20-7 (Task 75)

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DESIGN GUIDELINES FOR VERY LOW VOLUME LOCAL ROADS (< 400 ADT)

FINAL REPORT

Prepared For

National Cooperative Highway Research Program
Transportation Research Board
National Research Council

Timothy R. Neuman
CH2M HILL
Chicago, Illinois

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Table of Contents

Final Report – Geometric Design for Very Low Volume Local Roads

Tables

Tables (Continued)

Tables (Continued)

Tables (Continued)

Figures

Final Report – Geometric Design for Very Low Volume Local Roads

I. Introduction

The *AASHTO Policy on Geometric Design of Highways* (referred to by many as "The Green Book"), was developed as a fundamental reference document covering design policy for all highway types in the U.S. Developed by the American Association of State Highway and Transportation Officials (AASHTO), the AASHTO Policy represents a consensus view of appropriate design values and practices based on research and operational experience. The Policy, in combination with other companion AASHTO publications, addresses all aspects of geometric highway design, including cross section, horizontal and vertical alignment, intersections, roadside design and sight distance.

The AASHTO Policy was developed as a primary reference by engineers charged with construction and reconstruction of highways within state jurisdiction. Most states have adopted the majority of AASHTO's design recommendations and values within the framework of their individual state design manuals. In addition, over the years, the Policy has also become viewed as a key reference document for others, including county engineers and local municipal road designers. The AASHTO Policy thus has emerged as the key document used by most highway design professionals in the U.S., either directly or indirectly.

Beginning with the 1984 version of the Policy, AASHTO emphasized the importance of functional classification in development of design values for highways. Separate chapters were authored in which recommended design values for geometric elements were presented for each class of highway: freeways, arterials, collectors, and local roads and streets. Chapter V of the 1994 AASHTO Policy addresses design guidelines and criteria for local roads and streets.

In recent years, there has been increasing concern about funding and completing construction and reconstruction of roads and streets. Limited resources, increasingly restrictive environmental regulations, and public concerns about adverse effects of roads have affected agency project development. Within this context, design engineers seek ways to cost-effectively accomplish construction and reconstruction while maintaining a safe and adequate roadway.

The problem has become particularly acute at the local level, at which resources are especially limited. Furthermore, as much as ¾ of the centerline mileage of roads in the U.S. are local roads and streets carrying very low traffic volumes—on the order of 400 vehicles per day or less. For such roads with very low traffic volumes, there is a question about whether current design criteria and guidelines are reasonable or cost-effective.

NCHRP Project 20-7, Task 75 was performed to address concerns of those in the professional community regarding the design criteria and guidelines published in *the AASHTO Policy on Geometric Design for Very Low Volume Local Roads and Streets.* The concerns primarily relate to questions about the reasonableness of criteria in Chapter V of the AASHTO Policy for application to local roads and streets carrying 400 vehicles per day or less. Many believe such criteria are overly costly, restrictive and conservative. Recent research (NCHRP Report 362, and TRB Special Report 214) is cited as evidence that acceptable operations and safety can occur on highways with geometric dimensions less than previously thought acceptable.

Project Statement

NCHRP Project 20-7, Task 75 is titled *Geometric and Roadside Safety for Very Low Volume Local Roads (< 400 ADT).* [Note: Henceforth, the term ***LVLRoads*** will be used to refer to local roads with traffic volumes of 400 vpd or less.] The research project statement for this project contains the following paragraph, which summarizes the effort:

> *Improvements to roads with 0-400 vehicles per day to the Green Book criteria may not be a cost-effective approach to achieve overall operational and safety improvements on a system-wide basis. Several recent studies indicate that resources do not exist at the state and local level to meet the improvement needs for these very low volume local roads. A reduction in roadway width, design speeds and clear zone requirements could have a major impact on the total program by allowing additional miles of roadway to be constructed and reconstructed. The ability to fund additional miles of improvements could improve the overall system safety.*

Project 20-7, Task 75 was commissioned to look comprehensively at the range of geometric design criteria discussed in Chapter V, within the availability of budgeted resources.

Work Approach

The research approach was designed to maximize coverage within the resources established for the project. Five principle tasks were undertaken – *literature review, review of current practice, surveys of local agency engineers, operational and safety risk analysis, and development of guidelines.* This final report includes key findings from earlier reports that documented the above tasks.

An important part of the research was the prioritization of low volume local road problems. As documented in this report, the following geometric features were addressed with original research and most of the project resources:

- Stopping sight distance and design for vertical alignment
- Roadside design, including guardrail warrants/guidelines and clear zone dimensions
- Horizontal curvature

Other geometric elements of interest, including cross section and vertical alignment, are also addressed, although in more general terms. For these elements reliance is placed on a synthesis or reporting of work by others.

Units of Measurement

Existing data collected for this study was developed in both english and metric. In an effort to maintain data integrity, the information is documented in its original format.

All analyses performed as part of this study as well as subsequent models will be related only in metric units, consistent with the current format of the AASHTO Policy.

Purpose of Report

This report is intended for use by AASHTO to assess the content of Chapter V of the *1994 AASHTO Policy on Geometric Design of Highways*. Its intent is to provide suggestions and direction to AASHTO regarding potential, appropriate design policy values or approaches to derivation of geometric design values.

This report does not specifically constitute a recommended change in the AASHTO Policy. Any changes in the Policy, either in format or technical content, would be made following the normal process for design policy revisions.

II. Overview of Low Volume Local Road Design Issues and Practices

As defined for this project, the LVLRoad universe includes roads with average daily traffic volumes of less than 400 vehicles per day. The term "local" refers to the general or primary use of the road (see further discussion in Chapter III). Local roads serve primarily an access or local service function, as opposed to a through traffic carrying function such as is intended for arterials or collectors.

Despite the very low traffic volumes, there is substantial mileage of such facilities in the US. It is estimated that as much as 75 percent of total centerline mileage of public roads represents that of the LVLRoad classification. Such mileage occurs in both rural and urban areas, although the majority of mileage is rural in character.

This section of the report summarizes the current state of design practice for LVLRoads. It documents the results of a review of design practices, literature on geometry relative to traffic safety and operations, and a survey of county and other local engineers who design, maintain and operate LVLRoad systems.

LVLRoad Design Practices

An overview of current practice regarding LVLRoads provides a perspective on the range of variation or consensus. CH2M HILL compiled and reviewed geometric design guidelines and standards currently in use for very low volume roads. To accomplish this, published design manuals, policies and other materials were obtained from the following sources:

- AASHTO Policy on Geometric Design – 1994 Edition (Chapter V)
- United States National Forest Service
- Federal Highway Administration Federal Lands Division
- National Association of County Engineers

In addition, the principal investigator obtained design manuals and criteria from county and municipal sources identified by a survey distributed at the NACE Annual Convention in February 1996. Finally, the NCHRP 20-7 Project Advisory Panel provided input to the research by forwarding design standards from those panel members' respective agencies.

The focus of the review was, per the direction of the advisory panel and statement of research, design policies that address very low volume *local roads.* Arterial highway and collector design criteria were excluded from the analysis. The intent was to investigate design practices for local roads, as addressed by Chapter V of the AASHTO Policy. There was interest in the range of practice, in the background behind design valves, and in particular, in safety relationships to geometric design criteria.

The review of standards and criteria focused on a number of key issues. These were:

1. The applicability of the policy, including discussion/language referring to "standards," "criteria," etc., and the types of projects for which the policy applies;

2. Design Controls, including reference to functional classification, design speeds, terrain and other variables that define the parameters used for design;

3. Traffic Variables, with specific reference to breakdowns by Average Daily Traffic (with all roads less than 400 vehicles per day), as well as references to design vehicles;
4. Geometric Design elements, referring to design values for cross section, roadside, alignment, sight distance, and intersections.

Special attention was paid to discussion and presentation of special roadway elements or design features that are unique to very low volume roads. In particular, this includes criteria for design of one-lane roads and design practices for unpaved roads.

Table II-1 is a comprehensive spreadsheet that summarizes the major design criteria referenced in the published documents reviewed by CH2M HILL. The following is a summary of important differences and consensus features.

Design Policy Issues – Standards versus Guidelines

A major concern of many agency engineers is the stated nature and purpose of their respective design manual or document. The term "design standards" creates liability problems for many agencies confronted with very low volume roads and few resources. Engineers believe their agency is at risk if a roadway is found to not meet a "standard" if a crash occurs. It is noteworthy that, although the AASHTO Policy is very explicit in terms of its language, many local agency design manuals refer to the AASHTO Policy as "standards." Moreover, other design documents used on a widespread basis, including the *United States Forest Service (USFS) Preconstruction Handbook* and the Roads and Transportation Association of Canada (RTAC) Manual refer directly to their publication as "standards."

Design Policy Issues—Type of Project

A related issue is the type of project for which the design values apply. Again, the AASHTO Policy is explicit in its reference to major reconstruction or new construction, and its exclusion of projects that are 3R in nature (i.e., resurfacing, restoration or rehabilitation). Other published policies are less explicit, although one can infer from the discussion in them that the types of design projects envisioned are new roads.

Table II-1 shows that many state and local design policies consider their design criteria and/or standards to apply to 3R projects as well as new and major reconstruction. In some cases, text in the publications notes that the full values are desirable or ideal goals for 3R projects. This leaves the burden on the designer of a 3R project to demonstrate a need to avoid using a given design value. This issue is particularly significant, given that many geometric design criteria have changed over the years such that older highways designed to a given standard at the time may no longer meet the current standard.

Direct Reference to AASHTO Policies in Local Standards

Many state and local design policies refer in whole or in part to the AASHTO Geometric Design Policy or other policies, such as the Roadside Design Guide. In several other instances, although design tables or values are published for an agency, it is clear that AASHTO design values form the basis for that agency's design criteria.

	1994 AASHTO Policy (Chapter V)	United States Forest Service Road Preconstruction Handbook	National Park Service, Park Road Standards	Manual of Geometric Design Standards for Canadian Roads; Roads and Transportation Assoc. of Canada (Chapter H, Low Volume Roads)
Design Policy				
Discussion of Guidelines, Criteria and Standards	Referred to as Policy; presents "Design Guidelines"	Reference is made to "standards" in introduction to Chapter 4	Titled as "Standard"	Reference is made to establishment of "uniform national standards" and "provide standards for road agencies"
Types of Projects for which Policy Applies	New Construction and Major Reconstruction (3R is excluded)	For location, survey and design of Forest Service Roads	For location, reconstruction and 3R projects	New construction
Reference to State or AASHTO Design Policies	NA	NA	Tables are similar to AASHTO	NA
Design Controls				
Functional Classification	Reference to both access and through traffic; but no separation of criteria by these functions. "Special Purpose Roads"--Recreational, resource recovery (logging and mining)	Traffic Service Levels A-D specify types of traffic, vehicle mix, relative importance of user costs, surface type, alignment and topography. (See Exhibit 1, 4.1--5)	Four classes: Principal Park Road, Connector Park Road, Special Purpose Park Road, and Primitive Park Road	Service Functions A, B, & C. Type A: Rural roads to/within isolated communities - provide access; Type B: Rec. Rds-connect to external hwy network, scenic, and park areas, camp sites, etc.; Type C: Resource Recovery Rds - predominantly heavy vehicles.
Area Type	Rural and Urban	Rural only	Rural and Urban	Rural Only
Terrain	Level, Rolling, Mountainous	Not mentioned, but topography is identified as a major control	Flat, Rolling, Mountainous	Terrain discussed, but criteria not presented with terrain as a variable (other than grades)
Design Speed Range (km/hr)	Per Table V-1, 30 to 60 km/h--rural (other tables present design speeds to 100 km/h) 30 to 50 km/h--urban	No direct reference, but tables imply range of 10 mph to 50 mph	20-60 mph	30 km/h to 100 km/h
Adjacent Land Use	Reference to "industrial," "residential" land uses in discussions of design elements.	Not specified	Roadway to be designed around park features, "road is an end to a means"	Not specified
Level of Service or Other Operational Concerns	Not specified	Traffic Service Levels A-D specify types of traffic, vehicle mix, relative importance of user costs, surface type, alignment and topography.	"Park roads are for leisurely driving only, If you are in a hurry, you might do well to take another route now."	Not specified
Traffic Variables				
ADT Ranges (no, yes, specify)	ADT: 0-50 vpd, 50-250 vpd, 250-400 vpd for Design Speed criteria; no breakdown for other design elements (e.g., width)	No criteria stated	Principal Park Road, < 200 vpd, 200-400 vpd. Connector, < 400 vpd. Urban Parkway, < 4,000 vpd	Cross section standards have been developed for two-lane, two-way roads with ADT <100 vpd, 100-200 vpd and for one-lane, two-way roads with ADT less than 50.
Design Vehicles (Trucks, Agric., Logging, Passenger Vehicles, etc.)	No specific guidelines; reference to recreational design vehicles for Special Purpose Roads only	Different vehicles recommended for different design elements--e.g., passenger car for stopping sight distance, gravel truck for curve widening, RV for gradient, yarding equipment for clearances	Different vehicles recommended: 4W Drive, RV's, passenger cars, logging trucks.	No special design vehicles mentioned. Reference to wide trucks for resource recovery roads is made
DESIGN ELEMENTS				
Cross Section				
Lane Width	Per Table V-6, for two-lane roads 5.4 m to 6.6 m (Rural); per page 431, 2.7 to 3.6 m (Urban)	Reference to NCHRP 214 2-lane roads--9 to 12 feet. One lane roads-10 to 14 feet. Table V-14 for Special Purpose Roads	Table 10, 8' below 50 ADT, 9' 50-400 ADT	4.0 m for one-lane, two way roads (ADT varies by service function); for roads with design speeds of 50 km/h or less, shoulder width 0.5 m (paved roads)
Shoulder Width	Per table V-6, 0.6 m (Rural)	0 to 2 feet; the "minimum that can be economically constructed" is adequate.	Table 10, 1' below 200 ADT, 2' 200-400 ADT	Per Figures H.4.1 and 1b, 2-lane road widths vary from 5.6 m to 7.8 m for unpaved roads (function of ADT and truck volumes) lane widths of 3.0 m to 3.7 m for paved roads
Paved vs. Unpaved	Only reference regards cross slope	"Aggregate or native surface roads"--Design Speed 40 mph or less	ADT 0-200 vpd: surface may be dirt, gravel, or paved. ADT 200-400 vpd: surface type may be gravel or paved.	Paved, gravel, and earth. Earth roadway widths should be appropriate to provide for future gravelling
One Lane Roads	Reference in Special Purpose Roads only--"100 vpd maximum" (Design Speed 15 to 50 km/h)	"Most roads are single lane"	Primitive roads may be one lane. Table 7 provides stopping sight distance for one-lane roads, compiled from AASHTO.	ADT must be < 50 vpd and design speed < or = 50 km/hr. Roadway width = 4.0m, with turnout intervals < or = 300m. On resource development roads, ADT < or = 100 vpd and turnout spacing may be increased.
Roadside				
Clear Zone Policy	3 m "desirable" for rural, 0.5 m curb to obstruction for urban	4 feet for traffic Service Levels A&B; 0 to 4 feet for Traffic Service Levels C&D	Not specified	Clear right-of-width should be sufficient to accommodate construction of the road cross section. Suggested R-O-W widths for two-lane roadways are 20-30 m.
Guardrail/Barrier Policy	Noted for rural; but no specific guidelines. "Not used extensively" on local urban streets.	May be suitable as only practical...means of mitigating road inconsistency--refer to AASHTO Barrier Guide, NCHRP 214	Engineering Judgment	Not specified
Foreslopes	No specific guidelines	No specific guidelines (discussion pertains to drainage issues and erosion)	6:1 for high volume only. 4:1 for the first 10' desired.	Maximum foreslope of 2:1 in areas with earth grading. In mountainous terrain, 5:1 slope acceptable, where economical. Desirable for safety, slope 3:1. Max side slope of 1.25:1 suggested for rock grading.
Alignment				
Horizontal	No specific guidelines--Chapter III is assumed to apply for rural. 25 m minimum radius for urban.	See Exhibit 2, 4.31--5; Determine radius based on 'f'; f as a function of surface condition; Superelevation 6 to 12 percent, but none for low (<20 mph) speeds	Surface friction determined by seasonal use and surface type. AASHTO table used for max. Degree of Curvature.	Same design controls as for other roads (same as AASHTO Policy)
Vertical	Maximum Grade 7% to 16% for rural per Table V-4; Maximum Grade 8% in commercial, industrial areas; 15% for local streets in urban areas.	"Adverse environmental effects for grades in excess of 10% RV traffic--12% maximum; Motor Homes, vehicles pulling trailers--12% (< 300 feet); 4-wheel drive and high clearance vehicles--18%	Varies by topography and design speed. Ranges from 3 to 17% (table 3). Uses AASHTO values for vertical curves.	Varies with design speed-- 6 to 8 % for 100 km/h to 11 to 16% for 30 km/h; Crest vertical curves based on 150 mm object height rather than 380 mm (Canadian practice is based on taillights)
Sight Distance				
- Stopping Sight Distance	Per Table V-2, same criteria as other roads; per Table V-11, one-lane, two directional roads require twice stopping sight distance values as in Table V-2	Single lane roads require twice double lane road SSD; ht of object is 4.5 ft. (top of car). SSD formula varies for different traffic service levels	Uses AASHTO. Double stopping sight distance needed on one lane roads	Single lane roads require 2x the SSD of 2-lane rds. SSD model/criteria based on wet pav't surface conditions & fixed brake reaction time. For 30-50 km/hr, SSD is 30-65m.
- Passing Sight Distance	"Not anticipated" however on special purpose roads, provide "as frequently as possible". Chapter III criteria apply.	"Not a factor on single lane roads". Refers to AASHTO Policy for double lane roads	Uses AASHTO	PSD "is not considered to be a significant design element for low-volume roads"
- Intersection Sight Distance	Per p. 426 and Table V-9, provide 7 sec for passenger vehicle to cross uncontrolled road (both rural and urban)	"Sight distance along main road should be at least equal to SSD for the design speed of the main road"	Uses AASHTO	Not specified
Intersection Geometry	60° angle or better desirable (both rural and urban)	60° angle or better desirable; intersecting road should be at 6% grade or less	> 60 degrees, 6% grade or less	Not specified
Other Features Addressed	Cul-de-sac; Turnouts (Special Purpose Roads); Structures (loading, widths)	"Clearing Widths" ; Curve Widening; Turnout Spacing: Turnout Widths & Details	4' min. bikeways, curbs in urban areas	Turnout spacing and turnout design details

Table II-1
Design Criteria from State and Local Policy on LVL Roads

	Minnesota	Design Guide for Guardrail on State and Federal Aid Local Agency Projects, Michigan DOT	Section 8 Design Policies, State of Illinois	Wisconsin
Design Policy				
Discussion of Guidelines, Criteria and Standards	Referred to as a rule. Must be used for State aid projects by law.	Intended to be an aide for local agencies, design procedure.	Design Policies and General Requirements	Adopted by State Statute
Types of Projects for which Policy Applies	For New Construction, 3R and resurfacing	3R projects	New Construction and roadway "improvements"	New construction, Reconstruction, 3R
Reference to State or AASHTO Design Policies	Indirectly, Refers to MNDOT manual which refers to AASHTO	Michigan DOT standards, AASHTO, and MDOT local agency 3R project guidelines	Refers to AASHTO	Refers to AASHTO
Design Controls				
Functional Classification	None for rural, only for urban roadways which have collector and Arterial classification. ADT's below 400 are all collectors.	Not specified	Local road, collector, and Rural and Urban arterials	County Roads into Arterials, Collectors, & Locals also a Town Road Classification
Area Type	Urban, suburban and rural	Not specified	Rural and Urban	Rural and Urban
Terrain	Not specified	Level and rolling	Level and rolling	Level, Rolling & Mountainous
Design Speed Range (km/hr)	30-60 mph	Not specified	**Rural** ADT 0-250 vpd: local, 30 mph; collec., 40 mph; arterial, 50 mph. **Rural** ADT 250-400 vpd: local, 40 mph; collec., 40 mph; arterial, 50 mph. **Local** ADT 0-150 vpd, no design speed req'd.	60km/h
Adjacent Land Use	Urban areas rated by density as either "low" or "high"	Not specified	Not specified	Not specified
Level of Service or Other Operational Concerns	Not specified	Not specified	Not specified	Not specified
Traffic Variables				
ADT Ranges (no, yes, specify)	Rural - 0-49,50-149,150-399 Urban - 0-200,>200,Suburban - <1000,National Forest Hwys - 0-99, 100-750, Natural Preservation Routes - <300, 300-750, >750	ADT< or = 3,000 vpd.	Rural roadways 0-250 vpd and 250-400 vpd.	Local <250 vpd & 250-400 vpd, Town <100 vpd, 100-250 vpd, 250-400 vpd
Design Vehicles (Trucks, Agric., Logging, Passenger Vehicles, etc.)	Not specified	Not specified	Not specified	Not specified
DESIGN ELEMENTS				
Cross Section				
Lane Width	Rural: ADT 0-150vpd, lane width is 11'; ADT 150-399 vpd, lane width is 12'. Suburban lane width is 12'. Urban lane width is 12'.	Not specified	Rural Roadways - ADT 0-250 vpd: local, width=18'; collector., width=20'; arterial, width=22'. ADT 250-400 vpd: local, width=20'; collec., width=20'; arterial, width=22'. For local roads ADT 0-150 vpd, width=16'.	5.4m to 6.6m
Shoulder Width	Rural ADT 0-50 vpd, width is 1'; ADT 50-149, width is 3'; ADT 150-399, width is 4'. Suburban width is 6'. Urban width is 2'.	Not specified	Rural Roadways - ADT 0-400 vpd: local and collector., width=2' (use 4' if roadside barrier is utilized); ADT 0-400 vpd: arterial, width=4'.	0.6m to 1.5m
Paved vs. Unpaved	ADT 0-150, Aggregate. ADT >150, paved.	Not specified	Bituminous Treated Earth or Aggregate Surface	Paved and unpaved
One Lane Roads	Only urban one way	Not specified	Not specified	Only one way
Roadside				
Clear Zone Policy	Rural ADT<49, width is 7'; ADT 50-149, width is 9'; ADT 149-399, width is 15'. Suburban, width is 20'.	ADT range for all 0-750 vpd and slope 4:1-6:1. At 40 mph, 7-10'. At 45-50 mph, 10-14'. At 55 mph, 12-18'.	Not specified	Range between 2 to 3 m.
Guardrail/Barrier Policy	Not specified	Guardrail is "expected" at: high accident locations, bridge approaches on rural & suburban roads, certain bridge situations, and slopes steeper than 3:1 where embankment height is > 10'.	Not specified	Use AASHTO Roadside Design Guide. Guardrail not used for roadways with < 300 ADT.
Foreslopes	Rural ADT 0-50, 3:1. Rural ADT >50, 4:1. Suburban, 4:1.	Not specified	For 0'-6' width use 3:1 front slope. For 6' width or greater use 2:1 front slope.	3:1 max; 4:1 desirable
Alignment				
Horizontal	MNDOT Design Manual(AASHTO)	Not specified	At 30 mph, min radius is 251.85 with super transition length (trans length) = 18''. At 40 mph, min radius is 467.72', trans length = 21'. At 50 mph, min radius is 763.94', trans length = 24'.	AASHTO tables for design speed
Vertical	MNDOT Design Manual (AASHTO)	Not specified	At 30-40mph local roads & collectors: 7% level terrain, 8% rolling. At 50 mph local roads & collectors, 6% level terrain, 7% rolling. At 30-40 mph, arterials Not specified. At 50 mph, arterial, 4% level terrain, 5% rolling.	5% (80 km/h, level) to 16% (30km/h, mountainous)
Sight Distance				
- Stopping Sight Distance	MNDOT Design Manual (AASHTO)	Not specified	SSD at 30 mph is 200'. SSD at 40 mph is 275'. SSD at 50 mph is 400'	Refer to AASHTO
- Passing Sight Distance	MNDOT Design Manual (AASHTO)	Not specified	PSD at 30 mph is 1,100'. PSD at 40 mph is 1,500'. PSD at 50 mph is 1,800'.	Refer to AASHTO
- Intersection Sight Distance	MNDOT Design Manual (AASHTO)	Not specified	Minimum corner intersection sight distance - at 30 mph, 310'; at 40 mph, 415'; at 50 mph, 515'	Refer to AASHTO
Intersection Geometry	MNDOT Design Manual (AASHTO)	Not specified	Includes design plans for: Intersection Design of Side Road Approaches With Traffic Volumes of <= 400 vpd.	90 degrees +- 30
Other Features Addressed	Designated Forest Hwys and Natural Preservation Rtes (Class I.II & III), reducing design criteria. Lanes 11-12', Shoulders 1-6', In slope 3:1 or 4:1, Clear Zone 3'-15', Design speed 30-40 mph			

Table II-1
Design Criteria from State and Local Policy on LVL Roads

	Guidelines for Low Volume Roads and Streets within Washington State	Proposed new "Rules and Regulations for Metric Minimum Design Standards for Rural Roads, Municipal Streets, and State Highways, State of Nebraska	Nebraska Highway Law Consisting of Chapt. 39 Art. 21, Sect. 39-2215 and Art. 23, 24, and 25 of the Nebraska Revised, Reissued Statutes, Nebraska	Procedures for Classifications and Standards, Nebraska
Design Policy				
Discussion of Guidelines, Criteria and Standards	Design guidelines, procedures, and considerations	Rules and regulations	Functional classification; declaration	Functional classification
Types of Projects for which Policy Applies	New construction, "substantive improvements" (not spot reconstruction), and 3R projects	New project design and construction	Not specified, provides definitions for functional classes only	Classifications, referred to alternative sources for construction and maintenance detailed plans
Reference to State or AASHTO Design Policies	AASHTO, USFS Handbook, and WA State DOT Local Agency Guidelines and City and County Standards.	Refers to AASHTO	Refers to AASHTO	Refers to sections of the Nebraska revised statutes and minimum design standards.
Design Controls				
Functional Classification	Rural and Urban: minor collector or local access	1) New/reconstr. rural state hwys (ADT < 400 vpd),arterial collector, 2) 3R proj. on non-interst rural state hwys (ADT < 400 vpd), 3)scenic/recr/rural state hwys (ADT < 850 vpd), 4) Rural roads (ADT 0-50, 51-250, 251-400 vpd), arterial, collector, & local	1) Rural Highways (includes scenic highways) and 2) Municipal Streets. Sub headings: other arterial, collector, local, minimum maintenance roadways	1) Rural Highways (includes scenic highways); 2) Municipal Streets; and 3) Scenic-recreation roads. Sub headings for each category are: other arterial, collector, local, minimum maintenance roadways
Area Type	Rural and Urban. Ranges from flat to mountainous	Rural	Rural and Urban	Not specified
Terrain	Not specified	Level and Rolling	Not specified	Not specified
Design Speed Range (km/hr)	Varies from 20 (mountainous) to 60 mph (flat terrain)	See func. class above: 1) Arterial: level, Design Speed (DS) = 110km/hr, rolling, DS = 100km/hr. Collector: level & rolling, DS = 90km/hr, 2) DS = 90km/hr 3) DS = 80km/hr. 4) Arterial, DS = 65-80 km/hr. Collector, DS = 65-80 km/hr. Local, DS = 50-80 k/h	Not specified	Not specified
Adjacent Land Use	Not specified	Not specified	Not specified	Not specified
Level of Service or Other Operational Concerns	Not specified	Not specified	Not specified	Not specified
Traffic Variables				
ADT Ranges (no, yes, specify)	150 - 400 vpd	1)New/reconstr. rural state hwys (ADT < 400 vpd),arterial &collector, 2) 3R proj. on non-interst rural state hwys (ADT < 400 vpd), 3)scenic/recr/rural state hwys (ADT < 850 vpd), 4) Rural roads: arterial, collector, & local (ea. ADT 0-50, 51-250, 251-400)	Not specified	Not specified
Design Vehicles (Trucks, Agric., Logging, Passenger Vehicles, etc.)	Not specified	Not specified	Not specified	Not specified
DESIGN ELEMENTS				
Cross Section				
Lane Width	Roadway width (RW) incl. shoulder. For ADT < 150 vpd, Collector RW =22-24'; local access RW = 20-22'. For trucks below 10% and ADT < 150 vpd: collector width = 20' and local width = 18'. For ADT = 150-400 vpd, collector and local access RW = 24'.	1) New/recon rural state hwys, ADT< 400 vpd, width = 3.6m; 2) 3R projects ... hwy, ADT<400 vpd, width = 3.3m; 3) scenic-recr-rural state hwys, ADT<850 vpd, width = 3.3m,3.6m desirable; 4) Rural roads, all types, ADT<400, width = 3.0 - 3.3m	Not specified, Refer to the "Rules and Regulations for Metric Minimum Design Standards for Rural Roads, Municipal Streets, and State Highways, State of Nebraska.	Not specified, Refer to the"Rules and Regulations for Metric Minimum Design Standards for Rural Roads, Municipal Streets, and State Highways, State of Nebraska.
Shoulder Width	Shoulder width included in roadway width.	1) New/reconstr. rural state hwys, ADT<400 vpd, width = 1.2m (3m, if on Priority com. system); 2) 3R projects...hwy, ADT<400 vpd, width = 0.6m; 3) scenic-recr.-rural state hwys, ADT<850 vpd, width = 1.2m; 4) Rural roads, all types, ADT<400, width = 1-1.2m	Not specified, Refer to The "Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to The "Rules and Regulations ... State Highways, State of Nebraska.
Paved vs. Unpaved	Not specified	Paved	Not specified	Not specified
One Lane Roads	Not specified	Not specified	Not specified	Not specified
Roadside				
Clear Zone Policy	Refer to AASHTO	Not specified	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to the "Rules and Regulations ... State Highways, State of Nebraska.
Guardrail/Barrier Policy	AASHTO or other available design guidelines	Not specified	Not specified, Refer to the "Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to the "Rules and Regulations ... State Highways, State of Nebraska.
Foreslopes	Not specified	1, 2, 3) ADT<400 arter. & coll. FS=1:3-1:6 4) Rural arterial & collector: ADT=0-250, FS=1:2-1:3, ADT 251-400, FS=1:2-1:4. Rural local: ADT=0-50 FS=1:1.5-1:2; ADT 51-250, FS=1:2-1:3; ADT 251-400, FS=1:2-1:4	Not specified, Refer to The "Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to The "Rules and Regulations ... State Highways, State of Nebraska.
Alignment				
Horizontal	Refer to AASHTO	**Curve radius:** 1) ADT<400 vpd: arterial: level min R>=500, roll R>= 395m. Coll. R>=305m 2) 3R proj...hwy, R>=125m; 3) scenic/rec...hwys, R>=230m; 4) FOR art,coll, local ADT 51-400 R>=230m. FOR ADT<50: arterial R>=215m; coll R>=170m; local R>=75m.	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.
Vertical	Refer to AASHTO	Max grade: 1)Arterial: level min G>=3%, roll G>=4%. Coll: level G>=5.5%, roll G>=6.5% 2) 3R proj...hwy, Not specified 3) scenic/rec...hwys, G>=existing grade; 4) FOR art,coll, local ADT 51-400, G>=7%. FOR ADT<50: arterial G>=8%; coll G>=9%; local G>=10%.	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.
Sight Distance				
- Stopping Sight Distance	Refer to AASHTO	Not specified	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.
- Passing Sight Distance	Refer to AASHTO	Not specified	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.
- Intersection Sight Distance	Refer to AASHTO	Not specified	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.
Intersection Geometry	Not specified	Not specified	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.	Not specified, Refer to the"Rules and Regulations ... State Highways, State of Nebraska.
Other Features Addressed				Relaxation of Standards for design, construction, and maintenance when it possesses a special hardship on any segment of highway, road, or street.

Table II-1

Design Criteria from State and Local Policy on LVL Roads

	Iowa	Design Controls and Exceptions, Ohio Department of Transportation	Oklahoma	Subdivision Street Requirements, Virginia Department of Transportation
Design Policy				
Discussion of Guidelines, Criteria and Standards	"Rules" that require exceptions to exceed	Design controls	"Guidelines" which do not mandate the initiation of improvement projects.	Requirements
Types of Projects for which Policy Applies	New construction, reconstruction and 3R projects	New construction, reconstruction, major & minor pav't rehabilitation, two lane resurfacing, and 3R projects	New construction, reconstruction & 3R projects	New construction within the secondary system of state highways
Reference to State or AASHTO Design Policies	Refers to AASHTO Chapter 5	Refers to AASHTO	Refers AASHTO indirectly by using similar tables	Refers to AASHTO
Design Controls				
Functional Classification	Local, Collector, Arterial, Farm to Market	Rural and Urban. Sub classes: collectors and local roads	Refers to Oklahoma Policy and Procedures Manual.	Local subdivision streets
Area Type	Rural and Urban	Not specified	Not specified	Not specified
Terrain	Flat, Rolling & Hilly	Level, Rolling & Hilly	Level, Rolling & Mountainous	Level, Rolling & Mountainous
Design Speed Range (km/hr)	Urban, 30-55 mph, Farm-Market, 40-60mph Rural Coll., 30-40mph, Rural Local, 35-50mph	Collector & local, hilly: 20 mph. Collectors, rolling: 30 mph. Locals, rolling: 20-30 mph. Collectors, level: 40 mph. Local, level: 30-40 mph.	Paved - 20-50mph, Unpaved 20-30 mph	Not specified
Adjacent Land Use	Commercial, Industrial, residential & rural land use taken into consideration	Not specified	Not specified	Not specified
Level of Service or Other Operational Concerns	Not specified	Not specified	Not specified	Not specified
Traffic Variables				
ADT Ranges (no, yes, specify)	Urban classified by land use. Rural Local, <50 vpd, 50-250 vpd, 250-400 vpd	Collector & local ADT ranges: < 50 vpd, 50-249 vpd, and 250-400 vpd	ADT <50 vpd, 50-249 vpd, 250-399 vpd	ADT 0-250 vpd and 251-400 vpd
Design Vehicles (Trucks, Agric., Logging, Passenger Vehicles, etc.)	Not specified	Not specified	7000# wheel load for pavement design	Not specified
DESIGN ELEMENTS				
Cross Section				
Lane Width	Urban & Farm-Market, 10-12' Rural Coll.& Local, 10',	Rural: Arterial (ADT<400), lane width = 12'; Collector (ADT<400), width = 10' 11'; local roads (ADT 250-400), width = 10'-11' & (ADT <250), width = 9'-10'. Urban: arterial, width = 12'; collector, width = 11'; local, width = 10'-11',if ADT< 250, width=9'	For <250vpd, width is 9-10'. For 250-399 vpd, width is 10'.	For ADT 0-250 vpd, width is 5.4m. For ADT 251-400 vpd, width is 6.0m.
Shoulder Width	Urban, 6-8' Farm-Market, 2-8', Rural Coll., 2'Rural Local 2-3'	Rural: Arterial (ADT<400),shoulder width = 4'-8'; Collector (ADT<400), width = 4'-6'; local roads (ADT <400), width = 4'-6'. Urban: arterial, collector, and local, width = 1'-2' paved	For ADT<400, width is 2' (can be reduced in mountainous terrain)	For ADT 0-400 vpd, width is 2.1 m fill with gravel and 1.2m cut or fill without gravel
Paved vs. Unpaved	Urban paved, Farm-Market paved or unpaved, local not discussed	Paved	Gravel may be used for ADT < 400 vpd	Paved
One Lane Roads	Not specified	Not specified	Not specified	Not specified
Roadside				
Clear Zone Policy	Farm- Market, 6-12', Rural-Roadside Design Guide	For ADT<750 vpd: and design speed <40 mph, clear zone width = 8'.	2' min.	For ADT 0-250, R-O-W min. width is 12m. For ADT 251-400, R-O-W min is 15m
Guardrail/Barrier Policy	For roads with <200 ADT, can omit if justified by B/C analysis	Steel beam guardrail minimum barrier clearance of 7' when ADT < 400 and speed is < or = 40 mph	Use Roadside B/C to evaluate. Uses higher Accident costs.	Not specified
Foreslopes	Farm- Market, 3:1/4:1 Rural Coll. 3:1Rural Local 2:1	For ADT<750 vpd: and design speed <40 mph, clear zone width = 8' with a foreslope 4:1 to 6:1 (or flatter).	Cut - 3:1, Fill - 2:1	minimum slope of 3:1
Alignment				
Horizontal	AASHTO	Rural area - maximum degree of curve, design speed 20-40mph: 54° to 12°30'. Temporary & urban areas with design speed 20-40 mph, 72°45' to 11°45'.	Similar to AASHTO tables	For ADT 0-400 vpd: Level, min. curve radius is 40m; rolling, min. curve radius is 40m; mountainous minimum curve radius is 25m. No superelevation required.
Vertical	Urban, 5-11% Farm-Market 5-8% Rural Coll. 7-9%, local Not specified	Grades for 20 to 40 mph in urban and rural areas range from 7% to 15% but may be 2% greater in areas with ADT < 400	20mph - 7-16%, 30mph - 7-14%, 40mph - 7-12%, 50mph - 6-10%	For ADT 0-400 vpd: Level, min. grade is 7%; rolling, min. grade is 10%; mountainous, min. grade is 16%.
Sight Distance				
- Stopping Sight Distance	Refer to AASHTO	For 20 - 40 mph: 125' - 275' minimum. Preferred 125 - 325'.	Similar to AASHTO tables	For ADT 0-400 vpd, use 30 m
- Passing Sight Distance	Refer to AASHTO	Range 800' - 1,500' for 20 to 40 mph	Similar to AASHTO tables	Not specified
- Intersection Sight Distance	Refer to AASHTO	For 20 - 40 mph: 300' - 575'	Similar to AASHTO tables	For ADT 0-400 vpd, use 55 m
Intersection Geometry	Refer to AASHTO	Not specified	90 degrees +- 30	Not specified
Other Features Addressed	Benefit Cost analysis to justify design exceptions. BC<0.80=no improvement, BC>1.20=improvement warranted, BC0.80-1.20= requires further study		Hydrology, Drainage, Geotechnical, Pavement Design	

Table II-1
Design Criteria from State and Local Policy on LVL Roads

	Requirements and Specifications for Proposed Plats, Issued by: Board of County Road Commissioners, Genesee County, MI (In process of being reviewed)	Road Design Manual, County Road Association of Michigan	Policy and Guidelines for Design of 3R Projects Federal Aid System under Local Highway Agencies Control in Michigan	Standards for Construction of Road Improvements, Hamilton County, IN
Design Policy				
Discussion of Guidelines, Criteria and Standards	Standards and specifications	"This manual intended to serve as a guide in the design of streets, roads, and hwys."	Policy and Guidelines for Design of 3R projects.	Minimum standards
Types of Projects for which Policy Applies	Newly platted streets	New design and construction	3R projects on the collector roadway system (federal aid system under local jurisdiction).	New construction and 3R projects
Reference to State or AASHTO Design Policies	Michigan Department of Transportation, AASHTO, and ASTM	Highway Capacity Manual and AASHTO	Features not addressed in the study must meet AASHTO req'ts. "...wherever cost effective, a higher standard must be given serious consideration".	All construction must conform to the Standard Specs of the Indiana Department of Transportation and AASHTO.
Design Controls				
Functional Classification	1) Two Lane Sections: state & federal hwys, principal county local or primary roads, residential area streets, and commercial and industrial platted streets; 2) Boulevard platted streets; and 3) cul-de-sacs.	Local access: 1) Residential streets, 2) Local access roads, and 3) Industrial and commercial roads.	Collector roadway system (federal aid system under local jurisdiction)	Two lane roadways are classified as either collector (and commercial) or local.
Area Type	All roadways within Gennessee County	Rural and Urban	Not specified	Not specified
Terrain	Not specified	Flat roll in rural areas	Not specified	Not specified
Design Speed Range (km/hr)	Not specified	Local Access Road: **Rural** 30 - 40 mph. **Urban** 30 mph	For 3R projects a) speed to be commensurate with posted or regulatory speed on section improved or 85th percentile for horizontal/vertical curves in accordance with guidelines.	Local, Design speed = 25 mph. Collector, Design Speed = 40 mph
Adjacent Land Use	Not specified, assumed local residential and commercial	Not specified	Not specified	Not specified
Level of Service or Other Operational Concerns	Not specified	Design traffic were basically derived from HCM, level of service C in rural areas and LOS D in urban areas.	Not specified	Not specified
Traffic Variables				
ADT Ranges (no, yes, specify)	Not specified	Local Access Road: Rural Under 100 vpd, 100-250 vpd, 250-400 vpd and seasonal roads. **Urban** Under 750 vpd.	Roadway design based on one range from 0 to 750 vpd.	Not specified
Design Vehicles (Trucks, Agric., Logging, Passenger Vehicles, etc.)	Not specified	Specified for passenger vehicles and truck traffic.	Specified for passenger vehicles and truck traffic.	Not specified
DESIGN ELEMENTS				
Cross Section	Minimum R-O-W is 66' - 0" for most roads. County section-line and quarter section-line roads, minimum R-O-W is 100'-0'	Local Access Road: **Rural and Urban** minimum R-O-W of 66'		
Lane Width	a) Two-lane sections: width of 30' from back of curb (b/c) to back of curb. b) Boulevards: Minimum width, 30' b/c to b/c; Boulevard with median: minimum width, 100' b/c to b/c and maximum width 200' b/c to b/c. c) cul-de-sac. Not specified.	Local Access Road: **Rural** a) 2 lane seasonal roads @ 18' width, b) under 100 vpd & 30 mph, 2 lanes @ 20' width. c) 100 - 400 vpd & 30-40 mph, 2 lanes @ 20' width **Urban** ADT under 750 vpd, 2 lanes @ 28'.	For ADT less than 750 vpd, surface should be a minimum of 20 ft.	Local, Lane Width = not provided. Collector, Lane Width = 12', pavement, slope 1/4" per ft.
Shoulder Width	Specifications limited to curbed streets	Local Access Road: **Rural** a) 0 - 250 vpd, 4' earth shoulders b) 250 - 400 vpd, 6' earth shoulders **Urban** none specified.	For ADT less than 750 vpd shoulders should be 2 ft. width minimum. Designers should consider paving a portion of the shoulders for safety & maintenance benefits, but not required.	Local, Shoulder Width = not provided. Collector, Shoulder width = 3'-0"
Paved vs. Unpaved	Pavement to be bituminous aggregate or concrete with semi-rigid or limestone base	Paved	Not specified.	Local, Not specified. Collector, pavement section as req'd by ordinance
One Lane Roads	Not specified	Not specified	Not specified	Not specified
Roadside				
Clear Zone Policy	Not specified	Not specified	**A minimum consideration** to be given to determine cost-effectiveness of the following: 1) flattening sideslope > 1:3, 2) retain current side slope and widen lanes and shoulders, and 3) Remove, relocate, or shield isolated roadside obstacles.	Local, Not specified. Collector, 7'-0"clear zone with slope: 1/2" per ft.
Guardrail/Barrier Policy	For b) boulevards: high curb, flagstone walls or similar medians will not be permitted.	Not specified	Generally, when ADT is < 750, horiz. curve < or = 3 degrees, & truck traffic < or =25%: existing beam guardrail should be replaced when deficient by more than 3" (in no case shall existing railing be less than 24") or upgraded w/approved BCT End Sections.	Not specified
Foreslopes	Not specified	Not specified	Not specified	24:1 slope (1/2" per foot)
Alignment				
Horizontal	Maximum radii is provided only for two lane sections at platted street intersections as follows: principal county local or primary roads, 40' from b/c; residential areas, 30' from b/c.	Local Access Road: **Rural** Maximum curvature is not defined for ADT under 400. **Urban** ADT < 400 vpd maximum grade 8 %.	Generally stated for all roadway, use a max. superelevation rate of 0.06 ft/ft. Reference AASHTO. Note: Reconstruction should be considered only on horiz. curves > 15 mph below the 85th percentile of running speed and ADT > or = 750 vpd.	Local road curve radius = 150', tangent between reverse curves = 50'. Collector road curve radius = 200, tangent, reverse curves = 200"
Vertical	Sight distance is generalized for two lane sections. The maximum grade on any portion of street shall not exceed 6%, the minimum grade is 0.40%.	Local Access Road: **Rural** Maximum gradient is not defined for ADT under 400. **Urban** ADT < 400 vpd maximum grade 6 %.	Changes to crest vertical curves will only be made when crest hide major hazards, ADT in design year > 1,500, and design speed is > 20 mph below 85th percentile of surrounding area. Generally, all SAG curves can be maintained unless hazardous.	Local road: min. grade = 0.5%, max. grade = 7%. Collector: min grade = 0.5%, max grade = 5%
Sight Distance				
- Stopping Sight Distance	Sight distance is generalized for two lane sections. Sight distance over a crest shall be a minimum of 300' ; height of eye and height of object are equal to 3' 6".	Local Access Road: **Rural** ADT < 100 and seasonal roads, SSD = 200'. ADT 100-400, SSD = 200'-275'.**Urban** Not defined.	Designers should evaluate improvements to stopping sight distance where there is a reasonable chance it will be cost-effective.	Local Road, Sight distance = 250'. Collector, Sight distance = 650'
- Passing Sight Distance	Sight distance is generalized for two lane sections. Sight distance over a crest shall be a minimum of 300' ; height of eye and height of object are equal to 3' 6".	Local Access Road: Rural ADT < 100, PSD = 1,100'. ADT 100-400, PSD = 1,100'-1,500' Urban Not specified.	Not specified	Not specified
- Intersection Sight Distance	Minimum straightway to intersection of 150' with a maximum 3 % grade.	Not specified	Not specified	Not specified
Intersection Geometry	All newly platted streets which intersect with existing hwys or proposed platted streets will do so at 90 degree angles.	Not specified	Not specified	Local Road, Intersection angle min = 60 degrees, Collector, Intersection angle min = 75 degrees
Other Features Addressed			Signs, pav't markings, and traffic signal controls should be in accordance with the Michigan Manual on Uniform Traffic Control Devices.	

