NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT **141**

CHANGES IN LEGAL VEHICLE WEIGHTS AND DIMENSIONS

SOME ECONOMIC EFFECTS ON HIGHWAYS

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:

- TRANSPORTATION ADMINISTRATION
- TRANSPORTATION ECONOMICS
- HIGHWAY DESIGN
- PAVEMENT DESIGN
- ROAD USER CHARACTERISTICS
- TRAFFIC CONTROL AND OPERATIONS

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1973

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Highway Research Board with the approval of the Governing Board of the National Research Council, acting in behalf of the National Academy of Sciences. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.
The members of the advisory committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the advisory committee, they are not necessarily those of the Highway Research Board, the National Research Council, the National Academy of Sciences, or the program sponsors.
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FOREWORD

By Staff
Highway Research Board

This report is recommended to state highway officials responsible for planning and implementation of highway programs, as well as federal and state legislators responsible for policy decisions and actions on proposed legislation having an impact on legal vehicle weights and dimensions. It contains information on the principal factors involved in the construction, operation, and maintenance of the highway system that are related to vehicle weights and dimensions, as well as an analysis of these factors for their impacts on benefits and dis-benefits to highway users and non-users.

The Congress and the state legislatures have the continuing responsibility for considering legislation respecting legal maximum limits of motor vehicle weights and dimensions. When laws are changed, highway designers must take into consideration the effects of the new legal limits on such things as vehicle design, vehicle use of the highways, axle configurations, road axle weight distribution and frequency, and trucking practices. These factors, among others, affect management decisions relative to pavement design, bridge design, highway geometric design, over-all highway maintenance policies and procedures, methods of upgrading existing highways and bridges, and budgets for highway construction, betterments, and maintenance. Also affected are road user tax incomes and highway cost allocations.

The objectives of this project were to review critically past and current research and methodologies relating to the consequences of possible changes in legal vehicular weight and dimension limits and prepare a one-source document of information useful to highway departments and others in estimating the effects of changes in legislation relating to legal maximum weights and dimensions of vehicles. Based on the assemblage of existing knowledge, a methodology was to be recommended that identifies all decision points involved in reaching a conclusion regarding costs and benefits associated with changes in legal weights and dimension limits for vehicles.

The research team from Wilbur Smith and Associates compiled and updated data on truck configurations, truck transport economics, and state regulations on truck sizes and weights. This resulted in step-by-step procedures applicable to estimating truck traffic magnitudes should new limits on vehicle sizes be enacted, the economic effects on the trucking industry, and the benefits and disbenefits to the general public that included effects on capacity and efficiency of traffic flow and safety.

Highway engineers will find this report of special value in helping to determine the incremental cost impact on a state highway system of a proposed change in legal limits on vehicle sizes and weights.

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ACKNOWLEDGMENTS

The study reported herein was conducted by Wilbur Smith and Associates as part of NCHRP Project 19-3. Robert E. Whiteside acted as principal investigator, and John C. Cosby acted as project administrator.

Raymond L. Whitaker was responsible for the preparation of the study material and cost analysis methodology dealing with highway bridges and secondary structures. Rex Anderson and James L. Covil contributed their experience in the area of cost allocation and highway administration.

John Crosthwaite, transport economist, formerly with Wilbur Smith and Associates, was responsible for the area of transportation economics, and devised the method of predicting motor freight tons and ton-miles.

Dr. Ting Y. Chu, special consultant, prepared the portion of the report dealing with pavement design and cost analysis. Robley Winfrey, consulting civil engineer, was primarily responsible for the methods of forecasting the extent of equivalent axle applications resulting from a change in axle load limits and for updating the line-haul trucking costs.

The Federal Highway Administration is due special thanks for providing copies of unpublished documents of staff studies performed on the subject for information and guidance in this project.

Appreciation is also extended to the many state highway departments and state departments of transportation for their questionnaire responses and patient work in supplying research data and reports.

CHANGES IN LEGAL VEHICLE WEIGHTS AND DIMENSIONS

SOME ECONOMIC EFFECTS ON HIGHWAYS

SUMMARY State highway officials, state legislators, and others often are faced with policy decisions and actions on proposed legislation regarding changes in the legal vehicle weights and dimensions. The assembly of a procedure for determining the economic impacts of these changes on highways and highway freight transport and the identification of areas of further research were the principal objectives of this study.

This report has been prepared from a review of literature; it identifies principal factors involved in the construction, operation, and maintenance of the highway system that appear to relate to vehicle weights and dimensions. These factors have been analyzed for their impacts on benefits and disbenefits to highway users and nonusers.

Methods have been assembled from the state of the art that permit projection of estimated use of highway facilities by various classes of commercial vehicles, the division of motor freight among vehicle classes on principal types of highways, and estimated payloads these vehicles will transport.

In the assembled methodology, two separate estimates are proposed. One estimate projects the foregoing factors on the assumption that no change is made in present legal limits. Based on this assumption, the number of equivalent load applications by various classes of vehicles on the various highway systems required to carry the projected motor freight over the 20-year planning period is computed. The total load experience thus estimated is used to design new pavement construction and to determine the estimated remaining service life of existing pavements. Cost estimates reflect the impact of these factors, assuming no change in legal limits.

The second estimate repeats the process, with the assumption that the proposed limits are put into use. Reestimates of the total equivalent axle load applications required to carry the same total projected motor freight are made and applied to pavement design for new construction and to remaining service life of existing pavements. Bridge design standards are analyzed for the effects of new axle configurations permitted by the proposed change and likely to be used. An analysis is made of the impacts of these loads on new bridge construction and on existing bridge structures and decks. Cost estimates are made of the impacts that a change in legal limits would have on these elements. These computations then reflect the extent of application of proposed new limits by the motor transport industry. By this means, an incremental cost analysis, indicated by differences between the two cost estimates, can be made.

Pavement structural weakening is estimated by using equivalent 18-kip single-axle equivalence factors developed in the AASHO Road Test and modified to conform to local conditions where such correlations have been established. The total

anticipated equivalent 18-kip load applications are estimated. Methods of determining remaining service life from PSR or PSI pavement indices have been used to determine pavement reconstruction requirements. Design and cost factors for new pavements are analyzed in a similar fashion.

Two methods for estimating the cost impacts on existing and planned pavement structures, varying in detail and scope, are recommended. A numerical example of the application of each method is given.

A method was developed for estimating the cost impacts of changes in legal limits on bridges and secondary structures. By determining the overstress ratio for the anticipated new vehicle, a permissible overstress ratio criterion is assumed. Using this criterion, existing bridges in the inventory are categorized as: (1) those requiring replacement or strengthening, or (2) those that can be upgraded without change. Cost estimates of reconstruction and strengthening are based on a material quantity ratio (the ratio of materials for structures required under present weight limits and axle configurations to those required to meet new vehicular loads). The effects of new limits on fatigue life of structures and on maintenance costs are considered.

By application of these methods, an estimate can be made of impact on physical properties of the highway. By the incremental method, cost analyses can be made of economic impacts of changes in legal limits on the physical components of the highway system. Decisions can be based on the magnitude and character of these differences. Numerical examples of application of these methods appear in the appendices.

The impacts on highway geometric design of vehicle size, weight, and performance characteristics are analyzed. The interactions of truck length, acceleration capabilities, braking performance, offtracking characteristics, weight/horsepower ratios, gradeability, and traction are related to geometric design items such as: grades, truck passing lanes, lane widths, and curve widths on open highways and at intersections. These relationships are used to evaluate existing design practices and to determine desirable modifications thereto. Economic impacts of these modifications can then be projected after detailed analysis has been performed.

A method is presented for estimating the accident incidence rate for vehicles whose speed distribution is different from average highway speed, originally applied as warrants for truck climbing lanes. A numerical example of the application of this method is included.

Highway classification and needs studies and methods of highway cost allocation are reviewed to relate the sources of inputs to the assembled methods and to identify possible application of their outputs when one is considering means of minimizing physical cost impacts.

Oversize and overweight permit operations are reviewed. AASHO Policy recommendations for this type of regulation are summarized.

Benefits resulting from the changes in legal limits are identified. These include transport cost reductions that the motor freight industry might realize because of increased efficiencies resulting from liberalized limits. There is a general discussion of the economic contributions of the motor freight mode of transport. Types of commodities and cargo density are related to gross weights, axle weight limits, and vehicle cubic capacity.

Disbenefits, penalties that may be imposed on other users of the highway system as a result of increases in legal limits, are discussed. These disbenefits include changes in safety of the highway system, due primarily to possible widening of the disparities between truck/bus and passenger-car performance. "Social costs" related to noise, air pollution, property values, and vibration are discussed, and the current state of the art is summarized.

The study concludes that a cost/benefit analysis method can be applied as a limited decision factor within the present state of the art. The method for this application included here would scale the costs required to construct one mile of new highway, including a pro rata share of bridge structures, to the benefits to the truck operator in reducing the operating costs of moving motor freight over that hypothetical mile. In this manner, a basic but limited cost/benefit ratio can be computed.

Methods of determining both benefits and costs resulting from a change in dimensions are not presently mature enough to be included. Cost impacts related to social and environmental factors, safety, and geometric design are also not sensitive to changes in legal limits. Some of these factors must rely on technical judgments and some on subjective judgment, with little or no relationship to a technical base. Research into these identified gaps in knowledge is suggested, as are activities that might improve the applicability of current methods.

The study was not expected to make, nor did it arrive at, specific conclusions as to the desirability of changes in legal vehicle weights and dimensions. The recommended methods must first be applied by others to specific limit changes before conclusions can be drawn.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

The Congress and state legislatures have the continuing responsibility for considering legislation regarding legal maximum limits of motor vehicle weights and dimensions. When laws are changed, highway designers must take into consideration the effects of the new legal limits on such things as vehicle design, vehicle use of the highways, axle configurations, road axle weight distribution and frequency, and trucking practices. These factors, among others, affect management decisions relative to pavement design, bridge design, highway geometric design, over-all highway maintenance policies and procedures, methods of upgrading existing highway and bridges, and budgets for highway construction, betterments, and maintenance. Also possibly affected are road use tax income and highway cost allocations. However, absence of a clear definition of such things as the interrelationship between changes in the law and legal limits on the highway contributes to uncertainty and makes legislation and management decisions difficult. A further difficulty is that the knowledge helpful to making decisions on the many factors involved is relatively scarce and widely scattered throughout the literature and the disciplines. A synthesis of the knowledge and development of guidelines for evaluating the effects of such legislative changes could make this knowledge more usable by highway departments and others making decisions regarding the consequences of changes in the legal limits of vehicle weights and dimensions. Project 19-3 was formulated in recognition of this need.

PROJECT OBJECTIVES

Four principal project objectives were cited. The first objective was a critical review of past and current research and methods relating to the consequences of possible changes in legal vehicle weight. Information sources included legislative hearing records and published and unpublished material in highway departments, federal agencies, and consulting and research agencies.

The second objective was to evaluate the reliability, adequacy, ease of application, etc., of the methods and procedures identified in the review.

As a result of this effort, the program called for the assembly from existing knowledge of a recommended method or methods identifying all decision points involved in reaching a conclusion regarding costs and benefits associated with changes in legal limits of vehicle weights and dimensions. These methods were to include step-by-step procedures based on the review, evaluation, and synthesis of this established state of the art. Illustration by numerical example of the application of the recommended method was an essential part of this objective.

The fourth objective was the recommendation of additional research and development needed to fill gaps in present knowledge.

The project was to include preparation of a single document of information useful to highway departments and others in estimating the effects of changes in legislation relating to legal limits.

RESEARCH WORKING PLAN

The output of the 18-month study is this report, which contains the essential information developed during the study. This document describes the assembled step-by-step method that will enable highway officials and others to determine the economic effects of changes in legal vehicle weights and dimensions on highways and the administration of their highway departments. This report contains the results of application of this method to a numerical example to illustrate its use.

The study was divided into nine topical phases:

1. *Data Review.* Existing state-of-the-art data were assembled and a critical review of past and current research, present policies, and methods was performed.
2. *Data Assembly.* As the data review was conducted, summaries of the data were maintained. In addition, interviews with and questionnaires to selected state highway officials furnished data on their design and administrative procedures, problems, and other information related to the study.
3. *Truck Operations.* So that highway officials can develop an understanding of the role of motor freight operations within their states, traffic operations and truck transport economics were combined. This resulted in step-by-step procedures that can be applied to estimate the truck traffic magnitude should the new limits be enacted, the economic effects on the truck industry, and the benefits and disbenefits to the general public, including effects on capacity and efficiency of traffic flow and safety.
4. *Truck Configurations.* A means was developed of evaluating the possible new configurations of trucks to be used if new limits are enacted. Methods of estimating axle configurations, axle weights, gross weights, and other significant truck parameters that could be affected by the new limits were developed.
5. *Physical Effects.* A study was made of the effects of incremental increases in loads, both magnitude and frequency of application, on roadway pavements, subgrades, and soils, and on bridge structures and decks. This phase included the determination of changes in design practices required to reconstruct or strengthen existing structures and pavements, for new construction, and to maintain these facilities.
6. *Cost Impacts.* Once the magnitude, frequency of application, and nature of additional loads had been projected and the physical effects of these factors on the highway facilities had been determined between the former costs and those resulting from a change in limits, an incremental cost method was assembled.
7. *Highway Administration.* Cost impacts of changes were compared with benefits derived by both the truck industry and the general public. These comparisons were then used to develop recommended financial policies and other economic impacts on the highway departments. Methods of funding the additional costs were studied and compared for possible implementation.
8. *Cost/Benefit Analysis.* Methods were developed of determining the costs of supplying highway facilities against the general benefits from the new limits.
9. *Numerical Example.* The method was applied to demonstrate the principles of method application to typical assumed conditions.

BACKGROUND

Separately, each state has established laws and regulations as the carrying capacity of motor vehicles and traffic volume have increased. Laws were adopted that limited axle and gross weights, principally for safety and to protect highway facilities from possible damage by heavy vehicles. Numerous laws and regulations regarding limitations on width, height, and length of trucks, tractor-trailer combinations, and buses, together with limits on gross weights, axle weights, tires, and axle spacings, were enacted by each state to reflect its interpretations of needs, specific highway design practices, and widely varying soils, topography, and climatic conditions. These limits are given in Table 1.

These limits had an impact on truck design; some trucks would meet the legal requirements of one state but not of others. These variations multiplied the varieties of trucks that were produced to meet trucking needs and reduced the efficiency of interstate truck transport.

To study the problem and formulate recommendations as to policy that might be applied nationally, AASHO organized the Committee on Highway Transport, with charter to

> . . . investigate and evaluate the various transportation needs that should be served by the highway system in the United States; determine the degree to which these needs are met by the highway system in its current state of improvement, under existing regulatory laws; and recommend such policies, regulations, laws, and practices as may contribute to improve the efficiency of highway transportation with due regard for the conservation and cost of the highway plant.

Based on the efforts of this Committee, AASHO adopted its first policy concerning maximum dimensions, weights, and speeds of motor vehicles in November 1932.

To study the variations of these laws and regulations and to investigate the need for federal legislation of motor vehicle weights and dimensions, Congress, by the Motor Carrier Act of 1935 and the Transportation Act of 1940, directed the Interstate Commerce Commission (ICC) to make such studies and reports. Although the resulting report suggested that the ICC should be authorized to remove unreasonable discriminations against interstate commerce, federal regulation did not come about for almost 16 years.

An important and relevant program—the AASHO Road Test—was instigated by AASHO and conducted by the Highway Research Board with cooperation of many other agencies and organizations. The research, started in 1955, was "to furnish certain research information to aid in establishing an optimum balance between the best use and the best life of the highway," in addition to developing design practices of flexible and rigid pavements and evaluating certain bridge structural design practices. Among other applications, AASHO was to use the results of the tests to formulate recommended sizes and weights for highway vehicles. Test results were published in a series of special reports in 1962. These data were first reflected in the AASHO Dimensions and Weights Policy (*1*) of December 7, 1964 (revised January 15, 1968).

The first federal regulation of size and weight of motor vehicles was provided in the Federal-Aid Highway Act of 1956 and was applicable to the Interstate System (*2*). A section of this act specified a maximum single-axle weight of 18,000 lb, or a maximum tandem-axle weight of 32,000 lb and an over-all gross weight of 73,280 lb. The only dimensional limit, that of vehicle width, was specified as 96 in. The statute contains a "grandfather" clause that permits the use of individual state limits, in effect on July 1, 1956, which allow greater limits on public highways in a state than those given.

Subsequent to the enactment of this legislation in 1956, three Congresses have considered revisions to the size and weight limits of motor vehicles using the Interstate System. The second session of the 90th Congress, in Senate Bill S. 2658 (*3*), considered changes to the statute that would increase the width limit from 96 to 102 in., the single-axle load limit from 18,000 to 20,000 lb, and the tandem-axle load limit to 34,000 lb, and a maximum gross weight as determined by a "bridge formula."

The Subcommittee on Roads of the Committee on Public Works, U.S. Senate, conducted hearings in early 1968 on this proposed legislation (*3*).

