TRANSPORTATION TECHNOLOGY SUPPORT
FOR DEVELOPING COUNTRIES

COMPENDIUM 1

Geometric Design Standards for Low-Volume Roads

Modelos de Diseño Geométrico para Caminos de Bajo Volumen

Normes de Dimensionnement Géometrique pour Routes à Faible Capacité

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Notice
The project that is the subject of this report was approved by the Governing Board of the National Research Council, whose members are drawn from the councils of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. The members of the committee responsible for the report were chosen for their special competence and with regard for appropriate balance.

This report has been reviewed by a group other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

Cover photo: Switchbacks on approach to La Cumbre, Bolivia.

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Project Description

The development of agriculture, the distribution of food, the provision of health services, and the access to information through educational services and other forms of communication in rural regions of developing countries all heavily depend on transport facilities. Although rail and water facilities may play important roles in certain areas, a dominant and universal need is for road systems that provide an assured and yet relatively inexpensive means for the movement of people and goods. The bulk of this need is for low-volume roads that generally carry only 5 to 10 vehicles a day and that seldom carry as many as 100 vehicles a day.

The planning, design, construction, and maintenance of low-volume roads for rural regions of developing countries can be greatly enhanced with respect to economics, quality, and performance by the use of low-volume road technology that is available in many parts of the world. Much of this technology has been produced during the developmental phases of what are now the more developed countries, and some is continually produced in both the less and the more developed countries. Some of the technology has been doc-

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Descripcion del Proyecto

El desarrollo de la agricultura, la distribución de víveres, la provisión de servicios de sanidad, y el acceso a información por medio de servicios educacionales y otras formas de comunicación en las regiones rurales de países en desarrollo todos dependen en gran parte de los medios de transporte. Aunque en ciertas áreas los medios de ferrocarril y agua desempeñan una parte importante, una necesidad universal y dominante es para sistemas viales que proveen un medio asegurado pero relativamente poco costoso para el movimiento de gente y mercancías. La gran parte de esta necesidad es para caminos de bajo volúmen que generalmente mueven unicamente unos 5 a 10 vehículos por día y que pocas veces mueven tanto como 100 vehículos por día.

Con respecto a la economía, calidad, y rendimiento, el planeamiento, diseño, construcción y manutención de caminos de bajo volúmen para regiones rurales de países en desarrollo pueden ser mejorados en gran parte por el uso de la tecnología de caminos de bajo volúmen que se encuentra disponible en muchas partes del mundo. Mucha de esta tecnología ha sido producida durante las épocas de desarrollo de lo que ahora son los países mas desarrollados, y alguna se produce continuamente en los países menos y mas desarrollados. Parte de la tecnología se ha documentado en disertaciones, artículos, e

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Description du Projet

Dans les régions rurales des pays en voie de développement, l'exploitation agricole, la distribution des produits alimentaires, l'accès aux services médicaux, l'accès à l'information par l'intermédiaire de moyens éducatifs et d'autres moyens de communication, dépendent en grande partie des moyens de transport. Bien que les transports par voie férrée et par voie navigable jouent un rôle important dans certaines régions, un besoin dominant et universel éxiste d'un réseau routier qui puisse assurer avec certitude et d'une façon relativement bon marché, le déplacement des habitants, et le transport des marchandises. La plus grande partie de ce besoin peut être satisfaite par la construction de routes à faible capacité, capables d'accommoder un trafic de 5 a 10 véhicules par jour, ou plus rarement, jusqu'à 100 véhicules par jour.

L'utilisation des connaissances en technologie, qui éxistent déjà et sont accéssibles dans beaucoup de pays, peut faciliter l'étude des projets de construction, tracé et entretien, de routes à faible capacité dans les régions rurales des pays en voie de développement, surtout en ce qui concerne l'économie, la qualité, et la performance de ces routes. La majeure partie de cette technologie a été produite durant la phase de développement des pays que l'on appelle maintenant développés, et elle continue à être produite à la fois dans ces pays et dans les pays en voie umented in papers, articles, and reports that have been written by experts in the field. But much of the technology is undocumented and exists mainly in the minds of those who have developed and applied the technology through necessity. In either case, existing knowledge about low-volume road technology is widely dispersed geographically, is quite varied in the language and the form of its existence, and is not readily available for application to the needs of developing countries.

In October 1977 the Transportation Research Board (TRB) began this 3-year special project under the sponsorship of the U.S. Agency for International Development (AID) to enhance rural transportation in developing countries by providing improved access to existing information on the planning, design, construction, and maintenance of low-volume roads. With advice and guidance from a project steering committee, TRB will define, produce, and transmit information products through a network of correspondents in developing countries. Broad goals for the ultimate impact of the project work are to promote effective use of existing information in the economic development of transportation infrastructure and thereby to enhance other aspects of rural development throughout the world.

In addition to the packaging and distribution of technical information, personal interactions with users will be provided through field visits, conferences in the United States and abroad, and

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informes que han sido escritos por expertos en el campo. Pero mucha de la tecnología no esta documentada y existe principalmente en las mentes de aquellos que han desarrollado y aplicado la tecnología por necesidad. En cualquier caso, los conocimientos en existencia sobre la tecnología de caminos de bajo volúmen están grandemente esparcidos geograficamente, varían bastante con respecto al idioma y su forma, y no se encuentran facilmente disponibles para su aplicàción a las necesidades de los países en desarrollo.

En octubre de 1977 el Transportation Research Board (TRB) comenzó con este proyecto especial de tres años de duración bajo el patrocinio de la U.S. Agency for International Development (AID) para mejorar el transporte rural en los países en desarrollo acrecentando la disponibilidad de la información en existencia sobre el planeamiento, diseño, construcción, y manutención de caminos de bajo volúmen. Con el consejo y dirección de un comité de iniciativas para el proyecto, el TRB definirá, producirá, y transmitirá productos informativos a través de una red de corresponsales en países en desarrollo. Las metas generales para el impacto final del trabajo del proyecto son la promoción del uso efectivo de la información en existencia en el desarrollo económico de la infraestructura de transporte y de esta forma mejorar otros aspectos del desarrollo rural a través del mundo.

Además de la recolección y distribución de la información técnica, se proverán acciones recíprocas personales con los usuarios por

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de développement. Certains aspects de cette technologie ont été documentés dans des articles ou rapports écrits par des experts. Mais une grande partie des connaissances n'éxiste que dans l'esprit de ceux qui ont développé et appliqué cette technologie par nécessité. De plus, dans ces deux cas, les écrits et connaissances sur la technologie des routes à faible capacité, sont dispersés géographiquement, sont écrits dans des langues différentes, et ne sont pas assez aisément accessibles pour être appliqués aux besoins des pays en voie de développement.

En octobre 1977, le Transportation Research Board (TRB) initia ce projet, d'une durée de 3 ans, sous le patronage de l'U.S. Agency for International Development (AID), pour améliorer le transport rural dans les pays en voie de développement, en rendant plus accessible la documentation éxistante sur la conception, le tracé, la construction, et l'entretien des routes à faible capacité. Avec le conseil, et sous la conduite d'un Comité de Direction, TRB définira, produira, et transmettra cette documentation à l'aide d'un réseau de correspondants dans les pays en voie de développement. Généralement, l'aboutissement final de ce projet sera de favoriser l'utilisation de cette documentation, pour aider au développement économique de l'infrastructure des transports, et de cette façon mettre en valeur d'autres aspects d'exploitation rurale à travers le monde.

En plus de la dissémination de cette documentation technique, des visites, des conférences aux Etats Unis et à l'étranger, et other forms of communication.

Steering Committee

The Steering Committee is composed of experts who have knowledge of the physical and social characteristics of developing countries, knowledge of the needs of developing countries for transportation, knowledge of existing transportation technology, and experience in its use.

Major functions of the Steering Committee are to assist in the definition of users and their needs, the definition of information products that match user needs, and the identification of informational and human resources for development of the information products. Through its membership the committee provides liaison with project-related activities and provides guidance for interactions with users. In general the Steering Committee gives overview advice and direction for all aspects of the project work.

The project staff has responsibility for the preparation and transmittal of information products, the development of a correspondence network throughout the user community, and interactions with users.

Information Products

Three types of information products are prepared: compendiums of documented information on relatively narrow topics, syntheses of knowledge and

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medio de visitas de campaña, conferencias en los Estados Unidos de Norte América y en el extranjero, y otras formas de comunicación.

Comité de Iniciativas

El Comité de Iniciativas se compone de expertos que tienen conocimiento de las características físicas y sociales de los países en desarrollo, conocimiento de las necesidades de transporte de los países en desarrollo, conocimiento de la tecnología de transporte en existencia, y experiencia en su uso.

Las funciones importantes del Comité de Iniciativas son las de asistir en la definición de usuarios y sus necesidades, de productos informativos que se asemejan a las necesidades del usuario, y la identificación de recursos de conocimientos y humanos para el desarrollo de los productos informativos. A través de sus miembros el comité provee vínculos con actividades relacionadas con el proyecto y también una guía para la interacción con los usuarios. En general el Comité de Iniciativas proporciona consejos y dirección general para todos los aspectos del trabajo de proyecto.

El personal de proyecto tiene la responsabilidad para la preparación y transmisión de los productos informativos, el desarrollo de una red de corresponsales a través de la comunidad de usuarios, y la interacción con los usuarios.

Productos Informativos

Se preparan tres tipos de productos informa-

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d'autres formes de communication permettront une interaction constante avec les usagers.

Comité de Direction

Le Comité de Direction est composé d'experts qui ont à la fois des connaissances sur les caractéristiques physiques et sociales des pays en voie de développement, sur leurs besoins au point de vue transports, sur la technologie actuelle des transports, et ont aussi de l'expérience quant à l'utilisation pratique de cette technologie.

Les fonctions majeures de ce comité sont d'abord d'aider à définir les usagers et leurs besoins, puis de définir leurs besoins en matière de documentation, et d'identifier les ressources documentaires et humaines nécessaires pour le développement de cette documentation. Par l'intermédiaire de ses membres, le comité pourvoit à la liaison entre les différentes fonctions relatives au projet, et dirige l'interaction avec les usagers. En général, le Comité de Direction conseille et dirige toutes les phases du projet.

Le personnel attaché à ce projet est responsable de la préparation et de la dissémination des documents, du développement d'un réseau de correspondants pris dans la communauté d'usagers, et de l'interaction avec les usagers.

La Documentation

Trois genres de documents sont preparés:

practice on somewhat broader subjects, and proceedings of low-volume road conferences that are totally or partially supported by the project. Compendiums are prepared by project staff at the rate of about 12 per year; consultants are employed to prepare syntheses at the rate of 2 per year. At least two conference proceedings will be published during the 3-year period. In summary, this project aims to produce and distribute between 40 and 50 publications that cover much of what is known about low-volume road technology.

Interactions With Users

A number of mechanisms are used to provide interactions between the project and the user community. Project news is published in each issue of *Transportation Research News.* Feedback forms are transmitted with the information products so that recipients have opportunity to say how the products are beneficial and how they may be improved. Through semiannual visits to developing countries, the project staff acquires first-hand suggestions for the project work and can assist directly in specific technical problems. Additional opportunities for interaction with users arise through international and in-country conferences in which there is project participation. Finally, annual colloquiums will be held for students from developing countries who are enrolled at U.S. universities.

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tivos: los compendios de la información documentada sobre relativamente limitados temas, la síntesis del conocimiento y práctica sobre temas un poco mas ámplios, y los expedientes de conferencias de caminos de bajo volúmen que están totalmente o parcialmente amparados por el proyecto. El personal de proyecto prepara los compendios a razón de unos 12 por año; se utilizan consultores para preparar las síntesis a razón de 2 por año. Se publicarán por lo menos dos expedientes de conferencias durante el período de tres años. En breve, este proyecto pretende producir y distribuir entre 40 y 50 publicaciones que cubren mucho de lo que se conoce de la tecnología de caminos de bajo volúmen.

Interacción con los Usuarios

Se utilizan varios mecanismos para proveer las interacciones entre el proyecto y la comunidad de usuarios. Se publican las noticias del proyecto en cada edición de la *Transportation Research News.* Se transmiten formularios de retroacción con los productos informativos para que los recipientes tengan oportunidad de decir cómo benefician los productos y cómo pueden ser mejorados. A través de visitas semianuales a los países en desarrollo, el personal del proyecto adquiere directo de fuentes originales sugerencias para el trabajo del proyecto y puede asistir directamente en problemas técnicos

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des recueils dont le sujet sera relativement limité, des synthèses de connaissances et de pratique sur des sujets beaucoup plus généraux, et finalement des comptes-rendus de conférences sur les routes à faible capacité qui seront organisées complètement ou en partie par ce projet. Environ 12 recueils par an sont preparés par le personnel attaché au projet. Deux synthèses par an sont écrites par des experts. Les comptes-rendus d'au moins deux conférences seront écrits dans une période de 3 ans. En résumé, l'objet de ce projet est de produire et disséminer entre 40 et 50 documents qui couvriront l'essentiel des connaissances sur la technologie des routes à faible capacité.

Interaction Avec les Usagers

Un certain nombre de mécanismes sont utilisés pour assurer l'interaction entre le personnel du projet et la communauté d'usagers. Un bulletin d'information est publié dans chaque numéro de Transportation Research News. Des formulaires sont joints aux documents, afin que les usagers aient l'opportunité de juger de la valeur de ces documents et de donner leur avis sur les moyens de les améliorer. Au cours de visites semi-annuelles dans les pays en voie de développement, le personnel obtient de première main des suggestions sur le bon fonctionnement du projet et peut aider à résoudre sur place certains

Bibliography

Rather than listing the hundreds of relevant articles, reports, and books, the bibliographic section of each of the compendiums will list only those publications from which text has been selected or those basic publications that would have been selected had there been no restraints on the number of pages of selected text.

Index

The index in each compendium will make the specific elements of the wide assortment of information in each compendium quickly available to users. Each index will be a composite of subject terms, authors, and organizations whose work is relevant to the topic.

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específicos. Surgen oportunidades adicionales para la interacción con los usuarios a través de conferencias internacionales y nacionales en donde participa el proyecto. Finalmente, habrán diálogos con estudiantes de países en desarrollo que están inscriptos en universidades norteamericanas.

Bibliografía

En vez de numerar los cientos de artículos, informes, y libros pertinentes, la sección bibliográfica de cada uno de los compendios nombrará unicamente aquellos publicaciones básicas que hubieran sido seleccionadas si no hubiera un límite en el número de páginas de texto seleccionado.

Indice

El índice en cada compendio proporcionará al usuario una forma rápida de utilizar los elementos específicos de la gran variedad de información comprendida en él. Cada índice será un compuesto de tema, términos, autores, y organizaciones, cuyo trabajo sea pertinente al tema del compendio.

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problèmes techniques spécifiques. En outre, des conférences tenues soit aux Etats Unis, soit à l'étranger, sont l'occasion d'un échange d'idées entre le personnel et les usagers. Finalement, des colloques annuels sont organisés pour les étudiants des pays en voie de développement qui étudient dans les universités americaines.

Bibliographie

Plutôt que d'énumérer les centaines de livres, rapports, et articles qui touchent au sujet, la section bibliographique de chaque recueil comprendra seulement les documents dont une partie a été sélectionnée pour les Textes Choisis ou les documents de base qui auraient été choisis s'il n'y avait existé aucune restriction quant au nombre de pages.

Index

L'index de chaque recueil permettra au lecteur de trouver facilement et rapidement l'élément spécifique qu'il désire. Chaque index comprendra des mots-clés et des noms d'auteurs ou d'organisation dont les ouvrages touchent le sujet de ce recueil.

Foreword and Acknowledgments

This compendium is the initial product of the Transportation Research Board's project on Transportation Technology Support for Developing Countries under the sponsorship of the U.S. Agency for International Development. The objective of this book is that it provide useful and practical information for those in developing countries who have direct responsibility for the geometric design of low-volume roads. Feedback from correspondents in developing countries will be solicited and used to assess the degree to which that objective has been attained and to influence the nature of later products.

Acknowledgment is made to the following publishers for their kind permission to reprint the selected text portions of this compendium:

American Association of State Highway and Transportation Officials, Washington, D.C.;
International Bank for Reconstruction and Development, Washington, D.C.;
National Association of Australian State Road Authorities, Sydney;
Rural and Urban Roads, Scranton Publishing Co., Chicago;
Butterworth & Co. (Publishers) Ltd., London;
U.S. Department of Transportation, Washington, D.C.; and
U.S. Forest Service, Washington, D.C.

Prefacio y Agradecimientos

Este compendio es el producto inicial del proyecto del Transportation Research Board sobre Apoyo de Tecnología de Transporte para Países en Desarrollo bajo el patrocinio de la U.S. Agency for International Development. El objetivo de este libro es el de proveer información útil y práctica para aquellos en países en desarrollo quienes tienen responsabilidad directa por el diseño geométrico de caminos de bajo volúmen. Se pedirá a los corresponsales en los países en desarrollo información sobre los resultados, para utilizarse en el asesoramiento del grado al cuál se ha obtenido ese objetivo y para influenciar la naturaleza de productos subsequentes.

Se reconoce a los siguientes editores por el permiso dado para re-imprimir las porciones de texto seleccionadas de este compendio:

American Association of State Highway and Transportation Officials, Washington, D.C.;
International Bank for Reconstruction and Development, Washington, D.C.;
National Association of Australian State Road Authorities, Sydney;
Rural and Urban Roads, Scranton Publishing Co., Chicago;
Butterworth & Co. (Publishers) Ltd., London;
U.S. Department of Transportation, Washington, D.C.; and
U.S. Forest Service, Washington, D.C.

Avant-propos et Remerciements

Ce recueil représente le résultat initial du projet du Transportation Research Board sur la Technologie des Transports à l'Usage des Pays en Voie de Développement. Ce projet est placé sous le patronage de l'U.S. Agency for International Development. L'objet de ce recueil est de réunir une documentation pratique et utile qui puisse aider les responsables du dimensionnement géométrique des routes à faible capacité. La réaction des correspondants des pays en voie de développement sera sollicitée et utilisée pour évaluer à quel point le but proposé de ce projet a été atteint et pour influencer la nature des ouvrages à venir.

Nous remercions les éditeurs qui ont gracieusement donné leur permission de reproduire les textes sélectionnés pour ce recueil:

American Association of State Highway and Transportation Officials, Washington, D.C.;
International Bank for Reconstruction and Development, Washington, D.C.;
National Association of Australian State Road Authorities, Sydney;
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Butterworth & Co. (Publishers) Ltd., London;
U.S. Department of Transportation, Washington, D.C.; and
U.S. Forest Service, Washington, D.C.

Appreciation is also expressed to libraries and information services that provided references and documents from which final selections were made for the Selected Texts and Bibliography in this compendium. Special acknowledgment is made to the U.S. Department of Transportation Library Services Division and to the Library and Information Service of the U.K. Transport and Road Research Laboratory.

Finally, the Transportation Research Board acknowledges the valuable advice and direction that have been provided by the Steering Committee and is especially grateful to Clarkson H. Oglesby, Edward C. Sullivan, and Eldon J. Yoder, who provided special assistance on this particular compendium.

También se reconoce a las bibliotecas y servicios de información que proveen las referencias y documentos de los cuales se hacen las selecciones finales para los Textos Seleccionados y la Bibliografía en este compendio. Se hace un especial reconocimiento a la Library Services Division de la U.S. Department of Transportation y el Library and Information Service de la U.K. Transport and Road Research Laboratory.

Finalmente, el Transportation Research Board agradece el consejo y dirección valiosos provisto por el Comité de Iniciativas, con especial reconocimiento a los señores Clarkson H. Oglesby, Edward C. Sullivan, y Eldon J. Yoder, que prestaron ayuda especial para este compendio en particular.

Nos remerciements aussi aux bibliothèques et bureaux de documentation qui nous ont fourni les documents et les références utilisés dans les Textes Choisis et Bibliographie de ce recueil. Nous remercions spécialement la U.S. Department of Transportation Library Services Division et les Library and Information Service of the U.K. Transport and Road Research Laboratory.

Finalement, le Transportation Research Board reconnait la grande valeur de la direction et de l'assistance des membres du Comité de Direction et les remercie de leur concours et de la façon dont ils dirigent le projet, spécialement Messieurs Clarkson H. Oglesby, Edward C. Sullivan, et Eldon J. Yoder, qui ont bien voulu prêter leur assistance à la préparation de ce recueil.

Overview

Background and Scope

Geometric design is the art and science of determining the geometry of a roadway. Through experience, the highway engineering profession has learned that geometric design is not an exact science but that certain basic concepts and their application to geometric design can be standardized within various parameters. The parameters that affect the application of these basic concepts include (a) anticipated volume and composition of traffic to be served, (b) features of the terrain along the route selected, (c) intended quality of service to be provided, and (d) available funding. Once these parameters are determined, it is possible to establish a certain amount of standardization of design to ensure a satisfactory finished product.

Geometric design standards are determined before preliminary design of a roadway begins because they affect the route location and the preliminary cost estimate. Costs in turn affect the feasibility of continuing the design process and eventually the priority of the road construction. It is obvious then that a complete set of design standards must accommodate variation in at least the first three major parameters described in order to

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Vista General

Antecedentes y Alcance

El diseño geométrico es el arte y la ciencia de determinar la geometría de un camino. De larga experiencia los profesionales de la ingeniería vial han determinado que el diseño geométrico no es una ciencia exacta pero que ciertos conceptos básicos y su aplicación al diseño geométrico pueden ser uniformados dentro de varios parámetros. Los parámetros que afectan la aplicación de estos conceptos básicos incluyen (a) volúmen anticipado y composición del tránsito a servirse, (b) características del terreno sobre la ruta seleccionada, (c) calidad del servicio que se propone proveer, y (d) fondos disponibles. Una vez determinados estos parámetros, es posible establecer una cierta uniformidad o norma de diseño para asegurar un producto final satisfactorio.

Las normas de diseño geométricos se determinan antes de que comience el diseño preliminar de un camino porque afectan la ubicación de la ruta y la estimación preliminar de costo. A su vez, los costos afectan la posibilidad de continuar el proceso de diseño y eventualmente la prioridad de la construcción del camino. Está claro entonces que una serie completa de normas de diseño debe permitir una variación en por lo menos los primeros tres parámetros mas importantes descriptos con el propósito de ser útiles para

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Exposé

Historique et Description

Le dimensionnement géométrique est l'art et la science de déterminer géométriquement une route. Le génie civil a appris par expérience que le dimensionnement géométrique n'est pas une science exacte mais que certains concepts de base et leur application au dimensionnement géométrique peuvent être standardisés selon certains paramètres. Les paramètres qui influencent l'application de ces concepts de base sont: (a) le volume et la composition du trafic prévus, (b) les particularités de terrain le long de l'itinéraire sélectioné, (c) la qualité de service projetée, et (d) les moyens financiers disponibles. Une fois que ces paramètres sont determinés, il devient possible d'établir une certaine standardisation du dimensionnement qui garantit un resultat satisfaisant.

Les normes de dimensionnement géometrique sont déterminées avant le tracé préliminaire de la route parce qu'elles influencent directement son emplacement et l'estimation de son coût. Le coût à son tour influence la possibilité de continuer le processus d'étude et finalement les priorités de construction de cette route. Il est donc évident que l'ensemble de ces normes de dimensionnement doit pouvoir accommoder quelques variations, spécialement en ce qui concerne les trois paramètres les plus importants cités au be useful for a range of possible situations.

The first parameter, volume and composition of traffic, is treated by categorizing geometric design standards according to traffic volume increments, expressed as average daily traffic (ADT) or design hourly volume (DHV), for a particular yearly time period, either the year in which the road is first opened to traffic or some future year chosen for design purposes. This compendium does not address the methods of determining the traffic volume, but it does address the geometric design standards for specific traffic volumes once the volumes have been determined.

Since various definitions of low-volume roads exist, one specific definition must be adopted, however arbitrarily, in order that the writer and the reader may meet on common ground. The following definition of a low-volume rural road will be used throughout this series of publications:

Two levels of traffic volume can be identified: less than 50 ADT (class 1) and 50 to 400 ADT (class 2). Class 1 roads are generally unsurfaced or are graded in situ. Class 2 roads generally have granular surfaces, such as gravel, crushed stone, laterite, or stabilized soil; they generally do not have a high type of paved surface but, at the upper traffic volumes, may be primed, have single surface treatments, or be chip sealed.

The second parameter, terrain topography, is addressed by matching the geometric design stan-

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una variedad de situaciones posibles.

El primer parámetro, que es el volúmen y la composición del tránsito, se trata categorizando las normas de diseño geométrico de acuerdo con incrementos de volúmen de tránsito, representados como tránsito medio diario (ADT), o volúmen por hora de diseño (DHV), para un cuadro temporal anual en particular, en el año en que primero se abre el camino al tránsito, o en algún año futuro escogido por propósitos de diseño. Este compendio no se dedica a los métodos para determinar el volúmen de tránsito, pero sí trata sobre las normas de diseño geométricas para volúmenes de tránsito específicos una vez que se han determinado los volúmenes.

Ya que existen varias definiciones de caminos de bajo volúmen, se debe adoptar una definición específica, sin importar su arbitrariedad, con el propósito de que el escritor y el lector puedan encontrarse sobre terreno común. Se utilizará la siguiente definición para un camino rural de bajo volúmen en toda esta serie de publicaciones:

Se pueden identificar dos niveles de volúmen de tránsito: menos que 50 ADT (clase 1) y de 50 a 400 ADT (clase 2). Los caminos de clase 1 generalmente tienen superficie de tierra o son nivelados en situ. Los caminos de la clase 2 generalmente tienen una superficie granular, tal como ripio, piedra triturada, laterita, o tierra estabilizada; generalmente no tienen un tipo elevado de superficie pavimentado pero, en los volúmenes mas altos de tránsito, pueden ser tratados con capa de imprimación, selladora bituminos especial, o asfalto y agregado.

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debut de ce paragraphe, afin d'être valable pour un éventail de situations. Le premier paramètre, volume et composition du trafic, est traité en classant les normes de dimensionnement géométrique selon le volume de trafic, exprimé comme ADT (average daily traffic), le trafic moyen journalier, ou DHV (design hourly volume), trafic horaire de référence, pour une certaine année, soit la première année où la route est ouverte au trafic, soit une année future choisie arbitrairement. Ce recueil ne traite pas des méthodes utilisées pour déterminer le volume de trafic, mais il traite des normes de dimensionnement géométriques pour une valeur spécifique de volume de trafic une fois que ce volume a été déterminé. Il existe plusieurs définitions de route à faible capacité, donc une définition spécifique doit être adoptée de façon arbitraire peut-être, mais qui permettra à l'auteur et au lecteur de se comprendre. Voici donc la définition d'une route à faible capacité qui sera utilisée dans toute cette série de publications.

Deux niveaux de volume de trafic peuvent être identifiés: moins de 50 ADT (classe 1) et de 50 à 400 ADT (classe 2). Les routes de la classe 1 sont généralement le terrain naturel ou sont nivelées sans apport de matériaux. Les routes de la classe 2 ont des revêtements, soit graveleux, soit roche concassée, soit latérite ou sol stabilisé; en général, elles n'ont pas un revêtement de qualité supérieure, mais pour les limites maximales de volume de trafic elles peuvent être imprégnées ou avoir un enduit superficiel monocouche ou être gravillonnées.

dards to a generalized description of the terrain to be traversed. The usual categories are flat, rolling, and mountainous; they pertain to the actual route location chosen. For example, if a portion of a route follows a flat valley between two mountains, the terrain classification of concern for design is flat. The terrain classification may change several times along a route. Each change should be made at an obvious (to the vehicle operator) location where the route itself goes from one topographic classification to another. Since this decision is based on judgment and is intended to reduce construction costs in difficult areas, this compendium does not attempt to define more precisely the limits of each classification. References for proven methods to make the transition between areas of different design speeds on low-volume roads are lacking.

The third parameter, quality of service, here refers to the measure of safe speed, the limiting gradients, the general safety and comfort of the road user, and the relative economy of vehicle operation. This parameter is characterized by the use of various design speeds in geometric design standards. Design speeds vary as a function of route topography. Their primary use is to ensure that all elements of geometric design are harmonious and economical.

El segundo parámetro, que es la topografía del terreno, se trata igualando las normas de diseño geométrico a una descripción generalizada del terreno a cruzarse. Las categorías comunes son llano, ondulado, y montañoso; se refieren a la ubicación de ruta que realmente se ha escogido. Por ejemplo, si la porción de una ruta atraviesa una valle llano entre dos montañas, la clasificación del terreno que afecta el diseño sería llano. La clasificación del terreno puede cambiar varias veces a lo largo de una ruta. Cada cambio debe ocurrir en una ubicación óbvia (para el operador del vehículo) donde la ruta misma vá de una clasificación topográfica a otra. Ya que esta decisión se basa sobre opinión y es para reducir los costos de construcción en áreas difíciles, este compendio no tratará de definir mas precisamente los límites de cada clasificación. Están faltando referencias para métodos comprobados que establezcan la transición entre áreas de distintas velocidades de diseño en caminos de bajo volúmen.

El tercer parámetro, calidad de servicio, aquí se refiere a la medida de velocidad segura, las pendientes restrictivas, la seguridad y comodidad general del usuario del camino, y la economía relativa de operación del vehículo. Este parámetro se caracteriza por el uso de varias velocidades de diseño en normas de diseño geométrico. Las velocidades de diseño varían en función de la topografía de la ruta. Su principal utilidad es la de asegurar que todos los elementos del diseño geométrico son proporcionados y económicos. La asignación de una velocidad de diseño en

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Le deuxieme paramètre, la topographie, est traité en assortissant les normes de dimensionnement à une description générale du terrain à traverser. Les catégories ordinaires de terrain sont terrain plat, terrain vallonné, et terrain montagneux. Elles se rapportent à l'emplacement actuel de la route. Par exemple, si une section de la route suit une vallée plate entre deux montagnes, pour le dimensionnement cela sera considéré comme terrain plat. Bien entendu, le terrain peut changer de catégorie plusiers fois suivant le tracé de la route. Chaque changement devrait être fait à un endroit évident (au conducteur du vehicule) où la topographie de la route change d'une catégorie à une autre. Puisque cette décision est une question de jugement et a pour objet de réduire les frais de construction dans les terrains difficiles, ce recueil ne définira pas plus précisement les limites de chaque catégorie. Il est certain que les ouvrages de référence qui traiteraient de méthodes éprouvées pour faire la transition d'une catégorie de terrain à une autre dans le cas des routes à faible capacité sont singulièrement absents.

Le troisième paramètre, la qualité de service, ici s'en réfère à la vitesse de securité, le degré des pentes, et généralement tout ce qui affecte la securité et le confort de l'usager de la route et qui rend plus économique la conduite. Ce paramètre est caractérisé par l'usage de différentes vitesses de base dans les normes de dimensionnement géométriques. Les vitesses de base varient en fonction de la topographie. Leur principale qualité est

Assignment of a particular design speed to a route automatically sets limits on many of the geometric features of that roadway. Determination of the design speed of a route is perhaps the most arbitrary decision that is made in the preliminary evaluation of a proposed road, yet it is the fundamental determinant of the cost of road construction and maintenance. Since the subject of this project is low-volume roads, it follows that the subject is also low-cost roads. Some of the selected texts that follow address the relationship of costs to design speeds. Those texts urge the highway engineer to use a realistic design speed in order to get the most economical low-volume road that will meet the needs of the users. It must be recognized that on low-volume roads, since the design speeds are often low, operating speeds may substantially exceed design speeds on tangent sections.

Once the above parameters have been established, they can be applied to the basic concepts of geometric design. The results can then be applied over a wide range of conditions with assurance that the end product will be an economical and satisfactory design. These organized basic concepts are termed geometric design standards and are the subject of this compendium.

Many countries have developed geometric design standards of various degrees of complexity. These design standards classify roadway types either in an administrative sense, i.e., national roads, provincial

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particular a una ruta automaticamente fija límites sobre muchas de las características de aquel camino. Quizás la determinación de la velocidad de diseño de una ruta es la decisión mas arbitraria que se hace en la evaluación preliminar de un camino propuesto, sin embargo es el determinante fundamental del costo de la construcción y manutención del camino. Ya que el tema de este proyecto es caminos de bajo volúmen, sigue que el tema también es caminos de bajo costo. Algunos de los textos seleccionados que siguen tratan sobre la relación de costos y velocidades de diseño. Estos textos piden al ingeniero vial que utilice una velocidad de diseño práctico con el propósito de obtener el camino de bajo volúmen más económico que satisfaga las necesidades de los usuarios. Deberá reconocerse que en los caminos de bajo volúmen, ya que las velocidades de diseño son seguidad veces bajas, las velocidades operativas excederán substancialmente a las de diseño en secciones tangentes.

Una vez que se han establecido los parámetros mencionados arriba, pueden ser aplicados a los conceptos básicos de diseño geométrico. Los resultados entonces pueden aplicarse sobre una gran variedad de condiciones con la seguridad de que el producto final será un diseño económico y satisfactorio. Estos conceptos básicos organizados se llaman normas de diseño geométrico y forman el tema de este compendio.

Muchos países poseen normas de diseño geométricos de varios grados de complejidad. Estas normas de diseño clasifican tipos de

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d'assurer que tous les éléments de dimensionnement géométrique soient à la fois harmonieux et économiques. L'attribution d'une certaine vitesse de base à une route, automatiquement limite certains aspects de son dimensionnement géométrique. Cette détermination de la vitesse de base est sans doute la décision la plus arbitraire que l'on puisse prendre au moment de l'évaluation préliminaire d'une route proposée et, pourtant, cette décision est fondamentale et déterminera les frais de construction et d'entretien de cette route.

Le thème de ce projet est la construction de routes à faible capacité et, par conséquent, une construction qui ne soit pas dispendieuse. Certains des textes choisis pour ce recueil traitent du lien entre le coût de la construction et les vitesse de base. Ces textes encouragent l'ingénieur routier à être réaliste dans son choix de vitesse de base, de façon à construire de la façon la plus économique, une route à faible capacité qui donnera satisfaction aux usagers. On doit aussi reconnaître que sur les routes à faible capacité, dont les vitesses de base sont souvent basses, les vitesses de conduite réelles excèdent souvent de façon substantielle ces vitesses de base, surtout en ligne droite.

Une fois que tous ces paramètres ont été établis, ils peuvent être appliqués aux concepts de base du dimensionnement géometrique. Les résultats, alors, peuvent être appliqués à toute une gamme de conditions différentes, avec l'assurance d'aboutir à une solution finale économique et satisfaisante. L'ensemble de ces concepts de base s'appelle les normes de dimensionnement géométrique et forment roads, and so on, or by the level of use, i.e., major routes, feeder roads, and so on. The second classification depends on traffic volume and is the one used throughout this compendium.

This compendium was prepared after a search was made of the existing literature on geometric design standards in use throughout the world. As will become apparent, there are many variations. The earliest standards were modified, expanded, contracted, or discarded by those who came after to reflect the expanding data base of observations and research and to make general design standards fit specific problems.

Texts that were selected to be reprinted in full or in excerpted portions were chosen to illustrate (a) the parent stock from which today's geometric design standards are drawn, (b) the modifications that have evolved as it became apparent that low-volume roads required separate geometric design criteria, and (c) the increasing realization that design criteria and construction costs are constant companions.

The inclusion of a basic text does not imply that the geometric design standard policies of the issuing agency have not changed or will not change in the future. In fact, two publications from each of two agencies are included specifically to show the con-

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camino en forma administrativa, es decir, caminos nacionales, provinciales, y así sucesivamente, o por el nivel de uso, es decir, rutas principales, caminos vinculares, etc. La segunda clasificación depende de volúmenes de tránsito y es el que se utiliza en este compendio.

El compendio se preparó después de realizarse una búsqueda de toda la literatura existente sobre normas de diseño geométrico utilizados a través del mundo. Será aparente que hay muchas variaciones. Las primeras normas fueron modificadas, extendidas, reducidas, o descartadas por las subsiguientes para reflejar la base de datos de observaciones e investigación en incremento y para ajustar las normas de diseño generales a problemas específicos.

Los textos que fueron seleccionados para re-imprimición en toto o en porciones extraídas fueron escogidos para ilustrar (a) los temas originales de los cuales se extrayeron las normas de diseño geométrico de hoy día, (b) las modificaciones que resultaron cuando se comprendió que los caminos de bajo volúmen necesitaban criterios separados de diseño geométrico, y (c) que los criterios de diseño y costos de construcción siempre acompañan uno al otro.

La inclusión de un texto básico no implica que las políticas de las normas de diseño geométrico de la agencia que lo pública no ha cambiado o que no cambiará en el futuro. En realidad, se han incluido dos publicaciones de cada una de dos agencias especificamente para demostrar la revaluación contínua de la

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le thème de ce recueil.

De nombreux pays ont developpé des normes de dimensionnement géométrique plus ou moins complexes. Ces normes de dimensionnement classent les différentes sortes de routes soit du point de vue administratif, c'est á dire route nationales, routes départementales, etc., soit selon le niveau d'utilisation, c'est a dire route à grande circulation, pistes rurales, etc. La deuxième classification dépend du volume de trafic, et c'est celle qui est adoptée pour ce recueil.

Des recherches sur toute la littérature existante à travers le monde sur les normes de dimensionnement géométrique ont été faites pour les besoins de ce recueil. Il devient très vite apparent qu'il y a beaucoup de variations sur ce sujet. Les normes des plus anciens ouvrages ont été modifiées, développées, abrégées, ou rejetées au fur et à mesure de l'accroissement des connaissances acquises et de l'adaptation de ces normes générales à des problèmes spécifiques.

Les textes que nous avons décidé de reproduire, en partie ou en entier, dans cet ouvrage ont été choisis pour illustrer: (a) les concepts de base d'où ont été tirées les normes de dimensionnement géométriques d'aujourd'hui, (b) les modifications qui ont été faites quand il est devenu apparent que les routes à faible capacité demandaient des critères de dimensionnement géométrique spécifiques, et (c) le fait, de plus en plus évident, que les critères de dimensionnement et le coût de la construction vont de pair.

Le fait d'inclure un texte de base dans ce recueil n'indique pas nécéssairement que les normes de dimensionnement soient rigides et tinuing reappraisal of the relationship of economics to geometric design standards for low-volume roads.

Rationale for This Compendium

The three books dealing with geometric design standards that have probably received the widest distribution in the countries for which this compendium is written are, in order of original publication date,

1. *A Policy on Geometric Design of Rural Highways,* 1954, American Association of State Highway Officials (AASHO)—now the American Association of State Highway and Transportation Officials (AASHTO);
2. *A Guide to Highway Design Standards,* 1957, International Bank for Reconstruction and Development (World Bank); and
3. *Low-Cost Roads, Design, Construction, and Maintenance,* 1967, United Nations Educational, Scientific, and Cultural Organization (UNESCO).

The first of these, known also as the Blue Book, owes its wide distribution to its early publication and depth of coverage. The second book was published by one of the larger lending institutions in the world and, before it went out of print, was available in any country that received highway

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relación entre la economía y las normas de diseño geométrico en caminos de bajo volúmen.

Exposición Razonada Para Este Compendio

Los tres libros que tratan sobre las normas de diseño geométrico que probablemente han recibido la mas grande distribución en los países para los cuales se ha escrito este compendio son, en el órden de fecha de publicación original,

1. *A Policy on Geometric Design of Rural Highways* (Política Sobre el Diseño Geométrico en Caminos Rurales), 1954, American Association of State Highway Officials (AASHO)—ahora el American Association of State Highway and Transportation Officials (AASHTO);
2. *A Guide to Highway Design Standards* (Guía Para Normas de Diseño Vial), 1957, International Bank for Reconstruction and Development (Banco Mundial); y
3. *Low-Cost Roads: Design, Construction, and Maintenance* (Caminos de Bajo Costo: Diseño, Construcción, y Manutención), 1967, United Nations Educational, Scientific, and Cultural Organization (UNESCO).

El primero de éstos, que también se conoce como el Libro Azul, debe su amplia distribución a su temprana publicación y la penetración de su campo de aplicación. El segundo libro fué publicado por una de las instituciones mayores de préstamo en el mundo y, antes de agotarse, estaba disponible en cualquier país que recibía fondos viales. El tercer libro, que

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ne doivent pas être changées dans l'avenir. En fait, deux publications citées ici, de deux organismes différents, sont incluses spécifiquement pour montrer le réexamen constant du rapport existant entre le dimensionnement géométrique et le coût d'une route à faible capacité.

Objet de ce Recueil

Les trois livres qui traitent des normes de dimensionnement géométrique qui ont reçu la plus grande audience dans les pays pour lesquels ce recueil est écrit sont dans l'ordre chronologique,

1. *A Policy on Geometric Design of Rural Highways* (Methode sur le Dimensionnement Géometrique des Routes Rurales), 1954, American Association of State Highway Officials (AASHO)—devenue American Association of State Highway and Transportation Officials (AASHTO);
2. *A Guide to Highway Design Standards* (Un Guide de Normes de Dimensionnement Pour Routes), 1957, International Bank for Reconstruction and Development (Banque Mondiale); et
3. *Low-Cost Roads: Design, Construction, and Maintenance* (Routes Dans les Pays en Voie de Développement: Conception, Construction, et Entretien), 1967, United Nations Educational, Scientific, and Cultural Organization (UNESCO).

Le premier livre, appelé aussi le Livre Bleu, doit sa répartition à peu près universelle au fait qu'il est un des premiers à avoir été publié funding. The third book, which has been published in both French and English, has been available to all member nations of the United Nations, a larger group of countries than would normally be reached by the first two volumes.

The AASHO Blue Book was revised in 1965, and several supplementary volumes dealing with certain aspects of the same subject have been issued from time to time. The World Bank published a sequel to its 1957 book in 1975. This publication, *A Review of Highway Design Practices in Developing Countries,* is essentially a review of the design standards of some 150 highway projects financed by the World Bank between 1960 and 1970. The UNESCO book covers a wider range of highway-related subjects than the other publications; its treatment of geometric design is therefore, of necessity, somewhat more condensed.

The purpose of this compendium and the entire series of publications that is to follow is to enhance rural transportation in developing countries by providing improved access to existing information about the planning, design, construction, and maintenance of low-volume roads. The basic texts are not consistent in defining low-volume rural roads.

The definition of low-volume roads adopted for this project substantially agrees with the classification used in *A Review of Highway Design Practices*

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ha sido publicado en francés e inglés, ha estado disponible a todas las naciones miembros de las Naciones Unidas, un grupo mas grande de países que normalmente hubieran sido alcanzados por los dos primeros volúmenes.

El Libro Azul de AASHO ha sido revisada en 1965, y se han publicado de vez en cuando varios volúmenes suplementarios que tratan sobre varios aspectos del mismo tema. El Banco Mundial publicó una continuación a su libro de 1957 en 1975. Esta publicación, *A Review of Highway Design Practices in Developing Countries* (Un Repaso de las Prácticas de Diseño Vial en Países en Desarrollo), es esencialmente un repaso de las normas de diseño de unos 150 proyectos viales financiados por el Banco Mundial entre 1960 y 1970. El libro de UNESCO cubre una esfera de actividad mas amplia en temas relacionados con el camino que las otras dos publicaciones; por necesidad, entonces, su tratamiento sobre diseño geométrico es mas condensado.

El propósito de este compendio y la serie entera de publicaciones que seguirá es el de realzar el transporte rural en países en desarrollo proporcionando una mayor accesibilidad a la información en existencia sobre el planeamiento, diseño, construcción, y manutención de caminos de bajo volúmen. Los textos básicos no son consistentes en la definición de caminos rurales de bajo volúmen.

La definición de caminos de bajo volúmen que se adoptó para este proyecto substancialmente está de acuerdo con la clasificación

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et aussi au fait qu'il traite du sujet en profondeur. Le deuxième livre a été écrit par une des plus grande banque du monde et, avant que l'édition ne soit épuisée, était disponible dans tous les pays qui étaient autorisés à emprunter des fonds pour la construction des routes. Le troisième livre, qui a été publié à la fois en français et en anglais, peut être trouvé dans tous les pays membres des Nations Unies, donc dans un plus grand nombre de pays que les deux premiers.

Le Livre Bleu d'AASHO a été révisé en 1965, et plusieurs volumes supplémentaires qui traitent de certains aspects du même sujet ont été publiés. En 1975, la Banque Mondiale a publié une suite à son livre de 1957. Ce livre, intitulé *A Review of Highway Design Practices in Developing Countries* (Une Revue des Pratiques de Dimensionnement des Routes Dans les Pays en Voie de Développement), est essentiellement une revue des normes de dimensionnement de quelques 150 projets de construction de routes financés par la Banque Mondiale entre 1960 et 1970. Le livre de l'UNESCO couvre une plus grand étendue de sujets relatifs à la construction des route que les deux autres. Pour cette raison, les chapitres sur le dimensionnement géométrique sont plus condensés.

Le but de ce recueil et de la série entière de publications à venir, est d'améliorer les transport ruraux dans les pays en voie de développement en rendant plus accessible la documentation existante sur la conception, le dimensionnement, la construction, et l'entretien des routes à faible capacité. Les textes de base ne sont pas cohérents dans leur définition des routes rurales à faible capacité.

in Developing Countries. A further justification for the use of 400 ADT as the cutoff point is offered in ***Economics of Design Standards for Low-Volume Rural Roads*** (National Cooperative Highway Research Program Report 63), published in 1969 by the Highway Research Board (now TRB):

Another reason for adopting the 400-vehicle-per-day cutoff point relates to the AASHO standards, as given in *A Policy on Geometric Design for Rural Highways* (1965). Typical roadway cross sections set forth in these standards change at this traffic volume. Also, only for roads above this break point is the concept of "design hour volume" introduced.

In comparing these books, one immediately notes that the AASHO volume uses the U.S. system of measurement while the World Bank and UNESCO publications use the metric system. The conversion factors of 1 mile = 1.609 km and 1 foot = 0.305 m will suffice for most comparisons. The 1957 World Bank book also uses the degree of curvature as a criterion. Degree of curvature in the AASHO book and in the World Bank book have different meanings that must be carefully observed.

The degree of curvature used in the AASHO book is defined as the central angle subtended by an arc of 100 ft (30.5 m). The degree of curvature used in the World Bank volume is defined as

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que se utiliza en ***A Review of Highway Design Practices in Developing Countries.*** Una justificación adicional para el uso de 400 ADT como el punto de cierre se ofrece en ***Economics of Design Standards for Low-Volume Rural Roads*** (Economía Política de Normas de Diseño Para Caminos Rurales de Bajo Volúmen) (National Cooperative Highway Research Program Report 63), publicado en 1969 por el Highway Research Board (ahora el TRB):

Otra razón por adoptar el 400-vehículos-por-día punto de cierre se relaciona con las normas de AASHO, dadas en ***A Policy on Geometric Design for Rural Highways*** (1965). Las secciones transversales de camino típicas indicadas en estas normas cambian en este volúmen de tránsito. Además, es unicamente para los caminos por encima de este punto de cierre que se introduce el concepto de "volúmen por hora de diseño."

Al comparar estos libros es inmediatamente observable que el libro de AASHO utiliza el sistema de Estados Unidos de Norte América mientras que las publicaciones del Banco Mundial y la UNESCO utilizan el sistema métrico. Los factores de conversión de 1 milla = 1,609 km y 1 pié = 0,305 m será suficiente para casi todas las comparasiones. El libro de 1957 del Banco Mundial también utiliza el grado de curvatura como un criterio. El grado de curvatura en el libro de AASHO y en él del Banco Mundial tiene distintos significados y debe observarse cuidadosamente.

El grado de curvatura utilizado en el libro de AASHO se define como el ángulo central subtendido por un arco de 100 piés (30,5 m).

La définition des routes à faible capacité adoptée par ce projet s'accorde en substance avec les catégories utilisées dans le livre ***A Review of Highway Design Practices in Developing Countries.*** Une justification de plus pour avoir choisi 400 ADT comme limite supérieure peut se trouver dans le livre publié en 1969 par une division du Highway Research Board—maintenant appelé le Transportation Research Board (TRB)—***Economics of Design Standards for Low-Volume Rural Roads*** (Raisons Economiques Pour les Normes de Dimensionnement Pour les Routes Rurales à Faible Capacité) (National Cooperative Highway Research Program Report 63):

Une autre raison pour l'adoption de la limite supérieure de 400 vehicules par jour est liée aux normes AASHO données dans le livre ***A Policy on Geometric Design for Rural Highways*** (1965). Les changements de profil en travers types donnés dans cette méthode apparaissent à ce volume de trafic. De plus, le concept DHV n'est introduit que pour les routes dont le volume est supérieur à 400 ADT.

En comparant ces livres, on notera immédiatement que le livre d'AASHO utilise le système de mesure des Etats Unis, alors que les publications de la Banque Mondiale et de l'UNESCO utilisent le système métrique. Pour la plupart des comparaisons, il suffira de retenir que "1 mile" = 1,609 km et "1 foot" (1 pied) = 0,305 m. L'édition de 1957 du livre de la Banque Mondiale utilise aussi le degré de courbure comme critère. Le degré de courbure dans le livre d'AASHO et le degré de courbure dans le livre de la Banque Mondiale ont des sens différents qu'il faut observer avec soin. La the deflection angle for a 20-m (65.6-ft) chord. Most survey books show the conversion factors that are used to change from the arc definition to the chord definition. These conversion factors are small for large radii and increase as the radii decrease. Neglecting the chord-arc variation for simplicity, we find that in field layout the deflection angle is one-half the central angle; therefore the deflection angle for a 20-m curve is the same as the central angle of a 10-m curve (using the arc definition). For the same radius, the ratio of the degree of curvature used in the World Bank text to the degree of curvature used in the AASHO text is 10 m/100 ft = 32.808 ft/100 ft = 0.328 or about 1 to 3. The degree of curvature shown in the AASHO text should therefore be divided by 3 to find the comparable degree of curvature shown in the World Bank text.

Furthermore, the functions (in meters) of a unit one-degree metric curve (arc definition) are one-tenth of the corresponding functions (in feet) of the unit one-degree foot curve.

Discussion of Selected Texts

The first several items in the Selected Texts are presented in chronological order, beginning with *A Guide to Highway Design Standards,* published by

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El grado de curvatura utilizado en el volúmen del Banco Mundial se define como el ángulo de desviación para una cuerda de 20 m (65,6 piés). Casi todos los libros de deslinde muestran los factores de conversión que se utilizan para cambiar de la definición de arco a la definición de cuerda. Estos factores de conversión son pequeños para grandes radios y aumentan a medida que disminuyen los radios. Poniendo de lado la variación de cuerda-arco por razones de simplicidad, encontramos que en 12 disposición del trazado en campaña el ángulo de desviación es la mitad del ángulo central; por lo tanto, el ángulo de desviación para una curva de 20 m es igual al ángulo central de una curva de 10 m (utilizando la definición de arco). Para el mismo radio, la razón de curvas en el libro del Banco Mundial a las curvas en el libro de AASHO es 10 m/100 piés = 32,808 piés/100 piés = 0,328 o 1 a 3 aproximadamente. El grado de curvatura utilizado en el libro de AASHO debe ser entonces dividido por 3 para llegar al grado de curvatura utilizado en el libro del Banco Mundial.

Además, las funciones (en metros) de la unidad de un grado de la curva métrico (definición de arco) son una décima de las funciones correspondientes (en piés) de la unidad de un grado de la curva en piés.

Presentación de los Textos Seleccionados

Los primeros artículos en los Textos Seleccionados están presentados en órden cronológico, comenzando con *A Guide to Highway Design Standards,* publicado por el Banco Mundial en junio de 1957. Ya que esta publicación

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définition du degré de courbure dans le livre d'AASHO est l'angle central sous-tendu par un arc de 100 pieds (30,5 m). La définition du degré de courbure dans le livre de la Banque Mondiale est l'angle de déflexion pour une corde de 20 m (65,6 pieds). La plupart des carnets de levé indiquent les facteurs de conversion pour passer de la définition qui utilise l'arc à celle qui utilise la corde. Ces facteurs de conversion sont petits pour un large rayon et augmentent au fur et à mesure que le rayon diminue. Pour faciliter les calculs, la variation corde/arc sera ignorée ici, et nous trouvons que dans les tracés de terrain l'angle de déflexion est égal à la moitié de l'angle central—donc l'angle de déflexion pour une courbe de 20 m est équivalent à l'angle central d'une courbe de 10 m (en utilisant la définition de l'arc). Pour le même rayon, le rapport du degré de courbure utilisé dans le livre de la Banque Mondiale au degré de courbure utilisé dans le livre d'AASHO est de 10 m/100 pieds = 32,808 pieds/100 pieds = 0,328 ou à peu près de 1 à 3. Le degré de courbure que l'on trouve dans le livre d'AASHO devrait donc être divisé par 3 pour déterminer le degré de courbure comparable dans le livre de la Banque Mondiale. De plus on sait que la fonction en mètre d'une unité d'un degré de courbe métrique (en définition d'arc) représente le dixième de la fonction correspondante en pieds pour une unité d'un degré-pied.

Discussion des Textes Choisis

Les premiers textes choisis sont présentés en ordre chronologique en commencant par *A Guide to Highway Design Standards,* publié the World Bank in June 1957. Since this publication is currently out of print, the entire section on geometric design is reproduced here for convenience.

The second text is excerpted from ***A Policy on Geometric Design of Rural Highways*** (AASHO, 1965). Chapter 1 includes a summary of the remaining chapters. The material selected for this compendium consists of applicable data taken from the chapter abstracts given in Chapter 1. In addition, various tables, figures, and sections of the text from the chapters abstracted are included for clarification or amplification.

The third text is Chapter 3, Geometric Design, from ***Low-Cost Roads, Design, Construction and Maintenance,*** published by UNESCO in 1967 and translated into English in 1971. The chapter is reprinted in full. In spite of its title, this publication is concerned with roadway volumes up to 5000 vehicles/day, but the lower classifications (secondary roads and feeder roads) are in the range of this project and include the same ADT cutoff point (800 vehicles/day) used to differentiate between primary and secondary routes in the 1957 World Bank text.

The first three texts deal with rural roads in general and, while they note that certain standards may be relaxed for low-volume roads, they do not consider this type of roadway as a unique entity. The fourth text in this compendium is Part I of

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está por el momento agotado, se ha producido aquí la sección entera de diseño geométrico.

El segundo texto fué extraído de ***A Policy on Geometric Design of Rural Highways*** (AASHO, 1965). El primer capítulo incluye un resumen de los capítulos restantes. El material seleccionado para este compendio consiste de datos aplicables tomados de los sumarios de capítulos dados en el Capítulo 1. Además, se han incluído varias tablas, figuras, y secciones del texto de los sumarios de capítulos para clarificación.

El tercer texto es el Capítulo 3, Diseño Geométrico, de ***Low-Cost Roads: Design, Construction, and Maintenance,*** publicado por UNESCO en 1967, y traducido al inglés en 1971. El capítulo se reproduce en toto. A pesar del título, esta publicación trata con volúmenes hasta 5000 vehículos/día, pero las clasificaciones mas bajas (caminos secundarios y vinculares) estan dentro del alcance de este proyecto e incluyen el mismo punto de cierre de ADT (800 vehículos/día) que se utiliza para diferenciar entre rutas primarias y secundarias en el texto del Banco Mundial de 1957.

Los primeros tres textos tratan con caminos rurales en general y, mientras que toman nota de que se pueden remitir ciertas normas para caminos de bajo volúmen, no consideran a este tipo de camino como una entidad única en su género. El cuarto texto de este compendio es la Parte I de ***Geometric Design Guide for Local Roads and Streets*** (Guía Para el Diseño Geométrico de Caminos y Calles Locales) (AASHO, 1971), que se concierne

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par la Banque Mondiale en juin 1957. Comme cette édition est epuisée, la section entière sur le dimensionnement géométrique est reproduite ici.

Le deuxième texte est extrait du livre ***A Policy on Geometric Design of Rural Highways*** (AASHO, édition de 1965). Le premier chapitre contient des résumés de tous les autre chapitres. La matière choisie pour être publiée dans ce recueil consiste en données applicables, sélectionnées dans les résumés du premier chapitre. En outre, des tables, des chiffres, et quelques sections du texte des chapitres analysés sont inclus pour la clarification ou l'amplification.

Le troisième texte est le Chapitre 3, *Geometric Design* (Dimensionnement Géometrique), du livre ***Low-Cost Roads: Design, Construction, and Maintenance,*** publié par l'UNESCO en 1967 et traduit en anglais en 1971. Le chapitre est reproduit en entier. En dépit de son titre, cette publication traite des routes dont la capacité va jusqu'a 5000 véhicules par jour, mais les classements des routes à trafic plus modéré (routes secondaires) sont du domaine de ce projet et utilisent le même ADT (800 vehicules par jour) pour différencier les routes à grande circulation des routes secondaires et pistes rurales du livre de la Banque Mondiale de 1957. Les trois premiers textes sont sur la construction des routes rurales en général et, pendant qu'ils admettent que certaines normes peuvent être réduites pour les routes à faible capacité, ils ne considèrent pas ces routes comme appartenant à une catégorie particulière.

Le quatrième texte de ce recueil est la première partie (Part I) du livre ***Geometric***

Geometric Design Guide for Local Roads and Streets (AASHO, 1971), which concerns itself with road geometry for roadways with a current ADT of up to 750 vehicles/day with accommodation for future expansion.

The fifth text, *A Review of Highway Design Practices in Developing Countries* (World Bank, 1975), reprinted in full, concerns itself with basic rural roads of all capacities. It divides these roads into three categories and further subdivides category II into five classes. The basic concerns of this project are classes I and II of category II and category III.

The purpose of reducing geometric design standards for low-volume roads is basically a matter of economics. In 1970 the Institute for Transportation and Traffic Engineering prepared a report for the U.S. Department of Transportation entitled *Opportunities for Cost Reduction in the Design of Transport Facilities for Developing Regions.* The sixth text in this compendium consists of an excerpt from Chapter 2, Road Cost Analysis and Design Standards, by Lawrence Vance. In this excerpt the author points out the areas in which current AASHO standards seem excessive for initial rural road construction in developing countries, particularly in regard to roadway widths and maximum grades.

The seventh text is an article by W. H. Valentine,

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con la geometría vial para caminos con un ADT actual de hasta 750 vehículos/día con adaptación para un expansión futura.

El quinto texto, *A Review of Highway Design Practices in Developing Countries* (World Bank, 1975), reproducido en toto, se concierne con caminos rurales básicos de todas capacidades. Divide a estos caminos en tres categorías y además subdivide a la categoría II en cinco clases. Los intereses básicos de este proyecto son las clases I y II de la categoría II y la categoría III.

El propósito de reducir las normas de diseño geométrico en caminos de bajo volúmen es basicamente un asunto de economía. En 1970 el Institute for Transportation and Traffic Engineering preparó un informe para el U.S. Department of Transportation titulado *Opportunities for Cost Reduction in the Design of Transport Facilities for Developing Regions* (Oportunidades Para una Reducción de Costos en el Diseño de Servicios de Transporte Para Regiones en Desarrollo). El sexto texto en este compendio consiste en un extracto del Capítulo 2, Análisis de Costo Vial y Normas de Diseño, por Lawrence Vance. En este extracto el autor indica las secciones en donde las normas actuales de AASHO parecen excesivas para la construcción inicial de caminos rurales en los países en desarrollo, particularmente en lo que respecta a anchos de calzada y pendientes máximas.

El séptimo texto es un artículo por W. H. Valentine, *A Safe Sight Distance Requirement for Un-Laned Rural Roads* (Un Requisito de Distancia de Visibilidad de Frenado Para

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Design Guide For Local Roads and Streets (Un Guide de Dimensionnement Géométrique Pour les Routes Locales et les Rues) (AASHO, 1971), qui a pour sujet le tracé géométrique des routes dont le ADT va jusqu'à 750 véhicules par jour avec ajustements pour une expansion future.

Le cinquième texte, *A Review of Highway Design Practices in Developing Countries* (Banque Mondiale, 1975), reproduit ici en entier, traite des routes rurales de différentes capacités. Il divise ces routes en trois catégories et ensuite subdivise la catégorie II en cinq classes. Notre projet est spécialement concerné par les classes I et II des catégories II et III.

La raison pour laquelle on tente de simplifier les normes de dimensionnement géométrique des routes à faible capacité est une raison économique. En 1970, l'Institute for Transportation and Traffic Engineering a preparé un rapport pour le U.S. Department of Transportation *Opportunities for Cost Reduction in the Design of Transport Facilities for Developing Regions* (Moyens Utilisés Pour Réduire le Coût de la Construction des Infrastructures de Transport Dans les Régions en Voie de Développement). Le sixième texte de ce recueil est extrait du chapitre 2, *Road Cost Analysis and Design Standards* (Analyse du Coût et Normes de Dimensionnement d'une Route), par Lawrence Vance. Dans cet extrait, l'auteur signale les endroits ou les normes AASHO semblent excéssives pour la construction initiale des routes rurales dans les pays en voie de développement, particulièrement en ce qui concerne la largeur des routes et leur pente maximum.

A Safe Sight Distance Requirement for Un-Laned Rural Roads, published in *Rural and Urban Roads* in February 1968. This article, which was referred to in the excerpt that precedes it, concerns converging sight distance for single-lane facilities, a topic that is not considered in the selected AASHO publications.

The eighth text presented is an excerpt taken from *Policy for Geometric Design of Rural Roads,* published by the National Association of Australian State Road Authorities (NAASRA) in 1970. Section 8, Grades, is included in its entirety to demonstrate a different approach to determining maximum grades on low-volume roads. The excerpt uses the imperial system of measurement, but in 1972 an addendum using the metric system was published for the entire volume.

The ninth text contains the section of the *Metric Addendum* that pertains to the previous selection; also included is Appendix B, which contains conversion tables adopted by NAASRA.

The tenth and last text in this compendium is a paper taken from *Low-Volume Roads* (TRB Special Report 160). This report represents the proceedings of a TRB workshop held June 16 to 19, 1975, in Boise, Idaho, that was cosponsored by the Agency for International Development, AASHTO, the Federal Highway Administration, the Idaho Transpor-

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Caminos Rurales sin Trochas), publicado en el *Rural and Urban Roads* en febrero de 1968. Este artículo, que es mencionado en el extracto que lo precede, se concierne con la distancia de visibilidad convergente para medios de una trocha, un tema que no se considera en las publicaciones seleccionadas de AASHO.

El octavo texto que se presenta es un extracto de *Policy for Geometric Design of Rural Roads* (Política Para el Diseño Geométrico de Caminos Rurales), publicado por el National Association of Australian State Road Authorities (NAASRA) en 1970. La Sección 8, Grades (Pendientes), se incluye totalmente para demostrar una manera distinta de determinar pendientes máximas en caminos de bajo volúmen. El extracto utiliza el sistema imperial de medir, pero en 1972 se publicó un apéndice utilizando el sistema métrico para todo el volúmen.

El noveno texto contiene la sección del *Metric Addendum* (Apéndice Métrico) que se refiere a la sección previa; también se incluye el Apéndice B, que contiene tablas de conversión adoptados por NAASRA.

El décimo y último texto de este compendio es una exposición tomada de *Low-Volume Roads* (Caminos de Bajo Volúmen) (TRB Special Report 160). Este informe representa los procesos de un taller TRB llevado a cabo entre el 16 y 19 de junio, 1975, en Boise, Idaho, que fué co-patrocinado por la Agency for International Development, AASHTO, el Federal Highway Administration, el Idaho Transportation Department, el Banco Mundial, el International Road Federation, el National Asso-

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Le septième texte est un article de W. H. Valentine, *A Safe Sight Distance for Un-Laned Rural Roads* (Conditions de Visibilité Pour Assurer la Sécurité Pour des Routes Rurales Sans Marquage Longitudinal) publié dans *Rural and Urban Roads* de février 1968. Cet article, auquel on se rapporte dans l'extrait qui le précède, concerne les visibilités de distance convergentes pour les routes à voie unique, un sujet qui n'est pas abordé dans les écrits de AASHO que nous avons sélectionnés.

Le huitième texte présenté est un extrait pris dans *Policy for Geometric Design of Rural Roads* (Méthode Pour le Dimensionnement Géométrique des Routes Rurales), publié par le National Association of Australian State Road Authorities (NAASRA) en 1970. La section 8, Grades (Pentes), est reproduite entièrement pour montrer une approche différente pour déterminer les pentes maximales pour routes à faible capacité. Cet extrait utilise le système britannique de mesures, mais en 1972 un addenda utilisant le système métrique, a été publié.

Le neuvième texte contient la section de *Metric Addendum* (Addenda Metrique) se rapportant à la Section 8 précédente; l'appendice qui contient des tables de conversion adoptées par la NAASRA est aussi inclus.

Le dixième et dernier texte de ce recueil est extrait du rapport *Low-Volume Roads* (Routes à Faible Capacité) (TRB Special Report 160). Ce rapport représente les comptes-rendus d'une réunion du TRB à Boise, Idaho, du 16 au 19 juin 1975. Cette réunion était tenue sous le patronage de l'Agency for International Development, AASHTO, la Federal Highway Administration, le Idaho Department tation Department, the World Bank, the International Road Federation, the National Association of County Engineers, the National Science Foundation, the U.S. Army Engineer Waterways Experiment Station, the U.S. Forest Service, and the University of Idaho. The paper selected, ***The Forest Service's Computer-Aided Road Design System,*** by Thomas A. George, was chosen for two reasons. First, it provides an interesting overview of the many factors an engineer must consider in the design of a low-volume road whatever the geometric design standards used, and it stresses the fact that the computer is an aid to engineering, not a cure-all to replace sound engineering judgment. Second, it describes computer software developed specifically for low-volume roads operated under U.S. Forest Service conditions. This software is available for purchase by anyone who has suitable computer capability. The technical information concerning this computer program is included in the Bibliography of this compendium.

Bibliography

The Selected Texts are followed by a brief bibliography that contains reference data and abstracts for each of 17 publications. The first 10 references are for the publications in the Selected Texts. The remaining references are for publications that have been cited within the texts or that are otherwise closely associated with the selected items.

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ciation of County Engineers, el National Science Foundation, el U.S. Army Engineer Waterways Experiment Station, el U.S. Forest Service, y la Universidad de Idaho. La exposición seleccionada, ***The Forest Service's Computer-Aided Road Design System*** (El Sistema de Diseño de Camino Auxiliada por Computadora del Servicio Forestal), por Thomas A. George, fué escogida por dos razones. Primero, provee una vista general interesante de los muchos factores que debe considerar un ingeniero en el diseño de un camino de bajo volúmen cualesquiera normas de diseño geométrico se utilicen, y dá énfasis al hecho de que la computadora es una ayuda para la ingeniería, y no un cura-lo-todo para reemplazar un buén juicio ingenieril. Segundo, describe programas de computación desarrollados especialmente para caminos de bajo volúmen operados bajo condiciones del U.S. Forest Service. Estos programas pueden ser comprados por cualquiera que tenga capacidad de computadora adecuada. La información técnica que se refiere a este programa de computadora se incluye en la Bibliografía de este compendio.

Bibliografía

Los Textos Seleccionados son seguidos por una breve bibliografía que contiene datos de referencia y abstractos para cada una de 17 publicaciones. Las primeras 10 referencias son para las publicaciones de los Textos Seleccionados. Las referencias restantes son para publicaciones que han sido nombradas dentro de los textos o que de otra forma se asocian intimamente con las partidas seleccionadas.

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of Transportation, la Banque Mondiale, la International Road Federation, la National Association of County Engineers, la National Science Foundation, la U.S. Army Engineer Waterways Experiment Station, le U.S. Forest Service, et l'University of Idaho. L'article choisi s'intitule *The Forest Service's Computer-Aided Road Design System* (Tracé géométrique d'une Route à l'Aide de l'Ordinateur Dans le Forest Service) par Thomas A. George. Cet article a été choisi pour deux raisons. D'abord il donne une vue d'ensemble intéressante des différents facteurs qu'un ingénieur doit considérer quand il fait l'étude d'une route à faible capacité, quelques soient les normes de dimensionnement géométriques choisies, et ensuite, il met l'emphase sur le fait que l'ordinateur peut aider l'ingénieur mais n'est pas une panacée universelle qui puisse remplacer son jugement professionel. En dernier, cet article contient une description du logiciel développé spécialement pour les routes à faible capacité qui sont sous la responsabilité du U.S. Forest Service. Ce logiciel est en vente libre. La documentation technique concernant ce logiciel est incluse dans la Bibliographie de ce recueil.

Bibliographie

Les Textes Choisis sont suivis par une brève bibliographie qui contient les références et les analyses pour chacune des 17 publications. Les 10 premières références se rapportent aux publications des Textes Choisis. Les autres références se rapportent à des publications qui ont soit été citées dans les Textes Choisis, soit sont associées de très près avec eux.

Selected Texts

This section of the compendium contains selected pages from each text that is listed in the Table of Contents. Rectangular frames are used to enclose pages that have been reproduced from the original publication. Some of the original pages have been reduced in size to fit inside the frames. No other changes have been made in the original material except for the insertion of occasional explanatory notes. Thus, any errors that existed in the selected text have been reproduced in the compendium itself.

Page numbers of the original text appear inside the frames. Page numbers for the compendium are outside the frames and appear in the middle left or middle right outside margins of the pages. Page numbers that are given in the Table of Contents and in the Index refer to the compendium page numbers.

Each text begins with one or more pages of introductory material that was contained in the original publication. This material generally includes a title page, or a table of contents, or both. Asterisks that have been added to original tables of contents have the following meanings:

*Some pages (or parts of pages) in this part of

Textos Seleccionados

Esta sección del compendio contiene páginas seleccionadas de cada texto que se catalogaron en la Tabla de Materias. Se utilizan recuadros rectangulares para encerrar las páginas que han sido reproducidas de la publicación original. Algunas de las páginas originales han sido reducidos para entrar en los recuadros. No se han hecho ningunos otros cambios en el material original exceptuando algunas notas aclaradoras que de vez en cuando han sido agregadas. De esta forma, cualquier error que hubiera existido en el texto seleccionado ha sido reproducido en el compendio mismo.

Los números de páginas del texto original aparecen dentro de los recuadros. Los números de páginas para el compendio están fuera de los recuadros y aparecen en los márgenes medio izquierdo o medio derecho de las páginas. Los números de páginas que se dán en los índices del compendio se refieren a los del compendio.

Cada texto comienza con una o mas páginas de material de introducción que contenía la publicación original. Este material generalmente incluye una página título, un índice, o ambas. Los astericos que han sido agregados al índice original significan lo siguiente:

* Algunas páginas (o partes de páginas) en

Textes Choisis

Cette partie du recueil contient les pages sélectionnées de chaque texte qui est énuméré dans la Table des Matières. Les pages du texte original qui sont reproduites sont entourées d'un encadrement rectangulaire. Certaines pages ont dû être réduites pour pouvoir être placées dans l'encadrement. Le texte original n'a pas été changé sauf pour quelques explications qui ont été insérées. Donc, si le texte original contient des erreurs, elles sont reproduites dans le recueil.

La pagination originale apparaît à l'intérieur de l'encadrement. La pagination du recueil est à l'extérieur de l'encadrement, soit à droite, soit à gauche de la marge extérieure des pages et est celle qui est citée dans la table des matières et dans l'index du recueil.

Chaque texte commence par une ou plusieurs pages d'introduction qui étaient incluses dans le texte original. Ces pages sont généralement le titre, ou la table des matières, ou les deux. Des astériques ont été ajoutés à la table des matières d'origine pour les raisons suivantes:

* Certaines pages, ou portions des pages,

the original document appear in the selected text, but other pages (or parts of pages) in this part of the original publication have been omitted.

**All pages in this part of the original document appear in the selected text.

The selected texts therefore include only those parts of the original documents that are preceded by asterisks in the tables of contents of the respective publications.

Broken lines across any page of selected text indicate those places where original text has been omitted. In a number of places, the selected text contains explanatory notes that have been inserted by the project staff. Such notes are set off within dashed-line boxes and begin with the word NOTE.

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esta parte del documento original aparecen en el texto original, pero otras páginas (o partes de páginas) en esta parte de la publicación original han sido omitidas.

** Todas las páginas en esta parte del documento original también aparecen en el texto seleccionado.

Por lo tanto, los textos seleccionados unicamente incluyen aquellas partes de los documentos originales que están precedidas por asteriscos en el índice de las publicaciones respectivas.

Líneas de guiones cruzando cualquier página del texto seleccionado significan que en ese lugar se ha omitido texto original. En varios lugares el texto seleccionado contiene notas aclaradoras que han sido introducidas por el personal del proyecto. Tales notas están insertadas en recuadros de guiones y comienzan con la palabra NOTE.

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dans cet extrait du document original sont incluses dans les Textes Choisis, mais d'autres pages (ou portion de pages) de l'édition originale ont été omises.

** Toutes les pages dans cet extrait du document original sont incluses dans les Textes Choisis.

Les Textes Choisis, donc, incluent seulement ces extraits des documents originaux qui sont précédés d'un astérique dans les tables des matières des publications respectives.

Les lignes brisées sur les pages des textes choisis indiquent les endroits où le texte original a été omis. A certains endroits, les textes choisis contiennent des explications qui ont été insérées par le personnel attaché à ce projet. Ces explications sont entourées d'un encadrement en pointillé et commencent toujours par le mot NOTE.