Table II-1
Design Criteria from State and Local Policy on LVL Roads

	Standard Practices & Guidelines for Local Agencies, Indiana Assoc. of County Engineers	Mobile County, Alabama	Fredrick County, Maryland	Fresno County, California
Design Policy				
Discussion of Guidelines, Criteria and Standards	Classified as standard practices and guidelines for local agency roads	Guidelines	Ordinance adopted as law	Referred to as improvement standards
Types of Projects for which Policy Applies	"Road Improvements", Provides design specs for rehabilitation of pav't and, also, new construction of a collector street without curb.	Not specified, provides functional classification definitions only	New Construction	New construction
Reference to State or AASHTO Design Policies	Provides plan drawings to be used with Indiana DOT 1995 Standard Specs, Promulgated by the IACHES Standards Committee - July 1996	Refers to AASHTO Chapter 5	Uses AASHTO for Stopping sight distance, Intersections	Indirectly refers to AASHTO through graphs
Design Controls				
Functional Classification	Does not specify functional classifications. Provides specs for local roads (utilities), subdivisions (curb & gutter/culverts/drainage), county roads (traffic control/street lighting), and collector streets (cross-section).	Urban: Minor, Residential, Special Purpose. Rural: Minor, Residential & Special Purpose	Local, subcollector, collector, & "higher classification"	Local, collector, and arterial
Area Type	Not specified	Rural and Urban	Rural and Urban	
Terrain	Not specified	Not specified	Nominal, <9% and Hilly, 9-12%	Not specified
Design Speed Range (km/hr)	Not specified	Urban - 20-45mph. Rural - 15-55 mph	Rural, 30-60mph. Local & Subcollector, <30mph. Collector <40mph.	Local: 20-45 mph. Collector: 30 mph. Arterial: 45-60 mph.
Adjacent Land Use	Not specified	Commercial, Industrial, Residential, & Agricultural all define classification	Define density by zoning. Low density - Agric., R1, R3, Medium Density R5,R8, PUD's, High Density R12, R16	Not specified
Level of Service or Other Operational Concerns	Not specified	Not specified	Not specified	Residential and industrial
Traffic Variables				
ADT Ranges (no, yes, specify)	ADT only specified to determine length of guardrail required using a "guardrail design procedure cost approach method". Costs are broken out between 0-250, 250-800 vpd.	Not specified	"Guidelines only" Local ADT <250 vpd. Subcollector, ADT 200-2,500 vpd	Not specified
Design Vehicles (Trucks, Agric., Logging, Passenger Vehicles, etc.)	Not specified	"trucks" with no further classification	Garbage Trucks	Not specified
DESIGN ELEMENTS				
Cross Section	Collector Street w/out curb (Doesn't apply to existing 2 ROD R/W Roads)			
Lane Width	Collector Street: Lane width = 12' pavement with slope: 1/4" per ft.	Not specified	Rural, width is 22', Urban Local, width is 20'-32' Subcollector, width is 24'-32' Collector, width is 22-28'	Lane width = 10' - 12'.
Shoulder Width	Collector Street: 3' - 0" width, using #73 stone	Not specified	Rural only, 8' grass	Shoulder width = 3'- 8'.
Paved vs. Unpaved	Collector Street: Pavement section as required by local ordinances	Paved and Unpaved	Paved	Paved or gravel.
One Lane Roads	Not specified	Not specified	Not specified	Not specified
Roadside				
Clear Zone Policy	Collector street: 7'-0"clear zone with a 1/2" per foot slope, then 11'-0 ditch with 2'0"minimum distance between end of ditch backslope and start of sidewalk.	Not specified	Not specified	Not specified
Guardrail/Barrier Policy	Guardrail determined using one of the following two methods: 1) a Length of Need Procedure, which establishes a "clear zone", removing or shielding hazards within, and 2) a "Guardrail Design Procedure Cost Approach Method" (* procedure enclosed).	Not specified	Not specified	Not specified
Foreslopes	Collector Street: 24:1 slope (1/2" per foot)	Not specified	Rural 3:1 - All urban roads use curb & gutter	Not specified
Alignment				
Horizontal	Not specified	Refer to AASHTO	Local, R>125' with 50' min. tangent Subcollector, R>200/100' min tangent, Collector, R>350/100' min tangent.	Refers to AASHTO
Vertical	Not specified	Refer to AASHTO	Refer to AASHTO	Typically grade is less than 10%. Grade may be up to 15% for lengths less than 500 ft.
Sight Distance				
- Stopping Sight Distance	Not specified	Refer to AASHTO	Refer to AASHTO	Refers to AASHTO
- Passing Sight Distance	Not specified	Refer to AASHTO	Not specified	Refers to AASHTO
- Intersection Sight Distance	Not specified	Refer to AASHTO	Refer to AASHTO	Not specified
Intersection Geometry	Not specified	Refer to AASHTO	90 degrees +/- 15	60 to 120 degrees
Other Features Addressed	Collector Streets: Variations in pav't width shall be approved in writing by county, "clear zone" may be 5'-0" where required by ordinance	Focuses on Functional Classification and sets a design speed letting AASHTO Ch. V cover the details	Dead end roads less than 1800' in length, Bikepaths, sidewalk & terrace width	

Table II-1
Design Criteria from State and Local Policy on LVL Roads

	Mobile County Alabama Design Policy for Paving Dirt Roads 1998	Alabama County Road Design Policy
Design Policy		
Discussion of Guidelines, Criteria and Standards	Policy	Criteria that "must" be used for projects involving state funding
Types of Projects for which Policy Applies	For paving unpaved roads	New Construction and Reconstruction
Reference to State or AASHTO Design Policies	None	No reference for <2500 ADT
Design Controls		
Functional Classification	Local Roads (Range of classes given)	All County Roads
Area Type	All County Roads	
Terrain	Not specified	All types referenced
Design Speed Range (km/hr)	Residential -- 20 to 50 km/h Major Local -- 30 + 70 km/h	< 100 vpd -- 15 to 90 km/h 100 - 399 vpd -- 20 to 90 km/h
Adjacent Land Use	By inference per functional classes	Not Specified
Level of Service or Other Operational Concerns	Not Specified	Not Specified
Traffic Variables		
ADT Ranges (no, yes, specify)	1 to 750 ADT reported here	< 100 vpd 100 - 399 vpd
Design Vehicles (Trucks, Agric., Logging, Passenger Vehicles, etc.)	Not specified	Not specified
DESIGN ELEMENTS		
Cross Section		
Lane Width	5.4 m	2.7 m to 3.3 m (varies by design speed)
Shoulder Width	0.6 m	0.6 m
Paved vs. Unpaved	Paving unpaved roads	Not Specified
One Lane Roads	Not specified	Not specified
Roadside		
Clear Zone Policy	0.6 m	0.6 m to 2.4 m (varies by design speed)
Guardrail/Barrier Policy	Not specified	
Foreslopes	1 : 3 for Major 1:2 for Residential	
Alignment		
Horizontal	Consistent with AASHTO	Consistent with AASHTO for e_{max} =.08
Vertical	Maximum grade 16%	Maximum grade 18%
Sight Distance		
- Stopping Sight Distance	Based on 1080 mm eye height & 60 mm object height	Consistent with AASHTO
- Passing Sight Distance	Not specified	Not specified
- Intersection Sight Distance	Not specified	Consistent with AASHTO
Intersection Geometry	Not specified	Minimum 7.6 m corner radius
Other Features Addressed		

Table II-1
Design Criteria from State and Local Policy on LVL Roads

Design Controls

The basic roadway design approach is consistent among state and local agencies. Road or street designed is governed through establishment of fundamental design controls. Design controls include *functional classification, location or area type, terrain, design speed, and level of service* or other descriptors of operational quality. Design of all highway facilities, including very low volume roads, requires identification of certain fundamental controls that will "drive" the design.

Functional Classification

This research effort is limited to very low volume *local* roads, excluding arterials and collectors. The term "local" refers as much to the jurisdiction controlling the road as it does to its function. Indeed, within this context, the existing published guidelines indicate a differentiation of separate types of local roads. The AASHTO Policy considers "special purpose" roads as being different from other local roads. A special purpose road, according to AASHTO, includes recreational roads, resource development roads, and local service roads. The RTAC standards define in a similar manner three categories of low volume local roads. Category A includes local rural roads to or within an isolated community. Category B roads are defined as recreational roads, and Category C includes resource recovery roads.

Local roads in an urban or suburban setting also may have different functions, and classifications. The County Road Association of Michigan's published standards classify roads as being local access, industrial or commercial, or residential. Mobile County, Alabama has developed a functional classification system for its rural and urban local road system. Local roads are classified as major, minor, residential or special purpose. The classification is based on the local road's role in the network, adjacent land use, and the mix or type of traffic using it. Interestingly, Mobile County suggests that this system be considered dynamic and subject to revision on a periodic basis, presumably to reflect land use or other changes.

The US Forest Service establishes a form of functional classification for their roads that relates to the purpose and users of the roads. Four classes of *traffic service levels* (TSL) are defined, as shown in Table II-2. Note that the TSLs describe distinctly different ranges in the intended quality of service defined in terms of free flow operations, safety, road surface and user costs.

Area Type

Most agencies specify design values for local roads in both rural and urban settings. Differences tend to be associated with the lower speeds and right-of-way restrictions in urban areas.

Terrain

According to AASHTO, rural highways are classified according to their presence in level, rolling or mountainous terrain. Higher design speeds that are associated with level and rolling terrain, and design values for certain geometric variables vary accordingly. Most published guidelines reference level and rolling terrain in their geometric criteria. States with mountainous terrain also include specific guidelines for such terrain. Generally, grade and cross section are considered terrain-specific sensitive geometric variables.

Table II-2
Traffic Service Level Descriptions for Roads on the US Forest Service System

	A	B	C	D
Flow	Free flowing with adequate parking facilities.	Congested during heavy traffic such as during peak logging or recreation activities.	Interrupted by limited passing facilities, or slowed by the road condition.	Flow is slow or may be blocked by an activity. Two way traffic is difficult and may require backing to pass.
Volumes	Uncontrolled; will accommodate the expected traffic volumes.	Occasionally controlled during heavy use periods.	Erratic; frequently controlled as the capacity is reached.	Intermittent and usually controlled. Volume is limited to that associated with the single purpose.
Vehicle Types	Mixed; Includes the critical vehicle and all vehicles normally found on public roads.	Mixed; Includes the critical vehicle and all vehicles normally found on public roads.	Controlled mix; accommodates all vehicle types including the critical vehicle. Some use may be controlled to vehicle types.	Single Use; Not designed for mixed traffic. Some vehicles may not be able to negotiate. Concurrent use traffic is restricted.
Critical Vehicles	Clearances are adequate to allow free travel. Overload permits are required.	Traffic controls needed where clearances are marginal. Overload permits are required.	Special provisions may be needed. Some vehicles will have difficulty negotiating some segments.	Some vehicles may not be able to negotiate. Leads may have to be off-loaded and walked in.
Safety	Safety features are a part of the design.	High priority in design. Some protection is accomplished by traffic management.	Most protection is provided by management.	The need for protection is minimized by low speeds and strict traffic controls.
Traffic Management	Normally limited to regulatory, warning, and guide signs and permits.	Employed to reduce traffic volume and conflicts.	Traffic controls are frequently needed during periods of high use by the dominant resource activity.	Used to discourage or prohibit traffic other than that associated with the single purpose.
User Costs	Minimize; transportation efficiency is important.	Generally higher than "A" because of slower speeds and increased delays.	Not important, efficiency of travel may be traded for lower construction costs.	Not considered.
Alignment	Design speed is the pre-dominant factor within feasible topographic limitations.	Influenced more strongly by topography then by speed and efficiency.	Generally dictated by topographic features and environmental factors. Design speeds are generally low.	Dictated by topography, environmental factors, and the design and critical vehicle limitations. Speed is not important.
Road Surface	Stable and smooth with little or no dust, considering the normal season of use.	Stable for the predominant traffic for the normal use season. Periodic dust control for heavy use or environmental reasons. Smoothness is commensurate with the design speed.	May not be stable under all traffic or weather conditions during the normal use season. Surface rutting, roughness, and dust may be present, but controlled for environmental or investment protection.	Rough and irregular. Travel with low clearance vehicles is difficult. Stable during dry conditions. Rutting and dusting controlled only for soil and water protection.

Source: United States Forest Service Design Elements and Standards; United State Department of Agriculture Forest Service, Road Preconstruction Handbook

Design Speed

Design speeds for local roads are generally lower than higher class roads. Design speeds are as low as 10 mph for certain USFS roads, but lower end design speeds of 20 to 30 mph or 30 km/h are more common for rural local roads. In urban areas, design speeds as low as 20 mph or 30 km/h are typical. Also note in reviewing Table II-1 that some published criteria for design speeds are sensitive to average daily traffic, with lower design speeds allowable for lower volume facilities.

The issue of design speed as traditionally established creates problems for engineers concerned with very LVLRoads (i.e., those with 100 vpd or less). An appropriately selected design speed should produce 85th percentile operating speeds. However, actual measurement of operating speeds is generally impractical for very low volume conditions. Moreover, it is generally understood that there is a difference between design conditions exceeded by 15 percent of a higher volume (say, 2000 vehicles per day), and design conditions exceeded by a small percent of a very small total volume. These differences become significant given the importance of design speed in the derivation and actual values used for most design elements.

A related issue is the need for design flexibility in terms of allowing very low design speeds under constrained conditions. Design speeds as low as 20 km/h are considered desirable for logging roads in difficult terrain. Although the general design controls in the AASHTO Policy do not include a 20 km/h design speed, there is provision for such design under current AASHTO Policy for Local Roads considered "Special Purpose," unpaved roads.

Adjacent Land Use

A few published criteria reference adjacent land use as a control or design consideration. This is typically more prevalent in an urban area, with some specific criteria (i.e., see criteria for Iowa, noting commercial, residential and industrial land uses). The AASHTO Policy discusses industrial and residential land uses in the text on design elements; however little actual guidance is provided with respect to specific design criteria.

Level of Service

A normal design control is the level of service for design hour traffic. For low volume two-lane roads, this is by definition not an issue since volume levels are well below any thresholds for capacity or speed/flow considerations. Not surprisingly, few published design guidelines exist for level of service on low volume local roads. The Park Road Standards for the National Park Service explicitly note that park roads are for leisurely driving, inferring that a high level of service as defined by speed does not apply to their system. Table II-2 depicts the only set of design controls that were found that relate to operational controls for LVLRoads. The USFS defines four levels of traffic service, from A to D. Designers should identify the level of traffic service to be provided, which in turn defines the design approach and in some cases design values for surface type, cross section and geometry. As shown in Table II-2 and noted above, these traffic service levels appear to represent a combination of functional classification and operational controls for USFS roads.

Traffic Variables

Design variables for traffic typically include traffic volume and design vehicle selection and/or characteristics.

Traffic Volume.- Table II-1 shows how the various published standards or guidelines consider low volume local and other low volume roads. Most define a low volume road as being less than 400 vpd. One notable exception is the RTAC standards, which consider low volume roads to be less than 200 vehicles per day. Most published guidelines consider two or three ranges of traffic within the 0 to 400 vpd range. A typical lower end range is less than 50 or less than 100 vpd. There is typically another volume category that breaks at 250 vpd.

Design Vehicles.- The published national guidelines and standards offer the greatest insights on design vehicles and their effect on geometric design. AASHTO notes recreational design vehicles as a consideration in design of "Special Purpose Roads." Both the RTAC and National Park Service standards refer to logging trucks and other special resource recovery trucks, and note their effects on cross section, curve widening, and grades.

The USFS guidelines provide an excellent discussion of design vehicles for LVLRoads. Their design procedures note that different design vehicles may be appropriate for different design features of the same highways, as demonstrated in Table II-3.

Table II-3
Design Elements Related to Design Vehicle

Design Element	Possible Design Vehicle
Stopping Sight Distance	Passenger Car or Pickup Truck
Thickness of Pavement Structure	
(1) Campgrounds	Garbage or Other Service Truck
(2) Logging Road	Yarding Equipment or Construction Equipment
Curve Widening	Lowboy or Gravel Truck
Lateral or Vertical Clearance	Yarding Equipment
Gradient	Gravel Truck or Recreational Vehicles

Source: United States Department of Agriculture Forest Service, Road Preconstruction Handbook Amendment, Section 4.1, 5a

Design Elements

The basic geometric design elements of interest in LVLRoad design are cross section, roadside design, alignment (both horizontal and vertical), sight distance and intersection design.

Cross section design values and criteria were reviewed for lane width and shoulder width. LVLRoad criteria also address paved versus unpaved roads, and one-lane roads.

Lane Width

The 1994 AASHTO Policy reflects updated lane width criteria per the results of NCHRP Report 362. These are shown in Table II-4. The criteria were derived from a comprehensive evaluation of crash data related to geometric elements for roads with traffic volumes of 2,000 vpd or less. Recommended design values reflected cost-effectiveness and compatibility with other basic design controls such as level of service and design speed. The values in some cases represent a downsizing of design criteria from the previous Policy (1990).

The revised 1994 AASHTO criteria for lane width include an interesting feature. As described in a footnote, a lesser width dimension than the primary tabular value is considered acceptable for *reconstructed highways where alignment and safety of the existing road are considered satisfactory.* Recognition of differences between reconstruction and new construction in establishment of primary design criteria is a significant change by AASHTO over previous policies.

Other published width values for LVLRoads identify lane widths as low as 9 feet (i.e., Illinois, Ohio, County Road Association of Michigan). The USFS references an earlier research report, NCHRP Report 214, as the basis for lane width values. The USFS guidelines result in minimum lane widths as low as 9 feet, as shown in Table II-5.

Table II-6 compares three sets of local rural low volume road width dimensions—from AASHTO, RTAC, and the USFS. With the recent revisions to the AASHTO Policy per NCHRP Report 362, AASHTO lane width values are consistent, and indeed in some cases less than other published criteria.

Note that lane width or roadway width values are slightly greater for roads with truck traffic. Also, note that lower quality surfaces (i.e., aggregate or native) are recommended to have slightly greater widths due to the effects of vehicle tracking and maintenance requirements.

Widths for local urban roads are often expressed as a total width, including the curb dimension. These tend to range from 20 feet to 30 feet (see i.e., Frederick County, Maryland).

Table II-4
Minimum Width of Traveled Way and Graded Shoulders on Local Roads

Design Speed (km/h)	ADT Less Than 400	ADT 400-1500	ADT 1500-2000	ADT over 2000
		Width of Traveled Way (m)[c]		
30	5.4	6.0[a]	6.6	7.2
40	5.4	6.0[a]	6.6	7.2
50	5.4	6.0[a]	6.6	7.2
60	5.4	6.0[a]	6.6	7.2
70	6.0	6.6	6.6	7.2
80	6.0	6.6	6.6	7.2
90	6.6	6.6	7.2	7.2
100	6.6	6.6	7.2	7.2
		Width of Graded Shoulder—Each Side (m)[c]		
All Speeds	0.6	1.5[a,b]	1.8	2.4

Source: 1994 AASHTO Policy, Geometric Design of Highways and Streets

[a]Mountainous Terrain - ADT 400 - 600 5.4 m width and 0.6 m shoulders.

[b]May be adjusted to achieve a minimum roadway width of 9m for design speed of 60 km/h or less.

[c]Where the width of traveled way is shown to be 7.2m, the width of the traveled way may remain at 6.6m on reconstructed highways where alignment and safety results are satisfactory.

Table II-5
Lane Widths for Double-Lane Roads

Size and Type of Vehicle	Type of Road	Type of Surface	Design Speed (MPH) 10	20	30	40	50
Recreational, administrative and service: Vehicle width—			Minimum Lane Widths (FT)				
1. up to 6.5 FT to 8.0 FT wide	Recreation or administrative	All surface types	9	9	10	11	11
2. 6.5 FT to 8.0 FT wide			10	10	11	11	11
Commercial hauling and commercial passenger vehicles including buses. 8.0 FT wide or greater	Roads open to truck traffic or mixed traffic	Aggregate or native		11	12	12	12
		Bituminous pavement		11	11	11	12

Source: United States Forest Service Design Elements and Standards; United States Department of Agriculture Forest Service, Road Preconstruction Handbook, 4.24–13

Aggregate or native surface roads should not have design speeds greater than 40 MPH.

The additional width required for lower quality surfaces is necessary, because of the tracking corrections needed compared to a higher quality surface.

Table II-6
AASHTO, RTAC, and USFS Summary of Minimum Lane Width Values

	Design Speed (km/hr) 30	40	50	60	70	80	90	100
	Minimum Lane Width (m)							
AASHTO 1994 Design Policy, Chapter V, Local Roads and Streets								
ADT less than 400 vpd for all road surface types and all vehicle types.	2.7	2.7	2.7	2.7	3.0	3.0	3.3	3.3
RTAC, Manual of Uniform Design Standards for Canadian Roads								
ADT less than 100 vpd on earth and gravel roads with truck traffic less than 15 AADT	2.8	3.0	3.0	3.3	3.3	3.5	3.5	3.7
ADT less than 100 vpd on earth and gravel roads with truck traffic greater than 15 AADT	3.0	3.2	3.2	3.5	3.5	3.7	3.7	3.8
ADT between 100 - 200 vpd on earth and gravel roads with truck traffic less than 15 AADT	3.0	3.1	3.1	3.3	3.5	3.5	3.7	3.7
ADT between 100 - 200 vpd on earth and gravel roads with truck traffic greater than 15 AADT	3.2	3.3	3.3	3.5	3.7	3.7	3.9	3.9
ADT less than 200 vpd on paved surface road	3.0	3.1	3.1	3.3	3.5	3.5	3.7	3.7
USFS (ADT is not specified)								
All surface type roads with recreational, administrative, and service vehicles greater than/equal to 2.0 m wide	2.7	2.9	3.1	3.3	3.4	3.4		
All surface type roads with recreational, administrative, and service vehicles between 2.0 m and 2.4 m wide	3.0	3.2	3.4	3.4	3.4	3.4		
Aggregate or native roadway surface with commercial hauling and passenger vehicles, including buses, greater than/equal to 2.4 m wide		3.4	3.4	3.4	3.5	3.7		
Bituminous pavement with commercial hauling and passenger vehicles, including buses, greater than/equal to 2.4 m wide		3.6	3.7	3.7	3.7	3.7		

Shoulder Width

A review of Table II-1 shows that a minimum 2-foot design value or its metric equivalent for rural shoulder width is common. The USFS explicitly notes that economic feasibility is essential, and that an 0-m dimension (i.e., no shoulder) may be appropriate in some circumstances.

The importance of a minimum shoulder dimension is typically associated with pavement or roadway structural integrity and maintenance issues, as opposed to safety considerations. Note, however, that any shoulder dimension represents a positive contribution to the roadside clear zone (see discussion below).

Paved vs. Unpaved Surface

LVLRoads may be paved or unpaved. The published documents treat the subject of paved versus unpaved roads in various manners. Chapter V of the AASHTO Policy does not directly acknowledge unpaved roads (indeed, table headings referring to "pavement" suggest the criteria apply to paved roads only.) In the section on Special Purpose Roads, there is a reference to minimum horizontal curve design for gravel surface roads. Special Purpose Roads are considered to be recreational roads, resource development roads, and local service roads that are "lightly traveled.

The USFS standards, and other policies, note that unpaved roads should have design speeds of 40 mph or less. Also, the USFS criteria for varying traffic service levels refer to surface type, with the higher service levels considered typically to be paved, and the lower service levels often unpaved. Other criteria relate traffic volume to whether a road needs to be surfaced.

One Lane Roads

Substantial discussion of one-lane roads is limited for the most part to the national documents. AASHTO limits the discussion to "special purpose roads," and suggests that a maximum volume of 100 vpd and a maximum design speed of 50 km/h are appropriate. The National Park Service standards note that "primitive roads" (their lowest class) may be one lane. The USFS states that most of their roads are single lane. RTAC standards identify one-lane roads for certain service functions (see Table II-7). All of the above documents recommend roadway width values of single lane roads, ranging from 10 to 14 feet or the metric equivalent.

Roadside

Roadside design is among the more important elements of low volume road design in terms of both safety performance and construction cost. Roadside design includes clear zone policy, guardrail or barrier policy, and design of foreslopes or sideslopes.

Clear Zone.- It is in the area of clear zone recommendations that the AASHTO Policy tends to differ more from other agency guidelines. Chapter V of the AASHTO Policy notes that a 3 m (essentially 10 feet) clear zone is "desirable" for LVLRoads. Other published criteria call for lesser dimensions, ranging from 0 (i.e., no clear zone) to 2 or 3 feet (or 1 to 2 m). USFS criteria call for a 4-foot clear zone for traffic service levels A and B, and lesser dimensions for lower class roads. Illinois, Wisconsin and Oklahoma local road criteria allow dimensions of 0 to 2 feet for certain circumstances.

Table II-7
RTAC Manual of Geometric Design Standards
One-Lane/Two-Directional Road Design Criteria

Category	Maximum ADT	Maximum Design Speed Km/h	Roadway Width* (m)
Rural road systems and roads to or within isolated communities	50	50	4.0
Recreational roads	50	50	4.0
Resource development roads	100	50	4.0

*Where traffic barrier is used, increase roadway width by 0.5 m on traffic barrier side of roadway.
Roadway widths do not include roundings.

Guardrail/Barrier Policy.- The AASHTO Policy in Chapter V notes the use of guardrail, but offers no specific guidelines for its use in rural local roads. Guardrail "is not used extensively" in urban areas according to AASHTO. The USFS design handbook notes that guardrail may be the only means of mitigating a design inconsistency. It refers the designer to the AASHTO Barrier Guide, and to NCHRP Report 214. Some state policies provide more quantitative guidelines for the use of barrier. Wisconsin's manual notes that barrier is not used for roads with less than 300 vpd. Similarly, Iowa's policy notes that guardrail should be used only where economically justified on roads with less than 200 vpd. The Indiana Association of County Engineers provides a direct set of guardrail design procedures for roads with 0 to 1500 vpd. Other agencies either do not address barrier design, or refer the designer to the AASHTO Policies.

Foreslopes.- Minimum foreslopes for rural roads are addressed by a few of the published criteria. Foreslopes of 1:2 or 1:3 are generally referenced. Note that Chapter V of the AASHTO Policy gives no specific guidance, although the 3 m clear zone policy would be consistent with a minimum of 1:3 for any dimension outside the shoulder. Chapter IV of the AASHTO Policy notes that "consideration should be given to the use of a roadside barrier" where slopes steeper than 1:3 are used. The USFS handbook provides no specific guidelines, but does discuss foreslopes in terms of drainage and erosion, not traffic safety requirements.

Highway Alignment

Most published guidelines refer to design values for horizontal and vertical alignment. Alignment is generally linked to the roadway's design speed, as is the case with the AASHTO Policy.

Horizontal Alignment.- With one exception, the design guides that were reviewed established alignment design policies based on the AASHTO operational models for radius as a function of side friction and superelevation. Differences among state or agency policies generally relate to maximum superelevation rates of 0.06 versus 0.08 used by each agency.

An interesting exception is the alignment design policy for the USFS, which determines side slipping friction factor values from the following equation:

$$f = T_f - 0.2$$

where:

f = Side slipping friction factor.

T_f = Traction coefficient of the traveled way surface material.

Designers select side friction slipping values for the traction coefficient as shown in Table II-8. USFS procedures provide explicit guidelines for assumed side friction, such guidelines varying significantly based on the surface type and weather conditions. Note, for example, that one can choose to design alignment for a snow pack condition on the surface of a USFS road.

The USFS design procedure holds for lower nominal design speeds – on the order of 10 to 50 miles per hour (20 to 80 km/h). Discussion of horizontal alignment design in the USFS preconstruction manual suggests the procedure is to be consistent with the philosophy of AASHTO

Table II-8
Traction Coefficients For Use in Design of Horizontal Alignment on US Forest Service Roads

Material	Dry	Wet
Concrete	0.75-0.90	0.55-0.70
Asphalt	0.55-0.70	0.40-0.70
Gravel, packed, oiled	0.50-0.85	0.40-0.80
Gravel, loose	0.40-0.70	0.36-0.75
Rock, crushed	0.55-0.75	0.55-0.75
Earth	0.55-0.65	0.40-0.50
(Reduce earth T_f by 50 percent for wet clays)		
Dry packed snow	0.20-0.55	
Loose snow	0.10-0.60	
Snow, lightly sanded	0.29-0.31	
Snow, lightly sanded with chains	0.34	
Ice, without chains	0.07-0.12	

Source: USFS Design Elements and Standards ;United States Department of Transportation Forest Service, Road Preconstruction Handbook

Vertical Alignment.- Published vertical alignment guidelines vary widely, but tend to be consistent in terms of the parameters that affect the design values. Most agencies publish maximum desirable values for grades as a function of terrain, functional classification, and in some cases vehicle type. For low volume local roads, maximum grades as high as 11 to 16 percent are noted in various design guidelines.

Sight Distance

Design for sight distance includes stopping sight distance (SSD), passing sight distance (PSD), intersection sight distance (ISD) and decision sight distance (DSD).

Stopping Sight Distance.- Most agencies follow current AASHTO Policy for stopping sight distance on very low volume roads. Design values are based on the same "model " (a driver eye height of 3.5 feet and 6 inch object height) for SSD as other higher volume and higher class roads.

Two interesting exceptions are the USFS and RTAC design standards. Different SSD values are computed from USFS design formulas depending on the traffic service level of the road. Also, USFS procedures allow the designer to specify design for either a passenger car or truck, with the latter design vehicle resulting in longer braking distances. (Interestingly, in design for vertical alignment based on SSD, USFS procedures assume the same 3.5-foot eye height for both passenger car and trucks, representing an apparent inconsistency.) The RTAC design values for vertical alignment are based on a very short object height for very low volume roads. This is different from Canadian practice for higher volume roads, which uses taillight height for SSD design. The RTAC rationale is that on very low volume roads, maintenance may not be as good, and potholes or objects on the roadway surface may be more prevalent and hence of sufficient concern to dictate design policy.

Many design guides address SSD for one-lane roads. There is a consensus that one-lane roads should be designed with SSD equal to twice the value provided for a two-lane road.

Table II-9
USFS, AASHTO, and RTAC Summary of Stopping Sight Distance Criteria

	Design Speed (km/hr)							
	30	40	51	66	70	80	90	100
Agency	Minimum Stopping Sight Distance (m)							
USFS (converted mph to km/h)	25	37	50	64	82	101	---	---
AASHTO	30	44	57	74	94	113	131	157
RTAC	30	45	65	85	110	140	170	200

Passing Sight Distance (PSD).- Passing sight distance (PSD) is generally not considered of importance in design of very low volume roads, due to the infrequency of passing opportunities. Indeed, there is no explicit requirement for any highway type that PSD be provided. Its presence and the amount of PSD is one measure of the quality of design. PSD also affects the capacity of the road. Many published design guides explicitly note that PSD need not be provided on low volume roads.

Intersection Sight Distance (ISD).- Most published guidelines reference the AASHTO Policy. Interestingly, page 426 of Chapter V of the AASHTO Policy cites an ISD criterion that differs from the ISD models and values shown in other chapters. Table V-9, repeated here as Table II-9, calls for ISD based on 7 seconds for a passenger vehicle to cross an uncontrolled road.

Table II-10
Intersection Sight Distance Criteria for Local Roads

Design Speed (km/h)	Corner Intersection Sight Distance (m)[a]
100	210[b]
90	180
80	160
70	140
60	120
50	100
40	80
30	60

Source: AASHTO 1994 Policy
[a]Corner sight distance measured from a point on the minor road at least 5 m from the edge of the major traveled way and measured from a height of eye at 1070 mm on the minor road to a height of object 1300 mm on the major road.
[B]At 100 km/h stopping sight distance governs.

Intersection Geometry

Most published guidelines address intersections only in terms of the angle of intersection. The consensus, consistent with AASHTO, is that intersections should be provided with angles no less than 60 degrees. This criterion is common to higher class roads.

Other Features Addressed By Design Guidelines

At the bottom of Table II-1 is a summary of additional items of interest regarding the design of very low volume roads. A common roadway feature addressed in several publications is turnout design. Criteria for turnout spacing and the design of turnouts are addressed by AASHTO, the USFS, RTAC and other design publications. Turnouts are particularly important with respect to one-lane roads.

Vehicle Turnouts on Single Lane Roads

Turnouts are primarily used on single-lane, two-way roads to provide user convenience and safety while maintaining speed. Turnouts may also be used as turnaround areas, storage areas for disabled vehicles, and add space for construction and maintenance staging.

To determine the turnout spacing needed to attain the traffic flow characteristics for an intended level of service the USFS uses the following equation in combination with Figures II--1 and II-2.

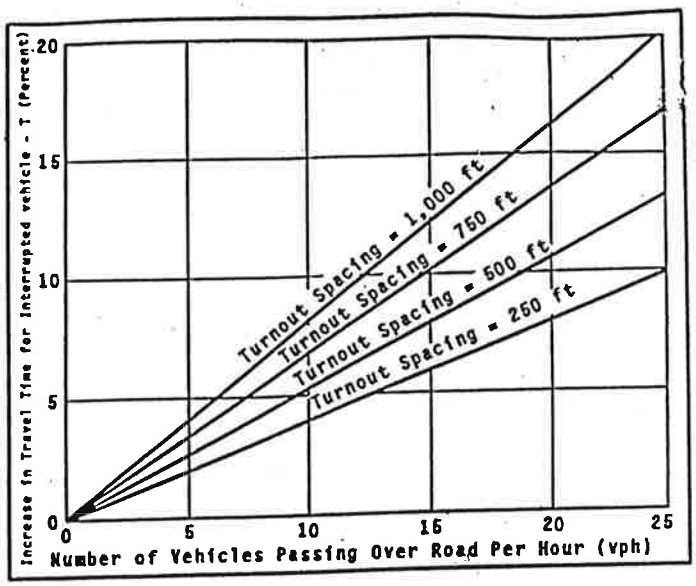
$$T = DS / 36$$

where:

T = Increase in travel time for the interrupted vehicle (percent);
D = Delay time per mile for the interrupted vehicle (seconds)
S = Design Speed (miles/hour)

After solving for T, the percent increase in travel time is compared against Figure II-1 to determine the turnout spacing for a certain number of passing vehicles. The vehicles per hour determined in Figure II-1 is then compared against Figure II-2 to determine the spacing required for a given level of service.

Figure II-1
Turnout Spacing



Source: United States Department of Agriculture Forest Service, Road Preconstruction Handbook 4.24 – 14

Figure II-2
Turnout Spacing

Traffic Service Level	Turnout Spacing	Operational Constraints
A	Make turnouts intervisible unless excessive costs or environmental constraints preclude construction. Closer spacing may contribute to efficiency and convenience. Maximum spacing is 1,000 ft.	Traffic: Mixed Capacity: Up to 25 vph Design Speed: Up to 40 mph Delays: 20 sec/mile or less.
B	Intervisible turnouts are highly desirable but may be precluded by excessive costs or environmental constraints. Maximum spacing is 1,000 ft.	Traffic: Mixed Capacity: Up to 25 vph Delays: Should be 30 sec./mile or less Use signs to warn non-commercial users of the traffic to be expected. Road segments without intervisible turnouts should be signed.
C	Maximum spacing is 1,000 ft. When the environmental impact is low and the investment is economically justifiable, additional turnouts may be constructed.	Traffic: Small amount of mixed Capacity: Up to 20 vph Design Speed: Up to 20 mph Delays: Up to 60 sec. Mile Road should be managed to minimize conflicts between commercial and non-commercial users.
D	Generally, only naturally occurring turnouts, such as additional widths on ridges or other available areas on flat terrain, are used.	Traffic: Not intended for mixed Capacity: Generally 10 vph or less Design speed: 15 mph or less Delays: At least 60 sec/mile expected Road should be managed to restrict concurrent use by commercial and non-commercial users.

Source: United States Department of Agriculture Forest Service, Road Preconstruction Handbook, 4.24--15

Note: On roads identified as being subject of the Highway Safety Act, intervisible turnouts or appropriate signing should be provided.

Professional Opinions of County Engineers

The experiences of many practicing engineers who work with low volume local roads and design criteria for them provide additional perspective here. Table II-11, shown below, reports on the results of a survey of county engineers with responsibility for low volume local roads. Although there are many opinions concerning the extent of low volume road design problems, there does appear to be a consensus on the following points:

1. Financial, public and environmental pressures are widespread, and make the task of implementing or maintaining low volume roads to a given "standard" very difficult.

2. Many local agencies are spending most or all of their resources on maintenance or reconstruction of existing low volume local roads. Many engineers said they were not constructing any new local roads.

3. The above not withstanding, there is concern for public safety and cost-effectiveness of design criteria. County engineers do not appear to be seeking an abandoning of design criteria, but do express a desire for those criteria to be defensible.

4. There is a recognition that the traffic type should play a major role in the design of a local road. Provision for agricultural vehicles and the unique aspects of residential and dead end streets are considered important.

Specific concerns of current design criteria focus on roadside clear zones, horizontal and vertical alignment, and cross section values. Several individuals expressed concerns regarding the availability of right-of-way to accomplish the design criteria. Their concerns relate to perceptions of the practicality of applying the published criteria to their typical problems.

Concern about the practicality of criteria for LVLRoads is at the heart of the issue. In general, county and other local road engineers understand and are supportive of the need for criteria. CH2M HILL did not uncover either through the survey or discussions with engineers a sense that LVLRoad criteria were unnecessary. What is generally believed, however, is that the criteria that do exist more often than not result in a high-cost, high impact solution *that produce no meaningful benefits.*

Summary of Current Design Practice for LVLRoads

Design criteria and the design process for very LVLRoads are substantially the same as for higher volume, higher class roads. Most published criteria, including the 1994 AASHTO Policy, treat LVLRoads the same as other roadway types. The basic design process is the same, wherein design controls are established for design speed, terrain, vehicle type, etc. Recommended minimum design values for roadway elements, to be implemented continuously or typically, are stated in the criteria. This continuity of approach is important. Consider Table II-12, which summarizes the basis of AASHTO design criteria for the major geometric elements. In all but design for the roadside and lane and shoulder widths on rural highways, geometric design is based on operational models of hypothetical, rational behavior rather than specific, quantifiable benefits. AASHTO operational models are for the most part insensitive to traffic volume (again, with the notable exception of lane and shoulder width).

City, State Participant Name Occupation	Preston, Minnesota Eugene Arling Filmore County Highway Department	St Cloud, Minnesota Doug Weiszharr Stearns County Engineer	Burlington, Kansas Ron Bonjour, P.E., RLS Coffee County Engineers Office	Port Angeles, Washington Don McInnes Clallam County Road Dept., Washington	Buffalo, Minnesota Wayne Fingalson Wright County Highway Department	Belton, Texas Richard Macchi, P.E. Bell County, Texas
Question						
1. Please list the approximate miles of roadway that you maintain.	480 linear miles	965 linear miles	950 linear miles	487.4 linear miles	530 linear miles	940 centerline miles
2. What percentage of these roads carry less than 400 vehicles a day?	50%	50-60%	98%	77%	5-10%	80%
3. What are your three biggest maintenance concerns?	Drainage, maintaining crushed rock on the roads, grading		Ditch maintenance, drainage, and the elimination of low water and plank (wood bridge) crossings. Problems with too much crushed rock on road. They have reduced the amount of crush rock used by about 25% over the last five years.	Maintaining Good Running Surface, Snow removal and sanding, and roadside drainage. Every year they add sand to the roads in the winter and must remove it from the ditches and shoulders in the spring.	Lack of funding, public demands, consistency of maintenance standards	Money, limited right-of-way, threat of law suits because of inadequate maintenance
4. To what extent do these reflect geometric design standards?	None.		Ditch grading and vertical geometry	Drainage of ditches, slopes are difficult to maintain.	Somewhat	Geometric design stds are greatly impacted by cost. See comments.
5. Do you have a clear zone policy?	Adopted State Standards for all roads that must be graded. Old roads do not meet state standards.	Adopted the State Policy	Follow the Low Volume Road Standards developed by Gene Russel at Kansas State University.	Adopted the State Policy	Yes. Follow State policies when using state funding.	As paved and gravel roads are upgraded, a 60 ft. clear zone is used (when feasible). There is no a uniform policy and the right-of-way width is usually dependent on the amount of land available for acquisition. r/w = "whatever they can get"
6. Do you have a guardrail policy?	Adopted State Standards. Typically use guardrail at bridges and in areas where a proper slope can not be achieved.	Adopted the State Policy	Follow State Standards.	Adopted the State Policy	Yes. Follow State policies when using state funding.	Guardrail is placed at approaches to all box culverts and bridges. In limited instances, guardrail is placed along the edge of road when there is a creek or ditch that is over five feet in depth and very close to the traveled way.
Maintenance Activities						
7. Please list your major maintenance activities.	Replacing signs and culverts, grading roads, plowing snow, crack batching asphalt, adding crushed rock to roadways, and replacing overlays.	Snow & Ice Removal, Surface Repair, Traffic Controls (including Signing)	Mowing r/w, clearing for sight distance at corners, grading, surfacing, chip and seal. They have undertaken a five year program to upgrade roadways to minor collector street standards.	Chip & Seal, winter maintenance, surface maintenance, pre-leveling, storm drainage, signing and painting, guardrail, vegetation control.	Snow & Ice Control, Bituminous patching, vegetation management, seal coating, and crack sealing	Annual seal coat program each summer, crack sealing (level up and pothole patching), mowing and brush/tree trimming, center line striping, reconstruction of roads that have failed due to traffic, age and/or lack of maintenance
8. What determines when a gravel road should be paved?	No set standard for when to pave a gravel road. However, they will NOT pave a gravel road unless it meets state standards.	Currently based on the number of complaints, The new Policy will soon require ADT to be greater than 300 vpd prior to paving.	They continue to use a two-part process established by KDOT in the 1960's when they had jurisdiction over local roads. For ADT>100vpd a double chip and seal is applied; for ADT>250vpd a three inch asphalt mix is applied to seal the road.	They are in the process of paving all roads because of the dust and maintenance concerns.	They no longer have gravel roads in the county. In the past, they felt that gravel roads with ADT>300vpd were to difficult to maintain.	Factors considered are number of residents along the road and ADT. There is no firm numbers when deciding when to pave a road. Decisions are based more on "who" and not "how many".

Table II-11
SUMMARY OF SURVEY OF RURAL ROAD SAFETY AND MAINTENANCE

City, State Participant Name Occupation	Preston, Minnesota Eugene Arling Filmore County Highway Department	St Cloud, Minnesota Doug Weiszharr Stearns County Engineer	Burlington, Kansas Ron Bonjour, P.E., RLS Coffee County Engineers Office	Port Angeles, Washington Don McInnes Clallam County Road Dept., Washington	Buffalo, Minnesota Wayne Fingalson Wright County Highway Department	Belton, Texas Richard Macchi, P.E. Bell County, Texas
Question Accidents and Safety						
9. Do you have insights on the use of accident records to identify and treat problems on your system?	Review accident reports semi-annually. They have used the data to soften curves in areas with a accident history. Typically changes to roadway are based on political pressures.	Receive semi-annual reports. Greatest use at intersection controls. Problems with officers reporting wrong accident locations.	Review Sheriff's accident reports which have been maintained since the late 1980's. However, they have so few accidents on their system that they know the problem areas. They have plans to use GIS to interface with the Sheriff's records.	To a minor extent they review the accident records. It helps them to determine locations for spot improvements.	They maintain a map of the county that shows all crash locations. They find this is a very useful tool for determining locations for improvement.	TDOT accident records are reviewed by Richard Macchi. Any roadway deficiencies that are mentioned are immediately checked out. Various aspects of the report such as locations, whether citations were issued, etc. are input into the computer for later retrieval and use in identifying high accident locations.
Maintenance Budget/Costs						
10. Can you share your maintenance records with us? If so, please send/fax to the address listed below. If not, please list the major cost items in your maintenance budget.	Yes. They will send a copy of their annual report.	They will send maintenance summary. If we need add'l information they will provide it.	Yes. They will send their annual report and several pages of a detailed cost summary. They have entire system broken down to 1/2 mile segments; each segment is assigned a project number.	Contact Steve Hauff at (360) 417-2306	They will provide a copy of the annual report. Minnesota breaks the roads into three categories: regular, municipal, and county roads.	Because of the way our budget is arranged, it is not possible to share the records
11. What are the sensitivities to cost? e.g. surface type, roadway width, topography, climate	Bad topography and soils will significantly increase the roadway cost.	Snow & Ice Removal are the biggest costs. Winter maintenance is double the cost of surface repairs.	Geographic location. Their costs are reasonable but a county engineer (friend) in LA must get his crushed rock from South America. He believes these variances make determining a standard unit cost impossible.		Traffic volume and percentage of trucks have a big impact on maintenance costs. Also weather conditions (snow, ice, and floods) vary maintenance costs on an annual basis. The cost of oil also affects how much patch work they will due in the summer.	
12. What is the average cost per mile to maintain your roads?	$4,000-$6,0000 per mile	Does not know off hand	Ranges per year but typically a surfaced road costs about $2,500/year and all other roads cost about $1,000-1,200/year.	Ranges per year but typically a surfaced road costs about $2,500/year and all other roads cost about $1,000-1,200/year.	$2,700-$3,000 / mile	$3,700/mile of road. There is no breakdown between paved and non paved roads.
13. What percentage of your overall budget is spent on maintenance (versus construction)?	State Aid allots 40% to maintenance and 60% to construction. Their budget is split about 50-50	50%	40%		20%	70%

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SUMMARY OF SURVEY OF RURAL ROAD SAFETY AND MAINTENANCE

City, State Participant Name Occupation	Preston, Minnesota Eugene Arling Filmore County Highway Department	St Cloud, Minnesota Doug Weiszharr Stearns County Engineer	Burlington, Kansas Ron Bonjour, P.E., RLS Coffee County Engineers Office	Port Angeles, Washington Don McInnes Clallam County Road Dept., Washington	Buffalo, Minnesota Wayne Fingalson Wright County Highway Department	Belton, Texas Richard Macchi, P.E. Bell County, Texas
Additional Comments	The biggest problem with attributing maintenance costs strictly to $/mile is that the cost of the individual automobile maintenance is not accounted for. The cost of maintaining a vehicle can be broken down by roadway type as follows: paved = $0.30-0.35/mile; asphalt = $0.45-0.50/mile; and crushed rock = $0.65-0.70/mile Also, it is considerably more expensive to maintain a rock road than a paved road. This is worsened by the fact that even though traffic volumes may remain the same, the size of farm machinery continues to increase as well as their travel lengths.		The Kansas 55mph statute on rural roads makes the excessive rock unsafe for motorists travelling at high speeds They are changing from crushed rock to pavement at approx. 35 miles per year. They are confident that this will reduce maintenance costs over the next few years. The process used to change from gravel to paved roads has been very effective for them. They feel that when the ADT> 250vpd the maintenance costs become too high to maintain a rock road. The biggest problem facing counties is the replacement of drainage structures. The gov't is not aware of how many structures need to be replaced. They have 5000 cross road structures in their jurisdiction. These costs will take up a considerable amount of the budget. Their oldest structure was built in 1910, however many counties have structures pre-dating the 1900's. As farm equipment continues to increase in size the plank bridges/culverts are becoming too narrow and can not support the increased loads. These are difficult to widen and often require reconstruction. Also suggests contacting Larry Emig from the Kansas Bureau of Local Roads at (785) 296-3861 Fax: (785) 296-2079. He is their contact person for all state projects We may also obtain information from LTAC (Local Technical Assistance Center) located in the Kansas State Univ.CE Dept.			The geometric design standards used, as you would guess, are greatly impacted by the amount of money we can spend to acquire proper right-of-way in which to construct the road. If it is necessary to cut off, sever, a piece of land from the parent tract in order to layout a proper horizontal curve, it will cost a great deal Likewise, in order to obtain a specific vertical curve, with required stopping or sight distance it will require the acquisition of substantial right-of-way to accommodate safe and maintainable back slopes. And them there's trees! You wouldn't believe how much a tree can cost. Then you do not have your clear zone. On the other hand if we do not build a road that is substantially in conformance with the "Green Book" we are opening ourselves up to a law suit.

Table II-11
SUMMARY OF SURVEY OF RURAL ROAD SAFETY AND MAINTENANCE

Table II-12
AASHTO Design Policy Background --
Primary Functional Basis for Geometric Design Criteria

	Safety	Operations	Sensitivity to Traffic Volume
Cross Section	✔ (Rural)		Yes
Roadside Design	✔		Yes
Horizontal Alignment		✔	No
Vertical Alignment		✔	No
Stopping Sight Distance		✔	No

The AASHTO design approach is further discussed in subsequent chapters of this report dealing with specific geometric elements. Under current design procedures and criteria for most agencies, low volume local roads are engineered and viewed as being no different from other highway types. While such an approach may be appropriate for higher class and volume facilities, basing design of LVLRoads on the same operational models and assumptions as higher class roads may explain why the resulting designs appear to not be cost-effective.

Literature Review of LVLRoad Design Criteria and Features

CH2M HILL staff conducted a comprehensive literature search and review of published literature on geometric design related to low volume roads, safety on low volume roads, and cost and maintenance of low volume roads. Reports and papers were reviewed with respect to their relevance to very low volume local roads. Thus, this review does not necessarily represent a comprehensive review of technical literature on a given geometric design topic.

The Northwestern University Transportation Library was the primary source of reference material. Additional literature was provided by members of the research panel.

A bibliography of key literature reviewed by CH2M HILL was published in a separate Task 1 Report. This section summarizes key findings from the literature review for the following subject areas: lane and shoulder width, horizontal and vertical alignment, sight distance, roadside design, construction and maintenance costs, and special features such as one lane roads, unpaved roads, and turnouts.

Lane and Shoulder Width

Lane and shoulder width have been among the most widely researched subject areas, particularly for two-lane rural highways. A recently published study (NCHRP Report 362 (70) noted in the literature review that over 70 studies had been identified pertaining to safety effects of lane and shoulder width. The reader is referred to Appendix B of that study for a complete summary of literature up to the time of that research.

Safety Relationships

The relationship between lane and shoulder width and highway crashes is among the most researched subjects. Many of the studies completed prior to NCHRP Report 362 produced similar findings. However, most previous studies relied on databases for higher-type facilities with traffic volumes greater than are found on very low volume local roads. Indeed, NCHRP Report 362 involved a specific look at very low volume roads, inclusive of all functional classes.

The findings for very low volume roads were somewhat different than the earlier studies, and counterintuitive to some. NCHRP Report 362 included collection and analysis of traffic, cross section and other data for roads with traffic volumes less than 2,000 vehicles per day. A significant portion of the NCHRP Report 362 database included roads with less than 400 vehicles per day, and also roads with very narrow lane and shoulder widths.

Figure II-3 shows the results of NCHRP Report 362 in terms of the safety effects of variable width values. For very narrow lane widths (on the order of 8 to 9 feet), *lower* accident rates were noted than for slightly greater lane widths. The principal researchers presented a number of possible explanations for this counterintuitive finding. The reasons could have been related to a presumed association between lower speeds and the much narrower lane widths, unreliability of traffic volume data for very low volume and/or local facilities, and small sample sizes for the very narrow widths (8 feet).

The researchers were not prepared to accept a conclusion that 8- and 9-foot lane widths were 'safer' than 10-foot lane widths. What *was* clear, however, was that there was no apparent or demonstrable benefit of 10-foot lane widths on lower volume roads versus 8- or 9-foot lanes widths, particularly when shoulders are very narrow. Indeed, the researchers concluded that shoulders are perhaps more important for very narrow roads than incremental lane width increases.

NCHRP Report 362 also established that a total roadway width (i.e., the sum of both lanes and shoulders) dimension of 30 to 32 feet is generally associated with safety effectiveness. Total width dimensions greater than 30 to 32 feet may be desirable from capacity or other perspectives, but should not be expected to produce measurable safety benefits over narrower dimensions.

NCHRP Report 362 also included shoulder width as a variable of interest. It was concluded that the presence of a shoulder is associated with significant accident reductions for roads with lane widths of 10 feet or less. Shoulder widths of at least 3 feet were found to be significant for roads with lane widths of 11 or 12 feet. The importance of a minimal (i.e., at least 2 foot) shoulder was carried into the development of recommended combinations of lane and shoulder width for low volume local roads, as shown in Table II-13.