It would be impractical to summarize all viewpoints of testimony heard; however, a few selected positions are cited here. The U.S. Department of Transportation (DOT) submitted during the hearings an amended version of S. 2658 which principally modified the "bridge formula" of the original bill, reducing a constant in that expression which, in essence, would reduce by 2,000 lb the maximum overall gross weight permitted the critical design vehicle. Representatives of the AASHO Transport Committee presented their current recommended policy on maximum dimensions and weights of motor vehicles at these hearings (*1*).

Representatives of American Trucking Associations testified generally in favor of S. 2658, although they entered statements rebutting testimony of others regarding safety of trucks and maintained that the results of the AASHO Road Test were "invalid as predictors of true pavement life."

Representatives of the American Automobile Association (AAA) opposed the bill, primarily on the question of safety. Kachlein (*3*) maintained that S. 2658 "is really an antisafety bill. Enactment of this legislation would increase present hazards and create new ones." AAA opposed a dual set of truck standards—one for Interstates and another for other routes. It also recommended: elimination of the "grandfather" clause by stipulating a time interval for all states to conform to the lower limits of existing federal law; comprehensive study to determine the effects of proposed increased sizes and weights on bridges and the direct costs to upgrade structures to conform to standards contained in the bill; that all costs occasioned by increases in sizes and weight be charged exclusively to the user group (heavy trucks) who occasioned such costs; and that a weight/horsepower ratio of a maximum 400 to 1 be required of all trucks.

The Association of American Railroads strongly opposed the bill, primarily for the following reasons: the "present existing inequitable" situation between motor carriers and railroads would be aggravated; proper assignment of cost burden required for safe highway operation was not identified in the bill; and need for further economic study of vehicle sizes and weights made action on the bill premature.

The Western Conference of the Council of State Governments introduced as testimony Council resolutions that called for immediate retirement of the Federal Government from the field of regulation of motor vehicle sizes and weights and the restoration to the individual states of their "rightful and traditional" authority in this field. The resolution further called for the individual states to reexamine their policies with respect to the size and weight of commercial vehicles in order to achieve more efficient and economic highway transportation services. If federal regulation were to continue, the Council called for the establishment of standards that would be consistent with the "geometrics and structural capacity of western highways now being built and the future needs of the western economy."

This bill passed the Senate but not the House of Representatives and therefore was not enacted by the 90th Congress.

The 91st Congress considered two bills—H.R. 11870 and H.R. 11619, which proposed amendments to Section 127 of Title 23, U.S.C. The former related to vehicle weight and width limitations on the Interstate System; the latter referred to an increase in the width limit for motor buses.

H.R. 11870 proposed a limit for single axles of 20,000 lb, a tandem-axle limit of 34,000 lb, and a maximum width of 102 in. The over-all gross weight of any group of two or more consecutive axles was established in the bill by the "bridge formula" previously recommended in testimony by DOT in hearings on S. 2658. A gross load limit of two consecutive sets of tandem axles spaced 36 ft or more apart was set at 68,000 lb, notwithstanding the results of the bridge formula.

H.R. 11619 proposed an amendment to the same Code to permit a 102-in. width of motor buses (motor vehicles designed to carry more than 10 persons).

Hearings on both bills were conducted by the Subcommittee on Roads of the Committee on Public Works, House of Representatives, in mid-1969 (*4*). The 835-page public record on these hearings almost tripled that of S. 2658. The record covers the testimony of 39 individuals, 17 written statements, and 118 other materials submitted

TABLE 1 STATE LEGAL MAXIMUM DIMENSIONS AND WEIGHTS OF MOTOR VEHICLES

Line	State	Width inches[1]	Height ft.-in.	Length-feet[2]					Number of towed units[3]			Axle load-pounds				Operating tire inflation pressure pounds per sq. in.
				Single unit			Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Single		Tandem		
				Truck	Bus	Semi-trailer or trailer						Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance	
1	Alabama	96	13-6	40	40	NS	55	NP	1	NP	NP	18,000	19,800	36,000	39,600	NS
2	Alaska	96	13-6	40	40	[7]NR	60	65	1	1	2	20,000		34,000		NS
3	Arizona	96	13-6	40	40	[7]NS	65	65	1	1	2	18,000		32,000		NS
4	Arkansas	96	13-6	40	40	NS	55	65	NR	NR	NR	[8]18,000		32,000		NS
5	California	96	[9]13-6	40	40	[71]40	60	65	NR	NR	NR	18,000		32,000		NS
6	Colorado	[10]96	[11]13-6	35	40	NR	[22]65	[22]65	1	2	2	18,000		36,000		NS
7	Connecticut	102	13-6	55	55	NR	55	NP	1	NP	NP	22,400	22,848	36,000	36,720	NS
8	Delaware	96	13-6	40	42	40	55	65	1	1	NP	[63]20,000		36,000		NS
9	Florida	96	13-6	[14]40	40	NS	55	55	1	1	NP	20,000	22,000	40,000	44,000	NS
10	Georgia	96	13-6	55	55	NR	55	55	NR	NR	NR	18,000	20,340	36,000	40,680	NS
11	Hawaii	108	13-6	40	40	NR	55	65	1	1	2	24,000		32,000		NS
12	Idaho	[10]96	14-0	[18]35	[18]35	NR	[19]60	[64]65	1	1	[64]2	[20]18,000		[20]32,000		NS
13	Illinois	96	13-6	42	42	42	55	[24]60	1	1	2	[23]18,000		32,000		NS
14	Indiana	96	13-6	36	40	NR	55	65	1	1	2	18,000	19,000	32,000	33,000	NS
15	Iowa	96	13-6	35	[6]40	NR	[26]55	[26]55	1	1	2	18,000	18,540	32,000	32,960	NS
16	Kansas	96	13-6	42' 6"	42' 6"	NS	55	65	1	1	2	18,000		32,000		NS
17	Kentucky	96	[27]13-6	[27]35	[27]35	NR	[29]55	[65]65	1	1	2	18,000	18,900	32,000	33,600	NS
18	Louisiana	96	13-6	35	[6]40	NR	60	65	1	1	NP	18,000		32,000		NS
19	Maine	[28]102	[31]13-6	55	55	NR	55	55	1	1	[39]NP	22,000		[28]36,000		NS
20	Maryland	[10]96	13-6	55	55	NR	55	[30]65	NR	NR	NR	22,400		[35]40,000		NS
21	Massachusetts	96	13-6	35	[6]40	NR	55	NP	1	NP	NP	22,400		36,000		NS
22	Michigan	96	13-6	40	40	NR	55	[25]65	1	1	2	[37]18,000		[38]32,000		NS
23	Minnesota	96	13-6	40	40	[66]40	55	55	1	1	NP	18,000		32,000		NS
24	Mississippi	96	13-6	35	40	NR	55	55	1	1	NP	18,000		28,650	[22]32,000	NS
25	Missouri	96	13-6	40	40	NR	55	[41]55	NR	NR	NR	18,000		32,000		NS
26	Montana	[10]96	13-6	35	40	NR	60	60	1	1	2	18,000		32,000		NS
27	Nebraska	96	13-6	40	40	[7]NR	60	65	1	1	2	20,000		34,000		NS
28	Nevada	96	NR	40	40	NR	70	70	NR	NR	NR	18,000	18,900	32,000	33,600	NS
29	New Hampshire	96	13-6	35	40	NR	55	55	1	1	NP	22,400		36,000		NS
30	New Jersey	96	13-6	35	[42]35	[7]40	55	55	1	1	NP	22,400	23,520	32,000	33,600	NS
31	New Mexico	[43]96	13-6	40	40	NR	65	65	1	1	2	21,600		34,320		NS
32	New York	[10]96	13-6	35	40	NR	55	55	1	1	NP	22,400		36,000		NS
33	North Carolina	96	13-6	35	[6]40	NR	55	55	1	1	NP	18,000	19,000	36,000	38,000	NS
34	North Dakota	[10]96	13-6	[6]40	[6]40	NR	[24]60	[24]60	1	1	2	18,000		32,000		NS
35	Ohio	96	13-6	40	[6]40	NR	55	65	1	NR	NR	19,000	19,570	[44]32,000	32,960	NS
36	Oklahoma	96	13-6	40	45	NR	55	65	1	1	2	18,000		32,000		NS
37	Oregon	96	[68]13-6	35	[22]40	[22]40	[21]50	[22]75	1	1	[22]2	[46]20,000		[46]34,000		NS
38	Pennsylvania	96	13-6	35	40	40	55	[34]55	1	1	NP	22,400	23,072	36,000	37,080	NS
39	Rhode Island	102	13-6	40	40	40	55	55	1	1	NP	22,400		NS		NS
40	South Carolina	96	13-6	[6]40	[6]40	NR	55	[54]55	1	1	NP	20,000		[55]32,000		NS
41	South Dakota	96	13-6	35	40	NR	65	65	1	1	2	18,000		32,000		NS
42	Tennessee	96	13-6	40	40	NS	55	55	1	[56]1	NP	18,000		32,000		NS
43	Texas	96	13-6	40	40	NR	65	65	NR	NR	NR	18,000		32,000		NS
44	Utah	96	14-0	45	45	45	60	60	NR	NR	NR	18,000		33,000		NS
45	Vermont	96	13-6	50	50	NS	55	55	1	1	NP	22,400	23,520	[57]36,000		NS
46	Virginia	96	13-6	35	40	NS	55	55	1	1	NP	18,000		[58]32,000		NS
47	Washington	96	13-6	35	40	40	[19]60	65	1	1	2	18,000		32,000		NS
48	West Virginia	96	[33]12-6	35	[6]40	35	50	50	1	1	NP	18,000	18,900	32,000	33,600	NS
49	Wisconsin	96	13-6	35	40	35	55	55	1	1	NP	18,000	[60]19,500	30,400	32,000	NS
50	Wyoming	96	13-6	50	40	NR	65	[34]65	1	1	2	18,000		32,000	[62]36,000	NS
51	District of Columbia	96	12-6	40	40	NS	55	55	1	1	NP	22,000		38,000		NS
52	Puerto Rico	96	13-6	35	40	NS	50	50	1	1	NP	NS		NS		NS
	AASHO Policy - 1946	96	12-6	35	40		50	60	1	1	NP	18,000		32,000		NS
Number of States	Higher	4	50	31	10		49	24	8	10		21		22		
	Same	48	2	21	39		3	4	44	39		31		28		
	Lower	0	0	0	3		0	24	0	3		0		2		
	AASHO Policy - 1968	102	13-6	40	40	40	55	65	1	1	2	20,000		32,000		95
Number of States	Higher	1	3	9	10	42	16	2	8	10	10	14		22		
	Same	3	47	22	39	7	33	19	44	39	19	6		28		[13]
	Lower	48	2	21	3	3	3	31	0	3	23	32		2		

NP—Not permitted. NR—Not restricted. NS—Not specified.

[1]Various exceptions for farm and construction equipment; public utility vehicles; house trailers; urban, suburban, and school buses; haulage of agricultural and forest products; at wheels of vehicles for safety accessories, on designated highways, and as administratively authorized.

[2]Various exceptions for utility vehicles and loads, house trailers, mobile homes and urban, suburban and school buses.

[3]When not specified, limited to number possible in practical combinations within permitted length limits; various exceptions for farm tractors, mobile homes, etc.

[4]Legally specified or established by administrative regulation.

[5]Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits:

(1) Minimum front overhang of 3 feet; minimum spacing from first to second axle of truck tractor 8 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semi-trailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhang assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups on total wheelbase as applicable.

C. Including statutory enforcement tolerance as applicable.

[6]Less than three axles 35 feet.

[7]Trailer 35 feet in New Jersey, 40' in Nebraska, Alaska and Arizona.

[8]Steering axle 12,000 pounds.

[9]Load on vehicle may exceed 13' 6" but not exceed 14' 0".

[10]Buses 102 inches on certain highways as administratively authorized.

[11]On class AA, or designated highways, 12 ft 6 in. on other highways.

[12]On Interstate system only. 79,900 lbs. on primary and secondary highways.

[13]Not yet specified in any state law.

[14]Two-axle truck 35 feet; three-axle truck 40 feet.

[15]Formula W=500 (LN/N minus 1 plus 12N plus 36 where W=Gross Weight L=Wheelbase in feet and N=Number of Axles. The formula provides for maximum gross weight allowed on any vehicle or combination.

[16]73,280 pounds maximum, except on roads under Rural Roads Authority 56,000 pounds maximum.

[17]800 (L plus 40) where L is distance between first and last axle of vehicle except that 700 (L plus 40) governs for any group of 11 or more consecutive axle whose L is 13 feet or less: Alternate Load Determination by table for vehicles of 3, 4 or 5 axles for L between 19 feet and 51 feet provided single axle load limited to 18000 pounds or less: 900 (L plus 40) on highways which have no structures with span of 20 feet or over.

[18]On designated highways 40 feet.

[19]Auto transports on designated highways 65 feet in Idaho, 70 feet in Washington if equipped as specified.

[20]Special limits for vehicles hauling including livestock; single axle 18,900 pour 66,000 pounds maximum at 21-foot axle sp spacing.

[21]60 ft. in special cases: Indiana, tru trailers on designated major routes.

[22]On designated highways only.

[23]On designated highways; 16,000 pou

[24]Truck tractor semitrailer drawing o highway or any other highway designated by

[25]On designated highways only. Auto rear of vehicle. Auto or boat transporter exce

[26]Auto and boat transports and three-

[27]On state maintained highways; on ot

[28]On Interstate System only maximun

[29]On state maintained highways; 30 fe

[30]On four lane highways only.

[31]Including load 14 feet; various excep

[32]But not less than 30 net brake horse

[33]Auto transports 13 feet 6 inches; ot

[34]Exception for poles, pilings, structur

[35]Less than 48-inch spacing, 36,000 po

[36]Subject to axle and tabular limits.

[37]Single axle spaced less than 9 feet fr

[38]On designated highways only and li does not exceed 73,250 pounds; 2 tandem ax

[39]Drive away, towaway operations as exceed 3 units in contact with the surface of

[40]On Interstate System 47,500 pounds

[41]Auto transports permitted 60 feet. therefrom and other designated routes.

[42]Or as prescribed by P.U.C.

[43]On designated highways 102 inches.

[44]32,000 pounds if over 4 feet but les feet.

[45]98' combinations required 1 net bhp

[46]On Interstate System logging vehicl 18,000 pound single axle and 32,000 pound t

COMPARED WITH AASHO STANDARDS (Prepared by AASHO, December 31, 1970)