INTERNATIONAL BANK FOR RECONSTRUCTION AND DEVELOPMENT

A GUIDE TO HIGHWAY DESIGN STANDARDS

June 1957

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Department of Technical Operations

PREFACE

This report offers specific guides for the selection and use of proper design standards in the preparation of plans and specifications for highway construction projects.

A large amount of information concerning highway design has been developed in English-speaking countries and is available in current technical publications. This report is not intended in any way as a replacement of technical manuals and reference texts. No attempt has been made to cover all phases of highway engineering or to supply detailed information on accepted principles of design. Its purposes are to offer guidance for establishing specific standards and specifications and to encourage the design engineer to make the most effective use of information from other sources.

Definite standards for highway design are presented, in which detailed dimensions and specifications of the various elements are indicated. Standards are listed for both primary and secondary routes and for different topographic conditions. Design speed and other important elements are specified within relatively broad limits, to permit the design engineer to select appropriate standards for any given situation. A discussion of the various considerations which should be recognized in making these selections constitutes an important part of the text. Design data, mostly in the form of graphs and charts, have been included to implement the use of the design standards.

Much of the published technical information on highway design is not readily adaptable to the needs of Engineers in countries where the metric system is used. In this report all standards and other design data, including charts and graphs, have been based upon the metric system. It is hoped that this feature will add to the convenience and general usefulness of the material.

CONTENTS

INTRODUCTION

Highway engineering has advanced rapidly during the past twenty years and continues to advance at a steady pace. New developments in the applications of scientific theories of motion to vehicle operation, studies of vehicle operational costs, analyses of accidents and of maintenance costs, and other studies of highway performance, constantly add to the knowledge which is available to the highway engineer. However, highway engineering is not an exact science. Wide variations in anticipated traffic volumes, differences of geography and topography, and many other variable factors, all tend to complicate the development of standardized concepts of design.

A large part of the accepted information on highway design standards is based on experience obtained from existing highway systems operating under relatively large volumes of traffic. Therefore much of the published information on these design standards represents situations and conditions in certain areas. However, the basic concepts and their application derived from these sources can be extended to highway requirements in other areas, providing due consideration is given to the vatiations already mentioned.

The Bank is now cooperating in highway construction programs involving more than 40 different projects in many countries which cover a wide geographic area. As a result it has been possible to obtain detailed information on many aspects of highway engineering in these countries. The present report is an attempt to apply this information to accepted principles of highway engineering and to the standards which have been adopted by several older highway agencies. It is hoped that the resultant design criteria will be useful in areas where there is as yet insufficient experience of design for the volumes of traffic which should be anticipated.

The fundamentals of highway design are based upon three considerations: (1) structural capacity to provide for the accommodation of specified weight; (2) volume capacity to provide for the movement of a specified number of vehicles over a stated period of time; and (3) traffic service, which comprises the measure of safe speed, the limiting gradients, the general safety and comfort of highway users, and the relative economy of vehicle operation.

DETERMINATION OF DESIGN CRITERIA

Project Planning

Good highway design must be founded upon basic information concerning the specific project under consideration. This information includes a general indication of the availability of funds for construction and future maintenance, the relative importance of the route and the amount and type of traffic which will be expected to use the facility at some time in the future. Proper selection of design criteria can be made only if this information is at hand.

An indication of the funds which should be made available for a specific project can be obtained only from a determination of the priority position of the project in a program. This determination may be made from a long-range plan which has been formally developed; or the design engineer may be forced to rely upon informal plans, or even upon vague ideas which exist in a highway department.

The development of a long-range highway construction program, which would include the projects to be selected for a current program, should be carried out with the use of specific engineering and economic studies. It is acknowledged that there may be many non-technical considerations, some of them valid, which are recognized in this type of planning. However, it is important that engineers organize all available data into usable form, that they obtain additional information which may be needed, and that they present complete and specific recommendations.

The classification of all routes in a highway system is also an engineering undertaking. The determination of the routes to be included in the primary or trunk system, the secondary network and the tertiary or feeder routes is important, not only as a first step in program planning, but also in the proper selection of standards to which specific highways are to be constructed and maintained.

In addition to route classification, good design must include consideration of future traffic usage. An estimate of the number of commercial vehicles which will use the highway is important, but predictions of future total traffic volumes are difficult to develop. Mathematical projections based upon records of increased volumes over past periods may later be found to have produced predictions which vary considerably from actual volumes. However, certain trends, together with experience in similar situations on other locations may offer sufficient information to insure the continued adequacy of a highway for some time in the future. Ten-year predictions are frequently used in this determination.

USE OF DESIGN STANDARDS

All highway design is based upon a series of design standards which are adopted formally before the design is started, or which are established informally during the development of the plans. In the past few years there has been a definite trend towards a greater use of specific standards of design which have been established by various highway agencies. These standards have been developed through the application of basic highway engineering principles, taking into consideration the level of traffic service which is to be provided and the cost of construction (or availability of funds). Pre-established standards may be used as a guide to all of the phases of highway design - from the preliminary location to the final detailed construction plans. This report is intended to illustrate the development and use of such a series of design standards and to offer a suggested list of standards for certain selected elements of design.

It has been found that the advantages which are likely to be derived from the use of proper pre-established design standards include the following:

i. The general design can be made to conform to a predetermined and specific level of traffic service and volume capacity for which future needs can be predicted.

ii. A proper balance between the standards for different elements of design can be provided on each project.

iii. A consistent level of traffic service can be obtained throughout the length of each individual sector.

iv. A formal appraisal can be made between various anticipated levels of traffic service and the resulting costs of construction.

When highway facilities become inadequate and need to be replaced it is usually for one of two causes - insufficient maintenance or obsolescence. Assuming that a highway will receive proper maintenance and periodic reconstruction of the pavement surface, obsolescence through the inadequacy of the original design standards becomes the only reason for any part of a highway ever having to be abandoned or rebuilt. Established standards of design, based upon past experience and proven principles of operation, offer the only practical means of translating the predictions of future needs into specific and detailed design of the various elements of the highway.

Good highway design implies that each element of the facility is designed in the proper proportion to the other elements. For example, higher design speed dictates longer curves, flatter gradients and wider pavements. Unbalanced design not only may produce dangerous highways but over-design of certain elements, in relation to the others, is likely to lead to uneconomical construction. Design standards help to establish this important balance in design.

A safe and efficient highway must provide a consistent level of traffic service throughout the length of a sector. An obvious example of this factor is that of horizontal alignment. It is recognized that a sharp curve, located in level terrain or at the end of a long tangent, is many times more dangerous than the same type of curve located in mountainous terrain, or in any sector containing many curves. In the same way, variations in pavement widths, or even in shoulder widths, may often produce undue hazards. Although general standards must be changed to meet changing conditions and situations, the number of these changes should be kept to a minimum, and they should occur only at logical locations. Different sets of design standards are normally used in mountainous, hilly and level terrain, but the proper application of even these different standards on the same project will produce fewer hazards than those which would result from indiscriminate changes in the geometric dimensions that localized situations might indicate.

Highways are never constructed to standards which are completely adequate. Highway design always includes a consideration of some logical balance between the recognized needs of present and future traffic and the cost of constructing the facility. The use of a set of pre-established design standards makes possible a formal engineering approach to this problem, by which various levels of traffic service which might be provided can be compared to the estimated cost of each type of construction.

Highway standards are usually applied to geometric design and to structural design of the pavement and base. Geometric design includes all of the visible dimensions of a highway facility - horizontal and vertical alignments and the cross-sectional elements. Structural design includes the determination of the dimensions which together establish the load-carrying capacity of the highway.

Certain portions of the highway structure constitute a more permanent investment than others. In descending order of their permanence, the various parts may be listed as the right-of-way, grade (cuts, embankments, etc.), large drainage structures, small culverts, sub-base, base, and pavement. It is apparent that the geometric design of a highway controls the more permanent parts of the investment. Horizontal alignment, together with the influence of the vertical alignment, establishes the location of the right-of-way and of the grade. Vertical alignment dictates the depth of the cuts and the height of the embankments. The standard cross-section defines the shape of the grade. It is for these reasons that the use of geometric design standards which have been carefully conceived and selected is so important.

If sufficient funds are not available to develop a project to reasonably adequate standards in all elements, it is frequently desirable to plan to invest proportionately greater amounts in the grade and drainage structures, and to design the pavement surface to relatively lower standards. Increased traffic volumes at some future date can then be served by an adequate highway facility, provided the pavement surface has by that time been built up to higher standards. There is little or no difference in the immediate traffic service offered by a high-type bituminous pavement and a well-constructed low-cost bituminous surface. "Stage construction," by which small investments can be made in periodic resurfacing operations, will eventually result in a pavement which may approximate the structural capacity of a high-type pavement. This method frequently offers a practical means by which current investments can be kept to a minimum, without materially reducing the service to the highway user. This type of planning is especially appropriate when the use of the facility is expected to increase rapidly during the first few years of service.

DEVELOPMENT OF DESIGN STANDARDS

In the development of the series of design standards included in this report, information from several sources was used. Specific and detailed design data were received from various highway departments which reported on construction projects having a total length of 5,434 kilometers. This information was carefully tabulated and analyzed. Accepted principles of highway engineering were applied to this information and the results were compiled in a tentative form. The resulting series of design standards were studied by engineers on the Bank staff and other engineers in the field and the items were corrected or adjusted according to the suggestions received. Therefore, the standards represent the result of a practical analysis of existing and approved standards and they should be generally applicable to highway projects in all areas.

SELECTION OF DESIGN STANDARDS

Most design standards are a direct function of design speed, traffic volume and weight capacity, vehicle operational costs, and relative traffic safety, qualified by a consideration of construction costs. For this reason, there is a definite correlation between the design criteria controlling each of the various elements of a highway facility, as they are specified in established standards.

Since there are wide variations in traffic needs and in topographic conditions, design standards which are intended for general application must contain provisions which will recognize these differences. Most of the elements which are included in the list of standards are defined between maximum and minimum specifications, which allows a rather wide degree of freedom for selection. In general, the selection of a higher standard for one element will indicate a higher standard for each of the remaining elements. For example, the selection of a relatively high design speed for a sector of highway in level terrain will dictate the selection of a specific standard to control the minimum radius of curvature. Less specifically, but equally important, is the selection of high standards for other elements, which might include the maximum gradient, the widths of pavement and shoulders, and the length of spiral easements at the ends of the horizontal curves.

Exception to this general concept of balanced design might include such major installations as tunnels and large bridges, which cannot be easily reconstructed to higher standards if future traffic volumes should warrant a higher type facility.

In considering the nature of the terrain in which a highway is to be located, certain differences in the balance of design between the various elements are indicated. Thus, in a mountainous area, the design speed is lowered to permit the selection of a more practical minimum radius of curvature, the maximum gradient is increased, and shoulders may be made more narrow, without decreasing the width of the pavement. All of these factors should be considered in direct relationship with the cost of construction.

The selection of the proper standards for the structural design of
14 the sub-base, base and pavement is usually determined by a number of considerations, including the type and volume of traffic, the prevailing weather and soil conditions, plans for future maintenance, and the availability of funds for construction and maintenance. Since pavements or untreated surfaces can be reconstructed or augmented at some future time without undue loss of the original investment, the immediate traffic requirements usually are assigned more importance in the selection of the surface type and thickness than in the selection of geometric standards.

Design standards should be regarded as goals to be attained, and not necessarily as specific rules which can never be violated. For example, if a maximum gradient of 6% is established for a mountainous sector, this selection might tend to control a large number of grades in that sector. However, the designer might find it completely impractical to consider such a grade at a certain location, and a steeper grade would be adopted. To have set the standard maximum grade for the entire sector at this steeper gradient might have eliminated or weakened the control of many other grades in the sector, with a resulting loss of the advantages derived from uniformity of design. It is an accepted practice to select standards which are appropriate, considering both the desired traffic service and construction costs, with the realization that there may be localized exceptions to these standards which can be justified. To lower the overall standards to meet these occasional local conditions is to lose the real benefits which can be derived from properly selected standards of design.

GEOMETRIC DESIGN

DEFINITION

Geometric design includes all of the visible dimensions of a highway facility, controlling the following elements of design:

A. Horizontal alignment.
   - i. Minimum radius of curvature (or maximum degree of curve).
   - ii. Minimum length of tangent between curves (reverse or compound curves).
   - iii. Horizontal sight distance. 1/

B. Vertical alignment.
   - i. Maximum gradient.
   - ii. Maximum length of control gradient.
   - iii. Length of parabolic curves at crests to obtain minimum stopping and passing sight distance.
   - iv. Length of parabolic curves at sags.

C. Cross-Section.
   - i. Width and crown of pavement.
   - ii. Width and slope of shoulders.
   - iii. Minimum depth and standard width of side ditches. 2/
   - iv. Slope of cut banks. 2/
   - v. Slope of embankments. 2/
   - vi. Amount of pavement superelevation.
   - vii. Curve easements and pavement widening on horizontal curves.
   - viii. Width of structures.
   - ix. Horizontal sight distance.

Geometric design is based upon three general considerations: volume capacity, degree of traffic service and cost. Cost includes both the cost of construction and the cost of future maintenance operations. All of these items must be considered in the proper use of geometric design standards.

1/ Also influenced by cross-section design.

2/ These elements are also functions of structural design.

The geometric design standards which are shown in Tables 1 and 2 are listed in two major categories according to route importance. Although the fundamental difference between primary and secondary routes is that of location or situation, for the purpose of assigning design standards the factor of traffic volumes also must be recognized. Therefore the standards in Table 2, for secondary routes, should be used only on routes which can be expected to have an average daily traffic of less than 800 vehicles during a period five years after construction.

Since the standards for primary routes are of greater importance, and the corresponding items for secondary routes follow the same general principles of design, most of the discussion which follows will be based upon the list for primary routes.

Certain standards are further divided into three classifications according to terrain. Although it is recognized that no specific definition can be derived or interpreted which would lead to a precise terrain classification in any specific sector, it is believed that the general terms "mountainous," "hilly" and "level and rolling" will be adequate for the purposes for which they are intended. The proper determination of the length and location of the individual design sectors depends upon the proper classification of the terrain. Changes in terrain may frequently be easily recognized and accurately located, while in other areas these changes may be difficult to identify and to assign to specific locations. The determination of the proper terrain classification forms an important part of the general consideration which must be given to the various factors which together influence the selection of practical design standards for highway construction projects.

The development of the geometric design standards which are listed in Tables 1 and 2 took into account the information which was submitted in the recent questionnaires from Bank-financed projects. Tables 3 and 4 show the close correlation between the information which was obtained from the field and the standards listed in Tables 1 and 2. This comparison indicates that the recommended standards of design are reasonable, within the ranges which are indicated. Most of the important elements of design are shown to be in general conformance with the accepted policies of the various highway departments which submitted information to the Bank.

GEOMETRIC DESIGN STANDARDS
PRIMARY ROUTES

Design Element	Mountainous	Hilly	Level and Rolling
DESIGN SPEED	40 KPH* - 55 KPH	55 KPH - 70 KPH	70 KPH - 100 KPH
Min. Radius of Curvature (Chart I)	50 m. - 100 m.	100 m. - 150 m.	150 m. - 300 m.
Max. Degree of Curve (Metric, 20 m.ch.) (Chart II)	23° - 13°	13° - 8°	8° - 4°
Maximum Gradient	9% - 7%	7% - 5%	4% 1/
Max. Length of Grades over Control Grad.	400m, over 6%	600m, over 4%	None
Pavement Width (plus widening at curves) (Table 5)	6.0 m. - 7.3 m.	6.3 m. - 7.3 m.	6.5 m. - 7.3 m.
Min. Shoulder Width, Cuts (Table 6)	1.0 m.	1.5 m.	2.0 m.
Min. Shoulder Width, Fills (Table 6)	2.0 m.	2.0 m.	2.0 m.
Rec. Shoulder Width, Cuts and Fills			2.5 m. - 3.0 m.

Design Element	Unclassified as to terrain		
Min. Length of Vertical Curves	See Charts III and IV		
Horizontal Sight Distance	See Chart VIII		
Pavement Crown	High-type $1\frac{1}{2}$ - 2%	Low-type 2% - 3%	
Shoulder Slope	5% - 10%		
Slope of Cut Banks (earth)	Under 1.5 m., 1:1 - 2:1	Over 1.5 m., 1/4:1 - $1\frac{1}{2}$:1	(Slopes, hor.:vert.)
Slope of Fills	Under 1.5 m., 2:1 - 4:1	Over 1.5 m., $1\frac{1}{2}$:1 - 2:1	(Slopes, hor.:vert.)
Superelevation (Table 7)	12% Max.		
Easements and Widening at Curves	See Tables 7 and 8		
Structure Widths; Under 25 m. Total span			
Clear Roadway Width	8.0 m., min.		
Clearance between trusses or handrails	9.0 m. - 9.5 m. **		
Structure Widths, Over 25 m. and under 250 m. Total Span			
Clear Roadway Width	7.8 m. min.		
Clearance between trusses or handrails	8.8 m. - 9.2 m. **		
Structure Width, Over 250 m. Total Span			
Clear Roadway Width	7.4 m. min.		
Clearance between trusses or handrails	8.4 m. - 8.7 m. **		

1/ In locations where the grade is predominantly flat, 3% maximum should be used. Minimum grade in cuts, 0.3%.

* or km/hour as used in many countries.

** Sidewalk recommended on one side.

TABLE 1

GEOMETRIC DESIGN STANDARDS
SECONDARY ROUTES

Design Element	Mountainous	Hilly	Level and Rolling
DESIGN SPEED	35 KPH - 50 KPH	50 KPH - 60 KPH	60 KPH - 80 KPH
Min. Radius of Curvature (Chart I)	40 m. - 80 m.	80 m. - 120 m.	120 m. - 200 m.
Max. Degree of Curve (Metric, 20 m.ch.) (Chart II)	25° - 15°	15° - 10°	10° - 6°
Maximum Gradient	9% - 7%	7% - 5%	5% 1/
Max. Length of Grades over Control Grad.	750 m., over 6%	None	None
Pavement Width (Table 5)	5.5 m. - 6.5 m.	5.8 m. - 7.0 m.	6.0 m. - 7.0 m.
Untreated Aggregate Surface Width	6.5 m. - 7.5 m.	6.8 m. - 8.0 m.	7.0 m. - 8.0 m.
Min. Shoulder Width, Cuts	1.0 m.	1.0 m.	1.5 m.
Min. Shoulder Width, Fills	2.0 m.	2.0 m.	2.0 m.
Rec. Shoulder Width, Cuts and Fills			2.0 m. - 2.5 m.

Unclassified as to terrain

Design Element			
Min. Length of Vertical Curves	See Charts III and IV		
Horizontal Sight Distance	See Chart VIII		
Pavement Crown	Low-type 2% - 3%	Untreated Aggregate 3% - 4%	
Shoulder Slope	5% - 10%		
Slope of Cut Banks (earth)	Under 1.5 m. 1:1 - 2:1	Over 1.5 m. 1/4:1 - 1½:1	(Slopes, hor.: vert.)
Slope of Fills	Under 1.5 m. 1½:1 - 3:1	Over 1.5 m. 1½:1 - 2:1	(Slopes, hor.: vert.)
Superelevation (Table 7)	12% (max.)		
Easements and Widening at Curves	See Tables 7 and 8		
Structure Widths; Under 25 m. Total Span			
Clear Roadway Width	7.4 m., (min.)		
Clearance between trusses or handrails	8.4 m., (min.)		
Structure Widths, Over 25 m.			
Clear Roadway Width	7.4 m. (min.)		
Clearance between trusses or handrails	8.4 m. (min.)		

1/ Minimum grade in cuts, 0.3%.

TABLE 2

TABLE 3

COMPARISON OF RECOMMENDED GEOMETRIC DESIGN STANDARDS WITH STANDARDS REPORTED FROM PROJECTS

Primary Routes

Element of Design	Standard	No. of Projects Within Range of Rec. Standards	No. of Projects Over Rec. Standards	No. of Projects Under Rec. Standards
Min. Radius of Curvature				
Mountainous	50 m.-100 m.	24	0	4
Hilly	100 m.-150 m.	9	1	11
Level and Rolling	150 m.-300 m.	17	1	10
Maximum Gradient				
Mountainous	9%-7%	23	3	0
Hilly	7%-5%	16	0	4
Level and Rolling	4%	1	2	15
Pavement Width				
Mountainous	6.0 m.-7.3 m.	19	0	0
Hilly	6.3 m.-7.3 m.	12	0	12
Level and Rolling	6.5 m.-7.3 m.	13	1	14
Shoulder Width*				
Mountainous	1.0 m. (min.)	1	9	7
Hilly	1.5 m. (min.)	3	6	17
Level and Rolling	2.0 m. (min.)	1	2	26
Clear Roadway Width				
Under 25 m. Total Span**	8.0 m. (min.)	3	7	21
25 m-250 m. Total Span**	7.8 m. (min.)	0	16	15
Over 250 m. Total Span**	7.4 m. (min.)	0	17	14

* Standards for cuts only.

** Standards compared to all structures reported in questionnaires.

Note: The total number of items which could be compared was not the same for each of the categories.

TABLE 4

COMPARISON OF RECOMMENDED GEOMETRIC DESIGN STANDARDS WITH STANDARDS REPORTED FROM PROJECTS

Secondary Routes

Element of Design	Standard	No. of Projects Within Range of Rec. Standards	No. of Projects Over Rec. Standards	No. of Projects Under Rec. Standards
Min. Radius of Curvature				
Mountainous	40 m.- 80 m.	5	1	2
Hilly	80 m.-120 m.	1	1	0
Level and Rolling	120 m.-200 m.	0	0	9
Maximum Gradient				
Mountainous	9%-7%	5	0	2
Hilly	7%-5%	1	1	0
Level and Rolling	5%	2	0	8
Pavement Width				
Mountainous	5.5 m.-6.5 m.	5	0	0
Hilly	5.8 m.-7.0 m.	4	0	1
Level and Rolling	6.0 m.-7.0 m.	5	0	0
Shoulder Width*				
Mountainous	1.0 m. (min.)	0	2	3
Hilly	1.0 m. (min.)	0	3	2
Level and Rolling	1.5 m. (min.)	1	2	3
Clear Roadway Width				
(all lengths)	7.4 m. (min.)	0	1	6

* Standards for cuts only.

DESIGN SPEED

Design speed can be defined as the maximum speed at which highway vehicles, operated by average drivers, can travel over a certain sector of highway, operating freely under normal traffic flows and under normal conditions, including wet weather, with a reasonable degree of safety. The selection of the proper design speed for an individual project, or for a specific sector of a project, should be based upon several factors, many of which cannot be determined precisely. These factors include the number of vehicles which will be using the facility, the design speed of adjacent sectors, the difficulty and expense of construction as influenced by topographic and geologic situations, and the amount of funds available for the work.

The selection of a proper design speed is the first important step in the development of a balanced design, since many of the major standards of design are dictated by this selection.

Changes in the design speed which are made necessary by changes in the type of terrain or other local conditions, usually establish the length of each design sector in which the design standards remain constant. In an idealized situation, a highway project beginning in a valley, traversing a mountainous area and ending in a valley on the opposite side, might be divided into five design sectors - two sectors in level and rolling terrain, two sectors in hilly terrain (foothills) and one in mountainous terrain, with three sets of design standards appropriate to the three general situations. Although it is important that individual design sectors should be of sufficient length to establish a general consistency of design for appreciable distances along a highway, it is not possible to establish specific guides for determining practical minimum lengths. In level terrain, design standards might not be changed throughout the length of an entire project, or a series of projects, while in rugged terrain a single project might be divided into several design sectors in conformance with changing terrain conditions. However, if design standards are changed to meet each local situation, the important advantages of consistent design may be lost.

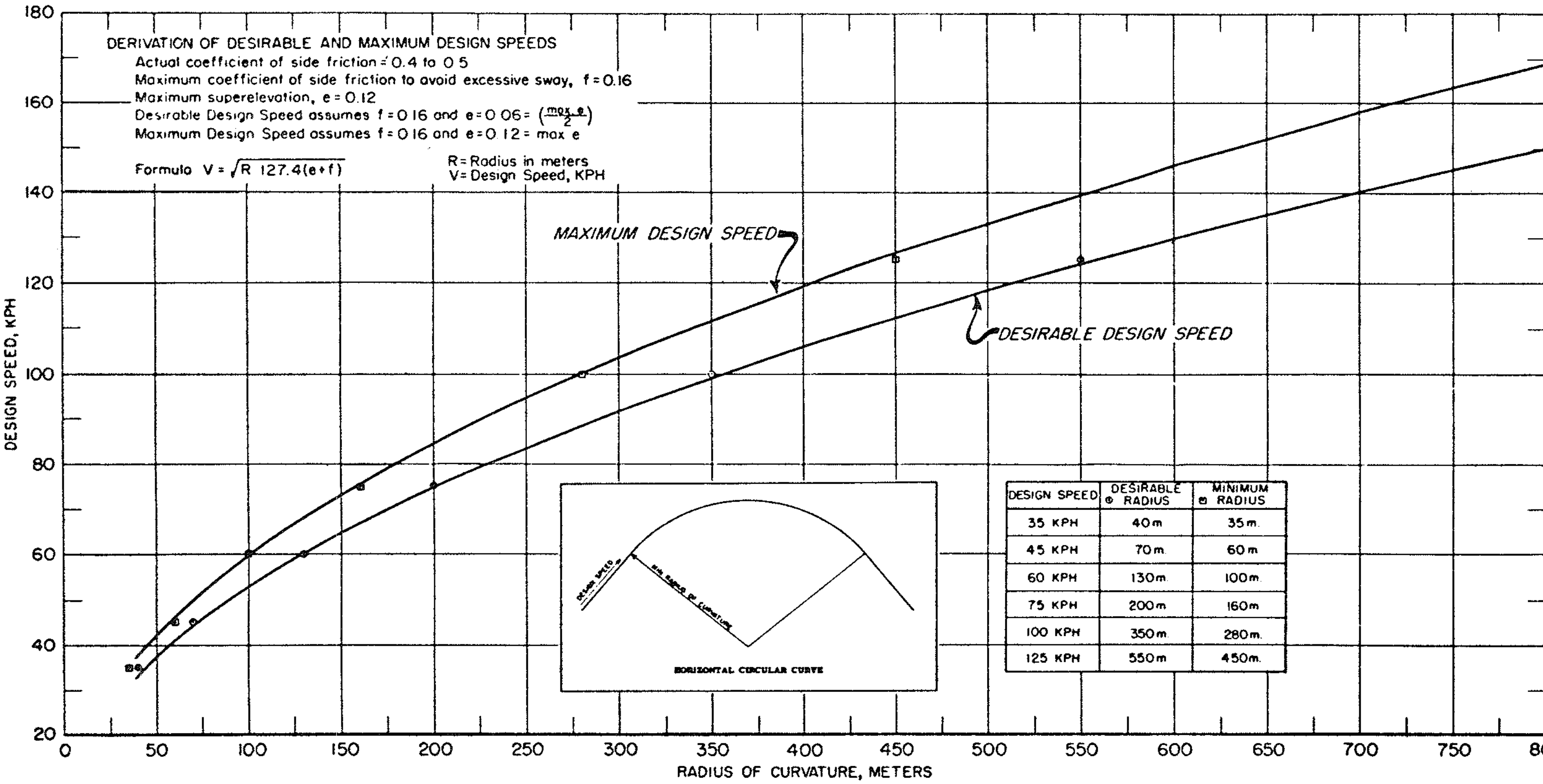
HORIZONTAL ALIGNMENT

After the selection of an appropriate design speed for a sector of a highway, the minimum radius of curvature (or maximum degree of curve) can be established mathematically, using the basic factors of superelevation and the coefficient of friction between the vehicle tires and the surface of the road.

Charts I and II show both the "Desirable" and "Maximum" conditions, with design speed related to the severity of the horizontal curves. The curve showing "Desirable" conditions arbitrarily assumes that the amount of maximum superelevation has been reduced by one-half. This is based upon the assumption that full superelevation is sometimes reduced along short lengths of the pavement, particularly at reverse curves, where an adequate distance for the transition from the superelevation to the normal cross-section may not be available. The assumption furnishes a convenient method for providing somewhat higher standards than are provided under "Maximum" conditions, in which full superelevation is assumed. In both cases, the actual average coefficient of side friction, determined by field measurements to be approximately 0.4, has been reduced to 0.16 in order to restrict the amount of side-sway to a safer limit. There is, therefore, an additional substantial factor of safety inherent in the data from which both curves were produced. This is intended to provide for errors in the operation of vehicles and for unusual pavement conditions.

It is recognized that curves in the same direction with different radii without an intervening tangent should be avoided because of the inability of the driver to recognize the sharper curve as he approaches it from the flatter one. Reverse curves also are undesirable, principally because there can be no adequate provision for the proper reversal of the superelevation between the two segments. In situations where these types of alignment may be indicated, it may sometimes be possible to introduce minimum lengths of tangents between the segments of the curves, thereby eliminating or reducing the problems. These tangents should be of sufficient length to provide for spiral easement curves and transition distances for superelevation.

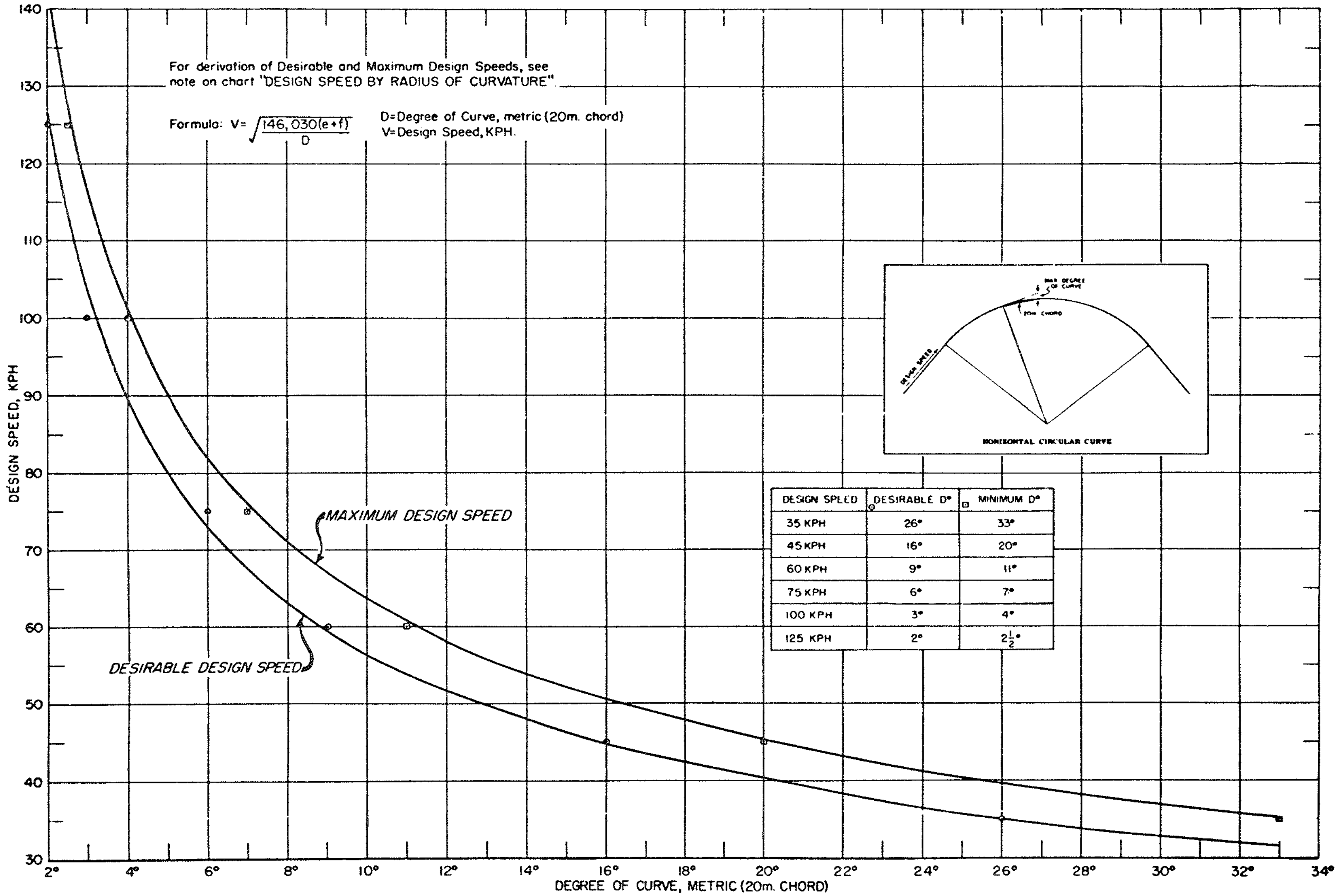
DESIGN SPEED (KPH) BY RADIUS OF CURVATURE (METERS)



DESIGN SPEED	DESIRABLE ⊕ RADIUS	MINIMUM ⊡ RADIUS
35 KPH	40 m	35 m.
45 KPH	70 m.	60 m
60 KPH	130 m.	100 m.
75 KPH	200 m	160 m
100 KPH	350 m.	280 m.
125 KPH	550 m	450 m.

Chart I

DESIGN SPEED (KPH) BY DEGREE OF CURVE (METRIC, 20m. CHORD)



DESIGN SPEED	DESIRABLE D°	MINIMUM D°
35 KPH	26°	33°
45 KPH	16°	20°
60 KPH	9°	11°
75 KPH	6°	7°
100 KPH	3°	4°
125 KPH	2°	$2\frac{1}{2}$°

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Chart II

VERTICAL ALIGNMENT

Design standards for vertical alignment include the determination of the maximum and control gradients and the lengths of vertical curves.

Maximum Gradient

The standard maximum gradient which is to be established for a certain sector should be considered carefully. It is not only an important factor in the consideration of cost but it may also control the horizontal alignment and the operating characteristics of the facility in general. For example, in certain types of mountain topography it may be possible to design relatively flat grades, with a minimum of heavy excavation, but only at the expense of many sharp horizontal curves, restrictions in horizontal sight distance and excessive increases in the length of road. In other situations, flatter gradients may be possible only by extremely heavy excavation, with higher construction costs and higher costs for future maintenance.

In rugged mountainous terrain, there are often one or more critical locations where it is not possible or feasible to design the gradient to a standard which would be appropriate for the remainder of the sector. Lowering the design standard for the sector to meet these special situations would tend to eliminate the advantages to be gained by establishing proper control on the remaining grades of the sector. Grades steeper than the established standard maximum can be justified, but they should be regarded as localized exceptions and justified as such.

The maximum standard gradients shown in Tables 1 and 2 include a rather broad range, and the selection of a single standard for a given sector should be made with full consideration of a proper balance between the resulting traffic service and construction costs.

Control Gradient

Field studies have shown that the total length of sustained steep grades may be as important, from the standpoint of operations, as the steepness of the grades. Both in climbing and in descending grades trucks need relief sections of flatter grades. Since the operating characteristics of loaded vehicles vary through a wide range, it is not possible to determine accurate values for the maximum length of sustained grades, nor to determine at what degree of steepness grades begin to cause operating difficulties. Also, topographic conditions sometimes make it impossible or impractical to reduce the length of steep grades at critical locations in mountainous areas. However, it has been found to be good design practice to establish a control gradient, at or above which the sustained length of the grade should be controlled. The control gradient usually is established at a lower percent of grade than the standard maximum gradient. It is assumed to be that grade upon which the average truck, loaded in conformance with the general terrain situation and the established design standards, will begin to experience undue difficulty in climbing. By establishing maximum lengths for these steep grades, difficulties which might be experienced by loaded vehicles may be reduced to a practical minimum.

Relatively steep grades which are located in areas having predominantly flat terrain may produce critical operational situations. These grades may act as control gradients for the sector, even if they are short in length. It is recommended that these grades be maintained at a maximum of 3%, whenever it is possible to do so.

Lengths of Vertical Curves at Crests

The length of vertical curves at grade crests is a function of the sight distance which is desired. Necessary sight distances are considered for two different purposes and are designated as design stopping sight distance and design passing sight distance.

Design stopping sight distance (vertical) for the purposes of this report can be defined as the distance which is required for a driver, traveling at the design speed, to bring his vehicle to a stop, after sighting an object 10 cm. high on the pavement over the crest of a vertical curve. This definition has been found to be practical in its application to actual situations in highway operation.

Chart III indicates the minimum lengths of parabolic curves which would connect the tangents of ascending grades at crests for various combinations of percents of grade and design speeds, to provide design stopping sight distance. These lengths have been derived from mathematical computations, considering the geometric functions of a parabola and using several proven empirical factors, including average driver perception-reaction time and braking distances for various speeds.

Since crest curves normally occur in heavy cut sections, it is customary to use the minimum lengths of vertical curve, in order to save excavation costs, if stopping distance is to be used as the criterion.

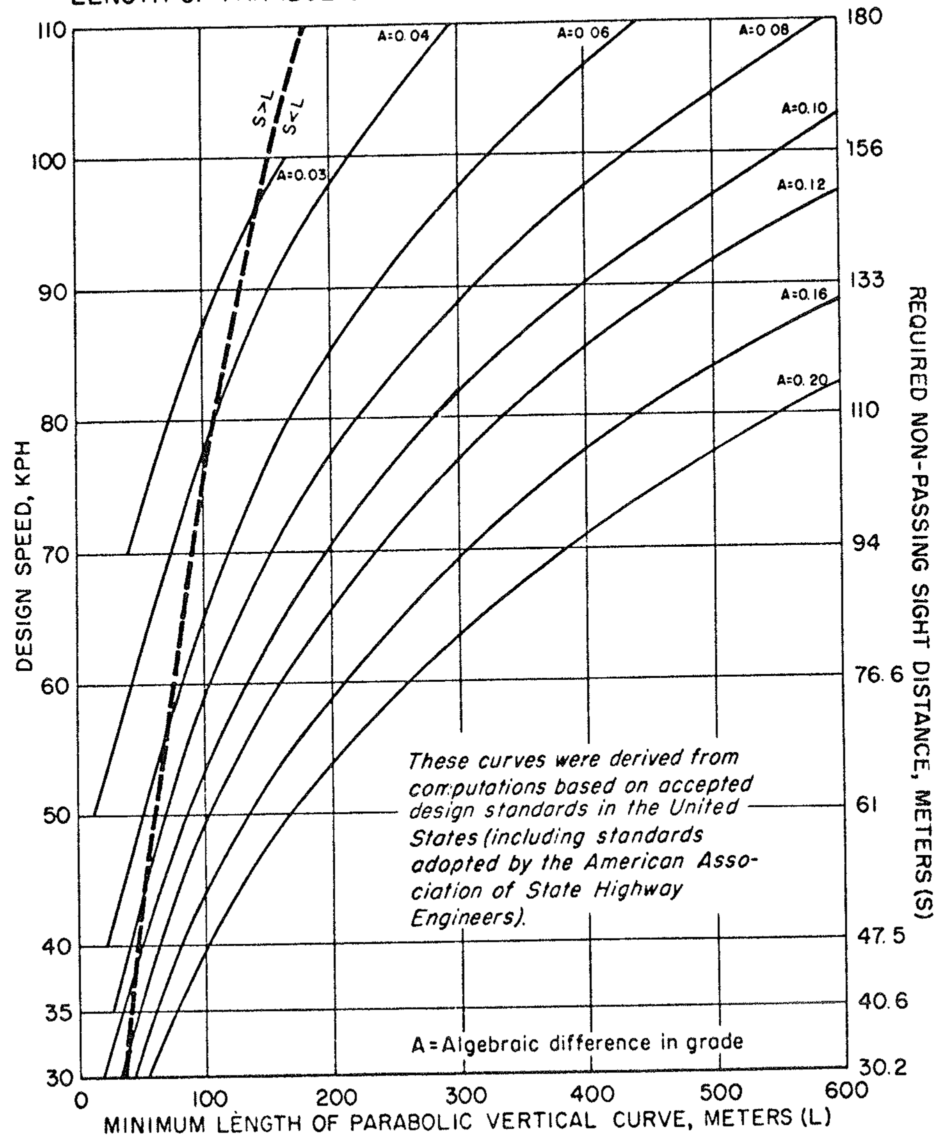
Design passing sight distance is considered where it is practical to do so, especially on two-lane high-type facilities. It is a much longer distance than stopping sight distance, and it is usually much more difficult and expensive to attain. The minimum lengths of vertical curves which are indicated on Chart IV were derived from a consideration of the geometric functions of a parabola and by the use of several factors which are inherent in a typical maneuver of one vehicle approaching and overtaking a slower vehicle, and returning to the proper lane, ahead of the approach of an oncoming car. All of these factors have been determined and proven by field investigations conducted by several highway agencies, using average situations and conditions. Although certain factors of safety have been incorporated into these determinations, it is believed that the information included in Chart IV represents practical minimum design requirements.

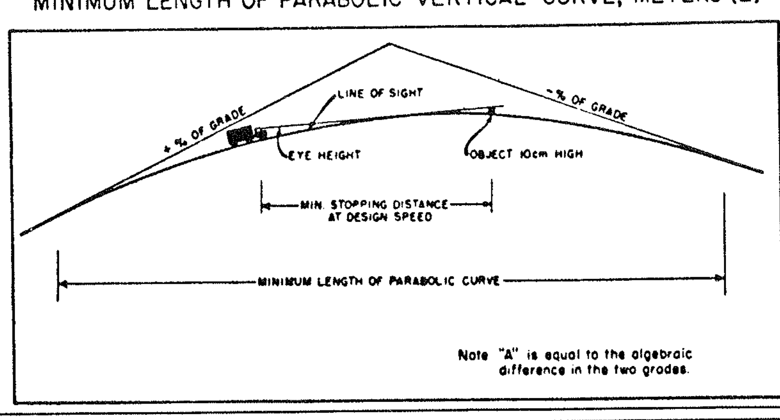
It is standard procedure to use the established design speed in the determination of adequate sight distance for passing, but there may be special situations in which the expected running speeds may be somewhat higher than the design speed. These situations may occur on sectors of a highway which have relatively good horizontal alignment of sufficient length to induce higher speeds for limited distances, and which are located between sectors having restrictive traffic characteristics. Changes in gradients at crests which are sufficiently small to permit consideration of establishing an adequate sight distance for passing, would not be likely to alert the driver and to cause him to reduce his speed to the design speed. Therefore in these locations it is recommended that the anticipated running speed be used in the place of design speed to determine the minimum lengths of the vertical curves which must be used to obtain adequate passing sight distance.

Chart III

NON-PASSING (STOPPING) SIGHT DISTANCE

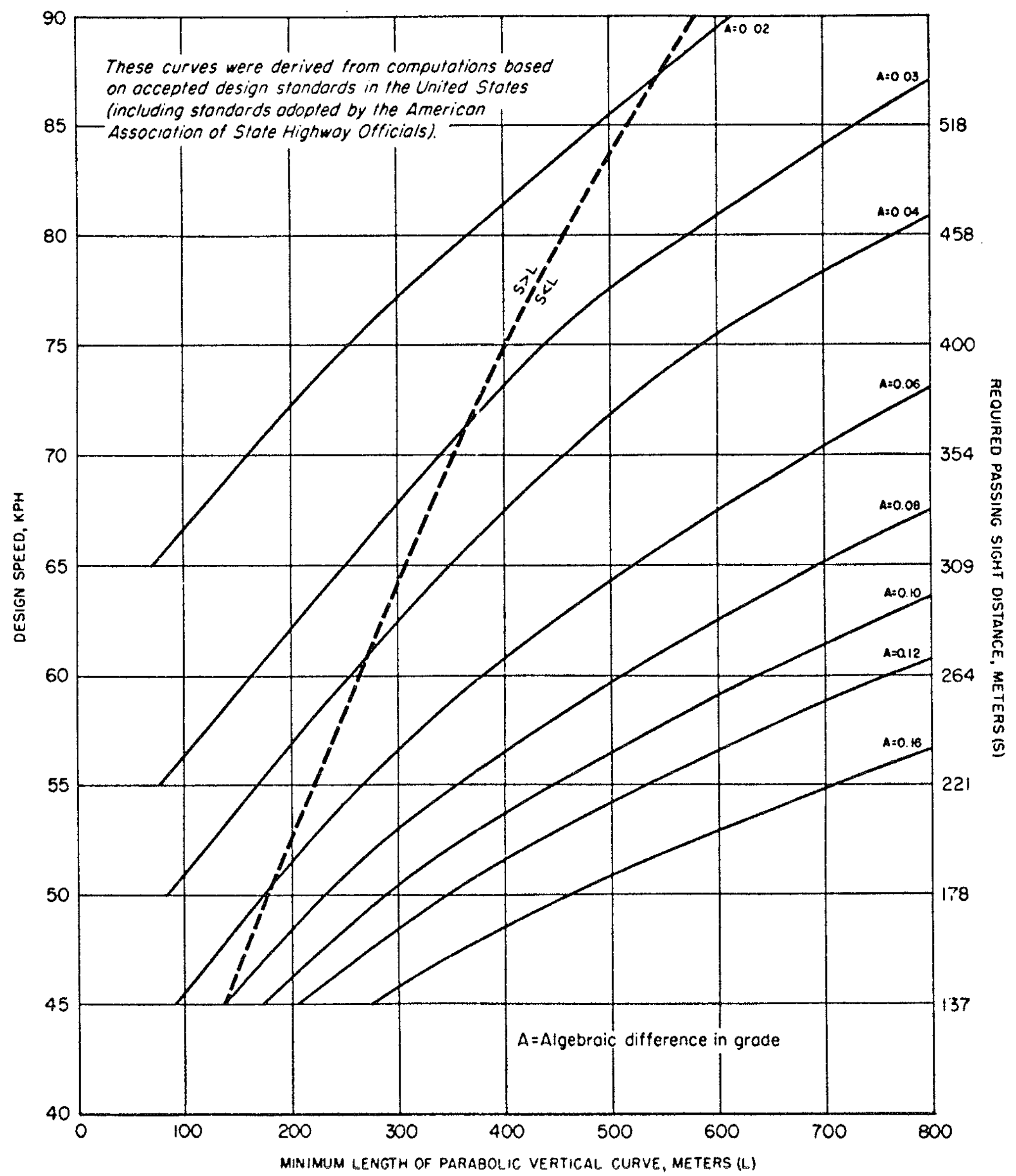
LENGTH OF PARABOLIC VERTICAL CURVES (CREST)

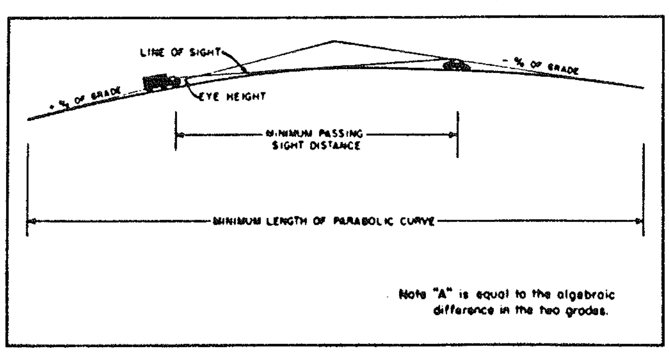




PASSING SIGHT DISTANCE

LENGTH OF PARABOLIC VERTICAL CURVES (CREST)





Although adequate passing sight distance adds to the safety and to the capacity of a highway facility, it is extremely difficult to obtain at critical situations in mountainous and even in hilly terrain. This design requirement is frequently used at minor changes of grade in rugged terrain to reduce the number of locations at which "No Passing" signs or pavement markings must be used. It has been found that a high frequency of no-passing zones produces traffic hazards out of proportion to the actual number, and that traffic capacity is also reduced. Although it is not usually possible to obtain adequate passing sight distance at major changes in grade, important operational advantages can still be gained by avoiding no-passing zones at less difficult situations which may occur on grades approaching but still at some distance from hill and mountain crests.

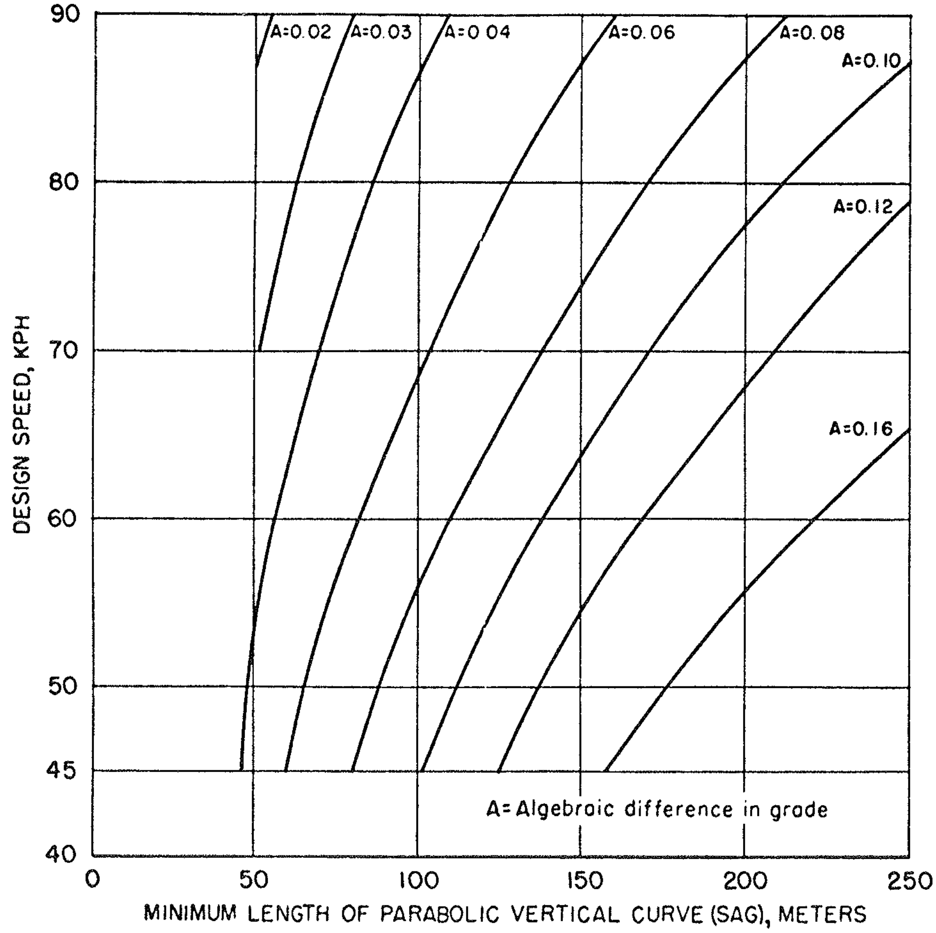
Lengths of Vertical Curves at Sags

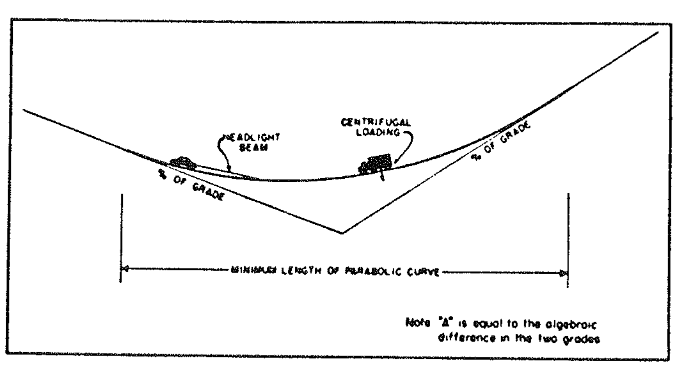
Although the length of parabolic vertical curves is not as critical at sags as at crest locations, certain minimum standards should be maintained. Chart V indicates the minimum length of sag curves at various design speeds for certain changes of grade. This information was derived from certain accepted concepts of vehicle operation in which the effect of headlight beams, centrifugal force and other factors were considered. Under ordinary conditions sag curves may be lengthened somewhat without large increases in cost, and frequently longer curves are used either for appearance or to raise the elevation of the lowest point on the grade.

Chart V

MINIMUM LENGTH OF PARABOLIC VERTICAL SAG CURVES

These curves were derived from computations based on accepted design standards in the United States (including standards adopted by the American Association of State Highway Officials).





CROSS-SECTION
(Typical Sections, Charts VI and VII)

Pavement Width

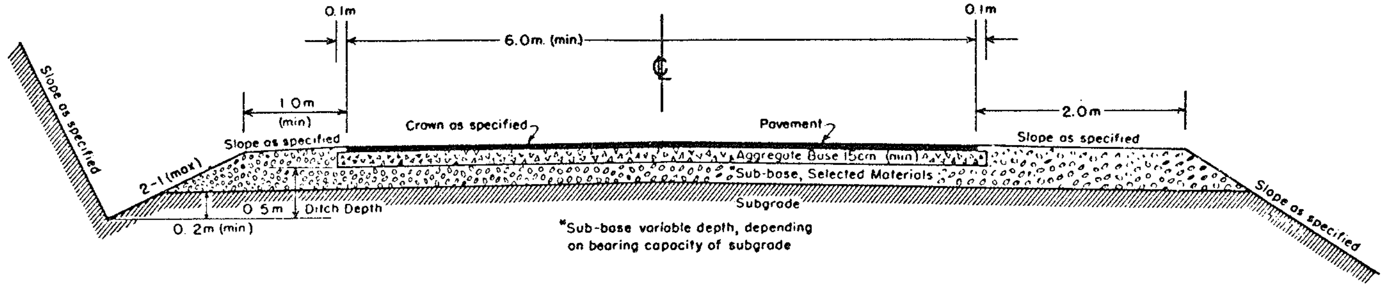
The proper width of a pavement surface is affected by many factors. For example, it is known that narrow shoulders, or obstructions near the edge of a pavement, reduce the effective pavement width by creating a psychological hazard. Also, it has been established that by using the clearance distance between two oncoming vehicles as a criterion for effective width, increases in pavement width beyond a certain dimension appear to offer diminishing returns. On the other hand, even beyond the point at which center clearance is no longer materially improved, it must be recognized that the additional paved surface width outside of the vehicle wheel tracks tends to protect the adjacent shoulder from undue traffic wear, and to provide a safer facility. Higher design speeds require wider pavements. It has been proven that wider pavements provide greater traffic capacity, through rather broad ranges of width. In the selection of a design standard for pavement width, all of these factors should be considered and balanced against construction costs.

The design standards for pavement widths shown in Tables 1 and 2 were developed from data taken from a number of sources. The ranges in each category were intended to be sufficiently broad to cover all normal situations. It will be noted that the standards are not greatly reduced in mountainous terrain, although design speeds are much lower. This is due to the fact that the standard widths of shoulders have been reduced in these difficult areas because of the high cost of grading, and it was determined that the resulting effective widths of the pavement therefore would be reduced by an amount which would conform to the lower design speeds. Also, because of the excessive erosion which can be expected along the pavement edges on steep grades, the pavement width was maintained to protect the shoulder from the action of traffic, which would tend to increase the severity of this problem. No provision was made to increase the effective width to provide for the lower standards of horizontal alignment, since pavement widening is specified on curves.

In order to illustrate the various combinations of pavement widths and shoulder widths which have proven to be practical in certain areas,

TYPICAL CROSS-SECTION IN MOUNTAINOUS TERRAIN

MINIMUM DIMENSIONS
PRIMARY ROUTES



TYPICAL CROSS-SECTION IN LEVEL AND ROLLING TERRAIN

MINIMUM DIMENSIONS
PRIMARY ROUTES

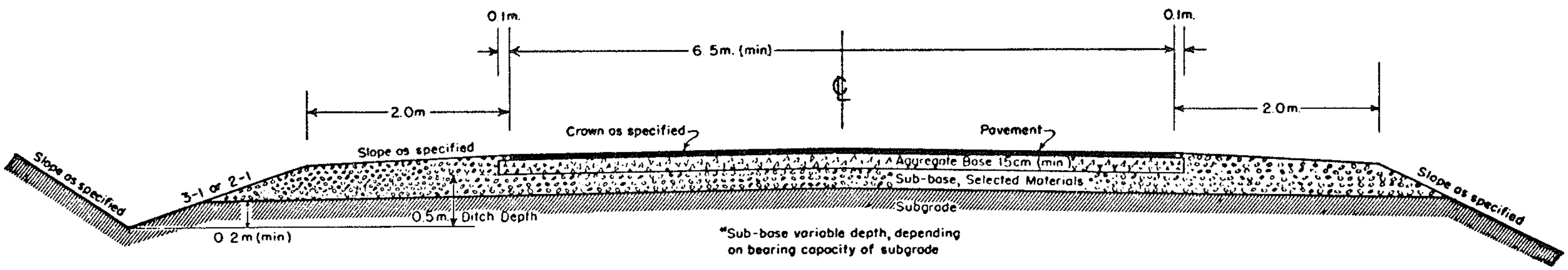
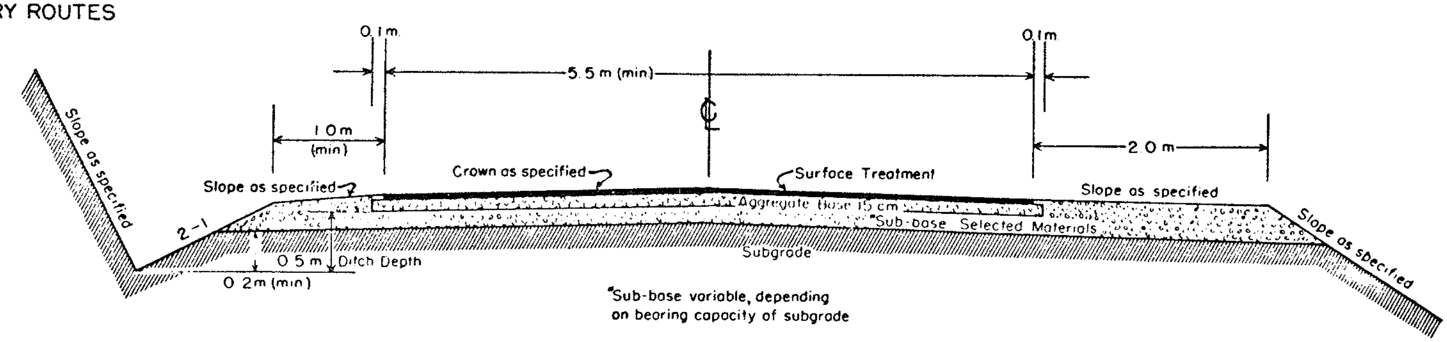


Chart VI

TYPICAL CROSS-SECTION IN MOUNTAINOUS TERRAIN

MINIMUM DIMENSIONS
SECONDARY ROUTES



TYPICAL CROSS-SECTION IN LEVEL AND ROLLING TERRAIN

MINIMUM DIMENSIONS
SECONDARY ROUTES

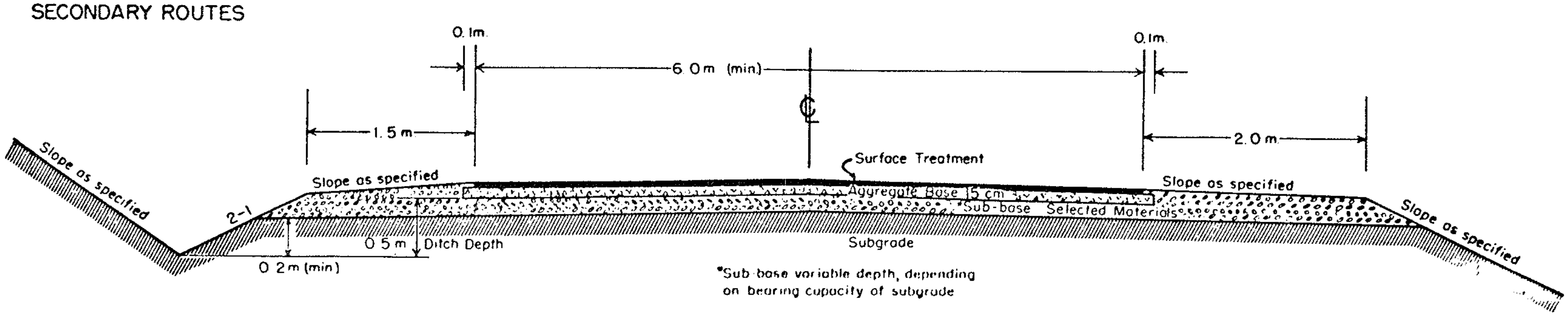


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Chart VII

TABLE 5

MINIMUM WIDTH OF SURFACING FOR TWO-LANE RURAL HIGHWAYS

(With adequate shoulders)

Design Speed	Average Daily Traffic for Design Period			
KPH	50-300	300-650	650-1200	1200-over
45	6.1 m.	6.1 m.	6.1 m.	6.7 m.
65	6.1	6.1	6.1	6.7
80	6.1	6.1	6.1	7.3
100	6.1	6.7	6.7	7.3

TABLE 6

MINIMUM WIDTH OF ROADWAY

	Average Daily Traffic for Design Period			
	Under 600	600-1200	1200-2500	over 2500
Shoulder Width	1.2 - 2.4 m.	1.8 - 3.0 m.	2.5 - 3.0 m.	3.0 - 3.6 m.
Pavement Width (Design Speed 80 KPH)	6.1 m.	6.1 m.	6.1 m.	7.3 m.
Total Width of Roadway	8.5 - 10.9 m.	9.7 - 12.1 m.	11.1 - 12.1 m.	13.3 - 14.5 m.

two additional tables have been included. Table 5 includes a list of recommended pavement widths for certain traffic volumes and indicated design speeds. These dimensions are based on the assumption that adequate shoulders have been provided. Table 6 was developed from data from the same source. It lists general recommendations for total roadway widths, for a design speed of 80 KPH.

The widths of untreated aggregate surfaces, included only in the standards for secondary routes (Table 2), are listed with those of the paved surfaces. The indicated width is intended to apply only to the full depth section of the aggregate surface, with the edges feathering out over the surface of the shoulder.

The crown on a pavement or aggregate surface must be adequate to provide rapid runoff for water falling on the surface; however, an excessive crown can become a hazard to traffic. The minimum amount of crown is a direct function of surface texture and the accuracy to which the surface can be constructed. As indicated in Tables 1 and 2, the amount of crown necessary for aggregate surfaces is at a maximum, while a minimum crown is provided for a high-type bituminous surface.

Shoulders

The shoulders on a highway serve two general basic functions -- the support and protection of the adjacent pavement and the provision of a space for the use of vehicles in emergencies, including the protection of traffic along high embankments. Shoulders also provide an area for the use of pedestrian and animal traffic. There is a direct relationship between the width of shoulders and the effective width of the pavement, and the design of shoulders therefore becomes a part of the general considerations of establishing pavement width. (See Table 6).

Since the established width of shoulders directly affects the width of the standard cross-section in both cuts and fills, it is a major influence in determining the amount and the cost of grading. For this reason, the selection of a standard shoulder width becomes a very important factor in establishing standards of design.

In mountainous terrain, where design speeds are low, the total width of the roadway (shoulders plus pavement) may be somewhat less than in areas where higher speeds are provided. Since pavement surfaces must be maintained at certain minimum widths, wide shoulders in rugged terrain are difficult to justify, particularly in cuts, because of excessive costs. However, shoulders less than 2.0 m. wide are not adequate for safe emergency stops. It is assumed that where standard widths are established which are less than 2.0 m., efforts will be made to provide wider sections of shoulders at frequent intervals along the highway, at distances which should not be longer than 200 m., and preferably 100 m., in order to provide locations for emergency parking. These sections of wider shoulders (2.5 - 3.0 m.) should be provided even in situations where relatively large amounts of additional excavation would be required to construct them.

The minimum shoulder widths which are listed as standard are intended to indicate the width which should be adopted as a minimum, and it is assumed that wider shoulders will be provided at all locations in which construction costs would not be unduly increased. In level and rolling terrain, a wider standard normal width is recommended in the list, in addition to the minimum width. Also, it is recognized that there may be some short sections, especially in mountainous and hilly terrain, in which the minimum standard width may be extremely difficult or expensive to construct, and that occasional exceptions to the standard may be justified. The proper selection of a practical standard will permit an economical use of the pavement surface and improve the overall inherent safety characteristics of the facility.

The slope of the shoulder surface should be sufficient to provide adequate runoff of the water falling on the pavement and shoulder area but an excessive slope may be hazardous to traffic. The proper standard slope should be determined by the type and texture of the shoulder surface.

Side Ditches

The standard side ditch cross-sections are shown in Charts VI and VII, Typical Cross-Sections. These minimum dimensions are intended to provide adequate drainage for the subgrade and pavement structure, with a practical inside slope for ease of mechanical maintenance and traffic safety. The standard cross-sections provide adequate runoff capacity for normal situations. In locations where runoff requirements exceed the capacity provided by the minimum section, the cross-sectional area may be increased by constructing a side ditch with a flat bottom, or the inside slope may be extended to provide both a wider and deeper side ditch. The use of additional cross-drain culverts may eliminate the need for larger side ditches.

Bank Slopes

The standard slopes for back-slopes in cuts, as shown in Tables 1 and 2 are intended to offer only a general guide for design. The type and stability of soil and the internal drainage situation should determine the slope which is used in design. In many mountainous locations, the slope is dictated largely by the existing slope of the original ground. Known slide conditions may indicate flatter slopes, or ledges to be constructed at the back of the side ditches to retain small amounts of slide material. Rock slopes may be constructed almost vertically (usually $\frac{1}{4}:1$), but they should either have benched areas part way up the slope, or sufficient space should be left at the back of the side ditches to retain fallen rocks.

Certain types of soils, frequently wind-deposited silts, lateritic or volcanic materials, are especially subject to severe erosion by water falling directly upon the cut slopes. In these situations the slopes are sometimes maintained at very steep angles in order to protect them from the direct effect of rainfall. In deep cuts, these steep slopes may bring about a series of localized failures, in which the high banks fail in shear under the weight of saturated soil above and behind the slopes, allowing the material in the banks to fall into the side ditches or even onto the roadway. When these types of soils are encountered, establishing the design slopes for individual cuts becomes a problem of obtaining the most practical balance between these two conditions.

There is seldom justification for constructing steep slopes in shallow cuts. The reduction of erosion, ease of maintenance and general appearance are all factors which encourage the use of flat slopes (2:1 or flatter) in shallow cuts.

Fill slopes should be somewhat flatter than the natural angle of repose of the material which makes up the embankment. Flatter slopes increase the stability of the subgrade and offer more support to the pavement.

Slopes on low fills are usually flattened for the same reasons as in shallow cuts.

Maximum Rate of Superelevation

If there were no practical limit to the amount of superelevation to which a pavement and shoulders could be constructed, there would be no theoretical limit to the speed at which any highway alignment could be travelled. However, the maximum amount of superelevation which can be employed with safety is limited by the fact that certain vehicles may not be traveling at the design speed, and that excessive superelevation would result in overturning, side sway or loss of control of the vehicle. In areas where there is little or no snow or ice the maximum rate of superelevation is usually considered to be 12%, as indicated in Tables 1 and 2. This value has also been used in the determination of minimum radius of curvature and other design standards, as listed in this report. Where snow and ice conditions can be expected, 10% or 8% is recommended.

Easement Curves

During the past ten years most highway agencies throughout the world have adopted some type of easement curves at the ends of horizontal curves. Most agencies use some type of modified spiral, since a spiral provides both a natural pattern for horizontal transition and an incremental requirement for superelevation which can be met by smooth vertical curves at the edges of the pavement. Modifications of a true spiral are made only for ease in field layout work. Most highway spiral formulae provide easement curves which are longer than are required for horizontal control, but which permit a proper transition distance for the superelevation.

Since most highway agencies have already adopted some method for establishing curve easements, and the various methods are described in many handbooks of highway engineering, no specific descriptions or standards are included in this report. However, Table 7, showing the

RATE OF SUPERELEVATION (%) AND MINIMUM LENGTH OF SPIRAL CURVE

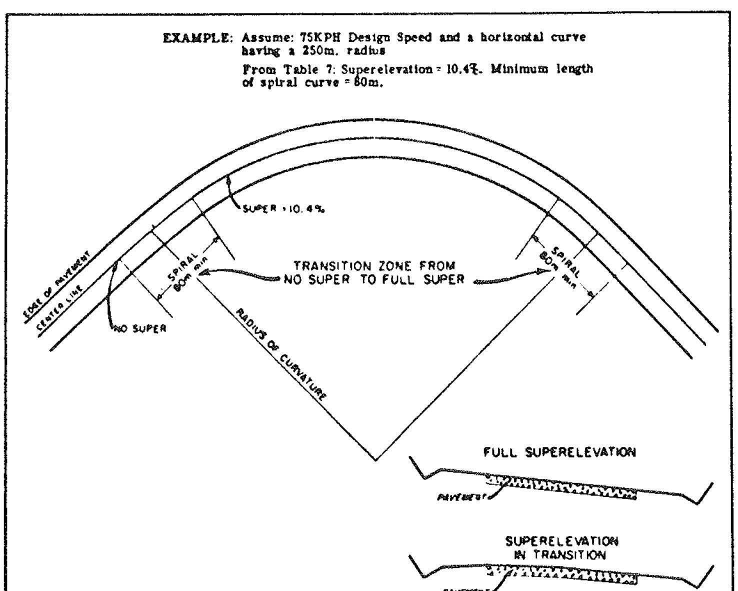
Maximum Superelevation - 12%

Radius m.	V = 45 KPH Super.,	L,	V = 60 KPH Super.,	L,	V = 75 KPH Super.,	L,	V = 90 KPH Super.,	L,
1000	-	30 m.	1.9 %	40 m.	2.8 %	45 m.	3.6 %	55 m.
800	1.5 %	30	2.7	40	3.8	45	4.8	55
600	2.1	30	3.5	40	4.9	45	6.2	55
500	2.6	30	4.2	40	5.9	45	7.4	65
400	3.2	30	5.2	40	7.2	60	9.2	75
300	4.0	30	6.7	50	9.2	75	11.2	90
250	4.7	30	7.7	55	10.4*	80*	12.0	90
200	5.9	35	9.1	60	12.0	90		
175	6.6	40	9.8	65				
150	7.3	45	12.0	65				
100	8.8	55						
80	9.8	60						
60	12.0	60						

"L" is the minimum recommended length of spiral and superelevation run-off.

Spiral curves not essential above heavy line.

* Example



rate of superelevation and the minimum lengths of easement curves for various radii or horizontal curves at different design speeds, is included as a general guide.

Pavement Widening at Curves

It is recognized that a driver experiences greater difficulty in staying in his proper lane when he is making a sharp horizontal curve. Also, large trucks occupy a wider space along the traffic lane on a curve, because of the overhang of the vehicle over the front axle and because of "off-tracking," which is the failure of the rear wheels to follow in the paths of the front wheels. For these reasons, it is standard practice to widen curves having a radius of 150 m. or less. This widening is placed on the inside of the curve.

In Table 8, the extra pavement width has been specified according to the severity of the curve and also according to the standard width of the pavement. Although no design speed is indicated in the table, the maximum speed appropriate to the indicated radius of curvature may be used.

Although pavement widening may start at the beginning of the curve, it may also be adjusted to complement the spiral easement curve which is introduced.

Structure Widths

A large structure at grade is likely to become a critical point along a highway. It is frequently impossible, or at least impractical, to extend the full width of a roadway section on to the structure, and some restriction in the free movement of traffic may be expected. However, a structure must be considered as one of the more permanent items in the investment which is being made and it is important that adequate roadway dimensions be established to provide safe operations for future traffic.

It is generally agreed by most highway agencies that longer highway structures should be designed to somewhat narrower roadway dimensions than are needed for shorter structures, for two reasons. First, economic considerations require that highway users accept more severe

TABLE 8

RECOMMENDED WIDENING ON INSIDE EDGE OF PAVEMENT AT HORIZONTAL CURVES

Radius of Curvature (meters)	Widening on 7.3m pavement (meters)	Widening on 6.7m pavement (meters)	Widening on 6.1m pavement (meters)
60 and less	.60	.90	1.20
80	.40	.70	1.00
100	.20	.50	.80
120 to 150	0	.30	.60

Widening shall be tapered off in a distance of 15 meters at each end of the curve.

restrictions in freedom of movement at a few locations where investments are high. Second, longer structures are usually easier for drivers to see or recognize, and therefore less danger is experienced in entering the zone of restricted movement than is experienced at entrances to shorter structures. Certain structure sites may produce situations which are exceptions to this general policy.

The standard clearance or width dimensions of structure listed in Tables 1 and 2 are divided into two categories. The clear roadway width is the width between the vertical faces of curbs or wheel guards. The clearance between trusses or handrails provides additional space for vehicle side clearance. Sidewalks are recommended, at least on one side.

Structures are classified into three lengths. Those having a total length of less than 25 m. have standard roadway widths and clearance widths between trusses greater than those for longer structures up to 250 m. in length. On structures longer than 250 m. total span, smaller widths are listed as standard.

Horizontal Sight Distance

The sight distance which is available to the driver across the inside of a horizontal curve frequently becomes a critical operational limitation. Although this sight distance may be restricted by such features as trees and undergrowth, or buildings, the primary consideration of this factor by the designer usually involves restrictions due to backslopes in cut sections. In these restrictions the sight distance may be increased either by using a curve with a longer radius, or by changing the cross-section design to provide a flatter back-slope, or to provide an increased distance out to the line of the side ditch. From the standpoint of the designer, horizontal sight distance frequently can be considered as a function of the cross-section design.