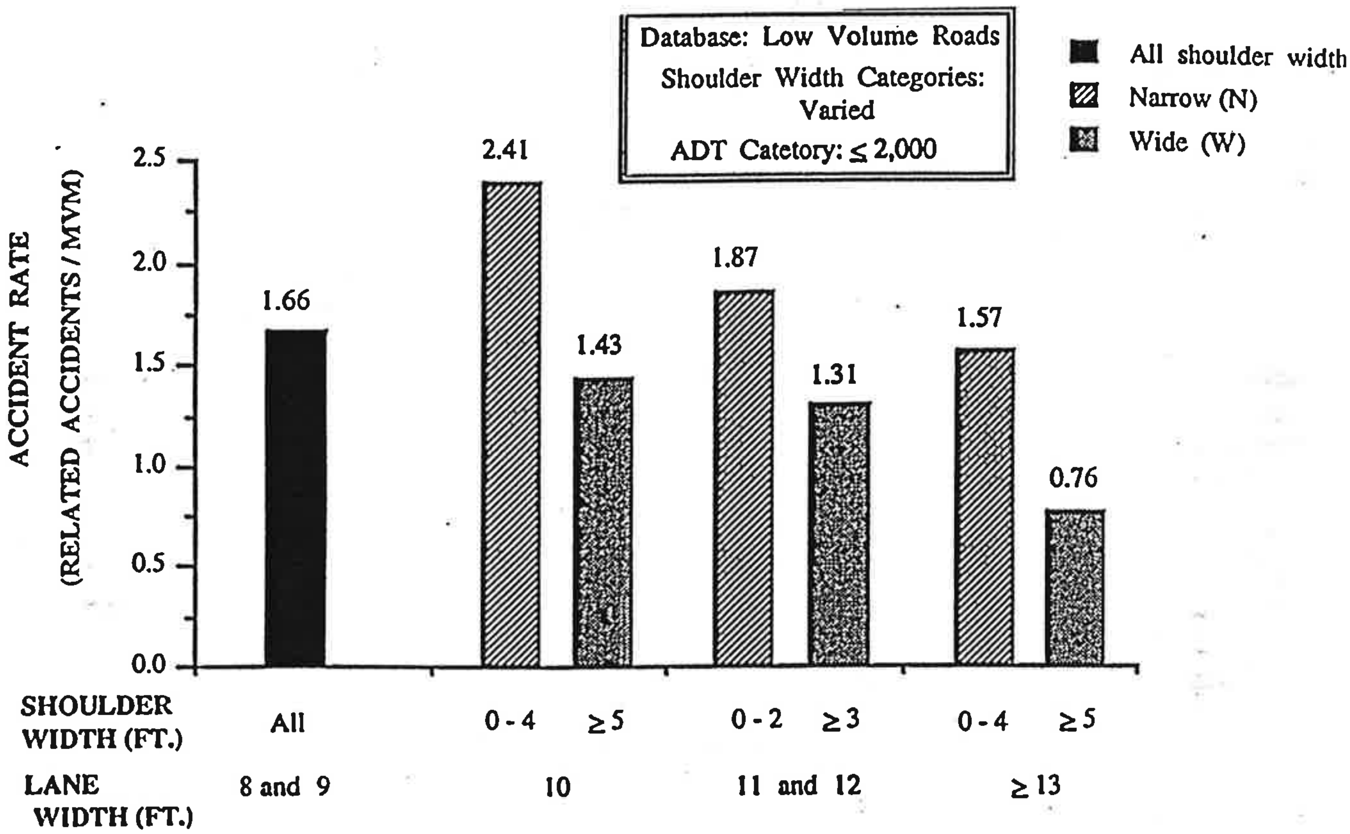
An important part of NCHRP Report 362 was the cost-effectiveness analysis of combinations of lane and shoulder width. Construction cost models were used in combination with crash reduction benefits associated with combinations of lane and shoulder widths. *Acceptance of incrementally greater widths as the basis for a minimum design value was predicated on 1) a measurable benefit; and 2) balancing that benefit against the known costs of providing for the additional width.*

The results of the cost-effectiveness analyses demonstrated that incremental lane and shoulder widths greater than minimums generally required traffic volumes greater than 400 vpd to justify their added costs of construction. For local roads with traffic volumes less than 400 vpd, cross section values of 9-foot lanes and 2-foot shoulders were considered optimal from a cost-effectiveness safety perspective. Only for level terrain, for local roads with design speeds of 50 mph were 10-foot lanes cost-effective.

Operational Relationships

Width-related low volume road operational concerns include vehicle type and speed effects. Concerns such as capacity common to higher volume facilities are not an issue for very low volume, lower class roads.

FIGURE II-3
Rates of Related Accidents by Lane and Shoulder Width from the Low-Volume Road Database



Source: Zeeger et. al. NCHRP Report 362

One previous study of low volume design criteria addressed width dimensions based on traffic operational requirements. NCHRP Report 214 recommended consideration of both traffic composition (percent of buses and trucks in the traffic stream) as well as the presence of agricultural vehicles in design of roadway widths. The need to provide for agricultural vehicles was judged to add 4 feet of total roadway width for the lower design speeds, as shown in Table II-14. NCHRP Report 214 (14) also noted operational needs associated with wider or special vehicles. Minimum shoulder dimensions for local roads were based in part on off-tracking and encroachment of large vehicles.

A review of design practice by many agencies showed reference to NCHRP Report 214 in their identification of recommended cross section design values.

Studies of driver operating behavior have established a relationship between roadway width and speeds on rural roads. NCHRP Report 362 incorporated these effects, noting that wider roadways (say, 11- or 12-foot lanes) are compatible with higher design speeds, and narrower roads (9-or 10-foot lanes) compatible with moderate or lower design speeds. Recommendations for lane and shoulder width in NCHRP Report 362 were based in part on these issues.

The research on cross section notes an operational effect of variable lane width on the capacity of a road. For the LVLRoads of interest, with traffic volumes of 400 vpd or less, incremental effects of width on capacity are not an issue. The design of such roads is not governed by capacity related considerations.

Horizontal Alignment

There have been a number of major research efforts on the safety of horizontal curves on 2-lane rural highways in recent years. Although the studies focused on higher class roads with higher traffic volumes, many of the findings and conclusions are applicable to very low volume roads.

NCHRP Report 374 (29) summarized the best recent work on horizontal alignment. This includes studies for FHWA by Glennon et al. (20) and by Zegeer, et al. (69). Both of these studies produced accident prediction models for isolated horizontal curves that include: degree and length of curve, total roadway width, and presence of spiral transitions. Other work by Glennon, et al. on "high accident" curve sites suggests that the roadside and pavement friction in the curve were also factors relating to safety. A recent synthesis of research on horizontal alignment and safety concluded that a wide range of factors have been shown to affect safety:

- Traffic Volume and Composition
- Curve Geometry
- Roadside Hazard on the Curve
- Cross Section on the Curve
- Stopping Sight Distance on the Curve and Approaches to the Curve
- Vertical Alignment on the Curve
- Proximity to Adjacent Curves
- Proximity to Other Critical Features such as Bridges, Intersections, Driveways, etc.
- Pavement Friction
- Signing and Marking

Transportation Research Board Special Report 214 on *Designing Safer Roads for 3R Studies* identified the work by Glennon and Zegeer as the most reliable. The TRB study looked at the cost-effectiveness of reconstructing horizontal curves to address safety problems, and concluded that traffic volumes of 1500 vpd or more were necessary to justify curve reconstruction for the normal range of conditions.

Glennon's and Zegeer's work highlighted the importance of the roadside with respect to curve safety. Note also that Glennon (and others before him) documented an increasing proportion of single-vehicle run-off-road accidents as traffic volume decreases. This strongly suggests that the importance of the roadside is even greater on very low volume road curves.

Table II-13(a)
Local Roads, Design Year ADT 0-2000 vpd (≤ 10% trucks)[1]

Design Speed (mph)	Cross Section Elements* (feet)	Design Year Traffic Volumes (ADT) in Vehicles per Day					
		0-250	250-400	400-750	750-1500	1500-2000	Comments
LEVEL TERAIN							
30	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 30 mph
40	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 40 mph
50	TW LW SW	(24) 10 2	(24) 10 2	(30) 11 4	(30) 11 4	(34) 11 6	*Minimum 10-foot lanes are appropriate for 50 mph
	TW LW SW	-	-	-	-	-	
ROLLING TERAIN							
30	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 30 mph
40	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 40 mph
50	TW LW SW	(24) 10 2	(24) 10 2	(30) 11 4	(30) 11 4	(34) 11 6	10-foot lanes are appropriate for 50 mph
MOUNTAINOUS TERAIN							
20	TW LW SW	(22) 9 2	(22) 9 2	(22)* 9 2	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 20 mph
30	TW LW SW	(22) 9 2	(22) 9 2	(22) 9* 2	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 30 mph
40	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 40 mph
50	TW LW SW	-	-	-	-	-	

TW——Total roadway width (lanes plus shoulders)
LW——Lane width
SW——Shoulder width (paved or unpaved)

*10-foot lanes and 5-foot shoulders may be considered as reasonable minimums for ADT greater than 600 vpd.

[1]Source: Zeeger et. al., NCHRP Report 362.

Table II-13 (b)
Local Roads, Design Year ADT 0-2000 vpd (> 10% trucks)[1]

Design Speed (mph)	Cross Section Elements* (feet)	Design Year Traffic Volumes (ADT) in Vehicles per Day					
		0-250	250-400	400-750	750-1500	1500-2000	Comments
LEVEL TERRAIN							
30	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 30 mph
40	TW LW SW	(24) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 40 mph
50	TW LW SW	(24) 10 2	(24) 10 2	(30) 11 4	(30) 11 4	(34) 11 6	10-foot minimum lanes are appropriate for 50 mph
	TW LW SW	-	-	-	-	-	
ROLLING TERRAIN							
30	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 30 mph
40	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 40 mph
50	TW LW SW	(24) 10 2	(24) 10 2	(30) 11 4	(30) 11 4	(34) 11 6	10-foot minimum lanes are appropriate for 50 mph
MOUNTAINOUS TERRAIN							
20	TW LW SW	(22) 9 2	(22) 9 2	(22)* 9 2	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 30 mph
30	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	9- or 10-foot lanes are appropriate for 40 mph
40	TW LW SW	(22) 9 2	(22) 9 2	(30) 10 5	(30) 10 5	(32) 10 6	10-foot minimum lanes are appropriate for 50 mph
50	TW LW SW	-	-	-	-	-	

TW——Total roadway width (lanes plus shoulders)
LW——Lane width
SW——Shoulder width (paved or unpaved)

*10-foot lanes and 5-foot shoulders may be considered as reasonable minimums for ADT greater than 600 vpd.

[1]Source: Zeeger et. al., NCHRP Report 362.

Table II-14
Minimum Road Width Values Based on Operational Requirements

Design Speed (mph)c/	Total Road Width Requirements (ft.) a/ b/			
	Lower % Busses & Trucks (as specified below) < 28% for 0-50 ADT < 12% for 51-100 ADT < 7% for 101-200 ADT NA for 201-400 ADT		Higher % Busses & Trucks (as specified below) ≥ 28% for 0-50 ADT ≥ 12% for 51-100 ADT ≥ 7% for 101-200 All % for 201-400 ADT	
	Infrequent Trips by Farm Machineryd/	Frequent Trips by Farm Machineryd/	Infrequent Trips by Farm Machinery	Frequent Trips by Farm Machinery
20 mph	18 ft.	22 ft.	20 ft.	24 ft.
25	20	24	22	26
30	20	24	22	26
35	22	24	24	26
40	22	26	24	28
45	26	26	26	28
50	30	30	30	30

Source: NCHRP Report 214
a/ 1 ft. = .305m
b/ Widths above 24 ft. (7.3 m) include appropriate shoulder widths.
c/ 1 mph = 1.61 km/h
d/ The determination of "frequent" and "infrequent" are at the discretion of the designer.

A number of other recently completed studies offer insights to safety and operations on lower speed curves such as might be expected on very low volume local roads. Harwood and Mason (25) analyzed AASHTO criteria for high and low speed curve design. They looked at operations of both passenger cars and trucks. For a passenger car, loss of control due to skidding is the critical event. Curves designed to AASHTO Policy generally incorporate a significant margin of safety against skidding. For operation of trucks, however, the findings were somewhat different. They concluded that rollover or loss of control skid may occur for critical trucks if their speed only slightly exceeds the design speed of a low-speed road. For example, on curves with a 32 km/h effective design speed, the most unstable trucks could roll over if their speed is as little as 1.6 km/h above the nominal design speed.

Anderson and Krammes (6) looked at speed reduction effects of horizontal curves. Their database included some very low volume roads, but was limited to collectors and arterials. Their study suggested a relationship between speed in a curve and curvature. This is consistent with work by Lamm (35) and others. A number of models were developed that related speed to curvature and accident to speed reductions.

An interesting aspect of these speed models is that, while they confirm a relationship between speed and curvature as is assumed by AASHTO design procedures, the sensitivity of speed relative to curvature differs from AASHTO assumptions. Drivers tend to "overdrive" curves, i.e., drive at speeds faster than is assumed by AASHTO. This behavior is particularly evident for curves consistent with 70 to 90 km/h speeds.

Design Speed and Design Consistency

Design consistency is a concern that has received increasing interest in recent years. Current AASHTO Policy for geometric features does not generally address issues of consistency with respect to the three dimensional features of the road. Consistency refers to the quality of the road, the message communicated to the driver, and how such message affects driver operations (primarily speed). Oglesby (46) has observed that the concept of design consistency is very important on low volume roads.

Much work has been done in recent years to develop measures of design consistency and translate them to geometric criteria. Lamm, et al. (35), in a number of research efforts have developed guidelines for speed and speed changes produced by alignment for continuous segments of highway. Leisch and Leisch (36) presented a theoretical model for speed consistency 20 years ago that has recently been tested with other European methods for speed consistency.

A number of authors (5, 31, 59) have developed speed change models or speed prediction models for use in evaluating design consistency. Such models typically relate horizontal curvature to speeds, although grades, pavement condition and roadway width have also been shown to affect speeds. Indeed, NCHRP Report 362 referred to roadway width and its expected effect of vehicle speeds in development of design criteria for width.

Wooldridge (68) reports on evaluation of a different driver model developed for two-lane rural roads. This model incorporates concepts of driver workload and expectancies with respect to intersections, lane drops, bridges, alignment, and width reductions. Wooldridge concluded that a model by Messer offered potential in terms of application to design consistency guidelines.

Vertical Alignment

Vertical alignment includes both grades and vertical curves. Design of the latter is governed by requirements for stopping sight distance. This section addresses grade and its effect on traffic operations. Sight distance design requirements are discussed below.

Roadway design has traditionally included constraints on maximum and minimum grades. Minimum grades are generally based on functional requirements for drainage. Maximum grades reflect concerns about effects of grade on vehicle speeds, which in turn influence the capacity and safety of the road.

There is little literature on the effects of grade on operations of LVLRoads. In one study, Jackson and Sessions (32) evaluated the speeds of loaded logging trucks on single lane roads to evaluate performance of such vehicles on curves and grades. They developed empirical relationships, and demonstrated that both loaded and unloaded truck speeds are significantly affected when grades are greater than 11 percent.

Sight Distance

A number of recently completed studies offer insights to safety and operational sensitivities regarding sight distance on very low volume roads. There are three types of sight distance of interest—passing, intersection and stopping. (Decision sight distance is not generally a concern on local roads or very low volume roads.)

Stopping Sight Distance (SSD) is among the most basic geometric features of the highway. Alignment design, particularly vertical curvature, is driven by SSD requirements. A number of studies have been performed over the past 15 years that have questioned the cost- and/or safety-effectiveness of current SSD requirements. Neuman (43) in a study of functional requirements of SSD noted a lack of cost-effectiveness in SSD design criteria for 2-lane rural roads with volumes much greater than those of this study. He established criteria for assessing improvements to SSD that suggest that nominal "deficiencies" as great as 20 km/h in SSD may be acceptable.

Fambro et al. (16) in NCHRP Report 400 established that very few highway crashes on roads with limited SSD are actually attributable to SSD. Their work, based on 55 mph roads, suggests that SSD does not become a factor in accident causation until SSD is less than 360 feet. Under current design policy and values, this translates to an effective deficiency of about 13 mph or 20 km/h. In addition, the SSD study by Fambro has documented that the frequency and severity of accidents involving 6-inch (150-mm) objects (representative of the AASHTO functional SSD model) are negligible.

Another interesting feature of the NCCHRP Report 400 study is the introduction of the concept of *maneuver sight distance.* Maneuver sight distance refers to an alternative sight distance operational model, potentially applicable to constrained situations. The concept is that drivers need to be provided a length and width of highway to maneuver around, but not necessarily brake to a stop in advance of an object or conflict. This and other aspects of NCHRP 400 are discussed at greater length later in this report in Chapter V.

Some studies have derived relationships between accident frequency and SSD at crest vertical curves. Paniatti and Council (50) in an HSIS study showed that accident frequency at crest vertical curves increases greatly where the grade differential ('A') is greater than 6 percent. Moreover, the accidents tended to cluster within 0.02 miles of the PI of the curve. However, this study involved roadways with traffic volumes much greater than 400 vpd.

There is limited information on the safety effectiveness of *intersection sight distance* (ISD). Most previous studies that showed a relationship between ISD and accidents involved higher volume intersections with stop control. A recently completed study by Neuman, et al (43) for the Michigan DOT provides insights on ISD and low volume intersections. This study of ISD-limited, stop controlled intersections of ramp terminals identified a threshold of 3,000 vehicles per day (total volume entering the intersection) as a minimum for which ISD appeared to be related to accidents. This volume level is well in excess of an intersection of two LVLRoads.

Roadside Safety

Significant research focus has been placed in recent years on the roadside. A number of studies looked specifically at roadside safety on very low volume roads. Studies addressed issues of clear zones, slopes and guardrail.

Viner (65) presents an overview of the rural roadside safety problem in a study of HSIS data for Utah. Table II-15 summarizes the six most frequent roadside crash types. Viner's study also noted a strong relationship between curvature and roadside crash frequency, a finding consistent with earlier research by others on highway curves. Data by ADT or functional class were not reported. Viner also reports a disproportionately greater risk associated with guardrail end impacts versus length of need impacts. He also notes that guardrail end crash severity is greater than impacts with the barrier itself.

Mak (39) reports on the methods currently employed to determine cost-effectiveness of roadside safety features. Figure II-4 is a summary of roadside encroachment rate studies for 2-lane rural roads. Mak notes the problems in attempts to predict roadside crash frequency. Note that in particular, the disparity in encroachment models for very low volume roads.

Probably the best known and most widely used design reference is the *AASHTO Roadside Design Guide (5)*. It includes, among other information, a discussion of the ROADSIDE model, which is used to predict and design for roadsides. This model was developed around research on encroachments (See Figure II-5).

Table II-15
Six Most Frequent Roadside Crash Types on Rural Roads

Accident Type	Percentage of Total Accidents
Overturn	27.5
Tree	19.3
Utility Pole	13.6
Embankment	9.3
Guardrail and other traffic rail	3.9
Ditch	3.0

Source: The Roadside Safety Problems, Viner, J.G. and the Federal Highway Administration.

An ASCE study of Local Low Volume Roads (3) addressed needs and concerns regarding roadside safety. This report, based on a synthesis of research and design guidelines by others, argues for establishment of "realistic" clear zone criteria, with consistent application of them. The use of traffic barriers is recommended only if they reduce the potential severity of accidents.

CENTRAC et al (13) reported on a survey of engineers in Washington on the issue of clear zone design criteria. Traffic and roadway variables were weighted by respondents, and prioritized. Table II-16 shows the results of the exercise. Speed and volume of traffic were considered the highest priority, with roadway classification and right-of-way availability next. The survey also addressed new construction versus existing facilities. Problems inherent to existing facilities were availability of right-of-way, location of utilities, and the presence of sidewalks. Also, many agencies noted that moving roadside objects to achieve a clear zone standard does not make sense if there was no history of problems with these objects.

Harwood performed a study published as NCHRP Report 247 (25) that addressed the system wide safety effects of various clear zone policies. Although the study involved roadways with a greater functional class and traffic volume than applies to this project, it does provide insights. Figure II-6 illustrates Harwood's findings on the relative frequency of a range of design policies. Great differences in crash experience associated with clear zone versus non-clear zone policies tend to narrow dramatically at ADT less than 1000 vpd.

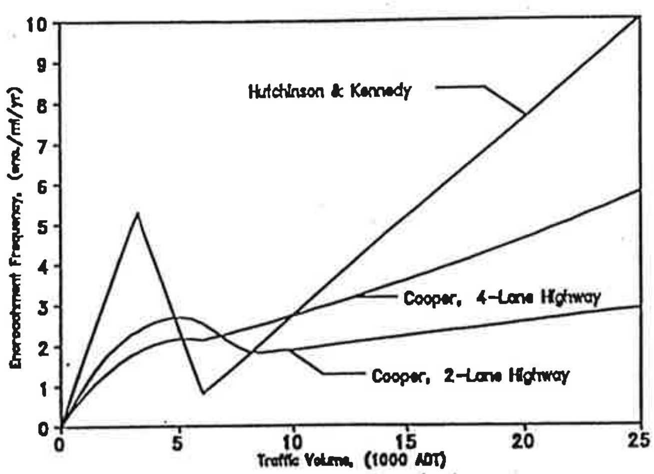
More recently, Stephens (60) looked at guardrail warrants for low volume roads. His unpublished study, NCHRP 22-A, was discussed in NCHRP Circular 416. Stephens developed a framework for considering roadside problems on very low volume roads. He suggested the use of the AASHTO ROADSIDE program in exercising the framework.

Stephens' procedure categorizes treatment alternatives into the following corrective methods: change the clear zone, remove or relocate the hazard, change the hazard, shield the hazard, or accept the risk. Alternatives are then prioritized as "obviously suitable or preferred treatment", "possibly suitable treatment", and "obviously not suitable treatment". Economic analysis is the primary consideration for suitability, but other factors such as functional feasibility, agency policy and available resources may also eliminate alternatives. Stephens notes that corrective measures, such as guardrail, will only be placed where they are highly cost-effective.

A more recent, comprehensive study of guardrail warrants by Miaou (42) was completed. This study used HSIS data. It concluded that guardrail in place of embankment was not generally warranted until traffic volumes were greater than 400 vpd. Figure II-7 is taken from this study.

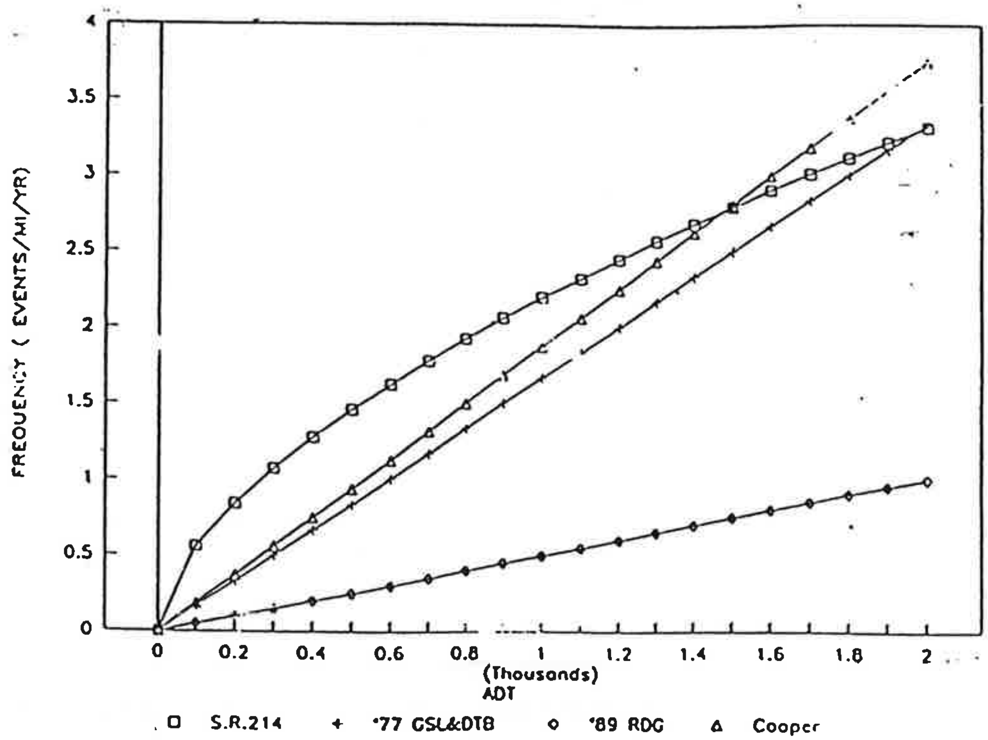
Hanna (24) looked at development of low-cost guardrail systems for very low volume roads. Four low service-level systems were developed. A combination of computer simulation, component testing and full-scale crash tests was conducted. The study concluded that use of guardrail to shield relatively small point hazards was justified only under severe conditions, with the hazard close to the roadway, and ADT greater than 500 vpd.

Figure II-4
Summary of Roadside Encroachment Rate Studies for Two-Lane Rural Roads



Source: Mak, K., Methods for Analyzing the Cost-effectiveness of Roadside Safety Features, 1995.

Figure II-5
Encroachment Frequency Curve



Source: AASHTO Roadside Design Guide

Table II-16
Survey Results of Current Clear Zone Practices Priority Rating of Factors Agencies Felt Important in Identifying Clear Zone Standards

Priority	Factor	Point Value Total
1	Speed of traffic	158
2	Volume of cars	153
3	Classification of roadway	139
4	ROW available	138
5	Terrain	133
6	Area type (rural vs. urban)	125
7	Abutting uses	116
8	Mix of trucks and buses	106

Source: CENTRAC, Clear Zones for Local Agencies, 1989

An earlier study by Arnold (7) for the Virginia DOT looked at design guidelines for guardrail on secondary roads. The study included a survey of practice (see Table II-17), which indicated a wide use of procedures or warrants different from AASHTO for low volume roads. The study produced a series of guardrail warrants for secondary roads for design speeds of 30 mph and 50 mph, as illustrated in Figure II-8.

It is interesting to note that many of the states with more rolling or mountainous terrain have warrants or open-ended exceptions that tend to exclude the use of barriers on low volume roads. Examples include West Virginia, Kentucky, North Carolina, Montana, Pennsylvania, Colorado and Alaska.

Finally, a recent study of guardrail warrants was performed by Wolford and Sicking (62) and published in Transportation Research Record 1599 (62). Wolford and Sicking used Highway Safety Information System data to look specifically at warrants for low volume roads. His findings, summarized in Figure II-9 note that guardrail to shield drivers from steep embankments is generally not cost-effective for volume ranges below 400 vpd.

Design Criteria, "Standards" and Functional Classification

A number of papers and reports have been published that address basic considerations in the formulation of design criteria for very low volume roads.

Weale, et al. (67) discuss design considerations for low volume roads. Their paper emphasizes the need for design criteria to be based on community needs, economy, safety and topography. They suggest that an "arbitrary" establishment of design speed and its resulting impact on design values is not recommended for very low volume roads. This argument is similar to one put forth by Oglesby (46), who recommended three conclusions:

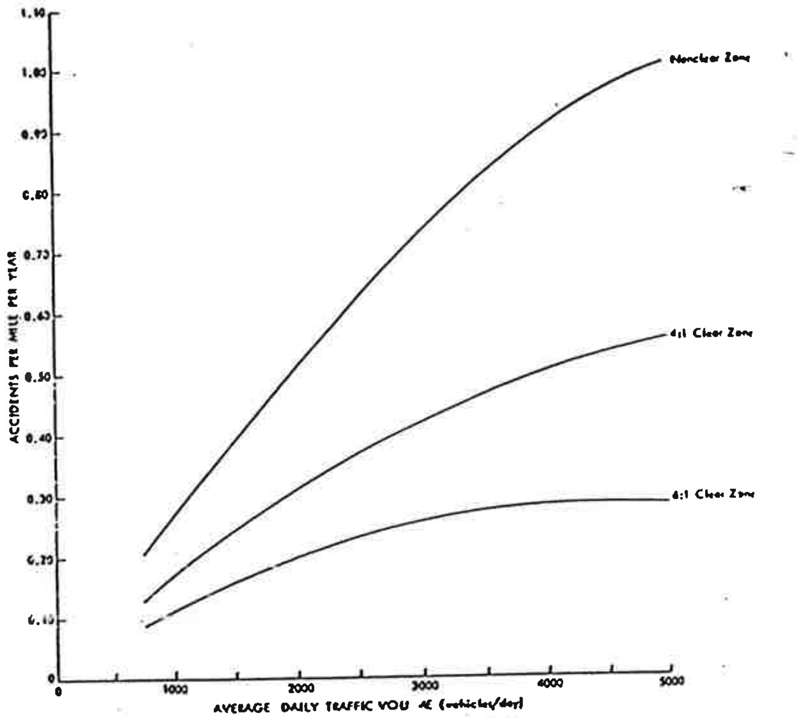
1. Prescribed "standards" for roadbed width need to be considered as maximums, not minimums.
2. The concept of a constant cross section should be abandoned. Emphasis should be on spot improvements.
3. Design features intended solely to reduce accidents on very low volume roads should be based on data or sound rational basis, not merely the belief that the road "seemed" dangerous.

Bews, et al reported on the development of design standards for very low volume Canadian roads.(11) The following are important findings from the Canadian effort:

1. Design speeds greater than 100 km/h were not considered justifiable. Selection of an appropriate design speed should be based on terrain, trip length, and service function.
2. They suggest an alternative design approach, requiring more research. It is considered unrealistic to balance all physical features of a low volume road to a consistent design speed, particularly in rolling or mountainous terrain.

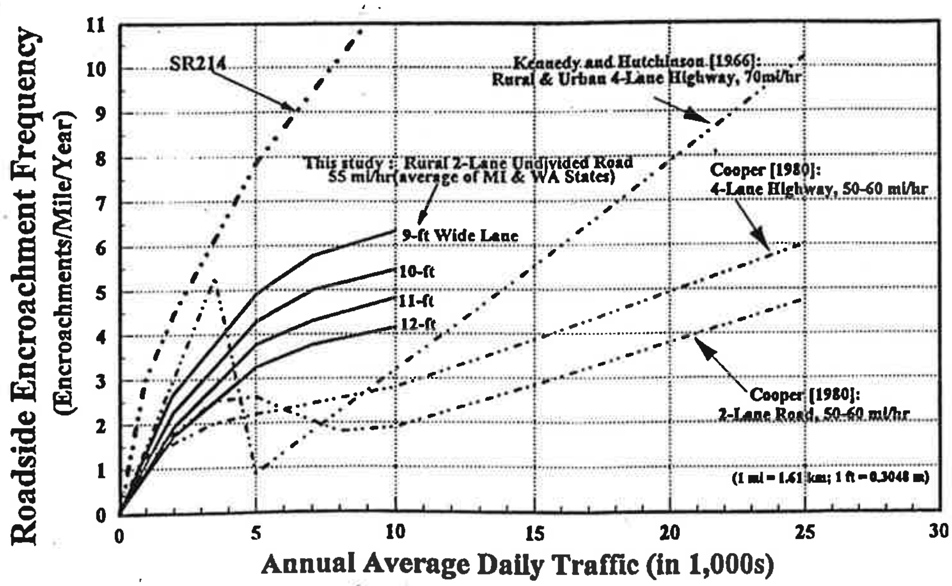
A report by the American Society of Civil Engineers discussed the subject of functional classification of very low volume roads. Table II-18, from this report, shows recommended functional classification based on vehicle use and traffic volume. Six road classes are established, with five of these representing "local" type functions.

Figure II-6
Relationship Between Single-Vehicle Run-Off Road Accidents Per Mile and ADT for Two-Lane Highways



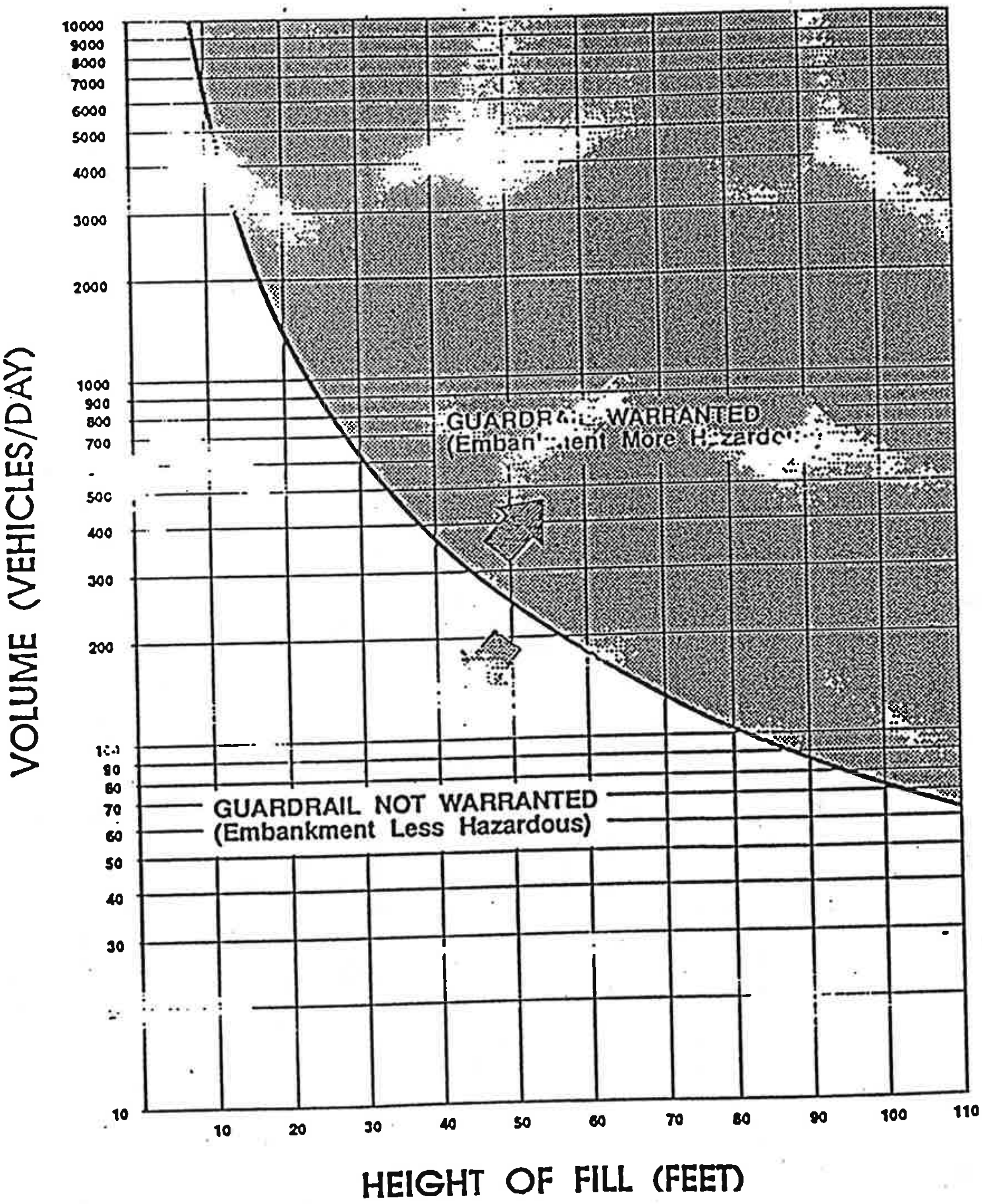
Source: NCHRP Report 247, 1982

Figure II-7
Roadside Encroachment Frequency



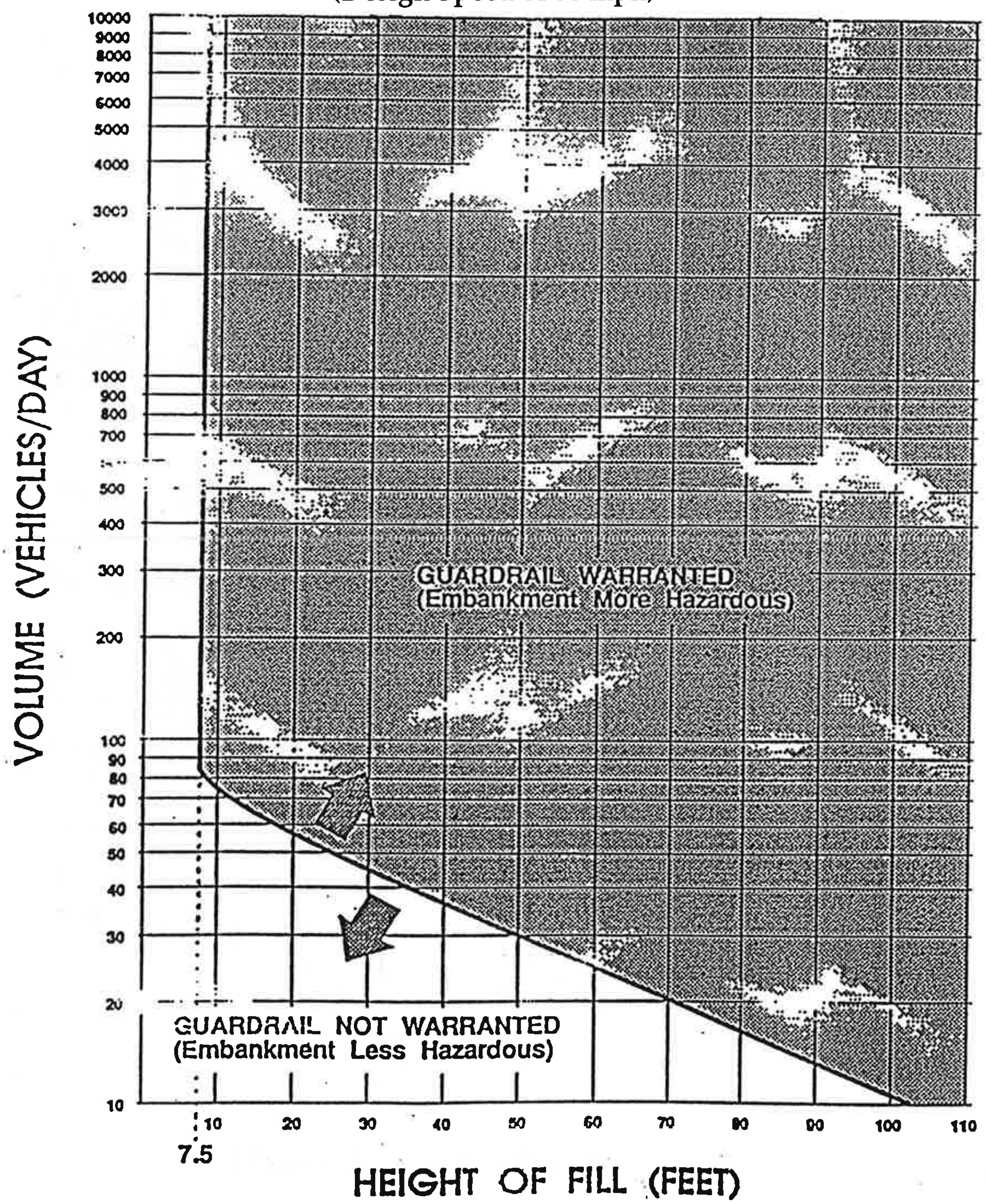
Source: Miaou, Shaw-Pin Transportation Research Record 1599, *Estimating Vehicle Roadside Encroachment Frequencies by Using Accident Prediction Models, 1997.*

Figure II-8 (a)
Guidelines for Guardrail on Secondary Roads
(Design Speed of 30 mph)



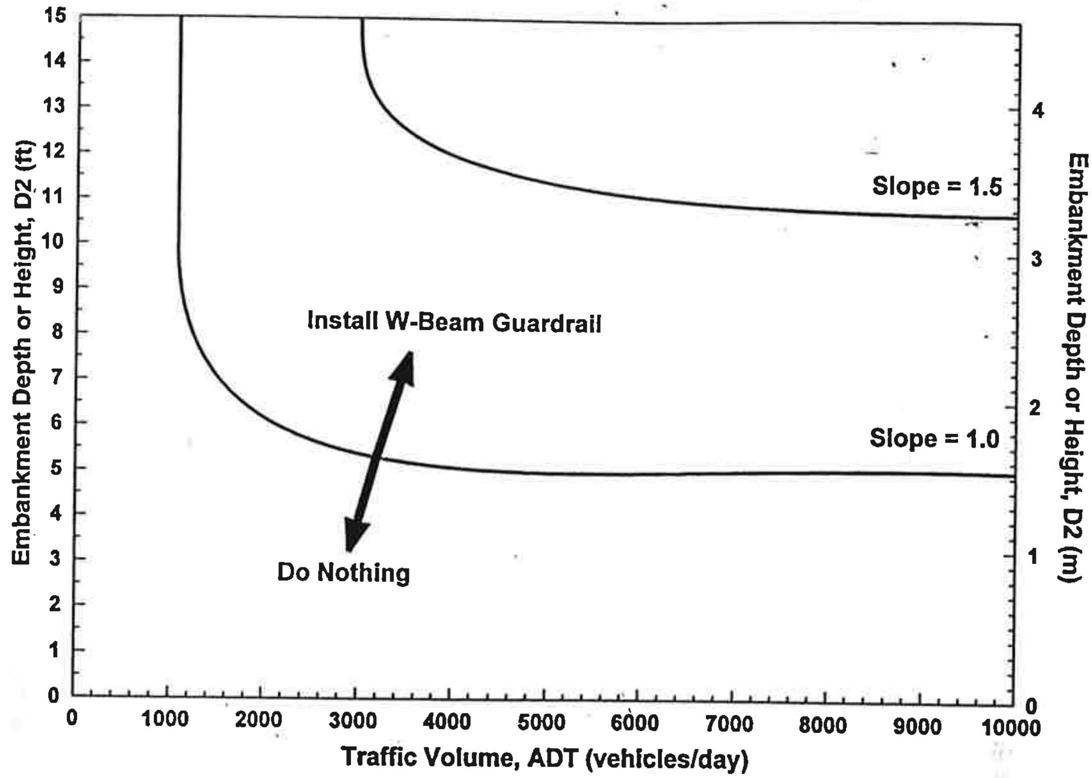
Source: Arnold, E.D., Guidelines for Guardrail on Low Volume Roads.

Figure II-8 (b)
Guidelines for Guardrail on Secondary Roads
(Design Speed of 50 mph)



Source: Arnold, E.D., Guidelines for Guardrail on Low Volume Roads.

Figure II-9
W-Beam Guardrail Beam for Embankments



Source: Transportation Research Record 1599; *Guardrail Need – Embankments and Culverts*

NCHRP 362 (70), reported a sensitivity of functional classification to accident rates on low volume roads. Zegeer's analyses showed functional classification to be a statistically significant variable in a number of basic accident models. Analyses of validation databases also showed apparently higher accident rates for local versus other class unpaved roads. Of course, the researchers noted that there is a strong relationship between functional class and other variables such as roadside. In other words, higher accident rates computed for lower class facilities are largely attributable to the fact that such roads tend to be narrower and with lower quality alignment and roadway width.

Construction and Maintenance Costs

Methods varied in determining construction and maintenance costs throughout the literature review. Although models were typically rated against a justifiable user cost, there was a difference in the variables used to calculate these indices. Some models placed significant emphasis on accident data, while others discounted this information entirely. Construction costs varied by seasonal conditions, roadway use, and pavement type. Other, less perceptible, cost differences resulted from varying public policies which determine the frequency of grading and roadside maintenance.

A widely used model is the World Bank's Highway Design and Maintenance Standards Model (HDM-III) discussed by Riley, M (53) and Kerall, H. (33). This model quantifies travel time savings that result from roadway improvements, as well as, the potential for economic development, and the increased ability to transport raw materials and resources to market.

The biggest drawback of HDM-III is that it looks specifically at the economic value of roads and does not incorporate safety issues into its model.

Mercier (40) discusses the Iowa Design Exception Process which generally follows the AASHTO design values, with some modifications to fit Iowa conditions. The benefit-cost determination worksheet is included in Appendix A. The general process for an agency to attain a design exception is as follows:

1. Acquire accident data for the roadway,
2. Prepare an estimate of the cost to bring the deficiency up to current standards,
3. Calculate the benefit-cost ratio, using accident data, cost estimates, and the prescribed benefit-cost determination process.

This process is used for both new construction and 3R projects, but the majority of agency requests are for 3R projects.

Special Features of Very Low Volume Roads

The literature search uncovered a number of recent papers on special features of very low volume roads. These include turnout design and operation, and one-lane roads.

A paper by Oh (48) looked at capacities of one-lane roads used by the USFS. The author developed speed-flow relationships for a number of sites, resulting in predictive speed models. Oh established that the theoretical capacity of a one-lane road could be as great as 400 vpd when traffic can be controlled with the use of CB radios and turnouts.

Zegeer, et al.(70) looked at the relative safety of paved vs. unpaved roads, and determined that, for roads with ADT less than 250 vpd, there is no appreciable difference in the accident experience for paved versus unpaved roads.

Summary of Findings From Literature

There is much recent research on safety and operations of rural roads, and some on urban roads. Little of the research is directed toward very low volume roads, or to roads classified as "local." Those studies that have been done tend to focus on construction and maintenance costs, or on surveys of engineers faced with low volume road problems.

Despite the relative lack of information on safety and operations of very low volume roads, there is sufficient research that can provide positive direction to many of the issues concerning LVLRoad design criteria. Indeed, a clear picture emerges when one combines what is known from surveys of engineers with the research on rural roads in general.

Table II-17
Summary of State Practice Guardrail versus Embankment on Low Volume Roads

Number of States	Warranting Conditions	Exceptions to Warrants for Low Volume Roads	Comments
16	Based on AASHTO Barrier Guide (Slope and Height of Fill)	None	
3 (Louisiana, N. Dakota, Tennessee)	Slopes Steeper than 3:1	—	All Volumes
1-Connecticut	Slopes Steeper than 6:1	—	All Volumes
3 (Indiana, California, Florida)	Height and Slope	—	Warrants differ from AASHTO (tolerate greater heights of fill)
2 (West Virginia, Kentucky)	AASHTO Barrier Guide	"Engineering judgment for low speed, low volume roads"	
1 (Rhode-Island)	AASHTO Barrier Guide	Engineering Judgment and Accident Analysis for low volume rural roads	
1-Hawaii	AASHTO Barrier Guide	For speeds less than 40 mph, determine need at field inspection	
1-Wisconsin	AASHTO Barrier Guide	Standard practice is to <u>not</u> install guardrail when current ADT is less than 300 vpd.	
3-Georgia, Idaho, Texas	"Georgia Curves Based on slope height of fill and traffic volume		
1-North Carolina	North Carolina curves—based on slope, fill height, length of need, volume, and speed		
1-Michigan	AASHTO Barrier guide	Federal Aid Local Agency Projects—warrants tolerate greater heights of fill	
1-Pennsylvania	Warrants based on slope, height of fill and traffic volume	Less than 400 vpd—2:1 slope requires 31 ft. height of fill to warrant barrier	3:1 or flatter—guardrail not warranted
1-Colorado	AASHTO Barrier guide	3-step approach takes into account volume	
1-Alaska	Warrants based on height vs. ADT	Special Design Curves for ADT <750 vpd	Barrier not warranted for ADT less than 300
1-Illinois	Warrants based on volume, slope and height of fill	Barrier not warranted for ADT <400 vpd	
1-Montana	Special Procedure		Hazard values computed based on height, alignment, climate, width of roadway, slope, ADT, grade

Source: Arnold, E.D. "Guidelines for Guardrail on Low-Volume Roads," 1990.

Table II-18
Summary of State Practice Clear Zone Width for Low Volume Roads[1]

Number of States	Practice as Reported	
27	Based on AASHTO Guidelines or Values	
12	Statement or Policy Providing Exceptions to AASHTO Guidelines	
	West Virginia	Engineering Judgment for low volume, low speed
	Kentucky	Need determined for low volume, low speed at Field Inspection
	Texas	16 ft. Desirable; 10 ft. minimum for roads with ADT < 750
	Idaho	10 ft. minimum for rural
	Wisconsin	Guardrail not used for ADT < 300 vpd
	Rhode Island	Engineering judgment and accident analysis for low volume, low speed
	Hawaii	12-ft. clear zone
	Alaska	Design speed < 40 mph, ADT < 750 vpd Clear Zone is 7 ft.; ADT > 750 vpd, Clear Zone is 10 ft.
	Ohio	Waives clear-zone requirement for roads with Design Speed less than 40 mph
	Florida	14 ft. desirable and 10 ft. minimum for all rural locals
	Illinois	ADT <400 vpd, 10-ft. clear zone
	Indiana	10-ft. clear zone for rural local roads

[1]Source: Arnold, E.D. "Guidelines for Guardrail on Low-Volume Roads," 1990

With respect to safety considerations, the following represents what is considered to be a strong consensus of the literature and opinions of engineers who operate and maintain the low volume road system:

1. The single most important consideration regarding safety on very low volume roads is the roadside. This is primarily by default, in that multiple vehicle crashes are rare where traffic volume is low.

2. There appears little justification for clear zone dimensions or policies such as are currently discussed in Chapter V of the AASHTO Policy, at least for roads with very low volumes. Studies of cost-effectiveness suggest that clear zone or guardrail policies on such roads do not begin to approach cost-effectiveness until volumes exceed 300 to 400 vpd.

3. With respect to alignment issues, the research on safety effects of horizontal curves offers some insights, although these must be carefully considered given that the underlying databases from which the published relationships were developed are for higher volume facilities. Curvature that is "too sharp" *may* be a problem, but

usually when such curvature is accompanied by other conditions such as poor roadside, poor pavement condition, or other confounding geometry.

4. Current design practice for stopping sight distance does not appear to represent cost-effective practice for very low volume roads. Indeed, there is much evidence that SSD design policy does not produce meaningful safety benefits for higher volume facilities. The SSD operational model and assumptions used for design may explain much of this finding.