Gross weight limit				Specified maximum gross weight-pounds[4]						Practical maximum gross weight-pounds[5]						Line
		Applicable to:		Truck		Truck-tractor semitrailer				Truck		Truck-tractor semitrailer				
Pounds per engine net horsepower delivered to clutch or equivalent	Type of restriction	Any group of axles	Total wheelbase only	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	
NS	Table		X					73,280		27,800	47,600	47,600	67,400	73,280	NP	1
NS	Table-tire cap.		X	40,000	54,000	60,000	74,000	88,000	100,000	28,000	42,000	48,000	62,000	76,000	90,000	2
NS	Table	Under 18'	Over 18'							26,000	40,000	44,000	58,000	[72]72,000	76,800	3
NS	Spec. maximum			30,000	44,000	48,000	62,000	73,280	73,280	26,000	40,000	44,000	58,000	72,000	73,280	4
NS	Table	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	5
NS	Formula-spec. lim.		X	30,000	40,000					26,000	44,000	44,000	62,000	76,000	76,000	6
NS	Spec. lim.-tire cap.			32,000	53,800	53,800	67,400	73,000	NP	30,848	44,720	53,800	67,400	73,000	NP	7
NS	Table-spec. lim.	X		30,000	65,000	48,000				28,000	44,000	48,000	64,000	73,280	73,280	8
NS	Table		X							30,000	52,000	52,000	73,271	73,271	73,271	9
NS	Spec. max.[16]							73,280	73,280	28,340	48,680	48,680	69,110	73,280	73,280	10
NS	Formula[17]	X								32,000	40,000	56,000	64,000	72,000	80,000	11
[45]NS	Table[20]	X								26,000	40,000	44,000	58,000	73,280	76,800	12
NS	Spec. lim.-tire cap.			36,000	[36]50,000	[36]50,000	[36]64,000	[36]73,280	[36]73,280	26,000	40,000	44,000	58,000	72,000	73,280	13
NS	Spec. lim.-tire cap.			36,000	50,000	54,000			72,000	27,000	41,000	44,000	58,000	[25]73,000	[25]73,000	14
NS	Table	X								26,540	40,960	45,080	59,500	73,280	73,280	15
NS	Table		X	[36]36,000	[36]50,000	[36]54,000	[36]72,000	[36]73,280	[36]73,280	26,000	40,000	44,000	58,000	72,000	73,280	16
NS	Spec. lim.-tire cap.			30,000	44,000	44,000	62,000	73,280	73,280	26,900	41,600	42,000	59,640	73,280	73,280	17
NS	Axle lim.-tire cap.									26,000	40,000	44,000	58,000	72,000	76,000	18
NS	Table-tire cap.	X		32,000	46,000	51,800	66,300	73,280	73,280	30,000	44,000	51,800	66,000	73,280	73,280	19
NS	Table		X			[36]55,000	[36]65,000	[36]73,280	[36]73,280	30,400	48,000	52,800	65,000	73,280	73,280	20
NS	Table-spec. lim.		X	[36]46,000	[36]60,000	[36]67,200	[36]73,000	[36]73,000	NP	30,400	44,000	52,800	66,400	73,000	NP	21
400	Axle lim.-tire cap.							73,280		26,000	40,000	44,000	58,000	72,000	138,000	22
NS	Table	X						73,280	73,280	26,000	40,000	44,000	58,000	72,000	73,280	23
NS	Table-tire cap.	X								26,000	[22]40,000	44,000	58,000	[22]72,000	[22]73,280	24
NS	Table		X							26,000	40,000	44,000	58,000	[22]72,000	[22]73,280	25
NS	Table–formula[15]	Under 18'	Over 18'	40,000	60,000	60,000	80,000	85,500	[70]105,500	26,000	40,000	44,000	58,000	73,280	76,800	26
NS	Table	X		40,000	60,000	60,000	80,000	85,500	95,000	28,000	42,000	48,000	62,000	77,500	80,500	27
NS	Table	Under 18'	Over 18'							26,900	41,600	45,800	60,500	75,200	76,800	28
NS	Table-spec. lim.		X	33,400	[40]55,000	52,800	66,400	73,280	73,280	30,400	44,000	52,800	66,400	73,280	73,280	29
NS	Axle lim.-tire cap.									31,520	41,600	55,040	65,120	73,280	73,280	30
NS	Table	Under 18'	Over 18'							29,600	42,320	51,200	63,920	76,640	86,400	31
NS	Formula		X					71,000	71,000	30,400	44,000	52,800	66,400	71,000	71,000	32
NS	Spec. lim.			31,500	49,875	49,875	67,200	73,280	73,280	27,000	46,000	46,000	65,000	73,280	73,280	33
NS	Formula	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	73,280	34
NS	Formula[59]		X							27,570	40,960	46,000	58,500	71,000	78,000	35
NS	Table	X		[67]18,000	[67]32,000	[67]36,000	[67]50,000	[67]64,000	73,280	26,000	40,000	42,000	60,000	73,280	73,280	36
NS	Table[47]	18' or Under	Over 18'					[47]76,000	[47]76,000	[69]28,000	[69]42,000	[69]48,000	[69]62,000	[69]76,000	[47]76,000	37
[32]450	Spec. lim.[49]			44,000	56,000	50,000	60,000	73,280	73,280	31,072	45,080	51,500	61,800	73,280	73,280	38
NS	Spec. lim.			[50]36,000	[51]44,000	[52]53,800	[53]67,400	73,280	88,000	30,400	44,000	53,800	67,400	73,280	88,000	39
NS	Spec. lim.			35,000	46,000	50,000	65,000	73,280	73,280	28,000	40,000	48,000	60,000	72,000	73,280	40
NS	Table	X								26,000	40,000	44,000	58,000	72,000	73,280	41
NS	Spec. lim.		X	30,000	44,000	48,000	62,000	73,280		26,000	40,000	44,000	58,000	72,000	43,500	42
NS	Table	X								26,000	40,000	44,000	58,000	72,000	72,000	43
NS	Table	X		36,000	51,000	54,000	69,000	79,900	79,900	26,000	41,000	44,000	59,000	74,000	79,900	44
NS	Table-tire cap.	X						73,280	73,280	31,520	44,000	55,000	66,400	73,280	73,280	45
NS	Table		X	36,000	50,000	54,000	68,000	70,000	70,000	26,000	40,000	44,000	60,000	70,000	70,000	46
NS	Table	Under 18'	Over 18'	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	47
NS	Table	X		36,000	54,000	54,000	[59]70,000	70,000	70,000	26,900	41,600	45,800	60,500	73,280	73,280	48
NS	Table[61]	X								27,500	40,000	47,000	59,500	73,000	73,000	49
NS	Table	X								26,000	44,000	44,000	62,000	73,950	[12]73,950	50
NS	Table-tire cap.	X						70,000	70,000	30,000	46,000	52,000	68,000	70,000	70,000	51
NS	Spec. lim.-tire cap.															52
NS	Table	X								26,000	40,000	44,000	55,470	61,490	71,900	
										29 22 0	29 21 1	26 24 1	51 0 0	51 0 0	45 0 6	
400	Table	X								28,000	40,000	48,000	60,000	72,000	86,500	
	Formula 5 Table 33 Spec. lim. 14	18	21							15 5 31	30 20 1	15 5 31	25 4 22	26 20 5	3 0 48	

timber and timber products, ores, concentrates, aggregates, and agricultural products
ıds, tandem axle 37,800 pounds, gross weight table; vehicle with 3 or 4 axles permitted
ıacing, vehicle with 5 or more axles permitted 79,000 pounds maximum at 43-foot axle

ıcks pulling house trailers and sectionalized buildings only; Oregon, truck tractor semi-

ınds on other highways.
ne trailer or tractor stinger-steered semitrailer or auto transport 65 feet. (on any 4 lane
the Department in Illinois; on highways designated by Commissioner in North Dakota)
and boat transporters permitted 65' plus an additional 3' for load beyond the front or
eding 60' may be operated on designated routes only.
ınit combinations permitted 60 feet.
her highways 11.5 feet high; trucks 26.5 feet and buses 30 feet long.
ı width 96 inches; tandem axle load 32,000 pounds.
et on other highways.

ıtions for vehicles hauling forest products and construction materials.
power.
ıer vehicles 13 feet 6 inches on designated routes.
al units, rowing shells etc., permitted 70 feet.
ıunds.

ım nearest axle limited to 13,000 pounds.
mited to one tandem axle in combination; otherwise 26,000 pounds. When gross weight
le assemblies shall be permitted 16,000 pounds per axle.
defined by the State P.U.C. may include a combination of saddlemount vehicles not to
the highway.

Double bottom units 65 feet on state primary and Interstate highways plus 5 miles

Body restricted to 96", additional 6" for tires only.
s than 8 feet apart; 24,000 pounds if less than 4 feet apart, 38,000 pounds if more than 8

per 400 lbs.
les limited to 19,000 pound single axle, 34,000 tandem axle, other vehicles limited to
andem axle.

[47] Governs gross weight permitted on highways designated by resolution of State highway commission.
[48] Does not apply if overall length of truck-tractor plus semitrailer does not exceed 50 feet.
[49] Single unit truck with 4-axle permitted 68,000 pounds.
[50] Axles spaced less than 6 feet 32,000 pounds; less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.
[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds; less than 20 feet 44,000 pounds; 20 feet or more governed by axle limit.
[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 53,800 pounds.
[53] Legal limit 67,400 pounds, axle spacing 27 feet or more.
[54] House trailers, auto transports, and double saddle mounts in daylight hours, 60 feet.
[55] On Interstate System; 36,000 pounds on other roads.
[56] Limited to 3,500 pounds.
[57] Three axle truck 55,000 pounds with no restriction on tandem. On Interstate 22,400 pounds single, 36,000 pounds tandem.
[58] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle.
[59] 38,000 plus 900 L but not greater than 78,000 where L is the distance in feet from front to rear axle.
[60] Axle load 21,000 pounds on 2-axle trucks transporting milk and dairy supplies from farm to market but not over Interstate System. 21,500 pounds on single axle, 35,000 pounds for groups of axles less than 7 feet apart, and for groups of 3 or more consecutive axles more than 9 feet apart, 4000 pounds more than in Table for vehicles transporting peeled or unpeeled forest products cut crosswise.
[61] On Class A highways. All axles of a vehicle or combination—73,000 pounds maximum. Wheel, axle, axle group and gross vehicle weights on Class B Highways are 60% of weights including tolerance authorized for Class A highways.
[62] Based on ruling of Attorney General.
[63] Axle load 21,700 pounds on 3-axle trucks. Total not to exceed 65,000 pounds. Does not apply on any Interstate Route or Turnpike.
[64] Except three or four unit combinations may use up to 98 feet on certain highways designated by the Board of Highway Directors. Combination must include on semitrailer.
[65] 60 feet for specially designed transports for motor vehicles, 65 feet for other combinations on designated highways by permit.
[66] 45 feet for trailers or semi-trailers constructred especially to haul livestock, boats or motor vehicles.
[67] Weight shown plus front steering axle not to exceed 18,000 pounds.
[68] Auto transports 14 feet by permit on designated highways.
[69] On Interstate System—26,000; 40,000; 44,000; 58,000 and 72,000 pounds respectively.
[70] Not permitted on Interstate System.
[71] Limitation does not apply to a semitrailer being towed by truck tractor providing the distance between the kingpin and the rearmost axle does not exceed 38'. The semitrailer, exclusive of attachments, shall not extend forward of the rear of the truck cab.
[72] Five axle units having 42 to 51 feet of wheelbase may gross 73,280 lbs. not to exceed the specified axle loadings of 18,000 and 32,000 lbs.

for the record. Despite this volume, the following is an attempt to provide a general scope of testimony.

AAA and the American Association of Railroads found nothing in these proposed bills to alter their opposition to S. 2658. Spokesmen for the National Association of Motor Bus Owners were in favor of both bills; they especially supported the increase in bus width to 102 in., pointing out that this change would permit wider seats that would substantially increase passenger comfort. They stated that 102-in. buses are in use in many urban and suburban transit systems and presently are permitted on the Interstate System and other federal-aid highways in eight states, on federal-aid highways other than Interstate System in seven states, and would be authorized, if legislation passes regarding the Interstate System, by present legislation in eight additional states.

AASHO presented testimony repeating its recommended policy, which was in general agreement with limits proposed. However, it disagreed with the "grandfather" clause included in H.R. 11870. AASHO had also added maximum tire-pressure limits to their recommendations for the first time.

State highway officials also expressed opposition to two separate sets of motor vehicle weights and size regulations. Special truck lanes, and in certain cases truck roadways, were recommended for policy consideration where special limitations and high truck volumes are encountered. AASHO policy retained the maximum tandem-axle load of 32,000 lb vs the proposed 34,000-lb limit by a margin of one vote. The question of 102-in. width for vehicles is left to the option of the individual state by current AASHO policy; it depends on the ability of the state's highway system to handle such widths, and on the state's prevailing soils and meteorological conditions. It was recommended that specific regulation of load on the steering axle at 10,000 lb be included. The question of fatigue stress on a majority of bridge structures was cited in the testimony. Research into bridge fatigue is required and was being undertaken at the time of hearings.

In commenting on the bill, DOT pointed to safety as its first concern, but its analysis of the record indicated that "there does not appear to be any statistical basis for assuming that vehicles of the size being proposed by these bills would be any more likely to be directly involved in accidents than the large vehicles presently on the roads." However, evidence is scarce on whether the presence of the larger vehicles might indirectly cause more accidents. DOT's work in establishing safety standards for trucks and buses was cited. DOT "would find it acceptable to allowing existing truck or other vehicle bodies to be widened to the new limit, without revising the running gear itself to the larger width." Similarly, the proposed increases in weights "should not be allowed except where they are accompanied by improved braking, higher power-to-weight ratios, wider vehicle tracks, and other beneficial features."

DOT's position on proposed weight increases was that the increase "would have relatively little significance in its effect on relative structural life of the highway." The greatest concern was the effect on the many bridges on the federal-aid system that are not built to Interstate standards, and that it was not then possible "to determine the ability of these bridges to carry either the present legal loads or the weights contemplated in the proposed legislation." As a matter of practicality, it was pointed out that trucks cannot operate exclusively on a limited highway system.

DOT held that the gross load limit of 68,000 lb in disregard of the bridge formula was objectionable, and recommended no exception be made to that equation. In place of the 70-ft limitation in length, DOT recommended a limit of 65 ft for vehicle combinations. The width limit of 102 in. was found to be acceptable, with certain safety features incorporated into the truck designs and the elimination of "tire bulge" in the specification. Deletion of the "grandfather" clause was recommended.

The bills failed to pass the 91st Congress.

Hearings were conducted in March 1971 by the same House Subcommittee on H.R. 4354, on a bill introduced in the 92nd Congress that was identical to H.R. 11619 except for the effective date of the "grandfather" clause.

Safety aspects of wider buses were the principal concern of DOT representatives: "Based on the extent of our analysis to date with regard to the potential benefits and safety hazards which would result from the proposed increased bus width, the Department cannot support enactment of H.R. 4354 at this time." Further investigation and report on the matter was promised.

The AAA representative appeared opposed to the question also, principally because of safety hazards that might be introduced and because wider buses would not be able to function on the Interstate System solely but must use other public roads that may not be built to Interstate standards.

The National Association of Motor Bus Owners pointed out the potential advantages to passengers of wider seats and aisles made possible by the additional width. The precedent of 102-in. buses in operation in many urban areas and in some states was entered into the record.

At the time of this report, no final action had been taken on this proposed legislation.

Table 2 summarizes these various proposed size and weight regulations, and compares them with other recommended laws and regulations shown in Figure 1 and given in Table 3.

In the Highway Revenue Act of 1956, Congress directed the Secretary of Commerce (who at that time administered the Bureau of Public Roads) to make a study and report on highway cost allocation. Section 210 of that act defined, in part, the purposes of this study which were:

> . . . to make available to the Congress information on the basis of which it may determine what taxes should be imposed by the United States, and in what amounts in order to assure, insofar as practicable, an equitable distribution of the tax burdens among the various classes of persons using the Federal-Aid highways or otherwise deriving benefits from such highways.

Reports resulting from this directive (sometimes referred to as the "Section 210 Study" or the "210 Study") provide an insight into the size and weight of motor vehicles (*5, 6, 7, 8*).

In cooperation with other federal offices and agencies,

TABLE 2

COMPARISONS OF SIZE AND WEIGHT LEGAL LIMITS WITH SELECTED RECOMMENDATIONS OF AASHO POLICY

LIMITS [a]	WIDTH (IN.)	HEIGHT (FT-IN.)	LENGTH (FT): SINGLE TRUCK	LENGTH (FT): SINGLE 2-AXLE OR 3-AXLE BUS	LENGTH (FT): SINGLE SEMI-TRAILER OR TRAILER	LENGTH (FT): SINGLE COMB. TRACTOR-SEMI-TRAILER	LENGTH (FT): OTHER COMB. OR TWO OR MORE UNITS	MAXIMUM PERMISSIBLE WEIGHT (LB): SINGLE AXLE	MAXIMUM PERMISSIBLE WEIGHT (LB): TANDEM AXLE	GROSS VEHICLE WEIGHT [e]
AASHO Policy, 1968 (1)	102 [b]	13–6	40	40	40	55	65	20,000 [c]	32,000 [c]	(A)
Industry Advisory Committee (2)	Cab or body=120 Over-all width at at tires=106	13–6	40	45	45	60	70	20,000	36,000	
Federal law (Interstate System) Section 127 of Title 23 U.S.C. (3)	96							18,000	32,000	73,200
Recent proposed federal legislation:										
S. 2658, 90th Congress (1968) (amend Section 127 of Title 23, U.S.C.)	102 [b]							20,000	36,000	(B)
H.R. 11870, 91st Congress (1969) (amend Section 127 of Title 23, U.S.C.) (trucks)	102 [b]						70	20,000	34,000	(C)
H.R. 11619, 91st Congress (1969) (motor buses)	102 [b]									
H.R. 4354, 92nd Congress (1971)	102							18,000	23,000 [d]	(D)
Ontario, Canada, legal limits (4)										

[a] References:
(1) AASHO (*1*).
(2) "Recommendations for Commercial Vehicle Sizes and Weights." Industry Advisory Committee, Automobile Manufacturers Association, Motor Truck Committee.
(3) "Federal Laws, Regulations and Other Material Relating to Highways." U.S. Bureau of Public Roads (1960).
(4) ARMSTRONG, M.D., ET AL. (*91*).
[b] Excluding tire bulge and approved safety devices.
[c] Local reductions permitted during periods of water saturation, frost, and other conditions.
[d] Ontario has a legal maximum for triple-axle weights of 40,000 lb on some combinations.
[e] As determined by bridge formula:
(A) $W=500\ (LN/N-1+12N+32)$ See Table 3 for computations of gross load.
(B) $W=500\ (LN/N-1+12N+40)$
(C) $W=500\ (LN/N-L+12N+36)$
(D) See Figure 1 for Ontario gross weight limits. Formula: $W_M=20+2.07\ B_M-0.0071\ B_M^2$

and state highway departments, the Bureau of Public Roads (BPR) made a comprehensive study of

(1) The effects in design, construction and maintenance of Federal-Aid highways from the use of vehicles of different dimensions, weights and other specifications, and the frequency of the occurrences of such vehicles in the traffic stream, (2) the proportionate share of design, construction and maintenance costs of Federal-Aid highways attributable to each class of user on such highways, and (3) any direct or indirect benefits occurring to any class, in addition to the benefits from the actual use of highways, which are attributable to highway expenditures (*3*).

Four different methods of allocating motor vehicle tax responsibility were included in these studies:

1. *Cost-Function Method.* This separates highway costs or other expenditures into three functional categories: (1) Weight Functions: costs that are affected by vehicle size or weight; (2) Mileage Functions: costs that vary with the use of highways, independent of size and weight; and (3) Vehicle Functions: costs that are independent of both vehicle size/weight and traffic volume that are distributed equally among all vehicles.