Design stopping sight distance (horizontal) is important from the standpoint of safety, and of the maintenance of a uniform speed. Chart VIII shows the required stopping distances for various design speeds.

Chart IX illustrates the situation which is assumed to establish stopping sight distance. Chart X indicates the minimum offset from the center line to the point of sight restriction, for various radii of curvature and for several design speeds.

It will be noted that on sharper curves the stopping sight distance requirements are met by a normal or nearly-normal cross-section, if the limiting speed for the specific curve is accepted. On flatter curves, the sight distance requirements for stopping call for a considerable change from the normal cross-section design, if the design speed appropriate to the radius of curvature is assumed. In practice, this situation may result in the flattening or the moving back of the back-slope in cuts on the inside of the curve, if the established design speed for the specific sector of highway is such that adequate sight distance would not otherwise be available. Conversely, on curves to be constructed in cuts, if a normal cross-section design is to be used, the horizontal stopping sight distance, rather than the ability of a vehicle to travel around the curve at a certain speed, will become the limiting factor in establishing the minimum radius of curvature for all except very low design speeds.

For these reasons it is especially important that the design be checked for adequate stopping sight distance at critical horizontal curves on highway sectors which have been assigned relatively high design speeds.

Horizontal sight restrictions other than those caused by back-slopes in cuts should also be checked, both by the designer and the engineer in the field during construction. For example, the amount of clearing which is to be done on the right-of-way may be increased at certain points to insure adequate sight distance.

In Charts IX and X it is assumed that both the vehicle and the object to be sighted are located on the curve and in the inside lane. This represents the most critical situation for a curve of a specified radius of curvature, since curves which are too short to accommodate this situation will require less offset distance from the center line to the point of sight restriction.

The provision for frequent passing opportunities adds to the capacity and to the inherent traffic safety characteristics of a highway. Adequate horizontal passing sight distance forms an important part of this consideration.

Chart XI illustrates the situation which is assumed to establish passing horizontal sight distance. Chart XII indicates the minimum offset from the centerline to the point of sight restriction, for various radii of curvature and for several design speeds.

Since a safe passing operation requires a relatively long sight distance, curves which approach the minimum radius of curvature for a specified speed usually cannot be constructed to provide adequate horizontal sight distance for passing in cuts. This fact is indicated in Chart XII. Also, the possibility of increasing the radius of curvature to provide a longer horizontal sight distance is usually superseded by other considerations which are important to the practical location and design of the highway. Therefore, the consideration which is given to obtaining adequate passing horizontal sight distance is usually confined to checking the offset distance to the point of sight restriction on the non-critical or flatter curves in the sector. At certain of these curves it may be found practical to obtain an adequate offset distance by flattening the backslope, or by widening the entire cross-section.

In Charts XI and XII it was assumed that both the passing vehicle and the approaching vehicle were located on the curve. As in the consideration of stopping sight distance, this represents the most critical condition for any given radius of curvature.

In some locations a series of relatively flat curves, including reverse curves, may offer possibilities for obtaining passing sight distance which cannot be determined readily by the use of formulae or graphs. In these conditions it may be possible to use large-scale plan sheets and a scaled straight-edge to check the available horizontal sight distance, assuming a normal cross-section design, and to determine the approximate amount of additional construction or clearing necessary to increase the sight distance at selected locations to avoid no-passing zones. In addition, visual inspection made in the field before and during construction may aid in establishing adequate passing sight distance at frequent intervals along the highway, including locations having generally poor horizontal alignment. The required passing sight distance for various design speeds can be obtained from the scale on the right-hand margin of Chart IV.

STOPPING DISTANCE for the determination of
HORIZONTAL SIGHT DISTANCE by Design Speed, KPH

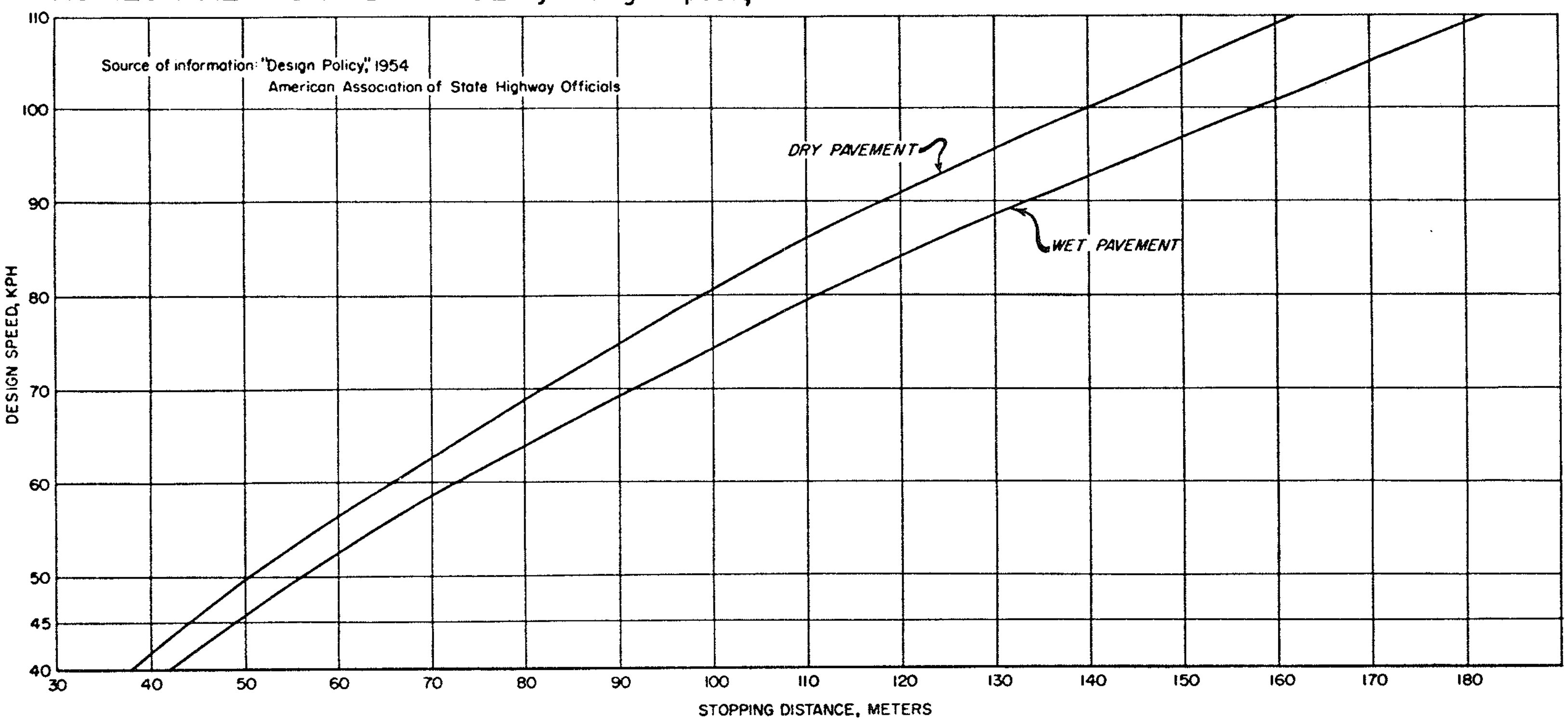
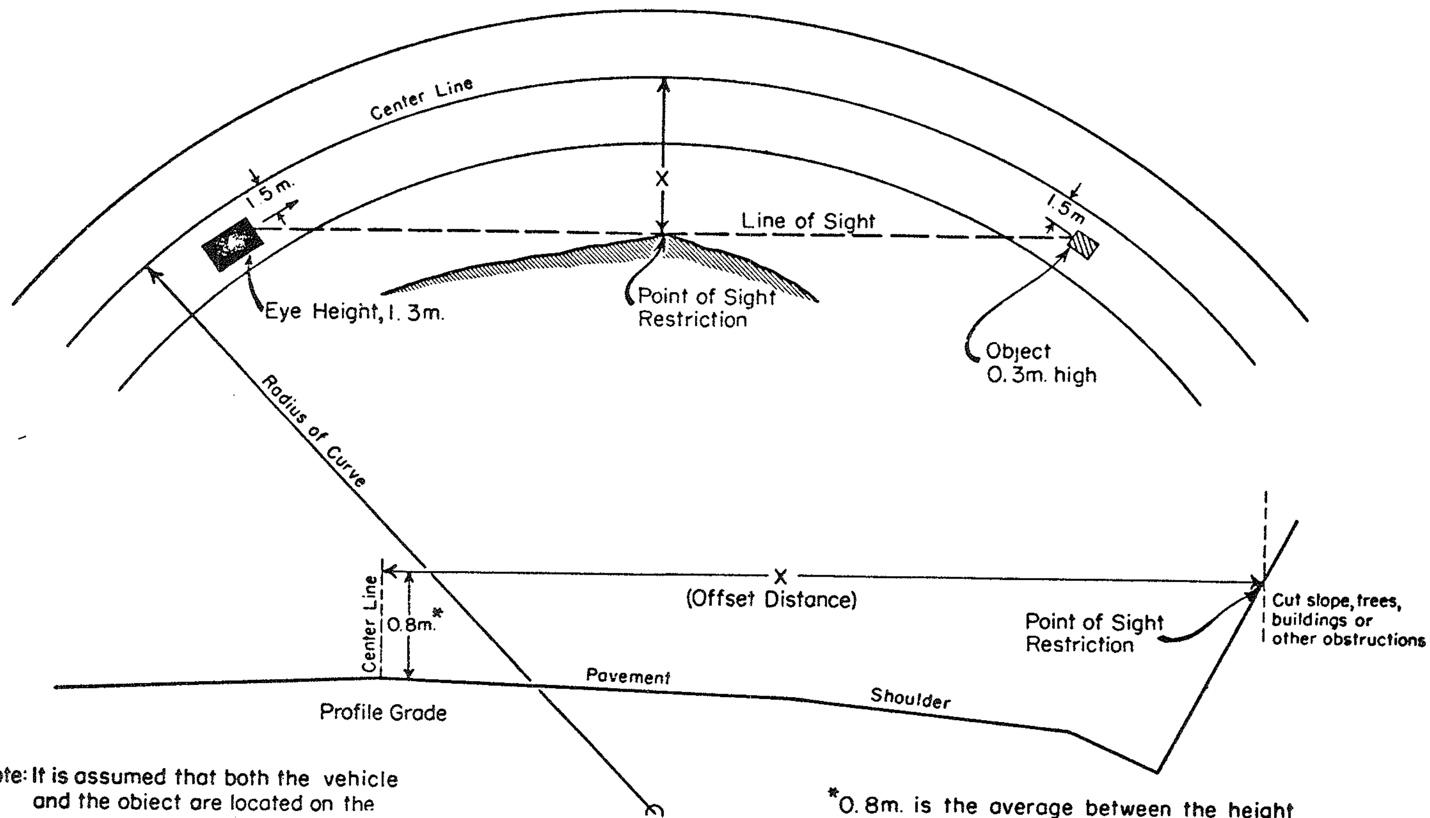


Chart VIII

RESTRICTED HORIZONTAL SIGHT as related to STOPPING SIGHT DISTANCE

(Ref.: Chart X)



Note: It is assumed that both the vehicle and the object are located on the curve (inside) and not on the adjacent tangents.

*0.8m. is the average between the height of eye and the height of the object

Chart IX

Minimum Offset Distance (Radial) to
HORIZONTAL SIGHT RESTRICTION
From Center Line on Curves, for Specified Design Speeds
STOPPING DISTANCE
(Ref.: Chart IX)

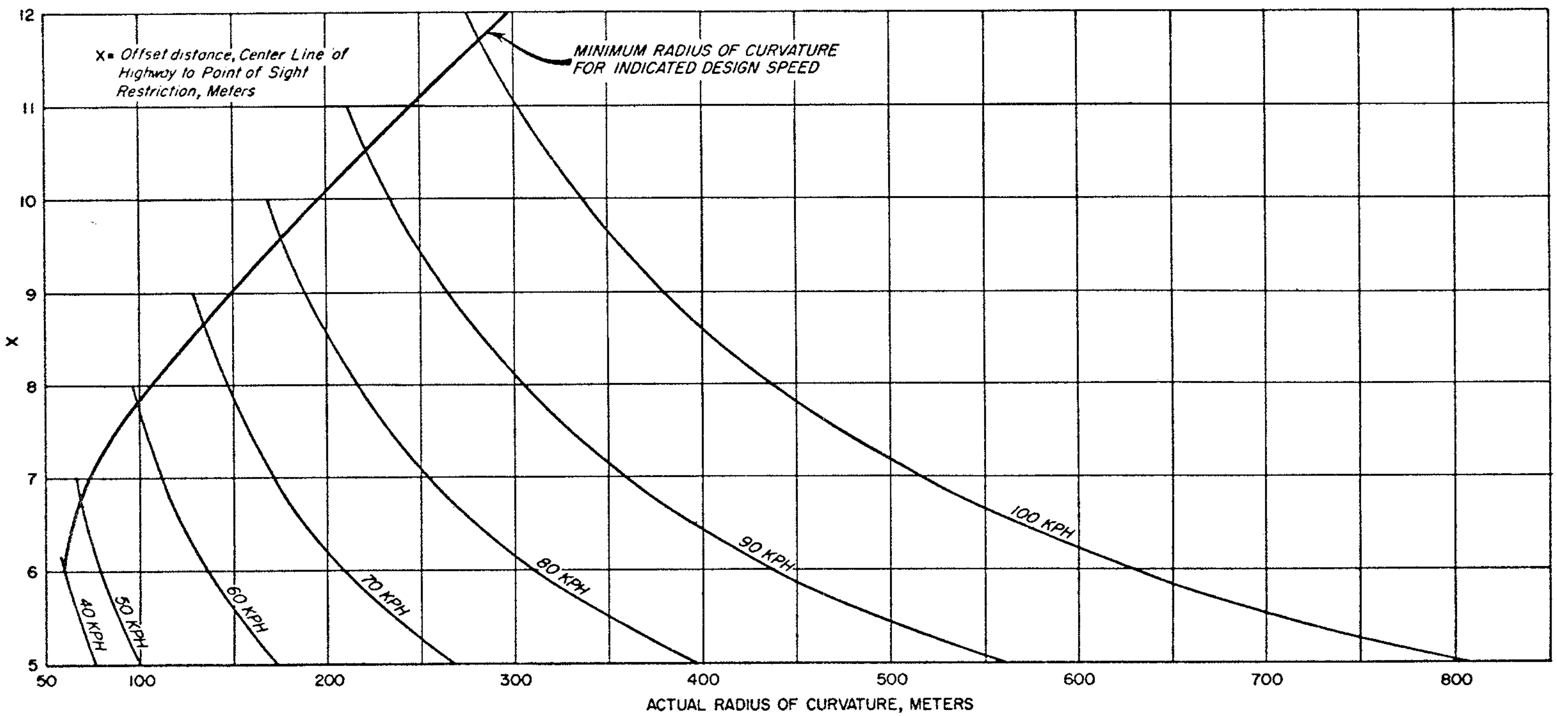


Chart X

RESTRICTED HORIZONTAL SIGHT as related to PASSING SIGHT DISTANCE

(Ref.: Chart XII)

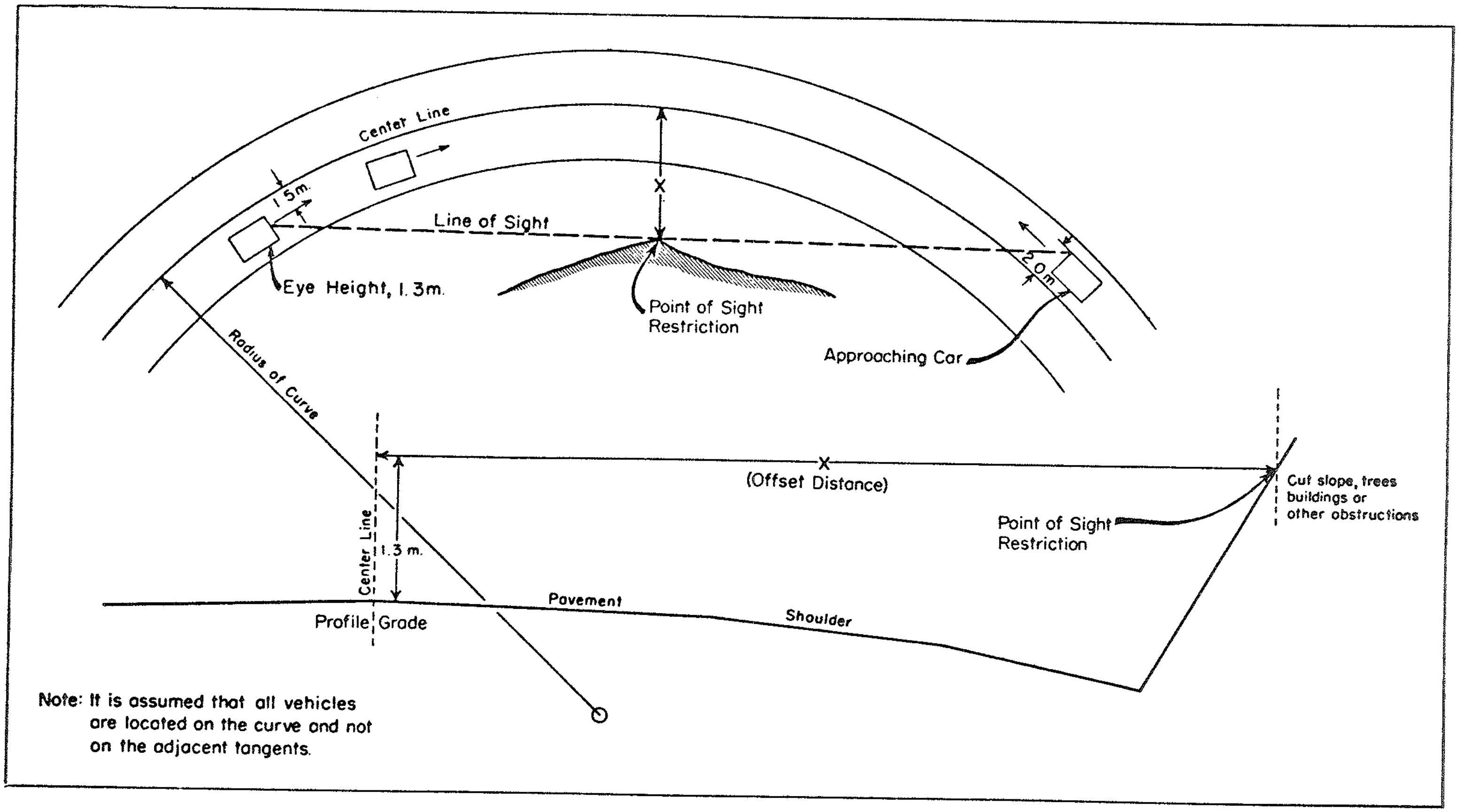
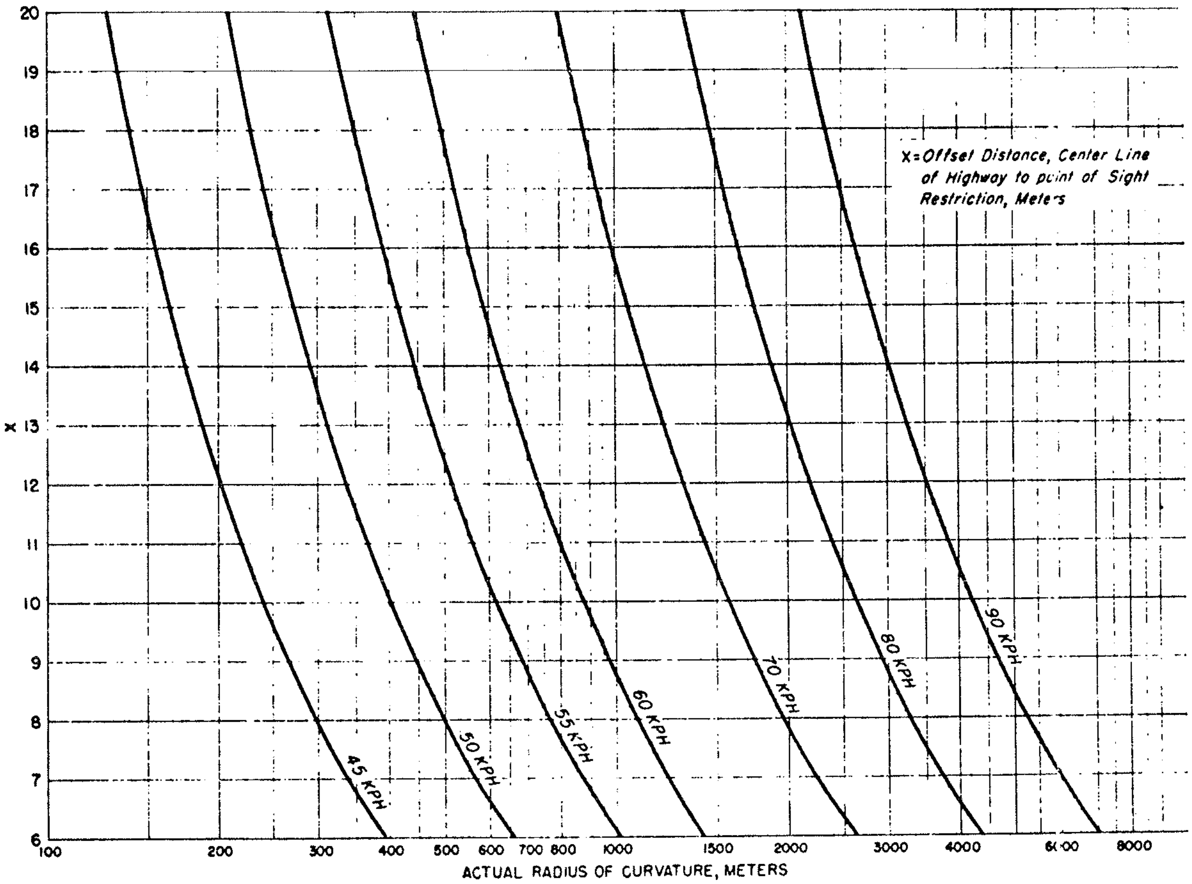


Chart XI

Chart XII

Minimum Offset Distance (Radial) to
HORIZONTAL SIGHT RESTRICTION
From Center Line on Curves, for Specified Design Speeds
PASSING DISTANCE
(Ref: Chart XI)



The established design speed for a specific sector of highway forms the basis for the design of the entire sector. It is assumed that the driver will recognize the various critical situations as they are approached, and that a balanced design of the various highway elements will tend to establish the normal running speed at a speed which will be approximately equal to the design speed. However, in sectors where difficult terrain has dictated the use of a low design speed, there may be certain sections of the highway which have relatively straight alignment for a sufficient distance to induce an appreciable increase in running speed. In these special situations it is recommended that the anticipated running speed be used in the place of design speed in the determination of adequate passing horizontal sight distance.

STANDARDS FOR SECONDARY ROUTES

The geometric design standards for secondary routes, listed in Table 2, are based upon lower design speeds and lower traffic capacity requirements. A generally lower level of traffic service is also assumed because of the fewer highway users to be served and the shorter average distances of travel which can be expected.

The lower speeds directly affect the standards for horizontal alignment and the requirements for sight distance. Other elements of design, less directly affected by speed, are derived from the application of fundamental principles of vehicle operation and driver performance to accepted design standards for primary routes, with appropriate reductions which are indicated by accepting lower traffic capacity and a lower standard of traffic service. These reductions result in generally lower costs.

The same important advantages of balanced and consistent design can be gained from the use of pre-established standards in the design of secondary routes as can be expected from their use on higher type facilities.

Road to Asmara, Ethiopia.

AMERICAN ASSOCIATION OF
STATE HIGHWAY OFFICIALS

A POLICY
on
GEOMETRIC DESIGN
of
RURAL HIGHWAYS

1965

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Preface

By Joseph Barnett

The Committee on Planning and Design Policies was organized in 1937 upon authority of the Executive Committee of the Association. The purpose of the committee is the formulation of administrative policies looking toward the incorporation in practice of highway design features which will result in maximum safety and utility. The committee outlines the program of work, after which personnel assigned by the Bureau of Public Roads under supervision of the Secretary investigates each subject and prepares tentative discussions with indicated design controls and guide values. These are criticized, revised, and supplemented by members of the committee until a policy on each subject acceptable to the committee is completed, after which it is submitted through the Committee on Standards to the Executive Committee for letter ballot by the States.

In the period 1938 to 1944 the Committee on Planning and Design Policies developed seven policies which were adopted by the Association and printed as separate brochures. In 1950 the group was reprinted without change and bound as a single volume entitled Policies on Geometric Highway Design. A Policy on Geometric Design of Rural Highways, adopted May 3, 1954, was a complete reworking of the separate policies into a cohesive single volume which completely superseded the group of seven former policies. This volume which came to be commonly known as the "Blue Book" received wide acceptance. Seven printings had been made by 1965. Inevitably some of the material failed to reflect the ever changing data in the dynamic art of highway engineering and the Executive Committee authorized the Committee on Planning and Design Policies to update the Policy. This present policy is the result. While a complete rewrite was not attempted the salient features and data were brought in line with present knowledge to such an extent that the volume may be considered a revision as the term is generally understood.

While all parts of the book were subject to updating it is well to state some of the principal items which were revised. The design vehicles were changed in dimension to conform to motor vehicle trends particularly semitrailers which were designed in accordance with wheelbase rather than length. The data curves of vehicle usage were based on vehicle miles of travel rather than on manufacturers' lists as in the 1954 publication. Data for design speeds of 75 and 80 miles per hour were added and sight distance design criteria were changed to result in some increase in lengths of vertical curves. Capacity data were revised to conform to the new capacity manual about to be published and the sections on climbing lanes revised accordingly. Design values for speed-change lanes were increased to reflect current operating experience. Examples of design were changed to reflect a decade of new design and construction.

The question arises as to whether the changes herein to update the Blue Book, after a period of a decade, are bold or radical enough to reflect sufficient vision so that highways constructed in rural areas with this book as a guide will be fully adequate for the life of the highway. The answer is that the contents are based on the facts and trends as they were found. To design highways for the future is not the province of the maker of guides and standards but rather that of the designer himself, who, in the planning and design stages, must choose values for those elements which

are basic to highway design from the data available to him and the trends which reveal those values. What will the likely volume of traffic be? What kind of traffic should be designed for? In relation to the terrain, the traffic, and the funds available, what type of highway should be planned and what should be the common denominator of assumed design speed? If these factors are chosen wisely, with proper consideration to values above minimums where justified for the conditions, then this Blue Book, as updated, will be an excellent guide for the use of the engineer in designing a highway in rural areas that will, for the least cost and with maximum safety, give high quality traffic and transportation service for the predetermined number of years. The 1954 Blue Book proved to be a valuable tool and served highway engineers well. It is hoped that this updated Blue Book will be equally serviceable.

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Chapter I

INTRODUCTION AND SUMMARY OF PRINCIPAL DESIGN CONTROLS AND GUIDE VALUES

Summarized from Chapters II through IX

Introduction

The art of modern road building is the result of gradual development to meet changing traffic needs and broadened knowledge gained through experience and research. In geometric design—design of the visible features of a road—early practice relied heavily on field location to provide a suitable road to meet standards for alinement, gradient, and other features at minimum cost. Prime recognition was given the cost of various elements of construction and availability of materials. This procedure was satisfactory for the needs of traffic at the time. Few could foresee the tremendous growth in motor vehicle transportation, and even if they could have foreseen it highway engineers could have done little about the additional requirements. The limited funds rightly were devoted to increasing the mileage of improved roads.

Greater numbers of vehicles, changes in their characteristics, a growing knowledge of driver behavior, particularly in the presence of other drivers, and increase in the number of accidents brought into prominence the geometric design of highways to provide maximum service with minimum hazard at reasonable cost. Increased knowledge in other phases of design and improvements in construction equipment, techniques, and materials made it economically feasible to provide facilities with standards considered too costly before then. The relation of location and design to costs, physical conditions, and fund availability will ever be important in highway engineering, but attention must be given to location and design to fit future traffic needs. A complete highway incorporates not only safety, utility and economy, but beauty as well. Care should be used in the selection of location and geometric elements so that the highway will be beautiful in appearance, compatible with its environment and give vehicle occupants pleasing views from the highway. This policy develops the guide values and details for geometric design which best fit the requirements of motor vehicles as they are constructed and operated currently, and insofar as it is possible to foresee, as they will be in the future.

Safe and efficient highway transportation requires increasing attention to enforcement and driver education as well as to engineering. Our highways necessarily involve conditions wherein drivers singly or collectively must accept regulations and conform with established operational patterns for their mutual benefit. The role of enforcement officials in this regard directly supplements that of the engineer in the design, construction, and operation of the highway. Effective training has demonstrated that proper driver attitude is a major item in enforcement which can be attained through organized effort toward education of all drivers. Discussion of enforcement and education features are not included herein, but it should be recognized throughout that they are essential supplements to engineering.

This Policy is limited to the geometric features of highway design as distinguished from structural design. It is intended as a comprehensive treatise on the geometric design of rural highways and encompasses practically all general controls and features, except those specifically related to urban conditions. The Policy on Arterial Highways in Urban Areas covers the details of urban design only, with reference to this volume for general guides and controls.

Summary

The following statements are abstracts of the general text and give the principal design controls and guide values established. These are listed in the order presented in the text, chapters II through IX.

DESIGN CONTROLS AND CRITERIA—Chapter II

Topography and Physical Features (*pages 52-53*)

Since topography and land use have a pronounced effect on highway location, geometrics, and determination of the type of highway, information regarding them should be obtained in the early stages of planning and design. This information together with traffic and vehicle data form the major controls for highway design.

Traffic (*pages 53-69*)

The design of a highway should be based on traffic data for that highway. The design hourly volume, DHV, should be representative of the future year chosen for design. It should be predicated on current traffic (existing and attracted) plus all traffic increases (normal traffic growth, generated traffic, and development traffic) that would occur during the period between the current and the future year chosen for design. A period of 20 years is widely used as a basis for design, for which the usual traffic increase on a highway improvement is in the range of 50 to 150 percent. Where the highway is to be a freeway, traffic increase is likely to be higher, in the range of 80 to 200 percent.

On minor, low volume roads, average daily traffic, ADT, often is sufficient. On most highways the DHV, usually the 30th highest hourly volume, is used for design. On highways with unusual or highly seasonal fluctuation in traffic flow it may be necessary to use a design hourly volume other than the 30th highest hour.

The design traffic data should include the following elements:

ADT—current average daily traffic, year specified.

ADT—future average daily traffic, year specified.

DHV—future design hourly volume, two-way unless otherwise specified. DHV usually equals 30 HV.

K—ratio of DHV to ADT, generally 12 to 18 percent.

D—directional distribution of DHV, one-way volume in predominant direction of travel expressed as percentage of total. D varies from about 50 to 80 percent of two-way DHV; average 67 percent.

T—trucks, exclusive of light delivery trucks, expressed as percentage of DHV. As an average on main rural highways, T is 7 to 9 percent of DHV and 13 percent of ADT. Where weekend peaks govern, the average may be 5 to 8 percent of DHV.

For important intersections, data should be obtained to show simultaneous traffic movements during both the morning and evening peak hours.

Design Vehicles *(pages 69-87)*

One or more of four design vehicles should be used as controls in geometric design, the dimensions of which are as follows:

Ref., table II-5

Design vehicle		Dimensions in feet					
Type	Symbol	Wheel-base	Overhang Front	Overhang Rear	Overall length	Overall width	Height
Passenger car	P	11	3	5	19	7	—
Single-unit truck	SU	20	4	6	30	8.5	13.5
Semitrailer combination, intermediate	WB-40	13+27 =40	4	6	50	8.5	13.5
Semitrailer combination, large	WB-50	20+30 =50	3	2	55	8.5	13.5

NOTE: For minimum turning paths, see figures II-3, -9, -11, and -12.

The vehicle which should be used in design for normal operation is the largest one which represents a significant percentage of the traffic for the design year. For design of most highways accommodating truck traffic, one of the design semitrailer combinations should be used. A design check should be made for the largest vehicle expected to insure that such vehicle can negotiate the designated turns, particularly if pavements are curbed. In special cases a design may have to be made to accommodate vehicles larger than the WB-50.

Design and Running Speeds *(pages 87-98)*

A design speed should be selected and used for correlation of the physical features of a highway that influence vehicle operation. The assumed design speed should be a logical one with respect to the character of terrain and the type of highway. Some geometric features of highway design, such as superelevation rate, critical length of grade, intersection curves, etc., require consideration of average running speeds.

Average running speed is the average for all traffic or component of traffic, being the summation of distances divided by the summation of running times.

Design speeds normally recommended and the corresponding average running speeds for the volume conditions shown are as follows:

MAIN HIGHWAYS

Ref., table II-6

Design speed, mph	Average Running Speed, mph		
	Low volume	Intermediate volume	Approaching possible capacity
30	28	26	25
40	36	34	31
50	44	41	35
60	52	47	37
65	55	50	
70	58	54	
75	61	56	
80	64	59	

The running speed for low volumes is a major design control for certain highway elements, as superelevation, intersection curves, and speed-change lanes.

As high a design speed as practicable should be used, preferably a constant value for any one highway. Where there is variation in terrain and other physical controls, changes in design speed for some sections of highway may be necessary. Design speeds of 75 and 80 mph are applicable only to highways with control of access or where such control is planned in the future.

NOTE: The omitted sections deal with design controls and criteria that are beyond the scope of this compendium.

ELEMENTS OF DESIGN—Chapter III

Stopping Sight Distance *(pages 134-140)*

Design stopping sight distance is the minimum distance required for a vehicle traveling near the design speed to stop before reaching an object in its path. It is the sum of the distances traveled during perception and brake reaction time and the distance traveled while braking to a stop on wet pavement. Sight distance at every point on a highway should be as long as possible and at least as great as the following:

Ref., table III-1

Design speed, mph	30	40	50	60	65	70	75	80
Minimum stopping sight distance, feet	200	275	350	475	550	600	675	750

Design speeds of 75 and 80 mph are applicable only to highways with full control of access or where such control is planned in the future. Stopping sight distance is measured from the driver's eye 3.75 feet above the road surface to the top of an object 6 inches high on the road surface.

The minimum stopping sight distances above reflect passenger car operation, but because of the greater height of eye and lower speeds of trucks the distances shown are applicable to trucks except in the case of a horizontal sight obstruction on a downgrade where a greater sight distance is desirable.

Passing Sight Distance *(pages 140-151)*

Design passing sight distance is the minimum distance required to safely make a normal passing maneuver on 2-lane highways at passing speeds representative of nearly all drivers, commensurate with design speed. Minimum passing sight distance for level or nearly level grades should be as follows:

Ref., table III-4

Design speed, mph	30	40	50	60	65	70	75	80
Minimum passing sight distance, feet: 2-lane highways	1100	1500	1800	2100	2300	2500	2600	2700

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Passing sight distance is measured from the driver's eye 3.75 feet above the road surface to the top of an object 4.5 feet high on the road surface.

Passing sight distance on 2-lane highways should be provided over as high a proportion of the highway length as feasible. This proportion should be greater on highways with high volumes than on those with low volumes.

Sight distances should be recorded on the plans for the construction or improvement of highways (pages 149-151).

Design Elements Related to Horizontal Curvature
(pages 152-155)

Table III-9, duplicated on the next page, gives design values for several geometric elements related to curvature for the common maximum superelevation rate of 0.10 foot per foot. (Tables III-7, -8, and -10 give the same data for maximum superelevation rates of 0.06, 0.08, and 0.12 foot per foot, respectively. Only one table is generally applicable in one State or area.)

Maximum Superelevation *(pages 155-157)*. A maximum superelevation rate (e) of 0.10 foot per foot is generally desirable and is used as a nationally representative value. Other rates may have application on some types of highways and in certain regions.

Values for Design Elements Related to Design Speed and Horizontal Curvature

Ref., table III-9

D	R	V=30 mph e	V=30 mph L-Feet 2-lane	V=30 mph L-Feet 4-lane	V=40 mph e	V=40 mph L-Feet 2-lane	V=40 mph L-Feet 4-lane	V=50 mph e	V=50 mph L-Feet 2-lane	V=50 mph L-Feet 4-lane	V=60 mph e	V=60 mph L-Feet 2-lane	V=60 mph L-Feet 4-lane	V=65 mph e	V=65 mph L-Feet 2-lane	V=65 mph L-Feet 4-lane	V=70 mph e	V=70 mph L-Feet 2-lane	V=70 mph L-Feet 4-lane	V=75 mph e	V=75 mph L-Feet 2-lane	V=75 mph L-Feet 4-lane	V=80 mph e	V=80 mph L-Feet 2-lane	V=80 mph L-Feet 4-lane
0° 15′	22918′	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	240	240
0° 30′	11459′	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200	.022	220	220	.024	240	240
0° 45′	7639′	NC	0	0	NC	0	0	RC	150	150	.024	175	175	.027	190	190	.029	200	200	.033	220	220	.036	240	240
1° 00′	5730′	NC	0	0	RC	125	125	.023	150	150	.032	175	175	.035	190	190	.039	200	200	.044	220	220	.048	240	240
1° 30′	3820′	RC	100	100	.021	125	125	.033	150	150	.046	175	190	.052	190	220	.058	200	260	.065	220	310	.071	240	350
2° 00′	2865′	RC	100	100	.028	125	125	.042	150	150	.058	175	230	.066	190	290	.074	220	330	.082	260	390	.089	290	440
2° 30′	2292′	.021	100	100	.034	125	125	.051	150	180	.069	190	280	.077	220	330	.086	260	390	.094	300	450	.099	330	490
3° 00′	1910′	.025	100	100	.040	125	125	.059	150	210	.079	210	320	.087	250	380	.094	280	420	.100	320	480	.100	330	500
3° 30′	1637′	.029	100	100	.046	125	140	.067	160	240	.087	230	350	.093	270	400	.099	300	450	.100	320	480	.100	330	500
4° 00′	1432′	.033	100	100	.051	125	160	.073	180	260	.093	250	380	.098	280	420	.100	300	450	D max=3.0°			D max=3.0°		
5° 00′	1146′	.040	100	110	.061	130	190	.084	200	300	.099	270	400	.100	290	430	.100	300	450						
6° 00′	955′	.046	100	120	.070	150	220	.092	220	330	.100	270	410	D max=4.5°			D max=4.0°								
7° 00′	819′	.053	100	140	.077	160	240	.098	240	350	D max=5.5°														
8° 00′	716′	.059	110	160	.084	180	260	.100	240	360															
9° 00′	637′	.064	120	170	.089	190	280	.100	240	360															
10° 00′	573′	.068	120	180	.093	200	290	D max=8.5°																	
11° 00′	521′	.073	130	200	.097	200	310																		
12° 00′	477′	.077	140	210	.099	210	310																		
13° 00′	441′	.080	140	220	.100	210	320																		
14° 00′	409′	.083	150	220	.100	210	320																		
16° 00′	358′	.089	160	240	D max=13.5°																				
18° 00′	318′	.093	170	250																					
20° 00′	286′	.097	170	260																					
22° 00′	260′	.099	180	270																					
24° 00′	239′	.100	180	270																					
		.100	180	270																					
		D max=25.0°																							

$e_{max}=0.10$

D—Degree of curve
R—Radius of curve
V—Assumed design speed
e—Rate of superelevation
L—Minimum length of runoff of spiral curve
NC—Normal crown section
RC—Remove adverse crown, superelevate at normal crown slope
Spirals desirable but not as essential above heavy line.
Lengths rounded in multiples of 25 or 50 feet permit simpler calculations.

Maximum Curvature *(page 157)*

Ref., table III-5

Maximum superelevation rate, foot per foot	Minimum radius in feet and maximum degree of curve for design speed, mph, of:							
	30	40	50	60	65	70	75	80
.06	270 21.0	500 11.5	830 7.0	1260 4.5	1480 4.0	1820 3.0	2200 2.5	2500 2.5
.08	250 23.0	460 12.5	760 7.5	1140 5.0	1340 4.5	1630 3.5	1970 3.0	2240 2.5
.10	230 25.0	430 13.5	700 8.5	1040 5.5	1230 4.5	1490 4.0	1780 3.0	2030 3.0
.12	210 26.5	395 14.5	640 9.0	960 6.0	1130 5.0	1360 4.0	1630 3.5	1850 3.0

Design Superelevation Rates *(pages 157-172).* On curves flatter than the maximum for a given design speed, the design superelevation rate should be as shown in applicable table III-7, -8, -9, or -10.

Transition (Spiral) Curves or Superelevation Runoffs *(pages 172-183).* A highway with built-in safety should include transition curves (spiral or equivalent) between circular curves of substantially different radii, and between tangents and circular curves with a degree of curvature greater than that indicated by the solid lines in tables III-7, -8, -9, and -10 for various design speeds.

Minimum lengths of spiral or superelevation runoff for various curves are also given in these tables. For convenience in design and location, lengths shown therein should be rounded in multiples of 25 or 50 feet.

Superelevation runoff for 2-lane highways should be effected in a length as determined by the following relative slope, but in a length not less than the minimum given in applicable table III-7, -8, -9, or -10:

Design Speed, mph	Maximum Relative Slope Between Profiles of Edge of 2-Lane Pavement and Centerline
30	0.66%
40	0.58%
50	0.50%
60	0.45%
65	0.41%
70	0.40%
75	0.38%
80	0.36%

Superelevation runoff lengths for pavements wider than two lanes should be as follows:

3-lane pavements:	1.2 times length for 2-lane highway
4-lane undivided pavements:	1.5 times length for 2-lane highway
6-lane undivided pavements:	2.0 times length for 2-lane highway

These lengths apply to divided highways with narrow medians. With wide medians, 2- and 3-lane lengths are applicable for each pavement.

On curves with spirals the superelevation runoff is effected on the spiral. On curves without spirals, from 60 to 80 percent of the length of runoff should be located on the tangent. See figure III-11.

Smoothly rounded edge of pavement profiles are the desired end in design of superelevation runoff, rather than exactness in fitting the above guide values.

Pavement Widening on Curves *(pages 183-187).* The minimum practical increment of widening, selected for design, is 2 feet. On highways with 12-foot lanes, pavement widening is not necessary on curves of 10 degrees or flatter.

Design values for widening of 2-lane pavements, one-way or two-way, on open highway curves are given in table III-12. On pavements wider than two lanes, widening should be increased in proportion to the number of lanes.

Pavement widening normally is applied on the inside edge of pavement where spirals are not used. Preferably, it should be attained over the superelevation runoff length and the inside edge should be a smooth curve. See figure III-12.

Sight Distance on Horizontal Curves *(pages 188-190).* All horizontal curves should be checked for adequate sight distance and, where necessary, adjustments made in alinement or cross section to provide not less than the minimum stopping sight distance. See figure III-13.

Design for passing sight distance, for the most part, is confined to tangent and very flat alinement conditions.

General Controls for Horizontal Alinement *(pages 190-192)*

1. Alinement should be as directional as possible, consistent with topography. A flowing line that conforms generally to the natural contours is preferable to one with long tangents that slashes through the terrain; exception may be made on 2-lane highways where passing sight distance should be provided over as large a percentage of the highway length as feasible.

2. Use generally should be made of flat curves, and curves of maximum degree avoided except for critical conditions.

3. Alinement should be consistent. Sharp curves at ends of long tangents and sudden changes from easy to sharply curving alinement should be avoided.

4. Curves should be sufficiently long to avoid the appearance of a kink. Curves should be at least 500 feet long for a central angle of 5 degrees, and should be increased 100 feet for each 1-degree decrease in the central angle.

5. Tangents or flat curvature should be used on high, long fills.

6. Compound curves with large differences in curvature introduce problems similar to those that arise at a tangent approach to a circular curve. In compounding, the radius of the flatter circular arc should be not more than 50 percent greater than the radius of the sharper circular arc. Where this is not feasible, an intermediate curve or spiral should be used to provide the necessary transition.

7. Abrupt reversal in alinement should be avoided by the use of sufficient length of tangent or spirals between the two curves.

8. Broken-back arrangement of curves (short tangent between two curves in the same direction) should be avoided.

9. Horizontal alinement should be coordinated with the profile.

Vertical Alinement—Profiles

Relation of Maximum Grades to Design Speed *(pages 194-195).* For main highways the following guide values apply:

RELATION OF MAXIMUM GRADES TO DESIGN SPEED

Main Highways

Ref., table III-13

	Design speed, mph							
Type of Topography	30	40	50	60	65	70	75	80
Flat	6	5	4	3	3	3	3	3
Rolling	7	6	5	4	4	4	4	4
Mountainous	9	8	7	6	6	5	—	—

Critical Lengths of Grades *(pages 195-203).* Where feasible, upgrades should not be of a length which causes loaded trucks to reduce speed unduly. The critical grade length is determined for a selected truck as that which will cause a 15-mph reduction in speed below the average running speed on the approach to the upgrade. On this basis, critical lengths of upgrades when approached by level or nearly level sections of road, are as follows:

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Ref., figure III-17

Upgrade, percent	3	4	5	6	7	8
Critical length of upgrade, feet	1700	1100	800	600	500	500

Where critical lengths of upgrade are substantially exceeded, consideration should be given to providing climbing lanes, particularly where truck volume is high; see chapter V.

Vertical Curves *(pages 203-209).* The length of a vertical curve to satisfy the requirements of minimum stopping sight distance, comfort, and appearance, should not be shorter than L = KA, where L is the length of vertical curve in feet, A is the algebraic difference of grades in percent, and the values of K are as follows:

Ref., tables III-14 and III-15

Design speed, mph	30	40	50	60	65	70	75	80
Minimum K value for:								
Crest vertical curves	28	55	85	160	215	255	325	400
Sag vertical curves	35	55	75	105	130	145	160	185

L in feet should be not less than 3 times the design speed in mph. See figures III-19 and -20.

Much longer and sometimes impracticable lengths of crest vertical curves are required to provide passing sight distance. Ordinarily, passing sight distance can be provided only on sections of road without crest vertical curves, or those having very small algebraic differences in grades.

General Controls for Vertical Alinement *(pages 209-211)*

1. A smooth grade line with gradual changes, consistent with the type or class of highway and the character of terrain, is preferred to a line with numerous breaks and short lengths of grade.

2. The "roller-coaster" or the "hidden-dip" type of profile should be avoided by gradual grades made possible by heavier cuts and fills or by introducing some horizontal curvature on relatively straight sections.

3. Undulating grade lines, involving substantial lengths of momentum grades, should be appraised for their effect upon traffic operation since they may result in undesirably high downgrade speeds of trucks.

4. A broken-back grade line should be avoided.

5. On long grades it is preferable to lighten the grades near the top of the ascent, particularly on low design speed highways.

6. Where at-grade intersections occur on highway sections with moderate to steep grades, it is desirable to reduce the gradient through the intersection.

7. Climbing lanes should be considered where the critical length of grade is exceeded and the DHV exceeds the design capacity on the grade by 20 percent in the case of 2-lane roads or by 30 percent in the case of multi-lane roads.

Combination of Horizontal and Vertical Alinement

(pages 212-214)

Horizontal and vertical alinement should complement each other. Both traffic operation and overall appearance of the facility should be considered in design.

Vertical curvature superimposed upon horizontal curvature or vice versa generally results in a pleasing facility. The following exceptions to these alinement combinations should be noted:

a. Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve.

b. Only flat horizontal curvature should be introduced at or near the low point of a pronounced sag vertical curve.

c. On 2-lane highways the need for safe passing sections at frequent intervals and for an appreciable percentage of the highway length often supersedes the general desirability for coordination of horizontal and vertical alinement.

Horizontal curvature and profile should be made as flat as feasible at highway intersections.

On divided highways, variation in the width of median and the use of separate profiles and horizontal alinements should be considered to derive design and operational advantages of one-way roadways.

NOTE: The omitted sections deal with geometric design elements that are beyond the scope of this compendium.

CROSS SECTION ELEMENTS—Chapter IV

Pavement

Surface Type *(page 222).* Types of surfaces broadly are referred to as high, intermediate, and low in considering effect on geometric design. A low design speed should not be assumed solely because of an initial low type surface.

Normal Cross Slope *(pages 223-225).* Cross slope is related to type of surface. The pavement or surface crown section may be formed by plane surfaces, curved surfaces, or a combination of the two. On other than superelevated sections, surface cross slopes normally should conform to the following:

Ref., table IV-1

Surface Type	Range in rate of cross slope	
	Inch per foot	Foot per foot
High	1/8 - 1/4	.01 - .02
Intermediate	3/16 - 3/8	.015- .03
Low	1/4 - 1/2	.02 - .04

Rates of cross slope should be as low as practicable for vehicle operation, but consistent with the accuracy of construction procedures and structural stability to insure proper drainage. Where two or more lanes are inclined in the same direction on multilane pavements, each successive lane outward from the high point preferably should be increased in rate of slope by about 1/16-inch per foot. Cross slopes greater than 1/4-inch per foot should be avoided on high type pavements. Cross slopes on curbed pavements generally should be not less than 3/16-inch and 1/4-inch per foot on high and intermediate type surfaces, respectively.

Lane Widths *(pages 225-227)*

In the interest of safety, efficiency, and ease of operation, lane widths of 11 to 13 feet are desirable, the larger values providing the additional freedom and ease of operation consistent with high volume facilities. The use of 12-foot lanes is predominant on most high type highways. The use of 10-foot and narrower lane widths should be discouraged and where they cannot be avoided they should be reserved for low volume roads.

Curbs

Types of Curbs *(pages 228-231).* Curbs are classed generally as barrier and mountable, figures IV-1 and -2. Barrier curbs may be used on bridges and at piers, walls, pedestrian refuge islands, and in some instances, on narrow medians, although there is an increasing tendency to eliminate barrier curbs along walls or faces of bridge parapets. Mountable curbs are used primarily on medians, at the inside edge of shoulders, although this is not recommended, and to outline channelizing islands in intersection areas. High visibility curbs are especially desirable along narrow medians, channelizing islands, at hazardous locations, and in areas subject to prolonged rains and fogs.

Curb Placement *(pages 231-234).* Mountable curbs may be located adjacent to through traffic lanes. Barrier curbs introduced on bridges or elsewhere should be offset 2 feet and preferably 3 feet from the edge of traffic lanes on main highways. Where barrier curbs are continuous along a highway, an offset of 1 to 2 feet is considered suitable.

Shoulders, Sidewalks and Guardrails

The term "shoulder" is variously used with modifying adjectives to describe certain functional or physical characteristics. The "graded" width of shoulder is measured from the edge of through traffic lane to the intersection of the shoulder slope. The "surfaced" width of shoulder is that part constructed to provide a better all-weather load support than afforded by the native soils. The "usable" width of shoulder is the actual width that can be used when a driver makes an emergency or parking stop. Well designed and properly maintained shoulders are necessary on rural highways with any appreciable volume of traffic.

Width of Shoulder *(pages 235-237).* In general a width of 10 feet is a desirable minimum. Heavily traveled and high speed highways should have a graded shoulder at least 10 feet and preferably 12 feet wide. In mountainous terrain where full shoulders are unduly costly a width of 6 feet is often used. A usable shoulder width not less than 4 feet, and preferably 6 or 8 feet, should be considered on low type highways. An additional width of about 2 feet outside the usable shoulder should be provided where guardrails, guide posts, walls or other vertical elements are used. Shoulder strips, surfaced sections, 2 to 4 feet wide, at the edge of through traffic lanes, are not intended to serve as shoulders but rather as a maintenance and a safety element of a wider shoulder.

Shoulder Cross Section *(pages 237-238).* Shoulders should be pitched sufficiently to remove surface water from the roadway, but not to the extent to make vehicular use hazardous. Shoulder cross slopes should be as follows:

Ref., table IV-3

Type of surface	Shoulder cross slope	
	Inch per foot	Foot per foot
No pavement edge curbs:		
Bituminous	3/8 - 5/8	.03 - .05
Gravel or crushed stone	1/2 - 3/4	.04 - .06
Turf	1	.08
With shoulder curbs at pavement edge:		
Bituminous	1/4	.02
Gravel or crushed stone	1/4 - 1/2	.02 - .04
Turf	3/8 - 1/2	.03 - .04

The break at the high edge of a superelevated section should be limited to an algebraic difference in the pavement and shoulder cross slopes of 0.07 foot per foot.

Shoulder Stability and Contrast *(pages 238-240).* If shoulders are to be utilized, they must be sufficiently stable to support occasional vehicle loads in all kinds of weather. Desirably, shoulder surface should contrast in color and texture with that of through traffic lanes.

Intermittent Shoulders or Turnouts *(page 241).* Adequate shoulders should be continuous along the highway, but where this is not economically feasible, consideration should be given to the use of intermittent sections of wide shoulder or turnouts that can be placed at favorable locations along the highway with little additional cost.

Sidewalks *(pages 241-242).* Justification of sidewalks in rural areas depends upon the volume of pedestrian and vehicular traffic, their relative timing, and the speed of vehicular traffic. Likely locations are at points of community development, such as schools, local businesses, and industrial plants.

In general, wherever the roadside and land development conditions are such that pedestrians regularly move along a rural main or high speed highway they should be furnished a sidewalk or path area well removed from the traveled way.

Guardrails and Guide Posts *(pages 242-244).* Guardrails should be provided at points of hazard. Generally such points are fixed objects along the pavement edge, fills, on steep grades, long through fills, fills on sharp curvature, along water courses, escarpments, along deep ditches in cuts and similar locations. Guardrail may be omitted where fill slopes are 4:1 or flatter. At less hazardous but similar locations, guide posts are desirable to outline the roadway.

Drainage Channels and Side Slopes *(pages 244-248)*

Safety, appearance, and economy in maintenance call for reasonably flat side slopes, broad drainage channels, and liberal warping and rounding of the cross section. Where terrain permits, roadside drainage channels built in earth should have side slopes not steeper than 4:1, and a rounded bottom at least 4 feet wide. When the height of cut or fill does not exceed about 4 feet, a side slope of 6:1 is desirable. Channel paving may be necessary on steep and on very flat longitudinal grades. Highway side slopes in cuts and fills should be rounded and gradually flattened toward the end of the cut or fill section. Suggested earth slopes for design are as follows:

Ref., table IV-4

Height of cut or fill, feet	Earth slope, horizontal to vertical, for type of terrain		
	Flat or rolling	Moderately steep	Steep
0- 4	6:1	4 :1	4 :1
4-10	4:1	3 :1	2 :1
10-15	3:1	2½:1	1¾:1
15-20	2:1	2 :1	1½:1*
Over 20	2:1	1½:1*	1½:1*

* In clay or silty soils subject to erosion, slopes steeper than 2:1 should be avoided.

Outer Cross Section Elements in Combination

(pages 248-251)

Typical combinations of pavements, shoulders, drainage channels, and side slopes are illustrated in figures IV-3 and -4. Appropriate rounding generally should be applied at the intersection of all planes.

NOTE: The omitted section deals with medians and frontage roads that are not relevant to low-volume roads.

HIGHWAY TYPES—Chapter V

Single-lane and 3-lane highways are considered inappropriate as parts of improved rural highway systems.

2-Lane Highways *(pages 259-276)*

Two-lane highways constitute about 90 percent of total rural highway mileage, varying from low cost, loose surfaced roads to high speed main arteries. A design guide for minimum widths of surfacing is suggested in accordance with traffic volume, design speed, and traffic composition, as follows:

MINIMUM WIDTHS OF SURFACING FOR 2-LANE HIGHWAYS

Ref., table V-1

Design speed, mph	Minimum widths of surfacing, in feet, for design volumes of: *				
	Current ADT 50-250	Current ADT 250-400	Current ADT 400-750 DHV 100-200	DHV 200-400	DHV 400 and over
30	20	20	20	22	24
40	20	20	22	22	24
50	20	20	22	24	24
60	20	22	22	24	24
65	20	22	24	24	24
70	20	22	24	24	24
75	24	24	24	24	24
80	24	24	24	24	24

* For design speeds of 30, 40 and 50 mph, surfacing widths that are two feet narrower may be used on minor roads with few trucks.

Usable shoulders 10 feet wide are desirable on all highways, but narrower widths are often considered on low volume highways in rugged terrain or where economic considerations otherwise govern. Suggested minimum widths of shoulder follow:

WIDTHS OF SHOULDERS FOR 2-LANE RURAL HIGHWAYS

Ref., table V-2

Design volume		Usable shoulder width, feet	
Current ADT	DHV	Minimum	Desirable
50-250	—	4	6
250-400	—	4	8
400-750	100-200	6	10
—	200-400	8	10
—	400 and over	10	12

Desirably, shoulders should be usable at all times regardless of weather conditions. On high volume highways they should be paved. Shoulders should be constructed to properly serve the type and volume of traffic on the particular highway system.

Design guides for cross section elements and right-of-way widths for 2-lane highways, as shown in figure V-1, are as follows:

Cross section element	Dimension of element, in feet, for 2-lane highways of the following general type:		
	Low	Intermediate	High
Surfacing	18 - 20	20 - 24	24
Usable shoulder	4 - 8	8	10
Roadway	26 - 36	36 - 40	44
Border	18 - 25*	20 - 30*	25 - 35*
Right-of-way	66 - 80*	80 - 100*	100 - 120*

* preferably wider.

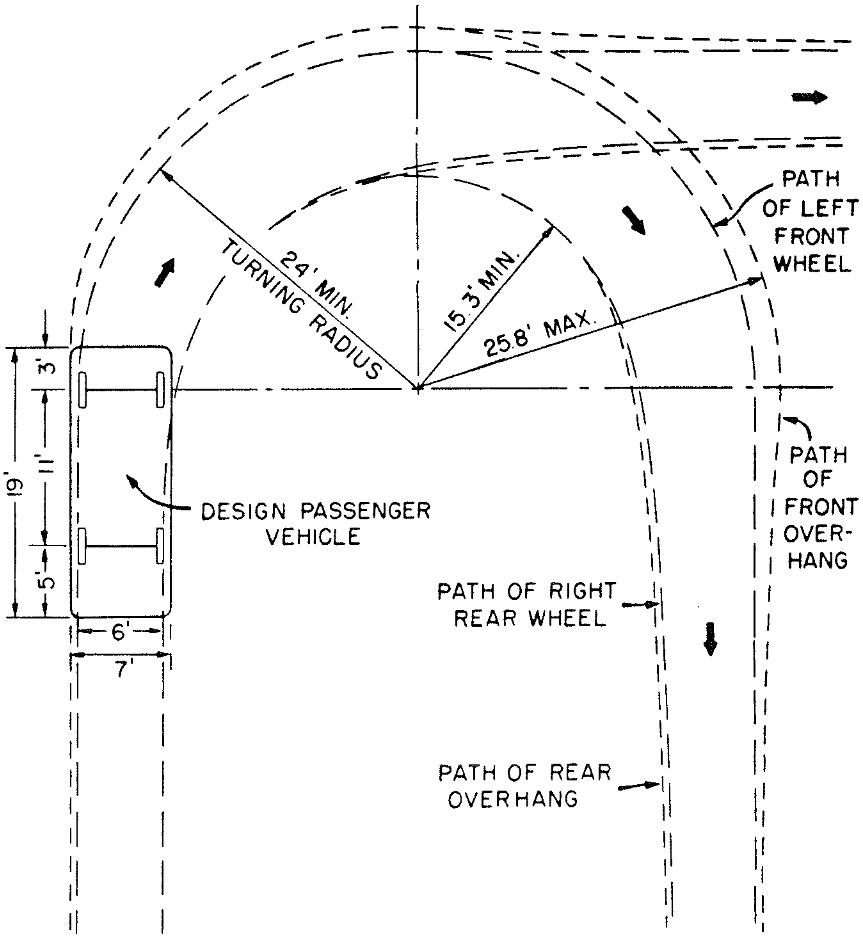
Horizontal and vertical alinement should be designed to provide at least stopping sight distance at all points, and sight distance safe for passing on as much of the highway as feasible. Consideration should be given to the construction of 4-lane sections where sight distances are limited.

Where the critical length of grade (see *pages 195-203* and figure III-17) is exceeded and design capacity is reduced because of climbing trucks, consideration should be given to providing a climbing lane where DHV exceeds the reduced capacity by 20 percent or more.

The climbing lane should not be less than 10 and preferably 12 feet wide. A shoulder 4 feet wide is considered adequate. It should be signed and marked. The climbing lane should begin near the foot of the grade at a point that would be determined by the speed of the trucks at the approach to the grade. Where practicable the climbing lane should end at a point beyond the crest where the truck can attain a speed of 30 mph.

Where it is anticipated that the DHV within a period of about 10 to 20 years will exceed design capacity of a 2-lane highway, the initial improvement should be patterned for ultimate development of a 4-lane divided highway. The initial 2-lane width should form one of the ultimate one-way surfaces. Details depend on the ultimate median width, see figure V-3.

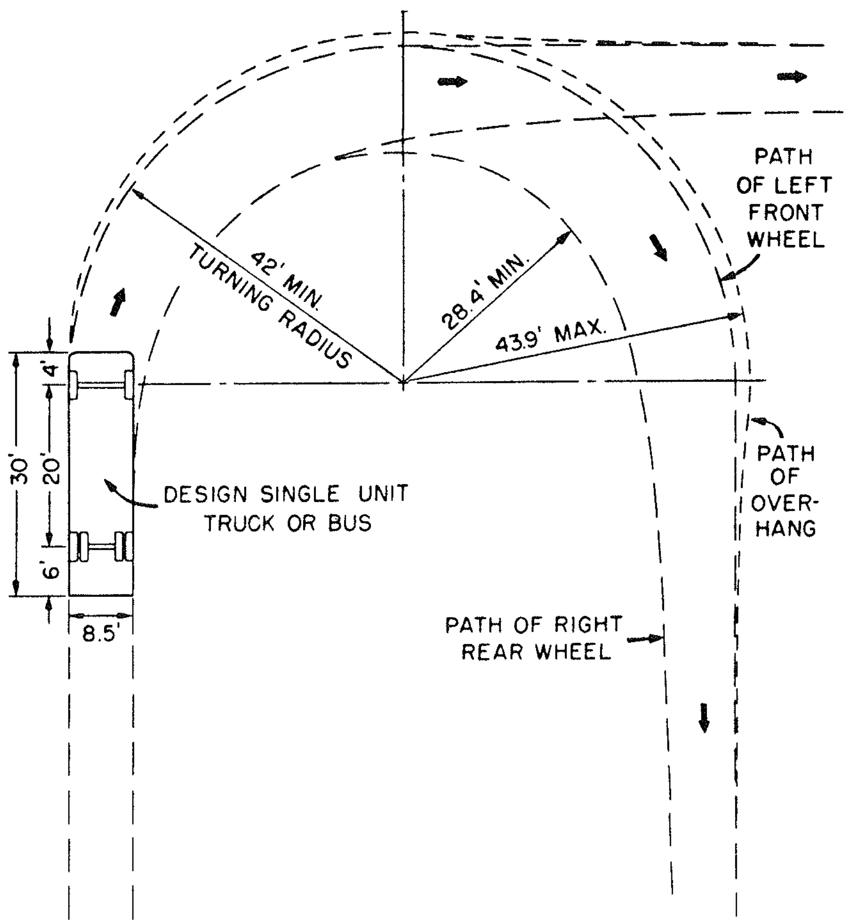
NOTE: The omitted section deals with 4-lane undivided highways.



P DESIGN VEHICLE

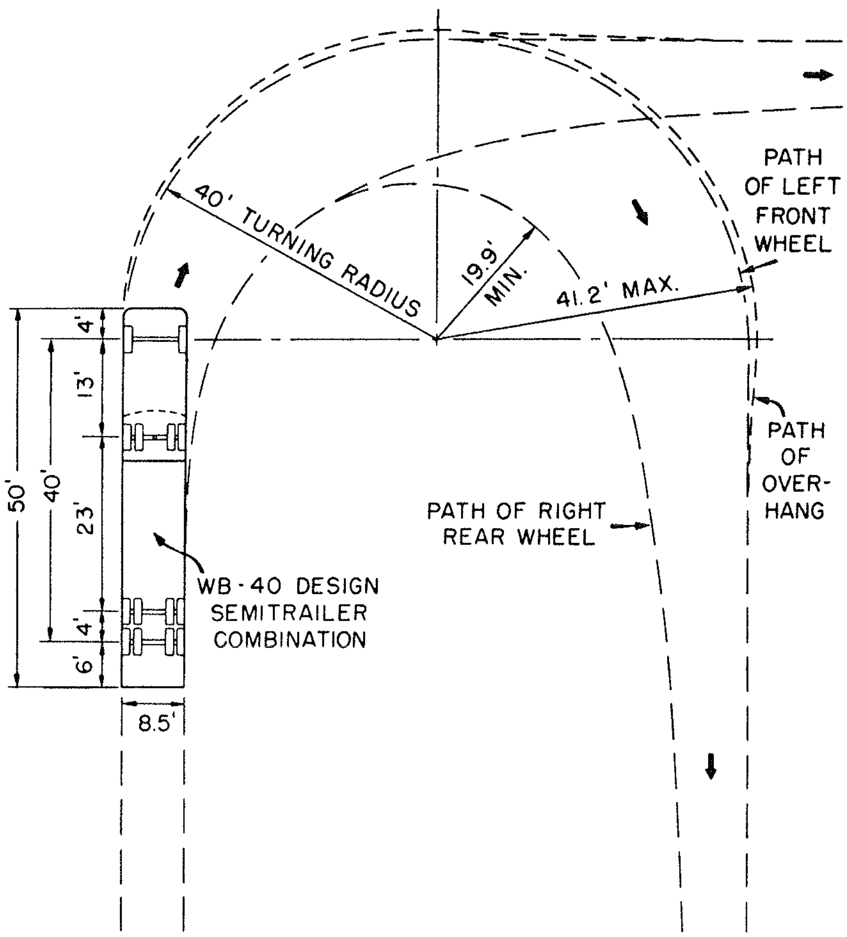
FIGURE II - 3

NOTE: This figure was referenced on page 4 of the text.



SU DESIGN VEHICLE
FIGURE II - 9

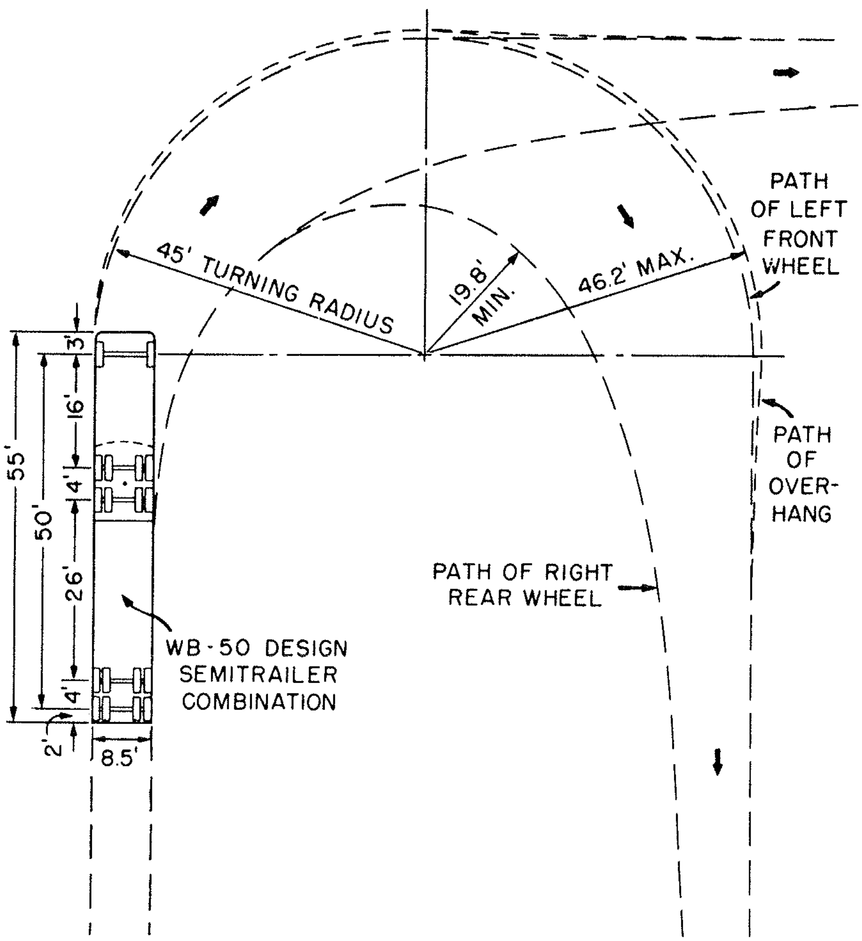
NOTE: This figure was referenced on page 4 of the text.



WB-40 DESIGN VEHICLE

FIGURE II-11

NOTE: This figure was referenced on page 4 of the text.



WB-50 DESIGN VEHICLE

FIGURE II - 12

NOTE: This figure was referenced on page 4 of the text.

NOTE: The table below was referenced on page 9 of the text.

TABLE III-1

MINIMUM STOPPING SIGHT DISTANCE

Design speed	Assumed speed for condition	Perception and brake reaction		Coefficient of friction	Braking distance on level	Stopping sight distance	
		Time	Distance			Computed	Rounded for design
mph	mph	sec.	feet	f	feet	feet	feet
		Design Criteria—WET PAVEMENTS					
30	28	2.5	103	.36	73	176	200
40	36	2.5	132	.33	131	263	275
50	44	2.5	161	.31	208	369	350
60	52	2.5	191	.30	300	491	475
65	55	2.5	202	.30	336	538	550
70	58	2.5	213	.29	387	600	600
75*	61	2.5	224	.28	443	667	675
80*	64	2.5	235	.27	506	741	750
		Comparative Values—DRY PAVEMENTS					
30	30	2.5	110	.62	48	158	
40	40	2.5	147	.60	89	236	
50	50	2.5	183	.58	144	327	
60	60	2.5	220	.56	214	434	
65	65	2.5	238	.56	251	489	
70	70	2.5	257	.55	297	554	
75	75	2.5	275	.54	347	622	
80	80	2.5	293	.53	403	696	

* Design speeds of 75 and 80 mph are applicable only to highways with full control of access or where such control is planned in the future.

NOTE: The following text is included since it modifies Table III-1 as shown on the preceding compendium page.

Effect of Grades on Stopping

When a highway is on a grade the standard formula for braking distance is:

$$d = \frac{V^2}{30\,(f \pm G)}$$

in which G is the percent of grade divided by 100, and the other terms are as previously stated. The safe stopping distances on upgrades are shorter and those on downgrades are longer. The extent of grade correction is indicated in table III-2.

TABLE III-2

EFFECT OF GRADE ON STOPPING SIGHT DISTANCE: WET CONDITIONS

Design speed, mph	Assumed speed for condition, mph	Correction in stopping distance—feet					
		Decrease for upgrades			Increase for downgrades		
		3%	6%	9%	3%	6%	9%
30	28	—	10	20	10	20	30
40	36	10	20	30	10	30	50
50	44	20	30	—	20	50	—
60	52	30	50	—	30	80	—
65	55	30	60	—	40	90	—
70	58	40	70	—	50	100	—
75*	61	50	80	—	60	120	—
80*	64	60	90	—	70	150	—

* Design speeds of 75 and 80 mph are applicable only to highways with full control of access or where such control is planned in the future.

These corrections are computed for wet conditions, the assumed design criterion condition. Use of the dry pavement friction factors (table III-1, lower) and full design speed results in calculated corrections about one-third the above; the smaller corrections applied to the lower computed stopping distances for dry pavements result in shorter lengths than the corrected stopping distances for wet pavements, consequently this distinction can be ignored.

On nearly all roads the grade is traversed by traffic in both directions but the sight distance generally is different in each direction, particularly on straight roads in rolling terrain. As a general rule the sight distance available on downgrades is larger than on upgrades, more or less automatically providing the necessary corrections for grade. This may explain why some design offices utilize stopping sight distances as determined for flat conditions without regard to grade corrections. Exceptions are one-way roads as on divided highways with independently designed profiles for the two roadways, for which the separate grade corrections are in order and the refinement in design is in keeping with the overall standards used.

Variation for Trucks

The minimum stopping sight distances as derived directly reflect passenger car operation and they might be questioned for use in design for truck operation. Trucks as a whole, especially the larger and heavier units, require a longer stopping distance from a given speed than do passenger vehicles. However, two factors tend to balance the additional braking length for trucks for given design speeds with that for passenger cars. First, on a vertical sight obstruction the truck operator is able to see substantially farther due to his higher position in the vehicle. Second, in nearly all cases trucks travel slower than passenger vehicles, either through regulation or by operator choice. Separate stopping sight distances for trucks and passenger cars, therefore, are not used in highway design standards.

There is one situation that should be treated with caution and every effort made to provide stopping sight distance greater than the minimum design value. This is the case of horizontal sight restriction occurring on downgrades, particularly at the ends of long downgrades. The greater height of eye of the truck operator is of little value, even when the horizontal sight obstruction is a cut slope, and on long downgrades truck speeds may tend to approach those of passenger cars. Even though the average truck operator is better trained than the average passenger car operator and is quicker to recognize hazards, it is desirable under such conditions to supply greater stopping sight distance than the design values, table III-1.

NOTE: The following text was referenced on page 9 of the text.