5. Design consistency (defining it, achieving it) is of great interest to researchers and practitioners. There is much recent research on vehicle speed behavior, on speed as a surrogate for safety, and on measuring consistency of highway alignment with respect to speed and speed changing.

6. Environmental and other issues relating to the right-of-way acquisition are serious considerations for LVLRoads. Design engineers have difficulty justifying the costs and impacts of widening or realignment in most cases.

An important theme that is apparent from the research is a concern over the reasonableness or applicability of the traditional design approach for very low volume roads. This approach includes establishment of basic design controls, among these being "design speed." An appropriate design speed is selected based in part on speeds that the majority of drivers are willing to drive. Design criteria for alignment, cross section and other features are determined with reference to design speed, with higher speeds tending to produce more costly and impacting dimensions.

This approach leads to what some believe are overly costly designs, or criteria that become too difficult to accomplish. Although a number of authors have expressed a dissatisfaction with the design speed concept as it applies to very low volume roads, none has suggested an alternative approach, other than to note that design consistency is what should be emphasized.

LVLRoad Maintenance Considerations and Practices

Maintenance activities are a major aspect of LVLRoad system operations. As noted previously, a survey of county engineers revealed that many if not most devote most of their time and budget to maintenance versus capital improvements. Maintenance needs and activities are affected by the physical attributes of the roadway system.

An additional phone survey was undertaken by CH2M HILL to characterize the issues involved with low volume road maintenance. This survey of county engineers was conducted to determine typical roadway maintenance conditions and their relative costs. Engineers were invited to discuss their field experiences and relate personal insights. Five of the six engineers who were contacted were responsible for road systems that carried less than 400 vehicles per day on over 50 percent of their roads.

Engineers were first asked to discuss their biggest maintenance concerns. Their responses suggested that the issues should be divided into two categories: policy and design/maintenance concerns. From a policy viewpoint, county engineers are concerned about lack of funding, limited purchasable right-of-way, and lack of consistency in maintenance standards. From a design/maintenance point-of-view, the greatest concerns are ability to maintain good running surfaces on both paved and non-paved roads, ability to keep ditches clear ~~of~~ debris, and the elimination of low water crossings.

Issues Related to Geometric Design Criteria

The engineers were asked how they perceive their maintenance concerns reflect geometric design standards. Most believed that the ability to achieve geometric standards is limited by budgetary constraints and the amount of purchasable right-of-way. Existing roads that do not meet geometric design standards are often surrounded by farmland. It is extremely difficult to purchase additional right-of-way from landowners to correct these deficiencies.

Based on the limited annual budgets, county engineers also recognize that it is not possible to correct all system deficiencies. Therefore they must use their best judgement in determining where to allocate the monies.

Overview of Maintenance Activities and Costs

The most common routine maintenance activities, excluding snow removal, are repairing pavement surfaces, grading non-paved roads, controlling vegetation growth, and performing traffic services. Traffic services tends to be a compilation of replacing signs, adding new signal controls, and adjusting timing at existing traffic signals. Ditch maintenance, such as replacing culverts and keeping ditches clear of debris, is also regularly performed.

Table II-19 provides an overview of the ranges of reported annual costs per mile for major maintenance activities. The total cost for maintaining both paved and non-paved roads varied from $2,700 to $7,510 per mile per year.

Table II - 19
Typical Maintenance Costs Experienced in Counties with Predominantly Low Volume Roads

Maintenance Item	Annual Cost Per Mile
Resurfacing	$200 - $1,2000
Snow Removal	$800 - $1,300
Minor Surface Repair	$350 - $800
Traffic Services	$550 - $650
Vegetation Control	$60 - $150
Reshaping	$70 - $130
Ditch and Culvert Maintenance	$50 - $100
Dust Treatments	$5 - $60

Although the retrieved data did not divide the costs between paved and non-paved roads, county engineers commented that it is considerably more expensive to maintain a non-paved road than a paved one. As traffic volumes increase on a non-paved road the maintenance costs increase to a point where it is more financially feasible to seal the road than to continue to add crushed rock. Because of this, several of the counties have programs in place to pave the roads classified as minor collector and above over a period of several years.

County engineers were asked to list any known cost sensitivities. The most common response was that maintenance costs are very sensitive to geographic location. Topography, weather, and soil conditions have a significant impact on the cost per mile to maintain a road. Winter maintenance is twice the cost of surface repairs in some areas of the country.

One contributing factor to the cost of winter maintenance results from the application of dirt and sand to roads during the snow season. Over time the dirt washes off the roads and clogs the ditches and culverts. This debris must be cleared out before the rain season begins to avoid washouts.

Another sensitivity to cost is the type of vehicle using the system. Counties are experiencing a surge in the size of farm vehicles and the distances they travel. As agriculture equipment continues to increase in size and weight, plank bridges and culverts are becoming too narrow and cannot support the increasing loads. Due to the age and type of the existing bridges and culverts, it is often difficult to widen these structures, requiring reconstruction. The wide loads are also deteriorating the roads at a faster rate than previously experienced.

The cost and supply of road materials also plays a key role in determining what types of roads will be constructed and maintained in a region. One county engineer commented that the amount of patchwork they do during the summer is dependent on the cost of oil that year. Another stated that they continue to maintain crushed rock roads because rock is very inexpensive in their area. However, another county engineer limits the amount of crushed rock he uses because it must be imported from another area.

Upgrading Unpaved Roads

They were then questioned as to what the determining factors are for upgrading to a paved road. Respondents gave a variety of reasons for the decision including dust and maintenance problems, political pressures, and an ADT greater than 250-300 vehicles per day.

One respondent discussed a two-part process, suggested by the Kansas Department of Transportation (KDOT), that his county has used for over 30 years to determine when a gravel road should be paved. The process requires a double chip and seal to be applied to roads with an ADT greater that 100 vpd. When the ADT becomes greater than 350 vpd a three inch asphalt mix is applied to seal the road. An added benefit of the two-part process is that it clearly establishes guidelines for residents who request pavement on very low volume roads.

Roadside

Engineers were asked to discuss the guidelines they use in determining their clear zone and guardrail policies. Most have adopted state policies. Kansas State University is in the process of completing a research project which may be used to develop a set of Low Volume Road Standards for use throughout their state. The established policies and standards are followed for new construction and road rehabilitation. However, due to budgetary constraints, older roads usually are not upgraded to meet these standards. It was noted that the older routes are typically collector streets and are traveled by drivers who are familiar with the roads.

Use of Accident Data to Support Maintenance Activities

County engineers were questioned regarding the use of accident records to identify and treat problems on their system. Each of the respondents currently uses bi-annual crash reports to evaluate potential problem areas on their system. High accident locations are also identified. A universal problem with crash records is the misrepresentation of crash locations in police reports. However, the county engineers stated that they are usually aware of most crash occurrences and able to interpret the accident location data.

The crash records were found to be most useful in making spot repairs and adjusting signal timing at intersections. In some instances, horizontal and vertical curvature was modified based on repeat crashes. Often politicians pressure the county to make spot improvements in locations that are perceived by their constituents as having safety problems, even though they are not high accident locations.

They also noted that in areas such as Kansas, where the state has mandated a 55-mph speed limit on rural roads, the crushed rock roads pose a safety hazard. Past experience has proven that surfaces with too little or too much crushed rock are unsafe for motorists travelling at high speeds.

Evolving Concerns

Future maintenance concerns were also discussed. The replacement of drainage structures throughout the counties may soon become an issue. Many of the existing culverts and bridges were built during the early 1900's and are considerably older than the design life. The costs to replace these structures will significantly impact the already limited budgets.

III. Framework for LVLRoad Design Criteria

Following a review of design practices and criteria in use by others, an assessment was made of the appropriate framework for consideration of LVLRoad design values. This chapter describes the framework, which was reviewed with the Project Advisory Panel and revised to meet comments and input from the panel.

Geometric design criteria for very low volume local roads should be considered within the following basic variables: *area type* (i.e., urban versus rural), *functional classification*, *design and operating speed*, *traffic volume*, and *terrain*. These basic variables were found by the researchers to represent significant, meaningful descriptors of the range of safety, operational and/or cost sensitivities for very low volume local roads. Note also that these variables are currently considered in the development and presentation of design criteria in the 1994 AASHTO Policy on Geometric Design.

Area Type

There are significant differences in the operating characteristics, constraints, costs, and configuration of basic geometric elements for roads in different locations. The former tends to include higher speed operations, open drainage, somewhat lesser right-of-way constraints, and little or no pedestrian activity along the roadway. Certain unique uses are also found in rural areas, primarily involving agricultural activities.

Low volume local roads in urban and suburban areas generally are more constrained in terms of speeds and right-of-way. Design criteria for very low volume roads should be developed and discussed separately for the rural and urban environment.

Functional Classification

The subject of this research and Chapter V of the AASHTO Policy is local roads, as different from collector or arterial roads or highways. According to AASHTO, a local road "primarily serves as access to the farm, residence, business or other abutting property." Even given this narrow classification, it is clear from previous work and discussions with Project Advisory Panel members that additional functional breakdowns are necessary.

There may be differences of opinion or different terms used to describe the various classes. However, there is a consensus among a number of agencies that local roads have a wide range of functions and can be classified accordingly. First, the term "local road" can refer to the jurisdiction controlling the design and maintenance of the road. Second, a common feature of all local low volume roads, is their primary function as access to abutting land uses. Certain "local" roads (as defined solely in terms of jurisdictional responsibility) may actually function as collectors or arterials within a confined area. These are not considered as part of this study.

Even with these two aspects of functional classification, there is still a wide range of possible different functions of local roads. The following is our proposed set of classifications, which correspond to the different types of land use being accessed, and to a degree, to the different types of roadway users. (Note that, regardless of the class or range in uses, this study is confined to local roads with traffic volumes less than 400 vpd).

Major Local

Major local roads serve a dual function of access as well as through or connecting service between two equal or higher type facilities. In rural areas, a major local road may have significant local continuity and length (say, 5 to 10 km or more), and may offer service at a relatively high speed. In urban areas the length of such a road would be shorter, but its function in through service as opposed to just local access is the same. Because of the possibility of through traffic, there may be a meaningful segment of traffic that includes unfamiliar drivers. Major local roads may thus in some respects function similarly to collector or even minor arterial roads, particularly in rural areas where even arterials carry low traffic volumes.

Major local roads will tend to have a paved or treated surface. In rural areas, unpaved major local roads may be more prevalent.

Minor Access

Minor local access roads serve essentially as access to adjacent land uses. Many of these roads would be cul-de-sacs or loop roads, with no through continuity. The length of a typical minor local access road is generally short (say, less that 5 km in a rural area, and even shorter in an urban area). Because their sole function is access, such roads are predominantly driven by familiar drivers.

Minor access roads generally serve residential or non-commercial land uses. Speeds are generally low for the local environment, given the purpose of the road and short trip lengths. As noted above, many of these roads end in culs-de-sacs or dead ends, thus negating the opportunity for significant speeds. Traffic is largely passenger vehicle or other smaller types. However, such roads need to be accessible by school buses, by fire and other emergency vehicles, and by maintenance vehicles such as snow plows and garbage trucks. Access roads serving commercial or industrial land uses are classified separately.

Minor local access roads will frequently be narrow, often functioning as one-lane roads. They can be unpaved, although in urban areas such roads are often paved.

Industrial /Commercial Access

Industrial or commercial access local roads serve developments that may generate a significant proportionate volume of truck or other heavy vehicles. Such roads primary or sole function may be to provide access from a factory or other commercial use to the local or regional highway network. A typical industrial/commercial access road would typically be short, and in many cases not serve as through routes. Industrial or commercial access roads exist in both urban or rural areas. They can be paved or unpaved.

Note that industrial or commercial access as a functional class may be considered by some as a subset or part of a more broadly defined "local access" class of road. For the purposes of investigating design sensitivities associated with larger vehicles, this separate class is considered useful.

Agricultural Access

Certain roads in rural areas primarily serve as access to fields and farming operations. Agricultural access roads are public roads with little or no continuity, with traffic generally limited to one or more farm operations. Vehicle types include tractors, combines and other large and slow-moving vehicles with unique operating characteristics. The driving population is repeat users, (i.e., familiar with the road and its characteristics). Such roads are often unpaved.

Agricultural access roads may also be viewed as a subset of minor local access roads. Again, retention of this separate classification will enable identification of unique design needs associated with agricultural vehicles such as combines.

Recreational and Scenic

Recreational and scenic roads serve specialized land uses such as parks. They occur in both urban and rural areas (i.e., an urban recreational road may be a roadway to a boat ramp). Traffic is open to the general public, and the users may generally be unfamiliar drivers. These types of roads would not carry significant volumes of truck traffic, but would carry recreational vehicles, passenger cars with boat and other trailers, etc. In many cases, these roads may carry highly seasonal traffic volumes.

Recreational and scenic roads may accommodate a wide range in speeds. Trip lengths may be fairly long. Such roads can be either paved or unpaved.

Resource Recovery

Resource recovery roads refer to local roads serving logging or mining operations. These would typically occur only in rural areas. (An example of an urban resource recovery road would be one serving a quarry. This might also be classified as an industrial or commercial access road, for reasons described below.)

Resource recovery roads are distinctly different from the other classes in that access to them may be restricted by permit. Only vehicles involved with the recovery activity would be allowed on them, and the driving population is essentially professional drivers with large vehicles. In some cases traffic control on these roads is enhanced through citizens' band radios, enabling the roads to be built and operate as single lane roads. Most resource recovery roads are unpaved.

Design Speed/Operating Speed

Speed has always been a primary, defining variable in the development and presentation of geometric design criteria. Current AASHTO Policy specifies design criteria as a function of design speed in 10 km/h increments. Designers select a design speed, which is to be appropriate given the conditions, and which is meant to represent a speed at which the majority of drivers will operate.

For the purposes of establishing a research framework, the recommended structure is to evaluate safety and operational sensitivities for speed in three ranges. *Low speeds* will include nominal design speeds of 20 to 50 km/h. *Moderate speeds* will include 50 to 80 km/h design speed roads. *High speeds* will include roads with nominal design speeds of 80 to 100 km/h.

The subject of design speed and its use/relevance to design has been continually debated. Ongoing research is once again addressing design speed issues. It is beyond the scope of this study to re-visit or question the use of design speed as a central input to design of all roads, including LVLRoads. Figure III-1 illustrates the ranges of speed considered appropriate for the area types and functional classification. Note that terrain is a factor in design speed.

Traffic Volume

Much of the research reviewed for this study showed traffic volume sensitivities related to safety. Lane and shoulder width, guardrail and clear zone criteria, paved versus unpaved roads and other criteria are volume-dependent within the range of 0 to 400 vehicles per day. Operational sensitivities related to traffic volumes are also evident. There is a significant difference between a road carrying, say, 75 vehicles per day and one carrying 350 vehicles per day in terms of interactions between vehicles and exposure to potential safety concerns.

There are indications that operational or safety sensitivities may occur for ranges within the 0 to 400 vpd limits. The following ranges were defined and applied to the LVLRoad study, and used in defining appropriate geometric criteria:

- 100 vehicles per day or less;
- 101 to 250 vehicles per day; and
- 251 to 400 vehicles per day.

Terrain

The retention of AASHTO definitions of *level, rolling,* and *mountainous* terrain is recommended for the evaluation and presentation of many of the criteria. Design speeds, and construction and maintenance costs are all influenced by the prevailing terrain.

LVLRoad Design Philosophy

The nature of the AASHTO Policy and its translation into design standards by some agencies has long created problems for those faced with constructing and maintaining low volume local roads. Problems concern the design values, and the types of projects for which the criteria apply.

The AASHTO *Policy on Geometric Design* discusses in its foreword the applicability of design criteria to all types of highways. The Policy notes that the criteria contained within apply both to new construction as well as reconstruction projects. It also suggests that designers should use the values as guidelines, acknowledging that local conditions or constraints may dictate a design.

The introduction to Chapter V states:

> *"In restricted or unusual conditions it may not be possible to meet these guide values. Every effort should be made to get the best possible alignment, grade, sight distance, and proper drainage that are consistent with the terrain, the development (present and anticipated), the safety, and the funds available." (p.417)*

Although it appears the intent of this language is to provide flexibility, AASHTO's reference to "restricted or unusual conditions" may not be sufficient to give local road engineers the confidence to use lesser design values. As noted in our survey of county and local engineers, restricted conditions are not unusual, but rather the norm.

Figure III-1
Guidelines for Design Speed for Low Volume Roads

	URBAN Design Speed (km/h)							RURAL Design Speed (km/h)									
	20	30	40	50	60	70	80	20	30	40	50	60	70	80	90	100
Major Local		(✓)[1]	✓	✓	✓	(✓)[2]				✓	✓	✓	✓	✓	(✓)[2]	(✓)[2]
Minor Access	(✓)[1]	(✓)[1]	✓	✓	✓				(✓)[1]	✓	✓	✓	✓	✓		
Industrial/Commercial Access	(✓)[1]	(✓)[1]	✓	✓	✓				(✓)[1]	✓	✓	✓	✓	✓		
Agricultural Access									(✓)[1]	✓	✓	✓	✓			
Recreational and Scenic	(✓)[1]	(✓)[1]	✓	✓	✓	(✓)[2]				✓	✓	✓	✓	✓	(✓)[2]	
Resource Recovery	(✓)[1]	(✓)[1]	✓	✓	✓			(✓)[1]	(✓)[1]	✓	✓	✓				

(✓)[1] Generally applies only to roads in mountainous terrain

(✓)[2] Generally applies only to roads in level terrain

As noted above, there are a number of problems with application of AASHTO values to the LVLRoad system. Design values have been derived based on general rationale, operational models, or a series of assumptions. Because of the intended use of the design values, such derivation has tended to be conservative with respect to issues of safety (a good example here is the historical basis for stopping sight distance). The design values err on the side of calling for *more* rather than *less*. Furthermore, design values are interpreted as being "minimums" to be applied continuously unless otherwise justified. This justification process entails evaluation and documentation of "design exceptions." County and other local engineers often do not have the time or resources to adequately document all design exceptions, particularly given that, under current criteria, such exceptions are the rule rather than the exception. Finally, the AASHTO Policy purports to place emphasis on "cost-effectiveness." This concept has been more broadly interpreted to include not only user benefits and costs, but to consider environmental factors, and to consider systemwide issues such as priorities for improvements, overall project justification, etc.

All of the above produce significant conflicts when one considers the unique nature of low volume local roads. By definition, low volume implies few users who will benefit directly from the service provided by the road. The difference in quality of a marginal or "substandard" design versus a "well-designed" low volume local road may be impossible to measure, for the only reason that so few use the facility that quantifiable operational measures are unattainable. Local implies familiar users, shorter trips, and for the most part lower speeds. All of these tend to negate the rationale for many of the conservative assumptions imbedded in design philosophy for other roads. Of course, there are always well identifiable costs one can assign to construction and maintenance of the low volume local road system. Moreover, many of these costs are unrelated to the level of traffic volume.

In short, there is clear justification for considering design of low volume local roads with a different framework than other segments of the system. This view was stated many years ago in NCHRP 63, *Economics of Design Standards for Low-Volume Rural Roads* by Oglesby, et al. The authors suggested, among other things, "abandoning the requirement that a constant cross-section be maintained for the full length of a road segment. Rather, the standards could suggest that narrower roadbeds be considered for straights (sic) than on curves or over crests where sight distances are limited......." Further on, it was suggested that "changing the philosophy for prescribing standards for roadbed width by recommending maximums instead of minimums."

Others, more recently, have suggested that the safety audit concept is more appropriate to low volume local roads. Focus should be placed on identifying and mitigating or avoiding combinations of geometric conditions or features that it is believed represents a greater risk to drivers. Here, the concept holds that such analysis during design can make the road safer without increasing its cost.

The above is not to say that low volume local roads are so different as to require or justify a completely different design approach. Low volume local roads are viewed as on the continuum of highways, with high volume freeways on the opposite end of the continuum. This means that reasonably appropriate, fundamentally sound principals apply to development of design criteria for low volume local roads, just as they do to all elements of the highway system.

It is beyond the scope of this research effort to revisit completely the design philosophy of low volume local roads. The intention is not to propose abandonment of recommended design values, nor to recommend adoption of maximum versus minimum set of values as was espoused by Oglesby. Nonetheless, the research approach does focus on taking a hard look at design criteria that 1) result in demonstrable costs or difficulties in their implementation, and 2) offer little or no hope of providing measurable, systemwide benefits in the low volume local road environment. In effect, the researchers suggest that the "burden of proof" be placed on a design approach for low volume local roads, particularly where the costs of that approach are readily apparent. Where such burden is not met, and the design criteria or approach appears overly conservative, alternative rationale and/or design values are suggested based on the research.

Measurable Benefits

This study will not address the economic justification for a road being constructed. It is thus implied that a certain minimum functional design characteristic is required. Within this context, the "benefits" of alternative geometric design criteria would generally be based on the following:

- Travel time savings or reduction in delay
- Comfort and convenience
- Reductions or savings in maintenance costs
- Reductions or savings in construction costs
- Reductions in crash frequency and/or severity

For the LVLRoad environment, the first two items are not considered of sufficient value or importance to affect decisions regarding design criteria. The short-trip nature of local roads implies that all travel on LVLRoads will be short in both distance and time. Congestion or delay on all but single lane roads is not a factor given the traffic volume levels. Marginal improvements to overall travel speeds produced, for example, by using a flatter horizontal curve, would be negligible for the volumes and speed differentials being considered.

Comfort and convenience are more difficult to quantify. For LVLRoads, the primary convenience is the existence of the road itself. Any presumed or identified benefits of comfort associated with normal highway design practice are discounted in considering LVLRoad design practice.

It is the contention of the researchers that design criteria for LVLRoads should be justified solely on the basis of 1) a demonstrable monetary benefit associated with savings in maintenance or construction costs; or 2) an estimated positive impact on highway crash frequency or severity. The former is identified based on the literature and expert opinions of local roads engineers. The latter is to be estimated using the best available information, adapted to a "risk assessment" approach outlined below.

Reconstruction Versus New Construction

The AASHTO Policy treats reconstructed highways the same as newly constructed highways in terms of the applicability of design criteria. Some states have refined this process, adopting, for example, "minimum" SSD values as sufficient for reconstructed roads, but requiring the full or "desirable" SSD values for new highways.

The researchers recommend that design criteria for LVLRoad consider in both form and design values the fundamental differences between new alignment and reconstructed roads. Of primary importance here is the presumed knowledge for reconstruction projects of the previous operational and safety history of the road.

Engineers responsible for the operation and maintenance of LVLRoads bear the responsibility of knowing and understanding the safety and operational characteristics of the road in question. The following information is necessary for reference in a reconstruction project:

- Traffic volumes (counted or estimated to reasonable accuracy) to enable classification within the LVLRoad definitions of 0-100 vpd, 101 to 25 vpd, and 251 to 40 vpd;
- Estimates or measures of operating speeds along the road; and
- Understanding of safety performance of the road, including known reported crashes, preferably over the past 10 years, but for at least 5 years; anecdotal or other evidence of safety problems from complaints by drivers, discussions with local police and road users, or physical evidence of roadside encroachments.

To the extent possible, LVLRoad design criteria for reconstruction projects should emphasize allowing the existing basic geometry and features to remain, if the costs or difficulty of upgrading or improving the road are substantial. This approach is valid as long as the design engineer demonstrates an understanding of the operational history of the road, and based on that history, judges that no undue risk will be afforded the traveling public.

Risk Assessment Guidelines for LVLRoad Design Policy

With the emphasis on safety as the primary measure for LVLRoads, it is necessary to establish a basis for quantifying safety. The term *risk assessment* refers to the evaluation of the relative safety of a design alternative, action or policy. Traditional measures of crash risk include crash rates and severity, or perhaps crash frequencies.

Safety Risk Assessment

In the case of very low volume local roads, traditional measures of safety may be of limited value. A crash rate of, say, 3.0 crashes per million vehicle kilometers sounds high compared to rates for other highway types. However, such a rate may translate into a very small number of crashes per year, given the extremely low volume. Consider, for example, Table III-1, which shows crash rates and expected crash frequencies for typical values for low volume local roads. One should expect that only one accident every 4 years or so per km is typical. For very low volume roads, a crash every 10 to 20 years per km may be the norm. Note also that not all such crashes will result in an injury or fatality. Clearly, the extreme infrequency of crashes produces the result that locating crashes or identifying "high crash" locations is essentially a meaningless exercise in a very low volume environment. [This is not to suggest that monitoring crash records should not be done. Indeed, such activity should always be undertaken as part of an ongoing program of road system maintenance. Reliance solely on site-specific data for risk assessment of LVLRoads will yield few insights.]

Table III-1
Quantifying Crash Risk on Low Volume Roads

Average Daily Traffic	Crashes per km per year for Given Crash Rate		Years for One Crash per kilometer	
	1.5 per mvk[a]	2.5 per mvk[b]	1.5 per mvk[a]	2.5 per mvk[b]
100	0.055	0.092	18.2	10.9
250	0.137	0.229	7.3	4.4
400	0.22	0.367	4.5	2.7

[a]Related Accident Rate (Single vehicle and opposite direction) for 10 ft lanes and narrow shoulders derived from NCHRP Report 362.
[b]Related Accident Rate for unpaved roads with ADT < 400 vpd, from NCHRP Report 362

This study addresses risk assessment associated with a given design policy in different terms. One parameter of interest is ***years to accumulate the risk of one additional crash.*** Or even more precisely, years for a serious (i.e., fatal or injury) crash. As will be shown, many existing design policies applied to low volume local roads will appear to produce such low risk that, for a given section of low volume road, the apparent risk per mile of an additional crash due to a given action requires as much as 50 years or more of operation of that road.

In some cases, risk assessment may be expressed in terms of probabilities of events occurring. Establishment of an alternative design approach or criterion may require acceptance of a defined level of risk of failure. (An example is the use of percentiles of driver perception/reaction times in stopping sight distance design). Reference to existing AASHTO design procedures or values, and estimation of risk levels associated with them, provides context and a basis for judging acceptable levels of risk for alternative design approaches applied to low volume local roads.

Capital Costs of LVLRoads – Input to Risk Assessment

Assuming the above approach is reasonable, it is necessary to establish a threshold or range of thresholds that would represent a level of cost-effectiveness for assessing the value of an alternative design policy or value. The following is a derivation of the basis for establishing a threshold for reference in the research.

Consider Table III-2, based on data taken from NCHRP Report 362. Construction cost data on a per mile basis were taken from NCHRP 362 for two-lane roads designed to accommodate less than 400 vpd, and less than 10 percent trucks. The difference between an 18-foot road with no shoulders and a 20-foot road with 4-foot shoulders is considered representative of a typical choice between a "substandard" design and a design that would meet criteria. Based on the cost model information from NCHRP Report 362, one can compute a difference in costs per mile (or km as converted here). Note that this cost varies with terrain, which reflects a range in expected earthwork costs. Thus, the *difference* or range in design quality between the two designs is $180,000 per mile to $240,000 per mile.

The cost model and data from NCHRP Report 362 were based primarily on higher class roads, with more costly pavement designs and overall greater earthwork than might

be representative of LVLRoads. Therefore, as a conservative assumption, the cost difference is multiplied by 60 percent, to reflect an expected lower overall cost structure for this type of road. When the per mile cost is annualized (at 6% discount rate over 20 years) and converted to a per kilometer cost, the resulting differences between high and low quality designs range from $5900 to $7800 dollars per km per year.

Table III-2
Construction Costs for Low Volume Roads - Costs per Mile (km)
(Based on Unit Costs)

Cross-Section	Terrain	Rural Highways[1]	Local Roads[2]
Representative			
'Substandard'	L	$560,000 ($350,000)	$336,000 ($210,000)
18-foot Traveled Way	R	$600,000 ($375,000)	$360,000 ($225,000)
0-foot Paved Shoulders	M	$960,000 ($600,000)	$576,000 ($360,000)
Representative			
'Basic'	L	$740,000 ($462,500)	$444,000 ($273,500)
20-foot Traveled Way	R	$790,000 ($493,750)	$474,000 ($296,250)
4-foot Paved Shoulders	M	$1,200,000 ($750,000)	$720,000 ($450,000)

[1] Source: NCHRP Report 362, costs per mile for highways designed for ADT <400 vpd and trucks <10% of ADT
[2] Assumed to be 60 % of costs of Rural Highways

The above calculation suggests that, as a general rule, the capitalized costs per km of constructing an upgraded LVLRoad alignment and/or cross-section versus a lesser design would be on the order of, say, $6000 to $9000. For upgraded geometry to be considered cost-effective in a systemwide or general sense, it should return a comparable value in terms of expected crash reduction savings over the life of the roadway.

Analyses performed in NCHRP Report 362 assigned a dollar value of $60,000 per crash (70). This reflected typical severity distributions for rural roads, and FHWA valuations on fatalities, injuries and property damage only crashes. A comparable value for lower speed, urban roads would be somewhat lower—perhaps on the order of $40,000 – due to the lower percentage of severe crashes.

An expected or predicted reduction in crashes of 0.15 per km per year (or, stated differently, one crash per about 6.5 years) would produce $6000 to $9000 in annual crash reduction benefits per km. This value compares with the annual per km capital costs noted above. Similarly, a crash reduction of 0.1 per km per year, or one crash every 10 years, produces benefits of $4000 to $6000 per km per year.

The above values for crashes suggest as a general guideline the following:

For urban or low speed LVLRoads --

An acceptable safety risk for LVLRoad design is given by an action or proposed approach that is expected to produce no more than ***one crash every 6 to 10 years per km.***

For rural or high speed LVLRoads

An acceptable safety risk for LVLRoad design is given by an action or proposed approach that is expected to produce no more than ***one crash every 10 to 15 years per km.***

The analysis is intended to be general in nature, and to provide an order of magnitude estimate of relative cost-effectiveness. Note, however, how these guidelines compare with the values from Table III-1. The *total* expected crash experience of a typical LVLRoad may range from one per every 11 years per km for ADT of 100 vpd, to one about every three years per km for ADT of 400 vpd.

Summary

Subsequent analyses presented in this report reference the general guidelines established above. Geometric design criteria that produce expected meaningful safety benefits consistent with reducing crashes every 6 to 10 years per km for lower speeds, and every 10 to 15 years per km for higher speeds, are considered justified. Design criteria that do not produce such benefits (i.e., based on literature synthesis), or produce lesser benefits for LVLRoad volume ranges are considered candidates for revision.

IV. Research Priorities and Plan

The research project scope of work noted a limitation in the subject areas that could be addressed by this project, with such limitation based on the available budget. Research priorities therefore were established. These priorities were based on assessment of the literature, and results of discussions with county and other local engineers. The NCHRP 20-7 Task 75 Project Advisory Panel was consulted and concurred with the priorities and research approach.

CH2M HILL recommended the following areas as being the focus of geometric and operational investigations for the balance of the research on LVLRoads:

- Design for stopping sight distance (SSD), including criteria for crest vertical curvature
- Roadside clear zone design criteria, including criteria for guardrail versus slope
- Design criteria for horizontal alignment

The research approach in all three areas was similar. As was noted in Chapter III, primary emphasis was placed on investigating direct or measurable safety relationships. Research involved documenting the historic technical background of the design criteria, assessing the latest literature regarding safety, performing appropriate risk assessments, and recommending guidelines for LVLRoads within the framework established in Chapter III.

Design For Stopping Sight Distance

Design criteria for stopping sight distance (SSD) affect both horizontal and vertical alignment on roadways. Current design methodology for SSD poses particularly significant problems for very low volume roads. This is because many existing roadways appear "substandard" with respect to current criteria. Such criteria have changed over the years, resulting in many roads becoming nominally substandard with respect to current AASHTO criteria.

The research focus with respect to SSD addressed three areas. Chapter V describes the findings:

- Overview of the AASHTO model for SSD, with reference to its relationship to safety and relevance to the unique operations of LVLRoads;
- Synthesis of literature on safety related to SSD, including derivation of threshold values for SSD as a function of design speed;
- Operational risk assessment of SSD using the recently published model for SSD that is expected to be the basis for AASHTO design policy;
- Safety risk assessment of variable SSD values for LVLRoad operating conditions using a SSD model developed by Neuman and cited by others;
- Development of a design approach to SSD for reconstruction and new construction of LVLRoads.

Roadside Design

The condition of the roadside has a significant influence on safety along LVLRoads. Roadside design requirements (Slope, drainage, utilities, clear zones) affect right-of-way requirements, maintenance and construction costs. The potential trade-offs in terms of safety versus cost are great, particularly for higher speed LVLRoads.

Roadside issues of concern in the LVLRoad environment include primarily the need for guardrail or other barrier, and continuous clear zone dimensions. The following research approach was developed. The reader is directed to Chapter VI:

- Overview of the history of roadside design criteria as published by AASHTO;
- Review of literature on safety of roadside design elements, including guardrail warrants and clear zone policies;
- Safety risk assessment of variable roadside design treatments using AASHTO's ROADSIDE analysis design procedures;
- Development of design guidelines for the roadside for LVLRoads.

Horizontal Alignment

Design of horizontal alignment is among the basic contributors to the geometry of the highway. AASHTO design procedures for horizontal curves have remained relatively unchanged for many years. Curve design is a function of the selected design speed of the road.

Research and experience on higher volume roads shows curves to represent special hazards to drivers, particularly on higher speed highways. Engineers responsible for LVLRoad design and maintenance note that design requirements for curves can represent substantial burdens.

Issues to be addressed include the reasonableness of current AASHTO design values for curve radius as a function of speed, for both new design and reconstruction of LVLRoads. The following research approach was developed. The reader is referred to Chapter VII:

- Overview of AASHTO operational "model" for design of horizontal curves;
- Review and synthesis of literature on safety and operations of horizontal curves;
- Operational risk assessment using alternative design approaches for horizontal curve design;
- Safety risk assessment of alternative curve design approaches using crash prediction models from the literature;
- Development of design guidelines for horizontal curves on LVLRoads.

Other Design Issues

Limited project resources restricted the research emphasis to the above three areas. However, additional insights from literature reviews were developed for the following subjects of interest to LVLRoad design engineers. These are addressed in Chapter VIII.

- Lane and Shoulder widths
- Guidelines and treatments for Paved versus Unpaved roads

V. Stopping Sight Distance

Stopping sight distance is among the basic design features of highways. The term sight distance refers to that length of the roadway visible to the driver. According to AASHTO, "the *minimum* sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path." This minimum sight distance has been referred to as stopping sight distance (SSD). Provision for SSD affects geometric design in all three dimensions.

SSD as a fundamental design feature has been part of highway design according to AASHTO since 1940. The 1940 Policy on Geometric Design of Highways included a discussion of SSD, along with the introduction of the simple operational model of a driver perceiving a fixed object and stopping the vehicle prior to encountering the object. Providing for SSD is considered fundamental to geometric design. SSD is among the 13 controlling, critical criteria reviewed by FHWA in consideration of design exceptions.

SSD is among the most influential factors affecting design of vertical alignment. Minimum design values for vertical curvature are based on providing SSD. Similarly, horizontal alignment and cross-sectional values within horizontal curves are controlled by the need to provide SSD through the inside of a curve.

This section of the report addresses design for SSD and its relationship to Low Volume Local Roads (LVLRoads). Included in this section is a review of the history of SSD, literature of the safety and operations related to SSD values, summary of practice by others with respect to SSD, a review of the geometric relationships involving SSD, risk assessment of alternative SSD design bases, and recommendations regarding design policy for SSD for LVLRoads.

History of SSD Design Parameters

An interesting aspect of the history of SSD design policy is its lack of sensitivity to certain, fundamental design controls. Design policy and design values for SSD are and have always been consistent across all highway types (two-lane, multi-lane, access controlled, etc.), location (rural versus urban) and functional classes. Furthermore, design levels of traffic volume are not considered in SSD design. In other words, for a given design speed, the same design values for SSD are considered appropriate regardless of type of highway or level of traffic volume. The fundamental operational "model" describing the requirements of SSD has remained unchanged for almost 60 years. As described in the 1940 AASHO Policy, and carried to the present day as noted in the 1994 Policy, SSD is the sum of two distances: the distance traversed by the driver in the time to perceive and react to the presence of an object by braking, and the distance traversed by the vehicle as it comes to a stop. Throughout the years, AASHTO's model for SSD design has been based on establishing parameters describing the performance of a nominally critical driver (i.e., one that is below average in terms of performance), and critical vehicle performance in combination with pavement condition and initial speed. Human factors studies, knowledge of pavement friction capabilities, and engineering judgment went into the establishment of design parameters for the SSD model. Other elements of SSD design policy have included dimensions associated with the height of a driver's eye, and the height of an object requiring the vehicle to stop. These dimensions are necessary to establish design controls for vertical alignment sufficient to produce minimum or adequate SSD.

Evolution of SSD Model Parameters

Although AASHTO's fundamental operational model has remained unchanged over the years, the individual parameters used to calculate SSD and determine vertical curvature have been revised over the years. Table V-1 demonstrates the evolution of AASHTO Policy in terms of changes in design parameters for eye height, object height, perception/reaction time, pavement friction, and even initial or operating speed assumptions.

Consider, for example, the model as described in the 1940 Policy. The vehicle fleet at the time resulted in a 4.5-foot eye height as being representative of a reasonably "low" eye. Interestingly, AASHO assumptions were that perception/reaction time varied with speed, and that a dry pavement was appropriate for design. A further policy decision was that SSD would be based on the assumption that a nominally critical vehicle operated at the design speed of the road.

Over the years, as the U.S. vehicle fleet changed, driver eye heights continued to be lowered. AASHO responded by lowering the design parameter for eye height, to its eventual 3.5-ft dimension incorporated in the 1973 and 1984 Policies, and metric equivalent of 1070 mm in the 1994 Policy. AASHO also adjusted their model to design SSD for a wet pavement, but coupled that more conservative assumption with a less conservative one, namely, that driver speeds in wet conditions were lower than the design speed. This change itself was again addressed by AASHTO in later years (1973), as evidence surfaced that suggested driver speeds did not decrease on wet pavement to the extent assumed. With the 1973 Policy, AASHTO chose to establish a range of speed behavior, leaving it to the designer and ultimately to users of the Policy as to which speeds and SSD values should be used for design.

The object height used for design purposes for vertical curve criteria has also changed over the years. Beginning with a 4-inch object, this was revised to a 6-inch object, and then most recently to a 150-mm object with the Metric version of the Policy published in 1994.

Evolution of SSD Design Values

As design parameters have changed, design values for SSD have also changed. Figure V-1 depicts the evolution of SSD design values by design speed, tracing these values from the 1954 AASHTO Policy to the present day. Design values are shown in the English system of units. SSD design values based on NCHRP Report 400 are also included for comparison purposes.

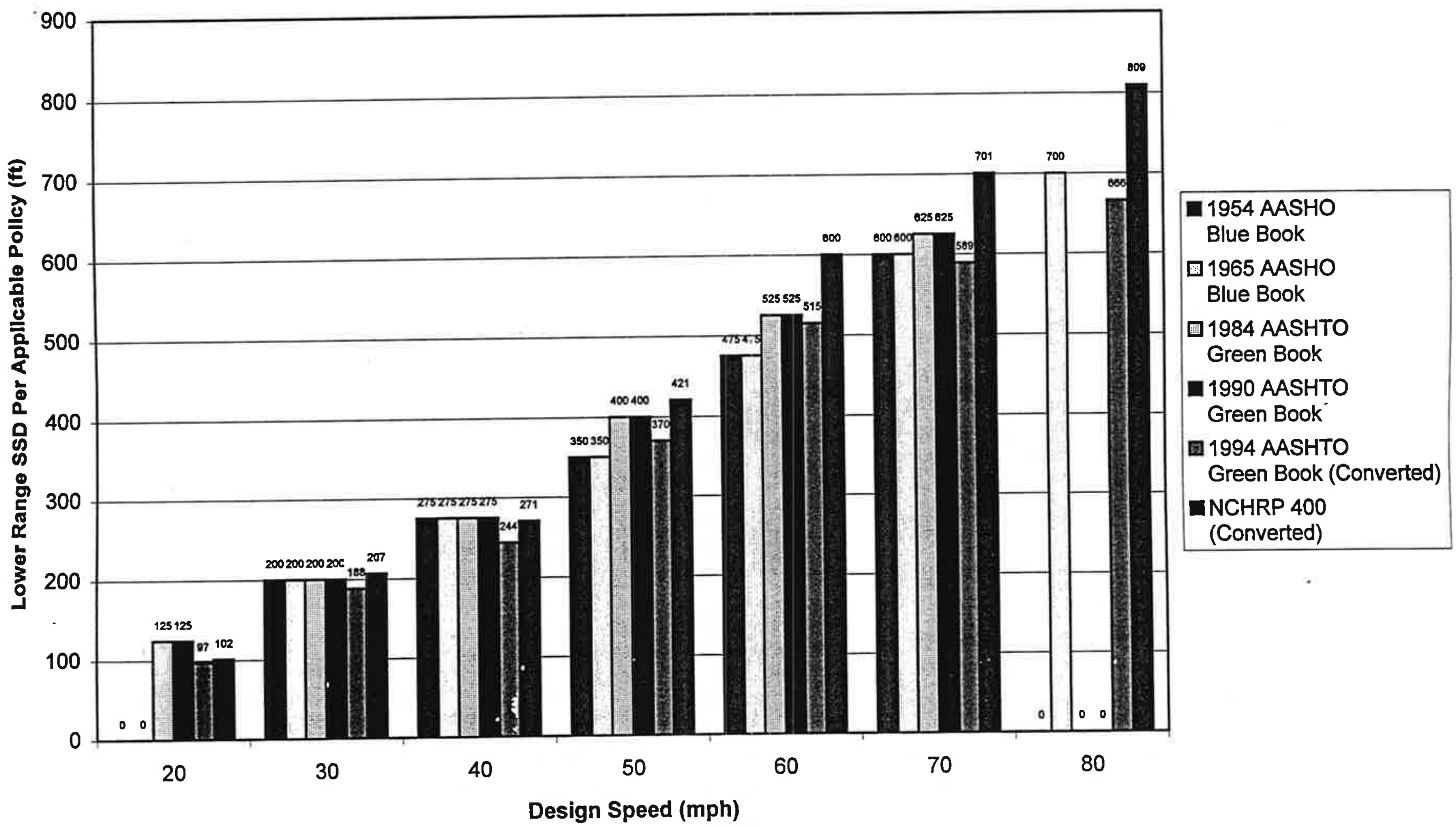
Figure V-1 shows the greatest change in SSD design values occurred in the 1984 Policy (which actually incorporated 1973 SSD Policy changes) described above. Desirable SSD increased significantly for higher design speeds. Accompanying these greater values was the establishment of a range of SSD with the 1984 and 1990 policies. Note, however, that even the lower range values for high design speeds resulted in greater SSD values than those published previously in the 1965 policy. For a 60 mph design, lower range SSD increased from 475 feet in 1965 to 525 feet in 1984. Whether by intent, or merely as a result of computing design values from parameter changes, AASHTO has increased SSD for a given speed over the past 40 plus years.

Table V-1
Evolution of AASHTO Stopping Sight Distance Policy

	DESIGN PARAMETERS					
YEAR	Eye Height (feet)	Object Height (Inches)	Perception/ Reaction Time (Seconds)	ASSUMED TIRE/PAVEMENT COEFFICIENT OF FRICTION	ASSUMED SPEED FOR DESIGN	EFFECTIVE CHANGE FROM PREVIOUS POLICY
1940	4.5	4	Variable -- 3.0 Sec. @ 30 mph to 2.0 Sec. @ 70 mph	DRY -- f Ranges from 0.50 @ 30 mph to 0.40 @ 70 mph	DESIGN SPPED	--
1954	4.5	4	2.5	WET – f Ranges from 0.36 @ 30 mph to 0.29 @ 70 mph	Lower Than Design Speed (28 mph @ 30 mph Design Speed; 59 mph @ 70 mph Design Speed)	No Net Change in Design Distance
1965	3.75	6	2.5	WET – f Ranges from 0.36 @ 30 mph to 0.27 @ 80 mph	Lower Than Design Speed (28 mph @ 30 mph Design Speed; 64 mph @ 80 mph Design Speed)	No Net Change in Design Distance
1973	3.75	6	2.5	WET -- f Ranges from 0.35 @ 30 mph to 0.27 @ 80 mph	Minimum Values – Same as 1965; Desirable Values – DESIGN SPEED	Increase in SSD of Up to 250 feet at 70 mph (Desirable)
1984 and 1990	3.50	6	2.5	WET -- f Ranges from 0.35 @ 30 mph to 0.27 @ 80 mph	Minimum Values – Same as 1965; Desirable Values – DESIGN SPEED	No Net Change from 1970
1994	3.50 (1070 mm)	6 (150 mm)	2.5	WET -- f Ranges from 0.40 @ 30 km/h to 0.28 @ 120 km/h	Minimum Values – Same as 1965; Desirable Values – DESIGN SPEED	No Net Change – Converted to Metric Units

Source: 1994 AASHTO Policy, Roadside Design Guide

Figure V-1
Stopping Sight Distance Design Values* - 1954 - Present



Perspective on SSD Design Parameters

The many changes over the years to SSD design parameters provide an interesting perspective on AASHTO's views of SSD. Background on the actual derivation of the original SSD model and its known design impacts provides insights that are relevant to this study.

Most design engineers believe that the basis for SSD or the AASHTO SSD model is fundamentally safety-driven. The model itself describes an intuitively clear, reasonable safety problem (collision avoidance with a small object). An axiom of roadway design has always been that a "safe" highway provides sight lines such that a driver should be able to brake to avoid a collision at all points along the highway.

It is evident, however, that no *direct* linkage to measures of crash avoidance has ever existed. Moreover, the original SSD model was actually never intended to reflect direct or quantitative measures of highway crashes. In a paper published in 1984, Neuman, et al (44) noted that, contrary to the conventional view of SSD, the originators of the SSD design procedure developed SSD design criteria based on a different concept of cost-effectiveness. In the 1940's, AASHO had no formal empirical evidence or research on incremental safety effects of varying SSD or lengths of vertical curves. The ability and data to formally measure and analyze the relative benefits of variable SSD did not exist. The policy writers also recognized that any given procedure for SSD design would have a significant effect on design and construction of vertical alignment. The longer the vertical curve the greater would be the construction cost.

The developers of the original policy thus rationalized a simple *operational* model – that a design vehicle driven by a design driver at the nominal design speed should be given the opportunity to safely brake to avoid collision with an object in the path of the vehicle. This simple operational model appeared fundamentally appropriate for any highway type, in any condition. (We note that the vast majority of roads over 40 years ago were two-lane in nature.)

With this rational, functional model established, it was then necessary to define the model's parameters so alignment design values could be computed. AASHO Policy makers selected an eye-height based on studies of the vehicle fleet at the time. The developers of the policy next looked at alternative object heights. The original policy developers noted that marginally shorter object heights (going to a 0-ft object at the limit) resulted in significant increases in required vertical curve lengths and their associated earthwork costs. They determined that an object height of 4 inches represented the optimal relationship between the amount of sight distance produced by a parabolic vertical curve, and the costs of constructing that vertical curve. The 4-inch object was thus selected as the basis for design, and K values for vertical curvature were computed accordingly.

AASHO's SSD design model was completely rational and appropriate for the extent of knowledge over 50 years ago. Indeed, recent research has reaffirmed the visibility of a driver-vehicle-object collision avoidance model as the basis for SSD design. It should be understood, however, that the original knowledge-base was primarily related to construction costs. Knowledge about human factors (e.g., the ability to perceive small objects at great distances, driver expectancy and its affect on behavior, etc.); and knowledge about the expected marginal differences in crash occurrence associated with different SSD values simply did not exist over 50 years ago.

What is interesting is that the AASHO policy writers believed it necessary to explain their selected SSD design parameters in safety-based terms. Language was placed in the Policy that referenced a 4-inch object as being of critical size to warrant collision avoidance. This suggests that safety was always a paramount concern, regardless of the actual rationale for the SSD design model.

[Addressing SSD parameters in direct safety terms may in retrospect appear unfortunate, as it has resulted in most design engineers lacking the understanding of the derivation of the AASHO/AASHTO SSD model. Interestingly, when the object height was changed by AASHTO from 4 to 6 inches in 1965, exactly the same language was used to describe the criticality of a 6-inch object.]