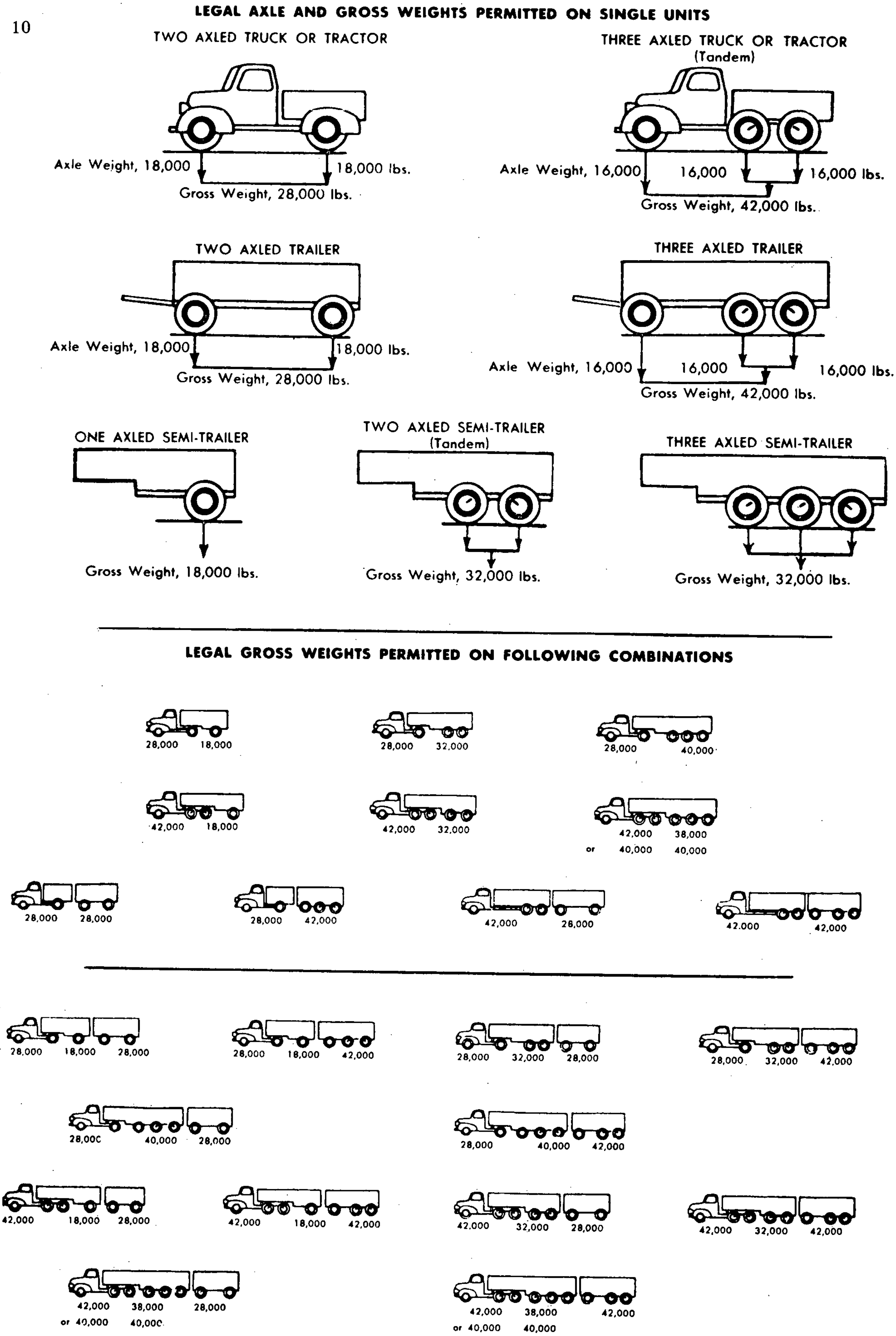


Figure 1. Permissible weights on motor vehicles in Ontario. Source: Ontario Department of Transport.

TABLE 3

PERMISSIBLE GROSS LOADS

BASED ON AASHO H15(44) BRIDGES WITH OVERSTRESSES NOT TO EXCEED 30 PERCENT FOR CERTAIN TYPICAL VEHICLES IN REGULAR OPERATION ILLUSTRATING WHEN TOTAL COMPUTED GROSS WEIGHT OR THE AXLE LOADINGS CONTROL THE PERMISSIBLE GROSS LOAD OF THE VEHICLE.

$$\text{Weight Formula } W = 500\left(\frac{LN}{N-1} + 12N + 32\right)$$

(A)—Permissible Gross Load of Vehicle Limited by Axles (B)—Computed Gross Load Controls

Distance in feet between the extremes of any group of two or more consecutive axles	Maximum load in pounds carried on any group of two or more consecutive axles								
	2 Axles	3 Axles		4 Axles			5 Axles		6 Axles
	Type 2	Type 3	Type 2-S1	Type 2-2	Type 2-S2	Type 3-S1	Type 3-2	Type 3-S2	All Types
4									
5									
6		32000							
7		32000							
8	~~32000~~	(B) 32000		Not Permitted	Not Permitted	Not Permitted			
9	~~37000~~	40500	40500						
10	~~38000~~	41500	41500						
11	~~39000~~	~~42000~~	42000						
12	~~40000~~	~~43000~~	43000	48000	48000	48000			
13		~~43500~~	43500	48500	48500	48500			
14		~~44500~~	44500	49500	49500	49500			
15		~~45000~~	(B) 45000	50000	50000	50000			
16		~~46000~~	46000	50500	50500	50500			
17		~~46500~~	46500	51500	51500	51500			
18		~~47500~~	47500	52000	52000	52000			
19		~~48000~~	48000	52500	52500	52500			
20		42,000 (A) ~~49000~~	49000	53500	53500	53500	Not Permitted	Not Permitted	Not Permitted
21	30,000 (A)	~~49500~~	49500	54000	54000	54000			
22		~~50500~~	~~50500~~	54500	(B) 54500	(B) 54500			
23		~~51000~~	~~51000~~	55500	55500	55500			
24		~~52000~~	~~52000~~	56000	56000	56000			
25			~~52500~~	56500	56500	56500			
26			~~53500~~	57500	57500	57500			
27			~~54000~~	58000	58000	58000			
28			~~55000~~	(B) 58500	58500	58500			
29			~~55500~~	59500	59500	59500			
30			~~56500~~	60000	60000	60000			
31			~~57000~~	60500	60500	60500			
32			~~58000~~	61500	61500	61500			
33			~~58500~~	62000	~~62000~~	~~62000~~			
34			~~59500~~	62500	~~62500~~	~~62500~~			
35			~~60000~~	63500	~~63500~~	~~63500~~			
36				64000	~~64000~~	~~64000~~			
37			50,000 (A)	64500	~~64500~~	~~64500~~			
38				65500	~~65500~~	~~65500~~		69500	
39				66000	~~66000~~	~~66000~~		70500	
40				66500	~~66500~~	~~66500~~		71000	
41				67000	62,000 (A) ~~67000~~	62,000 (A) ~~67000~~		(B) 71500	
42				68000	~~68000~~	~~68000~~	72000	72000	
43				68500	~~68500~~	~~68500~~	73000	73000	
44				69500	~~69500~~	~~69500~~	73500	73500	
45				~~70000~~	~~70000~~	~~70000~~	74000	74000	
46				~~70500~~	~~70500~~	~~70500~~	74500	~~74500~~	
47				~~71500~~	~~71500~~	~~71500~~	75500	~~75500~~	
48				~~72000~~	~~72000~~	~~72000~~	76000	74,000 (A) ~~76000~~	81000
49	Not Permitted	Not Permitted		~~72500~~	~~72500~~	~~72500~~	76500	~~76500~~	81500
50				~~73500~~	~~73500~~	~~73500~~	(B) 77000	~~77000~~	82000
51				~~74000~~			78000		82500
52				70,000 (A) ~~74500~~			78500		83000
53				~~75000~~			79000		83500
54				~~76000~~			82,000 (A) 79500		(B) 84500
55				~~76500~~			80500		85000
56			Not Permitted	~~77500~~	Not Permitted		81000	Not Permitted	85500
57				~~78000~~		Not Permitted	81500		86000
58				~~78500~~			~~82000~~		87000
59				~~79500~~			~~83000~~		87500
60				~~80000~~			~~83500~~		88000

Note: The loads are computed to the nearest 500 lbs.
The modifications consist in limiting the maximum load on any single axle to 20,000 lbs. and the load on any two axles spaced 8 ft. or less on centers to 32,000 lbs.
Loaded vehicles of 7 or more axles regardless of type and of wheelbase are not permitted.
Table I is applicable to gross loads of vehicles.
The permissible load on any group of 2 or more axles spaced 8 ft. or less between the centers of the extreme axles of the group is 32,000 lbs.
For the purposes of this table the steering axle is assumed to be 10,000 lbs. and 5 feet is subtracted from the maximum permissible vehicle length to arrive at the distance between the centers of extreme axles.

The procedure in the cost-function method is to allocate weight-function costs on a ton-mile basis, mileage-function costs on a vehicle-mile basis, and vehicle-function costs in proportion to the number of vehicles in each class.

In the final report (*7*) of the application of the cost-function method, the weight-function costs included: grading and drainage (intermediate and high-type roads), surface and base (intermediate and high-type roads), shoulders (high-type roads), and structures. All other expenditures were classified as mileage-function costs. In the distribution of costs, 77.2 percent were classified as weight-function and only 22.8 percent were classified as mileage-function. An allocation using this method approximated the gross-ton-mile method in imposing tax responsibility on heavier vehicles.

2. *Ton-Mile Method.* The gross-ton-mile method allocates cost responsibility among vehicles of different types, sizes, and weights, in proportion to the product of miles traveled times average operating gross weight. Although this method often has been used as a basis of cost allocation, it has been attacked repeatedly on the grounds that there is no proof or justification of the theory that either costs occasioned or benefits derived by the use of motor vehicles of different sizes are proportional to the product of distance traveled and vehicle weight.

The most convincing evidence against the ton-mile method was the results of the AASHO Road Test, where the number of applications of loads of different magnitudes did not appear to be linearly related to pavement deterioration. Wear was found to be exponentially related to increasing axle loads and their effects were related to "equivalent axle loads" where equivalency factors of different axle loads were related to that of a standard single-axle load of 18,000 lb.

3. *Incremental Method.* The basic concept here is that certain highway expenditures can be separated into increments of costs incurred in accommodating specific vehicles. Based primarily on AASHO Road Test results, each vehicle is assessed charges for those increments of expenditure creditable to a specific class of vehicles, with the heavier vehicle classes assessed for those increments of the highway system costs needed above a basic minimum system facility cost.

The incremental method is the most commonly recognized by both federal and state highway agencies for cost allocation. It is discussed in greater detail later.

4. *Differential-Benefit Method.* Here, each class of highway users should be taxed in proportion to the benefits received from the use of the highway. It appears that such benefits can be identified and at least approximately measured in terms of reductions in transportation costs, tangible and intangible, brought about by highway improvements. Unless the value exceeds or equals the corresponding costs, the improvement is not economically justified.

The major problem involved in the "210 Study" in applying this method was in the computation of benefits "differentiated according to weights and dimensions of vehicles" (*8*).

As pointed out in the S. 2658 hearing by the chairman of the AASHO Committee on Highway Transport (*3*):

> No cost allocation method actually gives a final and indisputable answer, for the problem has many ramifications, but the incremental method is so thoroughly grounded in results of highway engineering research and logic, that its findings command respect and confidence.
>
> Basically, the incremental method shows that the larger trucks, even at their present sizes and weights and not those that would be allowed under S. 2658, do not pay their total share of the highway costs.
>
> We do not raise this as a criticism, but point it out as a fact.

It must be emphasized that responsibility for establishing and enforcing appropriate vehicle size and weight limits rests principally with the individual states. State legislatures and agencies require a means of evaluating proposed changes in legal limits: (1) on the basis of impact on the economic and efficient use and appropriate financing of the highway system on a physical basis, (2) to account for total economic and social benefits and costs or disbenefits that all classes of highway users may enjoy or suffer, and (3), if possible, to account for these factors as related to the nonuser.

Costs of various types of roadway pavements, bridges, drainage and grading, and other physical properties are well understood in establishing the highway plant. The impacts of changes in axle weight and axle configurations occasioned by changes in legal limits related to these elements are relatively easily identified when compared to other aspects of this broad evaluation objective. This is due principally to the extensive research committed to the design approaches of these facilities and the experience gained in operating and maintaining them.

Transport economics and benefits of the motor freight industry are also fairly well defined in the literature, although their relationship to specific benefits to be passed on to the shipper is not as clear. This is because a variety of tariff regulations are imposed on some classes of the motor transport industry, yet other classes are not controlled because either the trucks are owned by private interests or the operators come under the "for hire" or contract category and therefore are not subject to regulation.

Other social costs—such as safety, travel-time costs, vehicle operating costs, noise, pollution, vibration, effects on property values, and rents of real estate along the highway—are not as well developed. These important issues should be included in any evaluation of changes in legal limits.

The methods assembled here should be useful in evaluating the economic impacts of either an increase or a decrease in legal limits. They should be applicable to all classifications of roads in a highway system. The methods should strike a balance between completeness and economic application, recognizing that many value inputs are based on projections or modeling techniques that always contain some possibility for error.

The objective of the study was not to arrive at any conclusions, per se, regarding changes in legal limits or the cost-benefits of any potential area of change. This can be accomplished only after addressing the specific case.

CHAPTER TWO

FINDINGS

INFLUENCES OF CHANGES IN LEGAL VEHICLE WEIGHTS ON BENEFITS AND COSTS TO THE HIGHWAY SYSTEM, THE HIGHWAY USER, AND SOCIETY

The fact that changes in legal vehicle weights and dimensions result in a complex interaction among the highway systems, transport economics, industry, and society undoubtedly explains the lack of published literature on the total problem and, indeed, the objective of this study. Determination of the economic impacts of a proposed change in legal limits involves a multitude of disciplines. One must understand the intricacies of and interactions among the influences and how they affect the economies of return to the highway users, the highway system costs in providing facilities responsive to highway use, and the costs and benefits to society resulting from a change in legal limits.

A network diagram that relates these changes in legal limits to the resulting benefit and cost elements is complex. Many interactions occur as the process proceeds from the input of the changes in legal limits to the quantification of benefits and costs. To obtain a complete solution to the problem, one must include in the analysis all possible interactions.

Figure 2 shows a generalized flow diagram of influences among the basic elements of the system that might be identified with benefit and cost components resulting from a change in legal limits (a more explicit identification of factors associated with these influences is developed later herein). One of the initial influences of a change in size and weight limits is on the vehicle's physical and performance characteristics. It is unlikely that new vehicles (using the new limits) will immediately appear on the highway, because of the time lapse in design, manufacture, and deployment. However, new vehicles eventually will appear that are responsive to transport demand of the various commodities, and that offer economic advantages to the transport factors and benefits to the motor freight industry and society.

The characteristics of the new vehicles and their use will influence pavements and bridges directly; the number of these new vehicles required to meet the transport demand, as well as their characteristics, will influence total traffic and geometric design. These two factors influence the cost of the physical elements of the roadway. Highway maintenance will be influenced by the same characteristics and the extent of vehicle use.

Geometric design is influenced by traffic operations of the new vehicles in the traffic mix and by traffic control and regulatory measures. Geometric design also is influenced by the vehicle performance characteristics.

Adverse geometric design and traffic operation generate noise, air pollution, and vibration, and unsafe congestion of the highway systems, if coupled with certain vehicle characteristics. These consequences must be translated to cost and benefit terms to permit a comprehensive analysis of the economic impacts.

Within this network, other economic, social, and community influences, not related directly to size and weight, are inputs. These must be identified and integrated into the problem solution.

If it is assumed that each of the many influences can be quantified and their interactions can be determined, it is possible to exercise the model on a "before" and "after" change basis over a given planning period (say, 20 years) to obtain the incremental cost and benefit impact of a change in legal limits. These costs and benefits are the cost differentials obtained by computations based on the assumption that (1) there is no change in legal limits ("before"), and (2) there is a change in legal limits ("after").

To accomplish this, the influences related to a change in legal limits and those external to these changes must be analyzed factorially. A method must be assembled that is sensitive to small changes to produce meaningful model responses which, in turn, permit synthesis of impacts of size and weight changes on each element of the network. These impacts must then be converted to costs or benefits to serve as basic inputs to a cost/benefit analysis. The degree of completeness of these analyses determines the degree of reliance in the decision-making process that can be placed on the cost/benefit analysis.

If the impact of a change is a decrease in costs, the impact is termed a "benefit" to that cost element. An increase in costs is interpreted as a "cost" to the element.

Intuitive Factorial Analysis of Influences of a Change in Legal Limits on Transport Industry and Social Benefits

Figure 3 shows how size and weight limits affect vehicle characteristics. Commodities to be carried as payload and cargo density greatly influence body type, axle configuration, and vehicle automotive design, engine type and size, transmission, and other features.

The vehicle characteristics that influence its use include cargo capacity, economy of size of the vehicle unit, and operating cost. These and other factors that determine the extent of vehicle use are shown in Figure 3.