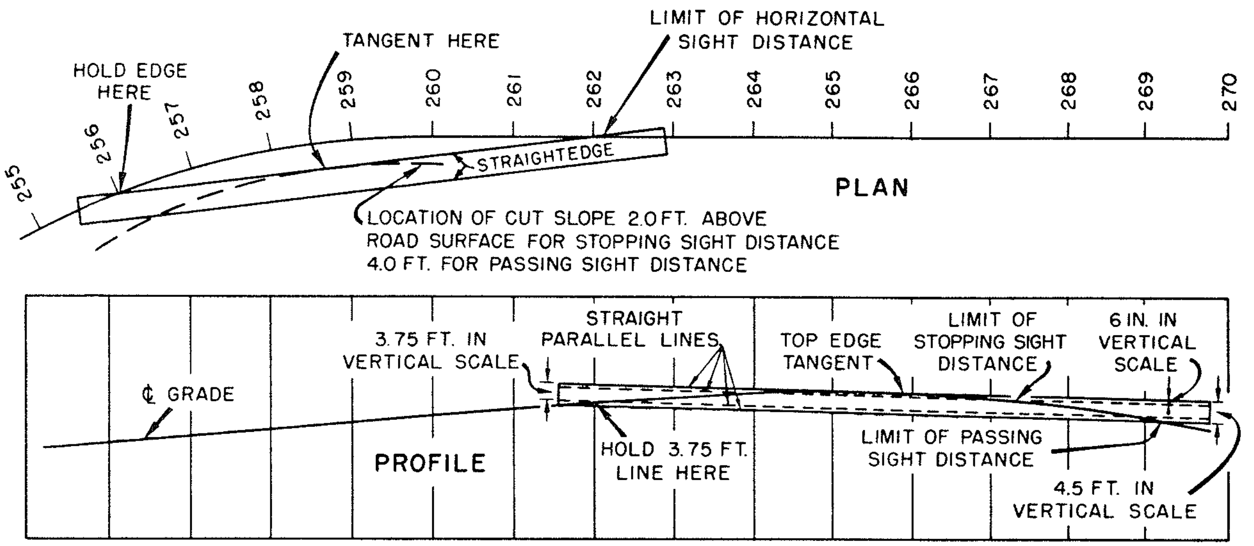
Measuring and Recording Sight Distance

The design of horizontal alinement and profiles using sight distance and other criteria in combination is given later in this chapter, particularly the detail design of horizontal and vertical curves. Sight distance, however, should be considered in the preliminary stages of design when the alinement, horizontal and vertical, is still subject to adjustment. By determining graphically the sight distances on the plans and recording them at frequent intervals, the designer can appraise the overall layout and effect a more balanced design by minor adjustments in the plan or profile. Methods for scaling sight distances are demonstrated in figure III-3. The figure also shows a typical sight distance record which would be shown on the final plans.

Since the view of the highway ahead may change rapidly in a short distance, it is desirable to measure and record sight distance for both directions of travel at each station. Both horizontal and vertical sight distances should be measured and the shorter lengths recorded. In the case of 2-lane highways, passing sight distance in addition to stopping sight distance should be measured and recorded.

Sight distance charts as in figures III-19 and -20 are used directly in design to establish minimum lengths of vertical curves. Charts similar to figure III-13 are used to determine the degree of horizontal curve or lateral offset therefrom. But once the horizontal and vertical alinements are tentatively established, the practical means of examining sight distances along the proposed highway is by direct scaling on the plans.

Horizontal sight distance on the inside of a curve is limited by obstructions such as buildings, hedges, wooded areas, high ground, or other topographic features. These generally are plotted on the plans. Horizontal sight is measured with a straightedge, as indicated at the upper left in figure III-3. The cut slope obstruction is shown on the work sheets by a line representing the proposed excavation slope at a point about 2.0 feet (approximate average of 3.75 and 0.5 foot) above the road surface for stopping sight distance and about 4.0 feet for passing sight distance. The position of this line with respect to the centerline may be scaled from the plotted high-



Station	255	256	257	258	259	260	261	262	263	264	265	266	267	268	269	270
STOPPING S.D. 3.75 FT. to 6 IN.		3000+ 600	3000+ 900	1600 800	700 700	550 600	550 550	600 500	650 500	750 600	850 3000+	950	1000	500 3000+		
PASSING S.D. 3.75 FT. to 4.5 FT.		3000+ 700	3000+ 1100	1600 1000	900 900	600 850	600 800	650 700	700 700	800 1600	900 3000+	1000	1050	900 3000+		

TYPICAL SIGHT DISTANCE RECORD

SCALING AND RECORDING SIGHT DISTANCES ON PLANS

FIGURE III - 3

way cross sections. Preferably the stopping sight distance should be measured between points on the one traffic lane, and passing sight distance from the middle of one lane to the middle of the other lane. Such refinement on 2-lane highways generally is not necessary and measurement to the centerline or pavement edge is suitable. Where there are changes of grade coincident with horizontal curves that have sight limiting cut slopes on the inside, the line of sight intercepts the slope at a level lower or higher than the assumed average height. In measuring sight distance the error in use of the assumed 2.0- or 4.0-foot height usually can be ignored.

Vertical sight distance may be scaled from a plotted profile by the method illustrated at the right center of figure III-3. A transparent strip with parallel edges 4.5 feet apart and with scratched lines 6 inches and 3.75 feet from the upper edge, in accordance with the vertical scale, is a useful tool. The 3.75-foot line is placed on the station from which the vertical sight distance is desired and the strip is pivoted about this point until the upper edge is tangent to the profile. The distance between the initial station and the station on the profile intersected by the 6-inch line is the stopping sight distance. The distance between the initial station and the station on the profile intersected by the lower edge of the strip is the passing sight distance.

A simple sight distance record is shown in the lower part of figure III-3. Sight distances in both directions are indicated by arrows and figures at each station on the plan and profile sheet of the proposed highway. To avoid the extra work of measuring unusually long sight distances as may be found on occasions, a selected maximum value may be recorded. In the example shown, all sight distances of more than 3000 feet are recorded as 3000 +, and where this occurs for several consecutive stations the intermediate values are omitted. Sight distances less than 1000 feet may be scaled to the nearest 50 feet and those greater than 1000 feet to the nearest 100 feet. The available sight distances along a proposed highway also may be shown by other methods. Several States use a sight distance graph, plotted in conjunction with the plan and profile of the highway, as a means of demonstrating sight distances.

Sight distance records for 2-lane highways may be used to advantage to tentatively determine the marking of no-passing zones in accordance with criteria given in the Manual on Uniform Traffic Control Devices. Marking of such zones is an operation rather than a design problem. No-passing zones thus established serve as a guide for markings when the highway is completed; the zones so determined should be checked and adjusted by field measurements before actual markings are placed.

Sight distance records also are useful on 2-lane highways for determining the percentage of length of highway on which sight distance is restricted to less than the passing minimum, which is important in evaluating capacity. With recorded sight distances as in the lower part of figure III-3, it is a simple process to determine the percentage of length with a given sight distance or greater.

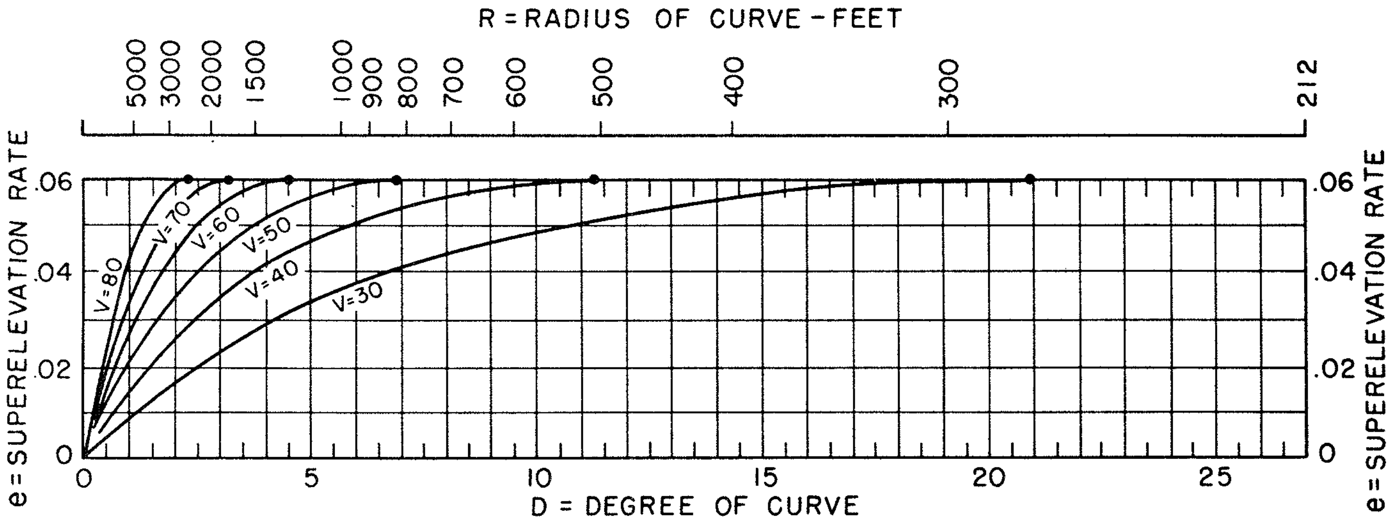
The sight distance record placed just below the profile or just above the profile on standard plan and profile sheets is useful information which takes up little space on the plans. It expedites design check or review, as usually necessary in two or more offices before plans are approved.

TABLE III-5

MAXIMUM DEGREE OF CURVE AND MINIMUM RADIUS DETERMINED FOR LIMITING VALUES OF e AND f

Design speed	Maximum e	Maximum f	Total (e+f)	Minimum radius	Max. degree of curve	Max. degree of curve, rounded
30	.06	.16	.22	273	21.0	21.0
40	.06	.15	.21	508	11.3	11.5
50	.06	.14	.20	833	6.9	7.0
60	.06	.13	.19	1263	4.5	4.5
65	.06	.13	.19	1483	3.9	4.0
70	.06	.12	.18	1815	3.2	3.0
75	.06	.11	.17	2206	2.6	2.5
80	.06	.11	.17	2510	2.3	2.5
30	.08	.16	.24	250	22.9	23.0
40	.08	.15	.23	464	12.4	12.5
50	.08	.14	.22	758	7.6	7.5
60	.08	.13	.21	1143	5.0	5.0
65	.08	.13	.21	1341	4.3	4.5
70	.08	.12	.20	1633	3.5	3.5
75	.08	.11	.19	1974	2.9	3.0
80	.08	.11	.19	2246	2.5	2.5
30	.10	.16	.26	231	24.8	25.0
40	.10	.15	.25	427	13.4	13.5
50	.10	.14	.24	694	8.3	8.5
60	.10	.13	.23	1043	5.5	5.5
65	.10	.13	.23	1225	4.7	4.5
70	.10	.12	.22	1485	3.9	4.0
75	.10	.11	.21	1786	3.2	3.0
80	.10	.11	.21	2032	2.8	3.0
30	.12	.16	.28	214	26.7	26.5
40	.12	.15	.27	395	14.5	14.5
50	.12	.14	.26	641	8.9	9.0
60	.12	.13	.25	960	6.0	6.0
65	.12	.13	.25	1127	5.1	5.0
70	.12	.12	.24	1361	4.2	4.0
75	.12	.11	.23	1630	3.5	3.5
80	.12	.11	.23	1855	3.1	3.0

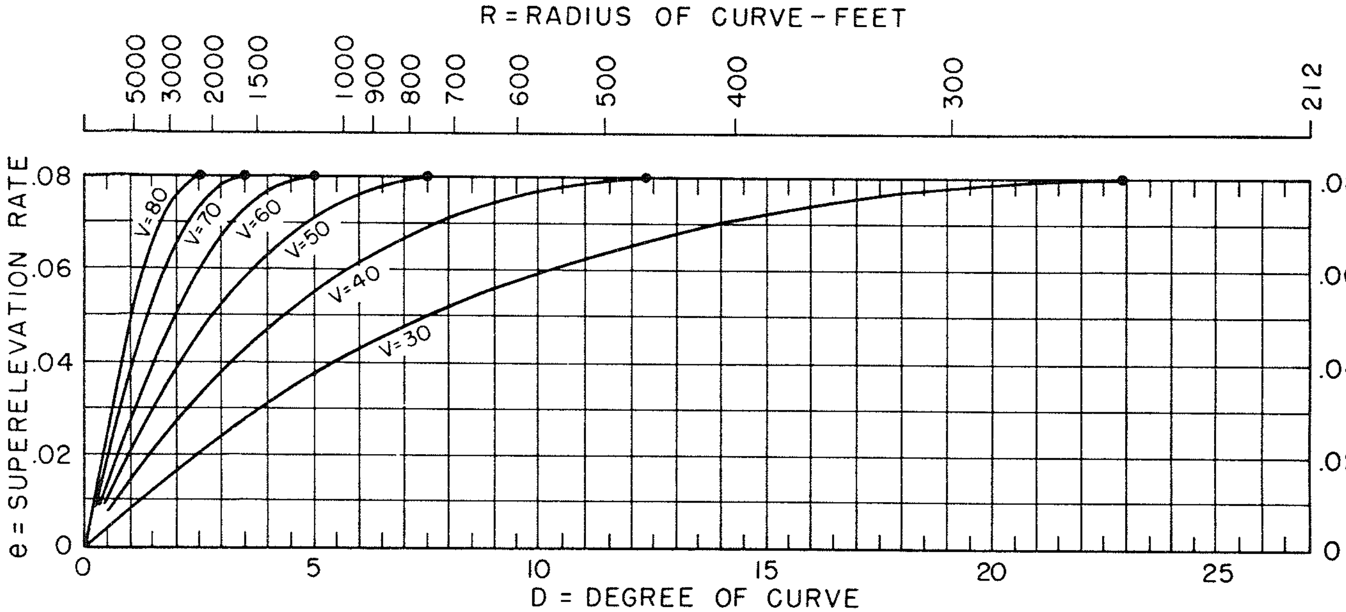
NOTE: Table III-5 above was referenced on page 11 of the text. The four figures and three tables on the following pages were also referenced on page 11 of the text.



DESIGN SUPERELEVATION RATES

FIGURE III - 7

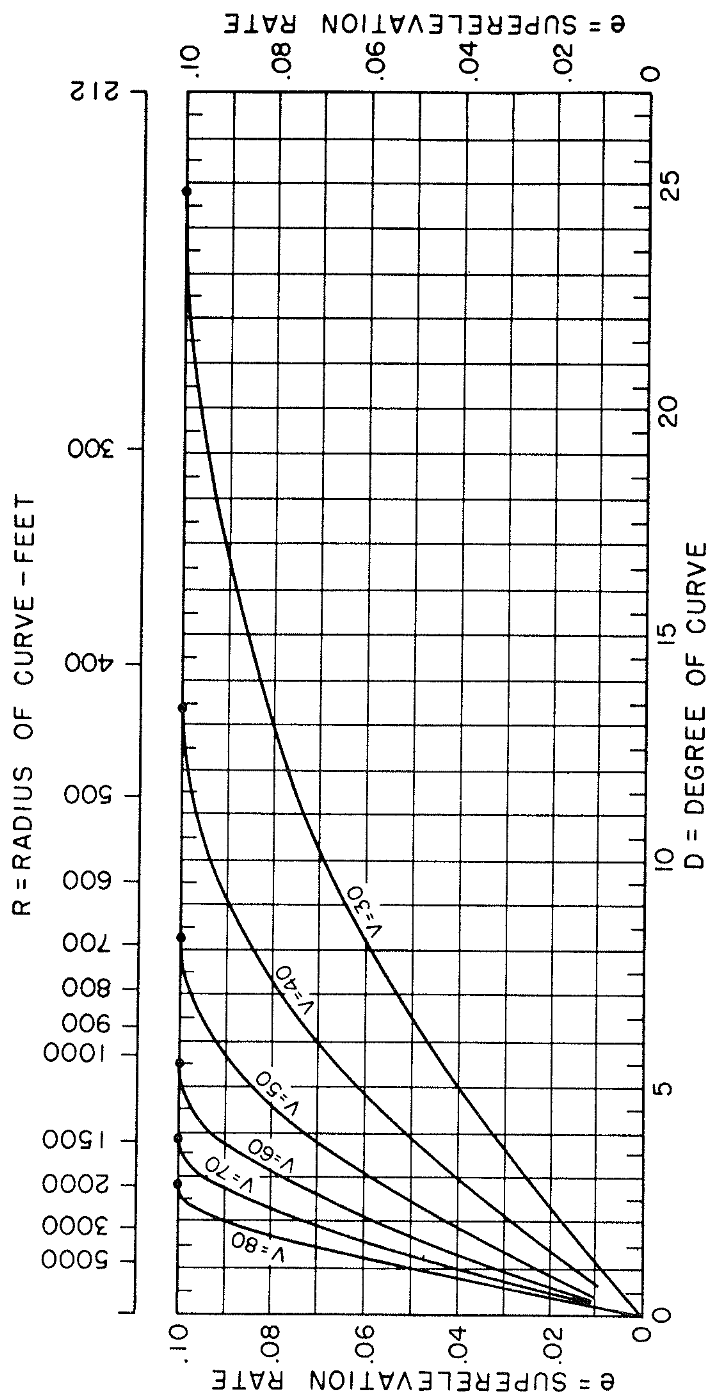
$e_{max.} = 0.06$



DESIGN SUPERELEVATION RATES

FIGURE III - 8

$e_{max.} = 0.08$

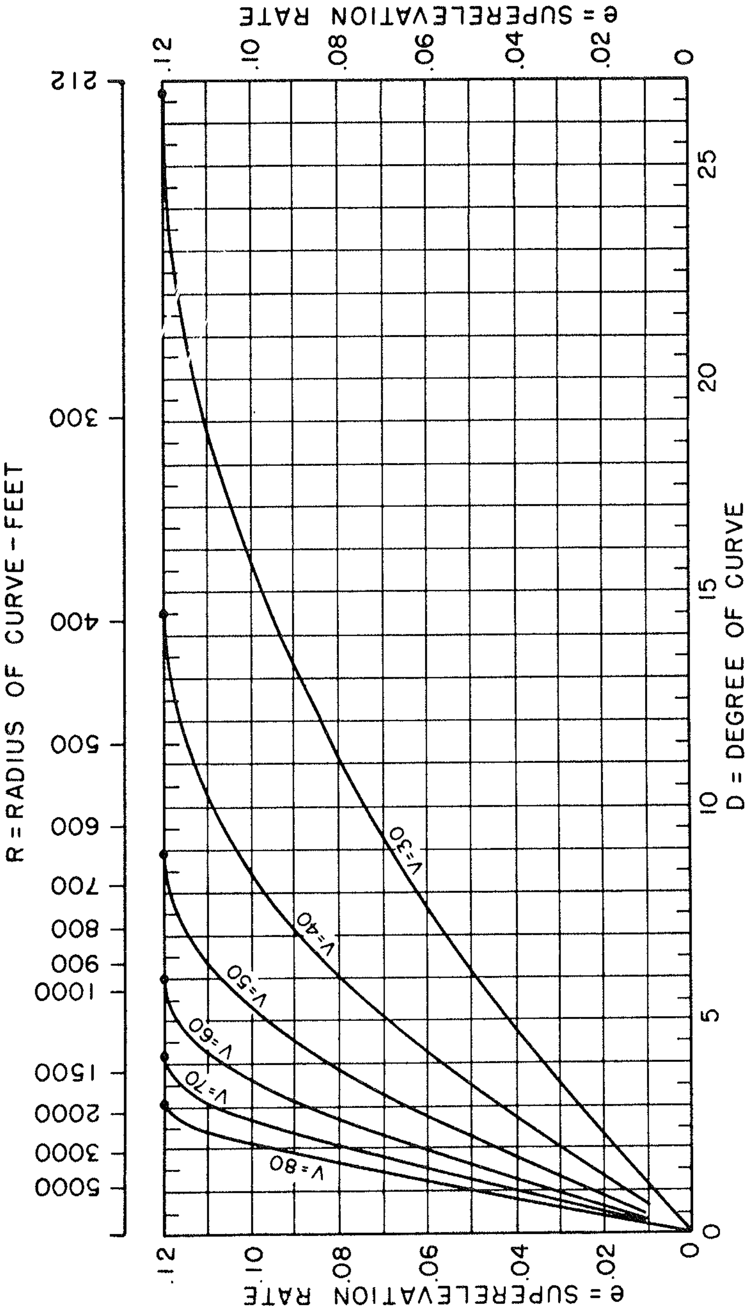


$e_{max} = 0.10$

DESIGN SUPERELEVATION RATES

FIGURE III - 9

100



$e_{max.} = 0.12$

DESIGN SUPERELEVATION RATES

FIGURE III - 10

TABLE III-7

Values for Design Elements Related to Design Speed and Horizontal Curvature

D	R	V=30 mph			V=40 mph			V=50 mph			V=60 mph			V=65 mph			V=70 mph			V=75 mph			V=80 mph		
		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet	
			2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane
0° 15′	22918′	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	240	240
0° 30′	11459′	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200	.021	220	220	.023	240	240
0° 45′	7639′	NC	0	0	NC	0	0	RC	0	0	.021	175	175	.023	190	190	.026	200	200	.030	220	220	.033	240	240
1° 00	5730′	NC	0	0	RC	125	125	.020	150	150	.027	175	175	.029	190	190	.033	200	200	.037	220	220	.041	240	240
1° 30′	3820′	RC	100	100	.020	125	125	.028	150	150	.036	175	175	.040	190	190	.044	200	200	.050	220	240	.053	240	260
2° 00′	2865′	RC	100	100	.026	125	125	.035	150	150	.044	175	180	.048	190	210	.052	200	230	.057	220	270	.059	240	290
2° 30′	2292′	.020	100	100	.031	125	125	.040	150	150	.050	175	200	.053	190	230	.057	200	260	.060	220	290	.060	240	300
3° 00′	1910′	.023	100	100	.035	125	125	.044	150	160	.054	175	220	.057	190	250	.060	200	270	.060	220	290	.060	240	300
3° 30′	1637′	.026	100	100	.038	125	125	.048	150	170	.057	175	230	.059	190	250	.060	200	270	D max=2.5°			D max=2.5°		
4° 00′	1432′	.029	100	100	.041	125	130	.051	150	180	.059	175	240	.060	190	260	D max=3.0°								
5° 00′	1146′	.034	100	100	.046	125	140	.056	150	200	.060	175	240	.060	190	260									
6° 00′	955′	.038	100	100	.050	125	160	.059	150	210	D max=4.5°			D max=4.0°											
7° 00′	819′	.041	100	110	.054	125	170	.060	150	220															
8° 00′	716′	.043	100	120	.056	125	180	.060	150	220															
9° 00′	637′	.046	100	120	.058	125	180	D max=7.0°																	
10° 00′	573′	.048	100	130	.059	125	190																		
11° 00′	521′	.050	100	140	.060	130	190																		
12° 00′	477′	.052	100	140	.060	130	190																		
13° 00′	441′	.053	100	140	D max=11.5°																				
14° 00′	409′	.055	100	150																					
16° 00′	358′	.058	100	160																					
18° 00′	318′	.059	110	160																					
20° 00′	286′	.060	110	160																					
		.060	110	160																					
		D max=21.0°																							

emax=0.06

D—Degree of curve
R—Radius of curve
V—Assumed design speed
e—Rate of superelevation
L—Minimum length of runoff of spiral curve
NC—Normal crown section
RC—Remove adverse crown, superelevate at normal crown slope
Spirals desirable but not as essential above heavy line.
Lengths rounded in multiples of 25 or 50 feet permit simpler calculations.

TABLE III-8

VALUES FOR DESIGN ELEMENTS RELATED TO DESIGN SPEED AND HORIZONTAL CURVATURE

D	R	V=30 mph			V=40 mph			V=50 mph			V=60 mph		
		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet	
			2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane
0° 15′	22918′	NC	0	0	NC	0	0	NC	0	0	NC	0	0
0° 30′	11459′	NC	0	0	NC	0	0	NC	0	0	RC	175	175
0° 45′	7639′	NC	0	0	NC	0	0	RC	150	150	.022	175	175
1° 00′	5730′	NC	0	0	RC	125	125	.021	150	150	.029	175	175
1° 30′	3820′	RC	100	100	.021	125	125	.030	150	150	.040	175	175
2° 00′	2865′	RC	100	100	.027	125	125	.038	150	150	.051	175	210
2° 30′	2292′	.021	100	100	.033	125	125	.046	150	170	.060	175	240
3° 00′	1910′	.025	100	100	.038	125	125	.053	150	190	.067	180	270
3° 30′	1637′	.028	100	100	.043	125	140	.058	150	210	.073	200	300
4° 00′	1432′	.032	100	100	.047	125	150	.063	150	230	.077	210	310
5° 00′	1146′	.038	100	100	.055	125	170	.071	170	260	.080	220	320
6° 00′	955′	.043	100	120	.061	130	190	.077	180	280	.080	220	320
7° 00′	819′	.048	100	130	.067	140	210	.079	190	280	D max=5.0°		
8° 00′	716′	.052	100	140	.071	150	220	.080	190	290			
9° 00′	637′	.056	100	150	.075	160	240	D max=7.5°					
10° 00′	573′	.059	110	160	.077	160	240						
11° 00′	521′	.063	110	170	.079	170	250						
12° 00′	477′	.066	120	180	.080	170	250						
13° 00′	441′	.068	120	180	.080	170	250						
14° 00′	409′	.070	130	190	D max=12.5°								
16° 00′	358′	.074	130	200									
18° 00′	318′	.077	140	210									
20° 00′	286′	.079	140	210									
22° 00′	260′	.080	140	220									
		.080	140	220									
		D max=23.0°											

D	R	V=65 mph			V=70 mph			V=75 mph			V=80 mph		
		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet	
			2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane
0° 15′	22918′	NC	0	0	NC	0	0	NC	0	0	RC	240	240
0° 30′	11459′	RC	190	190	RC	200	200	.022	220	220	.024	240	240
0° 45′	7639′	.025	190	190	.029	200	200	.032	220	220	.036	240	240
1° 00′	5730′	.033	190	190	.038	200	200	.043	220	220	.047	240	240
1° 30′	3820′	.046	190	200	.053	200	240	.060	220	290	.065	240	320
2° 00′	2865′	.057	190	250	.065	200	290	.072	230	340	.076	250	380
2° 30′	2292′	.066	190	290	.073	220	330	.078	250	370	.080	260	400
3° 00′	1910′	.073	210	320	.078	230	350	.080	250	380	.080	260	400
3° 30′	1637′	.077	220	330	.080	240	360	.080	250	380	D max=2.5°		
4° 00′	1432′	.079	230	340	.080	240	360	D max=3.0°					
5° 00′	1146′	.080	230	350	D max=3.5°								
6° 00′	955′	D max=4.5°											

emax=0.08

D—Degree of curve
R—Radius of curve
V—Assumed design speed
e—Rate of superelevation
L—Minimum length of runoff of spiral curve
NC—Normal crown section
RC—Remove adverse crown, superelevate at normal crown slope
Spirals desirable but not as essential above heavy line.
Lengths rounded in multiples of 25 or 50 feet permit simpler calculations.

TABLE III-10

VALUES FOR DESIGN ELEMENTS RELATED TO DESIGN SPEED AND HORIZONTAL CURVATURE

D	R	V=30 mph			V=40 mph			V=50 mph			V=60 mph			V=65 mph			V=70 mph			V=75 mph			V=80 mph		
		e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane
0° 15′	22918′	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	240	240
0° 30′	11459′	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200	.022	220	220	.024	240	240
0° 45′	7639′	NC	0	0	NC	0	0	RC	150	150	.024	175	175	.026	190	190	.029	200	200	.033	220	220	.036	240	240
1° 00′	5730′	NC	0	0	RC	125	125	.023	150	150	.031	175	175	.035	190	190	.039	200	200	.043	220	220	.048	240	240
1° 30′	3820′	RC	100	100	.022	125	125	.034	150	150	.047	175	190	.053	190	230	.059	200	270	.065	220	310	.072	240	360
2° 00′	2865′	RC	100	100	.030	125	125	.045	150	160	.062	175	250	.070	200	300	.078	230	350	.087	280	410	.095	310	470
2° 30′	2292′	.022	100	100	.037	125	125	.055	150	200	.076	210	310	.085	240	370	.095	290	430	.105	330	500	.113	370	560
3° 00′	1910′	.026	100	100	.044	125	140	.065	160	230	.088	240	360	.097	240	420	.108	320	490	.117	370	560	.120	400	600
3° 30′	1637′	.030	100	100	.050	125	160	.074	180	270	.098	260	400	.107	310	460	.116	350	520	.120	380	570	.120	400	600
4° 00′	1432′	.034	100	100	.057	125	180	.082	200	300	.106	290	430	.114	330	490	.120	360	540	.120	380	570	D max=3.0°		
5° 00′	1146′	.042	100	110	.068	140	210	.096	230	350	.117	320	470	.120	350	520	.120	360	540	D max=3.5°					
6° 00′	955′	.049	100	130	.079	170	250	.107	260	390	.120	320	490	.120	350	520	D max=4.0°								
7° 00′	819′	.055	100	150	.088	180	280	.114	270	410	.120	320	490	D max=5.0°											
8° 00′	716′	.062	110	170	.096	200	300	.119	290	430	D max=6.0°														
9° 00′	637′	.068	120	180	.103	220	320	.120	290	430															
10° 00′	573′	.074	130	200	.108	230	340	.120	290	430															
11° 00′	521′	.079	140	210	.113	240	360	D max=9.0°																	
12° 00′	477′	.084	150	230	.116	240	370																		
13° 00′	441′	.089	160	240	.119	250	370																		
14° 00′	409′	.093	170	250	.120	250	380																		
16° 00′	358′	.101	180	270	.120	250	380																		
18° 00′	318′	.108	190	290	D max=14.5°																				
20° 00′	286′	.113	200	310																					
22° 00′	260′	.117	210	320																					
26° 00′	220′	.120	220	320																					
		.120	220	320																					
		D max=26.5°																							

$e_{max}=0.12$

D—Degree of curve
R—Radius of curve
V—Assumed design speed
e—Rate of superelevation
L—Minimum length of runoff of spiral curve
NC—Normal crown section
RC—Remove adverse crown, superelevate at normal crown slope
Spirals desirable but not as essential above heavy line.
Lengths rounded in multiples of 25 or 50 feet permit simpler calculations.

NOTE: The following text includes several concepts that were not mentioned in the abstracted section that appeared on pages 11 and 12 of the text.

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Superelevation Runoff

Length Required

Superelevation runoff is the general term denoting the length of highway needed to accomplish the change in cross slope from a normal crown section to the fully superelevated section, or vice versa. To meet the requirements of comfort and safety the superelevation runoff should be effected uniformly over a length adequate for the likely travel speeds. To be pleasing in appearance the runoff pavement edges should not be distorted as the driver views them.

Some States employ the spiral curve and use its length in which to make the desired change in cross slope. Others do not employ the spiral, but empirically designate proportional lengths of tangent and circular curve for the same purpose. In either case, as far as can be determined, the length of roadway to effect the superelevation runoff should be the same for the same rate of superelevation and degree of curvature. The spiral curve simulates the natural turning path of a vehicle. On unspiraled curves, the average vehicle tends to traverse a similar transitioned path within the limits of the pavement.

Review of current design practice indicates the appearance aspect of super-

[18] A Practical Method for Improvement of Existing Railroad Curves—W. H. Shortt; Proceedings of the Institution of Civil Engineering, 1909.

elevation runoff largely governs the length. Required spiral lengths as determined otherwise often are shorter than that determined for general appearance, so that spiral formula values give way to longer empirical runoff values. A number of States have established one or more control runoff lengths within a range of about 100 to 600 feet, but there is no universally accepted empirical basis, considering all likely pavement widths. In one empirical expression with fairly wide use the required length is indicated in terms of the slope of the outside edge of pavement relative to the centerline profile.

Current practice indicates that for appearance and comfort the length of superelevation runoff should be such that a longitudinal slope (edge compared to centerline of a 2-lane highway) of 1 in 200 is not exceeded. Considered with plane sections, this can be stated that the difference in longitudinal gradient between an outside edge of a 2-lane pavement and its centerline profile should not exceed 0.5 percent.

In another source [19] the same 1 in 200 ratio is used for a design speed of 50 mph and higher. Where the design speed is 30 and 40 mph, relative slopes of 1:150 and 1:175, respectively, are used. These slopes correspond to relative gradients of 0.66 and 0.58 percent. To reflect the importance of the higher design speed and to harmonize with the flatter curving elements, both horizontal and vertical, it appears logical to extrapolate the changing relative slope to the higher design speeds, as follows:

Design Speed, mph	*Maximum Relative Slope Between Profiles of Edge of 2-Lane Pavement & Centerline*
30	0.66%
40	0.58%
50	0.50%
60	0.45%
65	0.41%
70	0.40%
75	0.38%
80	0.36%

The maximum relative slopes between profiles of the edges of 2-lane pavements are double those shown in the table.

Length of runoff on this basis is directly proportional to the total superelevation, which is the product of the lane width and superelevation rate. Table III-11 gives values for 2-lane highways with 10- and 12-foot lanes.

Review of current design practice shows use of *minimum* runoff lengths in the range of 100 to 250 feet, regardless of width and superelevation. This is practical recognition of the general appearance and the actual smoothing to be made on the pavement edge profile. Use of the smaller values in table III-11 results in undesirably abrupt edge of pavement profiles. The minimum lengths shown in the lower part of the table are assumed for design use. These minimum values approximate the distance traveled in 2 seconds at the design speed, reflecting the speed aspects of design, and should be used instead of the shorter lengths above the horizontal bars in the tabulation.

[19] Transition Curves for Highways—J. Barnett: Bureau of Public Roads, 1940. U. S. Government Printing Office.

While the values in table III-11 often will be the actual design control, it is recognized that for high type alinement longer superelevation runoff may be desirable. Further, the requirements of pavement drainage or smoothness in pavement edge profile may call for adjustment in values.

TABLE III-11

LENGTH REQUIRED FOR SUPERELEVATION RUNOFF—2-LANE PAVEMENTS

Superelevation rate, foot per foot	L—Length of runoff in feet for design speed, mph of:							
	30	40	50	60	65	70	75	80
	12-foot lanes							
.02	35	40	50	55	60	60	65	65
.04	70	85	95	110	115	120	125	130
.06	110	125	145	160	170	180	190	200
.08	145	170	190	215	230	240	255	265
.10	180	210	240	270	290	300	330	330
.12	215	250	290	325	345	360	390	395
	10-foot lanes							
.02	30	35	40	45	50	50	55	55
.04	60	70	80	90	95	100	105	110
.06	90	105	120	135	145	150	160	165
.08	120	140	160	180	190	200	210	220
.10	150	175	200	225	240	250	265	275
.12	180	210	240	270	290	300	320	330
Design minimum length regardless of superelevation	100	125	150	175	190	200	220	240

When a spiral curve is used the superelevation runoff usually will be effected over the whole of the spiral length. Depending on the formula and factors used, the length of spiral for a particular curve and design speed may be greater or less than the length of runoff shown in table III-11. For the most part, the calculated values for length of spiral and length of runoff do not differ materially. In view of the empirical nature of both, an adjustment in one to avoid two sets of values is desirable for purposes of design control. The length of runoff is applicable to all superelevated curves and it is concluded that this value also should be used for minimum lengths of spiral. This is consistent with design practice, and experience indicates that the values in table III-11 for lengths of runoff could also be satisfactory for use as lengths of spirals. Minimum design lengths are shown in tables III-7 to -10 in relation to degree of curve and superelevation rate. The values given are for 12-foot lanes. For other lane widths, the lengths of runoff vary in the proportion of the actual lane width to 12 feet. While shorter lengths could be applied for design with 10- and 11-foot lanes, considerations of uniformity and practical use of the empirically derived values suggest that the values for 12-foot lanes be used in all cases.

The length of runoff applicable to pavements wider than 2 lanes is subject to the same theoretical derivation as for 2-lane highways. On this basis the lengths for 4-lane highways would be double the derived values for 2-lane highways, and those for 6-lane highways would be triple. While lengths of this order may be considered desirable, especially with spirals where the changing superelevation rate is effected over its length, it frequently is not feasible to supply lengths based on such direct ratios. On the other hand, most engineers agree that superelevation runoff lengths for wide pavements should be greater than those for a 2-lane highway but no generally accepted factor or criterion for length ratio has been established.

On a purely empirical basis it is concluded that minimum design superelevation runoff lengths for pavements wider than 2 lanes should be as follows:

3-lane pavements: 1.2 times length for 2-lane highway
4-lane undivided pavements: 1.5 times length for 2-lane highway
6-lane undivided pavements: 2.0 times length for 2-lane highway

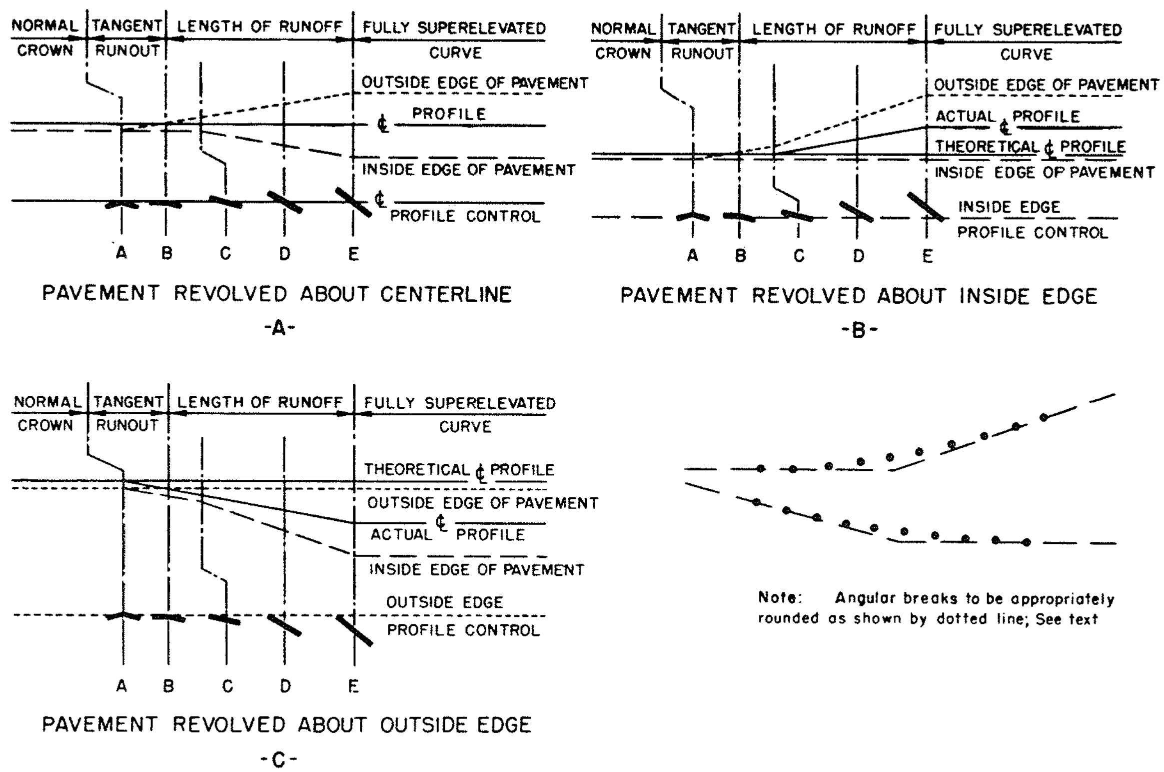
The 4-lane lengths shown in tables III-7 to -10 are determined on this basis. Proper design attention to obtain smooth edge profiles and avoid a distorted appearance may suggest lengths greater than these minimums. Runoff lengths for divided pavements are discussed under the section "Design with Medians."

Location with Respect to End of Curve

In alinement design with spirals, the superelevation runoff is effected over the whole of the transition curve. The length of runoff is the spiral length with the spiral T.S. at the beginning and the S.C. at the end. The change in cross slope begins by removing the adverse cross slope from the lane or lanes on the outside of the curve on a length of tangent just ahead of the T.S. This length is termed the tangent runout: see figure III-11. Between the T.S. and S.C. the pavement is rotated to reach the full superelevation at the S.C. By this design the whole of the circular curve has full superelevation. 107

In design of curves without spirals, the superelevation runoff also is considered to be that beyond the tangent runout. Empirical methods are employed to locate the superelevation runoff length with respect to the P.C. No method for division between the tangent and the circular curve can be completely rationalized. With full superelevation attained at the beginning of a circular curve (P.C.) the runoff lies entirely on the approach tangent, where theoretically no superelevation is needed. At the other extreme, placement of the runoff entirely on the circular curve results in a portion of the curve with less than the desired amount of superelevation. Most States design with part of the runoff length on the tangent and part on the curve. This compromise design control divides the undesirable features—the tangent receives some but not the maximum unneeded superelevation and the end portion of circular curve receives somewhat less than that needed.

In general, theoretical considerations favor the practice of placing a larger proportion of the runoff length on the approach tangent rather than on the circular curve. The resultant superelevation on the tangent is undesirable in that the driver may have to steer in a direction opposite to the direc-



DIAGRAMMATIC PROFILES SHOWING METHODS OF ATTAINING SUPERELEVATION

FIGURE III - 11

tion of the curve ahead to stay in line but the maximum side friction developed, which is equal to the rate of applied superelevation, is at all times below the rate of side friction considered comfortable and safe. A vehicle traveling at the design speed on the maximum degree of curve (maximum rate of superelevation) develops the maximum side friction considered safe and comfortable. To apply rates of superelevation less than the maximum at any point on the curve means that vehicles traveling at the design speed will develop side friction factors in excess of the allowable maximum. While the side friction factor developed upon the tangent is undesirable, the development on curves of friction factors greatly in excess of the design basis results in hazardous conditions.

The resulting side friction factor depends upon the actual vehicle path of travel. Regardless of the superelevation runoff, some form of transition path of travel can be expected. This actual transition path usually will begin well back on the tangent and will end beyond the beginning of the circular curve, depending upon the speed, sharpness of curvature, width available, and effect of other traffic. What might appear to be an undesirable cross slope upon the tangent actually compensates for the curvilinear path of the vehicle. And what may be considered lack of superelevation at the beginning of the circular curve proper is compensated for by the vehicle traveling a curvilinear path that is flatter than the roadway circular arc.

It is evident from the above that in alinement design without spirals the placement of the length of runoff with respect to the P.C. cannot be determined exactly from available practice and information. In general, design with 50 to 100 percent of the length of superelevation runoff on the tangent can be considered as suitable. For a more precise design control it is concluded that from 60 to 80 percent of the length of runoff preferably should be located on the tangent at curves without spirals.

Methods of Attaining Superelevation

Change in cross slope should be effected with edge-of-pavement profiles which are rounded to smooth flowing lines. The methods of changing cross slope are most conveniently discussed in terms of straight line relations and controls, but it is emphasized that these straight line profiles with angular breaks are to be rounded in the finished design as later discussed.

Three specific methods of profile design in attaining superelevation are practiced; namely, (1) revolving the pavement about the centerline profile, (2) revolving the pavement about the inside edge profile, and (3) revolving the pavement about the outside edge profile. Figure III-11 illustrates these three methods diagrammatically. The centerline profile, drawn as a horizontal line, represents the calculated profile, which may be tangent, a vertical curve, or a combination of the two. The small cross sections at the bottom of each diagram indicate the pavement cross slope condition at the lettered points.

Figure III-11A illustrates the method where the pavement section is revolved about the centerline profile. This general method is the most widely used in design since the required change in elevation of edge of pavement is made with less distortion than by the other methods. The usual calculated centerline profile is the base line and one-half of the required elevation change is made at each edge.

Figure III-11B illustrates the method where the pavement section is revolved about the inside edge profile. In this case the inside edge profile is determined as a line parallel to the calculated centerline profile. One-half of the required change in cross slope is made by raising the centerline profile with respect to the inside pavement edge and the other half by raising the outside pavement edge an equal amount with respect to the centerline profile. The third method where the pavement section is revolved about the outside edge profile involves similar geometrics, as shown in figure III-11C, except that the change is effected below the upper control profile.

The design controls for attaining superelevation are nearly the same for all three of the methods. Cross section A at one end of the tangent runout is a normal crown section. At cross section B, the other end of the tangent runout and the beginning of the spiral curve or length of runoff, the lane or lanes on the outside of the curve are made horizontal with the centerline profile. Based on the relative slopes previously established the tangent runout distance on 2-lane roads varies from about 25 to 40 feet for the average crown rate value of 0.012 foot per foot. Where the crown rate is 0.02 foot per foot about 35 to 60 feet are required. In practice, somewhat longer distances usually are desirable and should be used wherever possible. On curves where parabolic or circular crowns are not retained, the normal crown should be changed to straight cross slopes in the tangent runout length.

At cross section C the pavement is a plane, superelevated at the normal crown rate. Between cross sections B and C the outside lane or lanes change from a level condition to one of superelevation at the crown rate, which rate is retained on the inner lanes. Between cross sections C and E the pavement
110 section is revolved to the full rate of superelevation. The rate of cross slope at any intermediate point, cross section D, is proportional to the distance from section B.

Considering the infinite number of profile arrangements and in recognition of the specific problems of drainage, avoidance of critical grades, aesthetics, fitting the pavement to the ground, etc., the adoption of any specific axis of rotation, i.e., choice between methods A, B, or C in figure III-11, cannot be recommended. Each runoff section should be considered an individual problem to obtain the most pleasing and functional results. In practice, any longitudinal profile line for the axis of revolution may be the most adaptable for the problem at hand. In any case, a smooth edge profile is desired; see next heading.

In an overall sense the method of rotation about the centerline usually is the most adaptable. For example, in figure III-11A the change in longitudinal grade required for each edge profile with a relative slope of, say, 1 in 200 to the centerline, is 0.5 percent at point E. In figure III-11B, no change is required in the direction of the inside edge of pavement profile, whereas double the above amount, or 1.0 percent, is required in the outside edge of pavement profile at point E.

The method shown in figure III-11B is preferable to the other two in cases where the lower edge profile is a major control, as for drainage. With uniform profile conditions its use results in the greatest distortion of the upper edge profile. Where the overall appearance is to be emphasized, the method of figure III-11C is advantageous in that the upper edge profile—the edge most noticeable to drivers—retains the smoothness of the control profile.

The shape and direction of the centerline profile may determine the preferred method for attaining superelevation.

Design of Smooth Profiles for Pavement Edges

In the diagrammatic profiles of figure III-11 the tangent profile control lines result in angular breaks at cross sections A, C, and E. For general appearance and safety, these breaks should be rounded in final design by insertion of vertical curves. With the method of figure III-11A, usually short vertical curves are required. Even where the maximum relative slopes are utilized (minimum length of runoff) the length of vertical curve, required to conform to the 0.67 percent break at the 30 mph design speed and 0.4 percent break at the 70 mph design speed, need not be great. Where the pavement is revolved about an edge, these grade breaks are doubled to 1.33 percent for the 30 mph and 0.8 percent for the 70 mph design speed. Greater lengths of vertical curves obviously are needed in these cases. Positive controls cannot be cited for the lengths of vertical curves at the breaks in the diagrammatic profiles. However, for an approximate guide, the minimum vertical curve length in feet can be used as numerically equal to the design speed in mph. As the general profile condition may determine, greater lengths should be used where possible.

Several design procedures are followed by different agencies in the development of profiles for runoff sections. Some compute the edge profiles on the straight line basis of figure III-11 and insert vertical curves as eye adjustments in the field. Other agencies specify minimum lengths of vertical curves at the breaks for edge profiles. Some employ a selected reverse vertical curve for the entire transition section with resultant computed edge profiles. This method is laborious when the edge vertical curves are superimposed on a centerline vertical curve. It does provide essential controls to the designer and should yield uniformity of results.

Several agencies use a design procedure in which the runoff profiles are determined *graphically*. The method essentially is one of "spline line development." In this method the centerline or other base profile, which usually is computed, is plotted on a vertical scale of 1 in. = 2 ft. or 1 in. = 1 ft., and a horizontal scale in the range of 1 in. = 50 ft. to 1 in. = 20 ft. Superelevation control points are plotted in the form of the break points shown in figure III-11. Then by means of a spline, curve template, ship curve, or circular curve, smooth flowing lines are drawn to approximate the straight line controls. The natural bending of the spline, or if curve templates are used the obtainment of smooth flowing profiles without marked distortion, almost always will satisfy the requirements for minimum smoothing. Once the edge profiles—and lane profiles if required—are drawn in the proper relation to one another, elevations can be read for stations, half stations, quarter stations, or otherwise as necessary for construction control.

An important advantage of the graphical or spline line method is the intimate study possibilities it affords the designer. Alternate profile solutions can be developed with a minimum expenditure of time. The net result is a design that is well suited to the particular control conditions. The engineering design labor required for this procedure is a minimum. These several advantages make this method preferable over the other methods of developing profile details for runoff sections.

TABLE III-12

CALCULATED AND DESIGN VALUES FOR PAVEMENT WIDENING ON OPEN HIGHWAY CURVES

2-Lane Pavements, One-Way or Two-Way

Degree of curve	Widening, in feet, for 2-lane pavements on curves for width of pavement on tangent of: 24 feet						22 feet					20 feet			
	Design speed, mph						Design speed, mph					Design speed, mph			
	30	40	50	60	70	80	30	40	50	60	70	30	40	50	60
1	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.5	0.5	1.0	1.0	1.5	1.5	1.5	2.0
2	0.0	0.0	0.0	0.5	0.5	0.5	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5
3	0.0	0.0	0.5	0.5	1.0	1.0	1.0	1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5
4	0.0	0.5	0.5	1.0	1.0		1.0	1.5	1.5	2.0	2.0	2.0	2.5	2.5	3.0
5	0.5	0.5	1.0	1.0			1.5	1.5	2.0	2.0		2.5	2.5	3.0	3.0
6	0.5	1.0	1.0	1.5			1.5	2.0	2.0	2.5		2.5	3.0	3.0	3.5
7	0.5	1.0	1.5				1.5	2.0	2.5			2.5	3.0	3.5	
8	1.0	1.0	1.5				2.0	2.0	2.5			3.0	3.0	3.5	
9	1.0	1.5	2.0				2.0	2.5	3.0			3.0	3.5	4.0	
10-11	1.0	1.5					2.0	2.5				3.0	3.5		
12-14.5	1.5	2.0					2.5	3.0				3.5	4.0		
15-18	2.0						3.0					4.0			
19-21	2.5						3.5					4.5			
22-25	3.0						4.0					5.0			
26-26.5	3.5						4.5					5.5			

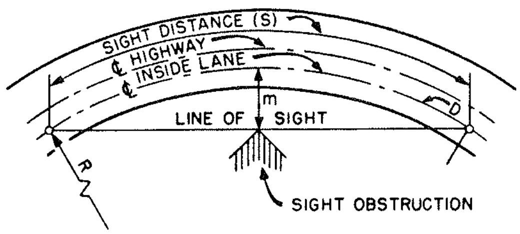
NOTE: Values less than 2.0 may be disregarded.
3-lane pavements: multiply above values by 1.5.
4-lane pavements: multiply above values by 2.
Where semitrailers are significant, increase tabular values of widening by 0.5 for curves of 10 to 16 degrees, and by 1.0 for curves 17 degrees and sharper.

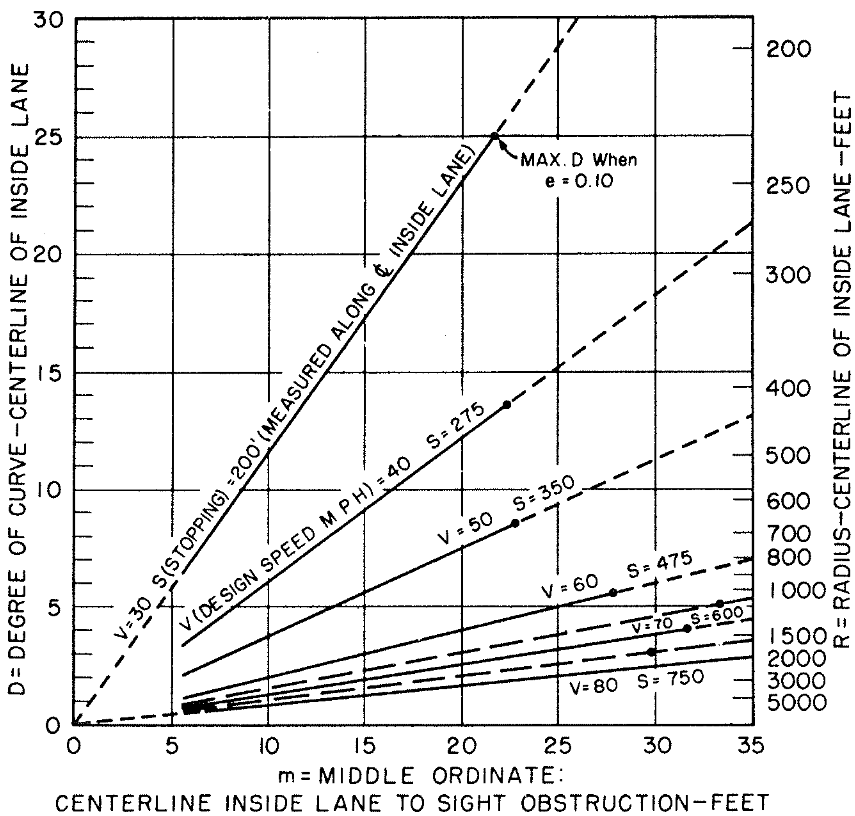
NOTE: Table III-12 on the preceding page and Figure III-13 below were referenced on page 11 of the text.

$$m = \frac{5730}{D} \text{ vers } \frac{SD}{200}$$

Also $m = R(\text{vers } \frac{28.65S}{R})$

And $S = \frac{R}{28.65} \cos^{-1} \frac{R-m}{R}$

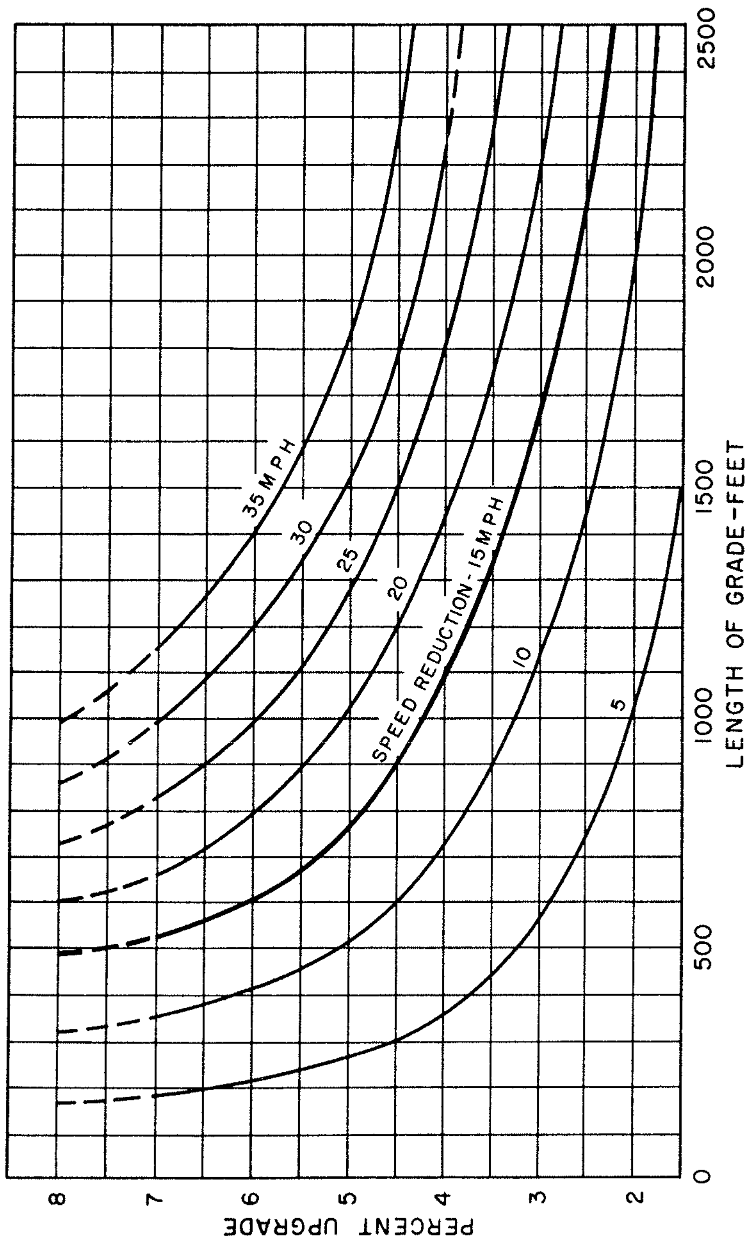




STOPPING SIGHT DISTANCE ON HORIZONTAL CURVES

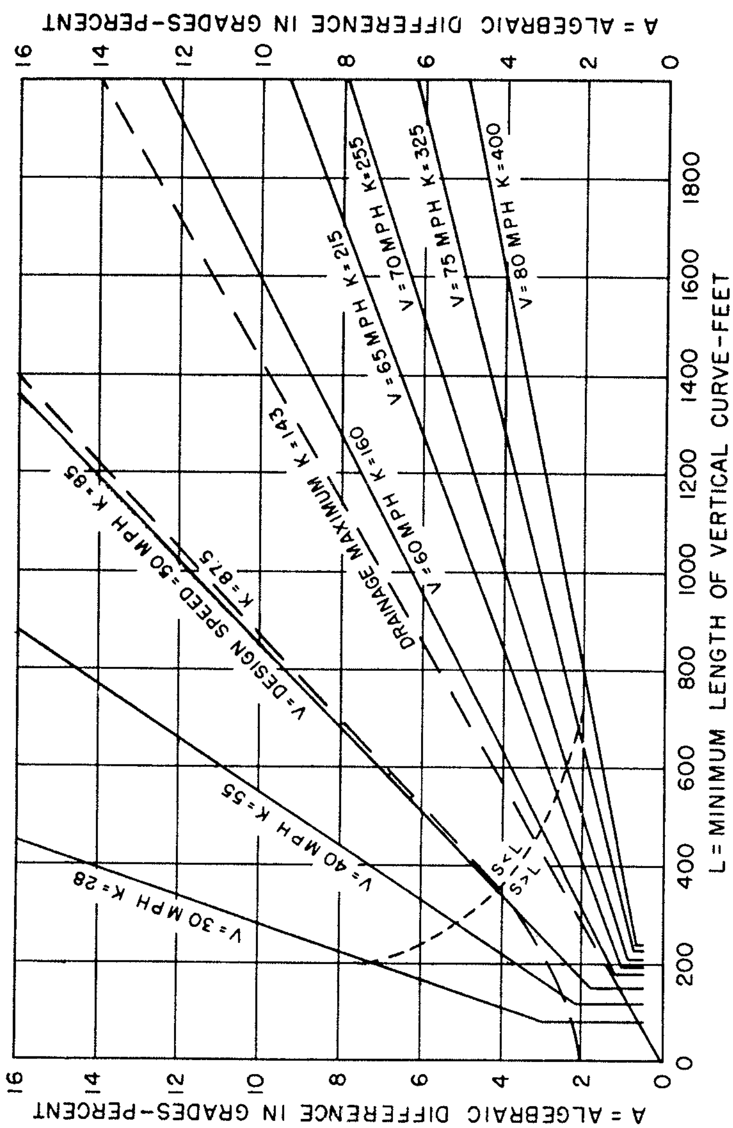
OPEN ROAD CONDITIONS

FIGURE III - 13

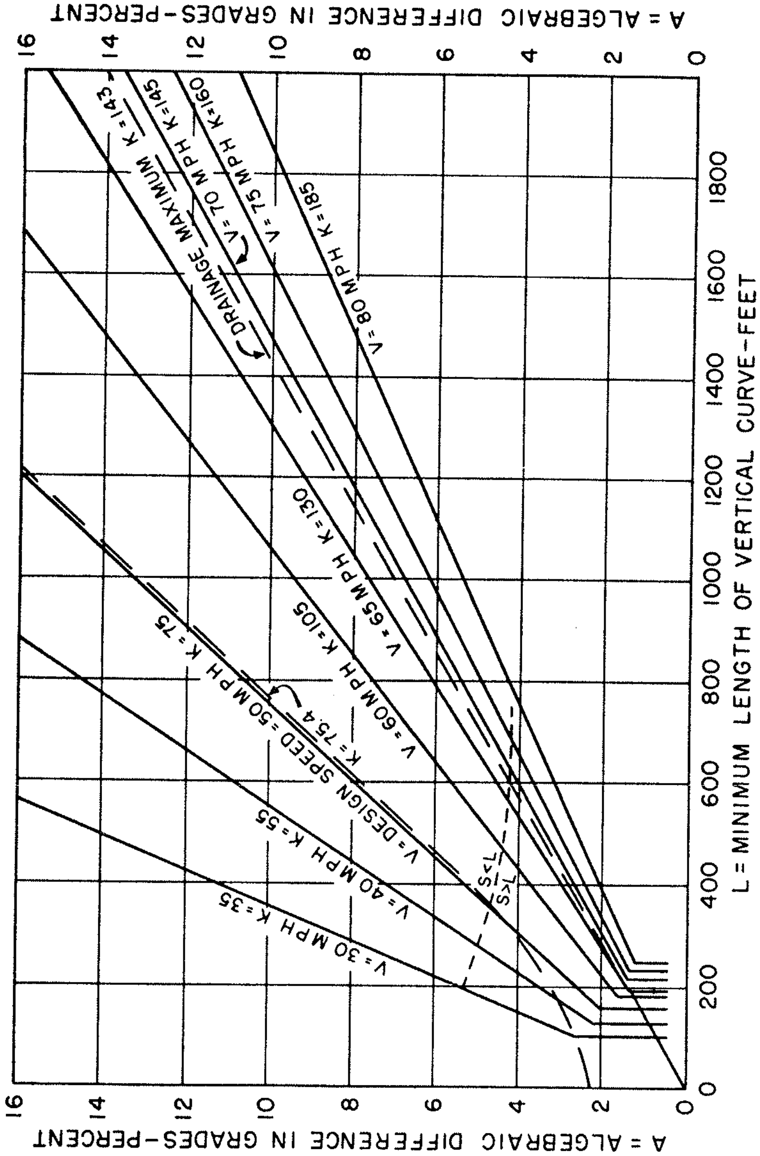


CRITICAL LENGTHS OF GRADE FOR DESIGN
ASSUMED TYPICAL HEAVY TRUCK OF 400 POUNDS PER HORSEPOWER
FIGURE III-17

NOTE: Figure III-17 on the preceding page and Figures III-19 and III-20 that follow were referenced on page 13 of the text.



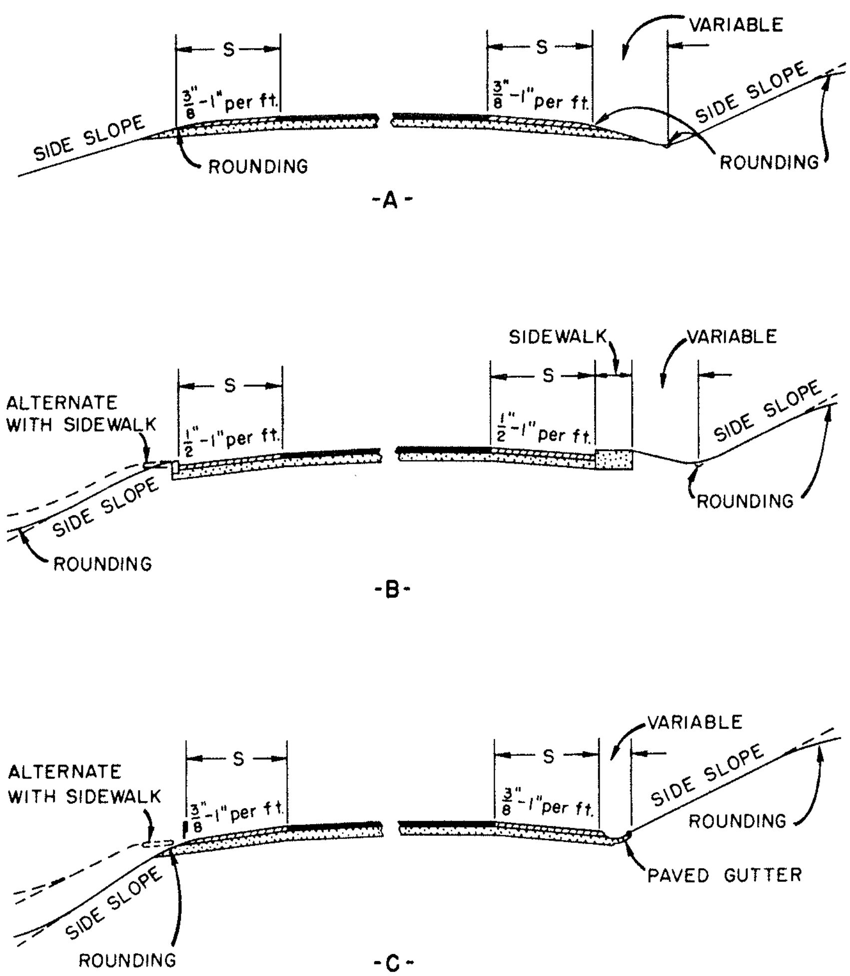
DESIGN CONTROLS FOR CREST VERTICAL CURVES
STOPPING SIGHT DISTANCE
FIGURE III-19



DESIGN CONTROLS FOR SAG VERTICAL CURVES

FIGURE III-20

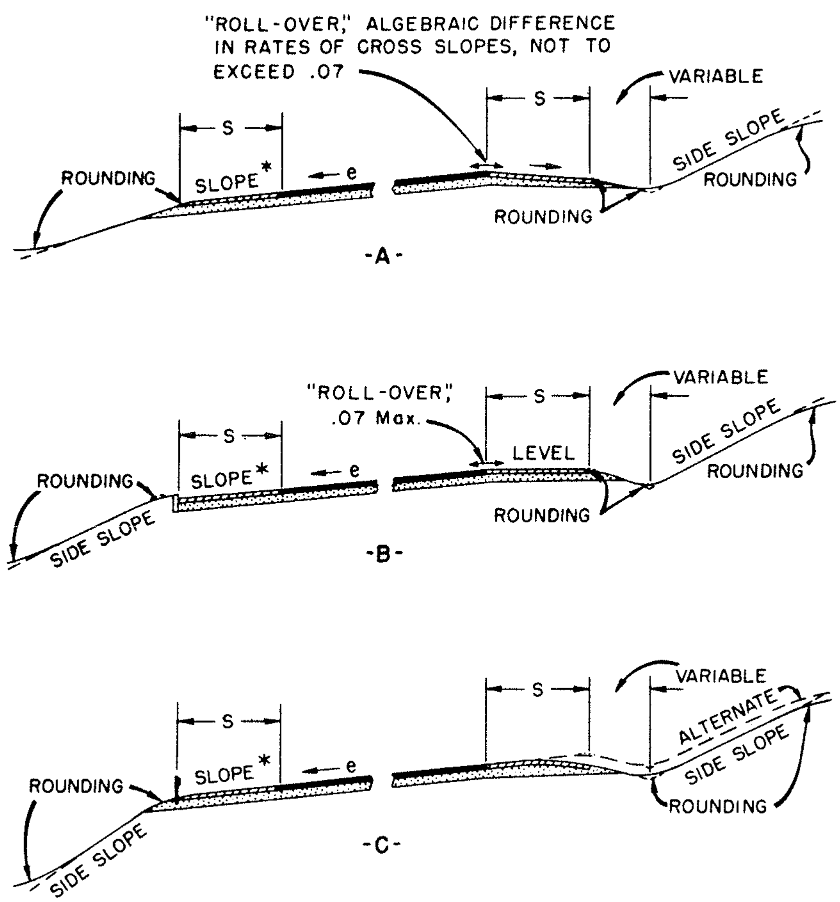
NOTE: Figures IV-3 and IV-4 below were referenced on page 19 of the text.



Note: S = Usable shoulder, 4' to 12' wide

TYPICAL CROSS SECTIONS – NORMAL CROWN

FIGURE IV - 3

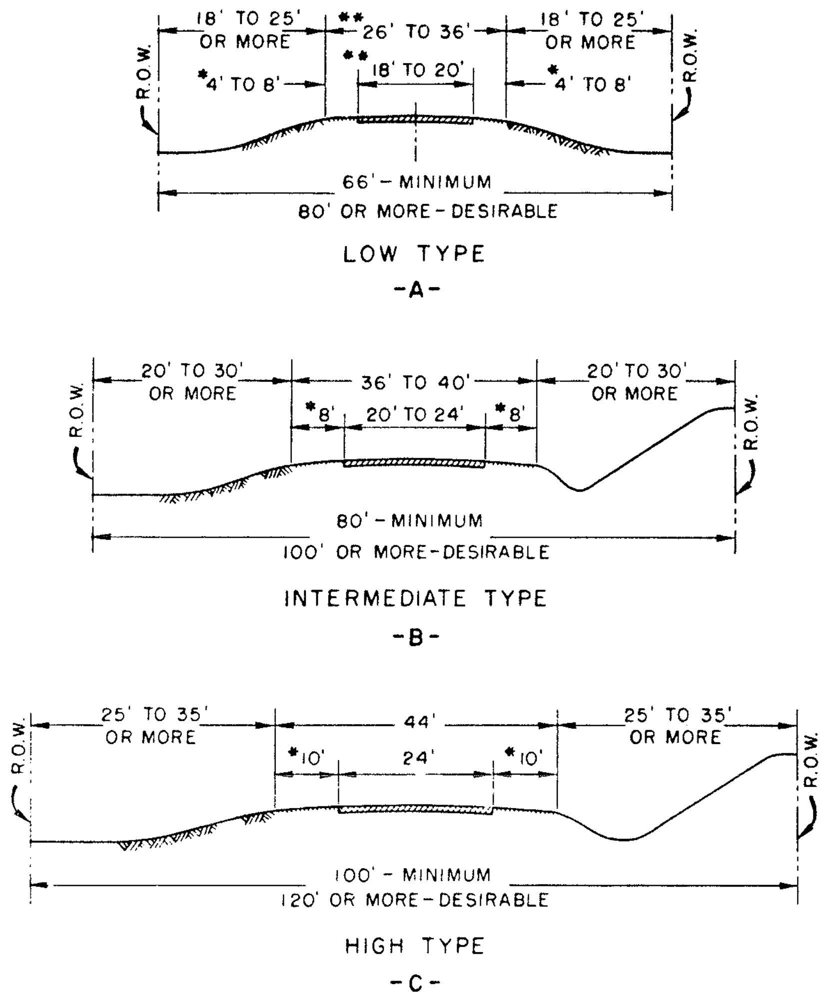


Note: S = Usable shoulder, 4' to 12' wide

*Slope = Superelevation rate (e) where greater than normal shoulder slope

TYPICAL CROSS SECTIONS—SUPERELEVATED

FIGURE IV - 4



* Usable shoulder width
** For low volume roads with few trucks

CROSS SECTIONS AND RIGHT-OF-WAY WIDTHS FOR 2-LANE RURAL HIGHWAYS

FIGURE V-1

NOTE: Figure V-1 above was referenced on page 21 of the text.

120

Western Hills Road, Nepal.

LOW COST ROADS

DESIGN, CONSTRUCTION
AND MAINTENANCE

Drafted by a group of international experts
L. ODIER, R. S. MILLARD,
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under the responsibility of UNESCO

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CONTENTS

PREFACE

Under the United Nations Development Programme, Unesco is carrying out a number of assistance projects in developing countries in the field of scientific and engineering research and standardisation. Unesco supports these projects under its Regular Programme by extensive studies in fields where activities appear to be most promising for the transfer of technologies to the developing countries. Some important fields of this type are standardisation, engineering codes of practice and guides for the design and construction, or manufacture, of engineering works.

The absence of such standards, guides and recommendations may severely hamper economic progress. Foreign standards and codes are often used although they may not be the most suitable for reasons of climatic conditions, or because of the different stage of economic and social development of the countries. The drafting and development of the principal standards and codes is a matter of prime importance and great urgency to the economic progress of developing countries.

The knowledge available in the industrial countries in the field of standards and engineering codes is immense. In order to apply this knowledge in developing countries, the available material must be judiciously selected, sifted and modified by experienced engineers and research workers, in co-operation with the international engineering associations and organisations for standardisation. This activity is an outstanding example of technical assistance where existing knowledge can be applied to development efficiently and with the least delay.

The present guide for the design, construction and maintenance of rural roads is the first attempt at providing engineering codes for use in the developing countries. Two other engineering studies in different fields are also under way and it is hoped that these studies are the forerunners of an extensive series of engineering codes covering a wide range of technical subjects applicable to various geographical regions.

This book has been drafted by an international working group of outstanding experts: L. Odier (France), R. S. Millard (United Kingdom), Pimentel dos Santos (Portugal) and S. R. Mehra (India). The document has been discussed and approved by the Low-cost Road Committee of the Permanent International Association of Road Congresses (PIARC). Unesco wishes to express its gratitude both to the experts and to PIARC for their valuable work and their most efficient co-operation.

1

INTRODUCTION

The construction or improvement of the road network is rightly regarded as one of the most effective ways of promoting economic development of a country; infrastructure in general, and transport infrastructure in particular, is unquestionably one of the decisive factors in development. It is clearly one of the major tasks of the leaders of a developing country to match their efforts in this direction with those made in other directions or, in other words, to set up this infrastructure in harmony with other development actions so that it is neither lagging too far behind nor racing too far ahead.