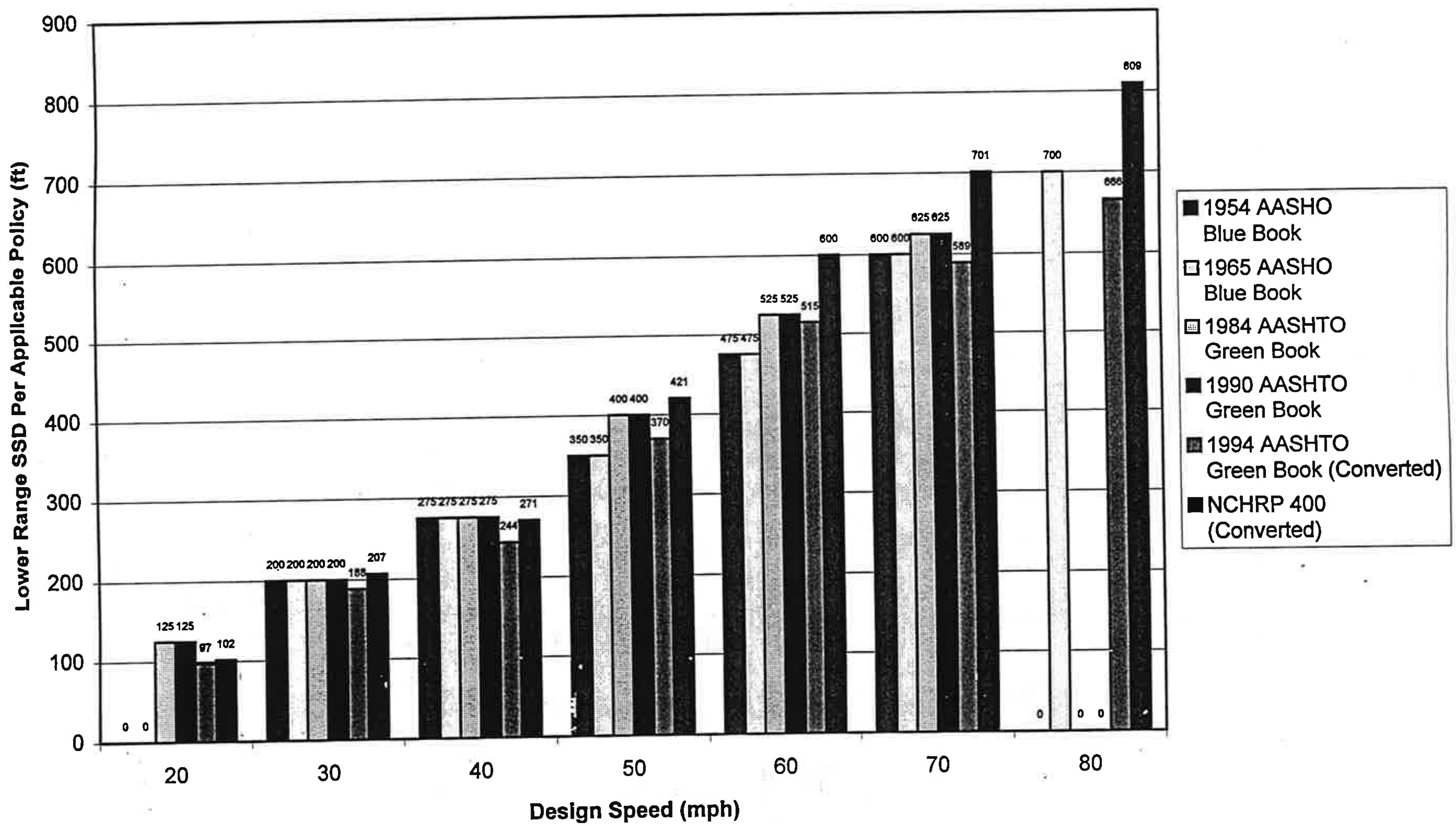
The history of changes to SSD parameters since 1940 provides further insights. By 1965, it became evident that the U.S. vehicle fleet had changed, and that eye heights were lowering. Rather than change just this dimension, however, the authors of the 1965 Policy were concerned that unilateral changing of eye height would greatly lengthen vertical curve dimensions, making construction more costly. There was no formal or even anecdotal information that suggested that lower eye heights were resulting in crashes or safety problems. Again, lacking much formal information on the relative safety merits of longer or shorter SSD values, AASHO again made judgments reflecting an understanding of the effects of SSD on construction of vertical curvature. Their solution to these issues was to change other parameters for object height and initial speed, compensating for the effect of lower eye heights on vertical curvature such that the net resultant effect on vertical curvature was minimized. In effect, AASHO recognized the need to "update" their SSD model parameters, yet also recognized that major lengthening of vertical curve or other design values would cost more without bringing, in their judgement, substantive or measurable safety benefits.

Since 1965, changes to SSD design values have similarly reflected intuitive concerns about the desirability or need for overly long vertical curves. The 1973 changes in the AASHTO models were based on a greater understanding of speed behavior in wet conditions. Despite evidence contradicting the assumption in the 1965 policy regarding driver speeds, it was the judgment of AASHTO in formulation of updated SSD values in 1973 that the older values based on the lower, presumably less conservative speed assumptions, could still be used. Thus was the creation of the range of SSD values, with lower end values based on the older speed assumption, and higher end values based on the assumption that driver speed was equal to design speed.

To summarize, as the SSD model parameters have changed, AASHTO has attempted to minimize the resultant impact on SSD and vertical curve design parameters. Regardless, there has been an increase in both SSD and K design values over the years, as illustrated in Figures V-1 and V-2.

It is clear that through the years, while retaining the original 1940 functional operational model, AASHO/AASHTO has attempted to update model parameters based on actual data, yet has resisted abandoning vertical curve lengths that were derived previously. It appears as if the judgment of AASHTO has been that vertical curves are long enough, and that whatever policy is established, cost-effectiveness in design is not served by significant lengthening of them. This judgement is supported by design engineers faced with reconstruction problems involving 'substandard' vertical alignment.

Figure V-2
Design Values for 'K' for Crest Vertical Curves – 1954 - Present



The above perspective is critical to a fresh evaluation of SSD, and SSD-related design requirements in the LVLRoad environment. A review of the history of SSD and its relationship to objective measures of safety shows the following:

1. The AASHTO SSD model is a simple, rational operational model. In essence, it has remained unchanged in almost 60 years. The model has always been considered appropriate for all highway types under all conditions. The volume or type of traffic on the road has never been a factor in design criteria for SSD, nor has the functional class of highway.
2. Design for SSD could not be originally linked to direct or quantitative measures of safety. The basis for SSD and vertical curve parameters was an analysis of construction cost-effectiveness.
3. Changes to parameters used to compute SSD values have been made over the years for a number of reasons. With each change, AASHTO has attempted to minimize the design-related impacts to vertical alignment criteria. Nonetheless, through the 1994 Policy, there has been a gradual lengthening of vertical curve design requirements.

Review of Knowledge on SSD Safety and Operations

A review of the literature over the past 30 years is revealing. Many studies have addressed or attempted to address the effect of SSD on highway crashes. As part of this study, the literature on SSD related to safety and operations was reviewed. This effort was helped by two recent studies—NCHRP Report 374 *Effect of Highway Standards on Safety* (29), and recently published NCHRP 400, *Stopping Sight Distance* (17).

Although many studies have looked at SSD or vertical alignment, few have successfully demonstrated a relationship between deficiencies in SSD and crashes. The authors of NCHRP Report 270 (47) in a matched pairs experiment attributed a 52 percent increase in accidents at "substandard" vertical curvature (relative to 1965 AASHO Policy values) compared to crashes on control sections along two lane roads. The SSD values in the substandard sites were less than 90 m for 90 km/h (55 mph) roads. The authors of the study noted, however, that the sample size was small, and that the results were not reliable. More recently, an HSIS study by Paniati and Council (50) showed that accident frequency at crest vertical curves increases greatly just at the crest of curves at which the grade differential ('A') is greater than 6 percent. They found that the recorded location of crashes tended to cluster within a very short distance of the PI of the crest – within 0.02 miles (about 30 m).

In NCHRP Report 374, an effort intended to identify the state of the art in highway safety-related to design geometry, the authors concluded that no meaningful relationships had been established between SSD and safety. They cited a SSD model developed by Neuman, et al (45) as the best available model describing SSD safety effects. Note, however, that this model as proposed at the time was hypothetical in nature, uncalibrated or confirmed by actual research. An earlier major study by TRB on 3R design standards, published as Special Report 214 (14), concluded at that time that the Neuman SSD model was the most useful tool for evaluating SSD relationships.

The Neuman model describes SSD safety in terms of not only the limiting geometry of, say, a vertical curve, but also the probability of an event occurring within the limits of sight restricted geometry. A critical event could be the presence of an object, vehicle in the road, etc. The model considers the location of a SSD "deficiency" relative to other roadway features to also be important. Poor SSD at or near, say an intersection, is considered to be inherently more problematic than poor SSD within a tangent.

Neuman's SSD model was used in TRB Special Report 214 to demonstrate the potential safety and cost-effectiveness of correcting locations with poor or nominally deficient SSD. Their analyses suggested that increasing sight distance at crests should be considered where traffic volumes are greater than 1500 vpd, the effective design speed of the crest is more than 20 mph (or about 30 km/h) below the 85th percentile operating speed, and there is a hazard or other risk producing feature such as an intersection, bridge, etc. near the crest of the curve.

More recently, studies performed for NCHRP Report 400 noted that very few crashes on roads with limited SSD are actually attributable to poor SSD. Tests of the relative frequency of objects struck showed that the AASHTO model does not relate well to actual crash occurrence on roads. An almost negligible number of crashes actually involved small objects, and these tended not to be severe crashes. Most of the objects struck were deer. Other objects included vehicles turning into the roadway or stopped.

Review of accident reports to determine probable causation was performed in NCHRP Report 400. Only 4 percent of accident narratives that were reviewed referenced limited SSD as a possible contributing factor. Most of the accidents with limited SSD as a contributing factor occurred on vertical curves with K values of 100 or less.

Additional studies, reported in NCHRP Report 400 noted that, for higher speed facilities, limited SSD did not appear to be a factor in accident causation until the available SSD was less than 360 feet. For 55 mph roads, this is the equivalent of a 13 mph (20 km/h) "deficiency" in SSD. Fambro's interpretation of the effect of SSD on crashes is illustrated by Figure V-3. Only under relatively severe conditions does limited SSD begin to have an actual effect on crashes.

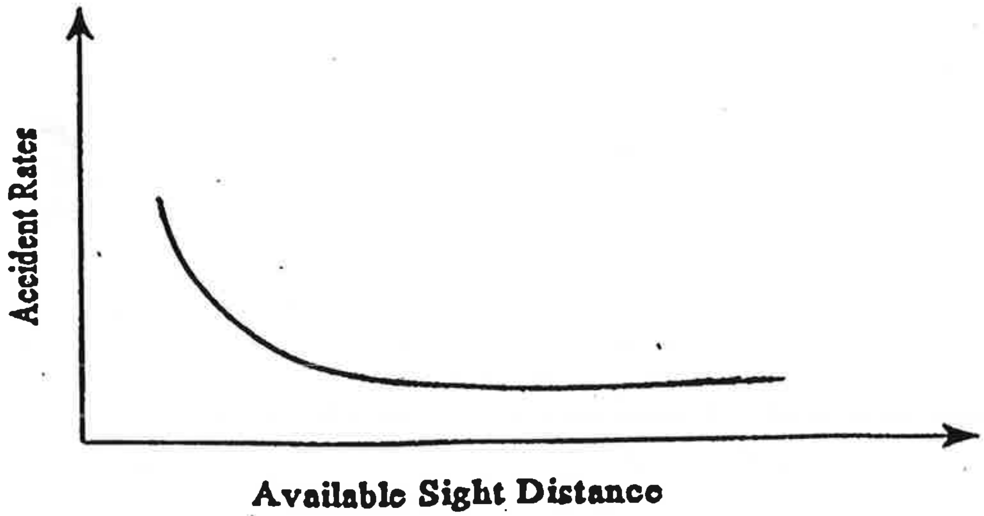
The authors of NCHRP Report 400 concluded the following (their emphasis):

> "The findings were that the accident rates on rural two-lane highways with limited SSD are similar to the accident rates on all two-lane rural highways. **Thus, LSSD does not appear to cause a safety problem."** Continuing on, **"...moderate reductions in minimum stopping sight distance do not appear to cause a safety problem."**(17)

Other work by Glennon and Neuman (45), discussed further in NCHRP Report 400, provides insights on safety related to SSD. If one wishes to characterize the actual risk of a crash associated with the highway's poor sight distance, a measure of that risk would be the length over which the SSD actually is at a minimum. The "profile" of SSD produced by parabolic vertical curvature results in relatively short highway dimensions over which an actual limiting condition may exist (see discussion below on the geometry of SSD). Note that this finding is entirely consistent with that of Paniati and Council regarding clustering of SSD related accidents.

Finally, other human factors research reported in NCHRP Report 400 confirms that the ability of drivers to discern a small object is limited once the object is greater than 130 m away. This suggests that nominal SSD values greater than 130 m, at least based on a small object definition, are essentially meaningless.

FIGURE V-3
Conceptual Relationship Between Available Sight Distance and Safety at Crest Vertical Curves



Source: NCHRP Report 400, Determination of Stopping Sight Distance

The reader should note that all the safety research cited above involved studies of roads with traffic volumes much greater than in the LVLRoad environment. Even under such cases, the literature appears to clearly demonstrate the following:

1. Within the normal range of design, measurable highway safety is insensitive to marginal differences in SSD. Previous studies show that only with a deficiency in SSD of at least 20 km/h, and perhaps as much as 30 km/h, can one expect to observe differences in actual safety performance attributable to SSD.
2. Studies that have uncovered safety sensitivities have involved sections of highways with much higher traffic volumes than occur in the LVLRoad environment.
3. Values for SSD greater than 130 m may have little practical meaning, given that humans have difficulty perceiving small objects at distances greater than 130 m.
4. Very small objects (as represented by AASHTO's 6 inch / 150 mm design dimension) do not represent significant safety risks to drivers. Greater dimensions such as those associated with a vehicle are better indicators of SSD safety risk.
5. Safety sensitivities may occur only in relatively short segments of highway. This reflects the geometry of highway design and the actual production of SSD along an alignment.

Review of Design Practice by Others

Current practice within North American design agencies were reviewed to establish the extent of consensus and uncover unique design practices for stopping sight distance. Chapter II contained an overall summary of this survey.

With respect to the issue of sight distance in design, the majority of agencies designing local or low volume roads reference current AASHTO design policy either directly or by implication. Design values are based on the operational "model" for SSD for lower volume and local roads the same as for higher classified facilities. However, as discussed below, there are some notable exceptions to the use of AASHTO Policy values.

Exceptions to AASHTO Design Model in North America

There are two notable exceptions to the use of AASHTO-based SSD design values. The US Forest Service (USFS) developed a range of design practice for SSD for a range of roadway types. (63) SSD for "higher class" two-lane roads (referred to as Traffic Service types A and B) is designed with a 2.5-second perception/reaction time, and 0.5 foot object height. Lower class roads (Traffic Service Levels C and D) are designed with a 2.0 second perception/reaction time and 0.5 foot object height. The lower perception/reaction time is attributed to the simpler operation on the lower class roads. Stopping distances are computed for two vehicle types, representing deceleration and braking for both a truck and passenger car. The USFS thus provides criteria for vertical curve design for both passenger car and truck usage, as well as for different classes of roads. (Interestingly, in the derivation of truck SSD, a 3.5-foot eye height is used.) Design of one-lane roads is similarly governed, with the use of a 4.5-foot object height representing the top of a passenger car, and twice the stopping distance.

Tables V-2 and V-3 show SSD values for USFS design procedures, and resultant vertical curve design criteria. Compared to current AASHTO Policy, USFS design for Traffic Service levels A and B results in an 18 percent reduction in vertical curve length for a 50 mph (80 km/h) design condition. Another interpretation of this is that USFS design criteria for an 80 km/h design speed are roughly equivalent to a 70 km/h AASHTO design (a 10 km/h differential). See Table V-4.

The geometric design manual published by the Roads and Transportation Association of Canada (RTAC) takes a different approach for local roads designed for low traffic volumes. RTAC considers the critical object height to be 150 mm. The rationale is that for lower class roads, surface debris, washouts or other surface anomalies are more likely to be the controlling condition. Interestingly, RTAC uses vehicle taillights (with a 380-mm dimension) for the design control of higher class roads. This compares with values studied in NCHRP Report 400.

USFS design criteria for stopping sight distance are lower than AASHTO. Considering the much greater object height, the design of vertical curvature for USFS roads would be significantly different from AASHTO. RTAC design values for SSD are greater for higher speeds.

International Practice in Design of SSD

NCHRP Report 400 reported on an international survey of design practice for highways. While the results reflect all roadway types, they are revealing with respect to where AASHTO criteria falls. Figure V-4, derived from values reported in NCHRP 400, shows that most countries with geometric criteria for SSD use design values from 5 to 20 percent less than those specified by AASHTO for speeds above 70 km/h.

Summary of Practice by Others

There is precedent in the design criteria employed by others for alternative design parameters and values for SSD. USFS criteria, developed specifically for application to lower volume roads in difficult terrain, are less severe (i.e., produce lesser SSD values and shorter vertical curves) than those of AASHTO. Compared to criteria internationally, AASHTO design values for higher class roads are conservative.

Geometric Relationships Involving SSD

A review of the relationships between SSD design values and roadway geometry provides further insights to consideration of SSD issues for LVLRoads. SSD operates in all three dimensions, influencing and being influenced by vertical alignment and horizontal alignment.

Vertical Alignment

United States and most international design practice calls for the use of parabolic vertical curvature between tangent grades. The curvature acts to limit the sight line of a driver on crest vertical curves. Vertical curvature on sag curves can also affect sight lines on unlit roads at night, given the limiting characteristics of headlight beams.

Although design values for SSD are firmly established in current Policy, and the need for SSD well understood, actual design of SSD is performed indirectly. SSD requirements for a given design speed are translated into minimum values for length of vertical curve, as a function of the two intersecting grades, as shown below:

$$K = L / A$$

where:

K = Length of vertical curve in meters per 1 percent change in grade
L = Length of vertical curve in meters
A = Algebraic difference in grades in percent

For parabolic vertical curves: ($S < L$)

$$L = AS^2/C$$

where:

S = Sight distance in meters
C = $200\,[(H_e)^{½} + (H_o)^{½}]^2$

and:

H_e = Height of driver's eye in meters
H_o = Height of object in road in meters

Table V-2
Criteria for Stopping Sight Distance (Two-Lane Roads) for Trucks Based on U.S. Forest Service Design Criteria[1]

Design Speed (km/h)	Traffic Service Levels A & B (P/R Time = 2.5 sec) SSD (m)	Traffic Service Levels C & D (P/R Time = 2.0 sec) SSD (m)
20	17.6	14.8
30	29.2	25.0
40	42.6	37.0
50	57.8	50.8
60	74.9	66.5
70	93.7	84.0
80	114.5	103.0

[1] Converted to metric from USFS Criteria in Road Reconstruction Handbook, 9/87.

Table V-3
Criteria for Crest Vertical Curve Design (Two-Lane Roads) based on U.S. Forest Service Design Criteria[1]

Design Speed (km/h)	Algebraic Difference in Grades	Traffic Service Levels A & B (P/R Time = 2.5 sec) L_{vc} (m)	Traffic Service Levels C & D (P/R Time = 2.0 sec) L_{vc} (m)
	5	15.2	15.2
20	10	15.2	15.2
	15	15.2	15.2
	5	17.1	17.1
30	10	17.9	17.1
	15	31.6	23.1
	5	22.9	22.9
40	10	44.8	33.8
	15	67.2	50.7
	5	34.6	28.7
50	10	82.5	63.8
	15	123.7	95.6
	5	68.7	51.9
60	10	138.3	109.1
	15	207.5	163.6
	5	108.5	87.0
70	10	216.9	174.0
	15	325.4	261.0
	5	161.7	131.7
80	10	323:4	263.3
	15	485.1	395.0

[1] Converted to metric from USFS Criteria in Road Reconstruction Handbook, 9/87.

Table V-4
Comparison of USFS and 1994 AASHTO Stopping Sight Distance Criteria

Speed (km/h)	1994 AASHTO Policy[1]	USFS Criteria[2]	
	SSD (m)	SSD (m)	AASHTO Equivalent Design Speed (km/h)
30	29.6	29.2	30
40	44.4	42.6	39
50	62.8	57.8	47
60	84.6	74.9	56
70	110.8	93.7	63
80	139.4	114.5	71

[1] Upper range SSD
[2] Traffic Service Levels A & B, Truck Deceleration

Figure V-4
Design or Operating Speed

Country	t_{pr} sec	20	30	40	50	60	70	80	90	100	110	120	130	140
		Design or Operating Speed (km/h) / Stopping Sight Distance (m)												
1994 AASHTO Policy	2.5	20	30	44	63	85	111	139	169	205	246	286		
COUNTRY														
Australia														
Normal Design	2.5	--	--	--	--	--	--	115	140	170	210	250	300	--
Normal Design	2.0	--	--	--	45	65	85	105	130	--	--	--	--	--
Restricted Design	1.5	--	--	--	40	55	55	--	--	--	--	--	--	--
Austria	2.0	--	--	35	50	70	90	120	--	185	--	275	--	380
Canada	2.5	--	--	45	65	85	110	140	170	200	220	240	--	--
France	2.0	15	25	35	50	65	85	105	130	160	--	--	--	--
Germany	2.0	--	--	--	--	65	85	110	140	170	210	255	--	--
Great Britain	2.0	--	--	--	70	90	120	--	--	215	--	295	--	--
Greece	2.0	--	--	--	--	65	85	110	140	170	205	245	--	--
South Africa	2.5	--	--	50	65	80	95	115	135	155	180	210	--	--
Sweden	2.0	--	35	--	70	--	165	--	--	--	195	--	--	--
Switzerland	2.0	--	--	35	50	70	95	120	150	195	230	280	--	--

Source: NCHRP Report 400

Designers typically reference design curves or tables giving the minimum length of vertical curve, or compute the minimum length given K.

What is not well understood by many, and is central to addressing issues of SSD, is the actual relationship between the resulting alignment and SSD, however it is defined. Contrary to the understanding of many, SSD produced by a given vertical curve is not constant throughout the curve, but varies. The minimum SSD for which the curve is intended to provide occurs over a distance much shorter than the length of curve itself. The values of the grades and their relative relationship influences the location of actual SSD. Finally, variable SSD exists for the two directions of travel.

The above points are illustrated through the use of *stopping sight distance profiles.* As shown in Figure V-5, a stopping sight distance profile can be produced by measuring the actual sight distance at each point along an alignment, and plotting it by stations. The results of this exercise provide perspective. For any given vertical curve, the extent of the highway over which the highway provides limiting or minimum SSD is much less than the total length of the vertical curve. Once past the crest of the curve, SSD increases very quickly, so that a driver on the departure portion of the vertical curve typically does not encounter a sight restriction.

Another significant characteristic of sight distance profiles from vertical alignment is the relationship between the length of highway with restricted sight distance and the amount of the restriction. The more restrictive the alignment (i.e., a very short vertical curve with large value of A), the shorter the length of the restriction. Thus, a "high risk" vertical curve, one producing significantly less than normal or required SSD per Policy, is mitigated by the fact that the length of highway over which the restriction exists is very short.

Figure V-6 demonstrates the relationship between length of sight-restricted alignment and values for A, for a range of design speeds. For a nominal 80 km/h alignment, even a moderate restriction (a 25 km/h deficiency, or in other words, an equivalent 55-km/h vertical curve) for a value of 8 for A (say, two 4 percent grades) produces a length of alignment at risk of only about 80 m.

Horizontal Alignment

SSD profiles can also be produced for horizontal curves. In limiting cases, the sight line from a driver to the road ahead may be obstructed by objects such as trees, embankment, buildings, etc. in the inside of the curve. A horizontal SSD profile is illustrated in Figure V-7.

Again, depending on the geometry of the curve (combination of length and radius), and the location of the object(s) limiting sight lines, the length of highway over which the sight distance is limited may be significantly less than the total length. Moreover, the sharper the horizontal curve, the shorter its length tends to be. Even with relatively small offsets to obstructions to the inside of the curve, drivers are confronted with a sight restriction over a very short segment of highway.

FIGURE V-5
Stopping Sight Distance Profiles for Vertical Curves

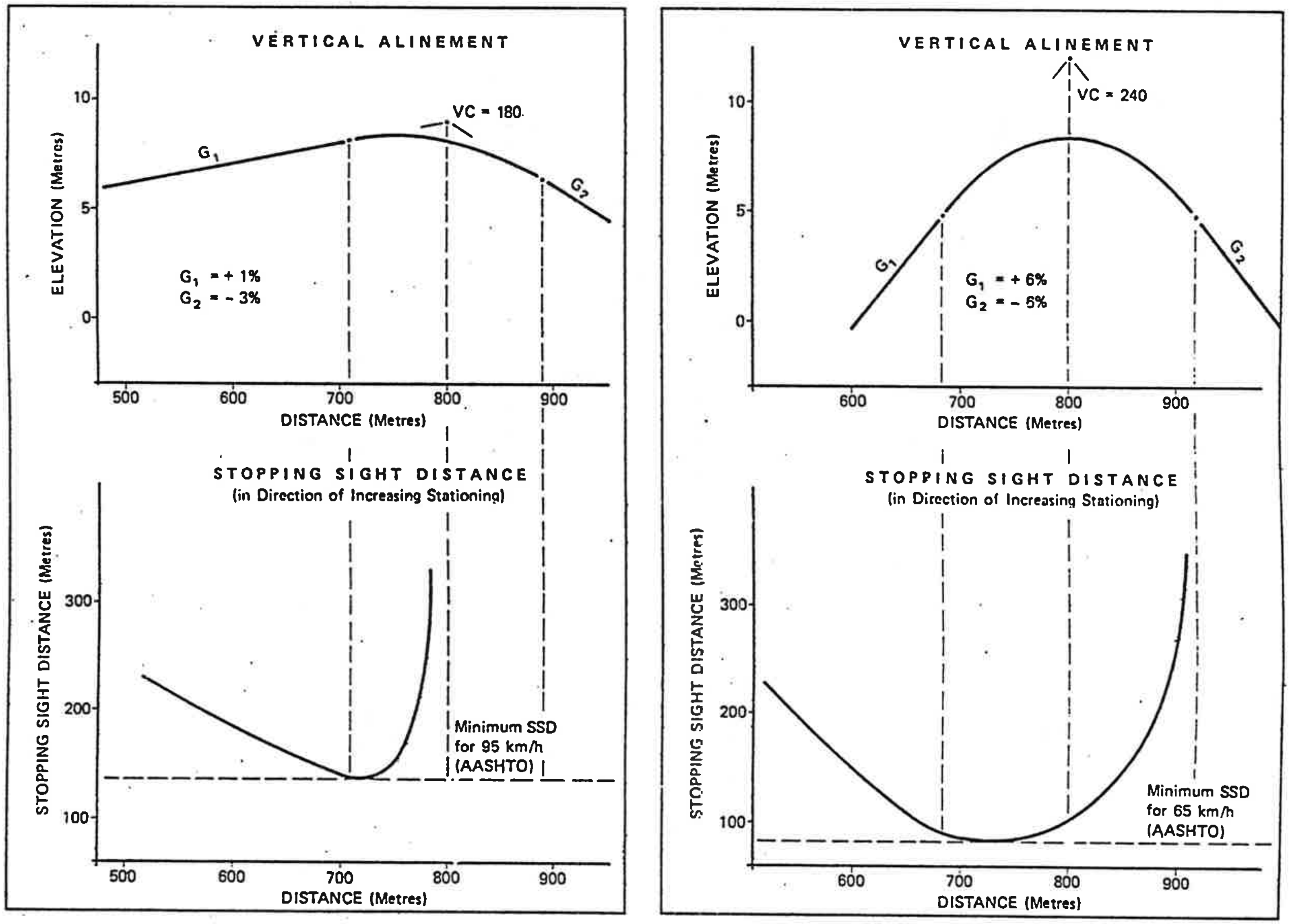


Figure V-6
Relationships Among Grades, Severity, and Length of SSD Restrictions on Vertical Curves

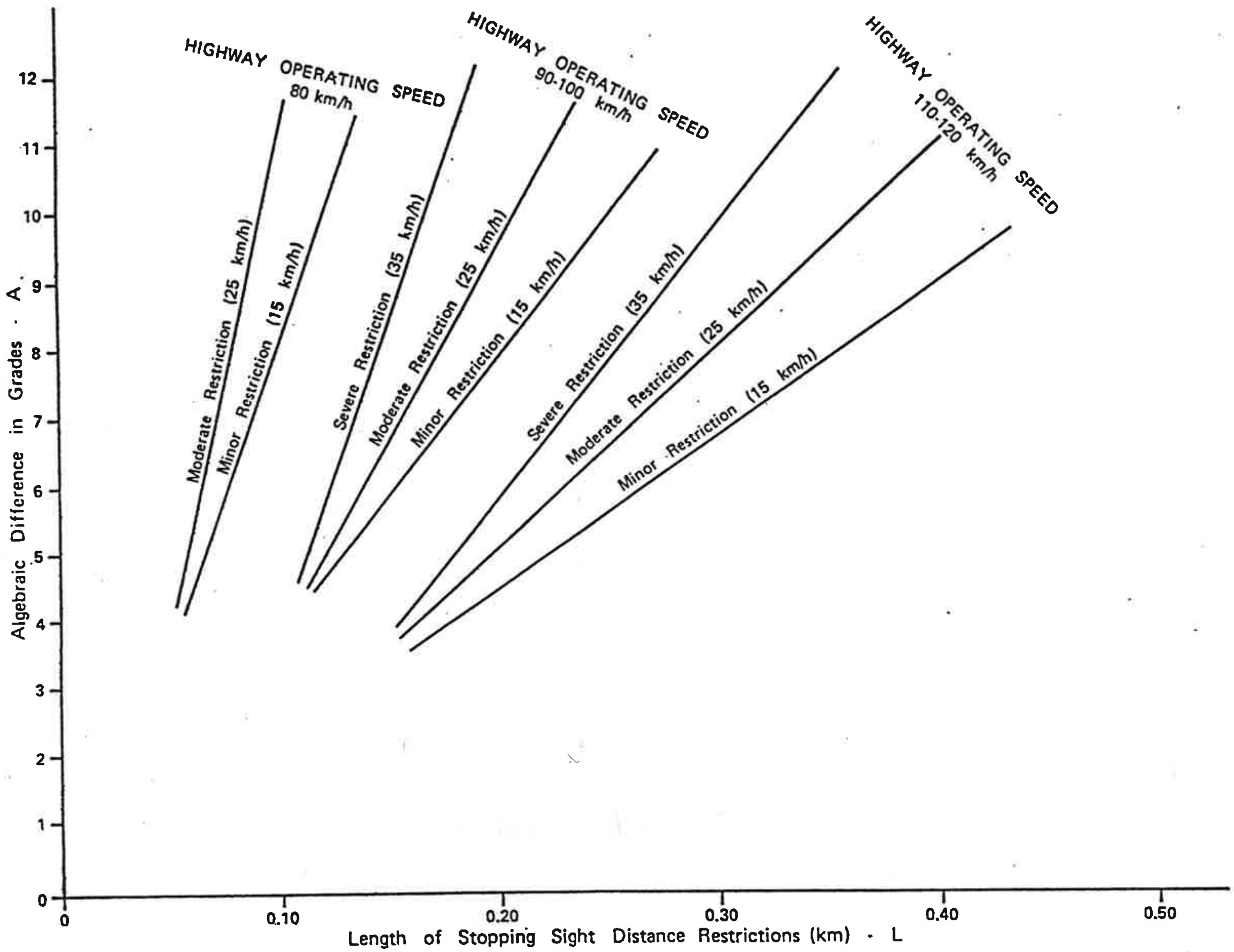
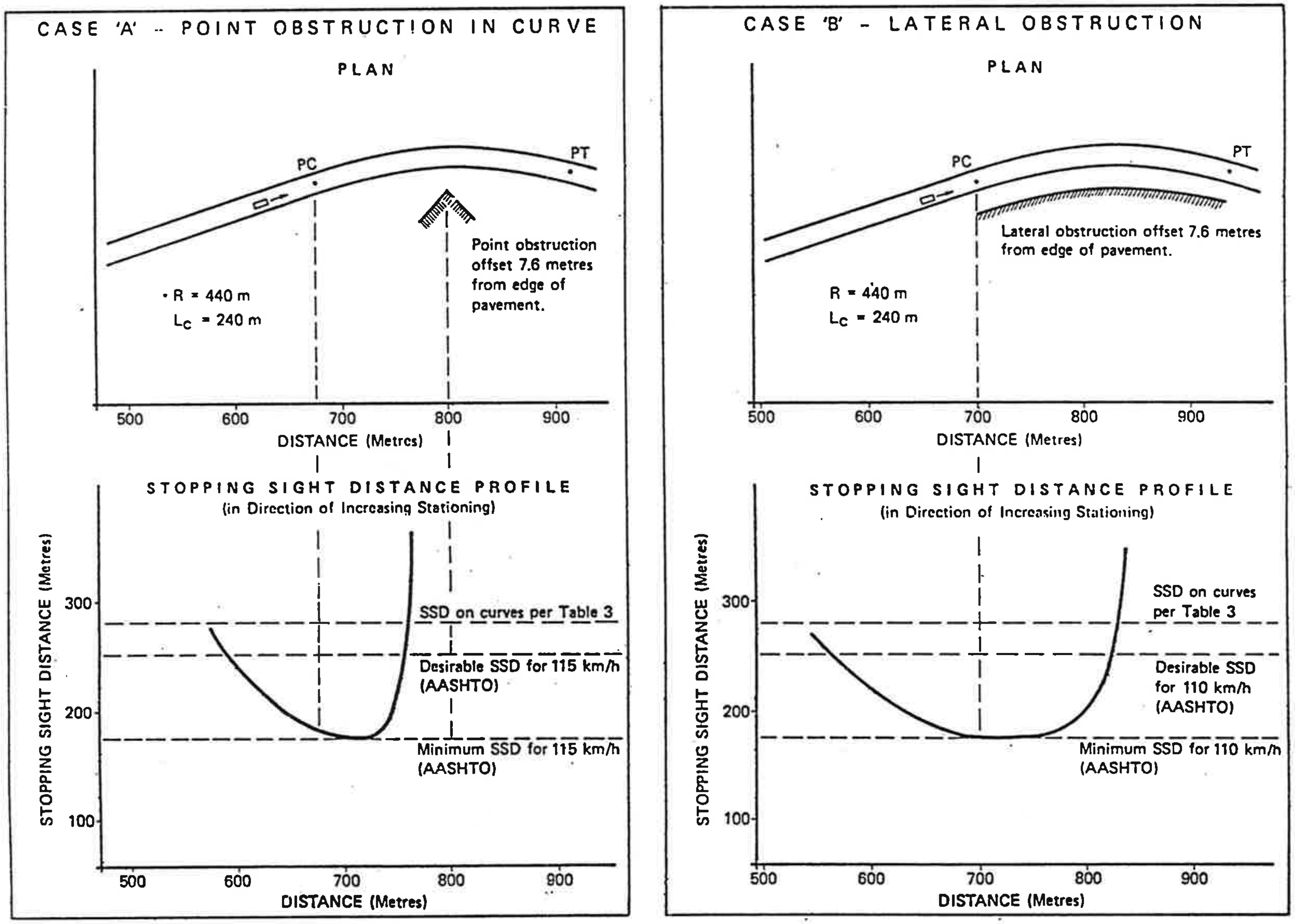


Figure V-7
Stopping Sight Distance Profiles for Horizontal Curves



Summary of SSD Geometric Relationships

The geometry of highway design is such that the actual length of a sight restriction is relatively short. Moreover, the greater the nominal deficiency, the shorter the length of road over which that deficiency exists.

Statement of the Problem—Stopping Sight Distance Design for Low Volume Local Roads

As a critical design element that influences all three dimensions of the highway, SSD is of great importance to those entrusted with maintaining low volume local roads. A survey of county engineers showed that many were concerned about limited right-of-way and avoidance of expensive relocations in reconstruction of local roads. Longer vertical curves and/or horizontal offsets for sight lines create potential problems, in that they both increase construction costs and right-of-way requirements.

Many engineers noted they perform little new construction. Their focus is maintenance and reconstruction as necessary on their existing system. Reconstruction needs are primarily associated with paving unpaved roads, replacing pavement, and reconstruction of culverts and drainage structures. Local roads engineers are generally cognizant of the cost-effectiveness of design policies and standards. Most recognize the need to maintain minimum standards. Also, many engineers expressed confidence in their knowledge of existing local safety problems. Such knowledge in some cases is through periodic review of crash records. In many others, it is based on anecdotal or general knowledge of troublesome locations, complaints from users, recurring maintenance problems, etc.

The vast majority of mileage of lower volume local roads was built many years ago. Some were constructed over 50 years ago. Some were designed to a prevailing standard at the time; many low volume local roads were not designed as much as constructed over the terrain. Recalling Figure V-1, a road designed and built to the prevailing design values for SSD and vertical curvature per the 1954 or 1965 AASHO Policy will not meet current 1994 criteria. The differences will be marginal, yet such locations are nominally "substandard."

AASHTO criteria for SSD design are among the more conservative for all highway types. The AASHTO operational model does not replicate well the unique conditions that occur on very low volume roads. In short, AASHTO criteria appear particularly unsuited as the formal basis for design for LVLRoads. Alternative procedures employed by other agencies, most notably the US Forest Service, specify SSD values and design criteria for vertical alignment that are less restrictive than AASHTO.

Research on the safety effects of SSD suggests that incremental or marginal differences in available SSD do not translate to safety problems for roads with substantially higher traffic volumes than those on the LVLRoad system. Sight restrictions occur over relatively short lengths of highway. Even for highly restrictive alignment, driver risk is not as great as is perhaps believed by many designers.

In short, there is ample evidence that acceptance of lesser design values as the basis for SSD design on LVLRoads would be reasonable. The question thus becomes one of how to arrive at and assess lesser SSD values, and then how to incorporate them within design policy for LVLRoads.

Risk Assessment of SSD

This section of Chapter V develops rationale and discussion to support alternative design approaches for SSD for LVLRoads. The approach taken in the research is referred to as risk assessment. The following is included:

- Investigation of NCHRP 400 SSD model parameters
- Safety model sensitivity analysis of alternative SSD values
- Summary of construction cost sensitivities for vertical curves
- Discussion of maneuver sight distance as an alternative to SSD

NCHRP Report 400 SSD Model

AASHTO is currently evaluating SSD design procedures and values resulting from NCHRP Report 400 by Fambro, et al. The recommended model for SSD in NCHRP Report 400 is similar to the existing AASHTO model in its rationale and formulation. SSD requirements are considered to be based on a vehicle encountering an object in the road. The "design event" is considered to be perception/reaction and deceleration to a stop. SSD design is considered constant across all functional classification and traffic volume ranges.

The equation is:

$$\mathrm{SSD} = (0.278)(\mathrm{V})(t) + 0.039\ \mathrm{V}^2/a$$

where:

SSD = stopping sight distance, m;
V = initial speed, km/h;
t = perception-brake reaction time, sec; and
a = deceleration, m/sec^2.

Perhaps the most significant aspect of the new, proposed model is the use of an object height representative of most vehicle headlights and taillights. This change from the 150-mm object reflects consideration of more real-world events and serious consequences for which SSD is presumably intended to address. The NCHRP Report 400 model proposed by Fambro, et al. appears destined to be the basis for future AASHTO Policy.

Another significant aspect of the new model is its derivation of deceleration and braking distances. As noted in Table V-5, Fambro's model formulation recommends the use of 95th percentile values for perception and brake reaction time, and 10th percentile values for driver deceleration.

The authors of NCHRP 400 recommended the design values for the various elements of their operational model based on their assessment of human factors and other operational research. Table V-5 summarizes the recommended SSD design values. It is notable that the recommended values for SSD are within the range defined by the current 1994 upper and lower SSD ranges. Adoption of these values by AASHTO would thus appear not to represent a major or radical revision to SSD policy for US highways. (With changing object heigh parameters, however, a reduction in vertical curve lengths would occur.)

It is not the intent of this study to question or revisit the NCHRP Report 400 recommendations. However, the formulation of the model, combined with an understanding of SSD in the LVLRoad environment, provides a means of testing the relative risk of alternative SSD values for LVLRoads.

Consider that the NCHRP Report 400 model results in a net probability of 0.005 (0.5 percent) that, for a given critical encounter with a vehicle, the driver will not react and be able to decelerate within the nominal design distance. (Note that this calculation is not a direct measure of crash occurrence. Other factors contributing to a crash event include the relative probability of a "design vehicle" stopped in the path of the vehicle, and the probability that this event occurs over a segment of road with limited sight lines.)

Crash risk and relative safety are directly related to traffic volume. It is reasonable, indeed, enlightening, to include volume considerations, at least in the context of very low volume roads, in evaluating SSD requirements. SSD design values derived using the NCHRP Report 400 model can be computed for different combinations of perception/reaction and deceleration associated with different percentiles of the driving population. For example, a 90th percentile value for perception-reaction time, and 15th percentile value for deceleration, results in SSD values that are about 10 percent lower than the NCHRP 400 recommended values. (Recall that the 90th percentile value of 2.0 seconds is consistent with that used by the USFS for design of lower class roads.) The "risk" associated with these values is 0.1 x 0.15, or 0.015 (1.5%). This risk is three times that of the NCHRP Report 400 model. Note, however, that if this risk is viewed on a volume-adjusted basis, taking into account traffic volume, an interesting picture emerges.

Consider Table V-6. The relative risk of the NCHRP Report 400 model for a given location is two driver/vehicle "systems" per day not meeting the criteria for an ADT of 400. For an ADT of 1000, the nominal risk is five; for ADT of 5000, the risk is 25, etc. Also shown in Table V-6 is the nominal risk of other possible SSD design approaches, including that cited above, as well as the use of a 90th percentile P/R time and 50th percentile deceleration. This latter combination is of particular interest, as it closely replicates SSD values from the 1965 AASHO Policy.

Table V-6 illustrates that *acceptance of alternative, less restrictive parameters for LVLRoads* within the framework of the NCHRP Report 400 model *produces an equivalent risk* to that associated with the model's values on the higher volume roads for which it is intended to apply.

Table V-6 also demonstrates an advantage of defining less restrictive criteria for the LVLRoad system. Acceptable risk levels can be quantified precisely because the traffic volumes are capped on the upper end. LVLRoads by definition do not carry traffic greater than 400 vpd. Selecting a less restrictive set of human factors for SSD computations is no different than, for example, relating roadway or shoulder width design criteria to traffic volume levels.

Other Factors Affecting Risk

The above analysis does not take into account other factors associated with the true safety risk of an alternative design approach for SSD. These factors include the actual probability of the SSD design event occurring, vehicle type distributions unique to LVLRoads, and the nature of drivers using LVLRoads.

Table V-5
Sensitivity of SSD to NCHRP Report 400 Model Input Value

Initial Speed (km/h)	1994 AASHTO SSD (m)	TTI SSD Model[1] (m)	90th Percentile Perception-Brake Reaction Time (2.0 sec) Deceleration (m/s^2)				95th Percentile Perception-Brake Reaction Time (2.5 sec) Deceleration (m/s^2)			
			50th Percentile (4.1)	15th Percentile (3.5)	10th Percentile (3.4)	5th Percentile (3.1)	50th Percentile (4.1)	15th Percentile (3.5)	10th Percentile (3.4)	5th Percentile (3.1)
30	30-30	31.0	25.2	26.7	27.0	28.3	29.4	30.9	31.2	32.2
40	44-44	45.9	37.5	40.1	40.6	42.4	43.0	45.6	46.2	47.9
50	57-63	63.1	51.6	55.7	56.5	59.3	58.5	62.6	63.4	66.2
60	74-85	82.5	67.6	73.5	74.7	78.7	75.9	81.8	83.0	87.0
70	94-111	104.2	85.5	93.5	95.1	100.6	95.3	103.3	104.9	110.3
80	113-139	128.2	105.4	115.8	117.9	125.0	116.5	126.9	129.0	136.1
90	131-169	154.4	127.1	140.3	143.0	151.9	139.6	152.8	155.5	164.5
100	157-205	182.9	150.7	167.0	170.3	181.4	164.6	180.9	184.2	195.3
110	180-246	213.7	176.3	196.0	200.0	213.4	191.5	211.3	215.2	228.7
120	203-286	246.7	203.7	227.2	231.9	247.9	220.4	243.9	248.6	264.6

[1] NCHRP Report 400 SSD model utilizes 95th percentile perception-brake reaction time and 90th percentile deceleration rate

Table V-6

Relative Risk of Alternative Design Approaches for SSD

Average Daily Traffic (vpd)	Relative Risk per Day of Alternative SSD Model Formulations		
	95%-tile Perception/Reaction Time; 10%-tile Deceleration P(Failure) = (.05)(.10) = .005	90%-tile Perception/Reaction Time; 15%-tile Deceleration P(Failure) = (0.10)(.15) = .015	90%-tile Perception/Reaction Time; 50%- tile Deceleration P(Failure) = (0.10)(.50) = .05
100	0.5	1.5	5.0
250	1.2	3.7	12.5
400	2.0	6.0	20.0
1000	5.0	15.0	50.0
5000	25.0	75.0	250.0

Source: NCHRP Report 400

The design event, a vehicle encountering another vehicle, is strongly sensitive to traffic volume levels. In very low volume conditions, the likelihood of one vehicle encountering another is small. Insights on these probabilities are offered by Table V-7, adapted from earlier work by Glennon in NCHRP Report 214. (19) The estimated number of head-on meetings per mile per day on a two-lane road is non-linear with respect to traffic—increasing exponentially from very few at volumes less than 100, to 100 at ADT of 400, to 15,000 at volumes in the 5000 range. In effect, adoption of the vehicle headlight criterion for very low volume roads would be no different in terms of actual safety performance than the current use of a small object, which has been shown not to relate well to safety. This suggests that the relative risk of lower SSD values cited above in Table V-7 is *overstated.*

Table V-7
Risk of Head-on Meetings on Low Volume Roads

ADT	Estimated Number of Head-on Meetings Per Mile Per Day[1]	
	Travel Speed = 30 mph	Travel Speed = 45 mph
50	2.3	1.5
100	9.3	6.2
200	37.0	25.0
300	83.0	56.0
400	148.0	100.0
1000	926.0	617.0
5000	23,148.0	15,432.0

Source: Derived from NCHRP Report 214
[1] Assumes 50-50 directional split on roadway and that roadway has traffic on it for only 18 hours per day.

Vehicle type and associated driver eye-height values are also of interest, particularly in the LVLRoad rural environment. A significant proportion of vehicles on such roads are pick-ups, four-wheel drives and other types with driver eye heights significantly higher than AASHTO model or NCHRP 400 model assumptions.

Finally, the issue of driver familiarity and its effect on SSD is also of interest. LVLRoads by definition carry local traffic, which implies that the vast majority of drivers would be familiar with the road. Sight line restrictions and relatively difficult alignment should be less of a surprise to this familiar driving population.

Summary of Risk Assessment of NCHRP Report 400 SSD Design Values

Values for LVLRoad SSD based on the NCHRP Report 400 model can be derived using different assumptions about driver perception/reaction and deceleration. Use of different parameters within the framework of the model can produce SSD values from 10 to 25 percent less that the NCHRP Report 400 values. The resulting operational/safety risk is consistent with the NCHRP Report 400 values applied to higher volume facilities.

Accident Risk Assessment Using Neuman SSD Model

The SSD accident model proposed by Neuman has been identified as a useful tool to characterize the potential risk associated with designing a road with different criteria for SSD. Neuman's model hypothesizes that crash risk for a location with limited SSD is as shown in below:

Model for Assessing Risk of 'Substandard' Stopping Distance
Risk Assessment of Substandard SSD

$$\Delta A = 365\ ADT\ (AR_{LVR} \times L_R \times F_{AR}) \times 10^{-6}$$

where:

ΔA = Annual additional accidents attributable to less than full standard SSD

AR_{LVR} = Low volume road accident rate (related accidents)

L_R = Length (m) of highway over which restricted SSD exists

F_{AR} = Factor reflecting risk associated with severity of SSD restriction and presence of hazard within restriction

For any location with nominal or critical SSD, one can estimate the relative safety risk as follows. The risk is characterized as the annual number of additional crashes potentially attributable to the presence and nature of a sight restriction. It is modeled as being directly related to the normal or expected crash rate for the highway type, the traffic volume along the road, the length of highway over which the sight restriction occurs, and a risk factor that is associated with the severity of the sight restriction and the nature of the alignment or other features within the length of highway with a sight restriction. Table V-8 shows the factors derived by Neuman. These factors are the hypothesized multiplicative risk factors for sight-restricted locations as a function of the severity of the restriction and hazard within the restriction. They were rationalized as consistent with distributions of accidents by location.

The model was developed for research on the cost-effectiveness of improvements to SSD. Although uncalibrated, it provides useful insights to the issue of alternative design approaches for SSD for LVLRoads. If one accepts that a road designed to current criteria operates in a typical manner relative to SSD-related crashes, the model provides a basis for estimating the potential additional crash risk for substandard or lesser SSD.

The severity of a sight restriction can be characterized as a differential in the nominal or effective design speed of the restricted alignment from the desirable or stated design speed. Thus, for example, a vertical curve can be effectively 15 km/h "deficient" relative to desired design speed (for example, a vertical curve that provides the equivalent of 65 km/h of SSD on an 80-km/h design speed road has a 15 km/h deficiency). In the case here, the analysis considers the nominal risk of accepting a different design basis for SSD than that currently applied (i.e., AASHTO). Thus, an alternative, less conservative design is considered as nominally "deficient" compared to AASHTO for the purposes of discussion.

Table V-8
Hypothesized Accident Rate Factors for Evaluation of SSD Restrictions

Character of Geometric Condition Within SSD Restriction	Severity of SSD Restriction[1]			
	0 km/h	15 km/h	25 km/h	35 km/h
Minor Hazard[2]	0	0.5	1.2	2.0
Moderate Hazard[3]	0.4	1.1	2.0	3.0
Major Hazard[4]	1.0	1.8	2.8	4.0

Source: Cost-Effectiveness of Improvements to Stopping Sight Distance Safety Problem

* Factor multiplied by average accident rate is accident rate attributable to the combined effects of the roadway geometry and SSD restriction.

[1] Severity refers to difference between available SSD and AASHTO SSD for 85[th] percentile speed.