Vehicle characteristics and operating costs combine with payload by vehicle class, freight tariffs, capital costs, and union contracts to influence transport economics. At this point, an example of major interaction is encountered. Transport costs and quality of service react with value of cargo, economic growth, accessibility of points of origin and destination by transportation mode, and other intermodal factors such as tariff regulations and transportation

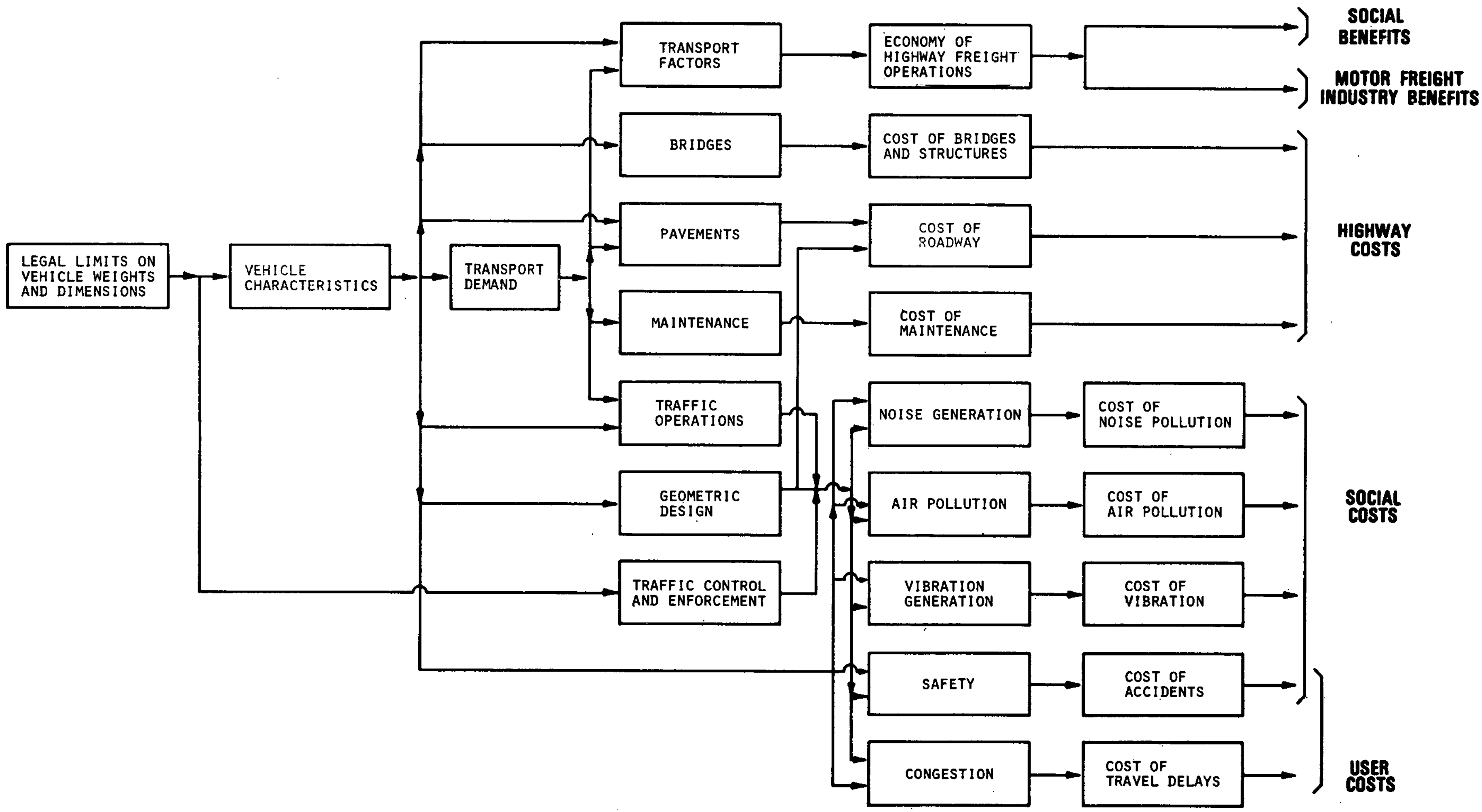


Figure 2. Influence flow between legal vehicle limits, basic elements, and benefit and costs to highways, users, and society.

policies. The total transport demand is fed back to determine the modal split moved within the state that can be assigned to motor freight.

Transport operations should produce earnings for the transport industry. Community, social net benefits are influenced by the resulting transport costs in goods movement and by the quality of service rendered. The economic development resulting from the use of high levels of service and from lower transport cost and fully accessible transportation facilities also leads to social benefits.

On Roadway Construction Costs

The input factors to vehicle characteristics and vehicle use are the same as those shown in Figure 3 when one is considering the impact on highway costs. These elements are related to the elements of roadway construction cost, as shown in Figure 4. The factors bearing on the determination of axle load applications on pavement structures are: practical maximum gross weight and axle configurations derived from the vehicle characteristics; number of vehicles, by class; and axle and gross weight distribution required to carry the projected payload over the planning period. These axle load applications are aggregated by vehicle class and highway system and applied to pavement design. In the design element, pavement type and soil support values influence the conversion of these load applications to equivalent 18-kip load applications or other design factors that it is anticipated will be applied to the pavement structure over the planning period. Allowances for environmental influences should be made in these designs. The output of this element is pavement components such as foundations, subbase, base, and wearing surfaces required to structurally support the anticipated loads on the various types of highway systems.

The new equivalent axle load applications for the various pavement designs permit identification of the impacts of these new loads on existing pavements. The length, structural type, and present serviceability, nominally available from a road inventory, together with pavement and geometric design standards, are used to determine reconstruction, improvement, and overlay needs. Applying the principle of incremental analysis, the existing system needs should be analyzed under "before" and "after" assumptions.

Highway geometric design is influenced by the factors shown in Figure 4.

Traffic operations are influenced by the factors shown in Figure 4.

There is a cause-and-effect interaction among geometric design, traffic operations, and traffic control and regulation. For simplicity, the major interactions are limited to the extent shown.

Factors of horizontal and vertical alignment influence roadway construction costs (see Fig. 4). Guardrails, drainage, and right-of-way acquisition are not assigned any cost differentials due to vehicle size and weight, although they still influence construction costs. Culverts in some instances may be influenced structurally by vehicle size and weight. Small changes in vehicle weights are not likely to influence their design, however. Unit costs and new construction needs also influence construction costs.

On Bridge Construction Costs

Bridge designs are traditionally analyzed using identified critical vehicle loads. Figure 5 shows the input factors to vehicle characteristics and vehicle use that would reflect a change in legal limits. Each likely vehicle configuration is analyzed for member stresses and moments produced by the individual vehicle loads on various bridge types. Those configurations producing critical design loads are then designated as "critical design vehicles." The influence of a change in legal limits on bridge design is measured by comparing the present critical design vehicles with the proposed design vehicles under the new limits. The critical loads are combined with current bridge design and the inventory of bridges to permit the evaluation of existing highway bridges. This analysis results in one of three decisions: (1) reinforce existing bridges; (2) construct new bridges to replace existing bridges; and (3) take no action on existing bridge structures presently suitable to carry the proposed new critical loads.

On Travel Costs, and User and Social Accident Costs

Figure 6 shows the factors influencing user, social, and traffic accident cost elements. The relationships and interactions of the regulation of vehicle weights and dimensions on vehicle design, geometric design, traffic operations, and traffic control and regulation are as discussed previously.

The theoretical traffic capacity of the roadway is compared with the actual traffic demand to determine the facility loading factor; this allows an estimate of travel time. When this factor is combined with the value of time, the travel cost on that facility could be estimated. Congestion should be also determinable.

Factors that influence highway safety are shown in Figure 6.

On Environmental Pollution

As Figure 7 shows, the generation of noise, air pollution, and vibration by highway traffic is influenced by factors related to vehicle characteristics and traffic operations, and, through interactions, to geometric design, and traffic control and regulation. Noise generated by traffic is a function of the vehicle's transmission design, body design, tire tread design, engine type, muffler design, and other noise countermeasures. The mode of engine operation includes whether the vehicle is accelerating, or pulling a grade. The vehicle's speed profile is determined by traffic operations. All are related to noise level, spectrum, and duration. Environment, meteorology, and topography help determine the level of noise at a given location with respect to the source of noise. These factors combine with the ambient noise level of the surroundings, the type of person-activity at that site, and the vehicle mix of the traffic stream to determine the magnitude and nature of the perceived disturbance. When they are combined with a factor translating the cost of noise impact, an estimate of the cost of noise pollution theoretically would be possible.

In the area of air pollution, the factors shown in Figure 7 combine to determine the nature and level of pollution and particulate emissions, and their dispersion and concentra-

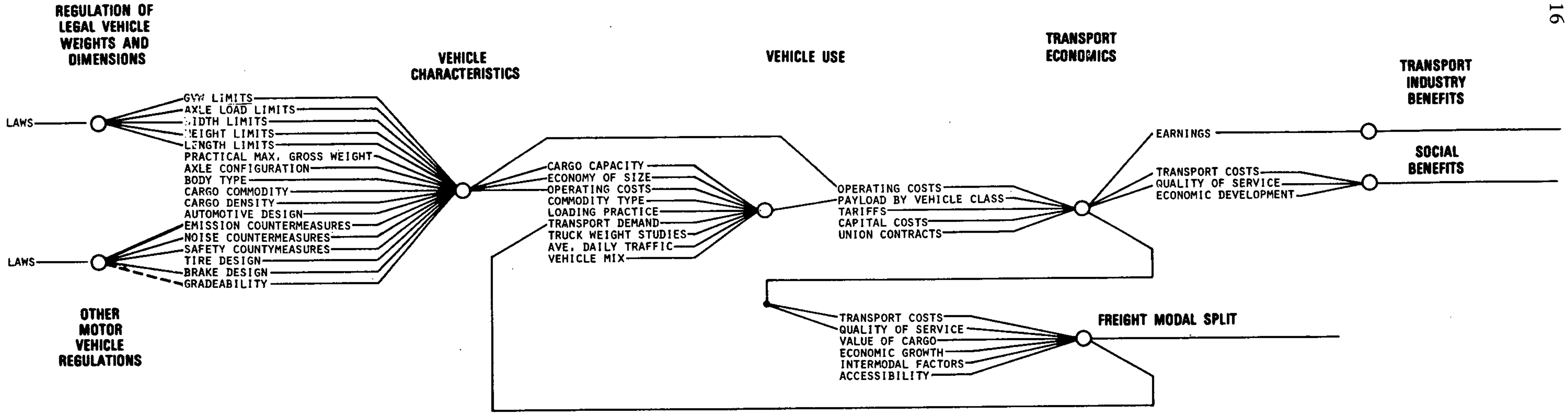


Figure 3. Factorial analysis: Influence of regulation of legal vehicle limits on transport benefits.

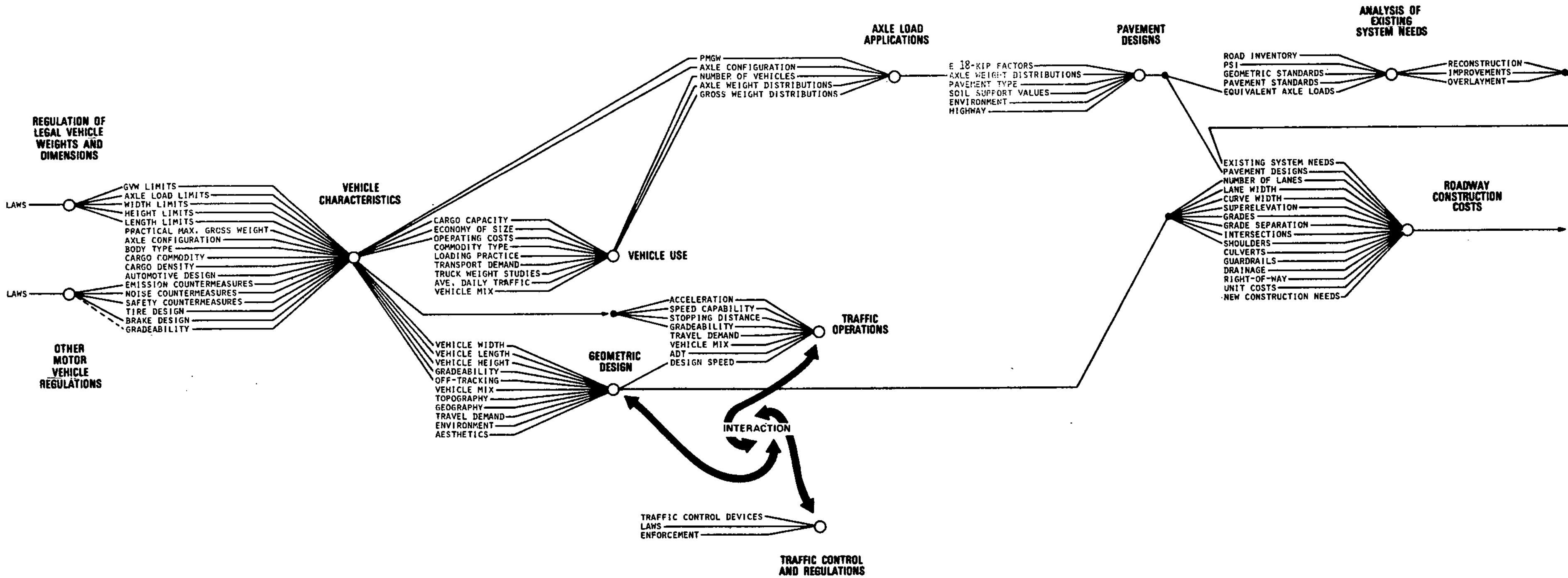


Figure 4. Factorial analysis: Regulation of legal limits and roadway construction costs.

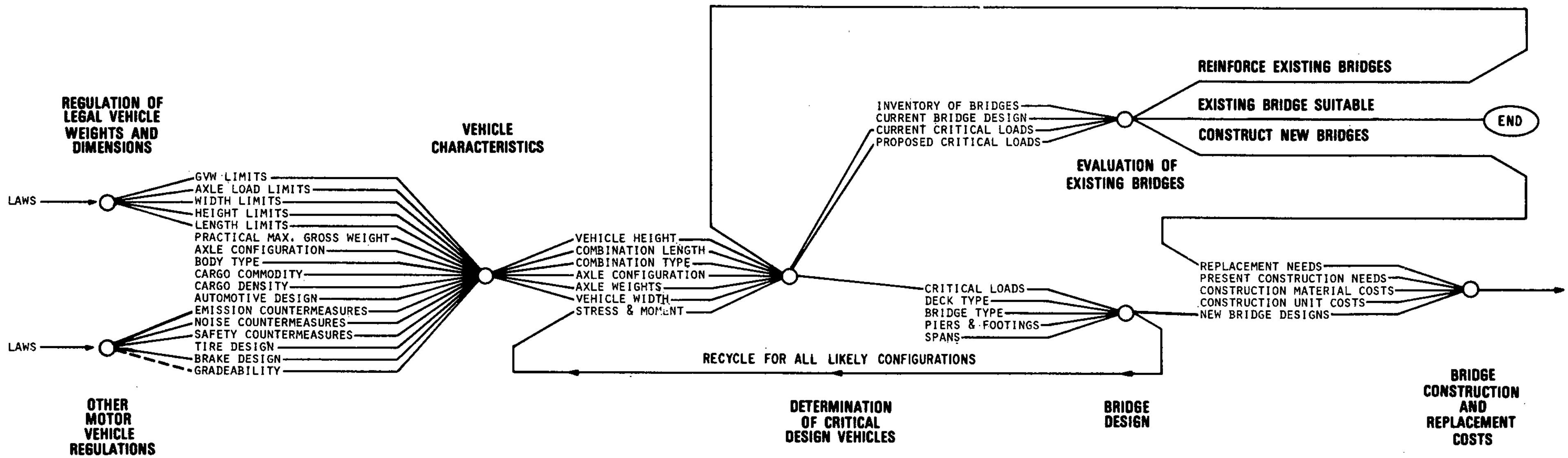


Figure 5. Factorial analysis: Regulation of legal limits and bridge construction and replacement costs.