The state of a road network in a country and its economic development are therefore closely linked, first because, in practice, the former is one of the conditions for the latter, but also, secondly, because of this essential harmony between different development actions.

These considerations explain why the design and construction of roads in developing countries have a special character. In these countries roads are not built solely under the pressure of traffic, that is to say, in order to meet a particular demand, but also and above all, within the framework of integrated development actions, which all combine together to create this demand and to satisfy it. Furthermore, and in the light of this continuous interaction between the service offered and the use which is made of it, the road must be designed to meet the needs in order to ensure the best return for the efforts made.

The economy of these countries is in fact very sensitive to road investments, which rapidly create entirely new trade. Any premature or excessive investment in a road would proportionately delay or restrict actions on other parts of the network, and this may have important effects on the economy.

While *economic* adaptation to development needs is one necessity, another is the adaptation of *techniques* in the light of the special conditions encountered, such as climate, geology and the nature of the soils, intensity and nature of traffic, structure and management of construction and maintenance services and the like. Joining hands at this point with economic necessity, road engineers have to discover how they can make the best use of their available resources—essentially men and natural materials; thus they are prompted to work out special techniques, often different from those used in the developed countries.

The economic adaptation of the road to special needs and its technical adaptation to local conditions are two complementary aspects which should dominate the planning of roads in developing countries. The operations of constructing a road are usually not too difficult once the planning and design have been performed. Analysis of the economic and social factors which should influence the planning, the location, the design and the maintenance of the road should be a prime consideration in establishing a road programme.

In view of the diversity of conditions encountered in the developing countries and the fact that most of these countries are in regions with a tropical or sub-tropical climate, this Manual has been deliberately limited to road problems affecting these regions. It has also been limited to the problems of rural roads, including, however, certain urban or suburban roads which have a rural character.

The Manual deals in succession with the following subjects:

Road planning.

Geometric characteristics.

Road materials and design of carriageways.

Drainage.

Road construction.

Road maintenance.

3

GEOMETRIC DESIGN

3.1 INTRODUCTION

The lay-out and other geometric characteristics of a road have a direct influence on its final cost and, therefore, on the overall cost of transports. It is the purpose of this chapter to set down the basic standards for the satisfactory design of roads in rural areas.

The task of the highway engineer in selecting geometric standards is not an easy one. He must deal with vehicle speed, human reaction time, dynamics, centrifugal forces, vehicle characteristics, human behaviour and psychology, alignments, grades, rights-of-way, curvature and transition curves, pavement width, friction between wheels and pavement, roadway cross-section, drainage problems, etc. The most fundamental of these factors, from the standpoint of design, are alignment and right-of-way. Unless proper and balanced judgement is exercised in choosing them, taking into account the function of the road in a reasonable future period, the highway will soon become inadequate, with corresponding waste of the investment made. Therefore, a suitable classification of the road to be designed must be established in advance, relating the function of the road to the expectation of traffic in a given reasonable future period of time.

Besides the function of the road and its foreseen traffic intensity, topography is another basic factor to be carefully weighed for the selection of a convenient design speed, which, in turn, influences decisively the geometric characteristics to be adopted. Design speeds could be as high as 110 km/h (or more) on important roads in flat country, or as low as 30 km/h on secondary roads in difficult terrain.

In the selection of a suitable location for the road, great attention must be given to the drainage conditions and the availability of local materials. These matters are dealt with in Chapters 2, 4 and 5.

3.2 TRAFFIC AND DESIGN SPEEDS

The design speed that is used for the correlation of the geometric characteristics and features of the road is dependent upon the terrain, the type and volume of traffic anticipated and the type of highway to be constructed. It is defined by the AASHO as a speed determined for design and correlation of the physical features of a highway that influence vehicle operation. It is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favourable that the design features of the highway govern. In large countries distances to be travelled are often great, and thus it is desirable to aim at the highest design speed practical in the circumstances. As a general rule, and for the benefit of future traffic, maximum design speeds should be used in determining horizontal alignment since costs are usually not critical.

Roads are usually classified by their function, and three classes, Primary, Secondary and Feeder roads are generally recognised.

Primary roads are of wider interest, carry relatively high volumes of traffic between main population centres or between major areas of production and centres of export and should be designed to facilitate fast traffic movement. Secondary roads provide access to higher-type roads and connect smaller communities and nearby areas in addition to serving adjacent property. Feeder roads are of local interest, providing access to adjacent property and to higher-type roads, besides facilitating community activities.

The choice of design standard is usually a matter for judgment in which one of the most important considerations is the expectation of traffic. The time scale to be considered for this choice is an important point since once the right-of-way has been acquired and the road constructed, it is often difficult to alter the horizontal alignment because of development taking place adjacent to the road, except in very sparsely populated areas. Due to the fact that traffic forecasts over a longer period are generally hazardous, design standards are frequently adopted with a view to meeting with the needs of traffic in ten years' time after completion of the construction. In the case of primary important roads, it is recommended to consider as long a period as possible, especially concerning the choice of the horizontal alignment design standards which control the right-of-way.

This recommendation applies more particularly where major bridges are involved, since they should be sited and designed so that they can be satisfactorily used for a period of twenty years or more. In the case of very large structures a considerably longer period should be allowed for. To save initial cost, it may be possible to design the bridges initially with capacity for traffic expected within, say ten years, but with provision for subsequent widening. In the conditions where widening of the foundations would be particularly difficult and expensive, it may be advisable to build the substructure for the ultimate width of the bridge in the first instance.

The standards suggested for different classes of road in this book are related to traffic intensities in the following ranges:

Primary roads	100–5000 vehicles per day
Secondary roads	50– 800 vehicles per day
Feeder roads	Less than 100 vehicles per day

This refers to 24-hour counts or, where these are not available, to 16-hour counts plus 6 per cent, and to 7-day averages. (When the night traffic is abnormally heavy, a 24-hour count should be used.)

Roads with traffic higher than 5000 vehicles per day must be the subject of a special design and are not considered here.

The corresponding design speeds are summarised, together with other pertinent data, in Table 3.1.

3.3 DESIGN RELATED TO VERTICAL ALIGNMENT

3.3.1 GENERAL

The vertical alignment of the road has a direct bearing upon the construction cost, the operating cost of vehicles using the road and the number of accidents. The following points must be carefully considered:

1. Good correlation with the horizontal alignment.
2. Provision of adequate sight distance over all crests.
3. Avoidance of very short sag vertical curves.
4. Avoidance of a short grade between two crests or two sag curves.

Table. 3.1 DESIGN CHARACTERISTICS FOR ROADS IN DIFFERENT TYPES OF TERRAIN

Characteristic / *Road type*	*Terrain*	*Design speed* km/h	*Min. radius of curvature** m	*Max. gradient* %	*Max. length of grade* m	*Formation width (Permanent surfacing and shoulders)* m	*Width of permanent surfacing* m
Primary	Flat or rolling	80–110	190–360	4	None	10–13	6–7·5
	Hilly	55–80	90–190	5–7	600 over 4%	10–13	—
	Mountainous	40–55	50–90	7–9	400 over 6%	8–10	—
Secondary	Flat or rolling	60–80	110–190	5	None	10–12	6–6·8
	Hilly	50–60	75–110	5–7	None	10–12	—
	Mountainous	35–50	35–75	7–9	750 over 6%	8–9	—
Feeder	Flat or rolling	50–60	75–110	7	None	7·5–8	5·5–6
	Hilly	35–50	35–75	7–9	None	7·5–8	—
	Mountainous	25–35	30–35	9–12	1 000 over 9%	7·5–8	—

* The absolute minimum radius of curvature shown here takes account of a superelevation of 10% and a sideway force coefficient of 0·16.

5. Avoidance of a short drop immediately before a long upgrade.
6. Avoidance of the combination of two vertical curves in the same direction (they should normally be replaced by a single vertical curve).

In short, the aim is to design a flowing alignment, following the terrain and merging itself harmoniously in the surrounding countryside, without sharp discontinuities.

3.3.2 MAXIMUM GRADE

Maximum grades to be used in different classes of roads and types of terrain are given in Table 3.1. The higher limits in mountainous terrain should be used only where either serious construction difficulties arise or the cost of works is very high, and provided that:

1. The section of road has a length not in excess of 1000 m.
2. The section is straight or, when curved, has a radius greater than 300 m.

   However, grades in excess of 6% should be avoided whenever possible wherever heavy vehicles are expected to use the roadway. Level grades in cut sections are to be avoided as far as possible in order to facilitate surface water drainage. The minimum longitudinal grade is normally 0·5%.

3.3.3 CRITICAL LENGTHS OF GRADE

Due consideration must be given to the fact that the length of the upgrade is an important factor in the cost of vehicle operation, besides also having some bearing on the accident rate on the road.

Therefore, sections of road with very steep grades should be limited in length. Recommended limits for maximum length of grade are given in Table 3.1.

Where it is necessary to exceed the critical length of grade on heavily trafficked roads, it is desirable to provide either safe passing distances on the rise or a climbing lane for the heavy vehicles. On more lightly trafficked roads, it may be necessary to prohibit overtaking on such lengths.

3.3.4 VERTICAL CURVES

Between two consecutive grades, vertical parabolic or circular curves should be introduced in order to provide safe sight distance, driving comfort and good drainage. Sight distance is provided as follows:

Non-Passing (stopping) sight distance. This is the distance required for safe vehicle operation, when opposite-direction traffic is not involved in passing operations. The minimum distance should be sufficient at the design speed for perception and brake-reaction time and for bringing the vehicle to a stop. According to AASHO, this distance is measured along the line of travel from a point 1·30 m high representing the eye of the driver to a point 10 cm high representing a stationary object on the road surface.

Passing sight distance. The distance required to allow safe overtaking at design speed in the face of oncoming traffic. This distance is measured along the line of travel between two points 1·30 m high above the pavement of the road, representing the height of the driver's eye and of an object ahead.

The minimum necessary length of vertical curves is obviously related to the design speed.

Figs. 3.1 and 3.2 provide, in a convenient form, data for designing parabolic curves over crests, taking into account both non-passing and passing distances. It can easily be seen that much longer vertical curves are required to provide passing than non-passing sight distance.

3.4 DESIGN RELATED TO HORIZONTAL ALIGNMENT

3.4.1 GENERAL

The horizontal alignment of a road must be carefully chosen in order to provide good drainage, avoiding where possible, soft or swampy areas, and to minimise earthworks. In addition, the location of the road may be affected by the availability of road-making materials and by plans for stage construction. With regard to this last point, the future strengthening of the road structure and, in some cases, the improvement of the vertical alignment may be made at a lower cost when the need arises if proper care and professional know-how has been used in the initial design.

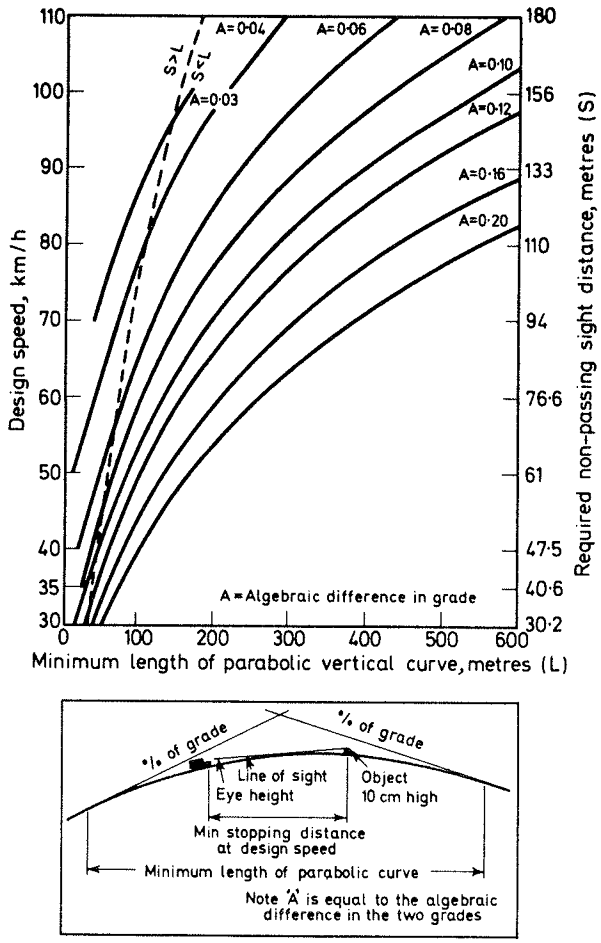


FIG. 3.1. *Non-passing (stopping) sight distance. Length of parabolic vertical curves (crest).*
These curves were derived from computations based on accepted design standards in the United States (including standards adopted by the American Association of State Highway Engineers).

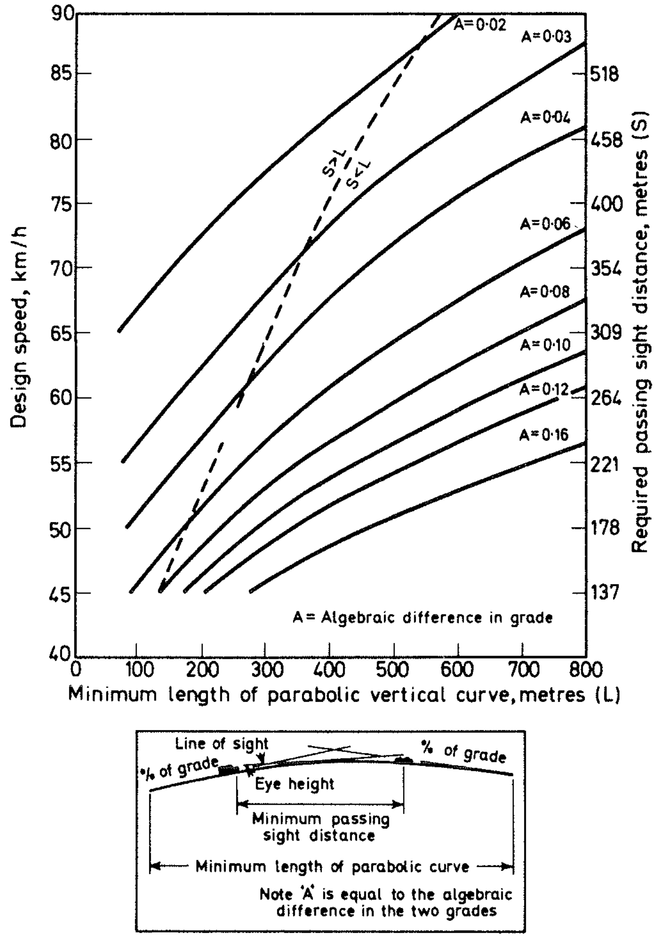


FIG. 3.2. *Passing sight distance. Length of parabolic vertical curves (crest).*
These curves were derived from computations based on accepted design standards in the United States (including standards adopted by the American Association of State Highway Engineers).

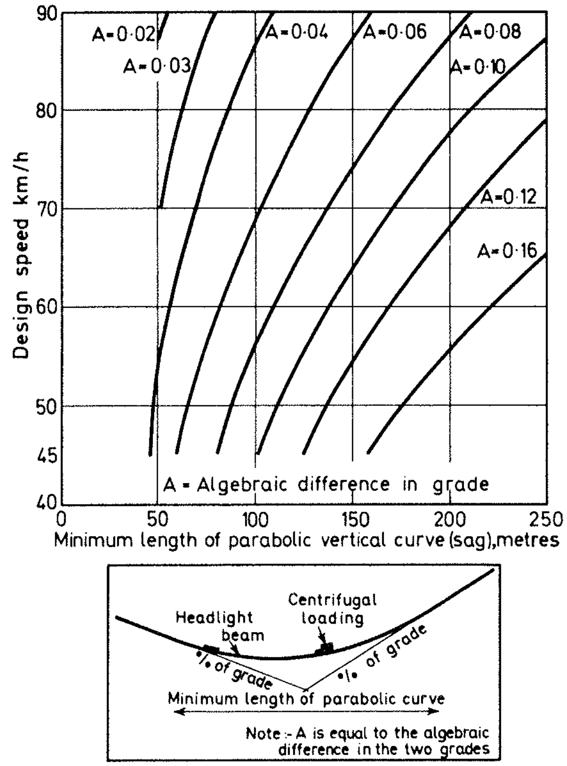


FIG. 3.3. *Minimum length of parabolic vertical sag curves.*
These curves were derived from computations based on accepted design standards adopted by the American Association of State Highway Officials.

In particular, the drainage structures and earthworks should be to a reasonably high standard as it is difficult and expensive to improve them later.

3.4.2 ALIGNMENT

Following the general principles stated above, the horizontal alignment should be as direct as possible, conforming with the topography of the terrain and, at the same time, fitting into the landscape to take advantage of aesthetic values, provided that no excessive extra cost results. To break monotony and avoid glare from headlights or a setting sun, the use of long straights is to be avoided; they should generally not exceed, 2500 m or 3000 m. If necessary, artificial curvature should be introduced, by deflecting the alignment approximately 4° left and right alternately by means of gentle curves.

At the end of long straights, short or sharp curves must be avoided as well as a short length of straight between two curves, especially when they turn in the same direction. In this case, the two curves should be substituted by a single or compound transition curve. The same procedure should be adopted instead of two successive curves in the same direction with pronounced difference in radius, length or degree of change in direction.

Between two curves which turn in opposite directions, transition straights should be used, with lengths depending upon the design speed and the characteristics of the curves. This is in order to provide satisfactory transition between the two curves.

3.4.3 HORIZONTAL CURVES

The largest radius of curvature compatible with the topographic conditions should be used whenever possible. Minimum radii of curvature are related to design speed.

In calculating the minimum radius of curvature for a given design speed, the maximum coefficient of friction is normally taken as $f = 0{\cdot}16$ and the maximum superelevation as $e = 0{\cdot}10$. The design speed is then given by the formula:

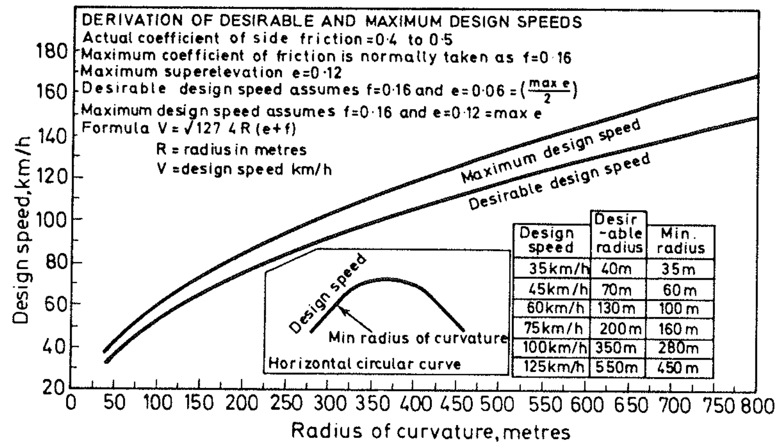
$$V = \sqrt{127{\cdot}4 . R(e + f)}$$

where V is in km/h and R in m.

Fig. 3.4 presents graphically the above information and suitable minimum radii for different classes of road are given in Table 3.1.

The minimum straight between two successive curves should be 100 m. When travelling around a horizontal curve, careful consideration must be given to ensure proper horizontal sight distance across the inside of the curve, i.e., the distance at which a driver can see an object lying in the roadway ahead, despite the occurrence of obstructions such as buildings, walls, cut slopes, etc.

Fig. 3.5 to 3.9 provide data concerning both non-passing



Design speed	Desir-able radius	Min. radius
35km/h	40m	35m
45km/h	70m	60m
60km/h	130m	100m
75km/h	200m	160m
100km/h	350m	280m
125km/h	550m	450m

FIG. 3.4. *Design speed (km/h) by radius of curvature (metres).*

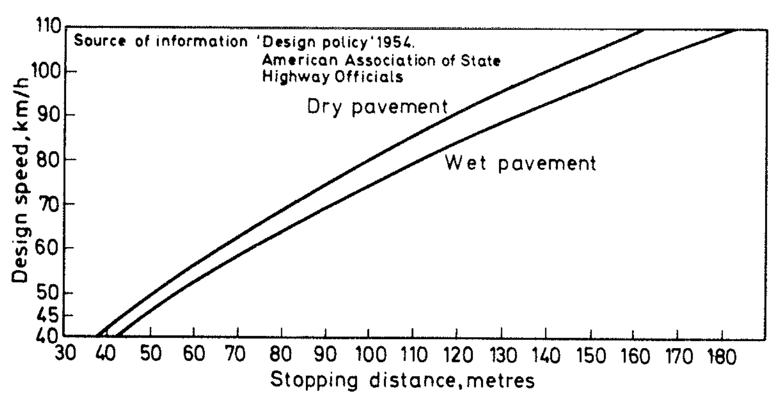


FIG. 3.5. *Stopping distance for the determination of horizontal sight distance by design speed, km/h.*

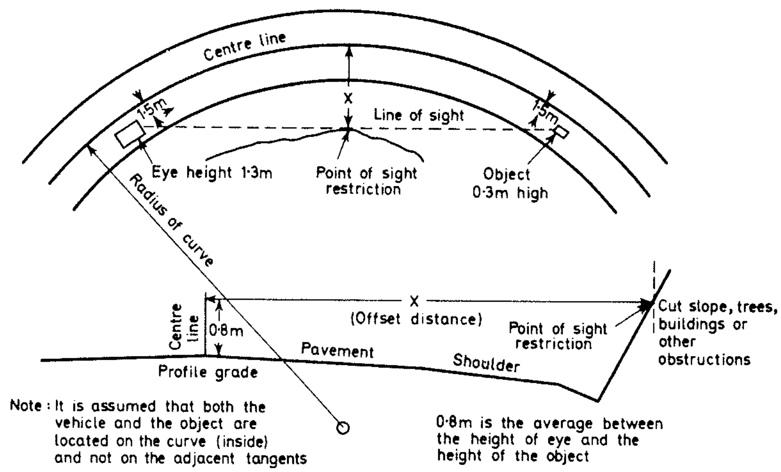


FIG. 3.6. *Restricted horizontal sight as related to stopping sight distance (see Fig. 3.7).*

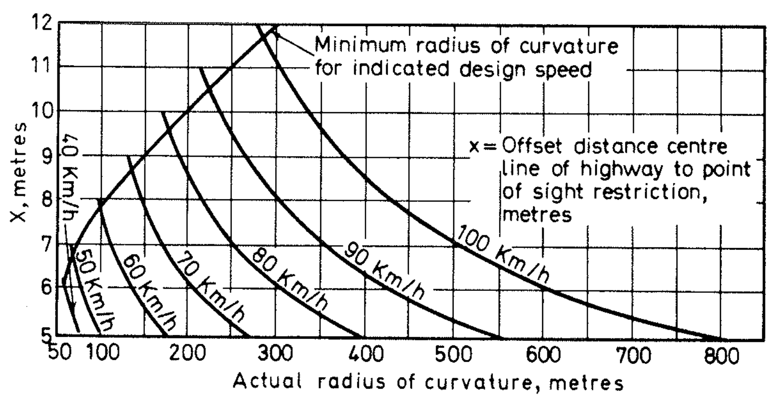


FIG. 3.7. *Minimum offset distance (radial) to horizontal sight restriction from centre line on curves for specified design speeds (stopping distance) (see Fig. 3.6).*

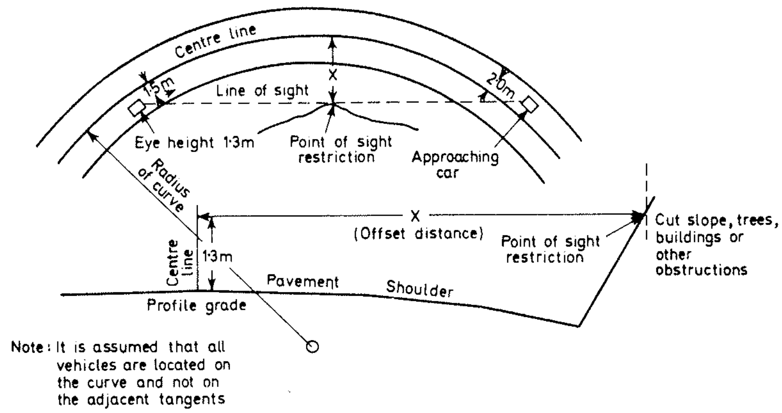


FIG. 3.8. *Restricted horizontal sight as related to passing sight distance (see Fig. 3.9).*

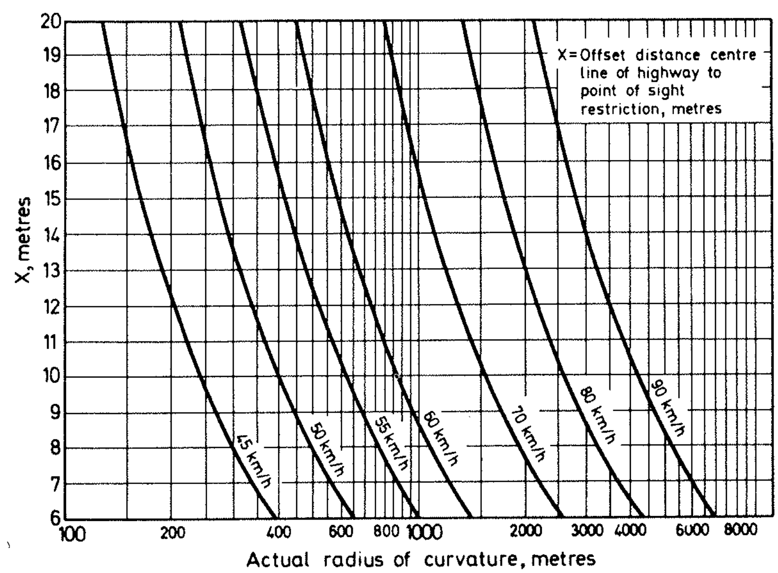


FIG. 3.9. *Minimum offset distance (radial) to horizontal sight restriction from centre line on curves for specified design speeds passing distance (see Fig. 3.8).*

(stopping) and passing horizontal sight distances to be adopted in design. Sometimes, suitable adjustment of the cross-section or even of the alignment may be required.

3.4.4 TRANSITION CURVES

Lengths of transition are necessary between straights and curves and between curves of different radii, both to enable the change from a camber to a superelevation to be made and to reduce the rate of change of centrifugal force. Transitions are usually made in the form of a spiral curve, although for greater radius of curvature, depending on design speed, circular curves could as well be used.

Table 3.2 provides values of the minimum length of spiral curve to be used in relation to the design speed, together with rates of superelevation up to a maximum of 10%.

3.4.5 COMBINATION OF VERTICAL AND HORIZONTAL ALIGNMENT

The complementary aspects of vertical and horizontal alignment must be considered in relation to the need to maintain sight distances and to preserve directional stability. When vertical and horizontal curves occur in combination or in close proximity to each other, it is recommended that the vertical curve should be wholly within or wholly outside the horizontal curve.

Care should be taken particularly to avoid sharp horizontal curves near the top of pronounced vertical curves, since drivers have little warning of the horizontal change in alignment. This is particularly dangerous on high-speed roads at night.

3.5 CROSS-SECTION

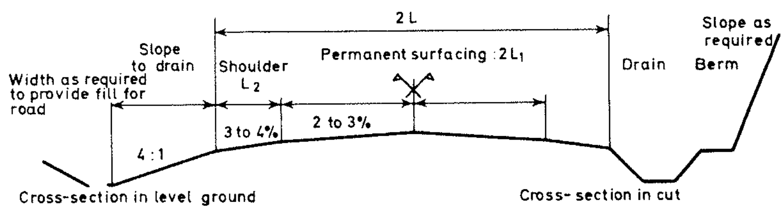
3.5.1 GENERAL

The cross-section to be adopted in the design of roads should be closely related to the classification of the road and the ultimate width of the carriageway in the last stage of evolution of construction. The following items should be particularly considered in selecting the cross-section for design.

1. *Right-of-way.* Road reserve to meet future needs of expansion during the life of the road.
2. *Carriageway width.* Depending on design speed, type of road and volume of traffic.
3. *Formation width.* Including carriageway and shoulders.
4. *Superelevation and widening at bends.* For counteracting centrifugal forces in curves.
5. *Drainage.* Providing the means for quick removal of surface water.

Recommended widths of formation and carriageway are given in Table 3.1 for different types of road and topographic conditions.

Whenever possible, consideration should be given to separate roadways for animal-drawn vehicles since these may be an important highway hazard in some developing countries.



	Flat, rolling or hilly terrain			Mountainous terrain		
	2L m	2L₁ m	L₂ m	2L m	2L₁ m	L₂ m
Primary roads	10–13	6–7·5	⩾ 2·0	8–10	6–7·5	⩾ 1·0
Secondary roads	10–12	6–6·8	⩾ 2·0	8–9	6–6·8	⩾ 1·0
Feeder roads	7·5–8·0	5·5–6·0	⩾ 1·0	7·5–8·0	5·5–6·0	⩾ 1·0

FIG. 3.10. *Typical standard cross-section of road. Formation width 2L.*

3.5.2 RIGHT-OF-WAY

Reasonable width of right-of-way is essential in order to avoid early obsolescence of the road. The following considerations must be borne in mind when establishing the width of road reserve.

1. That it must be adequate for construction (including drainage) and for subsequent maintenance.
2. That it should provide accommodation for future widening and for foreseeable minor realignments.
3. That it should provide accommodation for all necessary services, such as telephones, electric and other cables, etc., and where necessary for cattle driving.

According to the type of rural road, the following widths of right-of-way are suggested as examples:

Primary roads	50 m
Secondary roads	30 m
Feeder roads	20 m

In addition, regulations should be introduced concerning the limit of the building line.

3.5.3 STANDARD WIDTHS OF FORMATION AND CARRIAGEWAY

Fig. 3.10, showing a typical cross-section of road, is related to standard widths of formation and permanent surfacing as recorded in Table 3.1.

On roads with earth or gravel surfaces the running surface normally extends to the full width of the formation. Where it is anticipated that permanent surfacings will be provided before many years, the width of formation at the time of construction should be extended so that the correct width of formation will still be available after the height of construction has been increased by the provision of the permanent construction.

For feeder roads that are unlikely to carry more than fifty vehicles per day the minimum formation width may be reduced, but not below 6 m. For such roads a permanent bituminous surfacing of only 3·5 m may be used in order to enable funds to be spread efficiently over a greater distance in developing countries, where wider seals are not essential for existing light traffic.

The cross-section of the road should be maintained across minor structures (pipe and box culverts). Special cross-sections should be designed on bridges, taking account of traffic other than mechanical vehicles, in appropriate cases, such as pedestrians, bicycles etc. No reduction in the carriageway width should be accepted, except on long bridges where it may be justified to construct only single lane superstructure until such time as traffic volumes require widening.

In all structures over the road, namely tunnels, bridges, etc., the minimum vertical clearance of 5 m should be assured and the cross-section of the road maintained throughout.

3.5.4 SUPERELEVATION AND WIDENING IN CURVES

In order to counteract the centrifugal forces acting on a vehicle which is traversing a curve, it may be necessary to increase the cross-fall, and it is desirable to widen the road surface. The amount of superelevation provided is normally sufficient to balance up to half the centrifugal force, i.e.

$$S = \frac{V^2}{260R}$$

A maximum limit of superelevation is normally set in the interest of the safety of slow moving vehicles, particularly animal drawn vehicles, and an appropriate upper-limit would be 10 per cent.

The superelevation required depends on the design speed and on the radius of curve and typical superelevations are given in Table 3.3. The minimum radius of curve associated with different design speeds and classes of roads are given in Table 3.1. With curves of radius less than 150 m it is desirable to widen the permanent

Table 3.2 RECOMMENDED WIDENING ON INSIDE EDGE OF PAVEMENT AT HORIZONTAL CURVES

Radius of curvature (m)	*Widening on* 7·3 m *pavement* (m)	*Widening on* 6·7 m *pavement* (m)	*Widening on* 6·1 m *pavement* (m)
60 and less	0·60	0·90	1·20
80	0·40	0·70	1·00
100	0·20	0·50	0·80
120 to 150	0	0·30	0·60

Widening shall be tapered off in a distance of 15 metres at each end of the curve.

running surface and suggested amounts of widening are given in Table 3.2.

3.5.5 INTERSECTIONS

At intersections it is important to maintain adequate visibility so that vehicles on each approach road have sufficient sight distances on to the other roads. The sight distances required are essentially similar to the non-passing (stopping) sight distances required on curves. Because of the reduced speed of vehicles entering inter-

Table 3.3 RATE OF SUPERELEVATION (%) AND MINIMUM LENGTH OF SPIRAL CURVES

Radius m	*V* = *25* km/h *Super* %	*L*	*V* = *35* km/h *Super* %	*L*	*V* = *50* mh/h *Super* %	*L*	*V* = *80* km/h *Super* %	*L*	*V* = *110* km/h *Super* %	*L*
3 000	—	30	—	30	—	30	—	30	1·5	45
1500	—	30	—	30	—	30	1·6	45	3·1	55
1000	—	30	—	30	—	30	2·4	45	4·6	55
800	—	30	—	30	—	30	3·0	45	5·9	55
600	—	30	—	30	1·6	30	4·1	45	7·7	65
500	—	30	—	30	1·9	40	4·9	45	9·3	75
400	—	30	—	30	2·4	40	6·1	55		
300	—	30	1·6	30	3·2	40	8·2	65		
200	1·2	30	2·4	30	4·8	40				
150	1·7	30	3·1	30	6·4	50				
100	2·4	30	4·7	30	9·6	55				
80	3·0	30	5·9	35						
60	4·0	30	7·8	45						

Note.—Spiral curves *not* essential above the heavy line.

sections, the minimum radius of the transition curves at intersections of roads at grade may be as follows:

Primary roads	40 m
Secondary roads	30 m
Feeder roads	20 m

In the case of intersections of roads of different classes, the minimim radius corresponding to the lower class should be adopted.

3.5.6 DITCHES

The provision of adequate ditches and other drainage facilities is an essential feature of the road. Details are given in Chapter 4.

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GEOMETRIC DESIGN GUIDE FOR LOCAL ROADS AND STREETS

Part I

Approved by the Executive Committee on October 26, 1969

Part II, on November 7, 1970

Prepared by

Committee on Planning and Design Policies

American Association of State Highway Officials

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GEOMETRIC DESIGN GUIDE FOR LOCAL ROADS AND STREETS

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PREFACE

In August of 1945 the AASHO adopted a set of design standards for secondary and feeder roads which consisted of a single table of numerical values for various design elements. These standards were supplements to existing primary highway geometric standards. These two standards remained in effect until superseded by Geometric Design Standards for Highways Other Than Freeways, 1961, which single set of values was prepared to serve for both primary and secondary highways. Although these combined standards have served very satisfactorily for nearly ten years it became apparent that certain features unique to local roads were not adequately covered by the combined standards. Moreover, separate standards were desirable for administrative reasons. Accordingly, the Committee on Planning and Design Policies undertook the preparation of separate standards for local roads and streets.

Part I – Rural Highways was separately published in mid-1970. Part II – Urban Local Roads and Streets was completed late in 1970 for combination into this publication.

This guide represents a consensus of the members of many organizations that are responsible for the design and construction of local roads and streets throughout the United States. First drafts were based on a survey of existing practices in various State, county and city organizations. Comments on these drafts were solicited from the American Public Works Association, the National Association of County Officials, the National League of Cities, and within AASHO the Committee on Design, Construction and Maintenance of Secondary Roads, the Committee on Design, the Committee on Construction, and the Committee on Bridges and Structures. All comments received on the initial and later drafts were considered by the Committee on Planning and Design Policies to develop this final version.

DESIGN GUIDES FOR LOCAL ROADS AND STREETS

For purposes of design, highways should be classified by function with enumeration of design values for each functional type. The following guides are presented on a functional basis and are applicable to (1) collector rural roads and collector streets, and (2) local rural roads and local streets. In some cases, they may also apply for arterial roads and streets. In a jurisdictional highway classification, these guides apply generally to village or city streets, township and county roads, and State secondary roads and streets.

A local road or street is one that primarily serves for access to the farm, residence, business, or other abutting property. While local roads and streets may be planned, constructed, and operated with such service dominating, some of them properly include the "major street or highway" features of geometric design and traffic control measures to expedite the safe movement of through traffic. On these, the through traffic is local in nature and extent, rather than regional, trans-state or interstate. Local roads and streets constitute a high proportion of the highway mileage in the United States. The traffic volumes generated by the abutting land uses are largely short trips or a relatively small part of longer trips where the local road connects with highways of higher classifications. Because of the relatively low traffic volumes and the extensive mileage, design of local roads and streets is of a comparatively low order as a matter of practicality. However, to provide the requisite traffic mobility and safety, together with the essential economy in construction, maintenance and operation, they must be planned, located and designed so as to be suitable for the predictable traffic operations on them and consistent with the development and culture abutting the right-of-way.

The use of more liberal values than the minimums set out herein is recommended where it is economically feasible. In the special cases of tight or unusual conditions, it may not be practical to meet even these guide values. In all cases every effort should be made to get the best possible alinement, grade, sight distance, and, above all, proper drainage that is consistent with the terrain, the development (present and anticipated), and the funds available.

Drainage is most important, both on the pavement itself, as well as side and underdrainage. Unless this is adequately taken care of, maintenance costs will be unduly high. In snow country, it is necessary to design so that there is a place for plowed snow and the proper drainage to take care of it as it melts.

Safety is an important factor in all roadway improvements. However, on low volume roads, it may not be possible to get the obstacle-free roadsides that are desirable. Every effort should be made to provide as much clear roadside as is practical. The use of flatter slopes, the use of guardrail, and the judicious use of warning signs will help to achieve roadside safety. Utility poles should be kept close to the right-of-way line and in no case closer than the ditch line or well back of a curb.

For additional guidance and design of local roads and streets with higher volumes of traffic, reference should be made to "Geometric Design Standards for Highways Other Than Freeways," 1969, and "A Policy on Geometric Design of Rural Highways," 1965.

PART I - RURAL

Design Traffic Volumes

Highways shall be designed for specific traffic volumes as determined by accepted procedures. The design hourly volume, DHV, or average daily traffic, ADT, either current or projected to some future design year, will be the basis for design. Usually, the future design year is about 20 years from the date of completion of construction, but may be anywhere within a range of 10 to 25 years.

Design Speed

Geometric design features shall be consistent with a design speed selected as appropriate for conditions. Low design speeds are generally applicable to highways with winding alinement in rolling or mountainous terrain or where environmental conditions dictate. High design speeds are generally applicable to highways in level terrain, or where other environmental conditions are favorable.

Table 1
MINIMUM DESIGN SPEEDS

Type of Terrain	Minimum Design Speeds in MPH for Design Volumes of					
	Current ADT* Under 50	Current ADT 50-250	Current ADT 250-400	Current ADT 400-750 DHV 100-200	DHV* 200-400	DHV 400 and Over
Level	40	40	50	50	50	50
Rolling	30	30	40	40	40	40
Mountainous	20	20	20	30	30	30

*Note: *Current ADT is the annual average daily traffic expected after completion. DHV is the design hourly volume for the future design year, normally the 30th highest hourly volume about 20 years after completion.*

Level terrain is that condition where highway sight distances, as governed by both horizontal and vertical restrictions, are generally long or could be made to be so without construction difficulty or major expense.

Rolling terrain is that condition where the natural slopes consistently rise above and fall below the highway grade line and where occasional steep slopes offer some restriction to normal highway horizontal and vertical alinement.

Mountainous terrain is that condition where longitudinal and transverse changes in the elevation of the ground with respect to a highway are abrupt and where the roadbed is obtained by frequent benching or side hill excavation.

Terrain classification pertains to the general character of the specific route corridor. Roads in valleys or passes of mountainous areas that have all the characteristics of roads traversing level or rolling terrain should be classified as level or rolling. In general, rolling terrain conditions cause trucks to reduce to speeds below those of passenger cars on some sections of highway and mountainous terrain causes some truck operation at crawl speeds.

Sight Distance

Minimum stopping sight distance and passing sight distance shall be as shown in Table 2. Criteria for measuring sight distance, both vertical and horizontal, are as follows: for stopping sight distance, height of eye, 3.75 feet and height of object 0.5 foot; for passing sight distance, height of eye, 3.75 feet and height of object, 4.5 feet.

Table 2

MINIMUM SIGHT DISTANCES IN FEET

Design Speed, mph	20	30	40	50	60
Stopping sight distance:					
Minimum Stopping Sight Distance, feet	150	200	275	350	475
K value for:*					
Crest vertical curve	16	28	55	85	160
Sag vertical curve	24	35	55	75	105
Desirable Stopping Sight Distance, feet	150	200	300	450	650
K value for:*					
Crest vertical curve	16	28	65	145	300
Sag vertical curve	24	35	60	100	155
Passing sight distance:					
Passing distance, feet					
2-lane		1100	1500	1800	2100
K value for:*					
Crest vertical curve		365	686	985	1340

*NOTE: K value is a coefficient by which the algebraic difference in grade may be multiplied to determine the length in feet of the vertical curve which will provide minimum sight distance.

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Table 3

MAXIMUM GRADES

Type of Terrain	Design Speed MPH				
	20	30	40	50	60
Flat	7	7	7	6	5
Rolling	10	9	8	7	6
Mountainous	12	10	10	9	

Note: For highways with ADTs below 250, grades of relatively short lengths may be increased to 150 per cent of the value shown.

Grades

Maximum grades shall be shown in Table 3.

Alinement

Alinement between control points should be to as high a standard as is commensurate with the topography, terrain, the design traffic, and the reasonably obtainable right-of-way. Sudden changes between curves of widely different radii or between long tangents and sharp curves should be avoided. Insofar as feasible, the design should embody frequent passing opportunities. Where crest vertical curves and horizontal curves occur at the same location, there should be above-minimum sight distance design to assure that the horizontal curve is visible as drivers approach.

Depending on the maximum superelevation value, the maximum curvature for different design speeds is to be as shown in Table 4.

Table 4

MAXIMUM DEGREE OF CURVE AND MINIMUM RADIUS FOR DIFFERENT VALUES OF MAXIMUM SUPERELEVATION

Design Speed	Maximum e*	Minimum Radius (Rounded)	Max. Degree of Curve (Rounded)
MPH		Feet	Degrees
20	.06	115	50.0
30	.06	275	21.0
40	.06	510	11.5
50	.06	830	7.0
60	.06	1260	4.5
20	.08	110	53.5
30	.08	250	23.0
40	.08	460	12.5
50	.08	760	7.5
60	.08	1140	5.0
20	.10	100	58.0
30	.10	230	25.0
40	.10	430	13.5
50	.10	690	8.5
60	.10	1040	5.5
20	.12	95	62.5
30	.12	215	26.5
40	.12	400	14.5
50	.12	640	9.0
60	.12	960	6.0

**Note: e = rate of roadway superelevation, foot per foot*

Traveled Way Crown

Pavement or surfacing crown shall be adequate to provide proper drainage. Normally, cross slopes should be as shown in Table 5.

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Table 5

NORMAL PAVEMENT OR SURFACING CROSS SLOPES

Surface Type	Range in Rate of Cross Slope	
	Inch Per Foot	Foot Per Foot
High	1/8 - 1/4	.01 - .02
Intermediate	3/16 - 3/8	.015 - .03
Low	1/4 - 1/2	.02 - .04

High type pavements are those that retain smooth riding qualities and good nonskid properties in all weather under heavy traffic volumes and loadings, with little maintenance.

Intermediate type pavements are those designed to retain smooth riding qualities and good nonskid properties in all weather, but under lighter loads and lesser traffic volumes.

Low type surfaces are those with surface treated earth surfaces and those with loose surfaces such as shell and gravel.

Superelevation

For rural highways, superelevation should be not more than 0.12 foot per foot, except that, where snow and ice conditions prevail, the superelevation should be not more than 0.08 foot per foot.

Superelevation runoff is the length of highway needed to accomplish the change in cross slope from a normal crown section to a fully superelevated section. Minimum lengths of runoff are shown in Table 6. Adjustments in design runoff lengths may be necessary for smooth riding, surface drainage, and good appearance.

Table 6

MINIMUM LENGTH FOR SUPERELEVATION RUNOFF FOR 2-LANE PAVEMENTS

Superelevation Rate	L—Length of runoff in feet for design speed, MPH, of:				
	20	30	40	50	60
Foot per foot					
.02	50	100	125	150	175
..04	50	100	125	150	175
.06	50	110	125	150	175
.08	50	145	170	190	215
.10	50	180	210	240	270
.12	50	215	250	290	325

Number of Lanes

The number of lanes shall be sufficient to accommodate the design volume. For rural local roads, normally capacity conditions do not govern and two lanes are appropriate. Where more than two lanes are warranted to accommodate design traffic, determinations are to be made as indicated in "Design Standards for Highways Other Than Freeways."*

Width of Surfacing, Shoulder and Roadway

Minimum width of surfacing and of graded shoulder for various traffic volumes and design speed shall be as shown in Table 7.

Graded shoulder width is measured from the edge of surfacing (pavement) to the point of intersection of shoulder slope and side slope. Where a guardrail is used, the graded width of shoulder should be increased by about 2 feet. In mountainous terrain or sections with heavy earthwork, the graded width of shoulder in cuts may be decreased 2 feet.

The minimum roadway width is the direct sum of the surfacing and graded shoulder widths shown above. Desirable design widths include lanes at least 11 feet wide and shoulders 2 feet wider than the minimum shown in Table 7.

Table 7

MINIMUM WIDTH OF SURFACING AND GRADED SHOULDER

Design Speed, MPH	Width in Feet for Design Volume of:					
	Current ADT Less Than 50	Current ADT 50-250	Current ADT 250-400	Current ADT 400-750 DHV 100-200	DHV 200-400	DHV 400 & Over
	Width of Surfacing					
20	20	20	20	20	22	24
30	20	20	20	20	22	24
40	20	20	20	22	22	24
50	20	20	20	22	24	24
60	20	20	22	22	24	24
	Width of Graded Shoulder					
All	2	4	4	6	8	8

Note: Design volume in terms of mixed traffic. For design speeds of 50 mph or less, surfacing widths that are two feet narrower may be used on minor roads with few trucks.

Structures

The structural design of bridges, culverts, walls, tunnels, and other structures shall be in accord with Standard Specifications for Highway Bridges, 1969**.

**Available from American Association of State Highway Officials, 341 National Press Building, Washington, D.C. 20004 at 50¢ per copy.*

***Available from American Association of State Highway Officials, 341 National Press Building, Washington, D.C. 20004 at $9.00 per copy.*

Except as otherwise indicated herein, the dimensional design of structures shall also be in accord with this publication.

The minimum design loading for bridges shall be H-15 on local roads with current ADT volumes under 400. Where these volumes are 400 or greater, the minimum design loading shall be HS-20.

The clear roadway widths for new and reconstructed bridges shall be as shown in Table 8.

Table 8

CLEAR ROADWAY WIDTHS FOR NEW AND RECONSTRUCTED BRIDGES

Design Speed	ADT Volume	Minimum Clear Roadway Width of Bridge
50 MPH and over	750 or Greater	Approach Roadway Width
50 MPH and over	Under 750	Pavement Width + 6'
Under 50 MPH	400 or Greater	Pavement Width + 6'
Under 50 MPH	Under 400	Pavement Width + 4'

Notes: (1) *Where the approach roadway is surfaced for the full crown width that surfaced width should be carried across structures.*
(2) *On highways with a current ADT over 750, bridges with a total length over 100 feet may be constructed with a minimum clear roadway width of the surfacing width plus six feet.*

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Bridges to Remain in Place

Since highway geometric and roadway improvements encourage higher speeds and attract larger vehicles to the highway, it is important that existing substandard structures also be improved correspondingly. Because of their high cost, reasonably adequate bridges and culverts that meet tolerable criteria should be retained.

Where an existing highway is to be reconstructed, an existing bridge which fits the proposed alinement and profile may remain in place when its structural capacity in terms of design loading and clear roadway width are at least equal to the values shown for the applicable traffic volume in Table 9.

The values in Table 9 do not apply to structures with a total length greater than 100 feet. These structures should be analyzed individually, taking into consideration the clear width provided, traffic volumes, remaining life of the structure, design speed, as well as other pertinent factors.

Vertical Clearance

Vertical clearance at underpasses should be at least 14 feet over the entire roadway width, to which an allowance of 4 to 6 inches should be added for resurfacing.

Table 9

MINIMUM STRUCTURAL CAPACITIES AND MINIMUM ROADWAY WIDTHS FOR BRIDGES TO REMAIN IN PLACE

Traffic		Design Loading Structural Capacity		Roadway Clear Width Feet(1)	
Current ADT	DHV	Desirable Minimum	Minimum	Desirable Minimum	Minimum(2)
0-50	-	H-15	H-10	24	20
50-250	-	H-15	H-15	26	20
250-400	-	H-15	H-15	28	22
400-750	100-200	H-15	H-15	28	22
	200-400	HS-15	H-15	32	24
	Over 400	H-20	H-15	36	30

Notes: *(1) Clear width between curbs or rails, whichever is the lesser.*
(2) For design speeds of 50 mph or less, minimum clear widths that are two feet narrower may be used on minor roads with few trucks. In no case shall the minimum clear width be less than the approach surfacing width.

Right-of-Way Width

The procurement of rights-of-way to such widths as will accommodate the construction, adequate drainage and proper maintenance of a highway is a most important part of the overall design. Wide rights-of-way permit the construction of gentle slopes, resulting in more safety for the motorist and providing for easier and more economical maintenance. The procurement of sufficient rights-of-way at the time of the initial improvement permits the widening of the roadway and the widening and strengthening of the pavement at reasonable costs as traffic increases.

In developed areas, it may be more feasible to limit the right-of-way width to a practical minimum. In every case, however, the right-of-way should be not less than that required for all elements of the design cross section and appropriate border areas.

Side Slopes

The maximum rate of side slope will depend on the stability of local soils as determined by local experience. Slopes should be as flat as feasible and other factors should be considered to determine the design slope. Flat side slopes increase safety by providing maneuver area in emergencies, are more stable than steep slopes, aid in the establishment of plant growth, and simplify maintenance work. The intersection of slope planes should be well rounded.

Drivers who inadvertently leave the traveled way can often recover control of their vehicles if fill slopes and inslopes (see Figure 1) are gentle and drainage ditches are well rounded. Such recovery areas should be provided where terrain and right-of-way controls permit. Desirably, these recovery areas

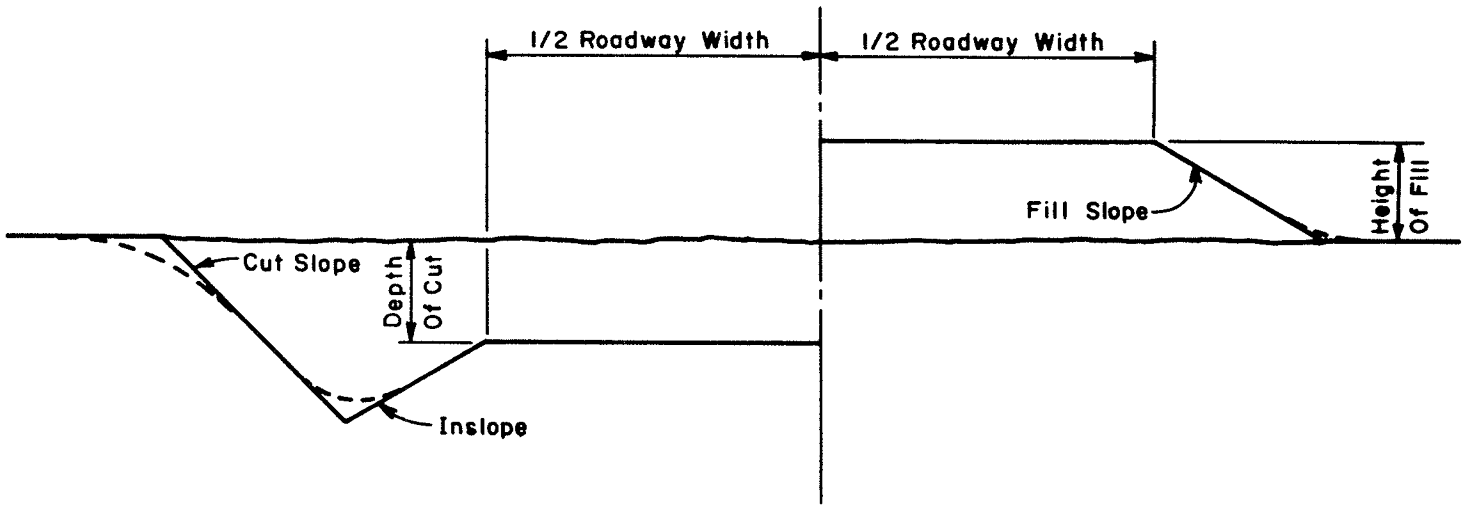


Figure 1

SLOPE TERMS
USED ON CUT AND FILL SECTIONS

should extend 10 to 20 feet beyond the outer edge of the shoulder, particularly on roads with a design speed of 50 mph or higher and ADT volumes of 750 or more.

Where this is not practical, effort should be made to provide combinations of rate and height of slope such that occupants of an out-of-control vehicle have a good chance for survival. Where controlling conditions (such as high fills, right-of-way restrictions, or the presence of rocks, watercourses or other hazards) make this infeasible, consideration should be given to the provision of guardrail, in which case use could be made of maximum rate of fill slope beyond.

Cut sections should be designed with adequate ditches. Preferably, the inslope should not be steeper than 4:1. The ditch bottom and slopes should be well rounded. The cut slope should not exceed the maximum required for stability.

Horizontal Clearances to Obstructions

On a local road with an ADT volume of 750 or more and with a design speed of 50 mph or greater, a clear roadside recovery area should be provided, preferably 10 to 20 feet or more outside the shoulder. Where the design speed is less than 50 mph, or ADT is less than 750, a clear roadside recovery area should be provided, preferably 10 to 15 feet or more from the edge of the through traffic lane. The clear roadside area should be an appropriate flat and rounded cross-section design. Exception may be made (1) in cut sections where fixed objects are located sufficiently up the cut slope so that there is little likelihood that they would be struck and (2) where guardrail protection is provided. The recovery area should be clear of all unyielding objects such as trees or unyielding sign supports, utility poles, unyielding light poles and any other fixed object that might severely damage an out-of-control vehicle.

To the extent feasible, where another highway or railroad passes over, the structure should be designed so that the pier or abutment supports have lateral clearance as great as the clear roadside area on the approach roadway.

Where it is not feasible to carry the approach roadway across an overpass or other bridge, an appropriately transitioned guardrail should be provided and securely anchored to the bridge rail. At hazardous locations, such as the outside of a sharp curve on low fill, where guardrail is not provided, a much greater horizontal clearance should be provided to any roadside obstruction.

Intersection Design

The location of intersections should be carefully selected to avoid steep profile grades and to insure that there is adequate approach sight distance to the intersection. An intersection should not be located on a short crest vertical curve, just beyond a short crest vertical curve, or on a sharp horizontal curve. Where there is no practical alternate to such a location, the approach sight distance on each leg should be checked carefully. Where necessary, cut slopes should be flattened and horizontal or vertical curves lengthened to provide additional sight distance. There should be sufficient sight distance to permit a passenger vehicle on the minor leg of the intersection to cross the traveled way without requiring the through approaching traffic to slow down. As a general rule, there should be a minimum of 6 to 7 seconds available to the driver crossing the through lanes. On this basis, the suggested corner sight distance for each design speed would be as follows:

Table 10

SUGGESTED CORNER SIGHT DISTANCE AT INTERSECTIONS

Design Speed, MPH	Minimum Corner Intersection Sight Distance, In Feet*
60	600
50	500
40	400
30	300
20	200

**Note:* *Corner sight distance measured from a point on the minor road at least 15 feet from the edge of the major road pavement and measured from a height of eye of 3.75 feet on the minor road to a height of object of 4.5 feet on the major road. See Figure VIII-5, Page 398, "A Policy on Geometric Design of Rural Highways".*

Intersections should be designed with a corner radius of the pavement or surfacing that is adequate for the larger vehicles anticipated; usually, a minimum edge radius of 50 feet is applicable. Where turning volumes are significant, consideration should be given to speed-change lanes and channelization.

Intersection legs that will operate under STOP control preferably should be at right angles.

Traffic Control Devices

Signs, pavement and other markings and, where pertinent traffic signal controls, are essential elements for all highways. Reference is made to the Manual on Uniform Traffic Control Devices for details of the devices to be used and, for some conditions, warrants for their use.

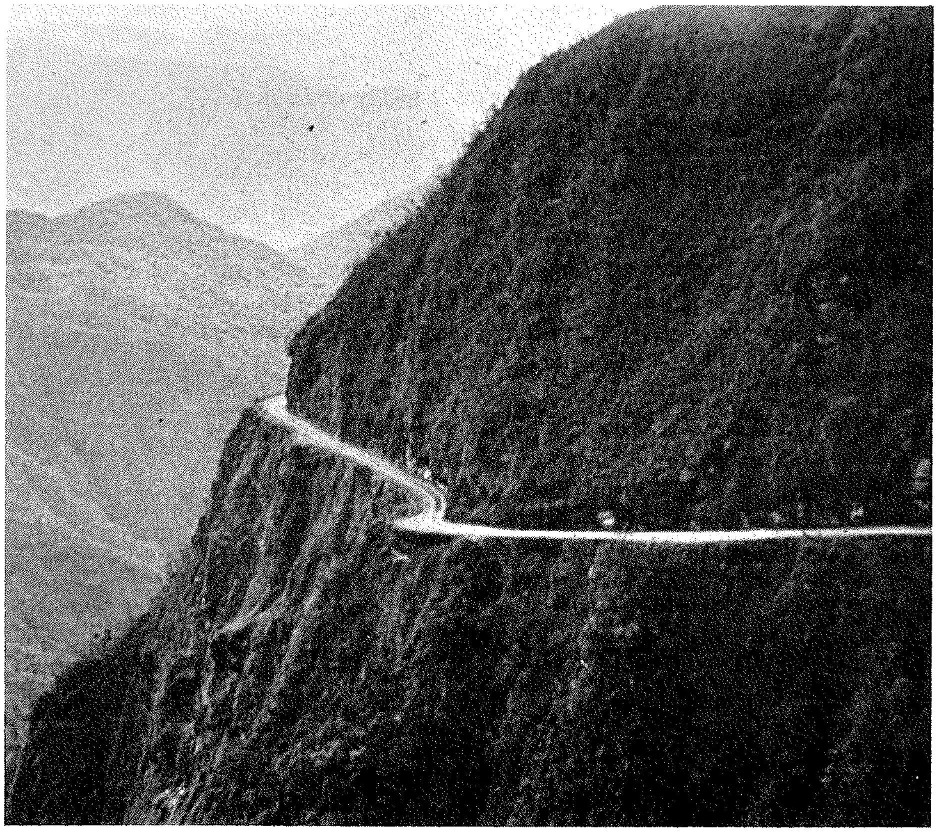
A route numbering or lettering system for local roads is highly desirable for non-local travelers and simplifies essential signing.

Erosion Control

All slopes and drainage areas should be designed with proper regard for the desired natural ground cover and growth regeneration on areas opened during construction. Seeding and mulching of slopes and the sodding of swales and other erodable areas should be a part of the local road design.

In roadside design, attention is to be given to the preservation of natural ground covers and desirable growth of shrubs and trees within the right-of-way. While specific design determinations are needed, such steps to mold the cross section to fit the areas to be preserved contribute much to maintenance economy and the refinement of the whole highway.

NOTE: The omitted text refers to urban roads and is outside the scope of this compendium.



Yolosa Road, Bolivia.

Frederick W. Cron

A Review of Highway Design Practices in Developing Countries

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Foreword

This review is a sequel to the World Bank's previous *Guide to Highway Design Standards* dated June 1957, which has been out of print for a number of years. This guide proved so popular, and the requests for a revised guide so numerous, that the Bank believes a further publication to be justified. The new publication is essentially a review of the design standards of some 150 highway projects financed by the Bank between 1960 and 1970.

The collaboration of the Federal Highway Administration of the U.S. Department of Transportation in providing the services of highly competent staff for the basic research, and for commenting on the draft study, is gratefully acknowledged. The author, Mr. Frederick W. Cron, has been a highway engineer in the U.S. Federal Highway Administration for more than 40 years, and draws mainly on his wide experience in the United States when commenting on the design practices followed in developing countries. Readers will appreciate that the circumstances in their own countries may not always be comparable to those in the United States, and an element of judgment by the highway design engineer may often be necessary in the light of specific circumstances.

The Bank has already published the first[1] of a series of reports on a comprehensive highway design study which should enable the design engineer to exercise his judgment on standards in a somewhat more rational manner than before; further sections are to be issued shortly. The report assesses the interrelationship between highway construction costs, vehicle operating costs and road maintenance costs. Adequate input data on traffic, soil

[1] See F. Moavenzadeh, et al. "Highway Design Standards Study, Phase I: The Model," *IBRD Staff Working Paper No. 96* (January 1971). This model is currently being revised in the light of results derived in Phase II of the study, which encompassed over two years of field research in Kenya by the UK Transport and Road Research Laboratory in conjunction with the Bank. Similar field research is being planned in India and Brazil.

strengths and costs should enable the designer to select an appropriate highway construction standard with the objective of minimizing total transportation cost.

Mr. Cron's review of design standards, as presented here, should nevertheless continue to prove useful as an interim design guide, and in cases where use of the highway design study model may not be appropriate.

S. C. HARDY
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ANNEXES

Preface

Practice in highway design evolves slowly over a long period, and crystallizes into formal "standards" only after a strong consensus has developed among informed engineers and administrators. This evolution proceeds at different rates in different countries so that at any particular time the highway standards for a large group of countries may cover a rather wide range of values for substantially the same traffic conditions. This variability became obvious early in this study and was one of its most interesting findings.

Despite the variations, there is an area of agreement among the standards of the sixty-three countries studied. In this report I have undertaken to sketch the bounds of this middle ground, and to suggest practical highway standards lying within these bounds for the guidance of planners in developing areas.

The guidance and advice of the Advisory Committee of distinguished highway engineers from the World Bank and the U.S. Department of Transportation is gratefully acknowledged. In particular I am indebted to Mr. S. C. Hardy of the World Bank, who was manager for the study, for his painstaking review of the report and for many excellent suggestions on the presentation of the findings.

To Mrs. Patricia J. Krohn I owe a special debt of gratitude for her plotting and analysis of the graphs, and also for typing the manuscript. For the very capable drafting of the graphs I thank Mr. William E. Krohn. Finally I wish to acknowledge the assistance of the Data Processing Branch, Region 8, FHWA for processing the large quantities of data generated by the study. This work included the writing of three special programs.

FREDERICK W. CRON
Lakewood, Colorado

I. A Sample of Highway Standards in Developing Countries

In the past twelve years a large proportion of the World Bank's [1] lending has been for transportation projects, and particularly for highways. These loans have given the Bank a unique opportunity to review the standards of highway design in a number of countries and, possibly, to determine whether there is a consensus among them regarding the principal design elements. This study examines these elements as they were applied in a large sample of the roads that have been built in developing countries in recent years.

When the World Bank receives an application from a member country for a highway loan a mission is sent to that country to evaluate the proposed project and recommend whether it is a suitable basis for a loan. The mission usually consists of a highway engineer and a transportation economist, who consider all aspects of the proposed project—engineering, economic, social, financial, and administrative—before making their recommendation to the Bank. Construction standards for the project are an important part of the data upon which the mission bases its recommendation, and usually are the same standards as those already in use by the borrowing country for other highways of the same type. Occasionally, however, existing standards may be modified for the specific project under consideration.

[1] Any reference to the "World Bank" or "Bank" in this publication includes both the "International Bank for Reconstruction and Development" and the "International Development Association."

The Basic Data for the Study

Between 1960 and 1970 the Bank sent over 150 missions to 63 countries to evaluate applications for highway projects. Most resulted in loans for design, maintenance or construction projects. The Mission Appraisal Reports for the construction projects are the sources of the information on standards used in this study. These reports identified 855 segments of highways totaling 69,433 kilometers in length and estimated to cost US$4.557 billion in the aggregate. The types of construction range from single-lane earth roads to six-lane elevated expressways, and from pedestrian suspension bridges to major toll bridges.

The basic unit is the *line item,* this being the smallest segment of road (or an isolated bridge in some cases) that can be identified in the appraisal reports. There are 855 line items, ranging in length from less than 1 kilometer up to 909 kilometers, but averaging about 85 kilometers. Ideally, a line item should represent a segment of road within which traffic and topography are reasonably uniform, but actually this is true only for the very short segments. The distribution among countries of the line items in the study is shown in Annex 1.

From the appraisal reports it is possible to identify the country, the termini, the length, the year of opening to traffic, the type of work (such as reconstruction, betterment, new construction), the functional classification (primary, secondary, feeder), the surfacing type, the width of surfacing and shoulder, the average annual daily traffic, the traffic growth rate and the estimated construction cost for every line item.

Generally, the estimated traffic on date of opening is given in the reports. In the few cases where it is not given it is possible to compute the traffic on opening by applying a growth rate to previous traffic counts.

From the reported information, a complete project-by-project analysis can be made of pavement type, width of surfacing and width of shoulders as these elements relate to opening traffic. Information on other highway elements such as design speed, sight distance, superelevation, and gradient is fragmentary or incomplete for many projects. We may nevertheless get some idea of prevailing design practice for these elements by examining the country standards, independently of their application to specific projects. (Table 1.1 is a good example of the information available in the country standards.) Here, however, we are again confronted with fragmentary information, as very few appraisal reports contain complete country standards such as are shown in Table 1.1. Most have only extracts from the country standards of the essential information applying to the project roads. Thus, we may have complete information on a particular road which will be built in flat terrain for an estimated traffic volume of 500 vehicles per day, but no information on the standards that would be used in that country for other terrains and different traffic volumes.

The Problem of Classifying Highway Standards

Road standards of different countries are often not set up in the same way and therefore cannot be directly compared without some rearrangement or processing. The most obvious differences are between countries using the English system of measures and those using the metric system. But there are many other differences.

For example, some countries apply road standards according to the *administrative* class of the road, as national, provincial, or local; while others use *function* (expressways, or primary, secondary and tertiary roads). There is considerable variation among countries in the number of design classes and in the traffic ranges for each class. A few use the terms light, medium, and heavy traffic without delimiting the actual volumes. A further complication: in some countries the traffic is expressed in equivalent passenger car units
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while in others it may be in truck units. Most countries, however, use average annual daily traffic of mixed vehicles: (*ADT*).[2] Some countries have only two brackets for topography—flat and rolling, and mountainous; while some have five. By far the largest number use three: flat, rolling, mountainous. Terms used to describe surfacing types differ from country to country.

With such a large number of variables some simplifying and grouping must be done before a meaningful analysis can be made. *Administrative* class, for example, does not appear to be significant as a classifying factor, as appraisal reports reflect few cases where a road was built to a higher standard than was justified by the potential traffic volume, merely because of its administrative classification.

Functional Classification

Functional class, on the other hand, appears to be useful for dividing the large number of roads in the sample into broad

[2] Average Annual Daily Traffic volume, sometimes abbreviated *AADT*. This is the total number of vehicles passing a given point in one year, divided by 365.

Table 1.1: Colombia: Sixth Highway Project, Ministry of Public Works—Highway Design Standards

Classification		Light Traffic								Medium Traffic								Heavy Traffic					
		LT-1				LT-2				MT-3				MT-4				HT-5			HT-6		
ADT		Up to 250				250-500				500-1000				1000-2000				2000-5000			Over 5000		
Terrain	—	H	M	R	F	H	M	R	F	H	M	R	F	H	M	R	F	H/M	R	F	H/M	R	F
Design speed	Kph	40	50	60	70	40	50	60	70	40	60	80	100	40	60	80	100	60/80	80/100	100/120	60/ 80	80/100	100/120
Width of pavement	m	6.00				6.00				7.00				7.00				7.00			2 of 7.00		
Shoulders																							
Left	m	050	1.00	1.00	1.50	1.00	1.00	1.00	1.50	1.00	1.00	1.50	2.00	1.50	1.50	2.00	2.50	2.50	2.50	3.00	1.00	1.00	1.50
Right	m	050	1.00	1.00	1.50	1.00	1.00	1.00	1.50	1.00	1.00	1.50	2.00	1.50	1.50	2.00	2.50	2.50	2.50	3.00	2.50	3.00	3.00
Total width	m	7	8	8	9	8	8	8	9	9	9	10	11	10	10	11	12	12	12	13	10.50	11	11.50
Maximum slope (gradient)	%	8	7	6	5	7	6	5	4	6	5	4	3	6	5	4	3	5	4	3	5	4	3
Minimum radius of curves	m	50	80	120	170	50	80	120	170	50	120	250	450	50	120	250	450	120/250	250/450	450/750	120/250	250/450	450/750
Non-passing sight distance	m	50	70	80	100	50	70	80	100	50	80	120	180	50	80	120	180	80/120	120/180	180/250	80/120	120/180	180/250
Passing sight distance	m	200	300	350	450	200	300	350	450	200	350	500	600	200	350	500	600	350/500	500/600	600/800	—	—	—
Width of bridges < 40	m	7		8		Same as total width																	
Between curbs > 40	m					8				9				9				10			Two of 9 each		
Design loading for bridges										H20-SI6-44													

Notes: H—Heavy mountainous.
M—Mountainous.
R—Rolling.
F—Flat.

Source: Ministry of Public Works, December 1969 categories of somewhat similar characteristics. Thus, at the top one can recognize a category of high-traffic-volume facilities, few in number and limited to a few countries, while at the bottom there are the tertiaries and special-purpose roads, the design of which is dominated not by actual or potential traffic, but by the fact that there are certain minimum dimensions below which one has no road at all. In between is a large group of ordinary highways, most of which are two-lane, serving both long distance and local traffic. Some general conclusions may be drawn from analyzing their characteristics. Most of these roads are classified as primary and secondary in the appraisal reports, but for the purpose of this study we can lump them together according to the traffic they carry. In the developing countries as in the more developed, a high-volume secondary road may carry more traffic than a low-volume primary road.

Traffic Classification

Traffic is the primary determinant for road standards, and for expressing standards in terms of traffic the *design class* system appears to be the most practical yet devised. This study analyzes standards according to five design classes for different volumes of expected traffic per day on opening:

Class I	Under 50 vehicles
Class II	50 to 400 vehicles
Class III	400 to 1,000 vehicles
Class IV	1,000 to 2,000 vehicles
Class V	Over 2,000 vehicles

Traffic is expressed as the ADT of mixed vehicles; and where other units such as equivalent passenger cars, appear in the appraisal reports these were converted to ADT of mixed vehicles before use in the study.

Terrain Classification

Three standard terms are used for topography, defined as follows:

Level terrain is that condition where highway sight distances, as governed by both horizontal and vertical restrictions, are generally long or could be made to be so without construction difficulty or major expense.

Rolling terrain is that condition where the natural slopes consistently rise above and fall below the highway grade line and where occasional steep slopes offer some restriction to normal highway horizontal and vertical alignment.

Mountainous terrain is that condition where longitudinal and transverse changes in the elevation of the ground with respect to a highway are abrupt and where the roadbed is obtained by frequent benching or side hill excavation.

Surfacing Classification

For the purpose of this study road surfaces are classified according to seven general types:

- Earth graded roads, unsurfaced
- Granular surfaces such as gravel, crushed stone, laterite and stabilized soil without surface treatment
- Prime and single surface treatment using liquid asphalt, tar or bituminous emulsion and surface aggregate or chippings
- Prime and double surface treatment
- Prime and surface treatments of three or more applications; also penetration macadam or bituminous macadam
- Road mix
- Asphaltic concrete or portland cement concrete

Design Elements

The data in the appraisal reports are sufficient for an analysis of 16 highway design elements, as listed below. For widths of surfacing, shoulder and roadway, and for type of surfacing, 62 countries in the sample are represented; but for the other elements the coverage is less complete.

Design Element	Countries Reported
Width of surfacing	62
Width of shoulder	62
Width of roadway	62
Type of surfacing	62
Design speed	55
Passing sight distance	13
Non-passing sight distance	30
Minimum radius	51
Maximum gradient	52
Pavement crown or camber	13
Superelevation	9
Shoulder slope	6
Width of bridge deck	24
Maximum axle load	37
Design live loading for bridges	46
Right-of-way width	25

Chapter II examines these elements in more detail and compares the values adopted for them in the country standards and the project road standards.

II. Comparison of Standards

The roads studied for this report fall into three broad functional categories:

I. A small group of expressways, freeways and toll roads carrying large traffic volumes.

II. A very large group of ordinary two-lane highways carrying a wide range of traffic volumes serving both local and long distance traffic.

III. A smaller group of low-traffic tertiary or special purpose roads existing primarily for land service.

Category I: Expressways and Toll Roads

Since 1960 the World Bank has participated in the financing of 2,430 kilometers of highway facilities of more than two lanes:

	Kilometers
4-lane undivided highways	59
2-lane expressways (usually as the initial stage of an ultimate 4-lane facility)	635
4-lane expressways, grade separated	1,707
6-lane expressways, grade separated	4
4-lane elevated expressways, grade separated	25
Total	2,430

This is 3.4 percent of the kilometrage of the whole sample for this study.