[2]Minor Hazard, Geometric Conditions: Tangent horizontal alignment, Mild curvature (> 600m radius), Mild downgrade (<3%), No driveways

[3]Moderate Hazard, Geometric Conditions: Minor intersection, Intermediate to controlling curvature, (300 m to 600 m radius), Moderate Downgrade (3%), Residential driveways

[4]Major Hazard, Geometric Conditions: Signficant intersection, Y-diverge, Sharp curvature (<300 m radius), Steep grade (> 6%), Narrow bridge, Narrowed pavement, One-lane roadway width

The length of the highway over which a sight restriction occurs can be taken from analysis of sight distance profiles, as described in Figure V-6. This dimension differs for different combinations of grade (A). The hazard within the restriction was hypothesized by Neuman as relating to the presence of, say, an intersection, sharp curve, narrow bridge, or other feature in combination with the limited sight distance. Note that hazards are by definition unusual conditions. Hence, most locations with sight restrictions will occur where the hazard as defined by the model is minor or moderate. The representative crash rate is taken from data from NCHRP Report 362 (70), adjusted to consider all crash types, to reflect effects of traffic volume on crash rates.

Table V-9 demonstrates the calculated effect of the deficiency in SSD. The reader should note that, as reported by Neuman, et al., the model was deliberately constructed to be conservative, (i.e., to overstate the potential effects of SSD geometry on safety).

The results in Table V-9 are converted in Tables V-10 and V-11 to express relative risk in terms of the number of years per location to experience one more crash. The tables reflect analysis of roads with traffic volumes of 101 to 400 vpd. The following is evident from review of Tables V-9 – V-11:

1. For moderate speed roads (such as in suburban or urban environments), an effective 15 km/h reduction of SSD holds little risk (one potential crash in every 30 to 120+ years) even where significant hazards exist.
2. For moderate speed roads, as much as a 25 km/h effective reduction in SSD may be acceptable for all but locations of major hazard.
3. For high speed roads, an effective 15 km/h reduction in SSD produces a nominal risk of one crash per 15 to 60+ years for minor to moderate hazard and the range of A values.
4. For high speed roads, even a 25 km/h reduction in SSD produces acceptable nominal risks (one crash per every 15 to 30 years) where minor hazards occur.
5. For high speed roads and a 25 km/h reduction in SSD, the nominal risk increases to one potential crash per every 5 to 10 years for moderate to high hazard locations.
6. For high speed roads, significant SSD reductions of 35 km/h are associated with additional risk levels of one crash every 5 to 10 years for all levels of risk.

Tables V-10 and V-11 also support research earlier cited regarding the sensitivity of crashes relative to values of A (difference in grades). Values of A less than 6 produce lower overall risk levels because of the geometry of the vertical curve. Alignments designed in flat to gently rolling terrain will in most cases involve such conditions.

Finally, Tables V-10 and V-11 address nominal risk levels for roads with traffic volumes between 100 and 400 vehicles per day. For very low volume roads (ADT less than 100 vpd), relative risk levels would clearly be much lower. Risk levels on the order of 50 to 150+ years for one additional crash would be associated with a 25-km/h reduction in SSD.

To summarize, although hypothetical in nature, the Neuman SSD model produces results that are entirely consistent with the safety research discussed previously. For roads with very low traffic volumes, the actual safety risk of lesser SSD than is normally provided is quite low, for SSD deficiencies as great as 25 km/h.

Maneuver Sight Distance

The existing AASHTO model, as well as the NCHRP Report 400 model, are arguably inappropriate for very LVLRoads, at least in terms of describing likely, critical events.

The concept of *maneuver sight distance* is discussed in NCHRP Report 400 and offered as a potential basis for design for sight distance with applicability in "constrained situations." Maneuver sight distance refers to the dimension required for a driver to perceive and maneuver around a constraint rather than brake to a full stop. Of course, the use of

Table V-9
Crash Risk Analysis Using Neuman SSD Model

80 KM/H A[4]	Amount of Restricted SSD (km/h)[1]	Length of Restriction[2] (km)	ADT	Accident Rate (mvk)	Risk Factor for Hazard Within Restriction[3] Minor	Moderate	Major	Crash Risk per Year for Range of Hazards Minor Delta A	Moderate Delta A	Major Delta A
4	15	0.055	250	2.8	0.5	1.1	1.8	7.03E-03	1.55E-02	2.53E-02
6	15	0.075	250	2.8	0.5	1.1	1.8	9.58E-03	2.11E-02	3.45E-02
8	15	0.100	250	2.8	0.5	1.1	1.8	1.28E-02	2.81E-02	4.60E-02
10	15	0.120	250	2.8	0.5	1.1	1.8	1.53E-02	3.37E-02	5.52E-02
12	15	0.140	250	2.8	0.5	1.1	1.8	1.79E-02	3.93E-02	6.44E-02

80 KM/H A[4]	Amount of Restricted SSD (km/h)[1]	Length of Restriction[2] (km)	ADT	Accident Rate (mvk)	Risk Factor for Hazard Within Restriction[3] Minor	Moderate	Major	Crash Risk per Year for Range of Hazards Minor Delta A	Moderate Delta A	Major Delta A
4	15	0.055	400	2.0	0.5	1.1	1.8	8.03E-03	1.77E-02	2.89E-02
6	15	0.075	400	2.0	0.5	1.1	1.8	1.10E-02	2.41E-02	3.94E-02
8	15	0.100	400	2.0	0.5	1.1	1.8	1.46E-02	3.21E-02	5.26E-02
10	15	0.120	400	2.0	0.5	1.1	1.8	1.75E-02	3.85E-02	6.31E-02
12	15	0.140	400	2.0	0.5	1.1	1.8	2.04E-02	4.50E-02	7.36E-02

80 KM/H A[4]	Amount of Restricted SSD (km/h)[1]	Length of Restriction[2] (km)	ADT	Accident Rate (mvk)	Risk Factor for Hazard Within Restriction[3] Minor	Moderate	Major	Crash Risk per Year for Range of Hazards Minor Delta A	Moderate Delta A	Major Delta A
4	25	0.050	250	2.8	1.2	2.0	2.8	1.53E-02	2.56E-02	3.58E-02
6	25	0.065	250	2.8	1.2	2.0	2.8	1.99E-02	3.32E-02	4.65E-02
8	25	0.075	250	2.8	1.2	2.0	2.8	2.30E-02	3.83E-02	5.37E-02
10	25	0.090	250	2.8	1.2	2.0	2.8	2.76E-02	4.60E-02	6.44E-02
12	25	0.105	250	2.8	1.2	2.0	2.8	3.22E-02	5.37E-02	7.51E-02

[1] Normal Deficiency in available SSD
[2] Per Figure V-6
[3] Per Table V-8
[4] Algebraic difference in grades

Table V-9
Crash Risk Analysis Using Neuman SSD Model

90/100 KM/H	Amount of Restricted	Length of		Accident	Risk Factor for Hazard Within Restriction[3]			Crash Risk per Year for Range of Hazards		
					Minor	Moderate	Major	Minor	Moderate	Major
A[4]	SSD (km/h)[1]	Restriction[2] (km)	ADT	Rate (mvk)				Delta A	Delta A	Delta A
4	35	0.100	250	2.8	2.0	3.0	4.0	5.11E-02	7.67E-02	1.02E-01
6	35	0.125	250	2.8	2.0	3.0	4.0	6.39E-02	9.58E-02	1.28E-01
8	35	0.145	250	2.8	2.0	3.0	4.0	7.41E-02	1.11E-01	1.48E-01
10	35	0.165	250	2.8	2.0	3.0	4.0	8.43E-02	1.26E-01	1.69E-01
12	35	0.190	250	2.8	2.0	3.0	4.0	9.71E-02	1.46E-01	1.94E-01

90/100 KM/H	Amount of Restricted	Length of		Accident	Risk Factor for Hazard Within Restriction[3]			Crash Risk per Year for Range of Hazards		
					Minor	Moderate	Major	Minor	Moderate	Major
A[4]	SSD (km/h)[1]	Restriction[2] (km)	ADT	Rate (mvk)				Delta A	Delta A	Delta A
4	15	0.100	400	2.0	0.5	1.1	1.8	1.46E-02	3.21E-02	5-26E-02
6	15	0.155	400	2.0	0.5	1.1	1.8	2.26E-02	4.98E-02	8.15E-02
8	15	0.205	400	2.0	0.5	1.1	1.8	2.99E-02	6.58E-02	1.08E-01
10	15	0.250	400	2.0	0.5	1.1	1.8	3.65E-02	8.03E-02	1.31E-01
12	15	0.300	400	2.0	0.5	1.1	1.8	4.38E-02	9.64E-02	1.58E-01

90/100 KM/H	Amount of Restricted	Length of		Accident	Risk Factor for Hazard Within Restriction[3]			Crash Risk per Year for Range of Hazards		
					Minor	Moderate	Major	Minor	Moderate	Major
A[4]	SSD (km/h)[1]	Restriction[2] (km)	ADT	Rate (mvk)				Delta A	Delta A	Delta A
4	25	0.100	400	2.0	1.2	2.0	2.8	3.50E-02	5.84E-02	8.18E-02
6	25	0.140	400	2.0	1.2	2.0	2.8	4.91E-02	8.18E-02	1.14E-01
8	25	0.170	400	2.0	1.2	2.0	2.8	5.96E-02	9.93E-02	1.39E-01
10	25	0.210	400	2.0	1.2	2.0	2.8	7.36E-02	1.23E-01	1.72E-01
12	25	0.245	400	2.0	1.2	2.0	2.8	8.58E-02	1.43E-01	2.00E-01

[1] Normal Deficiency in available SSD
[2] Per Figure V-6
[3] Per Table V-8
[4] Algebraic difference in grades

Table V-10
Summary of Safety Risk of Limited SSD on Low Volume Local Roads – 101 to 400 vpd
Moderate Speed Roads – 60 - 80 km/h

Design Condition	Years to accumulate 1 additional crash due to deficiency in SSD	
	$A < 6$[1]	$A > 6$[1]
15 km/h Deficiency in SSD		
Minor Hazard	120	60
Moderate Hazard	50	30
Major Hazard	30	20
25 km/h Deficiency in SSD		
Minor Hazard	50	30
Moderate Hazard	30	20
Major Hazard	20	15

[1] A is the algebraic difference in grades

Table V-11
Summary of Safety Risk of Limited SSD on Low Volume Local Roads – 101 to 400 vpd
High Speed Roads – 90 - 100 km/h

Design Condition	Years to accumulate 1 additional crash due to deficiency in SSD	
	$A < 6$[1]	$A > 6$[1]
15 km/h Deficiency in SSD		
Minor Hazard	60	30
Moderate Hazard	30	15
Major Hazard	15	10
25 km/h Deficiency in SSD		
Minor Hazard	30	15
Moderate Hazard	15	10
Major Hazard	10	5
35 km/h Deficiency in SSD		
Minor Hazard	15	10
Moderate Hazard	10	8
Major Hazard	8	5

[1] A is the algebraic difference in grades

maneuver sight distance for design implies that the road is sufficiently wide to enable maneuvering to occur.

Adoption of maneuver sight distance as a basis for design in the LVLRoad environment offers many potential advantages. First, it undoubtedly is a more realistic model of nominally critical behavior. Second, it offers the potential for justifying a less restrictive design basis for sight distance, without abandoning the concept of sight distance as a design concern for all roadways.

One important aspect of designing for maneuver sight distance is the identification of an object around which the driver must maneuver. The design model object height would control design values for vertical curvature. In NCHRP Report 400, it is noted that the appropriate object height should be 0, representing a pothole, washout or other pavement or surface problem. This is comparable with the design basis for sight distance for low volume roads employed by RTAC. Alternatively, greater height objects could be used for designing to maneuver sight distance.

Table V-12 shows design values for maneuver sight distance, as derived based on human factors research. Note that the time to maneuver increases as speed increases.

Table V-12
Maneuver Sight Distance (m)

Initial Speed (km/h)	Maneuver Time (sec)	Maneuver Sight Distance (m)
50	3.0	41.7
60	3.4	56.7
70	3.8	73.9
80	4.2	93.3
90	4.6	115.0
100	5.0	138.9

Source: NCHRP Report 400

An evaluation of alternative design models using maneuver sight distance was performed to determine the applicability of the concept for LVLRoads. See Tables V-13 and V-14.

Table V-13 shows the K values for vertical curve design for SSD as proposed by NCHRP Report 400, and maneuver sight distance (MSD) for a range of design object heights (0, 150 mm, and 600 mm). The values for K for the NCHRP Report 400 SSD model are based on a 600-mm object height. Note that by selecting a 0-m object height, the resulting values for K are actually greater than SSD K values. The use of alternative object heights produces lesser K values, which is presumably desired.

Table V-14 converts the K values to speed differentials to aid in the assessment of the criteria. Thus, for a given initial speed and K resulting from an object height, the effective "design speed" can be computed for a comparable vertical curve designed to the NCHRP 400 SSD model. The 0-m object height values are shown as negative speed differentials, in other words, the effective speed is greater than the SSD design speed.

Table V-13
Maneuver Sight Distance K-Values Compared to Equivalent NCHRP Report 400 Sight Distance K- Values for Crest Vertical Curves

Initial Speed (km/h)	Stopping Sight Distance (NCHRP Proposed) (m)	Stopping Sight Distance K-Value based on h_e=1080mm h_0=600mm (NCHRP Proposed)	Maneuver Sight Distance (m)	Maneuver Sight Distance K-Value based on h_e=1080mm h_0=0mm	Maneuver Sight Distance K-Value based on h_e=1080mm h_0=150mm	Maneuver Sight Distance K-Value based on h_e=1080mm h_0=600mm	Maneuver Sight Distance (600mm Object) Equivalent Design Speed of Stopping Sight Distance K-Values (km/h)
50	63.1	7	41.7	8	4	3	35
60	82.5	11	56.7	15	8	5	43
70	104.2	17	73.9	25	13	8	53
80	128.2	25	93.3	40	21	13	63
90	154.4	37	115.0	61	32	20	74
100	182.9	51	138.9	89	47	29	83

Source: NCHRP Report 400

Table V-14
Maneuver Sight Distance K-Values Compared to Equivalent NCHRP Report 400 Sight Distance K-Values for Crest Vertical Curves

Initial Speed (km/h)	Maneuver Sight Distance (m)	Maneuver Sight Distance (0mm Object) Equivalent Design Speed of Stopping Sight Distance K-Values (km/h)	ΔV (km/h) (V_{SSD}-V_{MSD})	Maneuver Sight Distance (150mm Object) Equivalent Design Speed of Stopping Sight Distance K-Values (km/h)	ΔV (km/h) (V_{SSD}-V_{MSD})	Maneuver Sight Distance (600mm Object) Equivalent Design Speed of Stopping Sight Distance K-Values (km/h)	ΔV (km/h) (V_{SSD}-V_{MSD})
50	41.7	53	-3	40	10	35	15
60	56.7	67	-7	53	7	43	17
70	73.9	80	-10	63	7	53	17
80	93.3	92	-12	75	5	63	17
90	115.0	105	-15	86	4	74	16
100	138.9	118	-18	97	3	83	17

Source: NCHRP Report 400

Use of a 150-mm object height with MSD values results in nominal speed differentials of 3 to 10 km/h compared to design according to SSD per NCHRP Report 400. A 600-mm object height and MSD values produces speed differentials on the order of 15 to 17 km/h for the range of initial speeds.

If one considers the results of Tables V-13 and V-14 in combination with the previous work on risk assessment and safety research, a rationale emerges for the consideration of an alternative model for design of vertical alignment within the LVLRoad environment. Design for MSD with a 600-mm object height produces vertical curve criteria that reflect a 15 to 20 km/h effective speed reduction compared to a SSD design baseline. This speed differential is within acceptable limits for lower volume ranges of the LVLRoad system.

Evaluation of Costs and Other Impacts of Alternative Design Criteria for SSD

Adherence to any design standard, value or criterion carries with it a cost. An objective of reasonable design guidelines should be to balance these costs with the benefits the guideline is intended to produce. Construction and maintenance costs are of primary concern to LVLRoads engineers.

An evaluation was performed of the incremental costs of reconstructing local road alignment to meet variable levels of vertical curve designs. Costs of removing pavement and regrading to improve vertical geometry were calculated for a range of curve criteria as discussed above. The evaluation used the NCHRP Report 362 cost model, adjusted to update values to 1998 and to reflect reconstruction versus new construction.

Table V-15 presents the results of the analysis, which illustrates the cost of reconstruction to full AASHTO vertical alignment criteria versus retaining a range of nominal "deficiencies" in vertical curve lengths. The costs are per vertical curve.

A review of Tables V-13 through V-15 demonstrates a fundamental problem with the cost-effectiveness of vertical curve design. Marginal nominal deficiencies are somewhat less costly to correct, yet are associated with minimal safety effectiveness. Conversely, to increase a vertical curve by 25 or 35 km/h requires a substantial amount of earthwork, affects a significant length of road, and is hence much more costly. Furthermore, it is only the significant nominal deficiencies that produce any meaningful safety benefits over time.

Overall, the costs of providing alignment consistent with full, nominal AASHTO criteria are substantial. For example, for roads in rolling or mountainous terrain with only moderately deficient geometry, each vertical curve could result in a cost of $250,000 to $650,000. This translates to an annualized cost of $20,000 to $50,000. Assuming two to three crest curves per km, the annualized reconstruction cost could be easily greater than $50,000. For perspective, consider that normal per km maintenance costs for low volume local roads amount to $2700 to $7500 for *all* maintenance activities, and that capital cost thresholds per km established previously are under $10,000.

Recommendations Regarding SSD Design for LVLRoads

Considering the unique characteristics of LVLRoads, there is ample evidence to support the use of a less conservative design basis for SSD for LVLRoads. Less conservative refers to acceptance of lesser design dimensions for sight distance and/or vertical curve geometry.

Table V-15

Estimated Costs to Reconstruct Vertical Curves (1998 Dollars)

Terrain	Effective Increase in SSD (km/h) Minor Deficiency (5 to 10 km/h)	Moderate Deficiency (10 to 20 km/h)	High Deficiency (20 to 30 km/h)
Level	$70,000[a]	$75,000[a]	---------------
Rolling	$210,000[b]	$250,000[b]	$280,000[b]
Mountainous	$520,000[c]	$650,000[c]	$770,000[c]

Source: NCHRP Report 362
[a] Length of alignment affected approximately 200m
[b] Length of alignment affected approximately 500m
[c] Length of alignment affected approximately 800m

Issues of what are reasonable design values and what format of the design criteria remain to be addressed. This section of the report outlines a suggested approach to sight distance design for LVLRoads.

With respect to design values, for lower class, very low volume roads, the evidence is that sight distance and vertical curvature criteria can be reduced by 10 to 25 percent (depending on speed) without expecting any measurable degradation in the safety performance of LVLRoads. Stated differently, as much as a 15 to 25 km/h reduction in the effective design speed of alignment (compared to current AASHTO SSD criteria or the proposed NCHRP Report 400 criteria) should be acceptable for the type and class of road addressed in this study. This finding reflects previous safety and human factors research, risk assessment that considers the low volume nature of such roads, and knowledge of the sight-restricting nature of highway geometry. The authors of this study consider this finding to be the most significant with respect to LVLRoads, with the actual format of design criteria less important.

With respect to format, there are many possible approaches to achieving consistent and cost-effective criteria. One would be to retain AASHTO model formulation and values for LVLRoads consistent with other roadway types, but to accept a range of speeds or speed differentials. A second approach, suggested by the authors as preferred, is to establish a separate set of criteria specifically formulated to address the unique conditions found on LVLRoads.

An important issue, with respect to format, concerns new versus reconstructed roads. The current AASHTO Policy considers both as equivalent within Policy. The majority of LVLRoad design problems involve reconstruction, with many miles of such roads built either to no effective standards or to standards that prevailed over 50 years ago. Consideration should be given to formulating design policy that acknowledges an operational and safety history of such roads, separate perhaps from policy for a new LVLRoad.

Another format issue concerns the definition of a design control for sight distance. This study does not advocate the abandonment of any design control that addresses requirements for the driver to see the alignment ahead. However, this study does recommend consideration of an explicit use of an alternative to stopping sight distance -- *maneuver sight distance* as the basis for design of very low volume roads (i.e., those with less than 100 vpd).

Table V-16 describes a rational, consistent approach to design for SSD for LVLRoads. The approach recognizes that LVLRoads are a part of the universe of all roads. As such, the structure of design policy for LVLRoads should be consistent or compatible with that for higher volume facilities. The approach also attempts to address what the authors believe is a structural shortcoming of current policy, and even of the proposed NCHRP Report 400 criteria. This shortcoming is the reasonableness of the same operational model for all conditions.

New Construction Design Criteria

For new construction of very low volume local roads (less than 100 vpd), neither the current AASHTO model nor the NCHRP Report 400 model describes a reasonably frequent, critical event. A maneuver sight distance model, with a 600-mm object height, provides a more reasonable design basis. It also produces vertical curve design requirements that are consistent with safety research results and risk assessments.

In designing for volumes at the "high" end of the LVLRoad spectrum (251 to 400 vpd), it is reasonable to apply the NCHRP Report 400 model, including eye and object height dimensions. However, acceptance of SSD values based on 50th percentile deceleration and 90th percentile brake reaction times is recommended as a means of enabling the use of less restrictive, more cost-effective design values for LVLRoads, at a reasonable, acceptable risk level.

Table V-16
Potential Structure of Low Volume Local Road Design Criteria for Stopping Sight Distance on Two-Lane Roads

	DESIGN TRAFFIC VOLUMES (VPD)				
Type of Project	0 – 100	101 – 250		251 – 400	> 400
New Construction	Maneuver Sight Distance[1]	Maneuver SD at 'lower risk' locations[2]	LVLR SSD for 'higher risk' locations[3]	LVLR SSD (NCHRP 400) model with different parameters[4]	AASHTO/NCHRP 400 Model & Resultant SSD Values
Reconstruction	Maneuver SD values or Existing SD unless evidence of SD-related safety problems exists[5]				

[1] For 2-lane roads
[2] Away from intersections, bridges, railroad grade crossings & sharp curves
[3] Intersections, narrow bridges, in advance of sharp curves or steep downgrades
[4] LVLR parameters – 50th percentile and 90th percentile brake reaction
[5] Where evidence of SD-related safety problems exist, design values for new construction apply

Design for the mid range—101 to 250 vpd, should be as flexible as possible. Previous research and the knowledge of those who operate the LVLRoad environment suggest that the risk of poor sight lines varies by location along the road. We suggest that this concept be incorporated within the framework of LVLRoad criteria. Maneuver sight distance values as a minimum design basis are acceptable at locations considered to be "low risk." This includes alignment that does not include an intersection, bridge, sharp curve, steep grade, railroad grade crossing, or other confounding geometric feature. Wherever such features do exist, a reasonable minimum design basis is to provide LVLRoad SSD as defined above for roads with up to 400 vpd. Of course, a designer should have the flexibility and may choose to apply the greater values continuously. The policy should not discourage this practice.

Table V-16 expresses criteria for SSD for 2-lane roads. A reasonable approach for single-lane roads would be twice the respective values for 2-lane roads. Most single-lane roads occur with traffic volumes less than 100 vpd. Thus, single-lane SSD design would be based on twice the maneuver sight distance for a given location.

Reconstruction Criteria

The recommended design criteria for LVLRoads requiring reconstruction is intended to provide the maximum flexibility to the designer. The geometry of SSD, costs of even marginal or incremental improvements and low traffic volumes combine to make reconstruction to a full SSD criterion not cost-effective except in unusual cases. Maximum flexibility carries with it a need for the designer to understand the safety and operating characteristics of the road, and in effect, to be able to identify a road or road segment as unusual.

Table V-16 outlines an approach that AASHTO should consider. As a minimum criterion, acceptance of either maneuver sight distance or the existing, prevailing sight distance is recommended *unless there is evidence of a site-specific safety problem attributable to inadequate sight distance.* What constitutes evidence of a problem can be left to the judgment of the designer or agency. The intent, of course, is that the designer make an effort to study, understand and document the existing conditions.

This design approach focuses the attention of any potential sight distance improvements to locations most likely to need or benefit from them. It should help to minimize or avoid the reconstruction of a road to a standard value merely for the sake of it, and to minimize design exception processing.

Other Issues

The above approach is recommended for the range of highway types within the overall local classification. Additional research may be needed to further investigate the value or propriety of alternative design values for, say, recreational roads, resource recovery roads, or industrial access roads. USFS and other sources suggest the potential for alternative design models for roads used primarily by trucks. Trade-offs are complicated. Larger vehicles have longer braking distances, yet are driven by professional drivers. Eye heights are much greater; suggesting shorter vertical curves would be more acceptable.

This chapter recommends a substantial reduction in design values for SSD and vertical curvature for all LVLRoads. Although even further reductions for *Resource Recovery* or *Industrial Access* roads are possible, within the resources available for this study, it was not possible to definitively address such roadway types.

VI. Roadside Design

The design of the roadside is recognized as among the most important features on LVLRoads. The roadside begins at the edge of pavement. Roadside design elements include the shoulder, slopes, drainage structures and ditches, and objects such as trees, buildings, etc.

In terms of geometric design, the key issues affecting LVLRoads are the need for or appropriate dimensions for clear zones, and the desirability or requirements for guardrail in locations of hazard. Most serious crashes on LVLRoads are single vehicle, typically stemming from an encounter with a roadside object. Provisions for a "safe" roadside produces conflicts with typical right-of-way and cost constraints associated with LVLRoads.

This section of the report addresses design issues involving clear zones, need for guardrail versus embankment, and roadside slope.

History of AASHTO Roadside Design Policies

A review of AASHTO design policies from 1940 to the present provides interesting insights on the evolution of knowledge on roadside safety, and its affect on roadside design.

The 1940 Policy on Geometric Highway Types did not expressly promote design of a safe roadside. Decisions regarding the roadside were discussed largely on cost and availability of right-of-way. Guardrail was recommended at "points of extreme danger." This included locations with a fill height greater than 6 feet and where changes in alignment forced a speed reduction (such as horizontal curves). The Policy stated that guardrail was not required where slopes of 4:1 or flatter were provided because "a driver forced onto such a slope has a chance of regaining control of the vehicle." The use of guardrail versus flatter slopes was based on the cost differential and available right-of-way. No mention is made of a clear roadside concept. The value of providing continuous shoulders to furnish support for the pavement and to accommodate stopped vehicles was recognized. Desirable shoulder widths of 8 to 10 feet were recommended, with a minimum shoulder width of 4 feet recommended in constrained areas.

The 1949 *Highway Practice in the United States of America* did not expressly promote design of a safe roadside outside of a few qualifying remarks. Recognition was made that "flat slopes should be used when possible;" however, criteria were not expressly provided. It was stated that slopes in stable material may be as flat as 3:1 or 4:1 and that slopes as steep as 1:1 were rarely satisfactory. Determination of roadway sideslopes for use in design was based on an economic, maintenance, and effect on right-of-way perspective rather than safety. The value of providing continuous shoulders to furnish support for the pavement and accommodation of stopped vehicles was recognized. Desirable shoulder widths of 8 to 10 feet were recommended. A minimum shoulder width of 4 feet is recommended in constrained areas. The use of guardrail is not discussed. No mention was made of a clear roadside concept.

The 1954 *Policy on Geometric Design of Rural Highways* did not expressly promote design of a safe roadside. This policy, however, was the first document to reflect a basic understanding of the relationship of roadside design and the severity of accidents. Again, the value of providing continuous shoulders to furnish support for the pavement and accommodation of stopped vehicles was recognized. It was stated that provision of shoulders allows for additional clearance to signs and objects and that "a large reduction in accidents could be expected with provision of adequate continuous shoulders." For high type facilities, a 10-foot shoulder was recommended. For lower type facilities, 6-8 foot shoulders (4-foot minimum) were recommended.

The 1954 Policy noted that guardrail should be used "where vehicles accidentally leaving the highway would be subjected to hazards." These hazards included fills with steep slopes, fills with sharp curvature, and locations where fixed objects are on the roadside. This was the first direct reference advocating the use of guardrail to shield vehicles specifically from roadside objects. The need for guardrail was still primarily related to sideslopes. When slopes of 4:1 or flatter are provided, guardrail may be omitted as "a driver has a chance of regaining control of the vehicle." Recognition was made that "flatter sideslopes are safer than steeper ones." Specifically, 4:1 and flatter sideslopes were identified as the threshold where the chance of an accident is reduced for motorists leaving the roadway. Suggested sideslopes for design were presented, varying from 4:1 to 2:1 based on height of cut/fill and the type of terrain.

The 1965 *Policy on Geometric Design of Rural Highways* dealt with roadside design issues in essentially the same manner as the 1954 Policy. A key difference was that suggested sideslopes for design vary from 6:1 to 2:1.

Design policies from the 1940's to mid 1960's tended to emphasize design effects on traffic operations and safety within the traveled way. The general view was that a well-designed road should be navigable for most any driver. There was little explicit recognition that roadside encroachments were to be expected and should be considered as part of the design process for the roadside.

The 1967 *Highway Design and Operational Practices Related to Highway Safety* was the first publication to expressly promote the design of a safe roadside. By the mid-1960's, travel in the United States had increased greatly, as had average speeds. Researchers and designers were beginning to build a knowledge base on the effects of geometric design on highway safety. This 1967 publication introduced the concept of the clear roadside as it exists today. It was stated that "roadsides should be designed for the eventuality that vehicles will leave the traveled way. The elimination of obstructions must be expedited by engineering the roadside for safety."

Research had indicated that roadsides having flat slopes and ample recovery areas appeared to have lower accident rates than those without these facilities. Rates of 6:1 or flatter were identified as being negotiable with a good chance of recovery (note the change from 4:1 previously). It was considered desirable to provide an unencumbered recovery area up to 30 feet from the edge of the traveled way. This dimension was chosen because research indicated that 80 percent of run-off-road accidents did not travel beyond this limit. To provide this recovery area, several states proposed maintaining a slope of 8:1 for 10 to 12 feet outside the edge of shoulder before starting a steeper slope. In addition, steps were recommended to eliminate heavy objects from roadside areas. This was emphasized in areas that were especially vulnerable (such as areas outside horizontal curves). This philosophy represented a shift from the use of guardrail to providing flatter slopes as a preferred treatment of "problem" areas.

For the first time, the philosophy of remove, relocate, reduce impact, redirect, delineate was presented. Objects within 30 feet of the edge of pavement were recommended to be moved or at a minimum converted to breakaway sign supports (reduce impact). Only in those cases where the result of striking the object (or leaving the roadway) would be more severe than striking a rail should guardrail (or another barrier type) be used. As a last resort, an object should be delineated to let a motorist know that there is a potential danger on the roadside.

The 1969 *Geometric Design Standards for Highways Other Than Freeways* drew heavily on the ideas presented in the 1965 Blue Book and 1967 Design Practice Manual. The idea of a clear recovery area, preferably about 30 feet, was again presented. This recovery area should be provided along high-speed rural highways and should be clear of unyielding objects, if practicable. If not, protective devices should be installed.

Significant resources were invested in the 1970s and 1980s on studying the dynamic effects of roadside encroachments. Research efforts led to refinements in design of guardrails and other barriers, and to more sophisticated procedures for determining barrier design requirements. There was evolving, a recognition that, poor or inappropriate barrier design represented a potential hazard to drivers.

The 1988 *Roadside Design Guide* was a compendium of the most recent information regarding roadside design. This document expanded on the clear roadside concept developed in the 1960's. Policies developed during the 1960's focused on providing an unobstructed roadside area extending 30 feet beyond the edge of the traveled way. Obstacles within this area were subsequently removed, relocated, redesigned, shielded, and/or delineated. Field studies indicated that in limited situations, a vehicle could encroach further than 30 feet onto the roadside.

An expansion of these concepts was provided in the *Roadside Design Guide*. This document presented the concept of a *variable* clear zone distance based on traffic volumes, speed, horizontal curvature and roadside geometry. Research on roadside encroachment and crashes was incorporated in the guidelines. (The variable clear zone concept was first presented in a 1977 AASHTO *Guide for Selecting, Locating, and Designing Traffic Barriers*). Slopes were classified as recoverable, non-recoverable, and critical. Recoverable slopes were identified as all embankment slopes 4:1 and flatter. It was acknowledged that vehicles that encroach on recoverable slopes can generally stop their vehicles and return to the roadway surface. A non-recoverable slope (3:1 to 4:1 embankment) is one on which a motorist will be unable to stop or return to the roadway easily. A critical slope was identified as one on which a vehicle is likely to overturn. This includes slopes steeper than 3:1.

The use of a roadside barrier was based on the premise that a traffic barrier should be installed only if it could be expected to reduce the severity of a potential accident. Typically, barrier warrants have been based on the subjective analysis of roadside conditions. If the consequences of a vehicle hitting a fixed object (or running off the roadway) is expected to be more serious than hitting a traffic barrier, than the barrier is considered warranted. Warrants were presented for barrier needs based on embankment height and sideslopes. Warrants were also provided for fixed objects on the roadside.

The *1996 Roadside Design Guide,* AASHTO's latest version, provides an update to the 1988 Guide in metric units. The key points and concepts of roadside design are unchanged.

Current Design Policy for Local Roads

Chapter V of the *AASHTO Policy on Geometric Design* addresses roadside design policy for Local Roads. Roadside design policy recommendations include recommendations for foreslopes and border areas, and dimensions for clear zones.

The discussion of foreslopes and an accompanying discussion of right-of-way widths suggests that "more is better" in terms of vehicle operations, maintenance and safety. Indeed, use of guardrail is suggested where a reasonable foreslope can not be provided. There are no citations of traffic volumes nor of design or operating speeds in the discussion of foreslopes on both Local roads and Special Purpose Roads.

Chapter V also discusses horizontal clearance dimensions. On page 425, it is stated that "a clear zone of 3 m or more from the edge of traveled way, appropriately graded and having gentle slopes and rounded cross-sectional design, is desirable." Further discussion suggests even greater dimensions at selected locations such as the outside of sharp curves.

In the section on special purpose roads, the 3 m clear recovery area is reiterated as being desirable for "higher order roads." The Policy concedes that for lower order roads and "where economic and environmental concerns are great" (p. 452), lesser dimensions may be appropriate. However, no specific values are cited, nor is there any discussion of traffic volumes, speeds, vehicle types or other factors that would help define where such lower values may be acceptable. Thus, the reader of the Policy tends to be left with the 3 m clear recovery dimension as the basic design value for Local Roads.

On page 438, in the section on urban local roads, even more definitive language is used in discussing the border area, which is used for utilities, sidewalks, etc. Here the Policy states "The border width may be a minimum of 1.5 m, but desirably should be 3.0 m or wider. Where the available right-of-way is limited and in areas of high right-of-way costs, as in some industrial and commercial areas, a buffer width of 0.6 may be tolerated."

The discussion of border areas in urban local road design focuses on typical roadside activities and requirements, as opposed to functional safety issues. No explicit mention is made, however, of the safety consequences of lesser dimensions, nor is there any discussion of volume or speed considerations. A designer faced with a compromise design decision regarding a lesser border area can find justification only by inference from the text. Moreover, as was discussed above, the explicit language suggests the minimum criteria are firm and should be followed.

Summary of Literature on Roadside Design for LVLRoads

Chapter II of this report included a comprehensive summary of LVLRoad roadside design literature. Clear zone, guardrail and embankment safety sensitivities were covered. The following is an overview of key findings. For more detail, the reader is referred to Chapter II.

Roadside Slope and Embankment versus Guardrail

The technical literature presents a strong consensus with respect to the cost-effectiveness of guardrail or other roadside barriers for LVLRoads. Previous studies by Stephens (60), Sicking and Ross (62) and others produced findings that barriers are not cost-effective at volumes under 400 vpd for normal applications in which culverts, trees or embankments are the primary roadside objects of interest.

A relatively recent study was performed using detailed HSIS data (see Figure II-9) from Michigan. The Michigan data included lower class roads with lower traffic volumes, and detailed information on the presence of guardrail. This study confirmed that roadside barriers for roads with traffic volumes within the LVLRoad realm are not generally cost-effective.

Finally, a general consensus of local and some state design practice for lower class roads suggests that guardrail is not typically found to be cost-effective for traffic volumes on the order of 400 vpd or less.

Clear Zone

The clear zone is defined as that portion of the roadside starting at the edge of traveled way that is free of obstructions and sufficiently flat to enable an errant vehicle to encroach without overturning. Shoulders are thus part of a roadside's clear zone.

Establishment of clear zone dimensions for design is based in part on knowledge of roadside encroachment rates and characteristics. Precise knowledge about rates of encroachment and relative distributions of offset distances from the traveled way is sketchy, in large part due to the difficulty of collecting reliable data. These problems hold for higher volume roads, and are particularly troublesome for the lowest volume roads. The range of encroachment study findings was presented in Figure II-4.

Uncertainty about encroachment characteristics is less important for higher volume facilities when considering design decisions, warrants, etc. For LVLRoads, however, depending on which study one uses, one can derive a significantly greater or lesser view of encroachment rates on the lowest volume roads.

Risk Assessment of Roadside Design

This section documents risk assessment of roadside design parameters for LVLRoads. Key geometric and design features selected for evaluation include the following:

- Clear zone dimensions
- Need for roadside protection (guardrail warrants)

Original planning for the research anticipated performing special accident studies using HSIS data. Review of the literature revealed similar studies had recently been accomplished. To supplement these studies, an alternative approach was undertaken that was similar to the risk assessment approach used for the other geometric features in the study (sight distance and horizontal alignment).

Risk assessment of roadside design elements utilized design procedures and tools currently used by AASHTO in site-specific roadside design analyses. AASHTO's "ROADSIDE" program, version 5.0 was used to model a range of hypothetical roadway, volume and speed conditions. The program was used to generate measures of crash frequency and severity associated with roadside objects, ranges of clear zone, and presence of guardrail. Comparison of modeled values per km of LVLRoad enabled judgments about the relative cost-effectiveness of design policies governing clear zone and use of guardrail. Note that the analyses are intended to supplement a synthesis of roadside design technical literature.

Study Design

The ROADSIDE program was used to predict the "years per injury or fatal crash per km" for the following LVLRoad conditions:

- Low to moderate design speed (50 km/h); and high design speed (110 km/h)
- Average Daily Traffic of 100 vpd, 250 vpd, and 400 vpd
- High Truck percentage (>30 percent); and low truck percentage (5 percent)

A range of roadside and alignment conditions were modeled as follows:

- Horizontal Curvature (250 m to 10,000 m (representing a tangent))
- Grade (0 percent to 6 percent downgrade)
- Clear Zone (0 m through 3 m in 0.5 m increments)
- Roadside Obstacles - assumed continuous throughout the section – (guardrail, 0.3 m diameter trees)

For the purposes of analysis, other roadway characteristics were held constant, using 2-lane roads, 3.0 m lane widths, and 1 km long section lengths. A hypothetical 1 km section length was run for each combination of traffic and geometric variables.

The study design resulted in running over 120 analyses of various combinations of roadside, alignment and traffic variables. Combinations of geometric variables were also run, including, for example, 400 m radius curve with 6 percent grade.

The various combinations of geometry and roadside conditions were intended to describe an "envelope" of overall roadside safety risk, using the tools and basic procedures currently considered by the design profession.

Performance Measures

The ROADSIDE model was developed for use in testing the safety-effectiveness of roadside treatments ranging from obstacle removal to embankment modifications to use of guardrail. Traditional use of the model involves development of expected costs of crashes, against which the capitalized costs of the design action and annual maintenance costs are balanced. Accident costs include costs of property damage only, injury and fatal crashes.

In the application here, the ROADSIDE model was used in a slightly different manner. In keeping with the design philosophy for LVLRoads, the primary concern was to estimate the safety risk as related to fatal and injury accidents, versus total accidents. Also, the model was not used for traditional cost-benefit analyses, but rather to generate "years to accumulate" one additional serious (i.e., fatal or injury) crash. This measure is consistent with the risk assessment approach described in Chapter III of this report.

Analysis Results

The ROADSIDE analysis exercise provides insights to relative safety risks of variable design conditions on LVLRoads. The results should be reviewed with appropriate caution. They reflect modeling of hypothetical conditions, and are intended for general reference purposes. Note, however, that they are based on the current AASHTO design procedures for roadside design, and hence have a certain measure of credibility.

Effects of Truck Volumes

The initial plan was to model different traffic conditions reflective of LVLRoads with high concentrations of truck traffic (i.e., resource recovery roads, industrial access roads, etc.) versus other types. The ROADSIDE model provides a means of adjusting traffic distributions. However, this variable was found to be insensitive to encroachment frequency or severity, a result that was confirmed by discussing the model with FHWA staff in Washington DC. Although high concentrations of truck volumes should result in different safety performance, the current version of ROADSIDE does not support an in-depth analysis of them. All analyses reported below are based on truck volumes representing 5 percent of total traffic.

Clear Zone Dimensions

Table VI-1 summarizes the results of safety risk analyses of variable clear zone dimensions for LVLRoads with "open" alignment (i.e., mild grades and curvature) for the range of design speeds, traffic volume levels and clear zones from 0 to 3.0 m. The table shows frequencies of injury crash expectancy assuming the presence of continuous roadside objects consistent with a row of trees (severity index of 3.2 at 50 km/h, and 5.0 at 110 km/h). The table expresses the modeled values of years to expect 1 injury crash per km.

The expected frequency of severe roadside crashes as modeled is extremely small—on the order of one crash every 40 years or more per km for all but higher speed roads with traffic volumes on the upper end of the LVLRoad spectrum. Indeed, even for a 0 m clear zone on a 110 km/h road with 400 vpd, the frequency of 1 severe crash every 14 years per km is at the threshold of marginal safety-effectiveness established in Chapter III.

Effect of Alignment and Clear Zone Dimensions on LVLRoad Safety

Table VI-2 shows a similar analysis for a hypothetical 1 km roadway with "severe" alignment as represented by sharp curvature and a steep grade. According to the ROADSIDE program, the alignment results in higher frequency of encroachments and hence more frequent crashes.

Even with such severe conditions, for lower volume LVLRoads (250 vpd and less), roadside crashes per km would occur once in 7 to 25 years. For ADT of 400 vpd on higher roads, frequencies are greater—once every 4 to 7 years for the range of clear zone dimensions. Note, however, that these conditions are theoretical. The curve radius of 250 m is representative of an 80 km/h design speed, 30 km/h less than the speed of the analysis. Also, it is unusual for alignment to be so severe as to be represented by this combination of curvature and grade over extended lengths (except, perhaps in mountainous terrain).

Risk Assessment of Alternative Clear Zone Dimensions

Tables VI-1 and VI-2 provide perspective on the relative frequency of roadside encroachments. In terms of clear zone dimensions, Tables VI-3 and VI-4 provide further insights. Tables VI-3 and VI-4 document the *change in risk* associated with marginally different roadside clear zone policies for the range of speed, volume and roadside conditions. Change in risk is of primary interest, as the real question is "What difference might one expect if a different clear zone dimension were used compared to AASHTO's 3.0 m clear zone?"

Table VI-1
Safety Risk Assessment of Alternative Clear Zone Design Dimensions for LVLRoads

['Open' Alignment – 0% Grade, No Curve]

Average Daily Traffic -- VPD	Years to Accumulate 1 Injury Crash per km for Clear Zone Dimension in Meters						
	0.0	0.5	1.0	1.5	2.0	2.5	3.0
	50 km/h Design Speed (Continuous Trees Along Roadside – SI = 3.2)						
100	93	132	170	212	262	320	390
250	37	53	68	85	105	128	156
400	23	33	42	53	65	80	98
	110 km/h Design Speed (Continuous Trees Along Roadside – SI = 5.0)						
100	55	60	67	74	82	89	97
250	22	24	27	30	33	36	39
400	14	15	17	19	20	22	24

Table VI-2
Safety Risk Assessment of Alternative Clear Zone Design Dimensions for LVLRoads

['Severe' Alignment – 250 m Radius Curve, 6% Downgrade]

Average Daily Traffic -- VPD	Years to Accumulate 1 Injury Crash per km for Clear Zone Dimension in Meters						
	0.0	0.5	1.0	1.5	2.0	2.5	3.0
	50 km/h Design Speed (Continuous Trees Along Roadside – SI = 3.2)						
100	26	37	48	60	74	91	111
250	10	15	19	24	30	36	44
400	6	9	12	15	19	23	28
	110 km/h Design Speed (Continuous Trees Along Roadside – SI = 5.0)						
100	17	18	20	23	25	28	30
250	7	7	8	9	10	11	12
400	4	5	5	6	6	7	7

Table VI-3
Safety Risk Assessment of Incremental Risk per Km of Alternative Clear Zone Dimensions

['Open' Alignment – 0% Grade, No Curvature]

Average Daily Traffic -- VPD	Change in Risk per Km for Alternative Clear Zone Design Dimensions		
	3.0 m to 2.0 m	3.0 m to 1.0 m	3.0 m to 0.0 m
	50 km/h Design Speed (Continuous Trees Along Roadside – SI = 3.2)		
100	> 150 years	> 150 years	> 150 years
250	> 150 years	> 150 years	> 150 years
400	> 150 years	65 years	35 years
	110 km/h Design Speed (Continuous Trees Along Roadside – SI = 5.0)		
100	> 150 years	> 150 years	120 years
250	> 150 years	90 years	50 years
400	100 years	50 years	30 years

Table VI-4
Safety Risk Assessment of Incremental Risk per Km of Alternative Clear Zone Dimensions

['Severe' Alignment – 250 m Radius Curve, 6% Downgrade]

Average Daily Traffic -- VPD	Change in Risk per Km for Alternative Clear Zone Design Dimensions		
	3.0 m to 2.0 m	3.0 m to 1.0 m	3.0 m to 0.0 m
	50 km/h Design Speed (Continuous Trees Along Roadside – SI = 3.2)		
100	> 150 years	85 years	34 years
250	90 years	32 years	13 years
400	65 years	21 years	9 years
	110 km/h Design Speed (Continuous Trees Along Roadside – SI = 5.0)		
100	> 150 years	60 years	42 years
250	50 years	24 years	17 years
400	48 years	18 years	9 years

Values in Table VI-3 were derived from Table VI-1 by computing the relative difference in severe crash frequency for a given set of conditions, associated with different clear zones. For example, the difference in a clear zone policy of 3.0 m versus 0 m for a 50 km/h road with 400 vpd can be estimated by comparing differences between a frequency of one injury-producing crash per km every 23 years (for 0 m) versus one every 98 years (for 3.0 m). This difference roughly computes to *one additional crash* expected every 35 years per km.

Stated differently, for a given set of conditions as defined by Tables VI-1 and VI-3, the expected difference in safety performance as predicted by the AASHTO ROADSIDE program between a road with a 3.0 m clear zone and a 0 m clear zone represents a "savings" of one crash per km in 35 years for a LVLRoad with 400 vpd.

From Table VI-3, for "open alignment," note that many of the entries are shown as ">150 years." Only for the higher volume ranges are risk measures of less than 100 years evident. Table VI-3 suggests that , under the alignment conditions modeled, a clear zone policy of 3.0 m does not produce a sufficient marginal safety benefit within the parameters established for this study. Indeed, even for the highest speed and volume conditions, the marginal risk of one additional severe crash per km every 30 years is at least twice the threshold values established in Chapter III.

For more severe alignment, the results are somewhat different. Table VI-4 shows a marginal safety performance analysis for the data described in Table VI-2. For lower speed and lower volume conditions, the change in risk associated with variable clear zone dimensions is generally less than one crash per km per 100 years. For the greatest difference, a 3.0 m versus 0 m clear zone policy, the difference in safety performance reaches the limits of marginal cost effectiveness as defined in Chapter III for volume ranges approaching 400 vehicles per day. For most other cases, however, the risk thresholds previously established suggest that, even with severe alignment, the risks of accepting a lesser clear zone dimension compared to AASHTO's 3.0 m are well within the thresholds established for this study.

Table VI-4 suggests that a reasonable clear zone dimension for safety for higher volume LVLRoads with severe alignment might be 1.0 m. From Table VI-4, one can estimate that the safety benefit of a 1.0 versus 0 m clear zone is one crash per km every 12 years for 50 km/h; and one every 20 years for 110 km/h. These calculations suggest a marginal safety-effectiveness for such conditions. Dimensions greater than 1.0 m, however, do not return benefits even close to the safety thresholds.

Effectiveness of Guardrail Versus Roadside Objects

Analyses similar to the above were also performed to investigate the expected safety performance of guardrail versus roadside objects. Eight cases are presented below in Tables VI-5 through VI-8 for lower speed roads, and Tables VI-9 through VI-12 for higher speed roads. The cases represent ranges in highway alignment, from "open" (with no grade and tangent alignment) to "moderate" (with varying combinations of grade and curvature) to "severe" (with 250 m radius curve and 6 percent grade).

ROADSIDE analyses were used to determine the expected frequency of total and injury-producing crashes per km for the range of ADT and two roadside cases - continuous guardrail, and 0.3 m diameter trees continuously along the roadside. The analyses essentially compare the difference between the safety performance of two types of roadsides with severity indices reflective of speed and the type of obstacle.

The tables show expected years to experience one injury-producing crash for the same set of conditions for each roadside design. The difference in crash experience, labeled "delta risk" in each table, reflects the benefits of the lower severity design (guardrail) versus an unprotected roadside.