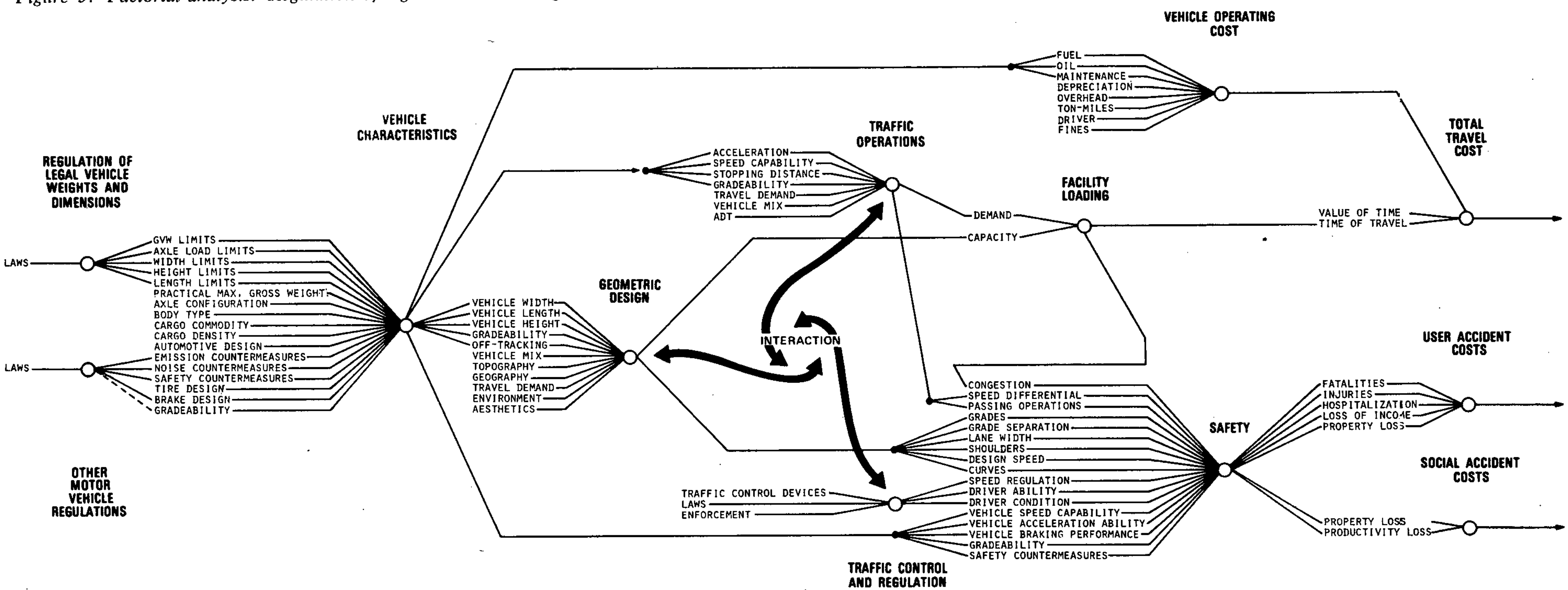


Figure 6. Factorial analysis: Regulation of legal limits and travel and accident costs.

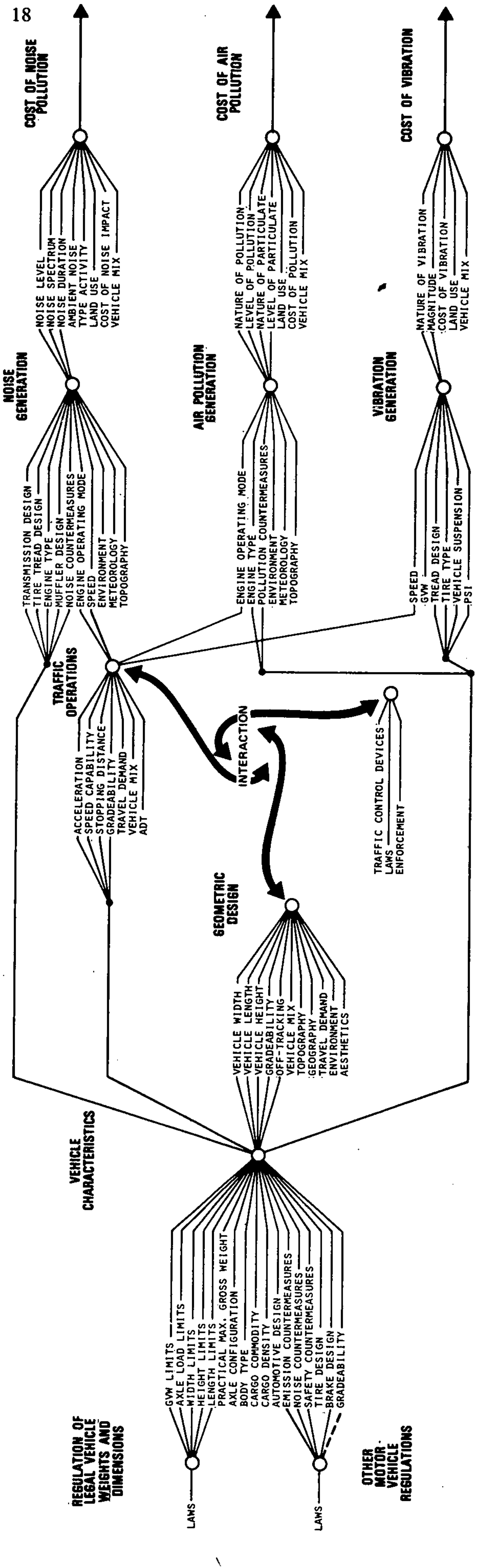


Figure 7. Factorial analysis: Regulation of legal limits and environmental costs.

tion. When these combine with land use, vehicle mix of the traffic stream, and a factor relating the costs of these pollution contributions, the cost of air pollution perhaps could be quantified.

Vibration generation is related to vehicle speed and weight; tire pressure, type and tread design; and the suspension system. The quality of the road surface, represented by the Present Serviceability Index (PSI), also influences the nature and magnitude of vibration. When these results are integrated with the adjacent land use, vehicle mix, and the resulting factor representing the cost of vibration, the total cost would be quantifiable.

The total environmental costs would be termed social costs because they affect both the highway user and the nonuser.

Assembly of Methods Permitting Quantification of the Various Factors of Influence

The next phase of the study was devoted to a literature search to assemble and evaluate existing methods that might permit some form of rational quantification of each element from the contributing factors. Where methods could be adapted to the quantification of costs and benefits, they were discussed and assembled in the report. In certain areas, new procedures and concepts were developed where such development was within the scope of the study and was needed to fill an existing gap in knowledge. Other areas of deficiencies were identified and evaluated for recommended future research efforts.

TRUCK OPERATIONS

Forecasting truck traffic after a change in vehicle size and weight as opposed to present truck traffic is but one problem highway engineers have had to face in past years. A change in highway facilities or legal-regulatory matters has always resulted in change in the character of highway use in the mix of traffic, including volumes and weights. Prediction of the effects of such changes is a constant problem. Transferred and induced travel often occur in much greater quantities than predictors would lead one to anticipate. Although no assured method has been uncovered in the literature to permit high predictive accuracy, a general understanding of the problem and suggested improvements are included to replace mere extrapolation of present traffic trends to future conditions.

Nature of Intercity Traffic

Fundamentally, to improve the accuracy of predictions of intercity truck traffic resulting from a change in regulatory provisions, demand for these movements must be determined or estimated. Under certain situations, the term "demand" can be challenged as not representing the actual value of transport services that may, in fact, be consumed under various cost levels. However, in this analysis "demand" should be interpreted to mean the actual amount of transport resources utilized.

The following is the preliminary groundwork necessary for providing a means of establishing the future demand for intercity goods and passenger movements. It consists of both descriptive and predictive elements. Also included is a brief state-of-the-art review of the formal modeling techniques that are or might be applicable to the measurement of future intercity goods movement demands.

The economic phase of commercial motor vehicle freight operation has several interlocking factors that affect demand:

1. The anticipated growth of highway freight and bus operation.
2. The number of vehicles that can take advantage of any increase in legal limits.
3. Savings to the vehicle operator in more economical operation through volumetric efficiency.
4. Economics resulting in line-haul operating costs.

As Jung (*9*) points out, economics may originate in one part of the shipper-trucker-customer chain, but the benefits accrue to other parts. For example, trucking companies will save on equipment use, fuel, driver time, and other inputs relative to output. But the benefits of these economies will accrue to consumers, either directly or through lower production costs of business firms. More indirectly, businesses will have to keep less inventory on hand with more flexible trucking service. They will be more certain of when shipments will arrive, and their operations can be conducted more smoothly.

Travel by Vehicle Category

The passenger car is the prime generator of miles of travel on all categories of U.S. highways. As Table 4 indicates, in 1969 passenger vehicles (less buses) accounted for 858.9 billion vehicle-miles, or 80.4 percent of total vehicular travel on the U.S. road system. The broad categories of vehicles considered in this study are commercial buses, single-unit trucks, and combination goods vehicles. Travel by these vehicles on the rural (intercity) road system accounted in 1969 for 0.2, 19.5, and 5.4 percent of total travel, respectively. For buses and combination goods vehicles, which are most directly affected by size and weight change considerations, the ratio of commercial bus travel to goods combination travel on the rural road system in vehicle-miles was 1 to 25. Thus, it appears that goods vehicles and their demands are the main subject for study. Bus passenger demands and buses are not to be ignored, but allocated analytical treatment in proportion to their importance to the highway plant.

Existing Intercity Freight Traffic

Over the period 1960 to 1969, the railroad share of total intercity ton-miles generated declined from 44.0 to 41.1 percent. Pipeline traffic increased from 17.4 to 21.7 percent of total traffic. The highway share of ton-miles remained comparatively stable, at between 21.7 and 21.3 percent. Freight movements by airways, while growing rapidly, remained relatively insignificant, accounting in 1969 for less than 0.2 percent of total ton-mile activity (Table 5).

Unfortunately, national transport statistics have certain imperfections, and it is difficult to present information on transportation activity without providing several qualifications regarding data interpretation. The most important of these, insofar as subsequent intermodal series are concerned, relate to the statistical allocation of traffic to pipelines and waterways. Owing to the practice of according to pipelines and waterways the shipments that originate by these modes, there is a tendency to overstate their importance.

Both railroad and motor vehicle activity achieved sustained growth over the period 1960 to 1969. The average annual rate of increase was 3.3 percent in railroad ton-miles, and 3.9 percent for highways.

Table 6 gives revenue data for the transport modes, 1960 to 1969. The revenues include receipts from passenger movements; therefore, a comparison with the modal share of ton-mile allocation is not valid. Data in Table 6 are limited to revenues accruing to carriers subject to regulation. Of course, a substantial volume of transport output is provided by unregulated or private carriers, particularly in the highway sector (*10*).

Railroad (operating) revenues declined in total carrier share, from 47.7 to 34.7 percent. However, airline revenues increased from 9.9 to 18.7 percent of total revenues, and revenues accruing to motor carriers of property increased from 33.7 to 39.2 percent.

The aggregate revenue data reflect but do not explain the complex relationships between transport costs, prices, and modal use. Table 7 shows the wide disparity in transport costs by the various transport modes, as adapted from Meyer (*11*).

Despite various industry regulatory devices that cause degrees of price rigidity, revenues do reflect to a certain extent the "value of service" provided by the various modes. That is, relative to the other transport activity indicators such as "tons hauled" and "ton-miles produced," the revenue measure provides an insight into the important aspect of intermodal service quality.

Recently, both railroad and trucking rates have increased significantly, to the extent of being listed in a 1971 Inflation Alert Statement prepared by the President's Council

TABLE 4

ESTIMATED MILES OF TRAVEL, BY VEHICLE CATEGORY AND ROAD TYPE, 1969

ROAD TYPE	TRAVEL (MILLIONS OF VEHICLE-MILES)				
	ALL PASS. VEHICLES [a]	COMM. BUSES [b]	SINGLE-UNIT TRUCKS	COMB. GOODS VEHICLES	TOTAL
Main rural	295,194	935	74,142	26,514	396,785
Local rural	97,649	193	28,172	1,580	127,594
All rural	392,843	1,128	102,314	28,094	524,379
Urban streets	446,015	1,879	64,927	11,345	544,166
Total	858,858	3,007	167,241	39,439	1,068,545

[a] Includes motorcycles, which accounted for 1.1 percent of total passenger travel.
[b] Excludes travel by school buses and military vehicles.
Source: FHWA, *Highway Statistics—1969*, p. 73.

TABLE 5

VOLUME OF INTERCITY FREIGHT TRAFFIC BY PUBLIC AND PRIVATE TRANSPORT AGENCIES, BY TRANSPORT MODE, 1960–1969

	INTERCITY FREIGHT TRAFFIC, BY TRANSPORT MODE					
YEAR	RAIL	MOTOR VEHICLES	INLAND WATERWAYS	PIPELINES (OIL)	AIRLINES	TOTAL
			(*a*) MILLIONS OF TON-MILES			
1960	579,130	285,483	220,253	228,626	778	1,314,270
1962	599,977	209,407	223,089	237,723	1,289	1,371,485
1964	666,207	356,298	250,165	268,655	1,504	1,542,829
1966	750,762	380,917	265,000	332,916	2,252	1,831,847
1968	756,800	396,300	291,409	391,300	2,900	1,838,700
1969	780,000	404,000	300,000	411,000	3,200	1,898,200
			(*b*) AS PERCENT OF ANNUAL TOTAL			
1960	44.06	21.72	16.76	17.40	0.06	100
1962	43.75	22.56	16.27	17.33	0.09	100
1964	43.18	23.09	16.21	17.41	0.10	100
1966	43.35	21.99	15.30	19.22	0.13	100
1968	41.16	21.55	15.85	21.28	0.16	100
1969	41.10	21.28	15.80	21.65	0.17	100

Source: ICC, *Transport Economics, Monthly Comment* (various eds.).

TABLE 6

OPERATING REVENUES OF CARRIERS SUBJECT TO FEDERAL REGULATION AND PERCENT SHARE, BY TRANSPORT MODE, 1960-1969

		MOTOR CARRIERS						
YEAR AND ITEM	RAIL	OF PASSENGERS	OF PROPERTY	TOTAL	WATERWAYS [a]	PIPELINES (OIL)	AIRLINES [a]	TOTAL
1960:								
Revenue [b]	10,226	667	7,214	7,881	427	770	2,129	21,433
Percent share	47.7	3.1	33.7	36.8	2.0	3.6	9.9	100.0
1964:								
Revenue [b]	10,603	802	9,155	9,957	405	865	3,095	24,925
Percent share	42.6	3.2	36.7	39.9	1.6	3.5	12.4	100.0
1969:								
Revenue [b]	11,955	1,007	13,500	14,507	450	1,103	6,438	34,453
Percent share	34.7	2.9	39.2	42.1	1.3	3.2	18.7	100.0

[a] Domestic.
[b] Millions of dollars.
Source: All modes except airlines, ICC, *Transport Economics, Monthly Comments* (various eds.). Airlines: 1960 and 1964, Federal Aviation Agency, *FAA Statistical Handbook of Aviation, 1966*. 1969, Civil Aeronautics Board, *Air Carrier Financial Statistics, 1969*.

of Economic Advisors (*12*). It appears that since mid-1967 the railroads have been allowed increases in freight rates of 43 percent. Since 1970 alone, rate increases cumulating 19 percent were granted.

Trucking rates generally have risen less than rail rates. Although there are uncertainties owing to the unreporting of unregulated carriers, average truck rates have risen by about 25 percent since 1967, with more than one-half this increase occurring since mid-1970.

With both rail and road, the rate increases were highly selective and the actual rates of increase were somewhat below the over-all values. Nonetheless, it is apparent that transport output as measured by the revenue series has been distorted by severe cost increases since 1967.

Also, it is apparent that road-rail competition is in a state of flux. The extent to which the railroads are applying increased revenues to changing price-quality relationships as a means to winning back traffic from the trucking industry is not clear. It appears, however, that they may be assisted in this objective by the intention of the Administration to give "attention to a number of proposals for stimulating competition by gradual deregulation of the transport industries" (*12*).

A third point, arising from the sheer magnitude of the price increases that will occur in the trucking industry, is the implication for increases in productivity. It is an economic truism that increases in prices should be offset by improvements in productivity; that is, in the use of the factors of production. Increasing the unit carrying capacity of the road transport fleet through weight and size liberalization would, other things being equal, appear to represent an increase in productivity. However, this point emphasizes that the scale or magnitude of the recent rate increases are a cause for concern in the measurement and subsequent attribution of the benefits that this research has summarized.

The effect on other modes of a change in legal limits of vehicles, however, is complex. Because the Federal Highway Administration (FHWA) had initiated a comprehensive research program to study these relationships, and because of the other broad objectives of this study, no further consideration of the modal competition aspects was attempted or required by the project objective. The applicable conclusions of the FHWA study should be made part of the methods used in judging the economic effects of changes in legal vehicle limits, when those methods are made available.

Importance of Truck Transport to the National Economy

The high and increasing dependence of the national economy on truck transportation is convincingly indicated by the statistics of national transportation. Much of the U.S. depends solely on highway transportation to serve its transport needs.

The most definitive information available on the character of intercity trucking movements is provided by the *Census of Transportation* for 1963 and for 1967. The data related only to shipments generated by manufacturing establishments, which account for an estimated 47 percent of intercity goods movements (this value does not include movements by pipeline).

TABLE 7

MINIMUM LONG-RUN MARGINAL COSTS OF FREIGHT TRANSPORT, 1960 [a]

TRANSPORT MODE	MINIMUM MARGINAL COST (1960 MILLS/ REVENUE TON-MILE)
Intercoastal tanker and lake bulk carrier	0.5
Pipeline	1.0
Barge (bulk commodities only)	3.0
Rail carload (bulk commodities)	7.0
Piggyback	9.0
Rail carload (manuf. commodities)	9.0
Truck	25.0
Airline	100.0

[a] The relationships between the costs are not the sole arbiters of shipper choice.
Source: Adapted from Meyer et al. (*11*), supplemented by information provided by the carriers.