All these roads except the four-lane undivided highways are access-controlled and a good proportion are toll facilities.

The estimated construction cost of these high-capacity facilities was US$1.692 billion, or 37.2 percent of the total cost of the whole sample. (A very substantial portion of this cost was for construction of freeways in Japan during the late 1950s and early 1960s.)

The rather fragmentary data on standards in the appraisal reports for these roads are summarized in Table 2.1:

Table 2.1: High, Median and Low Values for Various Design Elements of High-Capacity Facilities (Range of Traffic Volume on Opening: 5,200 ADT to 59,400 ADT)

	Flat	Rolling	Mountainous
Minimum Design Speed (14 Countries Reported)[a] km.p.h.			
High value	140	120	90
Median value	120	100	80
Low value	96	80	80
Minimum Radius (10 Countries Reported)[a]			
High value	1500 m.	1100 m.	300 m.
Median value	515 m.	400 m.	240 m.
Low value	292 m.	240 m.	120 m.
Minimum Non-Passing Sight Distance (9 Countries Reported)			
High value	350 m.	270 m.	140 m.
Median value	210 m.	183 m.	110 m.
Low value	190 m.	150 m.	100 m.
Maximum Gradient (12 Countries Reported) percent			
High value	4.0	5.0	6.0
Median value	3.2	4.0	5.0
Low value	2.0	3.0	4.0
Width of One Traffic Lane (14 Countries Reported)			
High value	3.75 m. (without regard to terrain)		
Median value	3.60 m. (without regard to terrain)		
Low value	3.25 m. (without regard to terrain)		
Width of Outside Shoulder (14 Countries Reported)[a]			
High value	3.50 m. (without regard to terrain)		
Median value	3.00 m. (without regard to terrain)		
Low value	2.40 m. (without regard to terrain)		
Width of Inside Shoulder (5 Countries Reported)[a]			
High value	2.75 m. (without regard to terrain)		
Median value	1.50 m. (without regard to terrain)		
Low value	1.50 m. (without regard to terrain)		
Minimum Width of Median Divider, Excluding inside shoulder (4 Countries Reported)			
High value	11.0 m.		
Median value	3.0 m.		
Low value	0.8 m.		

[a] Elevated urban expressways were excluded from this comparison.

All these highways had heavy duty pavements, principally of asphaltic concrete. They were designed to support single-axle loadings ranging from 8.2 metric tons to 16.0 metric tons.

Right-of-way widths were available for only four countries. These ranged from a minimum of 40 meters (Finland) up to 80 meters (Thailand).

Bridge live loadings were available for six countries. Three of these were AASHO H20-S16, with one each to British Standard 153 Type HA, Indian Road Congress "AA" and German DIN 1072.

The median values outlined are very close to those for similar facilities operating under similar traffic conditions in the more developed countries.

Category II: Two-Lane Primary and Secondary All-Purpose Roads

Since 1960 the World Bank has participated in the financing of 69,433 kilometers of road improvements of all types including the high-capacity facilities described above. Grouping these roads according to the five design classes adopted for this study the following table results:

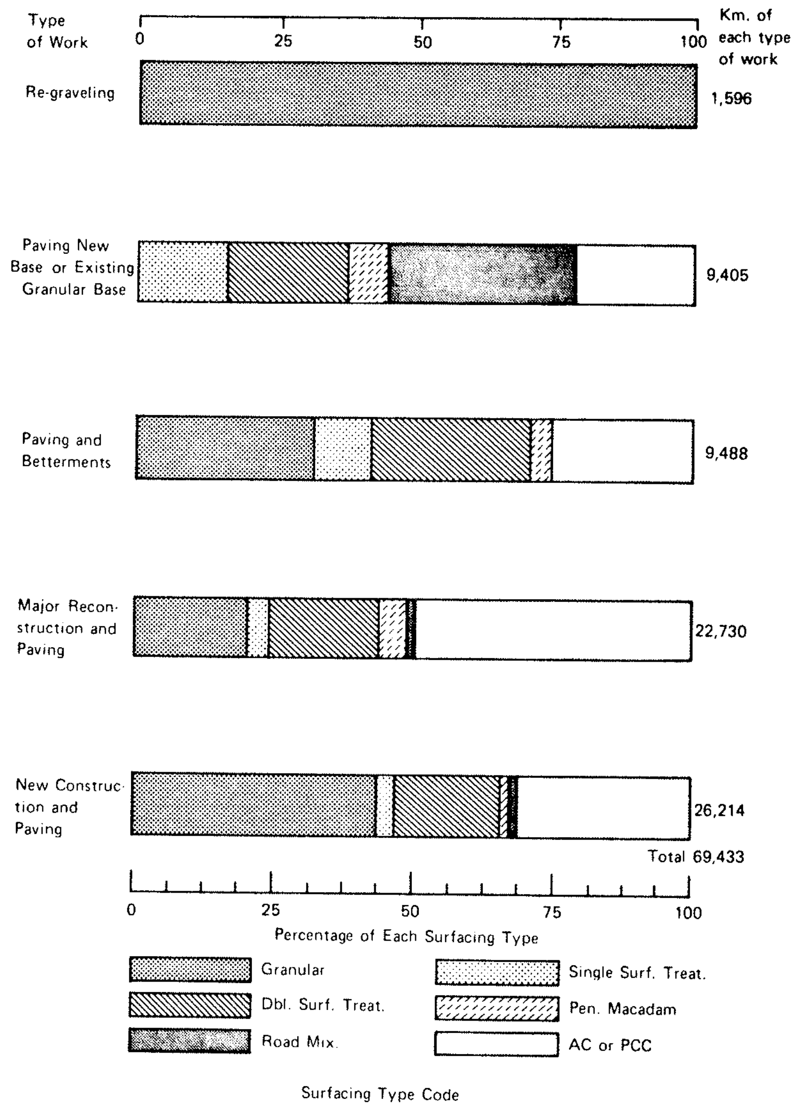
		Kilometers	Percent
Class I	Under 50 ADT	12,844	19
Class II	50 to 400 ADT	25,101	36
Class III	400 to 1,000 ADT	16,480	24
Class IV	1,000 to 2,000 ADT	6,984	10
Class V	Over 2,000 ADT	8,024	11
		69,433	100

It is interesting to note that almost 80 percent of the roads financed by the Bank carried less than 1,000 ADT on opening. About 38 percent of the total kilometrage was new construction, starting with completely new location, and most of it included some kind of dustless surface. From here, the improvements varied widely, but can be fitted into one of the five categories shown:

Type of Work	Length (Kilometers)	Percent
Replenishing the surface of existing gravel or granular-surfaced road	1,596	2
Paving placed on new base or existing granular-surfaced road	9,405	14
Re-paving of existing road accompanied by betterments or minor improvements in alignment or drainage	9,488	14
Reconstruction and paving	22,730	32
New construction and paving	26,214	38
Total	69,433	100

Figure 2.1 shows the distribution of this work by surfacing type.[a]

Figure 2.1: Distribution of Surfacing Types, by Types of Work



[a] Except for certain adjustments described in the text, Figures 2.1, 2.2, 2.3, 2.4, 2.6, 2.7, 2.10, 2.11 are based on the whole sample (Categories I, II and III), expressed in either line items or kilometers. Figures 2.5, 2.8, 2.9, 2.12, 2.13, 2.14, 2.15, 2.16 are based on the country standards irrespective of their application to specific projects.

Width of Surfacing [1]

The whole sample for this study consists of 855 line items totaling 69,433 kilometers. Of these, 31 items totaling 16 kilometers are for isolated bridge projects where the surfacing and shoulder widths of the approach roadways are not given in the data, and 3 items totaling 1,595 kilometers are regraveling projects of variable and unspecified width. For the remaining 821 line items definite surfacing and shoulder widths are available except for a few cases of granular-surfaced roads which are listed in the data as without shoulders. In these cases a shoulder of 0.5 meter on both sides is arbitrarily assumed, the rest of the width being considered as surfacing.

Figure 2.2 shows these 821 line items arranged in histogram form by design classes and 1/2-meter increments of surfacing width.

For Figure 2.3 the kilometers of road in the sample were arranged according to design classes and 1/2 meter increments of width.

These histograms show that the average widths of surfacing for the five design classes lie in the following ranges:

Design Class	ADT on Opening	Comparison by Line Items (Fig 2.2) (meters)	Comparison by Kilometers (Fig. 2.3) (meters)
Class I	Up to 50	4.0 to 4.5	4.5 to 5.0
Class II	50-400	5.5 to 6.0	5.5 to 6.0
Class III	400-1,000	6.0 to 6.5	6.5 to 7.0
Class IV	1,000-2,000	6.5 to 7.0	6.5 to 7.0
Class V	Over 2,000	6.5 to 7.0	7.0 to 7.5

Both histograms are affected to some degree by the fact that a few countries dominate the sample, and their design practices therefore have disproportionately greater weight in the analysis (see Annex 1). Nevertheless, these graphs, taken in conjunction with Figure 2.4, are useful indicators of the middle ground in width design in the 1960s and early 1970s.

The prudent designer will design a new road, or a major reconstruction of an old road, to somewhat higher standards than those required for current traffic in order to provide a "cushion" for future growth. For Bank-financed projects the amount of cushion thus provided is not stated in the appraisal reports but one can

[1] In this report the term "surfacing" is used for all road surfaces better than natural earth, including gravel, laterite, crushed stone, stabilized soil, surface treatments and road mix, plant mix and cement concrete pavements.

Figure 2.2:
Distribution of 821 Line Items by Increments of Surfacing Width

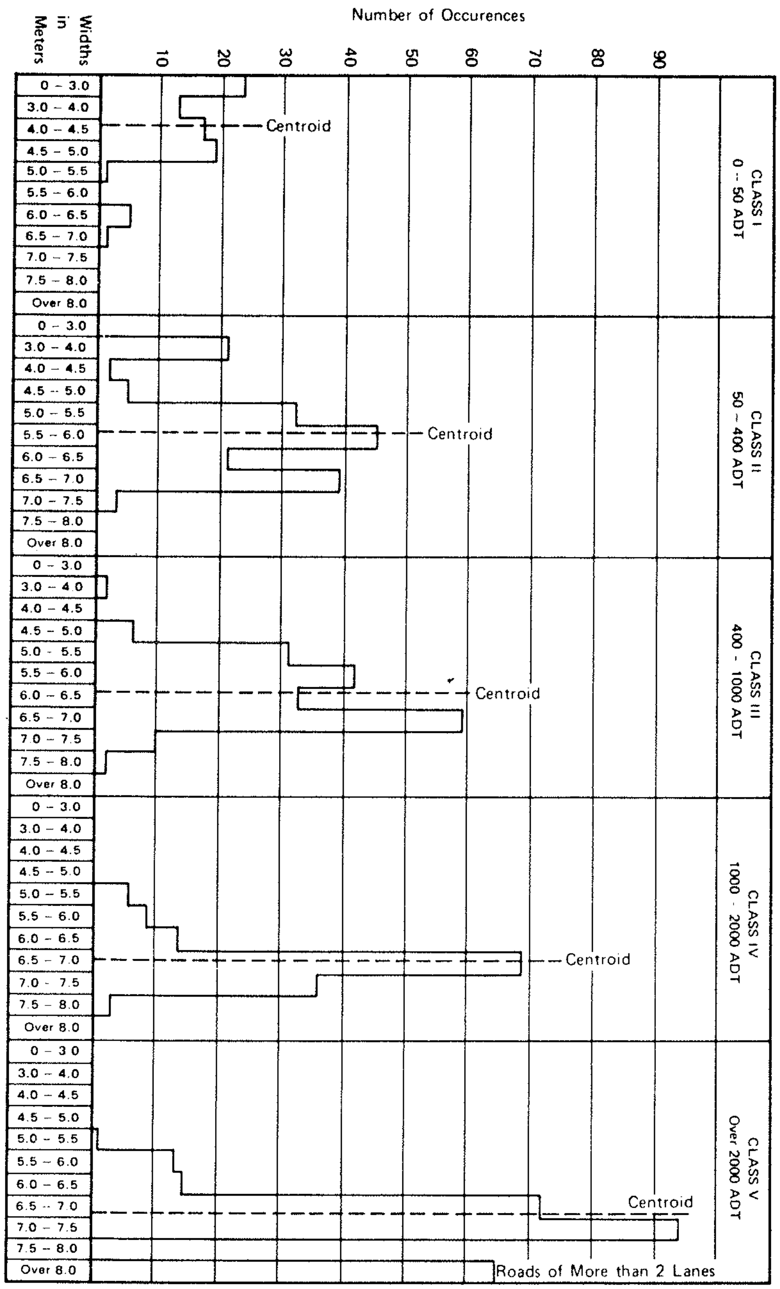


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Figure 2.3:
Distribution of Road Lengths by Increments of Surfacing Width

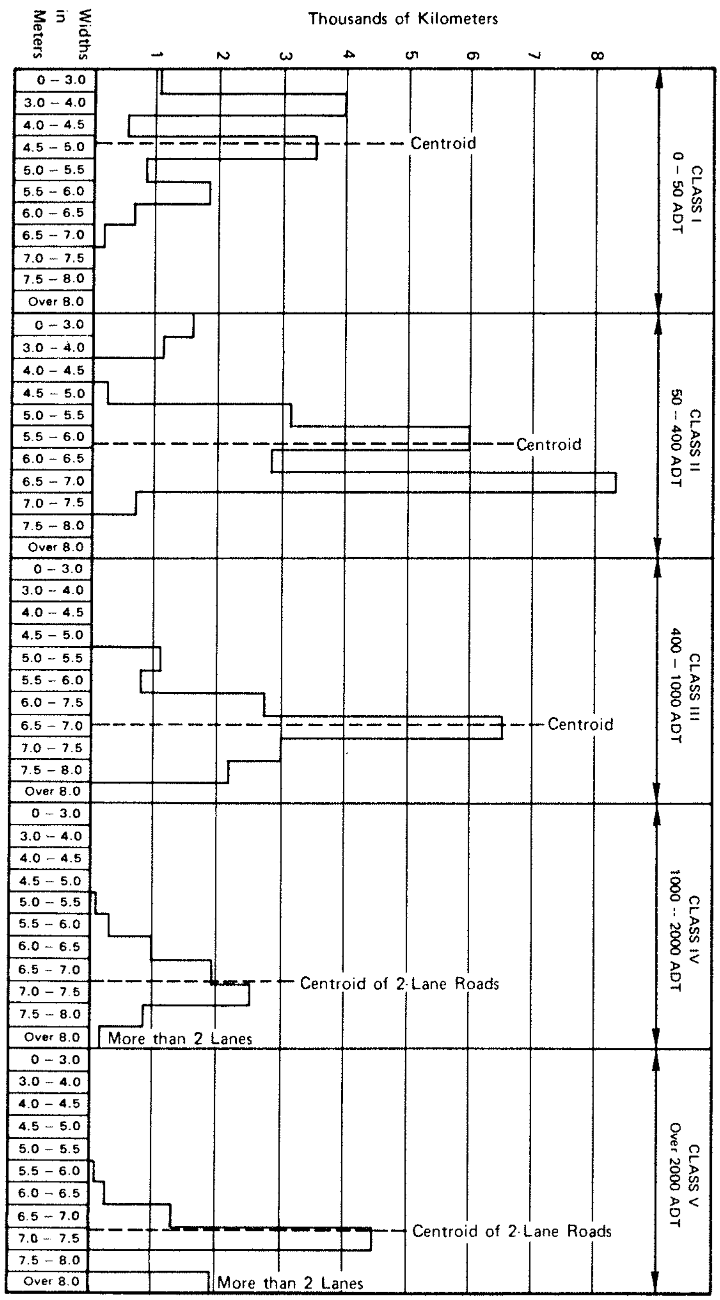

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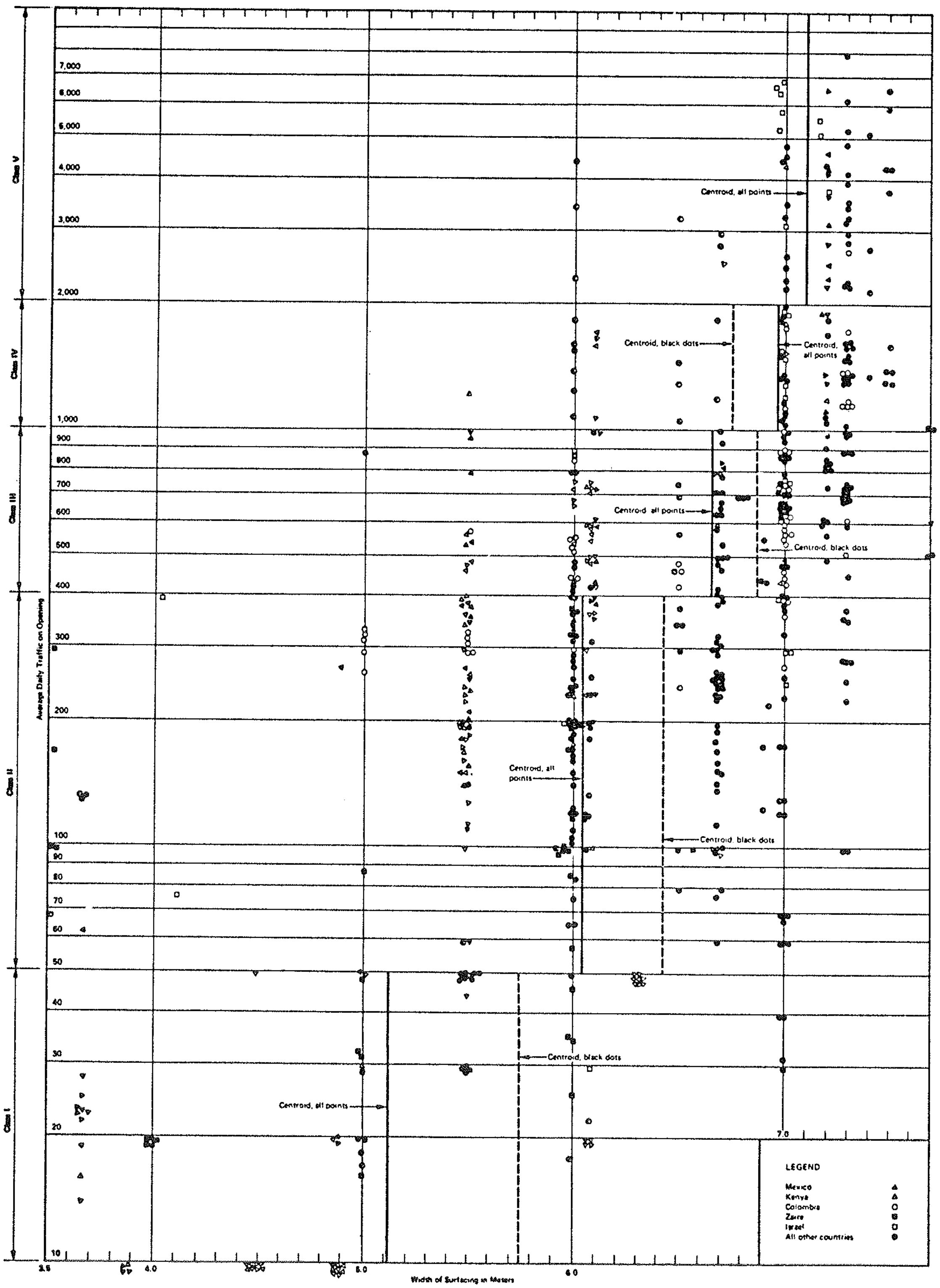
Figure 2.4: Surfacing Width versus Traffic on Opening for 657 Two-Lane Highway Segments



assume that it would be adequate to sustain two or three increments of traffic growth, during the economic life of the project which could be up to 20 years. In the analysis which follows it is assumed that the standards, although stated in terms of average daily traffic on opening, are intended to provide a reasonably adequate level of service life for about 7 to 10 years. A common practice, strongly supported by the Bank, is to follow the sound principle of stage construction by providing an initial facility that will handle the traffic expected in, say, the first 5 to 7 years, and still be capable of expansion to accommodate further traffic increase.

One cannot assume that a cushion for future growth exists for projects such as regraveling of existing roads. (Bar 1 of Figure 2.1) or paving of existing roads with betterments. (Bar 3 of Figure 2.1) These roads, of unknown age, may already have used up part of the cushion built into them when they were new. They are therefore omitted from the "scattergraph analysis" of Figure 2.4. Also omitted are the line items for roads of more than two traffic lanes and those for isolated bridge projects. After making the above adjustments there remain 657 line items totaling 55,904 kilometers located in 58 countries.

The modified sample, like the whole sample, is dominated by a few countries as shown below:

Country	Percent of Total Line Items in Modified Sample
Kenya	17.4
Mexico	10.0
Colombia	8.8
Zaire	4.7
Israel	4.2
All Others	54.9

To ascertain what effect this dominance may have on the result, the widths of projects in these countries are plotted with special symbols in Figure 2.4, to distinguish them from the others, which are plotted as solid black dots. In all cases the width of surfacing is plotted against ADT on date of opening. The centroids of the solid black dots falling within each design class are shown by dashed vertical lines and the centroids of *all* plotted points in each class, that is, the solid black dots plus the others, are shown by *solid* vertical lines.

Figure 2.4 shows that the average width in Class I is appreciably influenced by line items in Kenya and Zaire, the spread being 0.6 meter between the centroid with and without these items. This spread diminishes as traffic increases, becoming practically zero for Class V. The ranges rounded to the nearest 1/10 meter are:

Class I	5.1 to 5.7 m.
Class II	6.0 to 6.4 m.
Class III	6.7 to 6.9 m.
Class IV	6.8 to 6.9 m.
Class V	7.1 m.

Except for Class I, these values agree remarkably well with those of Figure 2.3.

Finally, Figure 2.5 analyzes the country standards, independently of how they may be applied to specific projects. This graph shows that topography has very little influence on pavement width.

For comparison, the values for width of surfacing suggested by the above analyses are collected into Table 2.2.

Width of One Shoulder

For the histogram analysis of shoulder widths the whole sample was adjusted as follows:

	Kilometers	
Whole sample		69,433
Deduct isolated bridges	16	
Deduct facilities of 3 or more lanes	1,949	
Deduct gravel-surfaced roads for which a shoulder width was not given in the data	4,798	
Total deductions		6,763
Kilometers in adjusted sample		62,670
Line items in adjusted sample		611

The adjusted sample therefore represents only two-lane roads for which a definite shoulder width is known. Figure 2.6 shows these 62,670 kilometers arranged in histogram form according to design classes and 0.3-meter increments of shoulder width. This histogram suggests a range of shoulder widths for the adjusted sample as shown in Table 2.3.

The same line items used for compiling Figure 2.6 are plotted as a scattergraph in Figure 2.7. As with Figure 2.4, the plots of line items in the five dominant countries are shown by separate symbols, to distinguish them from those of the other countries, which are shown by solid black dots. The vertical dashed lines are the centroids of the black dots in each design class, while the solid vertical lines are the centroids of *all* the plotted points in

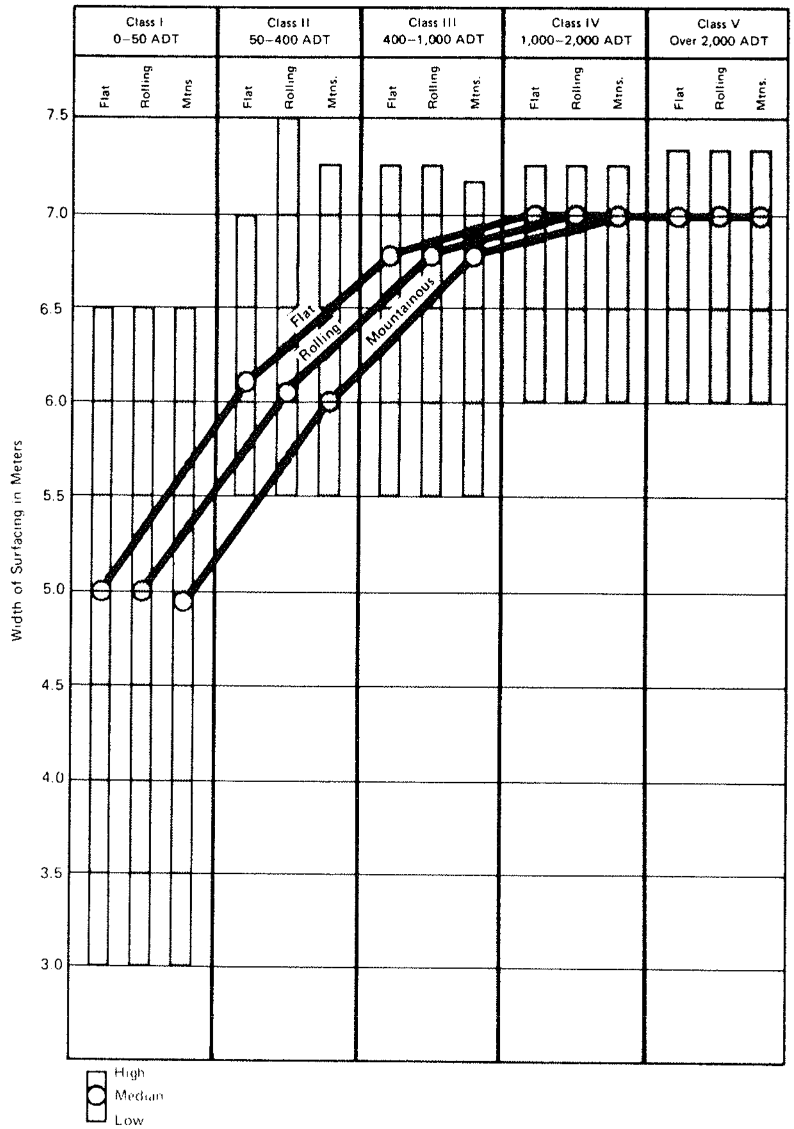
Table 2.2: Width of Surfacing, in Meters, for Two-Lane Roads as Suggested by Various Methods of Analysis (Rounded to nearest 1/10 meter)

Method	Class I 0-50 ADT	Class II 50-400 ADT	Class III 400-1,000 ADT	Class IV 1,000-2,000 ADT	Class V Over 2,000 ADT
Figure 2.2—Histogram of line items, all countries, (minus 4-lane segments)	4.0-5.0	5.5-6.0	6.0-6.5	6.5-7.0	6.5-7.0
Figure 2.3—Histogram of kilometers, all countries (minus 4-lane segments)	4.5-5.0	5.5-6.0	6.5-7.0	6.5-7.0	7.0-7.5
Figure 2.4—Scattergraph, all countries	5.1	6.0	6.7	7.0	7.1
Figure 2.4—Scattergraph, minus KEN, MXC, ZAI, ISR and CLM	5.7	6.4	6.9	6.8	7.1
Figure 2.5—Vertical bargraph, 56 countries	4.9-5.0	6.0-6.2	6.7	7.0	7.0
Practical range, all countries	4.0-5.1	5.5-6.2	6.0-7.0	6.5-7.0	7.0-7.5

Table 2.3: Width of Shoulder, in Meters, for Two-Lane Roads as Suggested by Various Methods of Analysis (Rounded to nearest 1/10 meter)

Method	Class I 0-50 ADT	Class II 50-400 ADT	Class III 400-1,000 ADT	Class IV 1,000-2,000 ADT	Class V Over 2,000 ADT
Figure 2.6—Histogram of kilometers, all countries, minus 4-lane segments and minus gravel roads with undifferentiated shoulders	1.0-1.3	1.0-1.3	1.3-1.6	1.6-2.0	2.0-2.3
Figure 2.7—Scattergraph, all countries	1.1	1.3	1.8	2.2	2.5
Figure 2.7—Scattergraph, minus KEN, MXC, ZAI ISR, CLM	1.1	1.5	2.1	2.2	2.4
Figure 2.8—Vertical Bar-Graph, 53 Countries	1.0	1.3-1.5	1.5-2.0	1.8-2.4	2.0-2.5
Practical range, all countries	1.0-1.1	1.0-1.5	1.3-2.0	1.6-2.4	2.0-2.5

Figure 2.5: Standards for Width of Surfacing in 56 Countries



each class. The practice of the five dominant countries has an appreciable effect only for Class III, operating to decrease the width of shoulder by about 0.3 meter.

Analyzing the country standards, independently of their application to specific projects results in the vertical bar graph, Figure 2.8. The range between high and low values is rather large, but

Figure 2.6:
Distribution of Road Lengths by Increments of Shoulder Width

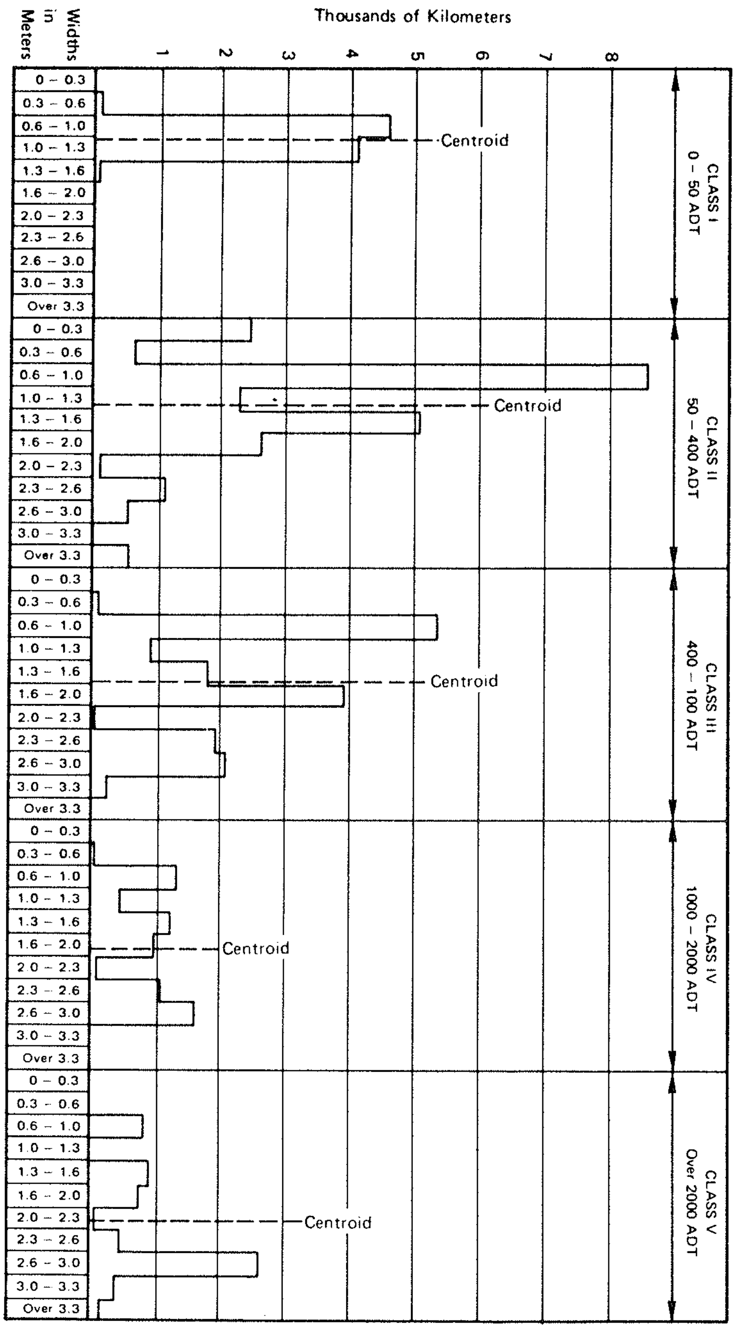


Figure 2.7: Width of One Shoulder versus Traffic on Opening for 611 Two-Lane Highway Segments

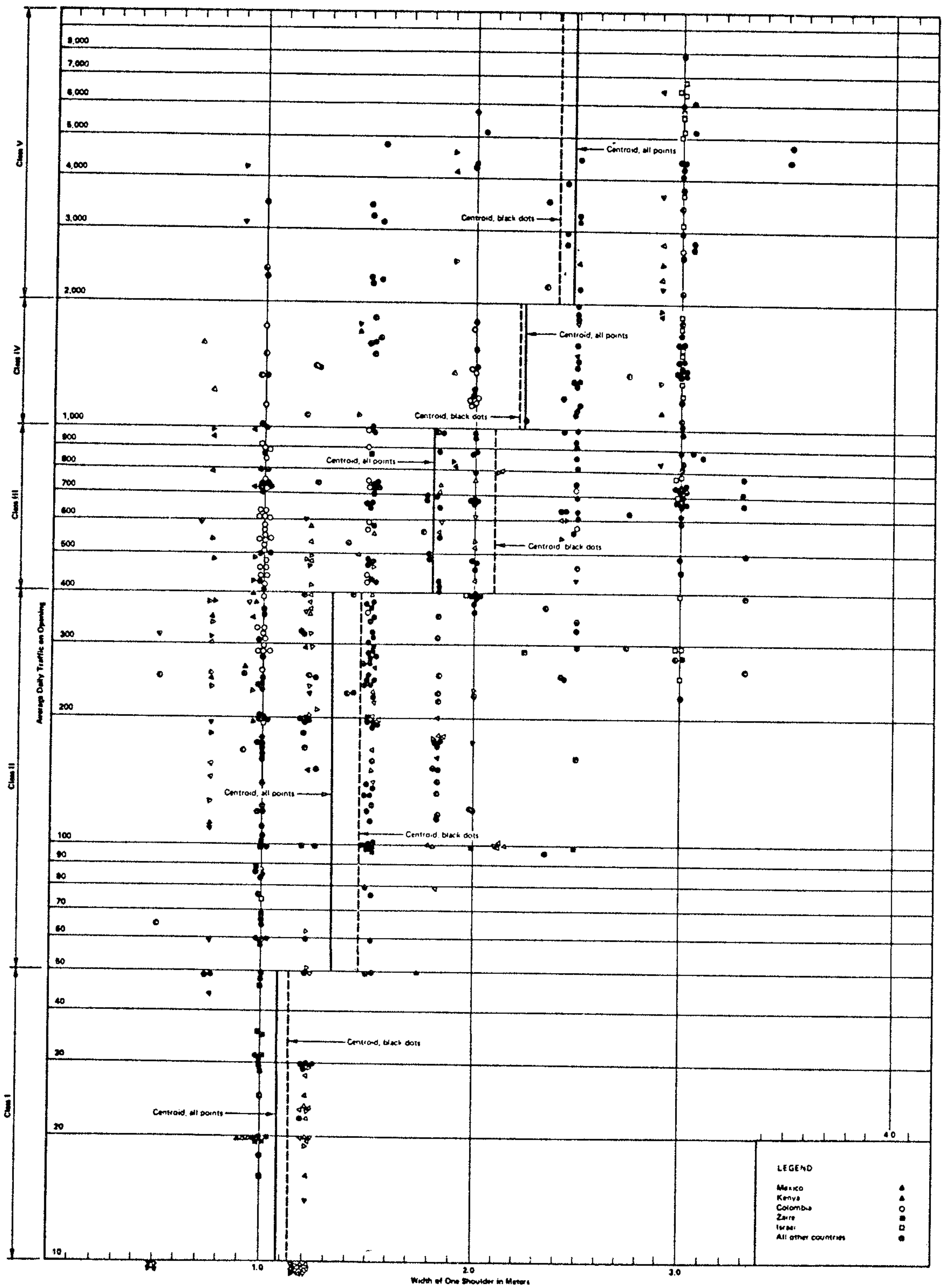
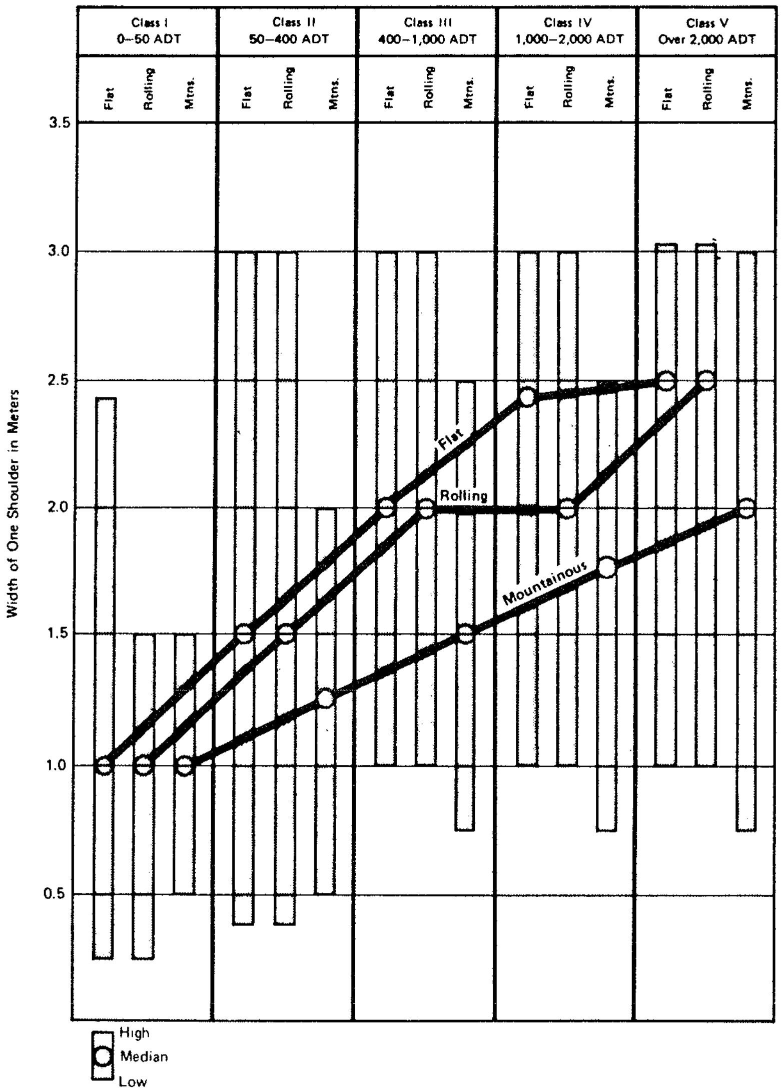


Figure 2.8: Standards for Width of one Shoulder in 53 Countries



the median values are consistent. They show that shoulder widths are generally not reduced appreciably until mountainous conditions are encountered. Comparing Figure 2.8 with Figure 2.5 it appears that rugged terrain is usually accommodated by narrowing the shoulder rather than the surfacing.

Right-of-Way Width

Right-of-way width seems to be related more to administrative classification than to either traffic or terrain type, and it may in addition be subject to considerable arbitrariness from country to country. These influences are reflected to some extent in Figure 2.9, especially in the extremely wide variations from high value to low value. The sharp rise in the median values for roads in Class V probably reflects the increasing possibility that these roads may someday be widened to four lanes or even provided with frontage roads for access control.

Surfacing Type

Figure 2.10 shows percentages of various surfacing types falling within 23 traffic brackets based on ADT at time of opening to traffic. This graph shows a tendency to use granular surfaces up to an ADT of 75 on opening. Bituminous surface treatments predominate between ADT 75 and ADT 400, and high-type paving, generally asphaltic concrete, for volumes over ADT 750. The use of bituminous surfacing on roads carrying less than 200 ADT may seem surprising, yet over 8,300 km. of such low-volume bituminous surfaces were constructed for traffic down to 75 ADT under the various IBRD projects, and were justified economically.

In Figure 2.11 surfacing types are plotted according to percentage of kilometers of each type in each of the five design classes.

Design Speed

As was noted earlier, a design speed cannot be identified in the appraisal reports for every line item. The most practical analysis therefore, is to analyze the country standards. The data available for this kind of analysis consist in part of complete country standards such as shown in Table 1.1 and partly of extracts from the country standards applying to the project roads. Thus, if the project roads in Country A happen to be in level terrain, Country A would be represented only in Column One "Flat" of Figure 2.12. In the same manner, Country B may be represented only in Column Two "Rolling" or Column Three "Mountainous." Only fifteen countries are represented in all three terrain classes and not all of these are represented in all five design classes.

While the spread from "low value" to "high value" shown by Figure 2.12 is quite wide, the medians fall into a consistent pattern, with a differential of about 20 km.p.h. between terrain classes.

Figure 2.9: Standards for Width of Right-of-Way in 25 Countries

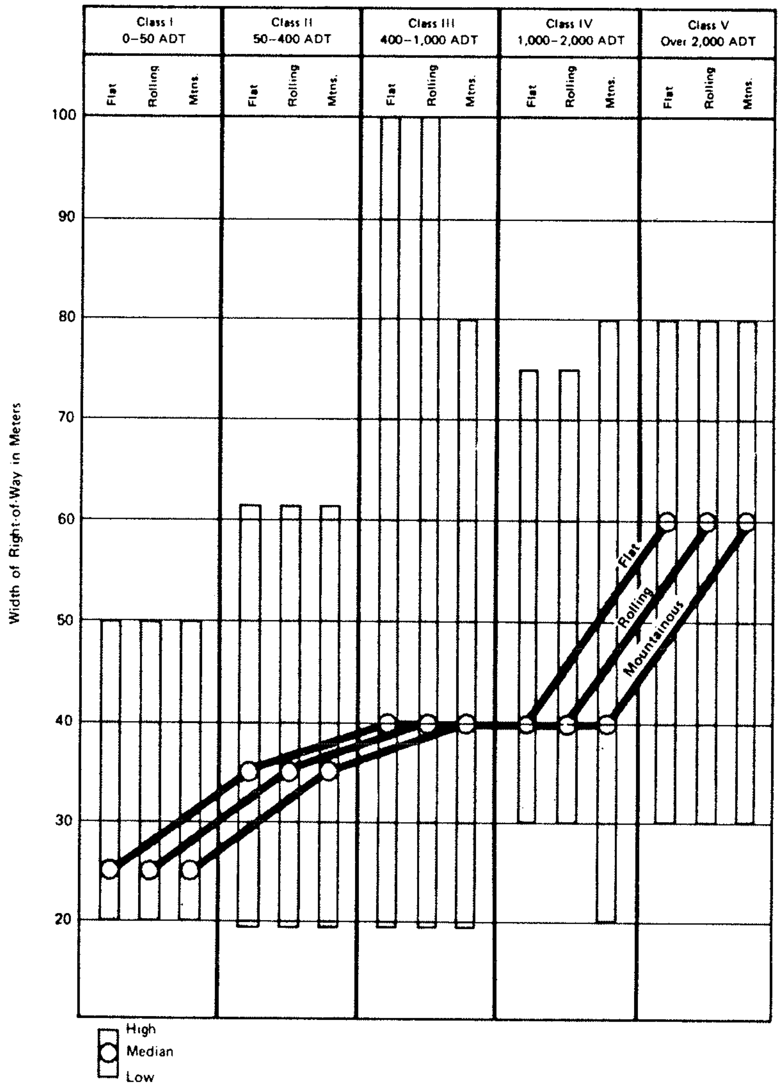


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Figure 2.10:
Percentages of Surfacing Types Falling within 23 Traffic Brackets

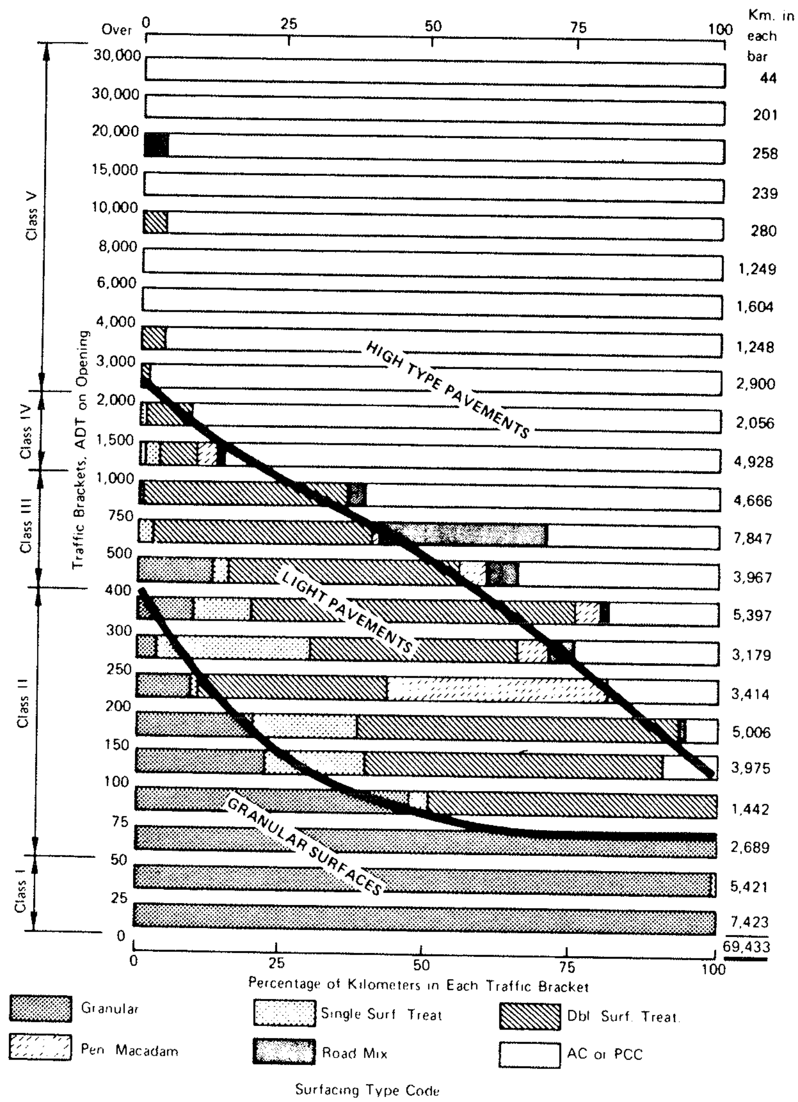


Figure 2.11:
Percentages of Surfacing Types Falling within 5 Design Classes

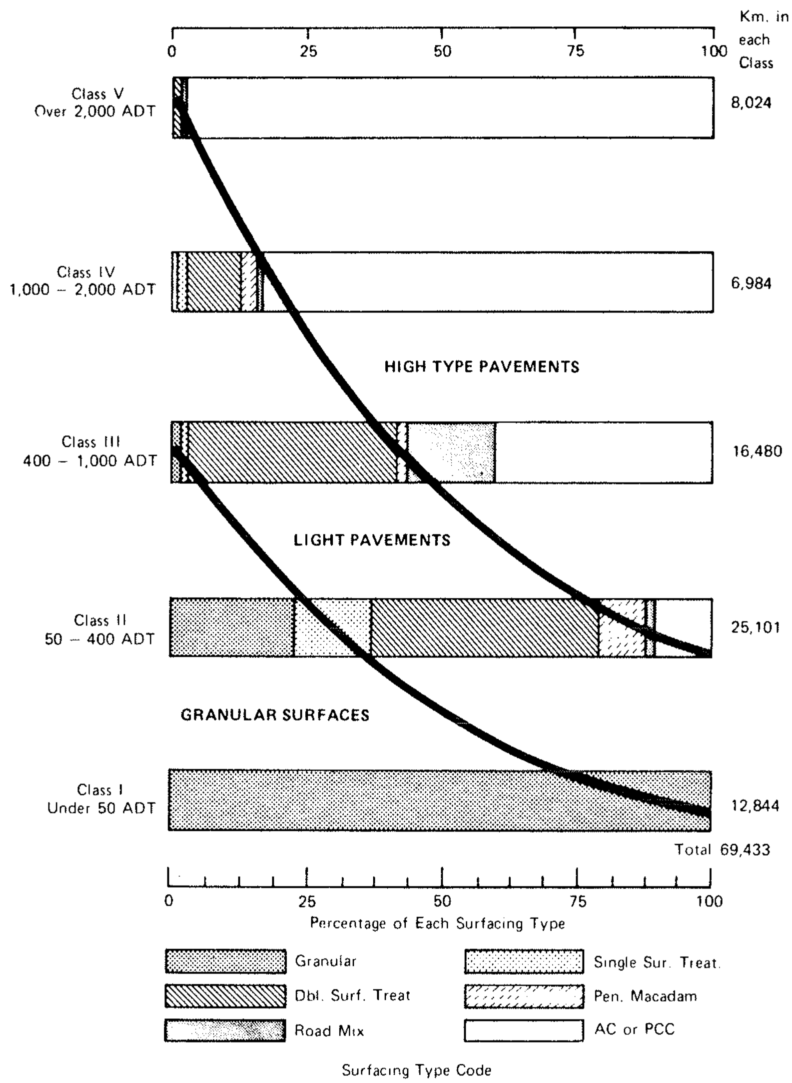
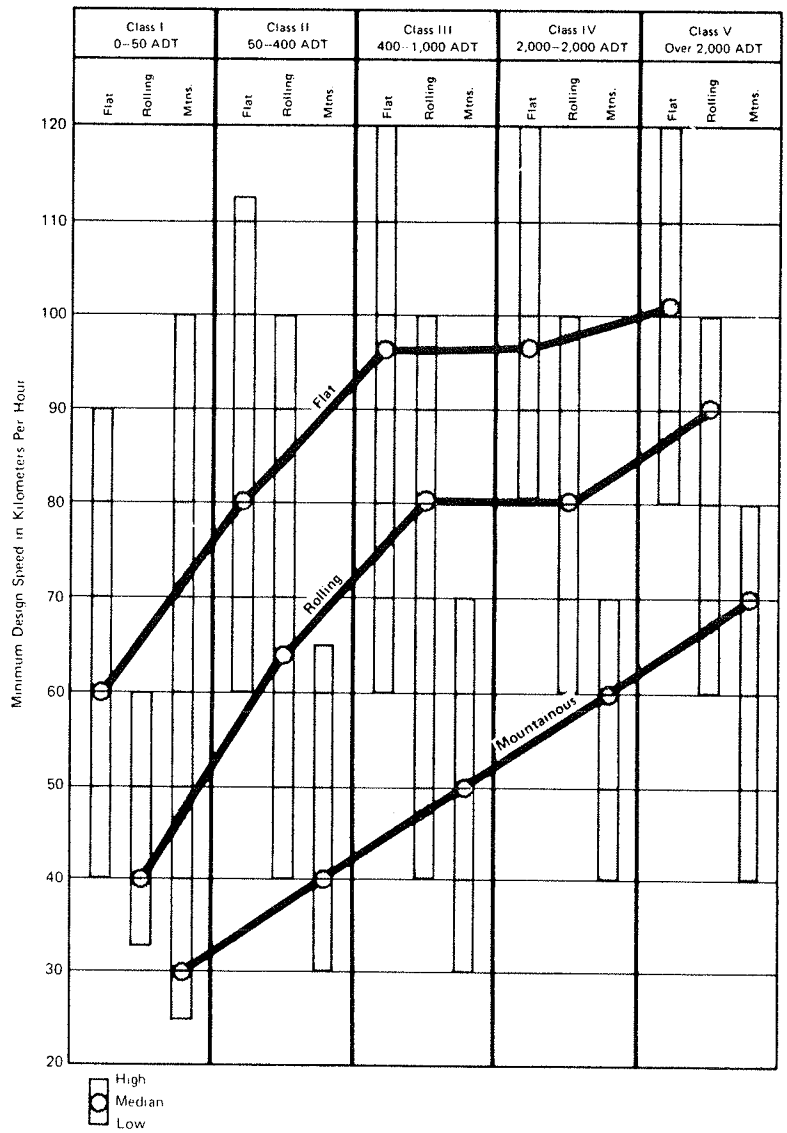


Figure 2.12: Standards for Design Speed in 55 Countries



Minimum Radius of Horizontal Curvature

Radius of curvature is a derivative of design speed, and is affected by either traffic volume or topography only to the extent that these determine the design speed. A moving vehicle rounding a curve generates a horizontal force which is resisted by the superelevation of the road surface and the side friction developed between the tires and the road. There are practical limits for superelevation and also for the amount of side friction that can be developed without discomfort to the vehicle occupants and these limits determine how long the radius must be for any given design speed.

The scattergraph plot of Figure 2.13 shows a rather wide spread in the radii corresponding to various design speeds, especially for the higher velocities. This spread may be due to differing assumptions as to maximum permissible superelevation and the permissible side friction factor. Unfortunately, the data do not permit a more extended analysis, since superelevation is reported for only nine countries. Of these, one country permits a maximum superelevation of 12 percent, four use 10 percent, two 8.5 percent and two 8.0 percent.

Sight Distance

In Figures 2.14 and 2.15 stopping and passing sight distances are plotted as scattergraphs against design speed. In both of these figures the average trend of practice in the sample countries is shown by the heavy dashed line. For comparison, practice in the USA as exemplified by the 1971 AASHO Standard is shown by light lines on these figures.

The USA practice for stopping sight distance was modified from the previous 1965 standard when a new "desirable" standard was adopted by AASHO [2] in 1971. The new standard compensates for a generally lower operator's eye height in new automobiles, and somewhat higher average road speeds during inclement weather than had previously been assumed. However, the 1965 standard was retained by AASHO as a "minimum." Figure 2.14 suggests that the practice in the sample countries is very close to that prevailing in the USA in 1965.

[2] This organization has recently changed its name to American Association of State Highway and Transportation Officials (AASHTO). In this report it will be referred to by the old designation, which was in effect during the period covered by the report.

From Figure 2.15 it appears that passing sight distances in developing countries are considerably shorter than USA practice in the lower speed ranges, but very close at higher speeds.

Maximum Gradient

Gradient standards for 52 countries are plotted in Figure 2.16. They show the usual wide spread from high value to low value, but the median values follow a consistent trend.

Crown or Camber for Bituminous Surface (13 Countries)

Highest value	3.0 percent cross slope in 1/2 width
Median value	2.0 percent cross slope in 1/2 width
Lowest value	2.0 percent cross slope in 1/2 width

Shoulder Slope (6 Countries)

Highest value	4.0 percent
Median value	4.0 percent
Lowest value	5.0 percent

Bridge Width, Curb-to-Curb, Short Bridges, 10-25 m. in length (24 Countries)

	Countries	Percent
Single-lane bridge	1	4
Approach surfacing width, plus less than 1 meter	7	29
Approach surfacing width, plus 1.0 to 1.5 meters	10	42
Same as width of approach roadway (surfacing plus shoulders)	6	25
Total	24	100

Bridge Width, Curb-to-Curb, Bridges over 25 m. long (24 Countries)

	Countries	Percent
Single lane bridge	3	12
Approach surfacing width, plus less than 1 meter	8	33
Approach surfacing width, plus 1.0 to 1.5 meters	10	43
Approach surfacing width, plus more than 1.5 meters	3	12
Total	24	100

Figure 2.13: Design Speed versus Radius in 55 Countries

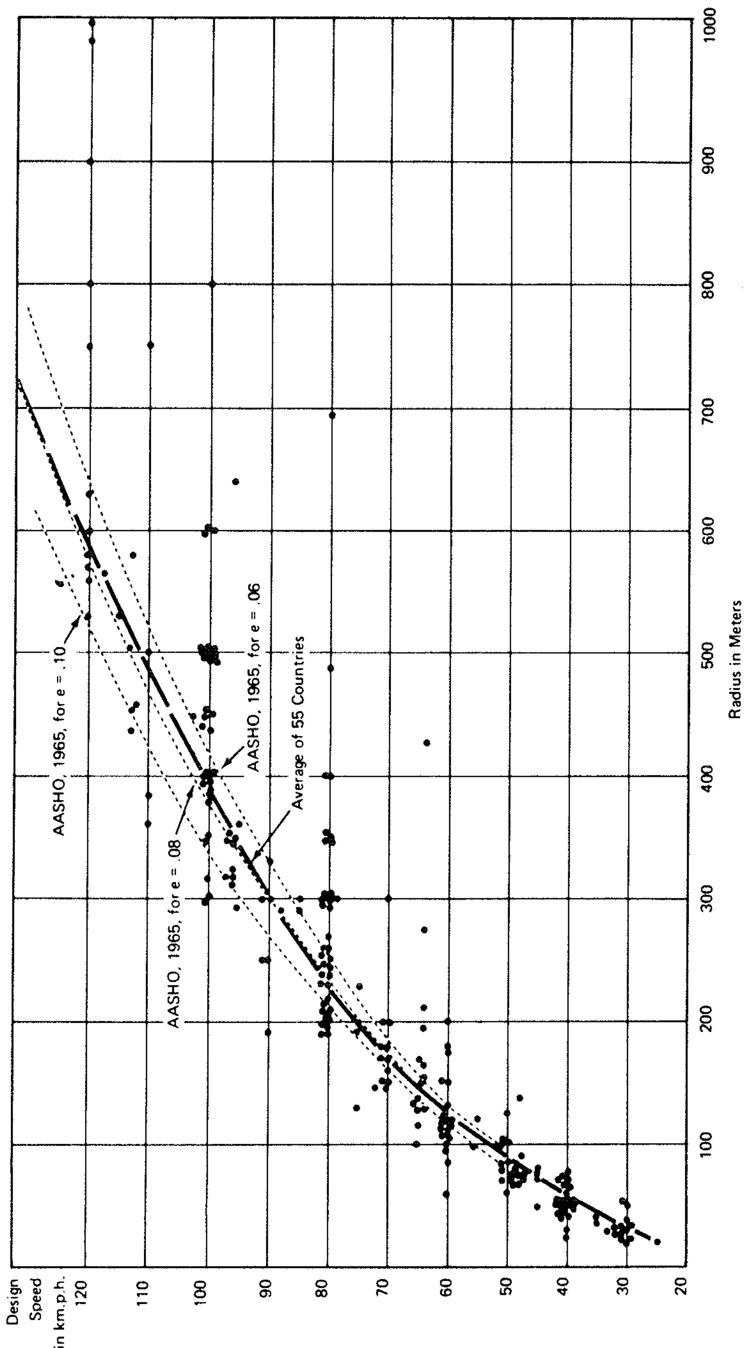


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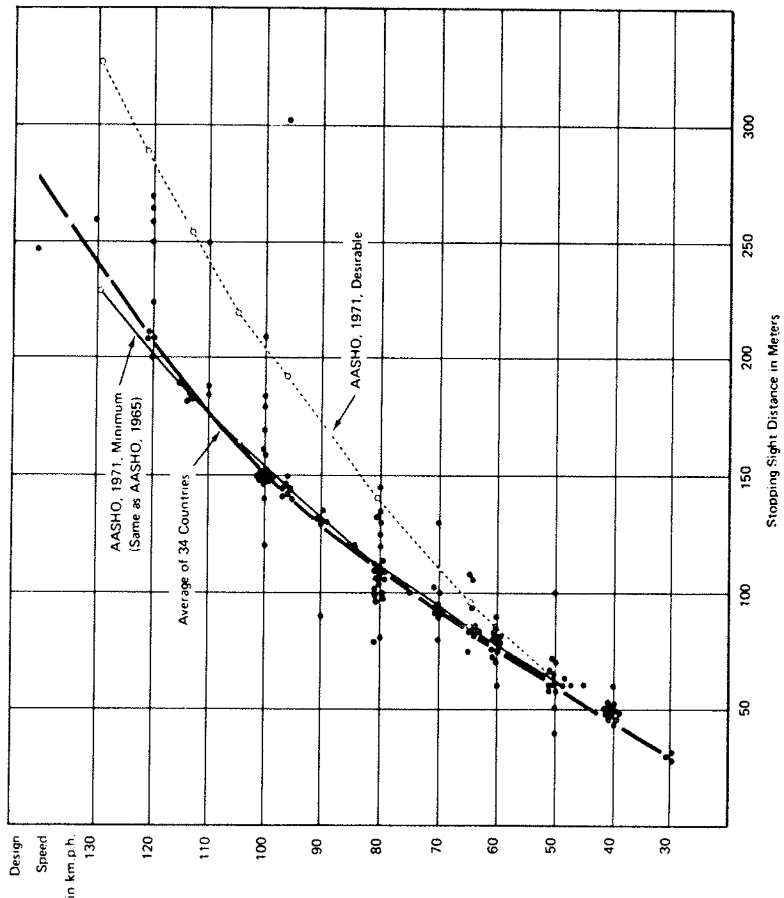
Figure 2.14:
Design Speed versus Stopping Sight Distance in 34 Countries



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Design Live Loadings for Bridges

A highway bridge must support its own weight, plus the live loads imposed by traffic. Vehicles of many sizes, weights and axle arrangements may use the bridge during its lifetime of 30 or more years. It would be laborious and impractical to design for these individually, so "standard live loadings" have been devised to simplify design. These loadings do not necessarily produce the same effect on a bridge as actual vehicles, but when the bridge is designed in accordance with them it should support any legal load that is likely to come upon the structure.

Figure 2.15:
Design Speed Versus Passing Sight Distance in 16 Countries

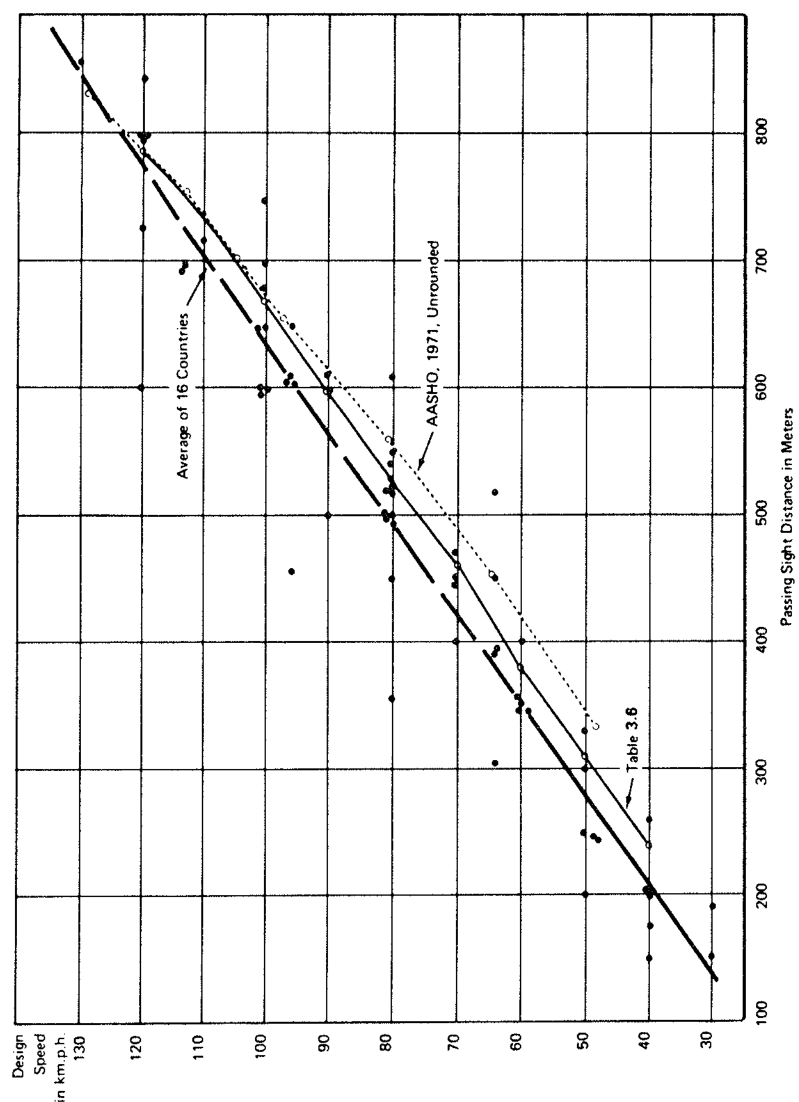

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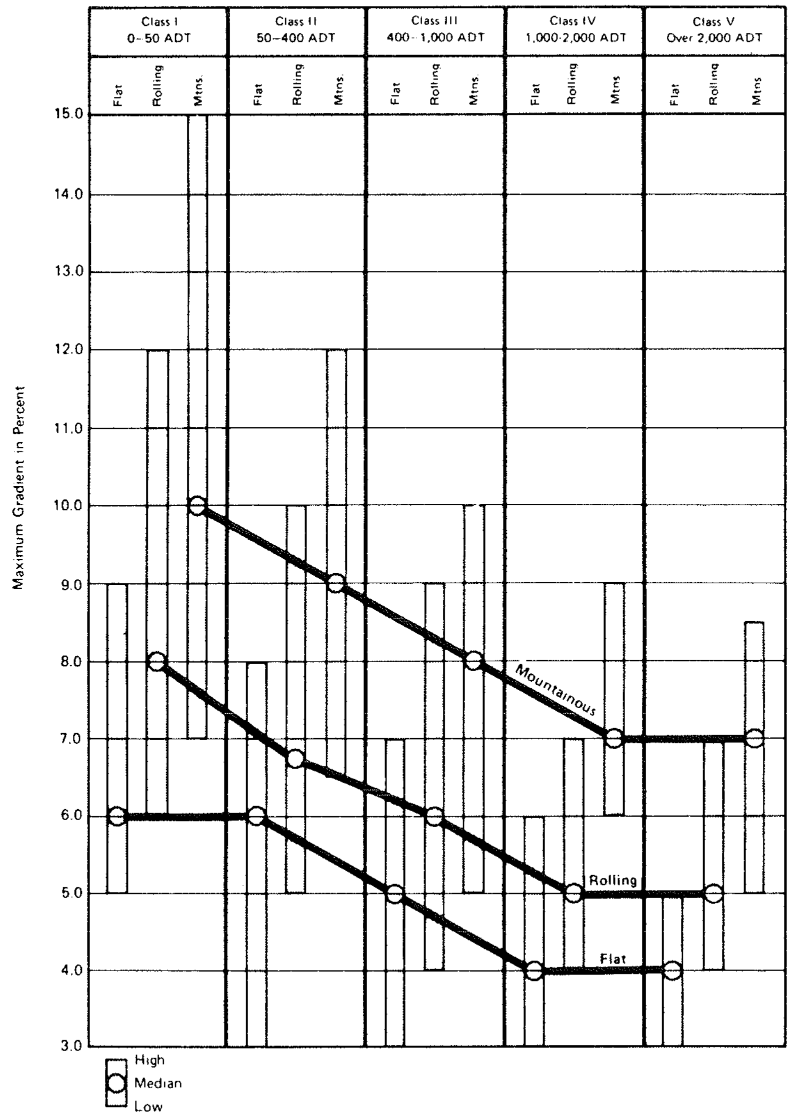
Figure 2.16: Standards for Maximum Gradient in 52 Countries



Standard live loadings vary considerably from country to country, but all are variations of three general types:

i. A simplified "design vehicle" of two or more axles with specified axle loadings and spacings; one or more of these vehicles may be placed anywhere on the bridge to produce maximum stresses in the bridge members.

ii. A uniform lane loading of a specified weight per square foot or square meter of deck.

iii. A single extremely heavy axle load (or knife-edge load or wheel load) which can be positioned anywhere to produce maximum stress, usually in conjunction with a uniform lane loading.

Although standard live loadings are primarily for the convenience of bridge designers, they may also serve as practical indicators of load-supporting capacity. Thus, a bridge designed according to AASHO live loading HS 20-44 will safely support a 36-ton 3-axle truck-semi-trailer combination in each lane.

The design live loadings used by 46 developing countries are given in the appraisal reports and are listed as follows:

Live Loading	Countries	Percent
AASHO H20-S16	18	39
AASHO H15-S12	13	28
British Standard 153, HA Loading	7	15
German Standard DIN 1072	1	2
French Ministry of Public Works "Cahier des Prescriptions Communes"	2	4
Indian Road Congress "A" or "AA"	2	4
NAASRA H20-S16	1	2
Other	2	4
Total	46	100

Standards for Structural Design of the Pavement

The duty of the pavement structure can be summed up in three words: traction, protection and distribution. It must provide a smooth surface of low rolling resistance, yet not slippery, for the passage of vehicles; it must protect the underlying soil from moisture saturation, erosion and abrasion; and it must spread the traffic loads evenly over the underlying soil without exceeding its strength.

The load-bearing capacity of the surfacing depends on the strength of the underlying soil or subgrade, and the strength of the pavement structure itself. Flexible surfaces, which predominate in developing areas, are composed of layers of materials, each layer stronger than those below. The thickness of the layers depends on the finished quality of the materials of which they are made and the loads that must be carried.

Highway traffic is a mixture of light, medium and heavy vehicles. All traffic occupies space and inflicts surface wear but only the heavier vehicles impose appreciable loads on the pavement structure. These heavier vehicles are designed to distribute the load over a number of axles and in practically every country the maximum load that can be carried by a single-axle or by tandem-axles is limited by law. Design of pavement structures therefore starts with the legal load limit, and the design must assume that the limit will not be exceeded, or at least not frequently or greatly exceeded.

The appraisal reports give "axle loadings for pavement design" (normally the legal limit) for 40 countries. These range from 8.0 metric tons per single-axle up to 13.0 metric tons per single-axle:

Metric Tons Single-Axle Load	Countries	Percent
8.0	4	10
8.16 (18,000 Pounds)	11	28
8.5	1	2
9.0 to 9.50	5	13
10.0	7	18
10.88 to 11.5	3	7
12.0	3	7
13.0	6	15
Total	40	100

The median value is 9.06 metric tons (20.000 pounds).

In some countries these pavement design loadings exceed the maximum legal single-axle limits, but generally the pavement design loading is the legal maximum.

Design thickness for the base and pavement structure can vary widely for the same physical conditions, depending on the method of design used. What little information regarding design methods appears in the appraisal reports is insufficient to establish a trend, but most methods involve a correlation between the soil strength as measured by a test such as the CBR Test, and an equivalent wheel or axle load, or repetition of axle loads.

Category III Tertiary and Special Purpose Roads

The World Bank participated in the financing of 11,222 kilometers of low-standard local roads in 15 developing countries over the study period. Typically, these roads carried less than 100 ADT. With few exceptions they had granular (gravel or laterite) surfaces, and a very considerable number were developed by upgrading existing tracks and trails. "Feeder Road" is perhaps the most generally-used term for these roads, but they are also referred to in the appraisal reports as Tea Roads, Forest Development Roads, Sugar Roads, Cotton Roads, Settlement Roads and Penetration Roads. The standards used for these roads are compared in Table 2.4 below:

Table 2.4: High, Median and Low Values for Design Elements of Tertiary and Special Purpose Roads (Traffic generally less than 100 ADT)

Terrain	Flat	Rolling	Mountainous
		km.p.h.	
Design Speed (13 Countries)			
High value	80	80	63
Median value	60	40	40
Low value	40	30	20
Maximum Gradient (14 Countries)			
		Percent	
High value	12.0	12.0	16.0
Median value	7.5	9.0	10.0
Low value	6.0	8.0	7.0

*Width of Surfacing** (15 Countries)		
High value	7.00 m.	(without regard to terrain)
Median value	5.00 m.	(without regard to terrain)
Low value	2.63 m.	(without regard to terrain)
*Width of One Shoulder** (15 Countries)		
High value	2.44 m.	(without regard to terrain)
Median value	1.00 m.	(without regard to terrain)
Low value	0.50 m.	(without regard to terrain)
*Width of Roadway (Surfacing plus Shoulders)** (15 Countries)		
High value	10.96 m.	(without regard to terrain)
Median value	7.00 m.	(without regard to terrain)
Low value	4.00 m.	(without regard to terrain)
Design Loading for Bridges (7 Countries)		
AASHO—H20-S16	2 Countries	
AASHO—H15-S12	2 Countries	
AASHO—H15	1 Country	
NAASRA—H20-S16	1 Country	
British Standard 153, Two Thirds of HA Loading	1 Country	
Single-Lane bridges are built initially on low traffic roads in at least 2 countries.		
Right of way (4 Countries)		
Desirable minimum, 50 m.	2 Countries	
Desirable minimum, 20 m.	2 Countries	

* For some projects the width of the roadway and the width of surfacing were the same; that is the road had no shoulder. In these cases 0.5 meter of the surface on each side was arbitrarily designated as "shoulder."

III. Discussion and Conclusions

Most countries have their own prescriptions for highway standards, and there are many similarities between these national standards. The activities of international associations such as the Permanent International Association of Road Congresses, the International Road Federation and the Pan American Highway Congresses have produced a healthy cross-fertilization of engineering thought and practice which has found expression in the national road standards. Another strong influence in recent years has been the activity of international engineering consultants. A number of these do a world-wide business and have an opportunity to study and evaluate the standards in different countries. The international lending organizations and the agencies for bilateral and multilateral economic aid exert yet another leveling influence on standards.

Standards As Guides for Designers

Today's standards are the distilled essence of the experience of engineers, administrators and motorists over the past 80 years of living with the motor vehicle. Highway standards change continually, though slowly, to adjust to changes in the vehicle as well as to the changing economic and technological status of society. Furthermore, for most countries the standards provide a wide range of flexibility within themselves. The word "standard" implies a much greater degree of uniformity and conformity than is found among most road standards; perhaps "guide" would be a better term.

Standards serve as guideposts for the designer, but they do not actually dictate the route or the design. They should be flexible, and the designer should apply them taking into consideration not only the estimated future traffic, but also the physical difficulties to be overcome, the road maintenance facilities, and the funds and other resources at his disposal. There are seldom enough funds available to build all the roads that are needed or to build to the most desirable standards; consequently the designer is constantly faced with the need to select standards that are high enough to move the traffic, but low enough to fit the budget—often no small task. A way out of this dilemma may sometimes be found in the "stage construction" approach which is discussed in more detail later in this chapter.