Table VI-5
Safety Risk Assessment of Guardrail Versus Roadside Objects for LVLRoads – 50 km/h Roads
['Open' Alignment – 0% Grade, Tangent]

Average Daily Traffic	Total Years per Impact per km	Years per Impact Causing Injury per km for Guardrail (SI = 1.9)	Trees (SI = 3.2)	Δ Risk
100	54	201	93	~ 110 years
250	22	80	67	~ 75 years
400	14	50	23	~ 50 years

Table VI-6
Safety Risk Assessment of Guardrail Versus Roadside Objects for LVLRoads – 50 km/h Roads
['Moderate' Alignment – 0% Grade, Radius of Curve = 400 m]

Average Daily Traffic	Total Years per Impact per km	Years per Impact Causing Injury per km for Guardrail (SI = 1.9)	Trees (SI = 3.2)	Δ Risk
100	33	120	56	~ 110 years
250	13	48	22	~ 45 years
400	8	30	14	~ 25 years

Table VI-7
Safety Risk Assessment of Guardrail Versus Roadside Objects for LVLRoads – 50 km/h Roads
['Moderate' Alignment – 5% Grade, Radius of Curve = 1700 m]

Average Daily Traffic	Total Years per Impact per km	Years per Impact Causing Injury per km for Guardrail (SI = 1.9)	Trees (SI = 3.2)	Δ Risk
100	34	124	58	110 years
250	13	50	23	50 years
400	8	31	14	~ 25 years

Table VI-8
Safety Risk Assessment of Guardrail Versus Roadside Objects for LVLRoads – 50 km/h Roads
['Moderate' Alignment – 6% Grade, Radius of Curve = 250 m]

Average Daily Traffic	Total Years per Impact per km	Years per Impact Causing Injury per km for Guardrail (SI = 1.9)	Trees (SI = 3.2)	Δ Risk
100	15	55	26	~ 50 years
250	6	22	10	~ 18 years
400	4	14	6	~ 11 years

Table VI-9
Safety Risk Assessment of Guardrail Versus Roadside Objects for LVLRoads – 110 km/h Roads
['Open' Alignment – 0% Grade, Tangent]

Average Daily Traffic	Total Years per Impact per km	Years per Impact Causing Injury per km for Guardrail (SI = 3.0)	Trees (SI = 5.0)	Δ Risk
100	43	76	55	> 50 years
250	17	30	22	90 years
400	11	19	14	48 years

Table VI-10
Safety Risk Assessment of Guardrail Versus Roadside Objects for LVLRoads – 110 km/h Roads
['Moderate' Alignment – 0% Grade, Radius of Curve = 400 m]

Average Daily Traffic	Total Years per Impact per km	Years per Impact Causing Injury per km for Guardrail (SI = 3.0)	Trees (SI = 5.0)	Δ Risk
100	26	46	33	125 years
250	10	18	13	45 years
400	6	11	8	29 years

Table VI-11
Safety Risk Assessment of Guardrail Versus Roadside Objects for LVLRoads – 110 km/h Roads

['Moderate' Alignment – 5% Grade, Radius of Curve = 1700 m]

Average Daily Traffic	Total Years per Impact per km	Years per Impact Causing Injury per km for Guardrail (SI = 3.0)	Trees (SI = 5.0)	Δ Risk
100	29	51	37	150 years
250	11	20	15	60 years
400	7	13	9	29 years

Table VI-12
Safety Risk Assessment of Guardrail Versus Roadside Objects for LVLRoads – 110 km/h Roads

['Moderate' Alignment – 6% Grade, Radius of Curve = 250 m]

Average Daily Traffic	Total Years per Impact per km	Years per Impact Causing Injury per km for Guardrail (SI = 3.0)	Trees (SI = 5.0)	Δ Risk
100	13	23	17	69 years
250	5	9	7	32 years
400	3	6	4	12 years

For lower speed roads, the use of guardrail produces very small incremental safety benefits for open and moderate alignment. Even at 400 vpd, the additional risk of not providing guardrail is only one additional crash per km over 25 years. For more severe alignment, the risk is greater—approaching the threshold of cost-effectiveness of 6 to 10 years for higher volume roads.

For higher speed roads, the results are somewhat similar. The added benefits of lower severity due to guardrail versus trees are minimal for open and moderate alignment. Even for the higher volume ranges within LVLRoad criteria, the "risk" of trees versus guardrail is only one more crash over 29 years. For severe alignment and 400 vpd, the risk is one more crash in 12 years, at the margin of the safety threshold.

The guardrail analyses modeled continuous trees as the object to be shielded. Very steep embankments, open bodies of water, etc. would represent greater risks. The severity indices for these as defined in ROADSIDE can be as great as 10. Risk assessment of these roadside objects would result in somewhat different results.

The results should be viewed with caution. They model continuous guardrail, and do not reflect the added risk associated with end treatments. Note, however, that they appear reasonably consistent with published literature on guardrail warrants wherein guardrail appears to begin justification at volume levels of 300 to 500 vpd.

Other Factors in Roadside Design

Providing additional roadside dimension may be desirable for a number of reasons separate from safety. Continuous shoulder or offset dimensions are used to provide additional structural support to the roadway surface. This may be particularly important on narrow and/or unpaved roads. Clearing the inside of sharp curves to provide minimum stopping sight or maneuver sight distance is also a roadside improvement. In urban areas, additional roadside dimension may be desired for utility placement, provision for sidewalks or storage of snow.

The relative lack of meaningful safety benefits associated with the concept of a clear zone should not in any way dissuade designers from providing additional dimension for the above or other reasons. Rather, the intent of roadside design policy for LVLRoads should be to provide flexibility to the designer to provide clear roadsides as they see fit.

Guidelines for Roadside Design Policy on LVLRoads

The literature and risk assessments support the view that LVLRoad design policy should be as flexible as possible. The roadside represents the greatest determinant of the safety of LVLRoads, for no other reason than multi-vehicle conflicts are rare. This does not mean, however, that safety can be optimized by establishing minimum design guidelines, guardrail warrants etc. Both the safety literature and risk assessments highlight the fact that run-off-road crashes on roads with very low volumes occur so infrequently as to make any minimum clear zone dimensions demonstrably not cost-effective. In many cases, an additional clear zone dimension results in the need to acquire more right-of-way, and to increase the cost of construction. Both factors are serious problems for designers working in the LVLRoad environment.

Interestingly, the analyses in this chapter, combined with work in other chapters of this report, do suggest ways for design engineers to be "safety cognizant" in their work. Roadside encroachments and potential safety problems are sensitive to the quality of the alignment. The "risk" of crashes is also greater for certain combinations of horizontal curve and speed (see Chapter VII). The following guidelines are suggested:

1. For LVLRoads, there is no minimum standard offset or clear zone dimension that can be characterized as "cost-effective." A 0 m clear zone may be typically acceptable for most types of LVLRoads.
2. Notwithstanding point 1, any clear zone dimension will produce some (albeit very small) safety benefit. Designers should not be discouraged from providing minimal clear zone dimensions of 1 m, 2 m or more when such designs can be readily provided at little or no additional cost. In short, the site-specific conditions and judgment of the designer should dictate clear-zone design for LVLRoads.
3. Designers should be encouraged to tailor the roadside design to specific conditions. Additional clear zone dimensions should be investigated and used in conjunction with sharp horizontal curves. (The information in Chapter VII on horizontal curves related to speeds can be used to identify "higher risk" horizontal curves.) Additional offsets within and along the outside of sharp curves will be more effective than standard or typical clear zone dimensions applied continuously. Where sufficient right-of-way

exists and a clear zone can be provided at a nominal, acceptable cost, designers should be encouraged to provide it.

4. The use of guardrail to shield or protect drivers from proximate hazards does not appear cost-effective for most LVLRoads. Guardrail itself is a hazard, and a significant proportion of impacts with it will produce injuries. The costs to maintain guardrail and the great infrequency of guardrail collisions suggest it is impractical for use when traffic volumes are very low.

5. Notwithstanding point 4 above, designers should use their judgment in deciding to place guardrails at locations of extreme risk. These may include very steep embankments, adjacent to open water, or other locations where the consequences of a run-off-road encroachment will always be a fatality. Design policy should not necessarily mandate the use of guardrail in such cases, but should encourage the designer to study the specific site and conditions.

The above guidelines appear appropriate for the range of functional classes of LVLRoads.

VII. Horizontal Alignment

Design of the horizontal alignment of a road is among the most fundamental geometric considerations. Horizontal alignment consists of tangents and circular curves. Some states and agencies use spiral transition curves between tangents and circular curves on high-speed roads. Such practice, however, is generally limited to arterial and other high type facility, and is not considered relevant to LVLRoads.

Horizontal alignment design considers the selection of appropriate circular curvature based on the design speed of the highway and other controls established from research or experience. Design of curvature also includes the use of superelevation, which is intended to counteract the acceleration felt by a driver traversing a curve. Maximum superelevation is determined by the policy of the design agency. This typically varies from 0.04 m/m to 0.12 m/m, with most agencies designing roads to a policy of 0.06 or 0.08.

AASHTO considers two design conditions-high speed and low speed design. The former is intended for use in primarily rural areas. Low speed design is intended for use at intersections and urban streets.

The AASHTO design model for horizontal curvature is expressed as follows:

$$e + f = V^2 / 127\,R$$

where:

e = superelevation (m/m)

f = side friction factor (design values established by AASHTO)

V = Design speed (km/h)

R = Curve Radius (m)

Minimum radius curves for a given design speed are designed with the maximum superelevation per agency policy. These are referred to as "controlling curves." Designers normally design curves milder than the maximum for a given design speed. These curves are designed with lesser superelevation values.

Historic Review of AASHTO Curve Design Policy

Fundamental AASHTO curve design policy has remained unchanged for over 50 years. Discussions in various versions of the policy note the typical behavior of drivers encountering curves. Also discussed are limitations on the performance of curve tracking, (i.e., the available friction produced at the tire-pavement interface). Referring to the 1954 AASHO *Policy on Geometric Design of Rural Highways (4)*, it is noted that

> *"...one criterion is the point at which the centrifugal force causes the driver to recognize a feeling of discomfort and instinctively acts - barring recklessness—to avoid higher speed. The speed at which discomfort due to the centrifugal force is evident to the driver can be accepted as a control on the amount of side friction for use in design."*
> *(p. 130)*

The AASHTO model for horizontal curve design developed in the 1940's and continued in use today is based on the following operational assumptions:

- The vehicle is represented as a point mass, with no differentiation between vehicle type (passenger car, truck, RV, etc.).
- The driver/vehicle system tracks a properly designed curve at the design speed of the highway.
- The driver/vehicle system tracks perfectly the alignment of the highway. In other words, the assumed or inferred dynamic forces are determined based directly on the radius of the curve and the speed.

The following are important features of the AASHTO design model and procedures:

- Minimum radius curvature is based on consideration of driver comfort. Side friction forces that result from the procedures fall well short of those that translate to loss of control. Figure VII-1 (p.122) from the Policy depicts the results of research on pavement/tire friction which have historically been referenced in AASHTO.
- There is no sensitivity of curve design policy for type of highway. Local roads are designed to the same set of values and assumptions as collectors, arterials and freeways. Multi-lane highways are designed the same as two-lane highways. Any differences in a design are reflected in the selection of a design speed.
- There is no sensitivity of curve design policy to traffic volume. A road with expected ADT of 250 vehicles per day is designed identically to one with 25,000 vehicles per day (for the same design speed and superelevation policy).
- There is no design interaction with other geometric features, including vertical alignment, or cross-section elements such as lane and shoulder width, or the roadside.

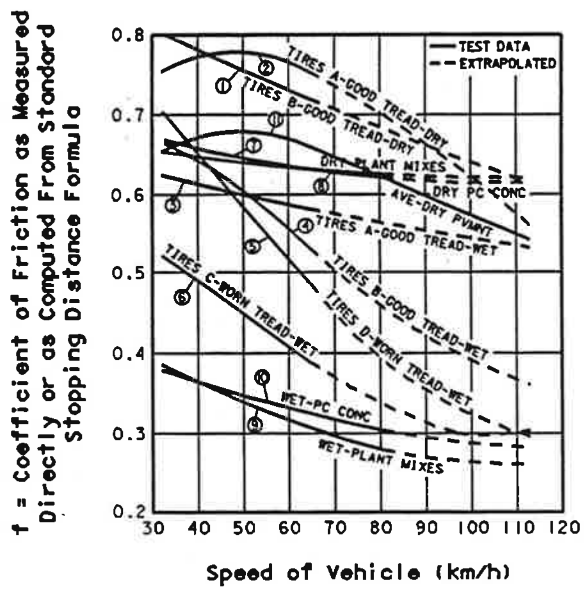
When the AASHTO/AASHO design model was first formulated, little was known about the incremental safety or overall safety implications of design. Although historic versions of the policy refer to "safe" design and selection of safe design speeds, there has been no direct relationship between safety performance and curve design policy.

Review of Literature on Horizontal Alignment

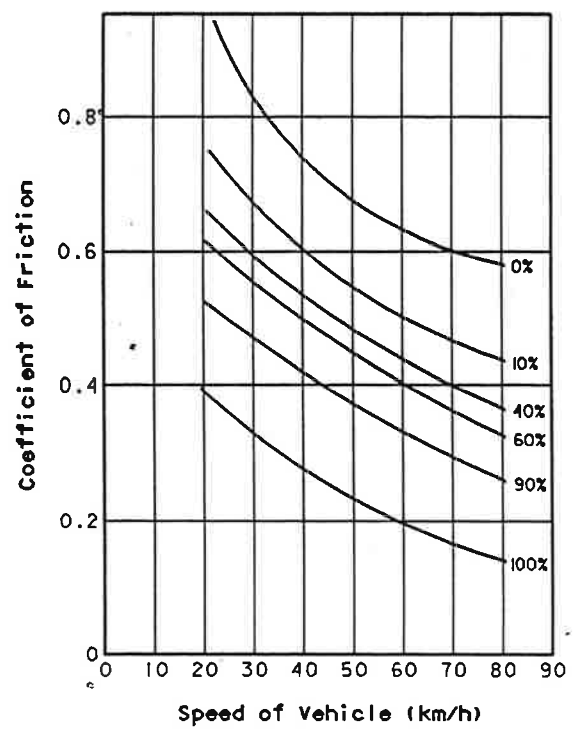
There have been a number of major research efforts on the safety of horizontal curves on 2-lane rural highways in recent years. Chapter II addressed much of the research on highway curve safety and operations. Although the studies focused on higher class roads with higher traffic volumes, many of the findings and conclusions are applicable to very low volume roads. A summary of key research relevant to the study is repeated here.

Curves have long been recognized as potential problem locations in terms of safety and crashes. NCHRP Report 374 summarized the best recent work on horizontal alignment related to safety. This includes studies for FHWA by Glennon et al. (20) and by Zegeer, et al (69). Both of these studies produced accident prediction models for isolated horizontal curves that include degree and length of curve, total roadway width, and presence of spiral transitions.

Figure VII-1
Variation in Coefficient with Vehicular Speed



A. Skid resistance for various tire and pavement conditions.



B. Skid Resistance of Pavement - Percent of Pavements Which Exceed Coefficient of Friction.

Glennon, et al Horizontal Curve Model

A major research effort undertaken by FHWA in the 1980's produced a substantial database of horizontal curves, their geometry and related crashes. From this work was derived a simple crash prediction model shown below:

$$A = Ar_s (L) (V) + 0.0336 (D) (V) \text{ for } L \geq L_c$$

where:

A = Total number of accidents on the segment

Ar_s = Accident rate on comparable straight segments in accidents per million vehicle miles

L = Length of highway segment in miles

V = Traffic volume in millions of vehicles

D = Curvature in degrees

L_c = Length of curved component in miles

Glennon's work was the first to establish the relationship between not only degree or sharpness of curvature and crashes, but also the length of curve or central angle of the curve. The Glennon model used accident rate as the predictive variable. Most uses of this model entail prediction of annual crashes.

Other work by Glennon, et al. on "high accident" curve sites suggests that the roadside and pavement friction in the curve were also factors relating to safety.

Zegeer, et al Horizontal Curve Model

Subsequent to Glennon's work, additional research was conducted for FHWA by Zegeer, et al, on horizontal curve safety. The following model was developed from that research:

$$A = [(1.552)(L)(V) + (.014)(D)(V) - (.012)(S)(V)]\,(.978)^{W-30}$$

where

A = Number of total accidents on the curve in a 5-year period

L = Length of the curve, in miles

V = Volume of vehicles in million vehicles in a 5-year period passing through the curve (both directions)

D = Degree of Curve

S = Presence of spiral

- S = 0 if no spiral exists
- S = 1 if there is a spiral

W = Width of the roadway (twice the lane width plus shoulder width) on the curve, in feet

Zegeer's model is similar in some respects to Glennon's in terms of identifying crashes as a function of curve geometric elements.

Both models were derived from databases that included traffic volumes greater than those on LVLRoads. Both models reflect operations on higher speed, rural roads. Glennon's and Zegeer's work both highlight the importance of the roadside with respect to curve safety. Note also that Glennon (and others before him) document an increasing proportion of single-vehicle run-off-road accidents as traffic volume decreases. This strongly suggests that the importance of the roadside is even greater on very low volume road curves.

Other Research on Curves

A recent synthesis of research on horizontal alignment and safety concluded that a wide range of factors has been shown to affect safety:

- Traffic Volume and Composition
- Curve Geometry
- Roadside Hazard on the Curve
- Cross-section on the Curve
- Stopping Sight Distance on the Curve and Approaches to the Curve
- Vertical Alignment on the Curve
- Proximity to Adjacent Curves
- Proximity to Other Critical Features such as Bridges, Intersections, Driveways, etc.
- Pavement Friction
- Signing and Marking

TRB Special Report 214, *Designing Safer Roads, Practices for Resurfacing, Restoration and Rehabilitation* (14), documented an analysis of reconstructing curves to treat existing safety problems. The report applied a model derived from the data by Glennon, et al. Significant conclusions relative to LVLRoads are:

- Flattening of too-sharp curves is generally not cost-effective at traffic volumes less than 750 vpd;

- Curve flattening may begin to be viable when the actual operating speeds on the curve are at least 15 mph (24 km/h) greater than the nominal or inferred design speed of the curve.

Low volume local road safety is distinctly different from higher volume types. As has been noted by Glennon in NCHRP Report 214, and repeated by others, for very low volume roads, the chances of head-on or other multi-vehicle encounters are minimal. Single vehicle crashes predominate.

A number of studies of very low volume roads have looked at alternative measures of horizontal alignment as input to safety index models. Ivey and Griffin (31) developed a safety index for low volume rural roads that includes a measure combining the effects of horizontal and vertical alignment. It is interesting that roadside and cross-section descriptors tend to dominate the safety index compared to the relative contribution of horizontal alignment.

Low Speed and Urban Roads

There is little evidence of safety sensitivities to curvature on low speed roads or streets, at any traffic volume level. Other factors that influence safety on such roads (typically in urban areas) tend to overwhelm or dominate crash causation. These include intersection location and design, access control, and median treatments for multi-lane roads.

Operational Research Related to Horizontal Alignment

It has been recognized that the actual performance of the driver/vehicle system may significantly differ from that assumed in the AASHTO Policy. Much research in recent years establishes a framework around which conclusions can be drawn about the operational characteristics of curvature.

Speed and Speed Relationships to Curvature

A number of other recently completed studies offer insights to safety and operations on lower speed curves such as might be expected on very low volume local roads. Anderson and Krammes (6) looked at speed reduction effects of horizontal curves. Their database included some very low volume roads, but was limited to collector and arterials. Their study suggested a strong relationship between speed in a curve and curvature. This is consistent with work by Lamm (35) and others. A number of models were developed that related speed to curvature and accident to speed reductions.

The Krammes research resulted in a model that predicts the 85th percentile speed on a curve as a function of the degree of curve, as outlined below:

$$V85 = 64.4 - 1.21\ DC$$

where:

V85 = The 85th percentile speed on the curve (mph)

DC = The degree of curve (English system)

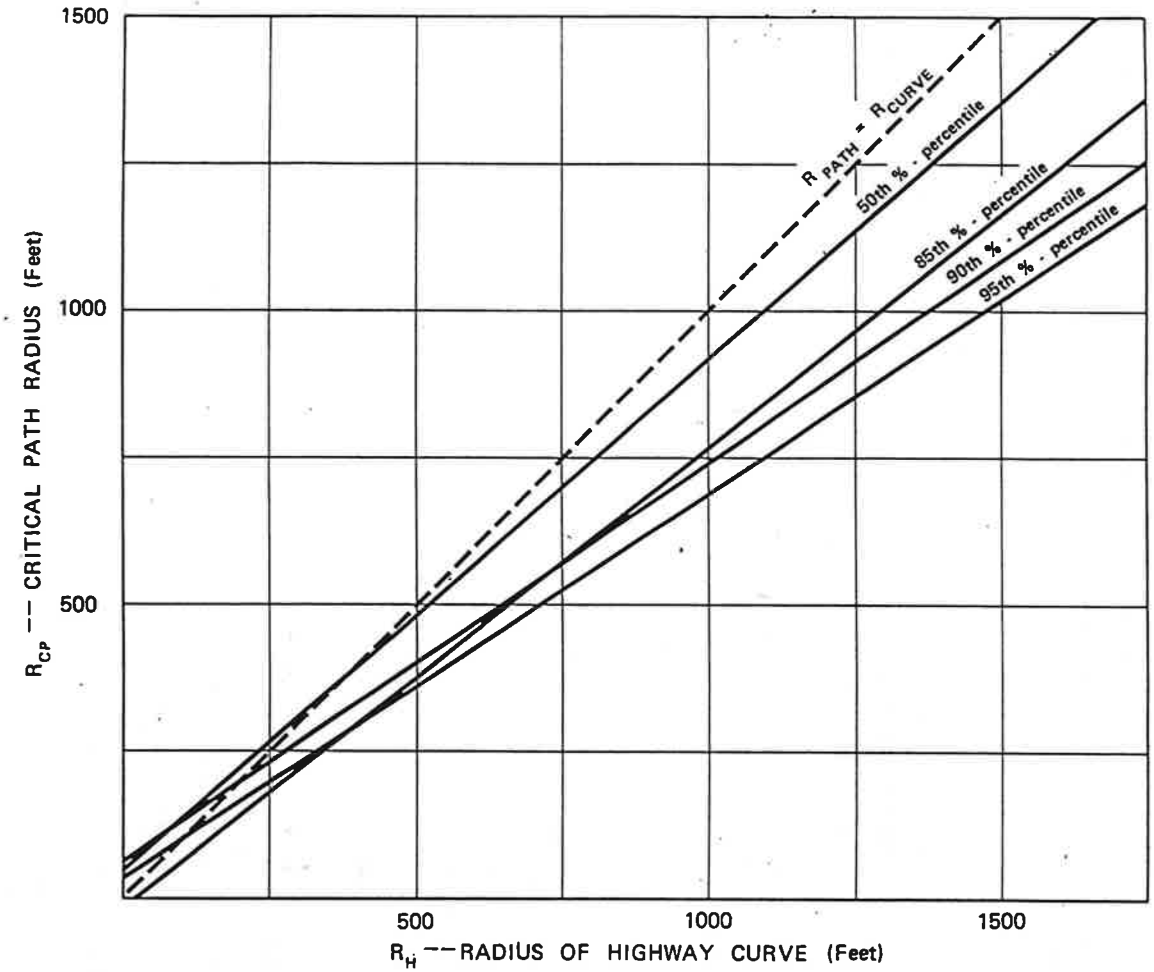
This model is consistent with other observations of speed behavior. Drivers tend to drive faster on low speed curves than the AASHTO design model suggests.

Harwood and Mason (25) analyzed AASHTO criteria for high and low speed curve design. They looked at operations of both passenger cars and trucks. For passenger car operations, skidding loss of control is more likely. They concluded that rollover or loss of control skid may occur for critical trucks if their speed only slightly exceeds the design speed of a low-speed road. For example, on curves with a 32 km/h effective design speed, the most unstable trucks could roll over if their speed is as little as 1.6 km/h above the nominal design speed.

Path/Vehicle Tracking Research

Dynamic forces on the driver and vehicle are a function of the vehicle path as well as its speed. Research completed in the past 15 years confirms that AASHTO assumptions about path behavior do not accurately reflect actual vehicle operations on curves. Glennon, et al. (20) completed studies of vehicle paths on two-lane rural highways. The studies noted the actual paths drivers tracked on unspiraled curves at speeds from 40 to 60 mph. The findings of the research are summarized on Figure VII-2. It was found that drivers track a significantly sharper path radius than the curve radius at some point during entry to the

Figure VII-2
Relationship Between Highway Curve Radius and Percentiles of Critical Path Radius



Source: Glennon, J.L., Neuman, T.R. and Leisch, J.E. "Safety and Operational Considerations for Design of Rural Highway Curves". FHWA Report. Washington, D.C., 1985.

curve. Moreover, the severity of this path was independent of the driver's speed. A 95th percentile driver path radius is as much as 30 percent smaller than the actual curve radius for controlling curve designs of 50 mph (about 80 km/h).

Design Speed and Design Consistency

Design consistency is a concern that has received increasing interest in recent years. Current AASHTO Policy for geometric features does not generally address issues of consistency with respect to the three dimensional features of the road. Consistency refers to the quality of the road, the message communicated to the driver, and how such message affects driver operations (primarily speed). Oglesby (46) has observed that design consistency is very important on low volume roads.

Much work has been done in recent years to develop measures of design consistency and translate them to geometric criteria. Lamm, et al. (35), in a number of research efforts have developed guidelines for speed and speed changes produced by alignment for continuous segments of highway. Leisch and Leisch (36) presented a theoretical model for speed consistency 20 years ago that has recently been tested with other European methods for speed consistency. Both the Lamm and Leisch studies identified speed differentials of 10 to 20 km/h as representing thresholds of desirable speed change behavior associated with alignment.

A number of authors (6, 35, 68) have developed speed change models or speed prediction models for use in evaluating design consistency. Such models typically relate horizontal curvature to speeds, although grades, pavement condition and roadway width have also been shown to affect speeds.

Indeed, NCHRP Report 362 referred to roadway width and its expected effect of vehicle speeds in development of design criteria for width.

Wooldridge (68) reports on evaluation of a different driver model developed for two-lane rural roads. This model incorporates concepts of driver workload and expectancies with respect to intersections, lane drops, bridges, alignment, and width reductions.

To summarize previous work, there is a consensus that horizontal alignment that produces speed reductions of greater than 20 km/h is considered undesirable. Reductions of 10 to 20 km/h may be marginal, depending on other conditions. Alignments that produce speed reductions of 10 km/h or less are considered "consistent."

Summary of Background on Horizontal Alignment

Within the context of design for low volume local roads, the following are key points with respect to horizontal curve design:

1. Current AASHTO design criteria are based on a hypothetical *operational* model. The model was developed independent of quantifiable estimates of safety or relative risk. The intent of the model is to produce a design wherein drivers under almost all circumstances will be able to comfortably track the alignment of the road without skidding or other loss of control. The model is insensitive to both highway type and traffic volume levels.

2. Selection of design values for 'f', which highly influences the derivation of minimum radius curvature, is based on providing a level of comfort and convenience to the driver. This is a critical point to understand regarding curve design for LVLRoads. The AASHTO Policy shows values for 'f' as a function of design speed, and discusses distributions of pavement friction. Yet, the basis for establishing 'f' is research on the *comfort* of drivers tracking the curve. Indeed, reference to data on actual pavement/tire friction suggests the criteria produce a substantial margin of safety against loss of control for passenger cars on most surfaces at high speeds.

3. Operational research over the past 20 years has resulted in many questions about the adequacy or accuracy of the simple AASHTO curve model. Drivers do not track the curve as designed, but rather effect a spiral path with an actual vehicular radius that is sharper than the designed radius. Drivers tend to drive faster through curves than the AASHTO comfort model suggests. Many researchers have shown that speed behavior is highly variable approaching or through curves.

4. Safety research on curves has also provided insights. Models have been developed that enable estimation of the relative safety benefits of different curve designs. The best models (see Zegeer's model for two-lane rural highways, for example) highlight the importance of other geometric elements in curve safety, such as the quality of the roadside, cross-section widths, and pavement friction. Of course, these models were derived on higher speed highways, with traffic volumes much greater than those considered for this project. Yet, the fundamental geometric relationships involving the interaction of the curve, roadside, and speed should apply to lower volume roads.

The above points represent a "mixed bag" regarding the applicability of current AASHTO design policy for LVLRoads. Designing for comfort, to the extent it results in constrained criteria, may be overly conservative and hence inappropriate for LVLRoads. There may be opportunities to design for sharper curves, using a different design philosophy than comfort. Conversely, the fact that observed driver behavior differs from that assumed by AASHTO, argues for caution in developing significant revisions to the curve design procedures. Also, under certain circumstances (i.e., truck operations at low speeds) current design policy offers little margin of safety. Finally, it is clear from safety research that curves continue to pose special safety problems, although these findings tend to reflect operation at traffic volume levels well above 400 vehicles per day.

Risk Assessment for Horizontal Curves

The potential for less conservative alignment design was investigated through a risk assessment based on operational and safety research discussed above. Two separate sets of analyses were conducted. An *operational* risk assessment approach was based on an alternative model for curve operations. *A safety* risk assessment investigated the potential additional risk of accepting smaller radius curve designs for LVLRoads.

Operational Risk Assessment

An alternative functional model is proposed for consideration and development for horizontal curve design on LVLRoads. Rather than designing for comfort, design for *margin of safety* from loss of control was tested. This approach is consistent with a philosophy for such roads which emphasizes functionality, safety and minimal cost. It is also consistent with design guidelines previously used by the US Forest Service, described in Chapter II.

A margin of safety design approach would use design values for f reflective of distributions of pavement friction rather than driver comfort. An appropriate margin of safety approach should also more closely reflect actual curve operations compared to the simple AASHTO operational model assumptions.

The researchers developed an alternative design model to test the viability and operational sensitivity of such a design approach. The following analyses were performed:

- Hypothetical values for side friction demand were calculated for a range of design speeds and maximum superelevation policies. The values were calculated based on various combinations of speed and path behavior from research cited above. For path behavior, a full range of path assumptions from the work by Glennon et al. was used. For speed behavior, the predicted 85th percentile speed from the research by Krammes was used, as well as the nominal design speed.

- Resultant values for 'f' demand were determined from values for actual pavement/tire friction supply taken from the research by Harwood and Mason, as shown in Figure VII-3. A margin of safety of 0.2 between friction supply and demand is established as a target or objective. This margin is consistent with that used by the US Forest Service design procedures.

- Design values for curve radius were computed using the various speed and path radius assumptions noted above, and the friction supply values based on margin of safety. Design values were produced for a range of design speeds and maximum superelevation policies of 0.04, 0.06, 0.08, and 0.10.

- Curve design radius values were compared to AASHTO design radius values to note differences and to study sensitivities relative to speeds and superelevation policy.

The results of the analyses are indicated in Tables VII-1 through VII-4. The combination of different operational assumptions based on actual versus AASHTO model assumptions produces interesting findings.

Analysis of Speed Effects

Table VII-1a through VIII-1d depict the results of an analysis based on driver speeds as modeled by the Krammes speed model, with a vehicle path assumed to be equal to the roadway path. The Krammes speed prediction model generates higher 85th percentile speeds for minimum curve radii for a lower AASHTO design speeds. Even with this input, the minimum radius path based on a margin of safety approach is less than given by AASHTO. The differences are greatest for the higher speeds. This is because the speed prediction model produces lower 85th percentile speeds for the higher design speed curves. (Care should be taken in interpreting the findings as meaningful for very low speed curves, given the nature and limitations of the speed model. In any event, lower design speed roads are often designed to the "low speed" friction curve in AASHTO.)

Figure VII-3
Design Values for 'f' Based on Margin of Safety

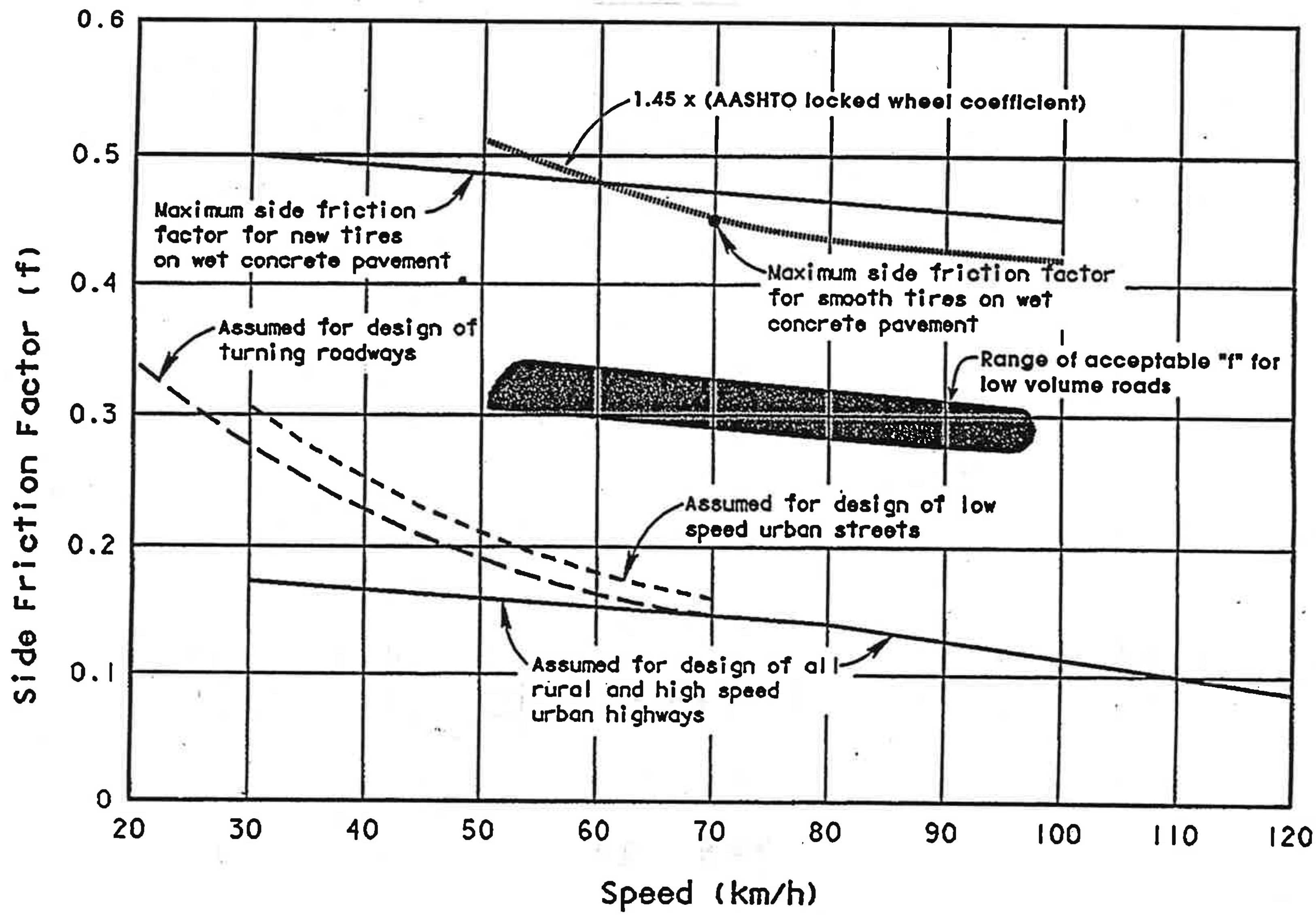


Table VII-2 shows that much smaller curve radii for higher nominal design speeds can be accepted for a margin of safety of 0.2. Indeed, compared with current AASHTO curve design policy, values for R_{design} represent an equivalent speed reduction of up to 18 km/h, with the higher speed reductions associated with higher design speeds. (For example, the margin of safety model for e_{max} of 0.10 and a design speed of 90 km/h suggest an acceptable radius of 206 m. Under current AASHTO Policy, a 206 m radius curve is equivalent to a 79 km/h curve. Thus, the margin of safety model represents an effective 11 km/h speed reduction.) Stated differently, accepting this alternative design model is equivalent to accepting a nominal lower design speed reduction of up to 18 km/h. Note that this effective speed reduction is consistent across the range of maximum superelevation policies.

Analysis of Path Effects

Tables VII-3a through VIII-3d illustrate the margin of safety approach for different vehicle path assumptions, as given by the relationships published by Glennon, et al. For this set of analyses the assumed speed is the design speed. The results are similar, but not identical to the speed model results. Smaller radius curves are acceptable for the full range of design speeds and maximum superelevation policies. Depending on which path model is considered acceptable, the differences amount to an equivalent design speed reduction of 3 to 10 km/h compared to current AASHTO curve design policy. Table VIII-4 provides one comparison using 85^{th} percentile path behavior and e_{max} policy of 0.10.

Risk Assessment – Probability of Loss of Control

The validity or usefulness of the alternative margin of safety approach requires careful consideration of available pavement friction. This includes both surface type and climate. For LVLRoads, pavement surface condition is an important variable. Many such roads are unpaved or in relatively poor condition. Washboarding, poor surface drainage and other problems may be prevalent, and should not be ignored in making design decisions.

The probability that a surface is wet or icy is highly variable. In certain parts of the country this may be less than 5 percent. In colder climes, for LVLRoads, the surface may be icy during much of the winter.

It is beyond the capabilities of this study to provide a complete probability analysis of an alternative operational model for curve design. A general illustration, however, indicates such a policy is reasonably conservative for the low traffic volume conditions or concern.

Consider the probability of an individual vehicle losing control during curve tracking as a function of four events - the speed, the path, the available pavement friction, and the condition of the pavement (wet or icy). A simplifying assumption (undoubtedly conservative) that the four are independent can be made. The probability of a curve tracking failure, producing skidding and potential loss of control, is given by the product of the four events. Assume an 85^{th} percentile driver speed, and 50^{th} percentile driver path. Available pavement friction is about 95^{th} percentile (only 5 percent are worse for the design values used), and the probability the pavement is wet is 10 percent.

$$P\text{ (loss of control)} = 0.85 \times 0.5 \times .05 \times 0.1 = 0.002125$$

A controlling curve with 100 vehicles per day will have 365 x 100 curve traversals per year, or 36,500. The total annual "risk" is thus 36,500 x 0.002125, or 78.

Table VII-1a
Derivation of Design Radii for Horizontal Curves Using 'Margin of Safety' Criterion and Modeled 85th Percentile and Speeds
($e_{(max)} = 0.04$)

V_{Design} (km/h)
50
60
70
80
90
100

English Units					
V_{Design} (mph)	V_{85} [1] (mph)	R_{AASHTO} (ft)	$f_{ms (V\ 85)}$ [2] f_{max} - 0.2; at V_{85}	R_{Design} (ft)
31	43.3	328	0.250	430
37	50.3	492	0.225	637
43	54.6	705	0.221	761
50	55.3	764	0.219	788
56	58.8	1230	0.217	896
62	60.1	1608	0.214	948

Metric Units				
V_{Design} (km/h)	V_{85} (km/h)	R_{AASHTO} (m)	$f_{ms (V\ 85)}$ f_{max} - 0.2; at V_{85}	R_{Design} (m)
50	69.6	100	0.250	131
60	81.0	150	0.225	194
70	87.8	215	0.221	232
80	89.0	280	0.219	240
90	94.6	375	0.217	273
100	96.7	490	0.214	289

1. V_{85} as given by Krammes Speed Model

$$V_{85} = 64.4 - 1.21\,(5730/R_{AASHTO}) \quad ; R\ (ft)$$

$$R_{Design} = \frac{(V_{85})^{\wedge 2}}{15(e + f)} \quad ; R\ (ft)$$

2. $F_{design} = f_{max} - 0.2$; per Figure X-X

Table VII-1b

Derivation of Design Radii for Horizontal Curves Using 'Margin of Safety' Criterion and Modeled
85th Percentile and Speeds
($e_{(max)} = 0.06$)

V_{Design} (km/h)	V_{Design} (mph)	V_{85} [1] (mph)	English Units R_{AASHTO} (ft)	$f_{ms (V 85)}$ [2] f_{max} - 0.2; at V_{85}	R_{Design} (ft)
50	31	40.9	295	0.258	351
60	37	48.7	443	0.228	550
70	43	53.6	640	0.222	678
80	50	55.3	764	0.219	731
90	56	58.1	1099	0.217	812
100	62	59.5	1427	0.215	859

V_{Design} (km/h)	V_{85} (km/h)	Metric Units R_{AASHTO} (m)	$f_{ms (V 85)}$ f_{max} - 0.2; at V_{85}	R_{Design} (m)
50	65.9	90	0.258	107
60	78.4	135	0.228	168
70	86.2	195	0.222	207
80	89.0	250	0.219	223
90	93.5	335	0.217	248
100	95.8	435	0.215	262

1. V_{85} as given by Krammes Speed Model

$$V_{85} = 64.4 - 1.21\,(5730/R_{AASHTO}) \quad ;\ R\ (ft)$$

$$R_{Design} = \frac{(V_{85})^{\wedge 2}}{15(e + f)} \quad ;\ R\ (ft)$$

2. $F_{design} = f_{max} - 0.2$; per Figure X-X

Table VII-1c
Derivation of Design Radii for Horizontal Curves Using 'Margin of Safety' Criterion and Modeled
85th Percentile and Speeds
($e_{(max)} = 0.08$)

V_{Design} (km/h)	V_{Design} (mph)	V_{85} [1] (mph)	R_{AASHTO} (ft)	$f_{ms (V 85)}$ [2] f_{max} - 0.2; at V_{85}	R_{Design} (ft)
			English Units		
50	31	38.0	262	0.270	275
60	37	47.5	410	0.237	474
70	43	52.3	574	0.222	604
80	50	55.3	764	0.220	680
90	56	57.5	1001	0.218	739
100	62	59.0	1296	0.216	785

V_{Design} (km/h)	V_{85} (km/h)	R_{AASHTO} (m)	f_{max} - 0.2; at V_{85}	R_{Design} (m)
		Metric Units		
50	61.1	80	0.270	84
60	76.4	125	0.237	145
70	84.2	175	0.222	184
80	89.0	230	0.220	207
90	92.5	305	0.218	225
100	95.0	395	0.216	239

[1]. V_{85} as given by Krammes Speed Model

$$V_{85} = 64.4 - 1.21\,(5730/R_{AASHTO}) \quad ; R\ (ft)$$

$$R_{Design} = \frac{(V_{85})^{\wedge 2}}{15(e+f)} \quad ; R\ (ft)$$

[2] $F_{design} = f_{max} - 0.2$; per Figure X-X

Table VII-1d
Derivation of Design Radii for Horizontal Curves Using 'Margin of Safety' Criterion and Modeled
85th Percentile and Speeds
($e_{(max)}$ = 0.10)

	English Units				
V_{Design} (km/h)	V_{Design} (mph)	V_{85}[1] (mph)	R_{AASHTO} (ft)	$f_{ms\ (V\ 85)}$[2] f_{max} - 0.2; at V_{85}	R_{Design} (ft)
50	31	36.2	246	0.280	230
60	37	46.0	377	0.239	417
70	43	51.2	525	0.224	539
80	50	55.3	764	0.220	638
90	56	56.7	902	0.218	674
100	62	58.5	1181	0.216	723

Metric Units				
V_{Design} (km/h)	V_{85} (km/h)	R_{AASHTO} (m)	$f_{ms\ (V\ 85)}$ f_{max} - 0.2; at V_{85}	R_{Design} (m)
50	58.3	75	0.280	70
60	74.1	115	0.239	127
70	82.4	160	0.224	164
80	89.0	210	0.220	194
90	91.3	275	0.218	206
100	94.2	360	0.216	220

[1]. V_{85} as given by Krammes Speed Model

$$V_{85} = 64.4 - 1.21\,(5730/R_{AASHTO}) \quad ; R\ (ft)$$

$$R_{Design} = \frac{(V_{85})^{\wedge 2}}{15(e+f)} \quad ; R\ (ft)$$

[2]. $F_{design} = f_{max} - 0.2$; per Figure X-X

Table VII-2
Comparison of AASHTO and 'Margin of Safety' Criterion Using Modeled Speed for Horizontal Curve

(e_{max} = 0.10)

Y_{Design} (km/h)	R_{AASHTO} (m)	R_{design}[1] (m)	Equivalent V_{AASHTO} for R_{Design}	ΔV (km/h)
50	75	70	49	---
60	115	127	63	---
70	160	164	71	---
80	210	194	77	3
90	275	206	79	11
100	360	220	82	18

[1] per Table VIII-1d for e_{max} = 0.10

Table VII-3a
Derivation of Design, Radii for Horizontal Curves Using 'Margin of Safety' Criterion and Modeled Path Behavior
($e_{(max)}$ = 0.04)

V_{Design} (km/h)				English Units				
	V_{Design} (mph)	R_{AASHTO} (ft)	$f_{ms\ (V\ Design)}$ f_{max} - 0.2; at V_{Des}	R_{Path} (ft)	R_{Design} for n^{th} %tile of Driver Population, (ft) 50th	85th	90th	95th
50	31	328	0.305	186.5	168	251	195	230
60	37	492	0.278	291.4	287	384	347	388
70	43	705	0.258	423.2	437	551	538	588
80	49.7	919	0.225	621.6	662	802	826	889
90	56	1230	0.218	808.1	874	1038	1096	1171
100	62	1608	0.213	1017.4	1112	1303	1399	1488

The equations below are in english measurements.
Data was converted between metric and english to perform the necessary calculations.

			Metric Units				
V_{Design} (km/h)	R_{AASHTO} (m)	$f_{ms\ (V\ Design)}$ f_{max} - 0.2; at V_{Des}	R_{Path} (m)	R_{Design} for n^{th} %tile of Driver Population, (m) 50th	85th	90th	95th
50	100	0.305	56.9	51	77	59	70
60	150	0.278	88.8	87	117	106	118
70	215	0.258	129.0	133	168	164	179
80	280	0.225	189.5	202	244	252	271
90	375	0.218	246.3	266	316	334	357
100	490	0.213	310.1	339	397	426	454

Equations: e = 0.04

$$e+f = \frac{(V_{Design})^2}{15^*(R_{Path\ for\ 50,\ 85,\ 90,\ 95\ percentile})} \quad ; R\ (ft)$$

$$R_{Path} = \frac{(V_{Design})^2}{15^*(e+f)} \quad ; R\ (ft)$$

Percentile of Driver Population	Slope (a)	Intercept (b)
95th	0.66	35
90th	0.69	52
85th	0.79	-12
50th	0.88	39

$R_{path} = a^*(R_{Design}) + b$
$R_{Design} = (R_{Path} - b)/a$
a = slope
b = intercept

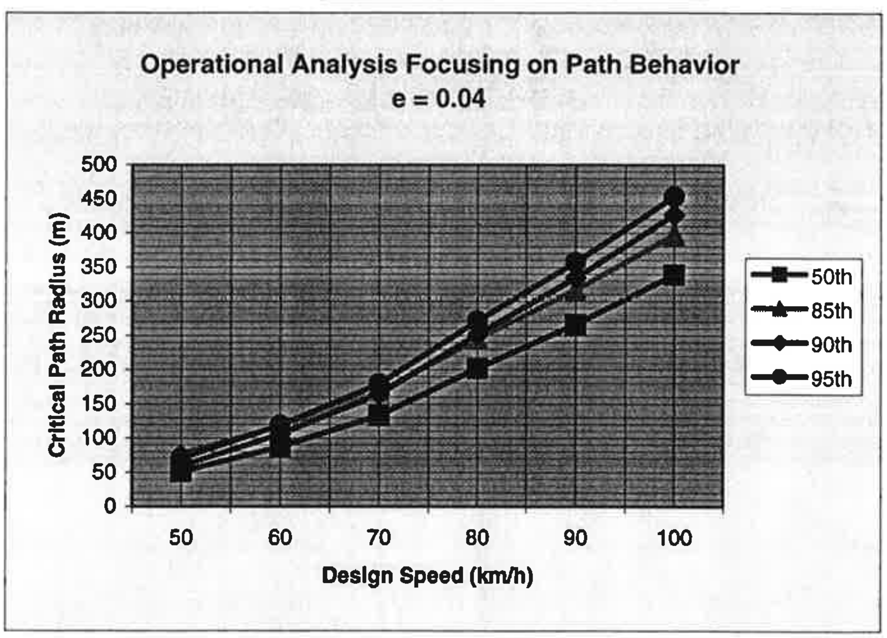


Table VII-3b
Derivation of Design, Radii for Horizontal Curves Using 'Margin of Safety' Criterion and Modeled Path Behavior
($e_{(max)} = 0.06$)

V_{Design} (km/h)	V_{Design} (mph)	R_{AASHTO} (ft)	$f_{ms\ (V\ Design)}$ f_{max} - 0.2; at V_{Des}	R_{Path} (ft)	R_{Design} for n^{th} %tile of Driver Population, (f 50th	85th	90th	95th
	English Units							
50	31	295	0.305	176.3	156	238	180	214
60	37	443	0.278	274.2	267	362	322	362
70	43	640	0.258	396.6	406	517	499	548
80	49.7	820	0.225	578.0	613	747	762	**823**
90	56	1099	0.218	750.0	808	965	1012	1083
100	62	1427	0.213	942.9	1027	1209	1291	1376

The equations below are in english measurements.
Data was converted between metric and english to perform the necessary calculations.