There is a marked difference in the proportion of shipments allocated to the various modes according to the type of activity measure that is considered. On a tonnage basis, 40.4 percent of manufacturing freight was shipped by highway in 1967 (Table 8). On the basis of ton-miles, which includes the important distance consideration, the proportion by highway drops substantially to 18.8 percent. The principal ton-mile generating mode is water. Large volumes of high-bulk commodities such as ores, grains, and fuels are shipped by water. A significant omission from the freight distribution is pipeline movements. These were not included in the shipper survey. The importance of pipeline movements was inferred by Table 5, which showed that

TABLE 8

CARRIER SHARES OF TOTAL INTERCITY SHIPMENTS BY MANUFACTURERS, 1963 AND 1967

TRANSPORT MODE	1967 VOLUME		RELATIVE SHARE (%)			
			1963		1967	
	TONS (MILLIONS)	TON-MILES (BILLIONS)	TONS	TON-MILES	TONS	TON-MILES
Motor carrier	369	99	26.0	13.8	26.6	14.6
Private truck	191	29	14.7	4.3	13.8	4.2
Total highway	560	128	40.7	18.1	40.4	18.8
Rail	454	250	33.1	36.5	32.8	36.9
Water	365 [a]	298 [a]	25.6	44.7	26.4	43.9
Other	6	3	0.6	0.7	0.4	0.4
Total	1,385	679	100.0	100.0	100.0	100.0

[a] Of the 365 million tons, 324 million were petroleum and coal products; about 41 million were all other products combined. Of the 298 billion ton-miles, 271 billion were petroleum and coal products; 27 billion ton-miles were all other products. For technical reasons, data on shipments by pipelines were not obtained by the survey and therefore are not included in the total. "Other" includes primarily air cargo, express, United Parcel, parcel post, and some "freight forwarded" that could not be allocated to the operating mode.
Source: Church (*31*, p. 6). Data are from *Census of Transportation*, 1963 and 1967.

(for 1968) 21.3 percent of total goods are shipped by this mode. This value, moreover, was for movements of oil and oil products and did not include natural gas.

Length of Haul

The variation in the respective allocations of shipments to highway largely reflects that road haulage use tends to be highest for short distances and to decline as distance increases. This is indicated by Table 9, which gives the highway share and average length of haul of selected commodities originated by manufacturers in 1963.

Both motor carrier and private trucking show a progressive decline in traffic carried as haul length increases. This inverse relationship tapers at a fairly consistent rate as hauls increase to 1,000 miles. Thereafter, the reduction in the proportion of traffic carried on highways is marked. The railroads, on the other hand, show an expanding traffic involvement as hauls increase, reaching a plateau at between 500 to 600 miles, after which traffic is essentially long-haul, involving distances of 2,000 miles or more. Water movements peak at haul lengths of between 1,000 and 1,500 miles. On many of the routes involving water, a unique situation prevails, because frequently no competing mode is available.

Size of Shipment

From the standpoint of road transport, three broad weight bands are significant. Shipments of less than 50 lb move principally by other means of transport—mostly express and parcel post—although the highway share is large, representing 42 percent of the total shipments in this size class. The second weight band extends from about 50 to 50,000 lb. Highway carriers account for more than 80 percent of the tonnage in all but one weight block in this broad range. The third weight band covers the range of 50,000 to 89,999 lb or more. Over this range, highway participation declines from 59 percent in the 50,000 to 59,999 block to 9 or 10 percent of tonnages in shipments larger than 89,999 lb (Table 10).

Shipment size data at the higher weight level, particularly in excess of 50,000 lb, are incompatible with prevailing truck weight regulations. It is likely that this inconsistency is related to the survey sampling procedure, which included sales invoices. Unless invoices were cross-checked with bills of lading, the shipment as listed probably would reflect the quantity of goods dispatched, and perhaps the mode. On this basis a shipment of 90,000 lb or more allocated to highway would not necessarily infer a unit load of this weight magnitude.

Geographic Area

The highway share varies widely among states and large industrial areas, ranging from 81.1 percent in Massachusetts to 14.4 percent in Texas. Variations in the "commodity mixes" appear to be a major cause of such differences. Shipments by the petroleum group in Texas, for instance, lowered the highway share from 43.8 to 14.4 percent, largely because of water carrier movements from Texas to the North Atlantic coast.

The highway share in Washington is lower than typical for most states, partly because of commodity mix, but also because of the length of haul. The average haul by all means of transport was 856 miles for shipments originating in Washington, as compared with roughly 250 to 350 miles in most other states.

TABLE 9

DISTRIBUTION OF GOODS ORIGINATED BY MANUFACTURERS, BY TRANSPORT MODE AND LENGTH OF HAUL, 1963

LENGTH OF HAUL (STRAIGHT-LINE MI)	TOTAL TONS (MILLIONS)	DISTRIBUTION BY TRANSPORT MODE (%) [a]						
		HIGHWAY						
		TOTAL	MOTOR CAR-RIER	PRIVATE TRUCK	RAIL	AIR	WATER	OTHER
Under 50	215	69.1	32.0	37.1	16.9	—	13.1	0.9
50 to 99	185	63.3	34.6	28.7	21.3	—	15.1	0.3
100 to 199	209	54.3	33.8	20.5	33.6	—	11.7	0.4
200 to 299	148	45.7	32.8	12.9	39.8	—	13.9	0.6
300 to 399	96	37.8	29.4	8.4	44.1	—	17.5	0.6
400 to 499	59	36.6	28.5	8.1	55.9	—	6.3	1.2
500 to 599	47	33.6	27.6	6.0	57.3	0.1	8.2	0.8
600 to 799	80	24.7	20.8	3.9	50.8	0.1	23.5	0.9
800 to 999	59	17.3	15.0	2.3	47.4	0.1	34.2	1.0
1000 to 1199	63	6.8	5.7	1.1	21.7	—	70.9	0.6
1200 to 1499	106	2.9	2.5	0.4	12.2	—	84.8	0.1
1500 to 1999	47	6.7	5.9	0.8	41.8	—	50.6	0.9
2000 or more	21	8.0	7.3	0.7	69.5	0.3	20.5	1.7
All distances	1,335	42.1	25.9	16.2	32.8	—	24.5	0.6

[a] Based on a probability sample of about one million shipping papers drawn from the files of about 10,000 manufacturing establishments, *1963 Census of Transportation*.
Source: Church (*14*, p. 1).

TABLE 10

DISTRIBUTION OF TOTAL MANUFACTURING SHIPMENTS, BY TRANSPORT MODE AND SHIPMENT SIZE, 1963

SIZE OF SHIPMENT (LB) [a]	DISTRIBUTION BY TRANSPORT MODE (%)						
	HIGHWAY						
	TOTAL	MOTOR CARRIER	PRIVATE TRUCK	RAIL	AIR	WATER	OTHER
Under 50	42.1	25.5	16.6	2.0	2.8	0.1	53.0
50 to 99	82.5	57.6	24.9	2.6	1.8	0.1	13.0
100 to 199	89.6	65.5	24.1	3.2	1.1	0.1	6.0
200 to 499	92.8	68.6	24.2	3.0	0.5	0.1	3.6
500 to 999	94.1	69.1	25.0	2.5	0.3	0.2	2.9
1,000 to 1,999	94.3	68.9	25.4	2.6	0.3	0.3	2.5
2,000 to 2,999	90.3	64.6	25.7	6.9	0.1	0.4	2.3
3,000 to 4,999	89.9	64.2	25.7	8.3	0.1	0.5	1.2
5,000 to 9,999	92.8	54.1	38.7	4.9	0.1	0.4	1.8
10,000 to 19,999	88.1	43.8	44.3	10.5	—	0.5	0.9
20,000 to 29,999	81.6	53.0	28.6	17.4	—	0.5	0.5
30,000 to 39,999	84.2	56.2	28.0	14.5	—	0.8	0.5
40,000 to 49,999	78.2	57.5	20.7	20.0	—	1.6	0.2
50,000 to 59,999	53.8	35.0	18.8	44.8	—	1.2	0.2
60,000 to 69,999	13.0	8.9	4.1	84.1	—	2.5	0.4
70,000 to 79,999	10.3	7.7	2.6	88.3	—	1.1	0.3
80,000 to 89,999	8.9	6.6	2.3	90.1	—	0.9	0.1
90,000 and over	10.2	6.2	4.0	60.5	—	28.7	0.6

[a] Classifications by size of shipment were based only on weights shown on bills of lading and sales invoices.
Source: Church (*14*, p. 2).

Manufacturing Plant Size

Small manufacturing plants tend to depend primarily on highway transportation for their shipments to customers or redistribution points. Processing and analysis of the *1963 Census of Transportation* data shows that the degree of reliance on highway transport tends to decline with an increase in plant size (*13*).

Traffic Allocation to Highway by Commodity Class

On a commodity-by-commodity basis, the variations in highway shares are extreme. Church notes that:

> In the extensive list of commodities shown in the Census Commodity Report the highway share ranged from 99.6 percent for ice cream and frozen desserts to about 3.1 percent of the tonnage for coke (*14*).

The highway share is almost invariably largest for distances of less than 200 miles, and declines as the distance increases (Table 11).

An initial step to developing the value of shipment to respective measures of highway involvement relationship was to manipulate the Standard Industrial Classification (SIC) for commodity groups, as used in the *Census of Manufacturers,* to approximate closely the Transportation Commodity Classification (TCC) used in the *Census of Transportation.* These adjustments resulted in some anomalies, in that in a few instances excessive decomposition of the industrial groups occurred.

Examples are the derivation of separate categories for "metal cans and miscellaneous metal products" and "industrial machinery except electrical." Also, the *Census of Transportation* made no direct reference to survey plants involved in the "printing and publishing" and "ordnance" industries. With these exceptions, the dislocation caused by matching the two classification series was not great.

Table 12 gives the value of shipments from manufacturing plants (1958 and 1968) that has been adopted as the measure of industrial activity. Also given is the real-term average annual rate of change in the value of shipments for the 10-year period of industrial category. The relative importance in terms of output of the industrial categories (expressed by the value of shipments) has been weighted by the respective highway ton-mile involvement to provide an indicator of relative highway involvement by industry group.

To ascertain the significance of rate of growth in output, a test was made including as a factor the percentage change in shipments, 1958 to 1968. It was found, however, that the size or amount of output was more significant. Even in those industries that exhibited very rapid growth, such as "ordnance" and "communications products and parts," a "highway involvement" weight was derived for these categories that was significantly below the much larger but slow-growing industries such as the food products group.

Table 13 gives the groups exhibiting the "highest road transport involvement." The "basic textiles and leather products" group is ranked first in terms of tonnage shipped by highway and highway ton-miles generated, and by ton-miles weighted by shipment size. Demonstrating the significance of shipment size, the "fabricated metal products" group is ranked second by "highway involvement" in spite of being seventh in highway ton and ton-mile rank order.

The ten industry groups given in Table 13 account in aggregate for 45.0 percent of total shipments from manu-

TABLE 11

HIGHWAY SHARE OF SELECTED COMMODITIES ORIGINATED BY MANUFACTURERS, BY MILEAGE BLOCK AND COMMODITY, 1963

	PERCENT OF TOTAL TONS ORIGINATED, BY MILES				
COMMODITY GROUP	UNDER 200	200–399	400–599	600–999	1,000 AND OVER
Thread and yarn	99	99	87	96	42
Misc. plastic products	96	94	92	94	63
General indust. machinery and equip.	93	88	87	83	77
Electric transmission and equipment	97	86	88	80	67
Containers, boxes, and related products	92	83	64	50	28
Misc. fabricated metal products	81	80	74	88	52
Glass and glassware	91	72	68	45	14
Beverages and flavoring extracts	89	69	44	18	15
Measuring and controlling instruments	81	92	79	80	39
Soap, detergents, etc.	90	76	63	56	45
Bolts, screws, rivets, washers, etc.	89	51	47	81	32
Nonferrous metal basic shapes	93	88	74	62	27
Plumbing fixtures and heating apparatus	93	94	67	69	21
Electronic components or accessories	65	91	67	55	39
Meat and poultry (fresh or frozen)	81	81	57	52	37
Women's and infants' clothing	76	62	72	64	57
Radio and television receiving sets	94	86	66	47	37
Misc. primary metal products	64	66	67	36	29
Tires and inner tubes	60	72	61	53	31
Plastics materials and plasticizers	83	73	59	37	19
Misc. chemical products	84	52	31	29	22
Steel works and rolling mill products	58	42	31	7	3
Metalworking machinery and equipment	27	30	36	71	53
Household appliances	65	61	29	15	11
Paper (exc. building paper)	67	37	27	19	9
Grain mill products	58	12	7	6	14

Source: Church, (*14*, p. 4).

facturers. Between 1958 and 1968, the over-all rate of increase in shipments for these categories with high levels of demand for highway use was 4.2 percent per annum. This rate is similar to that derived for the total spectrum of manufacturing activities in Table 12.

Future Demand for Transport

Over the long term there is a close, positive relationship between the rate of growth in gross national product (GNP) and the output of total intercity ton-miles of goods movement. This relationship was demonstrated by Kanwit.* The correlation is particularly good between commodity GNP and intercity ton-miles of activity. Figure 8 shows the historic performance patterns in these indicators for the period 1930 to 1960 and the anticipated growth of the three indicators to 1990. Also shown is the historic secular trend in intercity ton-miles by trucks.

Review of GNP Performance

When future performance of the U.S. economy is considered, the time frame or planning period is a long one. For highway planning purposes, time frames of 10 and 20 years are considered.

All forecasts are subject to varying types of uncertainty, and long-term forecasts have to deal with many. Some of these changes are beneficial and arise from the continuing process of social and economic evolvement.

The National Planning Association (NPA) has prepared forecasts for the U.S. economy, recognizing the need for public and private policymakers to address themselves to planning goals (*15*). As prerequisites, the NPA points to the necessity of seeking, as a major objective, high employment combined with sustained economic growth. These principal requirements are supported by associated assumptions relating to defense, world trade, and international liquidity, changes in productivity, monetary inflation, etc.

According to NPA forecasts, real economic growth is anticipated to average 4.4 percent per annum during the 1970's. This rate, together with an expected average annual increase in prices of 2.6 percent, indicates a monetary average annual increase in GNP of 7.0 percent. The NPA paper notes:

> Roughly half the growth will stem from increases in labor and capital, and the remainder from scientific and technological advances, improved management techniques, and other innovations which enhance productivity. The relative contributions of these various factors to growth in the years ahead are not expected to be significantly different than in the post World War II period to date.

Short-term effects may not be discounted over the longer time frame. This is dictated by the increasing influence of

* Unpublished working paper, BPR.

TABLE 12

RELATIONSHIP BETWEEN VALUE OF SHIPMENTS BY PRINCIPAL INDUSTRIAL CATEGORY AND HIGHWAY SHARE OF SHIPMENTS, 1958-1968

Shipper Code (TCC)	INDUSTRY GROUP	VALUE OF SHIPMENTS ($billion) 1958	1968	RELATIVE SIZE OF 1968 SHIPMENTS	HIGHWAY SHARE OF SHIPMENTS Total Tons Shipped	Total Ton-Miles Generated	COLUMN (C) X (E)	RANK ORDER BY HIGHWAY INVOLVEMENT (tons)	(ton-miles)	RANK ORDER, HIGHWAY TON MILES WEIGHTED BY RELATIVE SHIPMENT SIZE	AVERAGE ANNUAL RATE OF GROWTH IN SHIPMENTS 1958-1968 (1) (per cent)
		(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)
01	Meat and Dairy Products	26008	36129	0.604	0.690	0.528	0.319	10	10	5	1.3
02	Canned and Frozen Foods and other food products	26462	39059	0.653	0.409	0.251	0.151	22	19	17	1.9
03	Candy, beverage and tobacco	11146	17872	0.299	0.688	0.406	0.121	11	13	12	2.8
04	Basic Textiles and leather products	16311	27305	0.456	0.896	0.793	0.361	1	1	1	3.2
05	Apparel and related products	13113	22807	0.381	0.814	0.739	0.281	2	2	4	3.6
06	Paper and allied products	12897	22468	0.376	0.428	0.199	0.075	19	21	21	3.6
07	Basic chemicals, plastics materials, synthetic resins, rubber and fibers	11730	23275	0.389	0.358	0.197	0.077	23	22	23	5.0
08	Drugs, paints and other chemical products	11399	23096	0.386	0.538	0.383	0.148	15	16	13	5.2
09	Petroleum and coal products	12935	17768	0.297	0.158	0.025	0.007	24	24	24	1.2
10	Rubber and plastics products	6599	14553	0.243	0.727	0.625	0.152	9	8	9	6.1
11	Lumber and wood products -except furniture	7595	12850	0.215	0.414	0.121	0.026	21	23	25	3.3
12	Furniture, fixtures and miscellaneous manufacturers	9730	17512	0.293	0.766	0.615	0.180	6	6	7	4.0
13	Stone, Clay and glass products	9726	15915	0.266	0.622	0.406	0.180	14	15	15	2.9
14	Primary iron and steel products	18704	31277	0.523	0.429	0.281	0.147	18	18	16	3.2
15	Primary non-ferrous metal products	8103	16789	0.280	0.495	0.306	0.086	17	17	19	6.4
16	Fabricated metal products	17066	33013	0.552	0.744	0.621	0.343	7	7	2	4.7
17	Metal cans and misc. metal products	2164	4370	0.073	0.767	0.615	0.045	5	5	20	5.1
18	Industrial machinery except electrical	3427	6963	0.116	0.768	0.655	0.076	4	4	18	5.2
19	Machinery except industrial and electrical	19370	43410	0.726	0.662	0.526	0.382	12	11	3	6.3
20	Communications Products and parts	4060	12485	0.209	0.732	0.592	0.124	8	9	11	9.7
21	Electrical Products and Supplies	15487	34248	0.572	0.629	0.498	0.285	13	12	6	6.1
22	Motor vehicles and equipment	21472	49738	0.831	0.418	0.227	0.189	20	20	13	6.6
23	Transportation equipment except motor vehicles	16817	30119	0.503	0.537	0.502	0.252	16	14	8	3.9
24	Instruments, photographic equipment, watches and clocks	4418	10705	0.179	0.804	0.656	0.117	3	3	10	7.1
25	Printing and Publishing	12589	23438	0.392	n.a.	n.a.	0.074	n.a.	n.a.	22[2]	4.3
26	Ordnance	3686	11176	0.187	n.a.	n.a.	0.035	n.a.	n.a	24[2]	9.5
	TOTAL	323014	598340		0.404	0.188					4.2

n.a. - not available

(1) These are real term rates of increase. The G.N.P. price deflator was applied to the 1968 value of shipments

(2) The Census of Transportation excluded the printing and publishing and the ordnance industries. The "total industry" ton-mile highway involvement has been applied to derive weights for these sectors.