Standards for the Capacity-Related Elements of Design

The vehicle-carrying capacity of a highway is closely related to the width of the surfacing and shoulders, the design speed and the gradient, and to a lesser extent the width of the right-of-way. Despite the wide range of values for these elements of design, as shown in Chapter II, the median values are consistent and reasonable. With minor modifications, these median values, shown in Tables 3.1, 3.2 and 3.3 below, provide a useful benchmark for evaluating the standards that may be proposed for specific projects.

Table 3.1: Width of Right-of-Way for Various Design Classes

Traffic on Opening		Width of Right-of-Way (meters)
Class I	Less than 50 ADT	25 m.
Class II	50 to 400 ADT	35 m.[a]
Class III	400 to 1,000 ADT	40 m.[a]
Class IV	1,000 to 2,000 ADT	50 m.[a]
Class V	Over 2,000 ADT	60 m.[b]

[a] In mountainous areas additional width may be needed for cut and fill slopes or exceptional drainage measures.
[b] Additional width may have to be provided as required for future lanes, frontage roads, etc.

Table 3.2: Width of Surfacing and Width of One Shoulder for Various Design Classes, Including Allowance for Speed (Mixed Traffic on Date of Opening)

Design Speed (km.p.h.)	Class I 0-50 ADT	Class II 50-400 ADT	Class III 400-1,000 ADT	Class V 1,000-2,000 ADT	Class V Over 2,000 ADT
			Width of Surfacing (Meters)		
40	4.0	5.5	6.0	6.7	7.0
60	4.5	6.0	6.0	6.7	7.0
80	5.0	6.0	6.7	7.0	7.3
100	[a]	6.2	6.7	7.0	7.3
			Graded Width of One Shoulder (Meters)		
All Speeds	1.0	1.5	2.0	2.4	3.0

[a] Roads in Class I should not be designed for high speeds.

Table 3.3: Minimum Design Speeds and Maximum Gradients for Various Design Classes and Types of Terrain (Mixed Traffic on Date of Opening)

Terrain Type	Class I 0-50 ADT	Class II 50-400 ADT	Class III 400-1000 ADT	Class IV 1,000-2,000 ADT	Class V Over 2,000 ADT
	Minimum Design Speed (km.p.h.)				
Flat	60	80	90	95	100
Rolling	40	65	80	80	90
Mountainous	30	40	50	60	70
	Maximum Gradient (Percent)[a]				
Flat	6.0	6.0	5.0	4.0	4.0
Rolling	8.0	7.0	6.0	5.0	5.0
Mountainous	10.0	9.0	8.0	7.0	7.0

[a] Many countries have a sensible provision that the normal maximum gradients can be exceeded by about 1.0 percent for distances not exceeding 250 to 500 meters, in order to surmount occasional short stretches of difficult terrain.

Standards for the Velocity-Related Elements of Design

The standards for minimum radius and stopping sight distance are different from other highway standards in that they are principally dependent on design speed, rather than on traffic or terrain. Engineering judgment and consensus come into play in the selection of friction factors, superelevation rates and driver reaction times, but once these are determined the required values for these elements can be determined by mathematical calculation.

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Passing sight distance, on the other hand is dependent to a large degree on driver behavior—so much so that the standards adopted in the United States are based almost wholly on empirical studies of a large number of overtaking maneuvers.

Radius of Curvature Versus Design Speed

The radii of highway curves depend not only on the assumed design speed, but also on the superelevation, e, of the pavement and the amount of side friction, f_s, that can be safely and comfortably developed between the vehicle tires and the pavement surface. At maximum safe speed this relation may be expressed thus:

$$e + f_s = .0079\frac{V^2}{R}$$, where V is the speed in kilometers per hour and R is the radius in meters.

Countries where ice and snow are prevalent limit superelevation to 6 percent or 8 percent to preclude vehicles from sliding toward the center of the curve at low speeds. In warmer climates 10 percent or even 12 percent superelevation rates can be used, 10 percent being the median of the countries for which information appears in the appraisal reports.

According to extensive research, 0.16 is a desirable value for the side friction factor for speeds up to about 50 km.p.h. For higher speeds this value should be reduced for safety, reaching a minimum of 0.11 at a speed of 120 km.p.h.

The desirable minimum radii for various design speeds and a maximum superelevation rate of 10 percent are shown in Table 3.4 and also on Figure 2.13.

Table 3.4: Minimum Radii for Various Design Speeds

Design Speed (km.p.h.)	Side Friction factor "f_s"	Superelevation "e"	Minimum Radius in meters (Rounded)
40	0.16	0.10	50
50	0.16	0.10	76
60	0.15	0.10	114
70	0.15	0.10	155
80	0.14	0.10	211
90	0.13	0.10	278
100	0.13	0.10	344
110	0.12	0.10	435

Stopping Sight Distance

We have seen from Chapter II and Figure 2.14 that the average practice for stopping sight distance of the 34 developing countries for which data are available closely approximates the AASHO 1965 Standard. This standard assumed that the critical condition for stopping would occur during rainy weather when the pavement would be wet, and also that due to the inclement weather drivers would slow down and be travelling at about 80 to 90 percent of the design speed. The friction factor for forward skidding on a wet concrete surface, (which varies from 0.35 at low speeds to 0.28 at 110 km.p.h.) was assumed in the kinetic energy formula for braking distance. Driver perception/reaction time was assumed to be 2.5 seconds—well within the physical capability of most drivers. These assumptions expressed in metric units result in the minimum stopping distances shown in Table 3.5. These appear to be reasonable and practical for most highways except high-volume freeways, for which stopping distances should be based on the full design speed. The stopping distances of Table 3.5 are substantially the same as those of the Bank's 1957 *"Guide to Highway Standards."*

Table 3.5: Minimum Stopping Sight Distances for Various Design Speeds on Level Highway

Design Speed (km.p.h.)	Assumed Driving Speed (km.p.h.)	Perception-Reaction Values: Time (Seconds)	Perception-Reaction Values: Distance (Meters)	Coefficient of Friction for Wet Pavement (f_L)	Braking Distance on Level Road[a] (Meters)	Rounded Stopping Sight Distance[b] (Meters)
40	37	2.5	25.6	0.35	15.4	40
60	54	2.5	37.5	0.35	32.8	70
70	62	2.5	43.4	0.32	47.2	90
80	70	2.5	48.6	0.30	64.4	115
90	78	2.5	54.2	0.29	82.1	135
100	85	2.5	61.5	0.29	98.0	160
110	91	2.5	63.0	0.28	116.8	180
120	97	2.5	67.4	0.28	132.5	200

[a] Braking distance from the formula: $d = \frac{V^2}{254f_L}$

[b] Stopping distance is measured on the road from an assumed vehicle operator's eye-level of 1.15 meters (3.75 feet) above the pavement to an assumed obstacle projecting 15 cm. (6 inches) above the surface of the pavement ahead.

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Passing Sight Distance

The overtaking maneuver was the subject of elaborate studies in the USA in 1938 and 1941, using automatic detectors and recorders. Some of this research was repeated in 1957, substantially verifying the earlier work, even though in the interim highway speeds and vehicle performance capabilities had substantially increased. These studies, which are the basis for the standards currently in use in the USA, showed that few drivers used all acceleration of which their vehicles were capable, and there were appreciable variations in the amount of sight distance available to the overtaking drivers as they began the maneuver. The passing sight distances recommended for design were those that would accommodate the majority of the desired passing maneuvers. These are plotted in metric units on Figure 2.15 as a light dashed line. The horizontal displacement between this line and the trend line for the 16 developing countries suggests that somewhat lower passing sight distances may be appropriate in these countries, as shown by the intermediate curve, and Table 3.6.

Table 3.6: Minimum Passing Sight Distance for Various Design Speeds on Level Highway

Design Speed (km.p.h.)	Passing Sight-Distance (Meters)
40	240
50	310
60	380
70	460
80	530
90	600
100	670
110	740
120	790

Horizontal and Vertical Clearances for Bridges

The modern concept of highway bridges is to regard them not primarily as "structures," but, rather, as more expensive sections of roadway. Forty years ago highway locators went to great pains to save structure cost by selecting right-angle river crossings and avoiding bridges on curves. The result was often poorly aligned bridge approaches. Nowadays, the locator lays his line in the best location and the bridge engineer is expected to fit his structure to it. Only for exceptionally large or difficult crossings does the bridge site dictate the road location.

The same thinking applies to bridge deck widths, which formerly were no greater than that of the approach pavement. Gradually the clearance between edge of pavement and the bridge curb or parapet was increased until by 1945 the practice in the USA was to carry the entire roadway width, including shoulders, across bridges of 20 feet (6.1 meters) span or less, and to make longer bridges at least 2 feet wider on each side than the approach pavement. The present trend in the USA is to carry the full roadway width, including shoulders, across all bridges, excepting only the most monumental and costly structures.

In the developing countries, where financial considerations are usually stringent, bridge widths are generally narrower than in the USA. For example, out of 24 countries only 6 carry the full width or roadway across "short" bridges, defined as those of 10 to 20 meters length. The general practice for both short and long bridges seems to be to provide approach pavement width plus 1.0-1.5 meters.

Vertical clearances for under bridges were not readily identifiable from the study appraisal reports, but from a consideration of the dimensions of buses, trucks and their loads, a minimum of 5.0 meters is believed to be appropriate, unless special circumstances prevail.

Standard Live Loadings for Bridges

The data for the study show that the standard live loadings of the American Association of State Highway Officials are widely used in the developing countries. There are 5 standard AASHO loadings, but, with few exceptions, they cannot be definitely tied in with design classes based on traffic volume. Generally, however, AASHO Loading H20-S16 (now known as HS20-44) which is equivalent to a 36-ton truck-semitrailer combination on 3 axles, is used for "main roads" and high traffic volumes. For secondary and tertiary roads either AASHO H20-S16, or AASHO H15-S12 may be used, the latter being equivalent to a 27-ton semitrailer combination.

The problem in most of these countries is that all roads, down to the least important, carry relatively high volumes of heavy trucks and the trucks are commonly overloaded. For this reason, a number of countries use the highest loadings on all their permanent bridges. On very low-traffic roads the consequences of a structural failure, in terms of traffic interruption, would of course be less than for more important roads, and some reduction in standard design loadings might therefore be justified. This possibility has been taken into account in preparing Table 3.7 below.

Table 3.7: Design Horizontal Clearances and Loadings for Bridges

Design Class Traffic on Opening (ADT)	I 0-50	II 50-400	III 400-1,000	IV 1,000-2,000	V Over 2,000
Width, curb-to-curb for bridges of 20 meters length, or less	Single Lane (3.5 to 4.0 Meters)	Approach Roadway Width	Approach Roadway Width	Approach Roadway Width	Approach Roadway Width
Width, curb-to-curb for bridges over 20 meters long	Single Lane (3.5 to 4.0 Meters)	Approach Pavement + 1.5 m.	Approach Pavement + 1.5 m.	Approach Pavement + 2.0 m.	Approach Pavement + 2.0 m.
Design live loading, according to AASHO system	H15-44	HS20-44	HS20-44	HS20-44	HS20-44

The Structural Capacity of Pavements

Flexible pavement structures yield slightly under traffic and recover somewhat after the load has passed. Under this continual flexure the pavement is weakened by fatigue, and eventually has to be strengthened. Its useful life before strengthening depends upon the magnitude of the axle loads that pass over it and their number. So, to properly design a flexible pavement, one needs to know not only the legal axle load limit but also the number of such axle loads that will use the road, in the opening year and into the future through its economic life.

But traffic may be light both in volume and in the number of heavy axles during the early years of the road's economic life. Fortunately because of its layered construction a flexible pavement is ideally suited to stage construction. One can design the initial stage to take care of the heavy axles forecast for, say, the first 5 to 7 years of the road's life and add more layers or stronger layers later as traffic increases. One must be careful, however, that the initial stage is strong enough that it will not break up before the time comes to strengthen it.

Legal Load Limits

The most important single factor for designing the thickness of a highway pavement is the weight of the live loads it will be required to support. For all practical purposes the lighter loads—those imposed by automobiles, pickups and light trucks—can be ignored, as their effect on the result is practically nil. This is shown by Figure 3.1 which depicts findings from the AASHO Road Test of 1960.

According to Figure 3.1 one 18-kip (8.18 metric tons) axle load is equivalent in its destructive effect on the pavement to eight 10-kip. (4.55 m ton) axle loads. (The equivalency factors are 1.0 and 0.12 respectively.) Increasing the axle load from 18 kips to 22 kips (10.0 m ton) doubles the destructive effect, (equivalency factor = 2.0). A further increase to 26 kips (11.8 m ton) doubles it again. Figure 3.1 is a dramatic demonstration of the extent to which heavy axle loads and overloading shorten pavement life.

High load limits and overloading greatly increase the capital cost of road pavements and also the cost of maintenance. But extremely low limits may increase the economic cost of haulage and be a burden to development. As was noted in Chapter II the present range of "design axle loadings" (normally the legal limit) in 40 countries is between 8.0 and 13.0 metric tons per axle,

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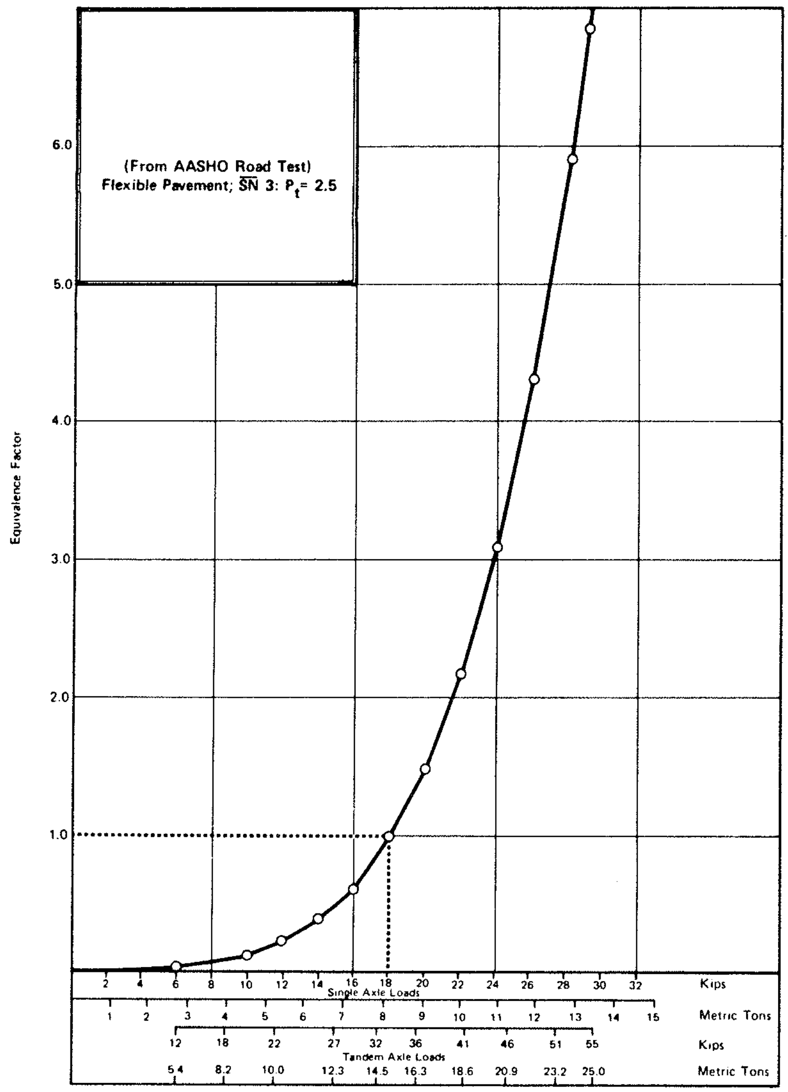
Figure 3.1: Equivalence Factors for Various Axle Loadings



with a median of around 9 m ton (almost 20,000 pounds). This median value of 9 m ton is a good compromise, and is suggested as the legal limit for roads which are to be given permanent pavement structures.

Pavement Design

The design thickness of flexible pavement is affected by the analysis methods used for design. The problem is so complex that despite over 50 years of intensive research, no completely rational method or procedure has been devised. At least a dozen empirical or semi-rational methods have been proposed and used in various parts of the world. In the United States the most generally used methods of thickness design have been the Group Index Method, the California Bearing Ratio (CBR) Method, and the AASHO Road Test Method, the latter predominating at present, by a wide margin. The method proposed by the British Road Research Laboratory and extensively used in African countries is essentially the same as the CBR Method. Another method now undergoing tests in Kenya is an adaptation of the AASHO Road Test Method to African conditions. The most recently developed method of thickness design is the Catalogue of Road Structure Types developed by the French Ministry of Public Works and Housing, which replaced previous methods in France in 1971 and has been proposed for use in Ivory Coast and in other African countries.

At present, the CBR method, in one or another of its variations, appears to be the most widely used in the developing countries. Perhaps the best way to view the pavement design methods now available is to regard them as valuable guides with the results subject to evaluation based on practical experience in the area. (There are a few agencies which use practical experience as the principal determinant for pavement thickness.)

A surprising finding of this study was the extent to which bituminous surfaces are used on roads that carry very low traffic volumes. Figure 2.10 shows that well over half of the kilometrage in the 75 to 200 ADT brackets has a dust-free surface. Such a surface has many obvious advantages, but at added cost. Higher speeds are possible with better utilization of vehicles, fewer accidents and less costly vehicle maintenance. Further, and in many instances most importantly, bituminous surfaces conserve surface aggregates, the loss of which from traffic and weathering may amount to over 4 centimeters per year from a granular surface.

However there are also disadvantages to bituminizing a low-traffic road. Bituminous pavements require a certain amount of traffic to keep the bitumen cement ductile and "alive," otherwise they may become dry and crumbly. More importantly, a light or medium strength subbase or base course may be severely distorted and rutted by heavy wheel loads. When this happens a granular surface can be easily restored to cross-section by adding more material and blading, but for a bituminous surface costly patching and overlays would be required. Even worse, if damage is severe, the bituminous surface may have to be scarified, the base reshaped and the surface replaced.

On balance, it would seem that roads carrying less than 200 ADT should be bituminized only if justified by a realistic economic analysis [1], and then only if the highway authority is prepared to invest in subbase and base courses capable of supporting the heaviest vehicles expected to use the road. Short stretches of road in mountainous terrain might be surfaced, regardless of traffic volume, where vehicle traction and storm water erosion would cause serious damage to an unpaved surface.

Suggested Design Standards for Two-Lane Highways

Taking into account the consensus of standards in the projects studied, and the author's own experience in highway design, Table 3.8 is a summary of suggested standards for the principal geometric elements of two-lane highways. These standards should provide a satisfactory level of service for a reasonable period after opening to traffic, assuming an increase in traffic of about 5 percent to 7 percent per year. Improvements to pavements, particularly in regard to width and type of surfacing, may be necessary at intervals of about seven to ten years, or at shorter intervals if traffic growth is very rapid.

Incremental Development of Highways

Historically the main roads of most countries have developed in stages as traffic increased: first as earth roads, then surfaced with gravel or stone and finally paved for all-weather traffic. Incremental development or stage construction is an important tool for enhancing the effectiveness of scarce supplies of capital in developing areas. There are, however, certain principles that should be observed in the development of highways by this method. The most important of these is that the original location of the highway should be carefully engineered to fit the terrain and the expected ultimate traffic. The potential savings in construction and maintenance expense and in road user vehicle operating costs are too great to leave the location to chance or in unskilled hands.

[1] Likewise the bituminous surfacing of roads carrying *more* than 200 ADT should be justified economically in terms of vehicle operating and road maintenance savings.

Table 3.8: Suggested Design Standards for Two-Lane Highways

Design Class		Class I			Class II			Class III			Class IV			Class V		
ADT on opening (Mixed traffic)		Under 50			50 to 400			400 to 1,000			1,000 to 2,000			Over 2,000		
Terrain		Flat	Rolling	Moun-tain-ous	Flat	Rolling	Moun-tain-ous	Flat	Rolling	Moun-tain-ous	Flat	Rolling	Moun-tain-ous	Flat	Rolling	Moun-tain-ous
Design speed	km.p.h.	60	40	30	80	65	40	90	80	50	95	80	60	100	90	70
Maximum gradient	%	6.0	8.0	10.0	6.0	7.0	9.0	5.0	6.0	8.0	4.0	5.0	7.0	4.0	5.0	7.0
Width of surfacing	m.	4.0 to 5.0 depending on design speed			5.5 to 6.2 depending on design speed			6.0 to 6.7 depending on design speed			6.7 to 7.0 depending on design speed			7.0 to 7.3 depending on design speed		
Width of one shoulder[a]	m.	1.0			1.5			2.0			2.4			3.0		
Total width of roadway[a]	m.	6.0 to 7.0			8.5 to 9.2			10.0 to 10.7			11.5 to 11.8			13.0 to 13.3		
Minimum radius	m.	As in Table 3.4 for 10 percent maximum superelevation														
Non-passing sight distance	m.	As in Table 3.5														
Passing sight distance	m.	As in Table 3.6														
Width of bridges L < 20	m.	3.5 to 4.0 (Single Lane)			8.5 to 9.2			10.0 to 10.7			11.5 to 11.8			13.0 to 13.3		
Between Curbs L > 20	m.	3.5 to 4.0 (Single Lane)			7.0 to 7.7			7.5 to 8.2			8.7 to 9.0			9.0 to 9.3		
Vertical clearance	m.	5.0			5.0			5.0			5.0			5.0		
Design live loading (AASHO or equiv.)		H 15-44			HS 20-44			HS 20-44			HS 20-44			HS 20-44		
Axle load for pavement design (Legal limit)	m ton				9			9			9			9		
Right-of-way width	m.	25			35			40			50			60+		
Surface Type (Assuming an adequate base)		Granular			Granular, Single or Double Surface Treatment			Multiple-layer surface treatment, Bituminous Macadam, Road Mix or Asphaltic Concrete			Asphaltic Concrete or Road Mix			Asphaltic Concrete or rigid pavement		

[a] Shoulder and roadway width may be reduced in rough mountainous terrain.

After the location is selected an adequate right-of-way wide enough to contain the ultimate road facility plus any temporary deviations should be acquired or reserved.

These two operations—the location and the reservation of the right-of-way—cannot be done in stages and must be adequately taken care of before any construction is done. Thereafter, there are a number of possible alternatives that will reduce the investment necessary for the early stages of development when traffic is light and the consequences of failure are correspondingly less. The usual options are:

—Use steeper gradients within the right-of-way to reduce the earthwork quantities

—Use short temporary detours to avoid the deepest cuts and fills

—Use narrower widths of grading and surfacing in the early stages of development

—Use fords, preferably paved, at favorable sites where low water flows predominate but where occasional large flows may occur

—Use ferries, pontoon bridges, or temporary bridges instead of permanent structures at river crossings

—Use one-way bridges or build a single-lane superstructure on a substructure capable of holding a two-lane superstructure

—Omit or defer the wearing course by operating the road first as an earth or granular surface, upgrading it as traffic increases.

It is not advisable to use temporary or undersized minor drainage facilities such as culverts and side ditches, or to reduce grading quantities by using steep, unstable cut and fill slopes.

The most commonly used and probably the most effective place to practice stage construction is in the base and pavement structure. Frequently, inferior but usable materials are locally available which can be used, in greater thicknesses, for subbase and temporary wearing courses instead of higher-quality more costly imported materials. These can be maintained as granular surfaces, adding local material from time to time to replenish the losses of wear and erosion. Eventually a good quality base and a dust-free pavement may be placed on the undisturbed initial subbase which has usually become through traffic compaction a much better subgrade than the original basement soil. For the reasons previously given in the discussion on pavement design, it is important not to change from a granular to a bituminous surface at too early a stage, especially if there are many heavy trucks in the traffic stream.

Levels of Service

The service of a highway to its users is a qualitative measure of the effects of a number of factors such as operating speed, travel time, traffic interruptions, freedom to maneuver and pass, driving safety, comfort and convenience and operating costs. Traffic research engineers seeking an objective measure of the quality of highway service have found two reasonably reliable indicators:

i) Operating speed under the existing traffic conditions; and
ii) the "volume/capacity ratio," that is, the ratio of the volume of traffic being carried by the highway under the existing conditions (service volume) to the volume under ideal conditions (capacity).

These indicators have been used to define six levels of service (LOS) shown diagramatically in Figure 3.2. LOS A is the condition where traffic is flowing freely at high speed and low volume. Speeds are controlled by legal limits, driver desires and physical roadway conditions, rather than by other vehicles in the traffic stream, and drivers can maneuver, overtake and pass practically at will. At LOS B operating speeds begin to be restricted somewhat by traffic conditions, but drivers still have reasonable freedom to select their speed and to maneuver.

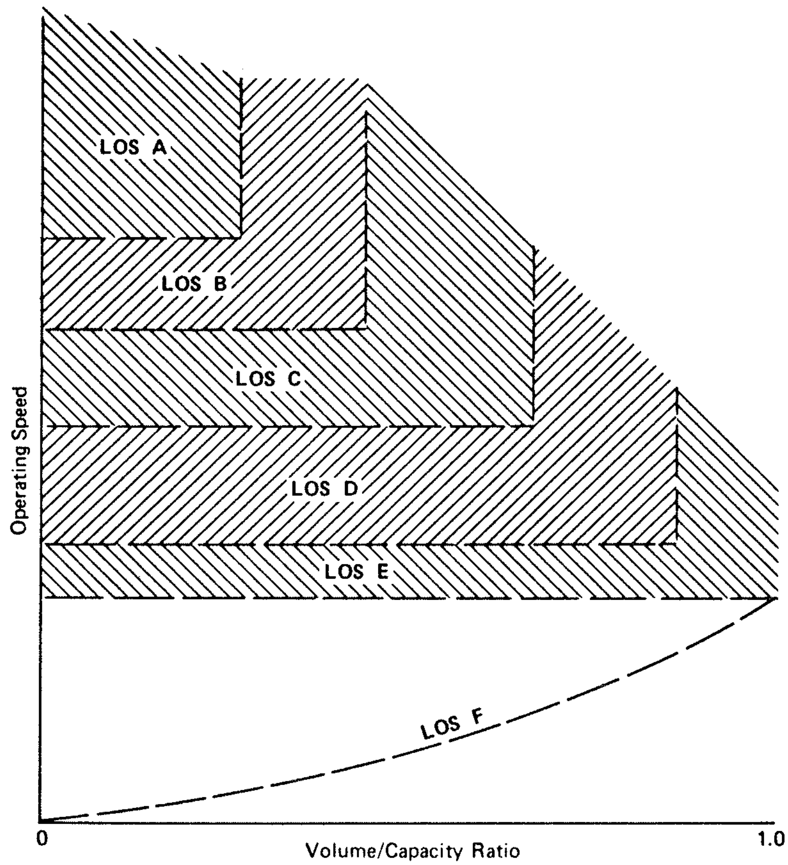
At LOS C traffic is still moving in the zone of stable flow and at a relatively satisfactory operating speed. At LOS D flow is stable but traffic is moving slowly. At LOS E flow is unstable, and speeds are slow, but maximum capacity is attained at about 2,000 vehicles per hour for two-lane two-way roads. LOS F describes intolerable or "forced flow" operation at low speeds and below capacity.

Level of service can be determined for any set of existing conditions that is, traffic volume, percentage of trucks and buses, width of road surface and quality of alignment, by applying known information on driver behavior and highway traffic capacity.[2] Such determinations are well within the accuracy of the basic traffic information available to highway planners and are considerably more accurate than even the best forecasts of future traffic.

A properly designed highway will have, on opening, a surplus of capacity so that as traffic increases with the years it will not become congested prematurely. The planners of new roads or

[2] See *Highway Capacity Manual* Special Report 87, Highway Research Board, Washington, 1965.

Figure 3.2: General Concept of Levels of Service



major reconstructions of old roads should select construction standards such that the residual level of service at the end of the project's planned economic life will not fall below a level acceptable to the users of the highway.

The standards for highways in Traffic Class I and the lower range of Traffic Class II, as defined in Chapter I, depend more on the practical minimum dimensions for roads than on capacity limitations. The level of service concept does not apply to these low-traffic tertiary roads, but for all other roads level of service analysis is a useful tool for selecting standards for proposed projects. Such analysis, taken in conjunction with the estimated traffic growth rate will immediately disclose whether the standards proposed for a given project are within realistic limits. The following levels of service are suggested:

Design Class	Traffic Volume on Opening ADT	Level of Service on Opening	Residual LOS 10 years after Opening
Class I	0-50	LOS Concept not applicable	
Class II	50-400	LOS Concept not applicable	
Class III	400-1,000	LOS B	LOS D
Class IV	1,000-2,000	LOS B	LOS D
Class V	Over 2,000	LOS B	LOS C

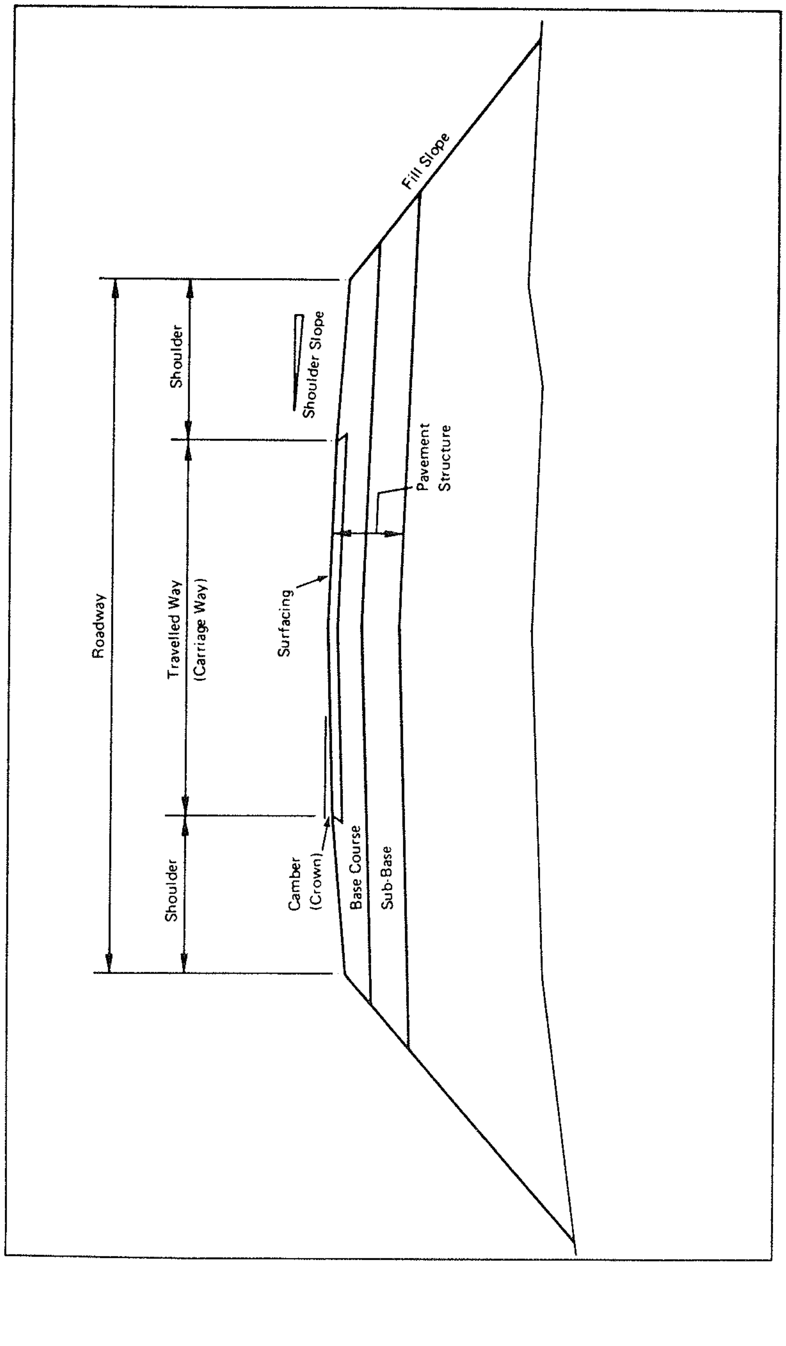
ANNEX I

DISTRIBUTION OF TOTAL SAMPLE BY COUNTRIES

	Country	Length in Kilometers	Percent of Total Kilometers	Number of Line Items	Percent of Total Line Items
1	Argentina	1,925.0	2.78	27	3.16
2	Botswana	651.8	0.94	7	0.82
3	Brazil	4,415.6	6.37	41	4.80
4	Cameroon	295.0	0.43	2	0.23
5	Central African Rep.	102.0	0.15	1	0.12
6	Chile	151.0	0.22	6	0.70
7	Colombia	2,125.3	3.07	59	6.90
8	Rep. of Congo	34.1	0.05	1	0.12
9	Costa Rica	57.1	0.08	3	0.35
10	El Salvador	338.0	0.49	11	1.29
11	Ethiopia	1,625.0	2.35	8	0.93
12	Fiji	115.8	0.17	2	0.23
13	Finland	4,501.5	6.48	14	1.64
14	Gabon	304.0	0.44	5	0.58
15	Guyana	54.1	0.08	3	0.35
16	Honduras	449.0	0.65	8	0.93
17	Iceland	47.4	0.07	8	0.93
18	India	1,059.0	1.53	16	1.87
19	Indonesia	200.0	0.29	1	0.12
20	Iran	4,098.0	5.92	25	2.92
21	Iraq	479.3	0.69	16	1.87
22	Israel	1,143.8	1.65	40	4.68
23	Ivory Coast	2,053.0	2.96	9	1.05
24	Jamaica	42.4	0.06	3	0.35
25	Japan	552.8	0.80	8	0.93
26	Jordan	18.0	0.02	1	0.12
27	Kenya	5,930.8	8.55	116	13.60
28	Korea	390.0	0.56	3	0.35
29	Lesotho	142.3	0.21	4	0.47
30	Liberia	120.7	0.17	3	0.35
31	Malagasy Republic	291.8	0.42	4	0.47
32	Malawi	298.4	0.42	3	0.35
33	Mali	1,444.0	2.08	28	3.28
34	Mauretania	200.0	0.29	2	0.23
35	Mexico	12,946.0	18.54	86	10.08
36	Morocco	526.0	0.76	15	1.75
37	Nepal	0.3		1	0.12
38	Niger	218.0	0.31	6	0.70
39	Nigeria	1,024.4	1.48	21	2.46
40	Pakistan	634.3	0.91	10	1.17
41	Panama	352.0	0.51	16	1.87
42	Papua/New Guinea	157.2	0.23	5	0.58
43	Paraguay	346.3	0.50	3	0.35
44	Peru	656.0	0.95	10	1.17

45	Philippines	160.0	0.23	1	0.12
46	Rwanda	79.0	0.11	2	0.23
47	Senegal	78.0	0.11	4	0.47
48	Sierra Leone	69.2	0.10	1	0.12
49	Somalia	374.0	0.54	2	0.23
50	Spain	764.5	1.10	9	1.05
51	Sri Lanka	1.2		1	0.12
52	Syria	683.0	0.98	6	0.70
53	Tanzania	1,557.6	2.24	15	1.75
54	Thailand	2,171.0	3.12	30	3.50
55	Trinidad/Tobago	27.7	0.04	4	0.47
56	Tunisia	277.0	0.40	12	1.40
57	Uganda	825.3	1.19	15	1.75
58	Upper Volta	97.0	0.14	6	0.70
59	Uruguay	485.0	0.70	3	0.35
60	Venezuela	337.7	0.47	5	0.58
61	Yugoslavia	1,222.6	1.77	31	3.62
62	Zaire	6,737.0	9.72	40	4.67
63	Zambia	978.4	1.41	7	0.82
		69,432.7	100.00	855	100.00

ANNEX 2
HIGHWAY TERMS



Institute of Transportation
and Traffic Engineering

OPPORTUNITIES FOR COST REDUCTION IN THE DESIGN OF TRANSPORT FACILITIES FOR DEVELOPING REGIONS

VOLUME 1

A Study Conducted under Contract No. DOT-OS-A9-004

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University of California
Berkeley, March 1970

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CHAPTER II

ROAD COST ANALYSIS & DESIGN STANDARDS

by

Lawrence Vance

NOTE: The preceding pages of the original text are concerned with general issues and economic considerations that are beyond the scope of this compendium.

D. Geometric Standards

The usual practice in road design is to set standards for:

1) Minimum horizontal curvature

2) Minimum vertical curvature

3) Road widths

a. Minimum lane width

b. Minimum roadbed width

c. and/or minimum shoulder width

4) Sight distances (minimum)

a. Passing

b. Non-passing

c. Approaching (single lane only)

5) Maximum gradient or maximum gradient as a function of grade length.

In addition, geometric standards are set for typical intersection layouts. In this paper intersection layouts will not be discussed as the intersection requirements for low volume rural roads are modest.

All of the geometric features above are normally specified as a function of design speed. Thus, design speed, a non-geometric road design element, will be included in the following discussion.

Balanced design is a keyword in modern road design. By "balance" we mean that all road geometrics should be set for the same design speed and the design speed should be a good approximation of actual operating speeds. The design speed should also be appropriate for the road surface provided. Balance between the design elements is as necessary in developing regions as it is in developed countries. One exception to the balanced design concept is the case of pavement staging where a road is constructed to high geometric standards but initially provided with a low type surface. This type of design unbalance seems justified for short periods of time (a few years) but is generally unacceptable for permanent construction.

Another exeception may be long gradients which force speeds below the design speed.

(1) Design Speed

Selection of a design speed is probably the most arbitrary and at the same time the most basic determination in the design of any road. The basic philosophy in the United States has been to set design speeds consistent with observed traffic speeds under similar conditions. Thus driver behavior and vehicle performance are the fundamental determinants of design speed. The alternative to this approach is to set design speeds in some manner without regard to the vehicle-driver characteristics. This approach leads to a massive enforcement problem if design speeds are significantly below driver-vehicle capabilities and desires. On the other hand, if design speeds are significantly above driver-vehicle capabilities there is economic waste associated with the over designed road.

It would appear that the basic approach to selecting design speeds for developing economics should be similar to the philosophy in the United States. Vehicles have similar capabilities world wide. Driver expectations, however, are probably somewhat different in emerging nations. As was the case in the U.S. during the development of the road network, drivers unaccustomed to high standard roads do not drive at high speeds even though their vehicles are capable of such speeds. Thus we find U.S. rural average speeds have increased steadily for many years yet maximum vehicle speed capabilities have changed much less. In part, this increase is due to improved roads but the expectations and education of drivers play a major role as well. Elimination of slow traffic such as animal drawn vehicles has also contributed to increased average speeds.

High design speed should be the exception not the rule for low volume roads in developing countries. As concluded in the 1955 PIARC Xth Congress:

"The main function of lightly trafficied roads is to provide access to main arteries rather than to allow high speed travel."

Significant cost savings can be realized by reducing geometric standards. Particularly, first costs of construction can be reduced although at the expense of higher vehicle and road operating costs.

In many developing countries roads must be shared by animal drawn vehicles and motor vehicles. Until separate ways can be provided for animal traffic, it appears unwise to construct high-speed facilities for joint use.

(2) Road Widths

Generally roadway construction costs increase with increasing width. In mountainous or other rough terrain where excavation is a major cost, changes in road width have a substantial effect on total cost.

Oglesby and Altenhofen* conclude that the 1965 AASHO standards of width for low volume, rural roads are excessive. They suggest lower standards with specified maximums rather than minimums. The trend of AASHO road & lane width recommendations from 1940 to 1965 is to wider lanes and shoulders and to elimination of single lane roads and pavements. AASHO 1965 desirable minimums are 12-foot lanes and 10-foot shoulders with absolute minimums of 9' lanes and 4' shoulders under low volume, low speed conditions with passenger vehicles only.

U.S. road width standards are important to the developing countries because our standards are repeatedly suggested as globally appropriate. It is evident from the standards actually used for low-volume U.S. roads that the AASHO width standards must be viewed as optimal not as minimums. Oglesby and Altenhofen have collected U.S. road standards for 2-lane and single-lane roads. Their plot of roadbed width standards for various traffic volumes is shown in Figure 10. It is evident from the figure that the present AASHO standards are high relative to rural practice. It is not evident why this is so from this plot. In Figure 11 the Oglesby and Altenhofen data have been plotted with design speed as a function of road width. Here it is apparent that all AASHO minimum roadway widths are adequate for speeds to about 60 mph although the design speed specified is lower. AASHO recommendations are based on U.S. experience where rural speeds reach 60-70 mph on lightly traveled roads wherever possible. Even on rural roads with geometric constraints reducing speeds to 20-30 mph, 60-70 mph speeds are observed on tangents between constraints. Thus we conclude AASHO width standards are based on the requirement of providing for high speed traffic regardless of established design speed.

* Oglesby, C. H. and I.A.J. Altenhofen, THE ECONOMICS OF DESIGN STANDARDS FOR LOW VOLUME RURAL ROADS. Stanford University, Report EEP-26, July 1967.

For developing countries the current AASHO width standards are excessive at least for initial rural road development. The Oglesby-Altenhofen data for pavement width is plotted against roadbed width in figure 11. In addition, some examples of foreign standards are plotted. The width recommendations of the Low Cost Roads section of the Permanent International Association of Road Congresses are also plotted. The PIARC low cost road width recommendations were made at the Istanbul meeting in 1955 as follows:

> "31. For single lane roads effective widths of 3.00-3.50 meter surfacings are considered adequate with wide enough shoulders to provide passing of cars coming from the opposite direction. For two-lane roads, effective surface widths within the range of 6-7 meters are considered.
>
> 32. Shoulders should be wide enough to prevent hazard to traffic. If possible 1.50-2.50 meter shoulders are recommended."

It is of interest to note that the original committee recommendation was for a width of 5.5 to 6.0 meters for two lane roads.

Roadbed and lane widths are a function of the dimensions of vehicles using the road, the speed at which they operate and the number of vehicles using the road per unit time. Usually roadway widths decrease as the terrain through which they pass becomes rougher. This is for economic reasons rather than technical.

Vehicle maximum width dimensions are regulated by virtually all nations. There is no international maximum standard width but of 77 countries surveyed by the International Road Federation, 49 have adopted 2.5 meters (8.2 feet). The Pan Am Highway Congress and the 1949 Geneva Convention also have adopted 2.5 meters. In Africa two countries use 2.3 meters (7.55 feet). In Europe, Norway uses a 2.20 meters (7.2 feet) limit and Switzerland a 2.3 meters (7.55 feet) limit. In South America and Asia, there is more variation from the 2.5 meters norm. Brazil, Venezuela and Iraq all have adopted a 2.6 meters (8.53 feet) width consistent with AASHO recommendations in the United States. In South America, Bolivia and Guyana have the lowest widths of 2.30 meters (7.55 feet) and 7.5 feet (2.29 meters) respectively. In Asia, Malaysia has the lowest width standard at 7.5 feet (2.29 meters).

Maximum vehicle widths set a minimum on roadway and lane widths. The absolute minimum roadbed width for one way operation is 9 feet and for two-way operation is 18 feet assuming a 2.5 meter vehicle. For single lane roads, intended for two way operation, a 9-foot width can be used with passing places provided at frequent intervals. Scottish practice on single lane roads in the highlands is to make the roadway 8-10 feet wide with passing places along about 60% of the road length. (The U.S. Army expedient road standards require a turnout at least every 1/4 mile[traveled way - 12' minimum.]TM 5-330). Three U.S. states still retain specifications for un-surfaced roads with widths from 14 to 16 feet. Such widths were considered adequate for two-way traffic in the United States prior to 1920. In that period 12 feet was considered the minimum possible width for two-way operation. Many state and federal specifications of the period used widths from 12 to 18 feet. In the early 20's, width specifications were increased, the 18 foot roadbed becoming a generally accepted minimum for two-way operation. Any roadbed width less than 18 feet today must be considered inadequate except for single lane roads and special circumstances such as roads used exclusively by vehicles narrower than 2.5 meters.

The speed of traffic has been recognized as a basic determinant of roadway width since motorized traffic became significant. In figure 13, Oglesby & Altenhofen's data on U.S. minimum road standards is used to show the relation between lane width and design speed. "Eyeball" regression curves indicate the relationships for three topographic types (mountainous, rolling and flat) for both paved surfaces and unpaved surfaces. It is evident that lane widths increase with design speed. Such standards reflect the designers' assumption that operation of vehicles at high speeds requires a greater lane width for safety than does low speed operation. As Oglesby & Altenhofen* have pointed out, this assumption has not been factually established for low-volume facilities.

Accepted practice among road designers has been to reduce geometric standards including roadbed width in rolling and particularly mountainous terrain. This is a recognition of the higher costs of a given standard in rougher terrain. Roadbed width reductions in rough terrain are primarily reductions in shoulder width for paved roads. There will also be some possible reduction in lane widths where design speeds are reduced significantly.

High volume traffic requires a wider roadbed according to traditional design practice. The primary requirement has been to provide shoulders of sufficient width that disabled or otherwise stopped vehicles can completely clear high volume traffic.

Roadway surface has a correlation to roadbed width. Loose surfaces are generally associated with lower speed operation and, thus, narrower roadbeds are adopted. Further, loose surfaces do not allow lane marking. It has been well established that lane markings will channel traffic.** At low volumes vehicles on unmarked surfaces will tend toward the center of the roadbed except when passing, while marking of lanes will channel the traffic more nearly to one side of the road. For low volume traffic - less

* Op Cit, page 69

** HRB, Bulletin 170. TRAFFIC BEHAVIOR AS RELATED TO SEVERAL HIGHWAY DESIGN FEATURES, Washington D. C., 1958.

than 100 vpd -- a narrow roadbed is acceptable even if opposing traffic must reduce speed to pass, since passing is infrequent at such volumes. (For a 5-vehicle per hour volume, vehicles pass every 12 minutes on the average. At 30 mph, this is once every 6 miles.)

(3) Gradients

The absolute maximum gradient which any powered vehicle can negotiate is determined by the coefficient of friction between the wheels and roadway, assuming adequate power is available. Under ideal conditions, this may allow operation on grades as high as 90% and some muds may reduce the maximum to 20% or below. Ice and snow can reduce the coefficient of friction to the point where a 5% grade is the maximum negotiable.

20% appears to be the maximum reasonable gradient for motor vehicle operation under favorable conditions. Grades of 20% will reduce the speed of all motor vehicles if the grade is of significant length. Some of the larger trucks with low power-to-weight ratio may be incapable of negotiating a long 20% grade.

U. S. practice in setting maximum gradient standards is to provide for a high level of service. In practice, high standard roads are not designed with grades which will significantly reduce passenger vehicle speeds below the design speed. On two-lane roads, gradients are either reduced to allow trucks to maintain design speed or, where not economically feasible, passing lanes are provided on ascending grades so that passenger vehicles are not impeded by slow-moving trucks. In practice, this level of service requires maximum gradient standards of 3-4% for design speeds in excess of 65 mph. AASHO 1965 standards for grades on rural highways are as follows:

RELATION OF MAXIMUM GRADES TO DESIGN SPEED

Main Highways

Type of topography	Design Speed, mph 30	40	50	60	65	70	75	80
Flat	6	5	4	3	3	3	3	3
Rolling	7	6	5	4	4	4	4	4
Mountainous	9	8	7	6	6	5	--	--

The level of service assumed in the AASHO standards appears excessive for low volume roads in developing regions. In Martinique, for instance, gradients as high as 30% are used with 20%-25% more common. Present practice is to use a maximum of 20%.*

UNESCO**suggested standards are similar to AASHO but extend maximum gradients to 12%.

It would appear that grades of 10%-20% are permissible for the lowest types of roads where such grades are short. Gradients over 10% should only be used in temperate or tropical climates and such gradients should be paved to provide adequate traction. For higher speed, high volume facilities the AASHO standards are more appropriate. The grade length criteria used by AASHO is conservative and probably should be revised where large, low-power-to-weight ratio trucks are not operated. See Figure 14.

* M. Deschamps quoted in PROCEEDINGS XIth International Road Congress, Rio de Janerio, 1959, page 215.

** UNESCO, ROUTES DANS LES PAYS EN VOIE DE DEVELOPEMENT, Editions Eyrolles, Paris, 1968

(4) Sight Distance

Three sight distance conditions must be considered:

1. Passing sight distance.
2. Stopping sight distance.
3. Converging sight distance (single lane facilities only).

Sight distance criteria for conditions 1 & 2 above are adequately covered by AASHO. The third condition applies only to single-lane facilities and is not considered by AASHO. The necessary criteria for converging sight distance has been published by W. H. Valentine in RURAL AND URBAN ROADS*. His article includes design charts similar to those published by AASHO for the other sight distance conditions.

The sight distance standards developed by AASHO & Valentine are appropriate for use on low volume rural roads, assuming reasonable design speeds are adopted, since sight distance criteria depend only on vehicle performance, driver performance and design speed. Average vehicle performance and driver performance will not change enough, worldwide, to significantly alter sight distance standards.

(5) Horizontal Curvature

The minimum possible road curvature is governed by the turning radius of the vehicles using the road. The AASHO passenger design vehicle requires a minimum road radius of 20 feet while the large semitrailer truck requires a minimum road radius of about 32 feet.

For expedient roads, the U.S. Army specifies a desirable minimum

* Valentine, W. H. "A Safe Sight Distance Requirement for Un-Laned Rural Roads", RURAL AND URBAN ROADS, February 1968, p. 34-35, 52.

radius of 150 feet (38.2°).* In Mexico, low standard roads are constructed with a minimum radius of 86 feet (67°).** The UNESCO*** suggested standards for local roads include a minimum standard radius of 25-35 meters in mountainous terrain, 35-75 meters in rolling terrain and 50-60 meters in flat terrain.

AASHO recommends road curvatures as a function of design speed and superelevation. Since the relationships are based on the physics of motion for a typical vehicle, they are appropriate for all road types. AASHO design curves for horizontal curvature do not include radii for non-superelevated curves. Such curves will prevail on low standard roads. The necessary curve can, however, be calculated from formulas provided in the manual.

(6) Vertical Curves

Vertical curvature is usually not standardized for low standard roads. Where vertical curvatures need to be specified, the AASHO methods, based on sight distance, are appropriate. If vertical curvatures are required for single lane facilities, the converging sight distance would need to be considered.

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* U.S. Army, PLANNING, SITE SELECTION, AND DESIGN OF ROADS, AIRFIELDS, AND HELIPORTS IN THE THEATER OF OPERATIONS - TM-5-330. Headquarters, Department of the Army, July 1963.

** Rodolfo, Felix V. and Jorge Gutierrez, RURAL ROADS IN MEXICO, a report to the II Pacific Regional Conference of the International Road Federation, Tokyo, 1964.

*** UNESCO, Op-Cit.

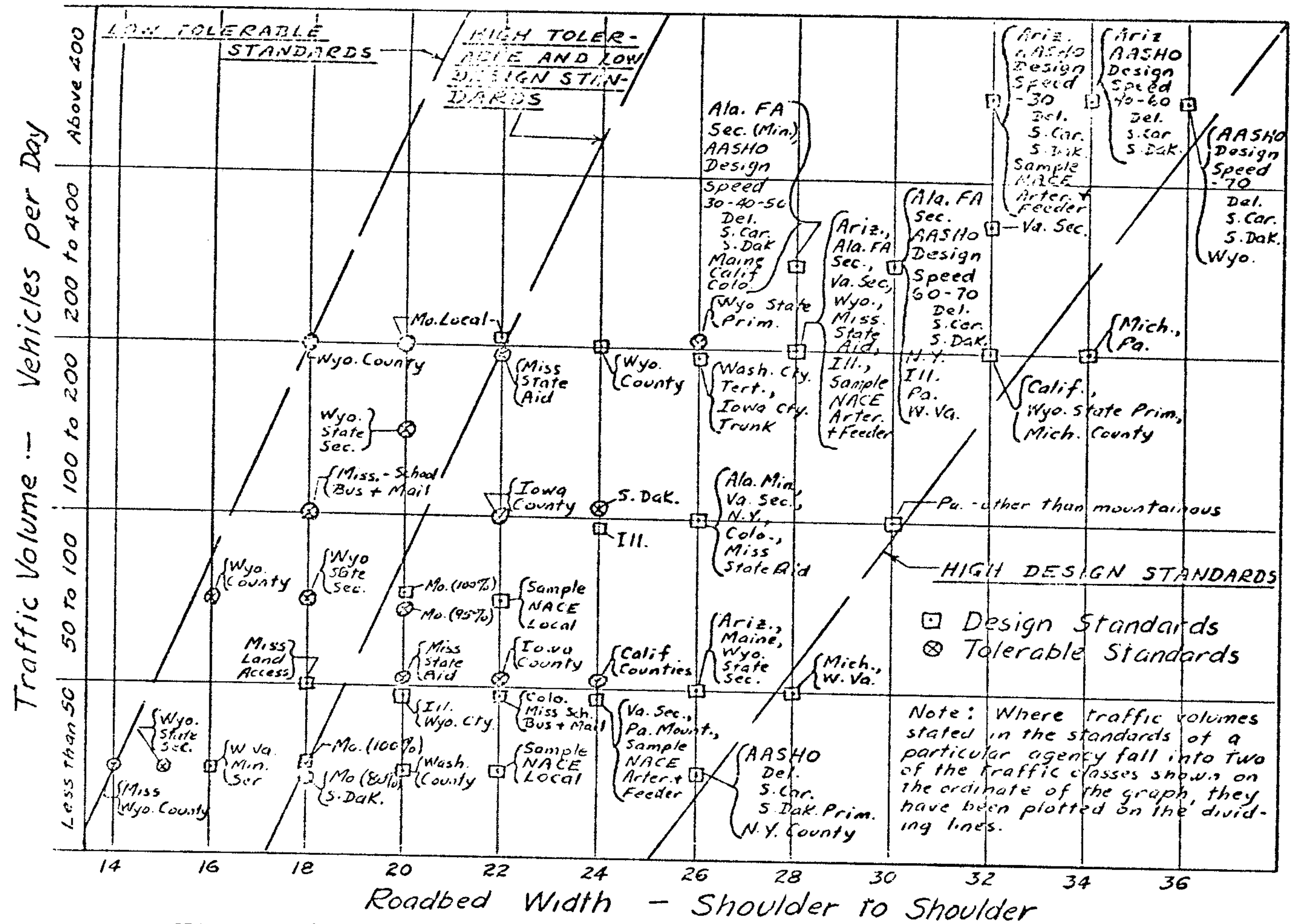

FIGURE 10 - Tolerable and Design Standards for Roadbed Width
(Based on data from a selected group of states)

From Oglesby and Altenhofen

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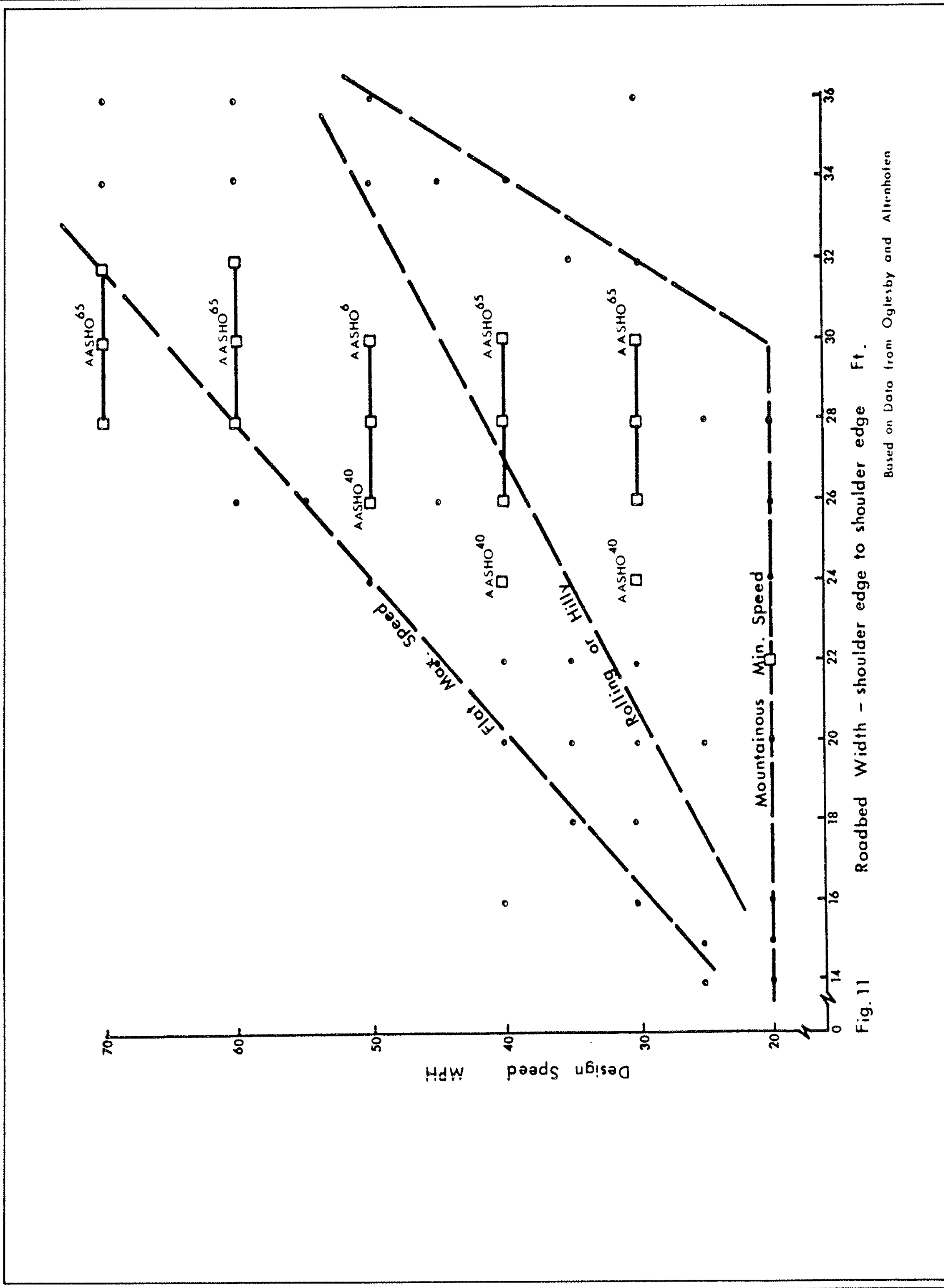
Design Speed MPH
70
60
50
40
30
20
0
14
16
18
20
22
24
26
28
30
32
34
36
Roadbed Width – shoulder edge to shoulder edge Ft.
Flat Max. Speed
Rolling or Hilly
Mountainous Min. Speed
AASHO65
AASHO6
AASHO40
Based on Data from Oglesby and Altenhofen

Fig. 11

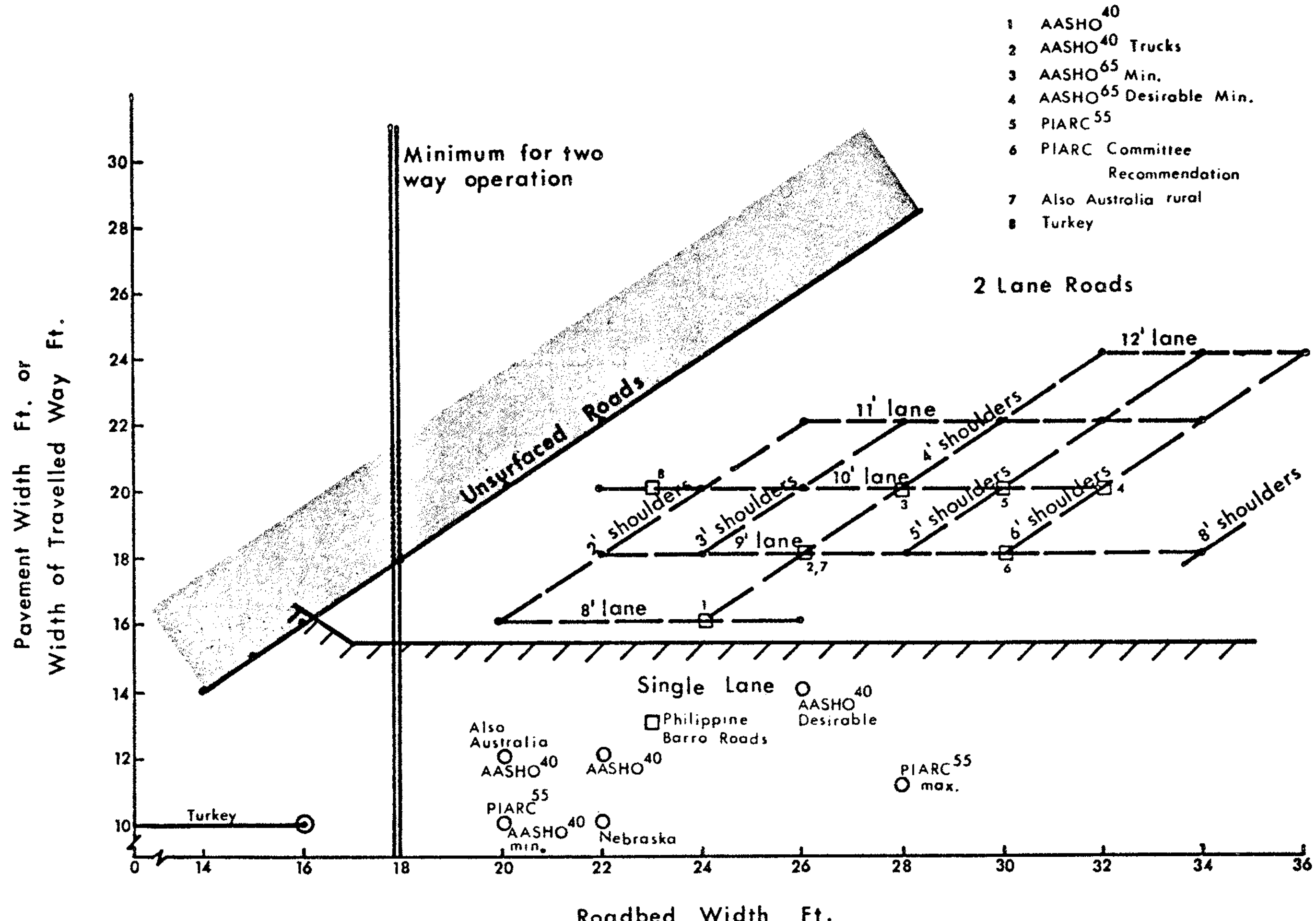

Fig. 12

Based on Data from Oglesby and Altenhofen

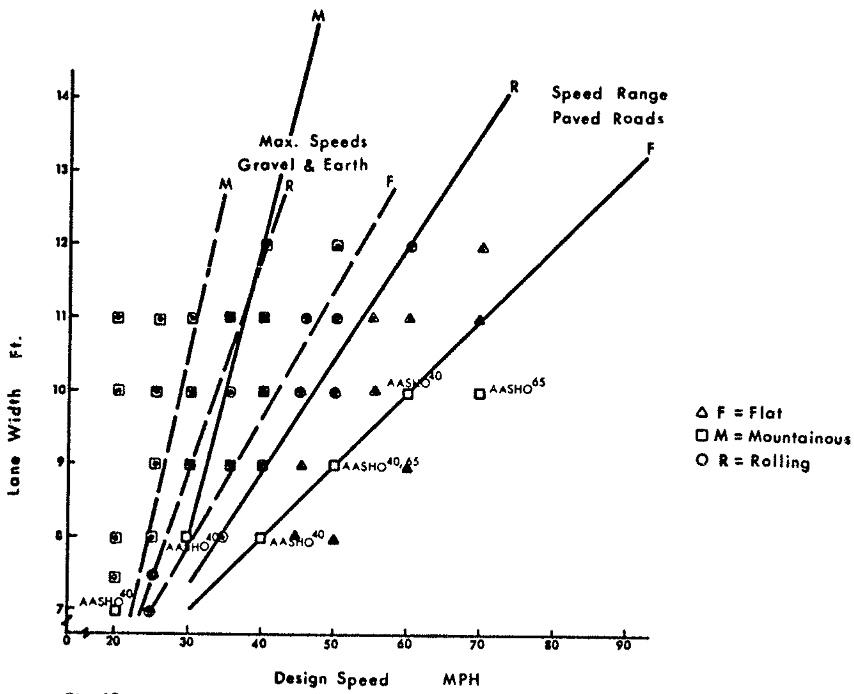

Fig. 13

Based on Data from Oglesby and Altenhofen

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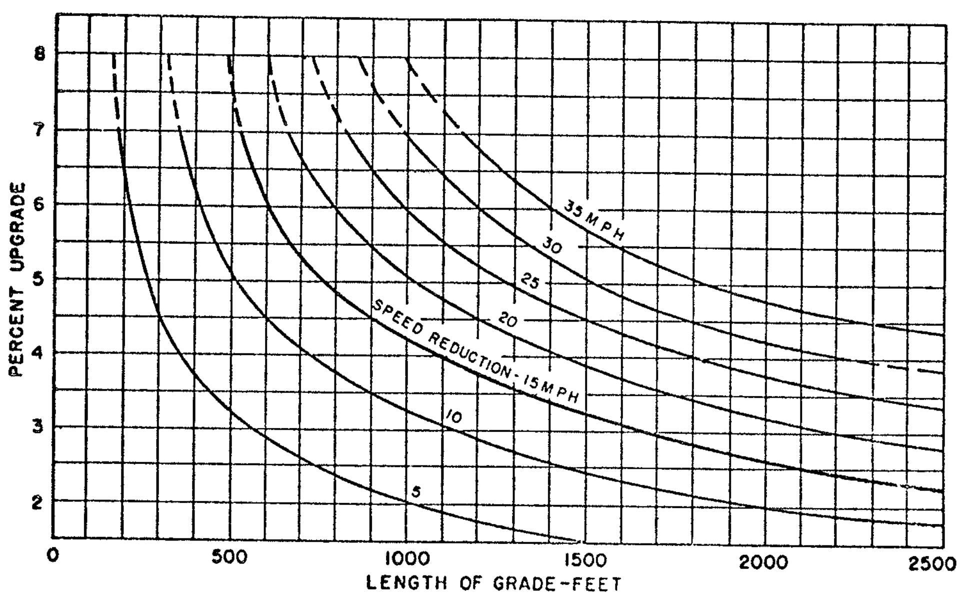


CRITICAL LENGTHS OF GRADE FOR DESIGN

ASSUMED TYPICAL HEAVY TRUCK OF 400 POUNDS PER HORSEPOWER

FIGURE 14

From AASHO

RURAL & URBAN

ROADS "U.S. REG. PDG."

FEBRUARY, 1968

ADMINISTRATION

ENGINEERING

MAINTENANCE AND OPERATION

DEPARTMENTS

Featuring

Rustic Look for Local Road

There are people who want a good road but just don't like all that smooth black asphalt spread between their green lawns. There is one solution to their problem—two courses of limestone chip-seal topped off with a course of crushed bank gravel to provide a "rustic" touch to the neighborhood.

Mini-sized Shopping Mall

Does your shopping district need sprucing up, but you don't want to go all the way to a full-fledged central shopping mall—well here is one solution to your problem, the mini-mall. Read how a small city combined the attractiveness of a "plaza effect" with some through traffic and limited vehicular parking to improve a business district that was on the way downhill.

Slurry Sealing Rural Roads

Slurry sealing has been used extensively in municipal areas as an economical resurfacing tool, providing a surface falling somewhat between a chip seal and a thin bituminous overlay. The State of Kentucky has been moving out onto long stretches of rural 2-lane roadways, using slurry machines to provide both resurfacing and de-slicking of the old asphalt pavements, and reports good results too!

Pavement Props

Got a concrete pavement that has its ups and downs? Here is a description of the proper step in "jacking" the slabs back into their proper place. Contains a description of corrective action for a variety of slab displacements—very useful.

Coming

Sodium Chloride and Calcium Chloride mix at the touch of a button.

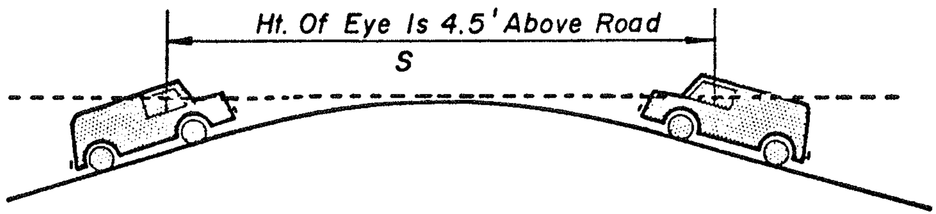
243

Cover

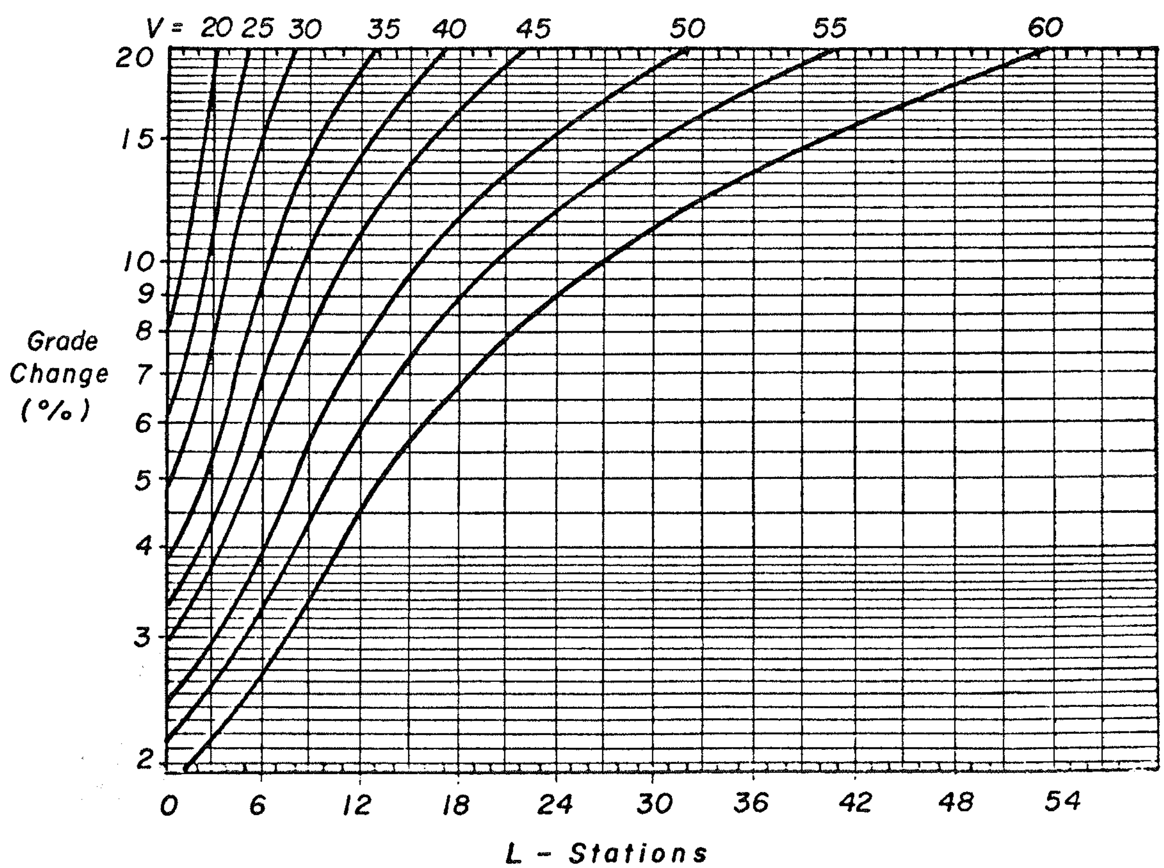
This Flaherty self-propelled chip spreader is moving right behind the asphalt distributor to lay down the seal with no "lost" time.

A Safe Sight Distance Requirement

Stopping Sight Distance
Crest Vertical Curves, Single-Track Operation



DESIGN VMPH	20	25	30	35	40	45	50	55	60
SIGHT DIST. Ft.	225	300	375	475	550	625	750	850	975



Rural & Urban Roads, February, 1968

By W.H. Valentine, P.E.
Forest Engineer
U.S. Forest Service
Elko, Nevada

For Un-Laned Rural Roads

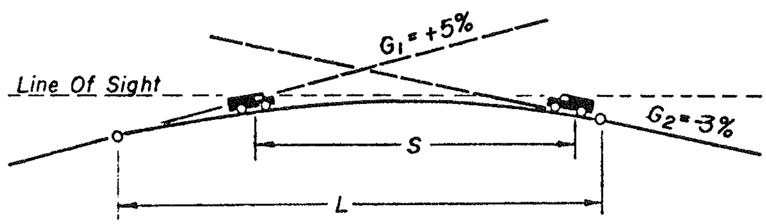
ENGINEERS have given much attention to designing safety into our modern high-volume freeways and highways. Their success is demonstrated by the decreasing accident rates on these highways.

Adequate sight distance is one of the first requirements in designing for safety. It is obvious that drivers must first see hazards to react safely to them. Consequently, all new highways are designed to provide adequate room for stopping a vehicle which is approaching a safety hazard—stopping sight distance. Likewise, passing opportunities are provided with adequate sight distance on two-way roads.