Metric Units

V_{Design} (km/h)	R_{AASHTO} (m)	$f_{ms\ (V\ Design)}$ f_{max} - 0.2; at V_{Des}	R_{Path} (m)	R_{Design} for n^{th} %tile of Driver Population, (n 50th	85th	90th	95th
50	90	0.305	53.7	48	73	55	65
60	135	0.278	83.6	81	110	98	110
70	195	0.258	120.9	124	158	152	167
80	250	0.225	176.2	187	228	232	**251**
90	335	0.218	228.6	246	294	308	330
100	435	0.213	287.4	313	368	394	419

Equations: $e = 0.06$

$$e+f = \frac{(V_{Design})^2}{15^*(R_{Path\ for\ 50,\ 85,\ 90,\ 95\ percentile})} ;\ R\ (ft)$$

$$R_{Path} = \frac{(V_{Design})^2}{15^*(e+f)} ;\ R\ (ft)$$

$$R_{path} = a^*(R_{Design}) + b$$

$$R_{Design} = (R_{Path} - b)/a$$

a = slope
b = intercept

Percentile of Driver Population	Slope (a)	Intercept (b)
95th	0.66	35
90th	0.69	52
85th	0.79	-12
50th	0.88	39

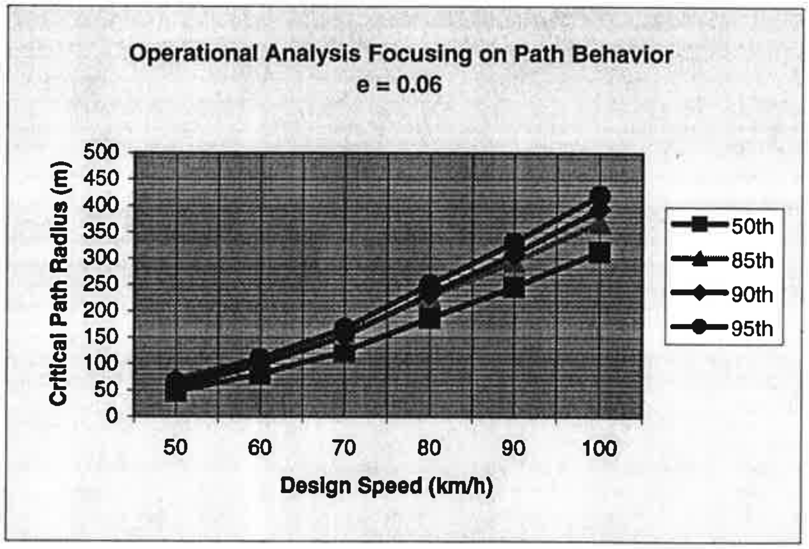


Table VII-3c
Derivation of Design, Radii for Horizontal Curves Using 'Margin of Safety' Criterion and Modeled Path Behavior

($e_{(max)} = 0.08$)

English Units

V_{Design} (km/h)	V_{Design} (mph)	R_{AASHTO} (ft)	$f_{ms\ (V\ Design)}$ f_{max} - 0.2; at V_{Des}	R_{Path} (ft)	R_{Design} for n^{th} %tile of Driver Population, (ft) 50th	85th	90th	95th
50	31	262	0.305	167.1	146	227	167	200
60	37	410	0.278	258.8	250	343	300	339
70	43	574	0.258	373.2	380	488	465	512
80	49.7	755	0.225	540.1	569	699	707	**765**
90	56	1001	0.218	699.6	751	901	939	**1007**
100	62	1296	0.213	878.5	954	1127	1198	1278

The equations below are in english measurements.
Data was converted between metric and english to perform the necessary calculations.

Metric Units

V_{Design} (km/h)	R_{AASHTO} (m)	$f_{ms\ (V\ Design)}$ f_{max} - 0.2; at V_{Des}	R_{Path} (m)	R_{Design} for n^{th} %tile of Driver Population, (m) 50th	85th	90th	95th
50	80	0.305	50.9	44	69	51	61
60	125	0.278	78.9	76	104	91	103
70	175	0.258	113.7	116	149	142	156
80	230	0.225	164.6	171	213	216	**233**
90	305	0.218	213.3	229	275	286	**307**
100	395	0.213	267.8	291	344	365	390

Equations: $e = 0.08$

$$e+f = \frac{(V_{Design})^2}{15^*(R_{Path\ for\ 50,\ 85,\ 90,\ 95\ percentile})};\ R\ (ft)$$

$$R_{Path} = \frac{(V_{Design})^2}{15^*(e+f)};\ R\ (ft)$$

Percentile of Driver Population	Slope (a)	Intercept (b)
95th	0.66	35
90th	0.69	52
85th	0.79	-12
50th	0.88	39

$R_{path} = a^*(R_{Design}) + b$

$R_{Design} = (R_{Path} - b)/a$

$a =$ slope

$b =$ intercept

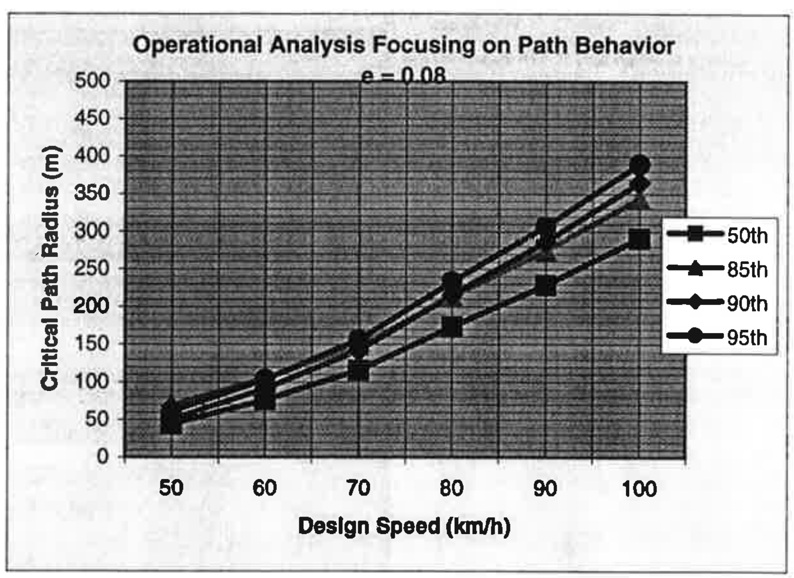


Table VII-3d

Derivation of Design, Radii for Horizontal Curves Using 'Margin of Safety' Criterion and Modeled Path Behavior

$(e_{(max)} = 0.10)$

English Units

V_{Design} (km/h)	V_{Design} (mph)	R_{AASHTO} (ft)	$f_{ms\ (V\ Design)}$ f_{max} - 0.2; at V_{Des}	R_{Path} (ft)	R_{Design} for n^{th} %tile of Driver Population, (ft) 50th	85th	90th	95th
50	31	246	0.305	158.9	136	216	155	188
60	37	377	0.278	245.1	234	325	280	318
70	43	525	0.258	352.3	356	461	435	481
80	49.7	689	0.225	506.9	532	657	659	**715**
90	56	902	0.218	655.6	701	845	875	**940**
100	62	1181	0.213	822.4	890	1056	1116	**1193**

The equations below are in english measurements.
Data was converted between metric and english to perform the necessary calculations.

Metric Units

V_{Design} (km/h)	R_{AASHTO} (m)	$f_{ms\ (V\ Design)}$ f_{max} - 0.2; at V_{Des}	R_{Path} (m)	R_{Design} for n^{th} %tile of Driver Population, (m) 50th	85th	90th	95th
50	75	0.305	48.4	42	66	47	57
60	115	0.278	74.7	71	99	85	97
70	160	0.258	107.4	109	141	133	147
80	210	0.225	154.5	162	200	201	**218**
90	275	0.218	199.8	214	258	267	**287**
100	360	0.213	250.7	271	322	340	**364**

Equations: $e = 0.10$

$$e+f = \frac{(V_{Design})^2}{15^*(R_{Path\ for\ 50,\ 85,\ 90,\ 95\ percentile})}\ ; R\ (ft)$$

$$R_{Path} = \frac{(V_{Design})^2}{15^*(e+f)}\ ; R\ (ft)$$

$R_{path} = a^*(R_{Design}) + b$

$R_{Design} = (R_{Path} - b)/a$

a = slope

b = intercept

Percentile of Driver Population	Slope (a)	Intercept (b)
95th	0.66	35
90th	0.69	52
85th	0.79	-12
50th	0.88	39

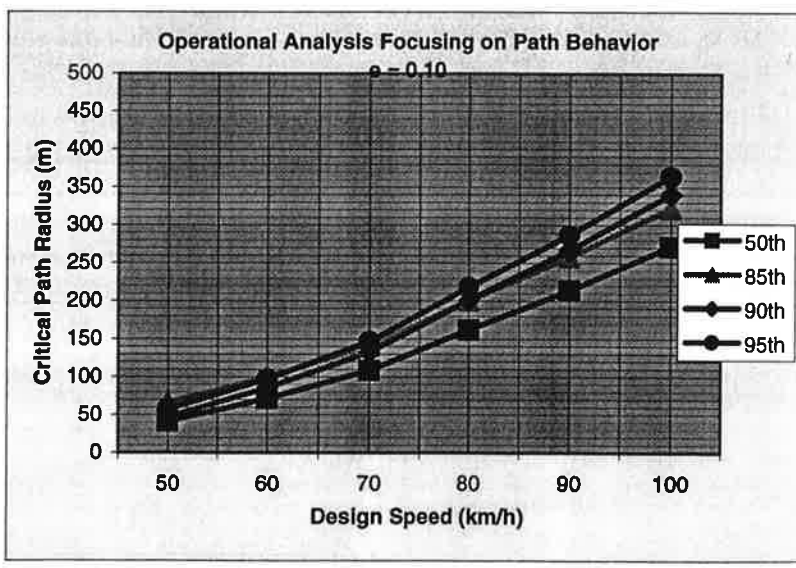


Table VII-4
Comparison of AASHTO and 'Margin of Safety' Criterion Using Modeled Path Behavior for

(e_{max} = 0.10)

Y_{Design} (km/h)	R_{AASHTO} (m)	R_{design}[1] (m)	Equivalent V_{AASHTO} for R_{Design}	ΔV (k5/h)
50	75	66	46	4
60	115	99	55	5
70	160	141	63	7
80	210	200	76	4
90	275	258	87	3
100	360	322	93	7

[1] per Table VIII-3d for emax = 0.10; 85th percentile path

Summary of Operational Risk
The alternative margin of safety approach using different models for both speed and path behavior produces somewhat similar and consistent results, particularly for high speed curves. Design for loss of control vs. comfort can enable the designer to accept smaller radius curves for a given design speed. Equivalent speed reductions on the order of 10 km/h for higher speed road curves are produced with this approach.

Safety Risk Assessment
A secondary approach to alternative horizontal curve design policy utilizes the crash prediction models described above from research by Glennon, et al. and Zegeer. Both models were exercised for a range of geometric and traffic conditions consistent with LVLRoad operations. The models were used to predict the number of accidents per year for different curve radius designs associated with alternative design policy approaches. Differences in expected crash frequency were then used to express the relative risk of one curve design versus another. The following is a summary of this safety risk assessment.

Inputs and Assumptions
The Glennon and Zegeer models predict curve-related accidents as a function of roadway width, radius or degree of curve, central angle or length of curve, presence of a spiral transition, and average daily traffic volume. Assumptions noted below reflect typical LVLRoad conditions, and would all tend to maximize or heighten the predicted crash experience on any one curve.

- Roadway width – 20 ft. (3.0 m)
- Curves are unspiraled

- ADT of 100 vpd, 250 vpd, and 400 vpd
- Curve radii for emax of 0.08 for ranges in AASHTO design speed of 100, 90, 80, 70, 60 and 50 km/h
- Ranges in central angle of curve of 10, 20, 30 and 40 degrees

The Glennon model expresses curve crash frequency as a function of the expected crash rate for a tangent alignment. For analysis purposes a rate of 1.5 per mvk was used.

Analysis Results

Necessary conversions between English and metric curve geometric definitions were made in exercising the models. Also, the Zegeer model expresses crash frequency as a predicted number of crashes per 5-year period on the curve; the Glennon model compute results in a crash rate prediction that can be readily translated to crashes per year. To facilitate comparisons between the two models, the Glennon model results were converted to a five-year period.

Tables VII-5a through VII-6d summarize the results of the modeling. Tables VII-5a through VII-5d show predicted crash frequencies over five years for ADT of 100, 250 and 400 vpd per Glennon; TablesVII-6a through VII-6d give comparable results based on the Zegeer model.

Although the models are different in form, they produce somewhat similar results. First, for the low traffic volume levels (100 vpd and 250 vpd), expected crash frequency is very small-less than 0.2 crashes per five-year period. This finding in itself suggests that marginally smaller curve radii should not be expected to result in a significantly greater risk. Second, there are interesting interrelationships among curve radius, central angle and crash frequency. For any given central angle, the effect of lengthening curve radius varies depending on the curve. The positive effects of a milder curve are somewhat offset by the added length of curve required for that central angle. Note that for many higher speed curves there is little computed benefit associated with larger radii. This effect is more pronounced in the Zegeer model, but is apparent in Glennon's model as well. This finding, which has been discussed in the literature, has implications with respect to reconstruction projects, in which curve flattening is being considered, and the central angle is fixed.

Risk Assessment

The findings from Tables VII-5 and VII-6 can be used to assess the relative risk of alternative design approaches for LVLRoad horizontal curves. The difference in expected frequencies between curves designed for different design speeds (for the same central angle) is a direct measure of risk. The "baseline" condition for analysis purposes is design according to AASHTO Policy. Consider the following example.

> *A curve on a roadway with 250 vpd is designed according to AASHTO for a design speed of 80 km/h. What is the estimated safety risk of accepting a smaller radius curve equivalent to a speed of 60 km/h? (Assume a central angle of 20 degrees)*
>
> *Estimated 5-year frequency per Zegeer model for 80 km/h is 0.105; and for 60 km/h is 0.135*
>
> *Difference in 5-year frequency is (0.135- 0.105) or 0.03. This is equivalent to **1** additional crash per **165** years of operation at 250 vpd on that curve.*

Table VII-5a
Predicted Number of Accidents in a 5-Year Period by the Glennon Model[1]
Average Daily Traffic = 100 vpd

Curve Central Angle (Degrees)	Curve Design Speed (km/h) (e_{max} = 0.08)					
	100	90	80	70	60	50
10	0.039	0.044	0.053	0.066	0.089	0.136
20	0.051	0.053	0.060	0.072	0.093	0.139
30	0.062	0.062	0.067	0.077	0.097	0.141
40	0.074	0.071	0.074	0.082	0.101	0.143

[1] Assumed Tangent Accident Rate = 1.5 acc/MVM

Table VII-5b
Predicted Number of Accidents in a 5-Year Period by the Glennon Model[1]
Average Daily Traffic = 250

Curve Central Angle (Degrees)	Curve Design Speed (km/h) (e_{max} = 0.08)					
	100	90	80	70	60	50
10	0.097	0.110	0.133	0.166	0.223	0.341
20	0.126	0.133	0.151	0.179	0.233	0.347
30	0.156	0.156	0.168	0.192	0.242	0.352
40	0.185	0.178	0.185	0.205	0.251	0.358

[1] Assumed Tangent Accident Rate = 1.5 acc/MVM

Table VII-5c
Predicted Number of Accidents in a 5-Year Period by the Glennon Model[1]
Average Daily Traffic = 400

Curve Central Angle (Degrees)	Curve Design Speed (km/h) (e_{max} = 0.08)					
	100	90	80	70	60	50
10	0.155	0.177	0.214	0.266	0.358	0.545
20	0.202	0.213	0.241	0.286	0.372	0.554
30	0.249	0.249	0.268	0.307	0.387	0.564
40	0.296	0.285	0.295	0.328	0.402	0.573

[1] Assumed Tangent Accident Rate = 1.5 acc/MVM

Table VII-6a
Predicted Number of Accidents in a 5-Year Period by the Zegeer Model[1]
Average Daily Traffic = 100 vpd

Curve Central Angle (Degrees)	Curve Design Speed (km/h) (e_{max} = 0.08)					
	100	90	80	70	60	50
10	0.029	0.030	0.033	0.039	0.049	0.073
20	0.044	0.042	0.042	0.045	0.054	0.076
30	0.060	0.053	0.051	0.052	0.059	0.079
40	0.075	0.065	0.059	0.059	0.064	0.082

[1] Assumed Roadway Width = 20 feet (6.0m)

Table VII-6b
Predicted Number of Accidents in a 5-Year Period by the Zegeer Model[1]
Average Daily Traffic = 250 vpd

Curve Central Angle (Degrees)	Curve Design Speed (km/h) (e_{max} = 0.08)					
	100	90	80	70	60	50
10	0.073	0.075	0.083	0.096	0.123	0.182
20	0.111	0.104	0.105	0.113	0.135	0.190
30	0.149	0.133	0.127	0.130	0.147	0.197
40	0.187	0.163	0.149	0.147	0.159	0.205

[1] Assumed Roadway Width = 20 feet (6.0m)

Table VII-6c
Predicted Number of Accidents in a 5-Year Period by the Zegeer Model[1]
Average Daily Traffic = 400 vpd

Curve Central Angle (Degrees)	Curve Design Speed (km/h) (e_{max} = 0.08)					
	100	90	80	70	60	50
10	0.117	0.120	0.132	0.154	0.198	0.291
20	0.178	0.167	0.167	0.181	0.217	0.303
30	0.238	0.213	0.203	0.208	0.236	0.315
40	0.299	0.260	0.238	0.235	0.255	0.328

[1] Assumed Roadway Width = 20 feet (6.0m)

Estimates of the relative risk of accepting smaller curve radii than AASHTO design policy are provided in Tables VII-7; VII-8 for the range of traffic volume levels and models employed, for critical design speeds of 100 km/h and 80 km/h. While both models produce somewhat different results, there are a number of fundamental conclusions common to both. The following conclusions are evident:

For very low traffic volumes (less than 250 vehicles per day), design speed differentials of 20 km/h (about 12 mph) or less, the nominal risk predicted by both models is very small-one crash per curve in well over 100 years according to the Zegeer model, and one in 60 to over 100 years according to the Glennon model. Even for greater speed differentials-30 km/h (about 18 km/h), the nominal risk according to both models is at most on the order of one more crash per curve per 25 to 30 years.

For traffic volume levels approaching 400 vpd, the Glennon model predicts a risk level that approaches the safety threshold for LVLRoads for speed differentials of up to 30 km/h; the Zegeer model predicts a much lower risk level. Note that both models predict all crashes; not just severe crashes. A nominal speed reduction of 30 km/h thus appears to represent a reasonable threshold of acceptable risk for all traffic volume levels within the LVLRoad framework.

Safety Risk Assessment—Some Caveats

The risk assessment analyses presented above should be reviewed with caution. They should be considered as providing general guidance on order of magnitude risks. In using the analyses, it should be understood that, first, the crash models themselves were developed for roads and traffic volumes beyond the range being studied here. There is always uncertainty in extrapolating a model to a condition not included in model formulation. Second, one should be aware that comparisons between design alternatives are based on the form of the models employed.

It can be argued that the risk analysis may be understating relative differences in crash experience. Curve crashes on LVLRoads are highly dependent on the roadside, which tends uniformly to be less forgiving than higher class roads. Conversely, the risk assessments may be significantly overstating crash experience if one considers traffic volume levels. Using the results for, say 40 vpd to model all conditions between 251 and 400 overstates the exposure of traffic volume and hence overall crash experience.

In any event, some confidence can be placed in the results given their overall consistency with findings published by others. As noted above, curve flattening has been shown to not be cost-effective for traffic volume levels under 1500 vpd, on the order of 5 to 10 times the volume levels studied here. Moreover, a relatively high speed differential of over 20 km/h has been shown to justify curve flattening. Finally, the results show reasonable consistency with the operational assessments in terms of sensitivities of speed to curve radius.

Table VII-7

Relative Safety Risk of Alternative Horizontal Curve Criteria for Low Volume Local Roads Based on (Glennon Model)

Curve Central Angle (Degrees)	Predicted Accidents on Curve (5 years) for Curve Design Speed = 100 km/h	Years to Expect One Crash on Curve at DS = 100 km/h	Predicted Accidents for Curve Design Speed = 90 km/h	Years to Expect One Crash on Curve at DS = 90 km/h	Predicted Accidents for Curve Design Speed = 80 km/h	Years to Expect One Crash on Curve at DS = 80 km/h	Predicted Accidents for Curve Design Speed = 70 km/h	Years to Expect One Crash on Curve at DS = 70 km/h
for Average Daily Traffic = 100 vpd								
10	0.03884	129	0.04417	113	0.05339	94	0.06639	75
20	0.05056	99	0.05322	94	0.06022	83	0.07158	70
30	0.06229	80	0.06228	80	0.06705	75	0.07678	65
40	0.07402	68	0.07133	70	0.07387	68	0.08198	61
for Average Daily Traffic = 250 vpd								
10	0.09709	51	0.11041	45	0.13347	37	0.16597	30
20	0.12641	40	0.13305	38	0.15054	33	0.17896	28
30	0.15573	32	0.15569	32	0.16761	30	0.19195	26
40	0.18504	27	0.17833	28	0.18468	27	0.20494	24
for Average Daily Traffic = 400 vpd								
10	0.09709	51	0.11041	45	0.13347	37	0.16597	30
20	0.12641	40	0.13305	38	0.15054	33	0.17896	28
30	0.15573	32	0.15569	32	0.16761	30	0.19195	26
40	0.18504	27	0.17833	28	0.18468	27	0.20494	24

Table VII-8
Relative Safety Risk of Alternative Horizontal Curve Criteria for Low Volume Local Roads Based on (ZegeerModel)

Curve Central Angle (Degrees)	Predicted Accidents on Curve (5 years) for Curve Design Speed = 100 km/h	Years to Expect One Crash on Curve at DS = 100 km/h	Predicted Accidents for Curve Design Speed = 90 km/h	Years to Expect One Crash on Curve at DS = 90 km/h	Predicted Accidents for Curve Design Speed = 80 km/h	Years to Expect One Crash on Curve at DS = 80 km/h	Predicted Accidents for Curve Design Speed = 70 km/h	Years to Expect One Crash on Curve at DS = 70 km/h
for Average Daily Traffic = 100 vpd								
10	0.02925	171	0.02996	167	0.03305	151	0.03856	130
20	0.04438	113	0.04165	120	0.04186	119	0.04526	110
30	0.05952	84	0.05334	94	0.05067	99	0.05197	96
40	0.07466	67	0.06503	77	0.05949	84	0.05867	85
for Average Daily Traffic = 250 vpd								
10	0.07312	68	0.07491	67	0.08262	61	0.09639	52
20	0.11096	45	0.10412	48	0.10465	48	0.11315	44
30	0.14880	34	0.13334	37	0.12669	39	0.12992	38
40	0.18664	27	0.16256	31	0.14872	34	0.14669	34
for Average Daily Traffic = 400 vpd								
10	0.11699	43	0.11985	42	0.13219	38	0.15422	32
20	0.17754	28	0.16660	30	0.16744	30	0.18105	28
30	0.23808	21	0.21335	23	0.20270	25	0.20787	24
40	0.29863	17	0.26010	19	0.23795	21	0.23470	21

Table V-9
Crash Risk Analysis Using Neuman SSD Model

90/100 KM/H	Amount of Restricted	Length of		Accident	Risk Factor for Hazard Within Restriction[3]			Crash Risk per Year for Range of Hazards		
					Minor	Moderate	Major	Minor	Moderate	Major
A[4]	SSD (km/h)[1]	Restriction[2] (km)	ADT	Rate (mvk)				Delta A	Delta A	Delta A
4	35	0.100	250	2.8	2.0	3.0	4.0	5.11E-02	7.67E-02	1.02E-01
6	35	0.125	250	2.8	2.0	3.0	4.0	6.39E-02	9.58E-02	1.28E-01
8	35	0.145	250	2.8	2.0	3.0	4.0	7.41E-02	1.11E-01	1.48E-01
10	35	0.165	250	2.8	2.0	3.0	4.0	8.43E-02	1.26E-01	1.69E-01
12	35	0.190	250	2.8	2.0	3.0	4.0	9.71E-02	1.46E-01	1.94E-01

90/100 KM/H	Amount of Restricted	Length of		Accident	Risk Factor for Hazard Within Restriction[3]			Crash Risk per Year for Range of Hazards		
					Minor	Moderate	Major	Minor	Moderate	Major
A[4]	SSD (km/h)[1]	Restriction[2] (km)	ADT	Rate (mvk)				Delta A	Delta A	Delta A
4	15	0.100	400	2.0	0.5	1.1	1.8	1.46E-02	3.21E-02	5-26E-02
6	15	0.155	400	2.0	0.5	1.1	1.8	2.26E-02	4.98E-02	8.15E-02
8	15	0.205	400	2.0	0.5	1.1	1.8	2.99E-02	6.58E-02	1.08E-01
10	15	0.250	400	2.0	0.5	1.1	1.8	3.65E-02	8.03E-02	1.31E-01
12	15	0.300	400	2.0	0.5	1.1	1.8	4.38E-02	9.64E-02	1.58E-01

90/100 KM/H	Amount of Restricted	Length of		Accident	Risk Factor for Hazard Within Restriction[3]			Crash Risk per Year for Range of Hazards		
					Minor	Moderate	Major	Minor	Moderate	Major
A[4]	SSD (km/h)[1]	Restriction[2] (km)	ADT	Rate (mvk)				Delta A	Delta A	Delta A
4	25	0.100	400	2.0	1.2	2.0	2.8	3.50E-02	5.84E-02	8.18E-02
6	25	0.140	400	2.0	1.2	2.0	2.8	4.91E-02	8.18E-02	1.14E-01
8	25	0.170	400	2.0	1.2	2.0	2.8	5.96E-02	9.93E-02	1.39E-01
10	25	0.210	400	2.0	1.2	2.0	2.8	7.36E-02	1.23E-01	1.72E-01
12	25	0.245	400	2.0	1.2	2.0	2.8	8.58E-02	1.43E-01	2.00E-01

[1] Normal Deficiency in available SSD
[2] Per Figure V-6
[3] Per Table V-8
[4] Algebraic difference in grades

Table VII-10a
Risk Assessment for Alternative Horizontal Curve Designs Based on Zegeer Curve Model
(Average Daily Traffic = 100 vpd)

	Years to Accumulate One Additional Crash for Curve by Accepting Lower Design Speed		
Δ	100 km/h to 90 km/h	100 km/h to 80 km/h	100 km/h to 70 km/h
10	6993	1316	537
20	N/A	N/A	5693
30	N/A	N/A	N/A
40	N/A	N/A	N/A
Δ	80 km/h to 70 km/h	80 km/h to 60 km/h	80 km/h to 50 km/h
10	908	306	126
20	1470	406	147
30	3866	603	177
40	N/A	1173	223

Table VII-10b
Risk Assessment for Alternative Horizontal Curve Designs Based on Zegeer Curve Model
(Average Daily Traffic = 250 vpd)

	Years to Accumulate One Additional Crash for Curve by Accepting Lower Design Speed		
Δ	100 km/h to 90 km/h	100 km/h to 80 km/h	100 km/h to 70 km/h
10	2797	526	215
20	N/A	N/A	2277
30	N/A	N/A	N/A
40	N/A	N/A	N/A
Δ	80 km/h to 70 km/h	80 km/h to 60 km/h	80 km/h to 50 km/h
10	363	122	50
20	588	162	59
30	1546	241	71
40	N/A	469	89

Table VII-10c
Risk Assessment for Alternative Horizontal Curve Designs Based on Zegeer Curve Model
(Average Daily Traffic = 400 vpd)

	Years to Accumulate One Additional Crash for Curve by Accepting Lower Design Speed		
Δ	100 km/h to 90 km/h	100 km/h to 80 km/h	100 km/h to 70 km/h
10	1748	329	134
20	N/A	N/A	1423
30	N/A	N/A	N/A
40	N/A	N/A	N/A
Δ	80 km/h to 70 km/h	80 km/h to 60 km/h	80 km/h to 50 km/h
10	227	77	31
20	368	102	37
30	966	151	44
40	N/A	293	56

Guidelines for Horizontal Curve Design on LVLRoads

There is clear justification for accepting less restrictive design criteria for horizontal alignment, within the philosophy of LVLRoad design discussed in Chapter III. The literature review, inspection of the AASHTO model and assumptions, and both operational and safety risk assessments produce generally consistent findings:

1. *For design of LVLRoads, acceptable operations are produced by smaller radius curvature than given by current AASHTO design policy as given in Table III-6 of the 1994 Green Book. Design radii based on the equivalent of a reduction of 10 to 20 km/h from current design policy may be sufficient for LVLRoads.*

2. *For LVLRoads carrying substantial truck traffic, such as industrial or resource recovery roads, somewhat greater radius curvature is recommended for design purposes to address the greater potential of vehicle rollover. Truck rollover potential is greatest at low speeds, and hence no change from current Policy appears appropriate. A difference of no more than 10 km/h from current design policy may be sufficient for higher design speeds for LVLRoads carrying truck traffic.*

3. *Design criteria for reconstruction of LVLRoads may allow for a design speed differential of up to 30 km/h, depending on the known history of the site and local conditions, including crash experience, speed characteristics, roadside character and pavement or surface conditions.*

New Construction Guidelines

Tables VII-11 through VII-13 show suggested LVLRoad design criteria for new construction. The tables assume that the AASHTO curve design model remains the basis for LVLRoad design. In unconstrained situations (left to the discretion of the designer), the use of current AASHTO design radii represents a desirable or upper limit for design. For constrained situations, LVLRoad criteria would allow the use of smaller radii as defined by an equivalent design speed differential. This differential is a function of design speed, traffic volume and pavement condition.

Tables VII-11 through VII-13 represent the research team's judgment and synthesis of the various studies. The tables show allowable curve radii for the range of superelevation policies. The far right column of each table shows the equivalent AASHTO curve design speed for reference. Depending on the condition, the tables show an effective acceptable 'speed reduction' of up to 20 km/h. Maximum speed differentials (representing the least restrictive design policy) are associated with lower volume (less than 250 vehicles per day) and higher speed LVLRoads. Acceptable speed differentials lessen for LVLRoad design on the "high end" of the volume range.

The less restrictive design policy is recommended for consideration for major local, minor access, and agricultural access roads. These would tend to be driven primarily by passenger car traffic and familiar users. More restrictive guidelines (i.e., greater allowable curvature) are recommended for roads intended to carry substantial truck traffic, which is more susceptible to rollover. Also, these guidelines are suggested for use on recreational or scenic roads, which are driven by unfamiliar drivers and by RVs and other similar vehicles that may not handle as well as passenger cars.

Table VII-11
Guidelines for Design of Controlling Horizontal Curves for LVLRoads
(Major Local, Minor Access, & Agricultural Access Roads)

Roadway Design Speed (km/h)	ADT ≤ 250 Vehicles per Day Acceptable Radius of Curve (m) in Constrained Conditions for Maximum Superelvation Policy of				Equivalent Curve Design Speed per Current AASHTO Policy (km/h)
	.04	.06	.08	.10	
30	20	20	20	20	NA
40	35	30	30	25	[30]
50	50	45	40	35	[35]
60	80	75	70	60	[45]
70	100	90	80	75	[50]
80	150	135	125	115	[60]
90	215	195	175	160	[70]
100	280	250	230	210	[80]

Table VII-12
Guidelines for Design of Controlling Horizontal Curves for LVLRoads
(Major Local, Minor Access, & Agricultural Access Roads)

Roadway Design Speed (km/h)	ADT 251 - 400 Vehicles per Day Acceptable Radius of Curve (m) in Constrained Conditions for Maximum Superelvation Policy of				Equivalent Curve Design Speed per Current AASHTO Policy (km/h)
	.04	.06	.08	.10	
30	20	20	20	20	NA
40	35	30	30	25	[30]
50	60	55	50	45	[40]
60	100	90	80	75	[50]
70	150	135	125	115	[60]
80	180	165	150	140	[65]
90	250	220	200	185	[75]
100	330	290	270	240	[85]

Table VII-13
Guidelines for Design of Controlling Horizontal Curves for LVLRoads
(Industrial/Commercial Access, & Resource Recovery & Recreational & Scenic Roads)

Roadway Design Speed (km/h)	ADT 251 - 400 Vehicles per Day Acceptable Radius of Curve (m) in Constrained Conditions for Maximum Superelvation Policy of				Equivalent Curve Design Speed per Current AASHTO Policy (km/h)
	.04	.06	.08	.10	
20	20[1]	20[1]	20[1]	20[1]	NA
30	35	30	30	25	[30]
40	60	55	50	45	[40]
50	100	90	80	75	[50]
60	125	115	100	95	[55]
70	185	165	150	140	[65]
80	215	195	175	160	[70]
90	280	250	230	210	[80]
100	375	335	305	275	[90]

[1] Subject to off-tracking requirements of design vehicle

Reconstruction Guidelines

For reconstruction of LVLRoads in general, use of the existing curve geometry should be considered acceptable in most cases unless there is evidence that the curve or its characteristics represent an unusual hazard. The following are general guidelines that reflect the risk assessment and special nature of reconstruction projects.

1. *For curves on low or moderate speed LVLRoads (design or estimated operating speed of 60 km/h or less), reconstruction using the existing curve geometry and cross-section is acceptable if the nominal design speed of the curve is within 30 km/h of the design or operating speed; and if there is no clear evidence of a safety problem associated with the curve.*

2. *For curves on higher speed LVLRoads (design or estimated operating speed greater than 60 km/h), reconstruction using the existing curve geometry and cross--section is acceptable if the nominal design speed of the curve is within 20 km/h of the design or operating speed; and if there is no clear evidence of a safety problem associated with the curve.*

3. *Acceptable substitutes for curve reconstruction include measures to reduce speed in the curve (signing, rumble strips, pavement markings), measures to improve the roadside within the curve (clearing slopes, widening shoulder through curve) and measures to increase pavement friction within the curve. Reconstruction employing any or all of these measures should be accompanied by appropriate before-and-after studies of their effectiveness.*

Evidence of a problem may be a history of curve related crashes (requiring at least five and desirably 10 years of known history); physical evidence of curve problems such as skid marks, scarred trees, substantial edge rutting or encroachments, etc.; a history of complaints from residents and/or local police, or measured or known speeds much higher than the intended design speed (30 km/h or greater). Even with such evidence, curve reconstruction actions should focus on low cost measures designed to control speeds, enhance curve tracking, or mitigate roadside encroachment severity. Except in rare circumstances, there are more cost-effective solutions to known LVLRoad curve problems than curve flattening and reconstruction. Design policy for reconstruction of such problems should emphasize such low cost measures, and should not emphasize or encourage more costly measures such as curve flattening.

VIII. Other Design Issues

Project resources limited the ability to address other design issues in greater depth. This chapter provides an overview of these issues, which include:

- Cross Section (Lane and Shoulder Width)
- Paved versus Unpaved Roads
- Single Lane Roads

A summary of literature and review of current practice are contained, as well as recommended guidelines from these sources.

Cross Section Design Criteria

Cross section elements include the lane and shoulder width.

Rural LVLRoads

Research reported in NCHRP Report 362 dealt with lane and shoulder width design criteria for roads with volumes of 2000 vehicles per day and less. The research addressed a full range of functional classification, including local roads. The latest AASHTO Policy (1994) reflected the results of this research for local roads. This was reported in Chapter II (see Table II-4).

NCHRP Report 362 primarily focused on safety and operational cost-effectiveness as the basis for minimum widths. The minimum lane widths included in the data bases assembled for the research were 8 to 9 feet (2.4 to 2.7 meters). Based on safety studies, it was determined that insufficient benefits would be derived for lane widths greater than 2.7 meters on lower speed, very low volume roads.

There are clearly other considerations in cross section design that may suggest greater width dimensions. Such considerations primarily include the functional requirements for greater width associated with wider and longer vehicles using the roadway.

Three of the six recommended functional classes of LVLRoads describe operations using vehicles substantially larger than passenger cars. Industrial/commercial access roads, agricultural access roads, and resource recovery roads are all used by large trucks, combines or other similar vehicles.

A number of design guidelines for roadway width note the importance of providing additional width for such operations. NCHRP Report 214, design standards published by RTAC, and the USFS design guidelines all address larger vehicles and their effect on cross section dimensions.

A detailed analysis of cost-effectiveness of variable width values was beyond the scope of this research. Table VIII-1 presents a synthesis of information presented in this report. This table can be used as general guidance in selecting appropriate traveled way widths for the six functional classes of LVLRoads.

Table VIII-1
Guidelines for Traveled Way Width for LVL Roads (meters)

Functional Classification of LVLRoads

Design Speed (km/h)	Major Local[1]	Minor Access[2]	Recreational & Scenic[1]	Industrial & Commercial Access[2]	Resource Recovery[2]	Agricultural Access[3]
20	---	---	---	6.0	6.0	6.6
30	---	5.4	5.4	6.0	6.0	7.2
40	5.4	5.4	5.4	6.4	6.4	7.2
50	5.4	5.4	5.4	6.8	6.8	7.2
60	5.4	5.4	5.4	6.8	6.8	7.2
70	6.0	6.0	6.0	7.0	---	8.0[4]
80	6.0	6.0	6.0	7.4[4]	---	---
90	6.6	---	6.6	---	---	---
100	6.6	---	---	---	---	---

[1] Based on width values derived from NCHRP Report 362 and published in 1994 AASHTO Policy on Geometric Design

[2] Based on total roadway width values from NCHRP Report 214 (see Table II-14 of this report, higher % trucks, infrequent trips by farm machinery) and values
from RTAC and USFS design guides

[3] Based on width values from NCHRP Report (see Table II-14 of this report, lower % trucks, frequent trips by farm machinery)

[4] Widths in excess of 7.2m may include shoulder

Note that designers should be afforded great discretion in the use of Table VIII-1. Marginal differences in an existing dimension or proposed dimension from that shown in Table VIII-1 may be completely acceptable. In the case of major local, minor access and recreational and scenic roads, the minimum dimensions are less than other minimum values from RTAC, USFS, and other sources. For the other three functional classes, the minimum values reflect off-tracking, passing and other operational requirements for the larger vehicles using these roads. The greater dimensions should *not* be construed as representing a safety-driven design requirement.

Urban LVLRoads

Urban LVLRoad width requirements are less safety driven and more related to basic operational requirements. Speeds are lower, trip lengths and lengths of local roads generally much shorter, and available right-of-way much less than in rural areas. The major functional requirements for all urban LVLRoads include the ability for vehicles to pass one another, to pass a parked or stopped vehicle; need to accommodate fire trucks and other emergency vehicles; and the need to accommodate in some manner occasional larger delivery vehicles. Other width-related dimensions outside the traveled way are associated with border areas for utilities and drainage, and in many cases sidewalks for pedestrians.

The Institute of Transportation Engineers has published design guidelines for widths of residential (i.e., local access) streets. These were considered, translated to metric dimensions and are provided here for reference in Table VIII-2.

Table VIII-2

Design Guidelines for Urban LVLR Cross Section Elements

Cross Section Element	Development Density Low	Development Density Medium
Right-of-way Width (m)	15	18
Traveled Way Width (m) (Two Way Traffic)	6.7 to 8.2	8.5 to 10.3
Curb	Optional	Yes
Sidewalk Offset from Face of Curb (m)	----	1.8
Sidewalk Width (m)	0	1.2 to 1.8

Derived from "Recommended Guidelines for Subdivision Streets," Institute of Transportation Engineers, 1984.

Surface Type

There is substantial mileage of unpaved roads in the U.S. local road system. Such roads are common in rural areas. Unpaved roads can be substantially less costly to construct. They are commonly used to access individual residences and farms across great distances. Traffic volumes are generally very low—often less than 50 vehicles per day.

Issues of concern regarding such roads include traffic volume and type warrants (i.e., at what traffic levels should roads be paved), operating speeds, the safety of unpaved surfaces, and maintenance considerations unique to unpaved roads. Many of these are interrelated. The following is an overview of knowledge obtained during this research that addresses some of these issues.

Safety Considerations

Research performed as part of NCHRP Report 362 addressed the issue of the safety of unpaved surfaces. Data from three states were collected and evaluated by the University of North Carolina Highway Research Center comparing crash rates of unpaved and paved surfaces. Other factors such as roadway width, average daily traffic and terrain were controlled for or included in the analysis. Functional classification was not available for a part of the database, and was therefore not included in the analysis.

The analysis established the following:

- Crash rates are greater for unpaved roads versus paved roads for lane widths of 9 feet or less, 10 to 11 feet, and 12 feet (see Table VIII-3)
- Crash rates are *lower* for narrower widths
- Crash rates are similar for paved versus unpaved roads for traffic volumes less than 250 vehicles per day; above 250 vehicles per day, paved roads showed significantly lower crash rates than unpaved roads. See Table VIII-4.

Most of the data for unpaved roads were on roads with less than 400 vehicles per day. The data for paved roads contained segments with traffic volumes greater than 400 vehicles per day.

Table VIII-3
Summary of Models Describing Relationship Between Accident Rates and Paved Versus Unpaved Surfaces on Low Volume Roads

Model Results Comparing Paved Versus Unpaved				
Road Type	**Lane Width (feet)**	**Model A[1] Least Squares Mean Rates of Related Accidents (per MVM)**	**Model B[2] Least Squares Mean Rates of Related Accidents (per MVM)**	**Model C[3] Least Squares Mean Rates of Related Accidents (per MVM)**
P_1 – Paved	≤ 9	1.20	1.13	1.32
P_2 – Paved	10-11	2.06	1.94	2.04
P_3 - Paved	≥ 12	1.99	1.99	2.09
U_1 – Unpaved	≤ 9	1.72	1.85	1.94
U_2 – Unpaved	10-11	3.95	3.95	3.85
U_3 - Unpaved	≥ 12	3.88	3.92	3.83

Pairwise Comparisons	**Model A P-Values**	**Model B P-Values**	**Model C P-Values**
$P_1 = P_2$	0.074	0.089	0.131
$P_2 = P_3$	0.802	0.866	0.872
$U_1 = U_2$	0.006	0.010	0.019
$U_2 = U_3$	0.921	0.958	0.984
$P_1 = U_1$	0.517	0.353	0.427
$P_2 = U_2$	0.001	0.001	0.001
$P_3 = U_3$	0.001	0.001	0.001

Source: NCHRP Report 362, Table C-10

[1] Model A contained roadway class, terrain and clear recovery distance.
[2] Model B added 'driveways per mile' to the Model A variables
[3] Model C added 'state' to the Model B variables

Interpretation of Safety Studies

As was noted in Chapter II, the results of NCHRP 362 produced somewhat counterintuitive findings about width versus crash rate for very narrow roads. This effect is seen in the paved versus unpaved analysis for lane widths of 9 feet and less. It would seem that such roads are undoubtedly local in nature, and driven at lower speeds, thus explaining the lower crash rates. The greater overall crash rate for unpaved roads is undoubtedly due to the interrelationships among surface type and other geometric features. Unpaved roads tend to be designed with poorer alignment, lesser clear zones, etc.

Table VIII-4
Relationship of Average Daily Traffic to Accident Rates for Paved Versus Unpaved Surfaces on Low Volume Roads

Paved Versus Unpaved Roads Within ADT Range			
Pavement Type	**Average Daily Traffic**	**Least Square Mean Rates of Related Accidents (per MVM)**	**Sample Size**
P_1 – Paved	≤ 250	3.71	86
P_2 – Paved	250-400	2.61	91
P_3 - Paved	> 400	1.96	430
U_1 – Unpaved	≤ 250	3.94	128
U_2 – Unpaved	250-400	4.16	23
U_3 - Unpaved	> 400	2.29	13

Comparisons	**P-Value**
$P_1 = P_2$	0.046
$P_2 = P_3$	0.032
$U_1 = U_2$	10.751
$U_2 = U_3$	0.013
$P_1 = U_1$	0.730
$P_2 = U_2$	0.015
$P_3 = U_3$	0.519
$P_2 + P_3 = U_2 + U_3$	0.024

The findings in Table VIII-4 provide insights toward establishment of guidelines for paving unpaved surfaces based on safety considerations. Note the following:

For unpaved roads with 250 to 400 vehicles per day, the added safety risk can be characterized by the difference in related crash rates (4.16 per MVM versus 2.61 per MVM for paved roads). Table VIII-5 shows calculations for traffic volumes within the 250 to 400 vpd range. The added risk of one additional crash per km is on the order of one per every 7 to 11 years.

Table VIII-5
Safety Risk of Unpaved Versus Paved LVLRoads for ADT of 250 – 400 VPD

Average Daily Traffic	Total Crashes per km (mile) per Year		Risk (Difference in Crashes per km/year)	Years to Accumulate 1 Additional Crash per km
	Paved[1]	Unpaved[2]		
250	0.148 (0.238)	0.236 (0.380)	0.088	~ 11 years
300	0.178 (0.286)	0.283 (0.456)	0.105	~ 10 years
350	0.207 (0.333)	0.330 (0.531)	0.123	~ 8 years
400	0.237 (0.381)	0.377 (0.608)	0.14	~ 7 years

[1] Based on 2.61 crashes per million-vehicle miles from NCHRP 362 (Table VIII-4)
[2] Based on 4.16 crashers per million-vehicle miles from NCHRP 362 (Table VIII-4)

Data from the same study suggest that about 50 to 60 percent of reported crashes for such conditions result in an injury or fatality. Taking this into account with the results from Table VIII-5, the safety risk of unpaved road surfaces is at the thresholds established for this study for LVLRoads. For traffic volume ranges of about 300 to 350 vehicles per day a saving of one severe crash per km every 10 to 15 years would be expected with a paved versus unpaved surface.

Other Considerations

The above findings are also consistent with discussions and general guidelines used by county engineers in decision-making on paving unpaved surfaces. Referring back to Chapter II, volume ranges on the order of 300 to 350 vehicles per day were identified as thresholds at which some engineers made decisions to pave unpaved roads based on maintenance considerations and in response to requests from local users.

USFS and other criteria, as well as the safety studies noted above, suggest that unpaved roads should operate at low to moderate speeds. Enforcement of speed limits on such roads is clearly impractical. Provision for costly solutions such as greater clear zones, milder slopes, etc. is normally inconsistent with the economic decision to build and maintain an unpaved road. Designers should therefore be aware that unpaved road operating speeds should be normally less than 70 km/h, and generally not greater than 80 km/h.

Regarding functional classification of LVLRoads, this research did not discover any guidelines that suggest paved roads should be used for certain types of functional LVLRoads. The speed and service guidelines would suggest that major local roads would more often than not be paved, but this is not considered mandatory. Resource recovery roads (e.g., logging roads) are frequently unpaved.

Horizontal Alignment Design

Unpaved roads have substantially different surface friction characteristics from paved roads. Also, unpaved roads are typically designed for low speed operation, without the use of superelevation. Design of horizontal alignment based on these conditions will differ from design of paved alignments.

Table II-8 in Chapter II presented friction surface characteristics used by the USFS in design of horizontal alignment. The design assumption is based on margin of safety from loss of control. For unpaved surfaces, the designer should have the option of specifying the surface condition (i.e., dry versus wet or icy). This should be left to the judgment of the designer. Based on the surface type (earth, crushed rock, packed snow, etc.) and assumed condition, a traction coefficient is selected and resulting curve radius determined.

Table VIII-6 was developed from USFS design guidelines to show the range of curve radii as a function of design speed and surface type for unpaved roads.

Table VIII-6
Minimum Radius of Curve (Meters) for Unpaved Surfaces (with No Superelevation)

	Traction Coefficient for Design				
T_f =	**0.7**	**0.6**	**0.5**	**0.4**	**0.3**
Design Speed (km/h)	←------ Dry Asphalt ------→ (0.7–0.6); ←---- Loose Gravel ----→ (0.7–0.4); ←------- Crushed Rock -------→ (0.7–0.6); ←-- Earth --→ (0.6)				Lightly Sanded Snow
20	----	----	----	20	30
30	----	20	25	35	65
40	35	30	40	60	115
50	40	50	60	90	----

Source: USFS Preconstruction Handbook (converted from English to metric values)

Using high values of traction coefficient for design allows the designer to select smaller curve radii than would otherwise be used. Of course, the selection of a high traction coefficient is consistent with a higher surface type, and/or with an assumption that poor surface conditions such as snow, ice or wet pavement are sufficiently infrequent as to not control the design.

Design Guidelines

The limited research and synthesis reported here suggests that unpaved roads can operate without undue safety risk for traffic volumes less than 300 to 400 vehicles per day. Unpaved roads are more appropriate where speeds are low to moderate. Judgments about paving unpaved LVLRoads should primarily be based on local conditions and constraints, such as costs and difficulty of maintaining such roads.

Single Lane Roads

Single lane roads may be used in constrained locations, where traffic volumes are very low. Many of the sources of design criteria reviewed for this study referenced single lane roads. The following is an overview of design and operational characteristics of such roads.

Traffic Operations

Single lane roads normally are used were traffic volumes are less than 50 vehicles per day. RTAC design standards suggest that up to 100 vpd may be serviced on single lane resource development roads; in such cases, professional drivers use the road, and these are often in contact by radio. Most of the design guidelines note that single lane roads should be designed to operate at low speeds - no more than 50 km/h.

Design Considerations

Single lane road widths are normally on the order of 3.5 to 4.0 meters. Such roads are often unpaved. USFS design guidelines suggest the need to plan for turnouts at regular intervals to allow safe passing of opposing vehicles. Some design guidelines suggest the need for twice the normal stopping sight distance as is provided for two-lane roads.

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