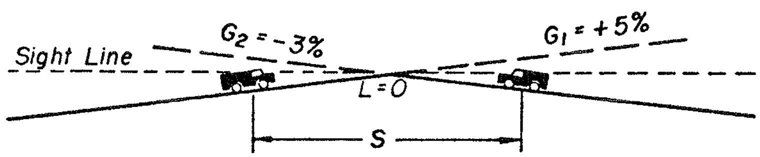
These two safety requirements —stopping and passing sight distances—are given appropriate attention in new road design. But the emphasis has been confined to high-volume highway situations. A real, existing, and potential hazard has not been recognized for rural low-volume roads—two vehicles approaching each other *in the same lane of travel!*

Figure one illustrates the problem, which is magnified by the perverse propensity of the average driver to drive down the *center* of a road that lacks lane control. Thousands of miles of paved and gravel-surfaced roads fall into this category. Depending on factors such as horizontal alignment and surface characteristics, these roads are suitable for speeds of 40 mph, 50 mph, and higher.

The hazard of two vehicles approaching one another at these speeds over blind summit curves in the same lane has not received proper attention from rural road authorities. Consider the large number of rural roads that have been paved in recent years without first improving vertical alignment. And, having been paved, still lack lane control marking.



Example 1: $V = 50$ mph; S From Table I = 750, $\Delta G = G_2 - G_1 = 8$
$S < L$ $L = \Delta G(S)^2/3600 = 8(750)^2/3600 = 1249.6$
Value Of L Taken From Chart = 1250.0
Note: If Computed L Is Less Than S, It Must Be Refigured Using Formula For $S > L$: $L = 2(S) - \frac{3600}{\Delta G}$
Chart Values Have Been Corrected For This Situation And Can Be Used Directly.



Example 2: $V = 20$ mph; S From Table I = 225; $\Delta G = G_2 - G_1 = 8$
$S < L$ $L = \Delta G(S)^2/3600 = 8(225)^2/3600 = 112.4$ Ft.
Here $S > L$ So Recompute using Formula 4
$L = 2(S) - 3600/\Delta G = 2(225) - 3600/\Delta G = 0$
Theoretically No Vertical Curve Is Necessary As Sight Line Is Above Vertex Of Grade Intersection.
Value Of L Taken From Chart = 0
Chart Value Automatically Corrects For The Change From $S < L$ To $S > L$.

In arriving at a suitable sight distance, it is necessary to use the sight path for the passing sight distance requirement and the stopping distance developed for the particular operating speed.

Table I shows total single lane sight distance requirements for various speeds.

Figure 2 provides a rapid means for selecting the required minimum length vertical curve for "Head on" stopping sight distance requirements. Transition from S <L, to S> L, has already been made in the curve computations.

Not only can these curves be

(Continued on page 52)

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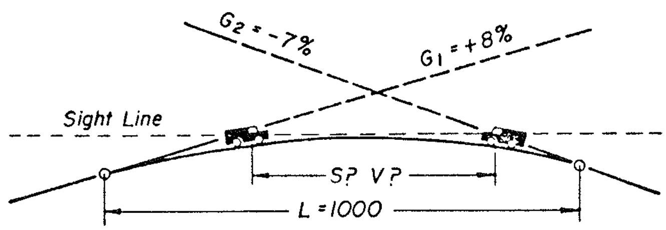
SAFE SIGHT DISTANCE *(Continued from page 35)*

used to determine design vertical curves for desired speeds and grades, they can also be used to determine what speeds are safe for existing roadways where no structural changes are to be made.

These examples are shown to indicate the usefulness of the chart in general highway work on rural roads. ■



Example 3: Existing 1000 Ft. Vertical Curve; $\Delta G = G_2 - G_1 = 15$
Determine The Minimum Available Sight Distance
And The Maximum Speed For That Sight Distance.

$$S < L \quad S = 60\sqrt{\frac{L}{\Delta G}} = 60\sqrt{\frac{1000}{15}} = 490$$

From Table I — *For S = 475 : V = 35*
For S = 550 : V = 40
Use V = 35mph

Solving Problem Directly From Chart:
For L = 1000 And ΔG = 15 : V Falls Between 35mph And 40mph –
Use V = 35 mph

Table I—Safe Sight Distance for Two Vehicles Approaching "Head On"

Design Speed	Assumed Speed	Single Vehicle Stopping Distance	Two Vehicle Stopping Distance	S. Rounded For Design
20	20	108	216	225
25	25	140	280	300
30	28	176	352	375
35	33	228	456	475
40	36	262	524	550
45	40	313	626	625
50	44	369	738	750
55	48	432	864	850
60	52	492	984	975

With the relationships shown in Table I, a solution for the minimum length vertical curve (L) for any grade change (ΔG) can be developed using these well-known equations:

for S > L

(1) $S = \frac{L}{2} + \frac{1800}{\Delta G}$

(4) $L = 2(S) - \frac{3600}{\Delta G}$

for S < L

(2) $S = 60\sqrt{\frac{L}{\Delta G}}$

(3) $L = \frac{\Delta G(S)^2}{3600}$

Where S = Safe sight distance for two vehicles approaching "head-on"

L = Minimum length of vertical curve required to provide this S at a particular grade change

ΔG = Algebraic difference between grade change 2 and grade change 1 ($\Delta G = G_2 - G_1$)

NATIONAL ASSOCIATION
OF
AUSTRALIAN STATE ROAD AUTHORITIES

POLICY FOR GEOMETRIC DESIGN OF RURAL ROADS

Fourth Edition
1970

FOREWORD

The National Association of Australian State Road Authorities works towards uniformity of practice in respect of design, construction and user aspects of roads and bridges, and with this purpose in view, it arranges for the preparation and publication of standards and general procedures.

The principles of design stated in the Policy are based on mathematical calculations, the results of experiments in this country and overseas, or stem from the experience of road design engineers. The method of presentation is to discuss an aspect of design in each section, such as Sight Distance or Vertical Curves, and show its application to undivided and divided roads.

The first edition of the Policy was published in 1955 and reissued in slightly revised forms in 1961 and 1967. This fourth edition has been enlarged to include the design of divided rural roads (excluding freeways and expressways) as well as two-lane rural roads. Roads and intersections in villages and towns which are lit by street lights are not discussed.

A bibliography of books consulted in the preparation of the Policy is given at the end of the book while a table of Notation at the beginning of the Policy lists the symbols used in the formulae quoted.

The technical terms used have the meanings assigned them in Australian Standard A16-1965, Terms Used in Road Engineering.

November, 1970

CONTENTS

Section 8—GRADES

8.1 GENERAL

What is usually sought for the higher design speeds on two-lane rural roads are grades which will allow a *design* motor car in top gear to maintain the design speed whilst climbing or descending without braking. Such grades are usually too steep for trucks or low-powered cars to ascend in top gear.

Steep grades cause disparity in the speeds of vehicles using a road and this leads to overtaking being attempted more often in hilly country than on level terrain.

8.2 UPHILL SPEEDS OF VEHICLES

On uphill grades exceeding 3 per cent some trucks and small cars cannot remain in top gear and maintain the speed of the design motor car. If the grade on a road is steep and the volume of slow traffic is heavy, a climbing or auxiliary lane will be needed for the traffic retarded by the grade—see Section 10.6.

The speed of the design car on various grades has been assessed. Tests were made on the performance in top gear of new 1963 model cars carrying a load of 3 hundredweight to learn their maximum speeds on various grades of sealed roads. From the results of these tests a design car was selected which is typical in hill-climbing power of the average new car using Australian roads in the mid-1960s. These results relate to new cars, the performance of used cars would be poorer.

The maximum upgrade which the design car can climb in top gear at the design speed is shown in column (a) of Table 8.1.

The design car could change over the years. There is a general increase in the developed horsepower of vehicles. These changes suggest that the design car of the future will have higher brake horsepower than it has now. Against this conclusion, however, smaller cars may become more popular and lower the horsepower of the design car.

8.3 DOWNHILL SPEEDS OF VEHICLES

Heavy commercial vehicles travelling downhill under favourable conditions of alignment and sight distance often travel at speeds greater than their usual running speed on the flat provided the grade does not exceed 6 per cent. On downhill grades steeper than 6 per cent truck drivers tend to reduce the speed of their vehicles.

The speed at which cars travel downhill varies with individual drivers and depends upon the steepness of the grade, alignment, visibility, and a driver's feeling of safety.

TABLE 8.1
General Maximum Grades

Design Speed—mph	Column (a) Maximum Upgrade[1] —per cent	Column (b) Maximum Downgrade[2] —per cent	Column (c) General Maximum Grade[3]—per cent
20	—	—	10[4]
30	9½	4½	6[5]
40	9	5½	6[5]
50	7	7	7
60	5	8	5
70	3	9½	3
80	—	—	3[6]

NOTES:
1. Maximum upgrade is that which a design vehicle can climb in top gear at the design speed.
2. Maximum downgrade is that which allows a car weighing 4000 pounds in average mechanical condition to descend at constant speed in top gear against engine compression.
3. General maximum grade is the lower value of columns (a) and (b).
4. At 20 miles per hour, a gear other than top gear has to be engaged for climbing and falling grades.
5. The values of 6 per cent for 30 and 40 miles per hour are slightly above the test figures. On a 6 per cent grade only light applications of the brakes are needed to maintain speeds of 30 or 40 miles per hour and therefore a 6 per cent grade is accepted for these speeds.
6. A general maximum grade of 3 per cent for 80 miles per hour design speed is considered appropriate by Australian and overseas authorities.

A desirable characteristic of a downhill grade is that a vehicle can maintain the design speed against engine compression in top gear without braking. For a car weighing 4000 pounds gross, in average mechanical condition, the constant speed attained downhill on a sealed road is given in column (b) of Table 8.1.

Grades used in design are set not only by vehicular performance but also by terrain.

8.4 GENERAL MAXIMUM GRADES

General maximum grades for each design speed or the steepest grade that the majority of cars can travel in top gear without loss of speed uphill and without gain downhill are shown in columns (a) and (b) respectively of Table 8.1. The general maximum grade is shown in column (c) as the lower value of the maximum uphill and downhill grades.

8.5 GRADES STEEPER THAN THE GENERAL MAXIMUM

8.5.1 General

At locations where flattening of the grade to the general maximum value would be costly or would significantly increase the length of the route, steeper grades may be used provided studies show a financial advantage in running costs, and the curvature can accommodate the downhill speeds that will result. Australian experience suggests a figure of 2 per cent above the general maximum grade is acceptable over short lengths in road design work.

The length of grade steeper than the general maximum which may be used depends upon the acceptable departure from the design speed. For ascending traffic the increase in grade should not result in a significant drop in speed. For descending traffic the speed should not increase to an extent which requires checking by an excessive use of brakes. These two requirements will generally be met if the ascending speed does not fall by more than 20 per cent and the descending speed is not increased by more than 10 miles per hour.

The acceptable length of rising grade steeper than the general maximum may be determined mathematically. The acceptable length of downgrade is found from road tests.

8.5.2 Uphill Grades—Excess Gain in Height

In making calculations relating to grades steeper than the general maximum, the height above that grade by which a vehicle is lifted is referred to as the excess gain in height—see Fig. 8.1.

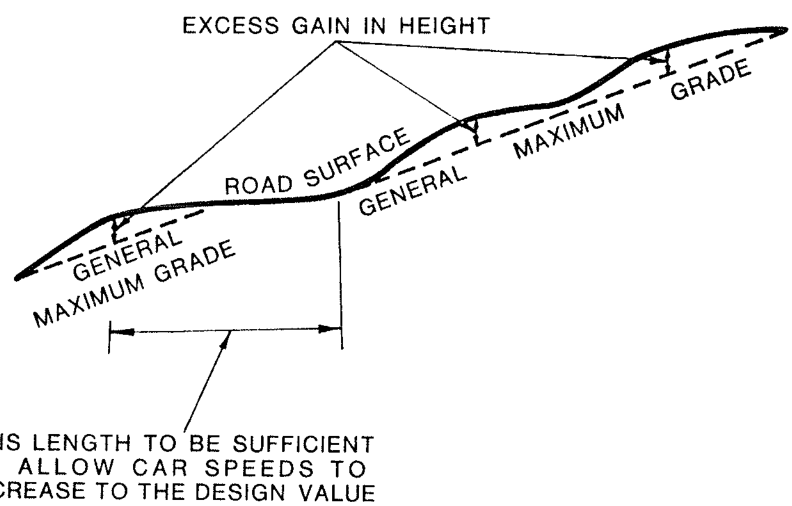
The excess gain in height is calculated as follows:

(i) If the engine output is fully used in climbing the general maximum grade, the energy available to lift a vehicle through an additional height is—

$$wh = \frac{wv^2}{2g}$$

where w = weight of vehicle (pounds).
h = additional height (feet).
v = speed of the vehicle (ft/sec).
g = acceleration due to gravity (ft/sec²).

plus a factor arising from the additional energy available owing to rotating parts,



LONGITUDINAL SECTION OF ROAD

Method of Applying Short Lengths of Grade Steeper than General Maximum Grade

Figure 8.1

(ii) The additional energy factor for cars may be taken as 5 per cent of the kinetic energy developed so that

$$wh = 1{\cdot}05\left(\frac{wv^2}{2g}\right)$$

(iii) Simplifying and converting speed to miles per hour, the excess gain in height which would cause a vehicle's speed to drop from a speed of V_1 at the beginning of a steep grade to V_2 at its end is:

$$H = 0{\cdot}035(V_1^2 - V_2^2)$$

where H = excess gain in height (feet).
V_1 = vehicle speed at start of grade (mph).
V_2 = vehicle speed at end of grade (mph).

This formula is used to find values of the excess gain in height which cause a 20 per cent drop in speed on steep ascending grades in the higher design speeds. Values for speeds of 50, 60, 70, and 80 miles per hour are shown in Table 8.2.

TABLE 8.2

Allowable Excess Gain in Height

Design Speed at Start of Grade	Speed at End of Grade Steeper than General Maximum	Excess Gain in Height
mph	mph	feet
50	40	32
60	48	45
70	56	62
80	64	81

The allowable length of grade steeper than the general maximum grade can be calculated using the values given in Table 8.2, for example:

For a 60-mile-per-hour design speed with a general maximum grade of 5 per cent, the allowable length of 7 per cent grade which may be used is—

$$\frac{\text{Excess gain in height}}{\text{Increase in grade per cent}} = 45 \times \frac{100}{2} = 2250$$
$$= \text{say } 2200 \text{ feet.}$$

8.5.3 Downhill Grades

The downhill speed of a car carrying five passengers and a driver (4000 pounds gross weight) will increase by 10 miles per hour above the design speeds on grades steeper than the general maximum grade when travelling in top gear against engine compression over the lengths given in Table 8.3.

TABLE 8.3

Test Results for Cars on Steep Downhill Grades

Design Speed—mph	Grade per cent	Approximate Length of Grade to increase speed by 10 mph above design speed—feet
30	6½	2500
	7	1500
	8	1000
40	7	50 mph not attained
	7½	3000
	8	2000

8.6 SUMMARY OF MAXIMUM GRADE REQUIREMENTS

Table 8.4 shows absolute maximum grades and general maximum grades and has been prepared by combining the values of uphill and downhill grades found by the calculations and tests given earlier.

If it is not possible to build at a particular site a general maximum grade as low as the one listed in the Table owing to cost, the design speed values for alignment and sight distance should not be lowered.

Where a grade steeper than the general maximum is used, horizontal curves occurring on it, or at the foot, should be designed for a speed at least 10 miles per hour greater than the design speed used elsewhere on the road.

Lengths of grade longer than those shown in the last column of Table 8.4 may be used but the standards of alignment and sight distance in such circumstances should be those for the likely speed.

TABLE 8.4

Length and Value of Maximum Grades

Design Speed	General Maximum Grade	Absolute Maximum Grade	Allowable Length of Grade Steeper than General Maximum Grade	
			Grade	Length
mph	per cent	per cent	per cent	feet
20	10	—*	—	—
30	6	8	6½ 7 8	2500 1500 1000
40	6	8	7 7½ 8	4000 3000 2000
50	7	8†	8	3200
60	5	7	6 7	4500 2200
70	3	5	4 5	6200 3100
80	3	5	4 5	8100 4000

* No absolute maximum grade is included for design speed of 20 miles per hour as in some circumstance a length of very steep grade may be necessary for economic or other reasons. The length of grade steeper than 10 per cent should be kept as short as possible.
† As calculated this value would be 9 per cent. 8 per cent is the desirable upper limit for this design speed.

Switchbacks on approach to La Cumbre, Bolivia.

NATIONAL ASSOCIATION

OF

AUSTRALIAN STATE ROAD AUTHORITIES

POLICY

FOR

GEOMETRIC DESIGN

OF

RURAL ROADS - 1970

(METRIC ADDENDUM)

July 1972

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July 1972

METRIC ADDENDUM
TO
POLICY FOR GEOMETRIC DESIGN
OF RURAL ROADS — 1970
CONVERSION TO SI UNITS

Metric conversion in the road construction industry is planned to take place over the periods shown in Appendix A to this addendum. The publication *Policy for Geometric Design of Rural Roads* will then be amended in accordance with the following tables which should be read in conjunction with any explanatory or qualifying statements contained in the text of the present imperial publication (Fourth Edition — 1970).

General

Throughout the publication where the text refers to dimensions or speed in general terms (i.e. without specifying a value) make the following substitutions:

(a) For inches (in) *read* millimetres (mm).

(b) For feet (ft) *read* metres (m).

(c) For miles (M) *read* kilometres (km).

(d) For miles per hour (mph) *read* kilometres/hour (km/h).

Where dimensions or speeds appear in the text, but not in tabular form, then reference should be made to the approximate conversions in Tables B1 to B4 and the specific conversions adopted by NAASRA in Table B5 all of which appear in Appendix B to this addendum.

TABLE 8.1, page 57

Delete existing Table and *substitute:*

TABLE 8.1
General Maximum Grades

Design Speed km/h	Column (a) Maximum Upgrade[1] per cent	Column (b) Maximum Downgrade[2] per cent	Column (c) General Maximum grade[3]—per cent
40	—	—	10
50	9·5	4·5	6
60	9	5	6
70	8	6	6
80	7	7	7
90	6	7·5	6
100	5	8·5	5
110	3·5	9·5	3·5
120	—	—	3
130	—	—	3

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Clause 8.5.2, Uphill Grades—Excess Gain in Height, page 58

(1) *Delete* the formula in sub-paragraph (i) (page 59) and *substitute:*

$$mgh = \frac{1}{2} mv^2$$

where m = mass of vehicle (kg)
g = acceleration due to gravity
= $9{\cdot}81\ m/s^2$
h = additional height (m)
v = speed of vehicle (m/s^2)

(2) *Delete* the formula in sub-paragraph (ii) (page 60) and *substitute:*

$$mgh = 1{\cdot}05\ (\tfrac{1}{2}mv^2)$$

(3) *Delete* the formula in sub-paragraph (iii) (page 60) and *substitute:*

$$H = 0{\cdot}00413\ (V_1 - V_2)$$

where H = excess gain in height (m)
V_1 = vehicle speed at start of grade (km/h)
V_2 = vehicle speed at end of grade (km/h)

TABLE 8.2, page 60

Delete existing Table and *substitute:*

TABLE 8.2

Allowable Excess Gain in Height

Design Speed at Start of Grade	Speed at End of Grade Steeper than General Maximum	Excess Gain in Height
km/h	km/h	metres
80	64	10
90	72	12
100	80	15
110	88	18
120	96	21
130	104	25

Clause 8.5.3, Downhill Grades, page 61

In paragraph 1, line 2, *delete* "(4000 pounds gross weight)" and *substitute* "(total mass 1800 kg)".

TABLE 8.3, page 61

Delete existing Table and *substitute:*

TABLE 8.3

Test Results for Cars on Steep Downhill Grades

Design Speed km/h	Grade per cent	Approximate Length of Grade to increase speed by 20 km/h above design speed—metres
50	6·5	760
	7·0	460
	8·0	300
70	7·0	80 km/h not attained
	7·5	900
	8·0	600

TABLE 8.4, page 62

Delete existing table and substitute the table that appears on the next page.

TABLE 8.4
Length and Value of Maximum Grades

Design Speed	General Maximum Grade	Absolute Maximum Grade	Allowable Length of Grade Steeper than General Maximum Grade	
			Grade	Length
km/h	per cent	per cent	per cent	metres
40	10	—*	—	—
50	6	8	6·5 7 8	800 500 330
60	6	8	7 7·5 8	1100 900 600
70	6	8	7 7·5 8	1300 1000 700
80	7	8	8	1000
90	6	8	7 8	1200 600
100	5	7	6 7	1500 750
110	3·5	5·5	4·5 5·5	1800 900
120	3	5	4 5	2100 1000
130	3	5	4	2600 1300

APPENDIX B

CONVERSION TABLES

Introduction: The conversions given in Tables B1 to B4 are approximate only but are sufficiently accurate for road design purposes. To obtain more precise conversions reference should be made to the factors given in Table 4 of Australian Standard 1155, Metric Units in Construction, published by the Standards Association of Australia.

TABLE B1
Speed of Vehicles

Kilometres per hour to miles per hour		miles per hour to kilometres per hour	
km/h	mph	mph	km/h
10	6·2	10	16·1
20	12·4	20	32·2
30	18·6	30	48·3
40	24·9	40	64·4
50	31·1	50	80·5
60	37·3	60	96·6
70	43·5	70	112·7
80	49·7	80	128·8
90	55·9		
100	62·1		
110	68·4		
120	74·6		
130	80·8		

TABLE B2
METRES TO FEET

metres	0	1	2	3	4	5	6	7	8	9
	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet
0	—	3·28	6·56	9·84	13·12	16·40	19·68	22·97	26·25	29·53
10	32·81	36·09	39·37	42·65	45·93	49·21	52·49	55·77	59·06	62·34
20	65·62	68·90	72·18	75·46	78·74	82·02	85·30	88·58	91·86	95·14
30	98·43	101·71	104·99	108·27	111·55	114·83	118·11	121·39	124·67	127·95
40	131·23	134·51	137·80	141·08	144·36	147·64	150·92	154·20	157·48	160·76
50	164·04	167·32	170·60	173·88	177·16	180·45	183·73	187·01	190·29	193·57
60	196·85	200·13	203·41	206·69	209·97	213·26	216·54	219·82	223·10	226·38
70	229·66	232·94	236·22	239·50	242·78	246·06	249·34	252·62	255·91	259·19
80	262·47	265·75	269·03	272·31	275·59	278·87	282·15	285·43	288·71	292·00
90	295·28	298·56	301·84	305·12	308·40	311·68	314·96	318·24	321·52	324·80
100	328·08	331·36	334·65	337·93	341·21	344·49	347·77	351·05	354·33	357·61

TABLE B3
FEET TO METRES

feet	0	1	2	3	4	5	6	7	8	9
	metres	metres	metres	metres	metres	metres	metres	metres	metres	metres
0	—	0·305	0·610	0·914	1·219	1·524	1·829	2·134	2·438	2·743
10	3·048	3·353	3·658	3·962	4·267	4·572	4·877	5·182	5·486	5·791
20	6·096	6·401	6·706	7·010	7·315	7·620	7·925	8·230	8·534	8·839
30	9·144	9·449	9·754	10·058	10·363	10·668	10·973	11·278	11·582	11·887
40	12·192	12·497	12·802	13·106	13·411	13·716	14·021	14·326	14·630	14·935
50	15·240	15·545	15·850	16·154	16·459	16·764	17·069	17·374	17·678	17·983
60	18·288	18·593	18·898	19·202	19·507	19·812	20·117	20·422	20·726	21·031
70	21·336	21·641	21·946	22·250	22·555	22·860	23·165	23·470	23·774	24·079
80	24·384	24·689	24·994	25·298	25·603	25·908	26·213	26·518	26·822	27·127
90	27·432	27·737	28·042	28·346	28·651	28·956	29·261	29·566	29·870	30·175
100	30·480	30·785	31·090	31·394	31·699	32·004	32·309	32·614	32·918	33·223

TABLE B4

INCHES TO MILLIMETRES

in	mm	in	mm
1	25	7	178
2	51	8	203
3	76	9	229
4	102	10	254
5	127	11	279
6	152		

TABLE B5
Specific Conversions adopted by NAASRA not stated elsewhere in this Addendum

1. LANE WIDTHS

Present Imperial Dimensions	Exact Metric Equivalent	**Adopted Metric Value**	Approximate Imperial Equivalent
feet	metres	**metres**	feet
12 (single lane)	3·6576	**3·7***	12·14
18 (two lane)	5·4864	**5·6**	18·36
20 (two lane)	6·0960	**6·2**	20·33
22 (two lane)	6·7056	**6·8**	22·31
24 (two lane)	7·3152	**7·4**	24·28

* Dual carriageways shall be in multiples of 3·7m

2. SHOULDER WIDTHS

4	1·2192	**1·2**	3·94
6	1·8288	**1·8**	5·91
8	2·4384	**2·4**	7·87
10 and emergency lanes width	3·0480	**3·0**	9·84

3. FORMATION WIDTH

20	6·0960	**6·1**	20·03
26	7·9248	**8·0**	26·20
32	9·7536	**9·8**	32·14
34	10·3632	**10·4**	34·11
40	12·1920	**12·2**	40·03
44	13·4112	**13·4**	43·95

4. SLOW LANE

10	3·0480	**3·1**	10·17
11	3·3528	**3·4**	11·16
12	3·6576	**3·7**	12·14

5. MEDIANS

absolute min. 30 ft	9·1440	**10**	32·80
desirable min. 50 ft	15·2400	**15**	49·21

TABLE B5 (cont.)

6. WIDTH OF ROAD RESERVES

Description	Present Imperial Dimension	Adopted Metric Value
	feet	metres
(a) Two-lane rural roads	100	30
(b) Divided rural roads	200	60
(c) Divided rural roads where carriageways are located and graded separately	300	90
(d) Minimum distance from toe of fill or top of batter to reserve boundary	25	10

7. DRAWING SCALES

Typical Uses	Existing Scale	Adopted Scale
Longitudinal Scales:		
Plans and Longitudinal Profiles	200 ft to 1 in (1:2400)	1:2000*
	100 ft to 1 in (1:1200)	1:1000
	40 ft to 1 in (1:480)	1:500
Vertical Scales:		
Profiles and Cross Section	20 ft to 1 in (1:240)	1:200*
	10 ft to 1 in (1:120)	1:100
	4 ft to 1 in (1:48)	1:50

* The non-preferred scales of 1:2500 and 1:250 might be considered as possible substitutes for the present scales 1:2400 and 1:240 in specific cases where their advantages outweigh the disadvantages.

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8. SURVEYING

(a) Pegging Intervals on Straights and Curves

Straights		Curves	
Terrain	Pegging Interval—m	Radius metres	Pegging Interval—m
flat	100	less than 150	10
general	50	150-1000	20
mountainous	20	over 1000	50

(b) Accuracy of Levels and Horizontal Measurements

Levels		Horizontal Measurements	
Roads and Bridges	Accuracy mm	Roads and Bridges	Accuracy mm
Rural roads	10	Rural roads	10
Urban roads	5	Urban roads	10
Bench marks and bridges	1	Urban roads (where required) and bridges	5

LOW-VOLUME ROADS

Proceedings of a workshop held June 16-19, 1975, in Boise, Idaho, by the Transportation Research Board and cosponsored by the Agency for International Development, American Association of State Highway and Transportation Officials, Federal Highway Administration, Idaho Transportation Department, International Bank for Reconstruction and Development, International Road Federation, National Association of County Engineers, National Science Foundation, U.S. Army Engineer Waterways Experiment Station, U.S. Forest Service, and University of Idaho

SPECIAL REPORT 160
Transportation Research Board
National Research Council
National Academy of Sciences
Washington, D.C.
1975

CONTENTS

Planning, Economics, and Operations

THE FOREST SERVICE'S COMPUTER-AIDED ROAD DESIGN SYSTEM

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The Forest Service has an integrated system of computer programs for design of roads, including low-volume roads. This system is known as the Forest Service road design system. System calculations begin with initial traverse input and end with construction quantities. This paper briefly describes each program. Several programs that are felt to be unique to this system are discussed in more detail. The designer's aid program outputs information for the designer to establish a horizontal alignment, based on a trial grade and template information. The roadway template and slope selection programs allow the designer to define his template in terms of width, curve widening, superelevation criteria, and turnouts for shoulder-to-shoulder dimensions and variable slope criteria for cut and fill slopes, based on depth of cut and fill. The earthwork adjustment program provides a quantity defined as unit mass, the amount of mass change that would result from lowering or raising an existing vertical point of intersection elevation 1 ft (0.3 m). Unit mass is supplied to the designer for use in manual adjustments, or an option in the program will automatically use the unit mass and perform another earthwork run.

For many years, the Forest Service and other agencies that design and construct low-volume roads designed these facilities by hand. At best, this was a slow, laborious process that produced designs, but many times the design did not fit the intended use or the on-the-ground conditions. This was because manual design is time-consuming, many items were overlooked in the interest of time, and there was lack of manpower in the small agencies involved in low-volume roads.

In the late 1950s, the Forest Service started using electronic computers to design roads. The computers were leased IBM 650s and IBM 1401s, and the design programs were the programs written by the Bureau of Public Roads. These programs were rather cumbersome to use and did not fully meet the needs of the Forest Service. During the early 1960s, the Forest Service pruchased CDC 3100s and wrote a completely new group of interrelated programs for computer-aided design of roads.

This paper gives a brief overview of this system and discusses three specific programs that are unique to the system and that help in the design of low-volume roads. Other agencies involved with low-volume roads may find this system helpful to their program.

The Forest Service road design system (FSRDS) consists of a series of interrelated computer programs for processing road designs from the initial traverse to construction earthwork quantities. The FSRDS is the result of close cooperation among many Forest Service personnel, each contributing his experience, knowledge, and specialized skills to develop a user-oriented computer-aided design system that is useful to the on-the-ground designer.

Several times so far I have used the term computer-aided design system. In the Forest Service, we have stressed that the FSRDS does not produce optimal or automatic designs. Many times, when a designer is introduced to computer methods, he has the preconceived idea that computers will provide optimal design and, if not optimal, at least automatic. Usually, this type of a designer looks at the computer system as a replacement and not as a potentially powerful partner. In designing the FSRDS, we were not so much concerned with producing automatic designs, optimal designs, or even feasible designs as we were with providing the designer with a tool to increase his creativity and to enable him to produce more imaginative and better designs more efficiently. Most design problems associated with transportation, and this includes low-volume roads, are complex and involve many variables. To produce even a good design, a designer must investigate a number of design alternatives, something that could not easily be done manually. Only when a number of feasible alternative designs have been made can engineering decision making be meaningful and effective. The objective of using a computer and its related tools is to produce effective computer-aided design that will expand a designer's productivity and creativity.

The Forest Service road design system is an integrated system of 30 design programs that accept design terrain data and design criteria and provide data for the designer to make a decision on his next course of action. The objectives in designing and creating this system were

1. To provide computer-aided assistance in road design, such that the manual and computer processes are complimentary and integrated.
2. To enhance designing by performing analyses and evaluating alternatives, the time requirements for which are prohibitive in the manual process.
3. To require simple enough input to allow use by noncomputer-oriented staff.
4. To provide concise and comprehensive output to allow the designer and his supervisor to evaluate the project and determine the next course of action.
5. To provide output containing all possible data of value to the designer.

The FSRDS can be broken into five general phases of consideration (Figure 1): traverse, topography, horizontal alignment-offsets-template simulation, earthwork, and post earthwork.

278 The traverse phase consists of the following programs:

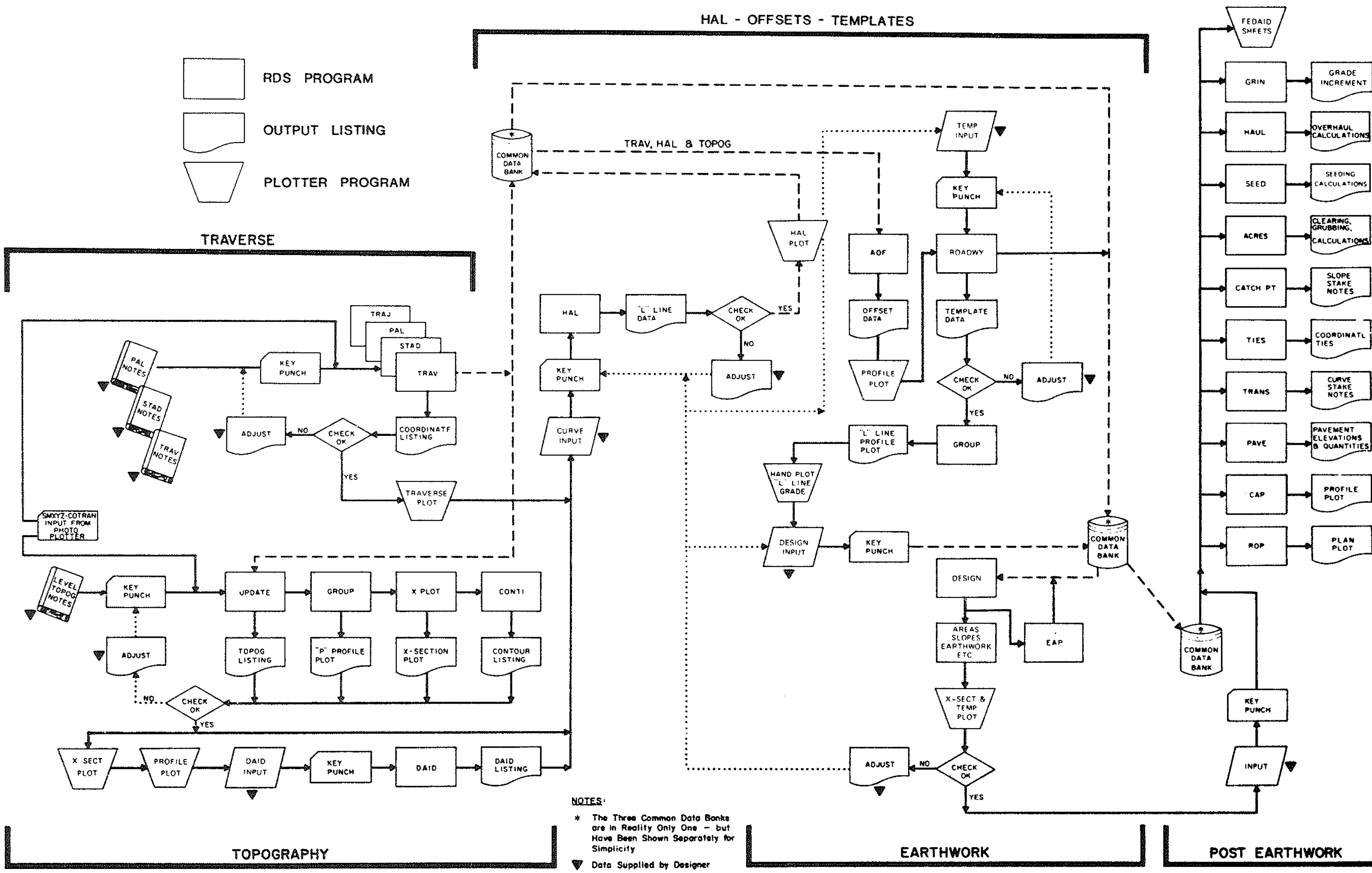
1. Traverse (TRAV) accepts varied input consisting of distance, horizontal or slope, bearings, azimuths, deflection angles, angles right, and coordinates; calculates coordinates, horizontal distance, bearings, deflection angles, and stationing.
2. Percent Abney level (PAL) accepts input consisting of slope distance, percentage of slope, and compass bearings; outputs stationing, horizontal distance, elevations, bearings, and coordinates.
3. Stadia (STAD) accepts input consisting of standard stadia survey data; outputs stationing, horizontal distance, elevations, bearings, and coordinates.
4. Traverse adjustment (TRAJ) accepts input from one of the three traverse programs (has option to adjust by either compass rule or transit rule); outputs adjusted bearings, distances, and coordinates.
5. Traverse plot (TRAP) is a digital line plot of the traverse data.

The topography phase consists of the following programs.

1. Update—This program reduces varied topography input to vertical rods and horizontal distances and prints out station, elevation, and all rod and distance readings. Varied input consists of vertical rods and horizontal distances and percentage of slope and slope distances or degree of slope and slope distances.
2. Ground profile plot (GROUP)—Input consists of elevations on file and produces either a printer plot of the profile or a line plot on a digital plotter.
3. Cross section plot (XPLOT)—This is a printer plot or line plot of cross section topography data on file.
4. Contour interpolation (CONTI)—CONTI produces a printout consisting of interpolated contours at a given interval from the topography data on file.
5. Designer's aid (DAID)—This program provides information that the designer can use as a guide for establishing horizontal alignment based on a trial grade and provides approximate earthwork and clearing quantities.

The horizontal alignment-offsets-template simulation phase consists of the following programs:

Figure 1. Forest Service road design system.



NOTES:

* The Three Common Data Banks are in Reality Only One — but Have Been Shown Separately for Simplicity

▼ Data Supplied by Designer

1. Horizontal alignment (HAL)—This program will accept field survey data for an existing horizontal alignment or coordinates and curve criteria for a design horizontal alignment. Output in both instances consists of stationing for the point of curvature, point of intersection, and point of tangency; coordinates for PC, PI, and PT, PI to PI distances, and pertinent curvature data.
2. Horizontal alignment plot (HALP)—This is a digital line plot of the horizontal alignment data.
3. Automatic offsets (AOF)—From the data on file, this program calculates the offset distance from a preliminary line to a design line, the skew angle at which the topography intersects the design line, the design line station and elevation, and X and Y coordinates.
4. Roadway template (ROADWY)—ROADWY computes turnouts, curve widening, and superelevation data and generates rod-distance templates. Curve widening, crown, and superelevation are combined with lane and turnout data to define the template.

The earthwork phase consists of two programs.

1. Design earthwork (DES)—Major input for this program is profile grade data, slope selection data, and compaction factors. From data on file, DES obtains topography and templates. The program computes profile grade for each station, end areas, volumes, unit mass for each vertical point of intersection (VPI), and features a printer plot of the mass diagram.
2. Earthwork adjustment program (EAP)—EAP is an optional program that requires no data input. The program automatically adjusts the profile grade in an attempt to optimize the relationship between haul and quantities.

The post earthwork phase consists of the following programs and usually requires no additional input, except for scales and other controlling criteria.

1. Catch point and profile grade plot (CAP)—This is a printer plot of the profile view, using four different characters, to represent the design grade, the design line ground profile, and the left and right top of cut and toe of fill points (catch points).
2. Road width and catch point plot (ROP)—This is a printer plot of the plan view, using four different characters, to represent the road centerline, the left and right edges of the road, and the left and right catch points.
3. Free haul and over haul quantities (HAUL)—This is a printer plot of the mass diagram giving quantities for each station, the mass of each station, the balance points, free haul points, and total cut, fill, free haul, and over haul between balance points.
4. Pavement elevations and quantities (PAVE)—This program calculates surface elevations at centerline, surface edge, and shoulder. It also calculates surfacing quantities in tons or cubic yards.
5. Design clearing and grubbing (ACRES)—This program calculates clearing and grubbing quantities and prints them out on a station-to-station basis and also keeps a running total.
6. Design seeding areas (SEED)—This program calculates seeding areas in square feet and acres and prints out on a station-to-station basis and also keeps a running total. An option in the program will calculate seed spread tables on an area increment specified by the designer.
7. Coordinate ties (TIES)—TIES provides field notes for establishing projected design line from a surveyed base line or photogrammetric control targets.
8. Transit notes (TRANS)—This program is intended to reduce field staking time by providing complete field notes in a form usable by field personnel to locate and stake a design line.
9. Slope stake notes (CATCHPT)—Slope stake notes for construction staking are computed and printed from the road design data on file. Notes are printed, two stations per page, and are designed for use as input documents for as-staked quantity calculations.

10. Grade increment (GRIN)—This program provides centerline grade elevations for evenly spaced stations.

11. Federal-aid plot (FEDAID)—This program provides machine-drawn federal-aid (plan and profile) sheets for the projects. Approximately 80 to 85 percent of all the data are machine drawn. Special notes and quantities must be hand drawn on the sheets.

Although the Forest Service road design system will handle the design of any undivided type of road, it was designed for low-volume roads, and there are several programs in the system that help design low-volume roads efficiently. These programs are designer's aid, roadway template and slope selection, and earthwork adjustment.

In essence, designer's aid allows the designer to make a quicky design on the base line information that has been collected. The minimum inputs are a trial profile grade, slope selection information, template information, and a compaction factor.

The computed output for each station is as follows:

1. Grade elevations;
2. Average side slope;
3. Topography limits, the distance left and right of base line centerline that the design template can be moved;
4. Cut and fill limits, how far the template can be shifted left and right of base line centerline, based on maximum cut and fill height;
5. Daylight offset, the distance left or right of base line centerline where the grade elevation intersects the topography;
6. Self-balanced offset, the distance the template is shifted to obtain a condition where cut quantities equal fill quantities; and
7. Approximate station by station earthwork quantities.

Several of these outputs can be valuable in establishing a horizontal alignment, especially the self-balanced offset distance. The designer can plot the various offset distances on the traverse plot and then fit a horizontal alignment to these points to meet the design criteria.

As was stated, these calculations are based on a trial grade; therefore, all output information must be reviewed in light of the grade that was established. If the original results are not as expected, new input can be submitted and the program rerun.

This program works best on 30 to 60 percent side slopes. Output data for steep or relatively flat terrain may seem questionable or contradictory. However, the data are usually correct in respect to the restraints introduced through input data.

The Forest Service road design system uses two separate functions to describe the total road template from catch point to catch point. The roadway template program is used to describe the roadway template from shoulder to shoulder. The template from shoulder to catch point is described by the slope selection program.

Input to the roadway template program is handled in three separate areas:

1. Superelevation and curve widening criteria—These criteria consist of curve widening factors, curve widening runoff distance, design speed, side friction factor, maximum and minimum superelevation, superelevation runoff criteria, and pivot information. The designer enters design criteria only where the criteria change from those currently used. If one set of superelevation and widening criteria will meet the designer's needs, that is all that needs to be entered. If he needs 10 sets of criteria, he enters 10 sets.
2. Template data—The designer enters lane width, crown, and shoulder widths. Again, an entry is only needed when and where the criteria change. It is not necessary to enter road width at each station.
3. Turnout criteria—The designer enters the taper distance in and out, the turnout length, the turnout width, which side of the road it is located on, and the slope of the turnout.

The program then takes all of the criteria and calculates a template for every station.

The program compares each template with the last calculated template. If they are the same, it discards the new template. In this manner, it is necessary to print out and store templates only where the template changes.

The printout consists of the following rod and distance shots, which define the template:

1. Centerline,
2. Left and right lane width plus curve widening and shoulder,
3. Left and right lane width plus curve widening, shoulder, and turnout, and
4. Pivot point if other than a previously printed shot.

The slope selection capabilities of RDS provide for automatic assignment of cut and fill slopes and ditch criteria. As in other design criteria input, the designer only needs to input slope criteria where there are changes. Data need not be entered at each station, unless the designers want to.

Input to this program consists of the following:

1. Slope specification control—This input controls the side on which the criteria are applied.
2. Typical section number—This allows the designer to use the same slope criteria later in the design by simply entering the typical section number. It is not necessary to write out the complete criteria.
3. Cut and fill slopes—One to three cut slopes or fill slopes or both can be assigned. They are assigned by the designer and used by the program on a depth criterion, measured from grade to catch point.
4. Ditch criteria—Side slope, depth, and width of the ditch are input.
5. Daylight—A daylight distance from shoulder can be entered. If the section can be daylighted within the limits specified, the program will automatically daylight the section.

Output from this program shows up on the design earthwork printout as the slope that was chosen for that station, along with any ditch criteria for that station.

Part of the output from the earthwork adjustment program is a quantity known as the unit mass. The unit mass is the amount of mass change that would result from lowering or raising an existing VPI elevation 1 ft (0.3 m). This quantity is supplied to assist the designer in making manual VPI changes.

The earthwork adjustment program (EAP) enables the computer to automatically do what the designer can do manually with unit mass, accurately compute VPI elevation changes to optimize earthwork. This new grade is automatically used to provide another earthwork run. Only the total design earthwork quantities are printed for the run with the VPIs submitted by the designer. The EAP run is listed in its entirety. EAP permits the designer one control; VPIs not to be changed can be specified. EAP automatically gives the first and last VPIs this status.

EAP was designed to simplify the process of obtaining balanced earthwork quantities. It accomplishes this by adjusting the elevations of VPIs automatically. The amount of adjustment for a VPI is determined by analyzing the mass diagram in the area to be changed. EAP functions by analyzing two items:

1. It analyzes the mass diagram to determine the amount of mass adjustment to be accounted for by each VPI, and
2. It determines the effect of each VPI on the mass corresponding to the grades adjacent to the VPI.

The desired elevation change for that VPI is then determined by relating these two analyses.

The theory of straight line fitting, employing the least squares method, is used to analyze the mass diagram. This method very nearly yields the least area between the mass diagram curve and the best fitting straight line.

Two consecutive grade tangents are analyzed as a single section of the mass diagram. When the analysis of these two tangents is complete, the program moves to the next tangent. The tangent of the first section and the adjacent new tangent form another section, which overlaps the previous section. The overlapping best fitting lines often intersect, which provides continuity that enables every VPI to be held accountable for a portion of the mass change required. It is this overlapping of the best fitting lines that enables the program to handle every possible type of mass drift.

The mass change required at a VPI is the difference between the intersection ordinates on each side of that VPI.

Next, EAP considers the effect of a VPI on the mass within its influence. For this, EAP assumes that the middle VPI of a two-tangent section is raised 1 ft (0.3 m). Each station is then considered in order to find its corresponding raise.

Cut and fill widths are then considered for each station. Cut width is adjusted by the compaction factor and added to the fill width to give an adjusted total width. Area changes for each station are then determined. From these come volume changes between stations. The sum of the volume changes over the two-tangent sections is the unit mass. The ratio of the accountable mass to the unit mass is the required elevation change.

A few weaknesses in EAP are apparent.

1. There is no grade limitation feature.
2. Accuracy drops off for larger elevation changes.
3. Prolonged restricted adjustment areas (holding the elevation on a series of VPIs) prevent the following areas from being balanced with respect to the original zero mass line.

None of these poses a serious problem. A rerun of EAP resolves problem 2 very easily. The designer can use the unit mass output of EAP to resolve the other two problems.

The Forest Service feels that the road design system is a very efficient and effective tool for helping the road designer and the on-the-ground land manager make effective decisions concerning road design and construction. They can fit the road to the ground with minimum impact and still take into consideration the safety of the road users.

This paper has presented a very brief look at the Forest Service road design system. It is capable of doing much more than was discussed. It is capable of designing roads other than divided highway, and it was specifically designed for low-volume roads. The system is continually being updated to meet new technology and new construction methods. It has been used for more than 10 years.

With the advent of high-powered programmable desk-top calculators (computers), the Forest Service started using them for low-volume road design. The main applications, at present, are for low precision surveys and designs, which do not have relatively high continuous service. We are using this type of equipment in remote areas that do not have access to large-scale computing systems.

The Forest Service is using this equipment in many places. At this time, they are accomplishing a small portion of the design, but we expect the volume of work on this equipment to increase. They are here to stay, and we are making plans to write efficient programs for these machines.

Western Hills Road, Nepal.

Bibliography

The following bibliography contains two sets of references. The first set consists of a reference for each selected text that appeared in the preceding part of this compendium. The second set consists of references to additional publications that either were cited in the selected texts or are closely associated with material that was presented in the Overview and Selected Texts. Each reference has five parts that are explained and illustrated below.

(a) Reference number: This number gives the position of the reference within this particular bibliography. It is used in the compendium index but should ***not*** be used when ordering publications.

(b) Title: This is either the title of the complete publication or the title of an article or section within a journal, report, or book.

(c) Bibliographic data: This paragraph gives names of personal or organizational authors (if any), the publisher's name and location, the date of publication, and the number of pages represented by the title as given above. In some references, the paragraph ends

Bibliografía

La siguiente bibliografía contiene dos series de referencias. La primera serie consiste en una referencia para cada texto seleccionado que apareció en la parte anterior de este compendio. La segunda serie consiste de referencias a publicaciones adicionales que fueron mencionadas en los textos seleccionados o que se asocian intimamente con el material que se presentó en la Vista General y los Textos Seleccionados. Cada referencia tiene cinco partes que se explican e ilustran abajo

(a) Número de referencia: Este número dá la posición de la referencia dentro de este bibliografía en particular. Se utiliza en el índice del compendio pero *no* deberá utilizarse al pedir publicaciones.

(b) Título: Este es el título de la publicación completa o el título de un artículo o sección dentro de una revista, informe, o libro.

(c) Datos bibliográficos: Este parágrafo dá los nombres de autores personales o organizacionales (si hay alguno), el nombre del editor y su direccion, la fecha de publicacion, y el numero de paginas representadas por el título en la parte (b). En algunas referencias el parágrafo termina con

Bibliographie

La bibliographie qui suit contient deux catégories de références. La première catégorie consiste en une référence pour chaque texte choisi qui est inclus dans la partie précédente de ce recueil. La deuxième catégorie contient des références pour des documents qui ont soit été cités dans les textes choisis, ou soit sont étroitement associés avec des écrits qui sont présentés dans l'Exposé ou les Textes Choisis. Chaque référence est composée de cinq parties qui sont expliquées et illustrées ci-dessous

(a) Numéro de la référence: Ce numéro indique la position de cette référence dans cette bibliographie. Ce numéro est indiqué dans l'index du recueil mais *ne doit pas* être utilisé pour les commandes de publications.

(b) Titre: Cela indique ou le titre du livre entier, ou le titre d'un article ou d'une section d'une revue, un rapport, ou un livre.

(c) Données bibliographiques: Ce paragraphe indique les noms des auteurs personnels (quand il y en a) ou des auteurs collectifs (organisation), le nom de l'editeur et son adresse, la date de l'edition, et le nombre de pages qui sont incluses sous le titre dans (b). Certaines with an order number for the publication in parentheses.

(d) Availability information: This paragraph tells how the referenced publication is available to the reader. If the publication is out-of-print but may be consulted at a particular library, the name of the library is given. If the publication can be ordered, name and address of the organization from which it is available are given. ***The order should include all information given in parts (b) and (c) above.***

(e) Abstract: This paragraph contains an abstract of the publication whose title was given in part (b).

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un número de pedido para la publicación en paréntesis.

(d) Disponibilidad de la informàción: Este parágrafo explica que la publicación referenciada está disponible al lector en una de dos formas como sigue. (1) La publicación esta agotada pero puede ser consultada en la biblioteca indicada donde se sabe que se posee una copia. (2) La publicación puede ser pedida de la organización cuyo nombre y dirección están indicados. ***El pedido deberá incluir toda la información dada en las partes (b) y (c).***.

(e) Resumen. Este parágrafo es un resúmen de la publicación cuyo título se dió en la parte (b).

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références se terminent par un numéro entre parenthèses qui indique le numéro de commande.

(d) Disponibilité des Documents: Ce paragraphe indique les deux façons dont le lecteur peut acquérir les documents: (1) L'édition est épuisée, mais une certaine bibliothèque détient ce document et il peut être consulté. (2) Le document peut être commandé à l'organisation dont le nom et l'adresse sont indiqués ici. ***L'ordre de commande doit inclure toutes les informations données dans les parties (b) et (c).***

(e) Analyse: Ce paragraphe est une analyse du texte dont le titre est cité dans la partie (b).

Illustration Ilustración Illustration

(a) Reference number
(a) Número de referencia →
(a) Numéro de la référence

Reference 5

(b) Title
(b) Título →
(b) Titre

A REVIEW OF HIGHWAY DESIGN PRACTICES IN DEVELOPING COUNTRIES

(c) Bibliographic data
(c) Datos bibliográficos →
(c) Donnees bibliographiques

Cron, Frederick W. Washington, DC: International Bank for Reconstruction and Development; 1975 May. 57 p.

(d) Availability information
(d) Disponibilidad de la información →
(d) Disponibilité des documents

Order from: International Bank for Reconstruction and Development, 1818 H Street, N.W., Washington, DC 20433.

(e) Abstract
(e) Resumen →
(e) Analyse

The design standards of some 150 highway projects financed by the International Bank for Reconstruction and Development between 1960 and 1970 are reviewed, and areas of agreement between the standards of the 63 countries studied are identified; practical highway standards based on these areas of agreement are sketched for the guidance of planners in developing countries. The roads discussed here, fall into three functional categories: a small group of expressways, freeways and toll roads carrying large volumes of traffic; a very large group of 2-lane highways carrying a wide range of traffic volumes serving both local and long distance traffic; and a smaller group of low-traffic tertiary or special purpose roads existing primarily for land service. Comments are made on the problem of classifying highway standards, and on the comparison of standards. Conclusions regarding standards for the capacity- related elements of design and standards for the velocity-related elements of design (radius of curvature, stopping sight distance, passing sight distance) are discussed, as well as the horizontal and vertical clearances for bridges. The standard live loadings for bridges, the structural capacity of pavements and legal load limits are covered, and conclusions relating to pavement design, design standards for 2-lane highways, incremental development of highways, and levels of service are presented.

The order should include all information given in parts (b) and (c) above.

El pedido deberá incluir toda la información dada en las partes (b) y (c).

L'ordre de commande doit inclure toutes les informations données dans les parties (b) et (c).

SELECTED TEXT REFERENCES

Reference 1
A GUIDE TO HIGHWAY DESIGN STANDARDS

International Bank for Reconstruction and Development, Department of Technical Operations:Washington, DC; 1957 June. 76 p.

Out-of-print; may be consulted at U.S. Department of Transportation, Library Services Division, Room 2200, 400 Seventh Street, S.W., Washington, DC 20590.

Specific guidelines are offered for the selection and use of proper design standards in the preparation of plans and specifications for highway construction projects. Standards for geometric and structural design of highways are presented in which detailed dimensions and specifications of the various elements are indicated. Design speed, horizontal alignment, vertical alignment, cross-section, pavement design, embankments, subgrade, subbase, and base course are covered, as well as the selection of pavement type, the design of pavement thickness, and design loading for structures. Surface and subsurface drainage and vehicle design and traffic operating characteristics are also considered. The standards are listed for both primary and secondary routes and for different topographic conditions. The design elements are specified within relatively broad limits to permit the design engineer to select appropriate standards for any given situation. The various considerations which were recognized in making the selections are also discussed. Design data in the form of graphs and charts are included.

Reference 2
A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS - 1965

American Association of State Highway Officials. Washington, DC; 1966. 650 p.

Order from: American Association of State Highway and Transportation Officials, 444 North Capitol Street, N.W., Suite 225, Washington, DC 20001.

This book is an update of the 1954 publication commonly known as the "Blue Book " (see Reference 12). While all parts of the book were subject to updating it is well to state some of the principal items which were revised. The design vehicles were changed in dimension to conform to motor vehicle trends particularly semitrailers which were designed in accordance with wheelbase rather than length. The data curves of vehicle were based on vehicle miles of travel rather than on manufacturers' lists as in the 1954 publication. Data for design speeds of 75 and 80 miles per hour were added and sight distance design criteria were increase in lengths of vertical curves. Capacity data were revised to conform to the new capacity manual and the sections on climbing lanes revised accordingly. Design values for speedchange lanes were increased to reflect current operating experience. Examples of design were changed to reflect a decade of new design and construction.

Reference 3
LOW COST ROADS; DESIGN, CONSTRUCTION AND MAINTENANCE

Odier, L.; Millard, R.S.;dos Santos, Pimentel; Mehra, S. R. London: Butterworths; 1971. 158 p. (Sponsored by UNESCO).

Out-of-print; may be consulted at U.S. Department of Transportation, Library Services Division, Room 2200, 400 Seventh Street, S.W., Washington, DC 20590.

Engineering codes relating to road planning, design, construction, materials, and maintenance for use in the developing countries are presented here. Basic principles of road construction and maintenance policy including social and economic aspects, master plan and feasibility studies, as well as stage construction are covered. Traffic and design speeds, design related to vertical alignment and horizontal alignment, and cross section elements are discussed; design principles for unimproved roads, improved roads, roads with permanent surfaces, and flexible pavements are set forth. The drainage of the road is considered including the control of erosion, and the stability of embankments and cuttings. Defensive measures during wet weather construction are noted. The location and waterway requirements for bridges and culverts are discussed, and the principal factors to be considered in the design of bridge foundations and structures are indicated. Notes are provided on construction operations and plant which include preliminary and detailed surveys, setting out, earthworks, compaction, quarrying, soil stabilization, bituminous surfacing and concreting. The discussion of road maintenance distinguishes between short-term, largely manual maintenance and long-term maintenance usually involving the use of mechanical equipment. Methods of estimating costs are outlined and special consideration is given to the choice between manual and mechanised methods or combinations of the two.

Reference 4
GEOMETRIC DESIGN GUIDE FOR LOCAL ROADS AND STREETS

American Association of State Highway Officials, Committee on Planning and Design Policies. Washington, DC: American Association of State Highway Officials; 1971. 24 p.

Order from: American Association of State Highway and Transportation Officials, 444 North Capitol Street, N.W., Suite 225, Washington, DC 20001.

Standards for geometric design of urban and rural local roads and streets are set forth. In 1961 AASHO published a single set of values for both primary and secondary highways, but over the intervening decade it seemed appropriate for design and administrative reasons to issue separate standards for local roads and streets.

Reference 5
A REVIEW OF HIGHWAY DESIGN PRACTICES IN DEVELOPING COUNTRIES

Cron, Frederick W. Washington, DC: International Bank for Reconstruction and Development; 1975 May. 57 p.

Order from: International Bank for Reconstruction and

Development, 1818 H Street, N.W., Washington, DC 20433.

The design standards of some 150 highway projects financed by the International Bank for Reconstruction and Development between 1960 and 1970 are reviewed, and areas of agreement between the standards of the 63 countries studied are identified; practical highway standards based on these areas of agreement are sketched for the guidance of planners in developing countries. The roads discussed here, fall into three functional categories: a small group of expressways, freeways and toll roads carrying large volumes of traffic; a very large group of 2-lane highways carrying a wide range of traffic volumes serving both local and long distance traffic; and a smaller group of low-traffic tertiary or special purpose roads existing primarily for land service. Comments are made on the problem of classifying highway standards, and on the comparison of standards. Conclusions regarding standards for the capacity- related elements of design and standards for the velocity-related elements of design (radius of curvature, stopping sight distance, passing sight distance) are discussed, as well as the horizontal and vertical clearances for bridges. The standard live loadings for bridges, the structural capacity of pavements and legal load limits are covered, and conclusions relating to pavement design, design standards for 2-lane highways, incremental development of highways, and levels of service are presented.

Reference 6
OPPORTUNITIES FOR COST REDUCTION IN THE DESIGN OF TRANSPORT FACILITIES FOR DEVELOPING REGIONS

University of California, Institute of Transportation and Traffic Engineering. Berkeley, California; 1970. 406 p. (Sponsored by the U.S. Department of Transportation, Office of the Assistant Secretary for Policy and International Affairs and the U.S. Agency for International Development; report #PB-207520).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

The document provides the results of an examination of traditional designs of transport facilities in the developing countries with a view to reducing total initial and/or operating costs, or to reducing costs devoted to the imported elements. The topics include: Road cost analysis and design standards; road construction cost model; a road maintenance cost model; opportunities for cost savings in highway engineering design; economics of one-way bridging; potential cost savings in the design of water crossings; potential cost savings in the design and use of ground vehicles; opportunities for cost reductions in aircraft, airways, and airports; potential cost savings in the selection of waterway and harbor techniques; harbors and associated facilities; economic models for choice of transport techniques in developing countries; tradeoffs between construction costs and maintenance costs.

Reference 7
A SAFE SIGHT DISTANCE REQUIREMENT FOR UN-LANED RURAL ROADS

Valentine, W.H. Rural and Urban Roads, Vol. 6, No. 2, 1968 February; pp. 34-35, 52.

Order from: Scranton Publishing Company, 434 South Wabash Avenue, Chicago, Illinois 60605.

In arriving at a suitable sight distance, it is necessary to use the sight path for the passing sight distance requirement and the stopping distance developed for the particular speed. In this article, the total single lane sight distance requirements for various speeds are tabulated, and a rapid means for selecting the required minimum length vertical curve for "head on" stopping sight distance requirements is illustrated in a figure. These curves can be used to determine design vertical curves for desired speeds and grades, as well as to determine what speeds are safe for existing roadways where no structural changes are to be made.

Reference 8
POLICY FOR GEOMETRIC DESIGN OF RURAL ROADS.

4th ed. National Association of Australian State Road Authorities. Sydney; 1970. 88 p.

Out-of-print; see Reference 17.

The principles of design stated in the policy are based on mathematical calculations, experimental results, and engineering experience. They have been gathered to promote uniformity of design practice. Separate sections cover: factors influencing geometric design, the influence of speed on road design, width of pavement and formation, cross section, sight distance, horizontal alignment, vertical curves, grades, design form, and divided rural roads. A bibliography and an appendix on curb types and uses are included.

Reference 9
POLICY FOR GEOMETRIC DESIGN OF RURAL ROADS-1970 (METRIC ADDENDUM)

National Association of Australian State Road Authorities. Sydney; 1972 July. 34 p.

Out-of-print; see Reference 17.

The document provides metric conversion tables for design speed, grades, radius of horizontal curves, etc. shown in the imperial system in Reference 8 above.

Reference 10
THE FOREST SERVICE'S COMPUTER-AIDED ROAD DESIGN SYSTEM

George, Thomas A. Low-Volume Roads; Proceedings of a Workshop Held June 16-19, 1975 in Boise, Idaho. Washington, DC: Transportation Research Board; 1975; pp. 75-81. (Special Report 160).

Order from: Transportation Research Board, 2101 Constitution Avenue, N.W., Washington, DC 20418.

The Forest Service has an integrated system of computer programs for design of roads, including low-volume roads. This system is known as the Forest Service road design system. System calculations begin with initial traverse input and end with construction quantities. This paper briefly describes each program. Several programs that are felt to be unique to this system are discussed in more detail. The designer's aid program outputs information for the designer to establish a horizontal alignment, based on a trial grade and template information. The roadway template and slope selection programs allow the designer to define his template in terms of width, curve widening, superelevation criteria, and turnouts for shoulder-to-shoulder dimensions and variable slope criteria for cut and fill slopes, based on depth of cut and fill. The earthwork adjustment program provides a quantity defined as unit mass, the amount of mass change that would result from lowering or raising an existing vertical point of intersection evaluation 1 ft (0.3m). Unit mass is supplied to the designer for use in manual adjustments, or an option in the program will automatically use the unit mass and perform another earthwork run.

ADDITIONAL REFERENCES

Reference 11
a. ROAD DESIGN SYSTEM

U.S. Forest Service, Engineering Staff. Fort Collins, Colorado; 1978. Magnetic tape. (Item #PB-279500).

b. ROAD DESIGN SYSTEM - ENGINEERING COMPUTER APPLICATION HANDBOOK

U.S. Forest Service, Engineering Staff. Fort Collins, Colorado; 1978 April. Various paging. (Report #PB-279501)

Order either from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

The primary purpose of the handbook is to instruct the user in submitting road design data to the computer center and in interpreting computer print-outs. The road design system (RDS), when used properly as a tool by the designer, will enable him to study more alternate designs and thus improve his end product —a well-designed road. The RDS as explained in this Handbook consists of a series of interrelated computer programs with the purpose of performing the many tedious and repetitious mathematical calculations required in road design. It does not design roads, but performs mathematical calculations based on the data input.

Reference 12
A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS

American Association of State Highway Officials. Washington, DC; 1954. 655 p.

Out-of-print; may be consulted at U.S. Department of Transportation, Library Services Division, Room 2200, 400 Seventh Street, S.W., Washington, DC 20590.

In the period 1938 to 1944 the Committee on Planning and Design Policies developed seven policies which were adopted by the Association and printed as separate brochures. In 1950 the group was reprinted without change and bound as a single volume entitled Policies on Geometric Highway Design. A Policy on Geometric Design of Rural Highways adopted May 3, 1954, was a complete reworking of the separate policies into a cohesive single volume which completely superseded the group of seven former policies. This volume which came to be commonly known as the "Blue Book" received wide acceptance.

Reference 13
GEOMETRIC DESIGN STANDARDS FOR HIGHWAYS OTHER THAN FREEWAYS - 1969

American Association of State Highway Officials, Committee on Planning and Design Policies. Washington, DC: American Association of State Highway Officials; 1969. 15 p.

Order from: American Association of State Highway and Transportation Officials, 444 North Capitol Street, N.W., Suite 225, Washington, DC 20001.

Geometric highway design deals with the dimensions of those highway features such as alinement, grades, widths, sight distance, clearances and slopes as distinguished from structural design which deals with such features as thickness, composition of materials and load carrying capacity. Frequently, available finances do not permit the construction of the ideal in highway design. Where economy is necessary it is suggested that it be practiced on some feature other than the principal geometric features. The roadway section can be improved and widened at reasonable cost. The surfacing can be widened and strengthened at any future date that finances will permit. But the geometric features of alinement, grade, and sight distance, when once molded into the landscape and tied down by right-of-way and surfacing, are most difficult and expensive to correct. It appears, therefore, that in discussion and decision on these principal geometric features a generous factor of safety should be added, and unquestioned adequacy rather than strict economy should be the criterion. These standards are intended to be applied as general design controls regardless of the system of which the highway is a part.

Reference 14
ECONOMICS OF DESIGN STANDARDS FOR LOW-VOLUME RURAL ROADS

Oglesby, C.H.; Altenhofen, M.J. Washington, DC: Highway Research Board; 1969. 93 p. (NCHRP Report 63).

Order from: University Microfilms International, 300 North Zeeb Road, Ann Arbor, Michigan 48106.

The report examines current standards for roadbed widths (shoulder break to shoulder break) and surface types. Both standards for highway needs studies and new road designs were presented. The rationale for setting the current standards, which combines scientific principles, engineering judgment, and financial and political compromises was discussed by the researchers. Construction, maintenance, vehicle operating, and accident costs were derived for various typical low-volume roadway designs. In addition, the economic effects of standards for vertical and horizontal alinements also were examined.

Reference 15
A POLICY ON DESIGN STANDARDS FOR STOPPING SIGHT DISTANCE

American Association of State Highway Officials, Standing Committee on Engineering Policies. Washington, DC: American Association of State Highway Officials; 1971. 15 p.

Order from: American Association of State Highway and Transportation Officials, 444 North Capitol Street, N.W., Suite 225, Washington, DC 20001.

The stopping sight distance values adopted in design, policies and standards have been reexamined and retained as minimum design values, and a second set of higher values has been added to be used as a basis for design wherever it is practicable so to do. Observations have shown that a higher value for initial speed and lower coefficients of friction should be used, and that drivers need to see somewhat further ahead than present minimum values in order to have more maneuvering time at decision points. The modified design values are based on the decision to: retain the 2.5 seconds for perception and reaction time for all speeds; retain the same relationship between the actual speed of the vehicle and the coefficients of friction on wet pavements; change to the use of full design speed as the initial speed for braking distance calculation; retain the 3.75 foot height of driver's eye criteria; and retain the 0.5-foot height of object criteria. The increase in stopping distances for the various design speeds are significantly greater for the following reasons: braking distance increases as the square of the speed; distance traveled during perception-reacting increases directly as the speed of the vehicle; and the friction factor diminishes with an increase in speed.

Reference 16
HIGHWAY DESIGN MANUAL

Roy Jorgensen Associates, Inc. Washington, DC: U.S. Agency for International Development; 1975. 228 p. (Highway Manual AID/OTrC-1420).

Order from: Office of Engineering, U.S. Agency for International Development, 320 21st Street, N.W., Washington, DC 20523.

This manual has been developed to provide guidance and assistance to highway design personnel in the practices and procedures for the detailed design of highways and the preparation of contract plans. The principal objectives of the manual are: to document highway administration policies with regard to standards of design and procedures for development of contract plans; to define criteria to guide judgments and decisions made by highway design personnel; to describe the most effective design techniques and procedures and to present charts, tables and other information found to be useful by designers. The material in the manual has been reviewed and edited by a committee of highway administration personnel who are knowledgeable of current design requirements and procedures. In this first edition, emphasis has been placed principally on compiling and documenting policies and procedures currently being followed as a result of various memoranda and verbal instructions. New and improved procedures have been incorporated in the manual when it was evident that they would be more effective. This initial publication should not be considered a completed comprehensive design manual - rather it is a first step toward orderly documentation of policies, procedures, instructions and guides. Those persons who use the manual can contribute to its continuing improvement by submitting suggestions to the Director of Engineering, for ways in which the manual can be made more useful and practical.

Reference 17
POLICY FOR GEOMETRIC DESIGN OF RURAL ROADS

5th ed. National Association of Australian State Road Authorities. Sydney; 1976. 92 p.

Order from: National Association of Australian State Road Authorities, P.O. Box J141, Brickfield Hill, N.S.W. 2000, Australia.

This edition does not differ in any of the technical requirements from the previous edition (see Reference 8) however the opportunity has been taken to publish a metric edition of the policy and to include an index.

Index

The following index is an alphabetical list of subject terms, names of people, and names of organizations that appear in one or another of the previous parts of this compendium, i.e., in the Overview, Selected Texts, or Bibliography. The subject terms listed are those that are most basic to the understanding of the topic of the compendium.

Subject terms that are not proper nouns are shown in lower case. Personal names that are listed generally represent the authors of selected texts and other references given in the bibliography, but they may also represent people who are otherwise identified with the compendium subjects. Personal names are listed as surname followed by initials. Organizations listed are those that have produced information on the topic of the compendium and that continue to be a source of information on the topic. For this reason, postal addresses are given for each organization listed.

Numbers that follow a subject term, personal name, or organization name are the page numbers of this compendium on which the term or name ap-

Indice

El siguiente índice es una lista alfabética del vocablo del tema, nombres de personas, y nombres de organizaciones que aparecen en una u otra de las partes previas de este compendio, es decir, en el Vista General, Textos Seleccionados, o Bibliografía. Los vocablos del tema que se listean son aquellos básicos necesarios para el entendimiento de la materia del compendio.

Los vocablos del tema que no son nombres propios aparecen en letras minúsculas. Los nombres personales que aparecen representan los autores de los textos seleccionados y otras referencias dadas en la bibliografía, pero también pueden representar a personas que de otra manera están conectadas a los temas del compendio. Los nombres personales están listeados como apellido seguido por las iniciales. Las organizaciones nombradas son las que han producido información sobre la materia del compendio y que siguen siendo una fuente de información sobre alguna parte o el alcance total del compendio. Por esta razón se dan las direcciones postales para cada organización listeada.

Los números que siguen a un vocablo del tema, nombre personal, o nombre de organización son los números de página del com-

Index

Cet index se compose d'une liste alphabétique de mots-clés, noms d'auteurs, et noms d'organisations qui paraissent dans une section ou une autre de ce recueil, c'est à dire dans l'Exposé, les Textes Choisis, ou la Bibliographie. Les mots-clés cités sont ceux qui sont le plus élémentaires à la compréhension de ce recueil.

Les mots-clés qui ne sont pas des noms propres sont imprimés en minuscules. Les noms propres cités sont les noms des auteurs des textes choisis ou de textes de référence cités dans la bibliographie, ou alors les noms de personnes identifiées avec les sujets de ce recueil. Le nom de famille est suivi des initiales des prénoms. Les organisations citées sont celles qui ont écrit sur le sujet de ce recueil et qui continueront d'être une source de documentation. Les adresses de toutes ces organisations sont incluses.

Le numéro qui suit chaque mot-clé, nom d'auteur, ou nom d'organisation est le numéro de la page où ce nom ou mot-clé parait. Les numéros écrits en chiffres romains se rapporpears. Roman numerals refer to pages in the Overview, Arabic numerals refer to pages in the Selected Texts, and reference numbers (e.g., Ref.12) refer to references in the Bibliography.

Some subject terms and organization names are followed by the word ***see.*** In such cases, the compendium page numbers should be sought under the alternative term or name that follows the word ***see.*** Some subject terms and organization names are followed by the words ***see also.*** In such cases, relevant references should be sought among the page numbers listed under the terms that follow the words ***see also.***

The foregoing explanation is illustrated below.

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pendio donde el vocablo o nombre aparecen. Los números romanos se refieren a las páginas en la Vista General, los números arábigos se refieren a páginas en los Textos Seleccionados, y los números de referencia (por ejemplo, Ref. 12) indican referencias en la Bibliografía.

Algunos vocablos del tema y nombres de organizaciones están seguidos por la palabra *see.* En tales casos los números de página del compendio se encontrarán bajo el término o nombre alternativo que sigue a la palabra *see.* Algunos vocablos del tema y nombres de organizaciones están seguidos por las palabras *see also.* En tales casos las referencias pertinentes se encontrarán entre los números de página indicadas bajo los términos que siguen a las palabras *see also.*

La explicación anterior esta subsiguientemente ilustrada.

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tent aux pages de l'Exposé et les numéros écrits en chiffres arabes se rapportent aux pages des Textes Choisis. Les numéros de référence (par exemple Ref. 12) indiquent les numéros des références de la Bibliographie.

Certains mots-clés et noms d'organisations sont suivis du terme *see.* Dans ces cas, le numéro des pages du recueil se trouvera après le mot-clé ou le nom d organisation qui suit le terme *see.* D'autres mots-clés ou noms d'organisations sont suivis des mots *see also.* Dans ce cas, les références qui les touchent se trouveront citées après les mots-clés qui suivent la notation *see also.*

Ces explications sont illustrées ci-dessous.

Illustration **Ilustración** **Illustration**