SOURCE: Value of Shipments; Bureau of Domestic Commerce; Industry Profiles 1958-1968 United States Department of Commerce, November, 1970.
Highway Share of Shipments; Bureau of the Census, 1967 Census of Transportation, Volume III, Part 1, Commodity Transportation Survey.

the price-wage spiral on the spectrum of economic achievement during the last three years. The increase in inflation requires a reassessment of the NPA projections. Using alternative forecasting procedures, the following values were developed to accommodate inflation:

ITEM	GPN AVG. ANN. GROWTH RATE (%)	INFLATION (ANN. INCREASE IN GNP IMPLICIT PRICE DEFLATOR; %)
Judgment model	4.4	2.6
Target model	5.0	2.0

These values are reasonable when it is considered that between 1958 and 1968 the GNP price deflator rose at an average rate of 2.0 percent per annum. In contrast, the rate of inflation since 1968 has increased significantly, averaging since 1958 about 5.0 percent per annum. The Government recognizes that a continuation of such rates of price increase will, if sustained, be both economically and politically unacceptable.

However, a return to the relatively more euphoric price regime of the early and middle 1960's is unlikely. This may be attributed in part to the worldwide inflationary phenomenon, and perhaps more importantly to the need for expanding the home economy with a view to providing adequate employment opportunities. Unfortunately, high employment and price stability are not concurrently consistent objectives.

Recognizing this upward trend in prices, it is proposed to incorporate a rate of inflation equivalent to 3.2 percent per annum. Thus, under the auspices of a 7.0 percent average annual monetary increase in GNP, a real-term increase of 3.8 percent per annum is indicated.

Accordingly, Table 14 compares the respective forecasts of GNP as developed by Kanwit et al. (*16*), and as derived using the modified NPA growth factor. Recognizing the time scale and the uncertainties involved, the two series do not differ greatly. In fact, they show a reassuringly close relationship to each other. Although the BPR study (*16*) did not anticipate the high rate of growth that prevailed through the first half of the 1960's, it did select a rate that closely approximated actual performance during the latter half-decade. Moreover, it anticipated the high rate of monetary inflation that was to become a characteristic of the late 1960's and that continues as a dominant feature of the economy in the 1970's.

As Table 14 indicates, Kanwit et al. anticipated a real-term GNP of $1,549 billion for 1990. Under the modified NPA approach, a level of $1,570 billion is anticipated for the same year. This is only 1.4 percent above the Kanwit value and, given the uncertainties involved, the difference between the two forecasts is within the margin of error to be anticipated.

Macro-Economic Transport-Related Models

The predictive aspect of the various macro-economic transport-related models consists broadly of three interrelated phases or steps:

1. The forecasting of demand for transportation.
2. The prediction of the way in which the transport network will be used ("modal split" aspect).
3. The influence of transport policies.

On a national or regional scale, it is generally considered that the most difficult and elusive of these three phases is the forecasting of demand for transportation.

A number of analytic approaches have been applied to over-all multimodal transportation studies. These developed relatively slowly during the 1960's, particularly in the last five years. The "Harvard" project has dealt with a number of developing nations (*17, 18*); the Northeast Corridor Project involved large-scale regions and urbanized areas, primarily in the interurban context (*19*); the DOT Intercity-Intraurban-Interface ("I^3") project developed methods for dealing with a large-scale urbanized metropolitan area in the context of freight and passenger flows to and from the rest of the U.S. and the "rest-of-the-world" (*20*); and the PENNDOT (Pennsylvania Department of Transportation) System draws heavily on the others in its formulation (*21*). All of these are total systems, in that they:

1. Deal in a "macro" sense with the economic structure of the largest geographic unit they represent (e.g., nation, megalopolis, state, or metropolitan area), and the relation of this economic structure to that of other economic structures with which they are most interrelated (in a trade, resource mobility, and economic "sphere of influence" sense).
2. Permit the disaggregation of the large geographic units into smaller, economically homogeneous units, permitting the interrelationship of the structure of these smaller subeconomies with those of the larger unit.

TABLE 13

SUMMARY OF INDUSTRIAL CATEGORIES WITH HIGHEST HIGHWAY TON-MILE GENERATION CHARACTERISTICS, 1958–1968

SHIPPER CODE AND INDUSTRY GROUP	AS % OF TOTAL VALUE OF 1968 SHIPMENTS	RANK ORDER: BY HIGHWAY INVOLVEMENT: BY TONS	BY TON-MILES	HIGHWAY TON-MILES WEIGHTED BY RELATIVE SHIPMENT SIZE	AVG. ANNUAL GROWTH RATE, 1958–1968 [a]
04 Basic textiles and leather products	4.6	1	1	1	3.2
16 Fabricated metal products	5.5	7	7	2	4.7
19 Machinery except industrial and electrical	7.3	12	11	3	6.3
05 Apparel and related products	3.8	2	2	4	3.6
01 Meat and dairy products	6.0	10	10	5	1.3
21 Electrical products and supplies	5.7	13	12	6	6.1
12 Furniture, fixtures, and misc. manufactures	2.9	6	6	7	4.0
23 Transportation equipment except motor vehicles	5.0	16	14	8	3.9
10 Rubber and plastics products	2.4	9	8	9	6.1
24 Instruments, photographic equipment, watches and clocks	1.8	3	3	10	7.1
Total selected industries	45.0				4.2

[a] Real-term growth rates.

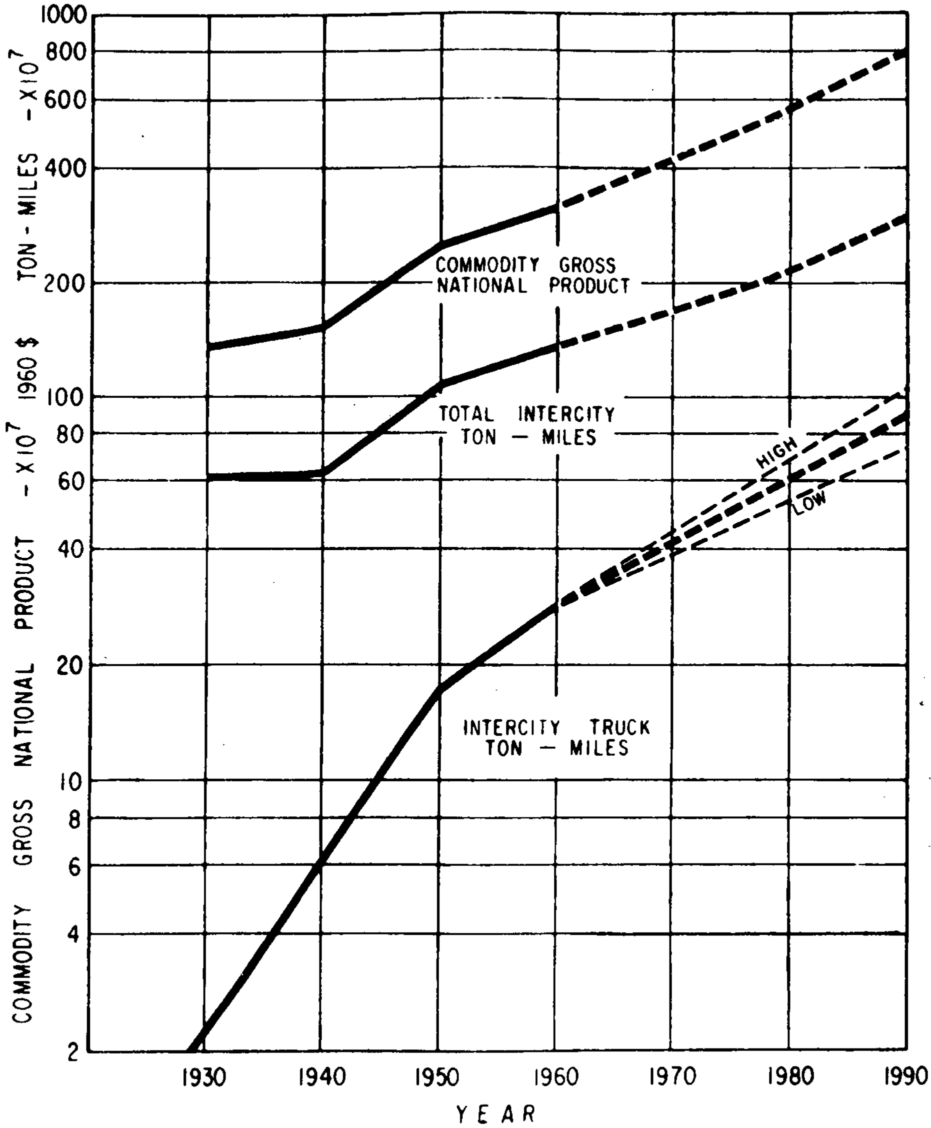


Figure 8. Commodity GNP, total intercity ton-miles, and intercity truck ton-miles, 1929–1990.

3. Allow the variation, through insertion of alternative site or "policy variable," to test for the implications of alternative policy variables; and also permit the evaluation of the sensitivity of the results of variations in the underlying parameters of the system.

4. Permit the "feedback" of the effects of the transportation system on the economic development of the sub-economies.

5. Perform the following calculations in the transportation system:
   - (a) Generate the traffic flow (in volume, by type of commodity) originating at transportation "nodes."
   - (b) Distribute the traffic flow (in volume, by type of commodity) to terminating transportation "nodes."
   - (c) Divide the traffic flow between partially competing, partially complementary modes (i.e., the "modal share" computation).
   - (d) Assign the traffic flow over various routes of the network.
   - (e) (Optional) Select one or more carriers (where there are competitive carriers for a given mode and route assignment).

6. Produce estimates of the following freight traffic flows, each one related to a calculation in Item 5:
   - (a) *Diverted traffic*—that component of existing traffic that has changed from its previous path of travel to another route without a change in origin, destination, or mode of transport.
   - (b) *Converted traffic*—that component of existing traffic that, for either part or all of its previous path of travel, has changed its mode of transport without a change in origin or destination.
   - (c) *Shifted traffic*—that component of existing traffic made up of journeys whose "desire lines" have

TABLE 14

FORECAST OF GROSS NATIONAL PRODUCT IN 1960 PRICES

	BPR		NPA-MODIFIED FORECAST [a]	
YEAR	($ BILLIONS)	INDEX (1960 =100)	($ BILLIONS)	INDEX (1960 =100)
1950	363.9	72.4	363.9	72.4
1955	449.2	89.4	449.2	89.4
1960	502.6	100.0	502.6	100.0
1962	540.8	107.6	548.0	102.4
	Forecast:			
1965	599.6	119.3	636.0	107.4
1970	715.6	142.4	756.2	130.9
			Forecast:	
1975	860.3	171.2	910.0	
1980	1046.7	208.3	1095.0	
1985	1273.4	253.4	1320.0	
1990	1549.3	308.2	1570.0	

AVERAGE ANNUAL COMPOUND RATE OF INCREASE (%)

BPR		NPA [a]	
1950–1955	4.1 [b]	1950–1955	4.1 [b]
1955–1960	2.3	1955–1960	2.3
1960–1965	3.6	1960–1965	4.8
1965–1975	3.7	1965–1970	3.6
1975–1990	4.0	1970–1990	3.8

[a] Forecast rate of average annual increase modified by the researchers to accommodate a higher anticipated rate of inflation (see text).
[b] Rate influenced by the Korean War.
Source:
Bureau of Public Roads: Kanwit, et al. (*16*).
National Planning Association (modified forecast): Al-Samarrie, et al. (*15*).

shifted due to a change in origin and/or destination (e.g., relocation of a producer and/or buyer).

(d) *Induced traffic*—that component of added traffic that did not previously exist in any form, but which results in unique response to the provision of improved transportation facilities.

(e) *Natural traffic*—that component of added traffic which did not previously exist in any form, but which results merely from the "natural" growth in traffic-generating activity independent of network modification.

7. Permit the evaluation of the results by use and application of cost and benefit estimates to users of the system and, indirectly, to nonusers who are affected by the system.

These relatively complete systems are not the only ones that merit attention in review of such systems; others exist, but these are representative of the complexity, cost, and potential power of such systems.

Some Component Subsystems

Some of the more pertinent questions concerning each of these systems and their components are:

1. *Forecasting Commodity Demand.* The future spatial pattern of economic activities frequently is considered a major determinant of commodity (freight) flow patterns. Future production and consumption, as well as the marketing and distribution of each commodity grouping, in each relevant economic subarea, and between economic subareas, is typically the foundation for the projecting of future commmodity flows. Some studies rely heavily on standard economic analysis projection techniques—trends, market analysis, and knowledge of industry structure. Others consider, also, "desire lines" and the available modes of transport in their demand estimating procedure.

More analytical, reproducible methods of forecasting future production, consumption, and trade, particularly where national trade and exchange policies prevail, rely heavily on economic models. Examples of such models are:

(a) *Harvard Macro-Economic Model* (*22*). Under sponsorship of the International Bank for Reconstruction and Development, this model was developed for the Republic of Colombia. Subsequent applications have been limited. The lack of more widespread use is thought to stem from budget and data constraints, not from the ineffectiveness of the procedures.

The Harvard model is used to stimulate the functionings of an entire economy. Its basis is a national input-output table that can be used to determine the amount of each constituent input required to produce a single unit of output of each commodity. The input-output table thus incorporates by means of its technological coefficients the interindustry transactions that will be generated by any given set of demands for final products (*23*).

The model is an econometric computer model. In addition to the input-output table, it consists of a series of equations expressing the interrelationships between the environmental variables that adjust to fit various economies, the program parameters by which the model is calibrated, and the controllable variables by means of which the planner can influence over-all growth.

A modification of the Harvard model has been developed to consider national policy parameters affecting international commerce (*24, 25*).

(b) *Modification of Interindustry/Econometric Approach to Incorporate Freight Traffic Zones* (*19*). The traditional input-output model assumes constant technological coefficients; that is, fixed proportions in the ratios of dollar expenditures for the different inputs of each industry. In the Northeast Corridor and PENNDOT adaptations, underlying production functions were estimated and used to forecast changes in the proportions of physical quantities of inputs, and hence freight movements, that would result from change in prices and changes in the cost of transportation.

These models required that the entire geographic area and its periphery be subdivided into a set of freight traffic zones or nodes which become the origins and destinations of all freight movements. The model describes the flows of goods between the nodes of the geographic area and its periphery. Thus, the more numerous the zones identified, the more complete will be the description of freight flows within the area. However, certain practical considerations limit the number of nodes that should be considered. For certain purposes it may be desirable to use the model to analyze traffic flows among as few as four regions of the entire area. For other purposes, a more detailed analysis is required.

(c) *Simplified Freight-Forecasting Procedure* (*20*). Because many of the data required for input-output models, or their econometric adaptations (which require time series of the same data), are difficult to obtain, considerably simpler computational procedures have been developed that use the basic interindustry and interactance concepts of these models. One such model (called the "I^3 Demand Model") is used by DOT to provide estimates of goods and passenger traffic flows to and from a study area.

From base-year estimates of such intercity flows, together with internal, intraurban goods flows, the model projects these demands on a transportation system for selected future years. Because typically it is possible to develop estimates for only one point in time in the past, the model does not necessarily use any type of extrapolation of past trends. The approximate nature of the base-year traffic flow estimates also typically rules out all but the most simple relationships with basic socioeconomic measures, such as population and employment. A model assumption is that these relationships are constant throughout the study area and will remain so over time. These rudimentary measures of the demand placed on the study area's transportation network provide a point of departure for more extensive research to identify the more complex causal relationships which do, in fact, give rise to these phenomena as "demand" in the sense of economic theory. The model structure is sufficiently flexible to permit the introduction of calibrations reflecting these complex relationships whenever it becomes possible to determine them. In the meantime, the model produces internally consistent preliminary estimates of traffic flow demands likely to be placed on the area's transportation network. The order of magnitude for the several flows predicted by the model is reasonable to the extent that it permits a general evaluation of operations within the area's transportation system. The I^3 Demand Model determines the relationships that existed in a predetermined base year between selected socioeconomic variables for specified originating and terminating areas, and the traffic flows of goods and persons exchanged by the "centroids" of study area subdivisions and the "centers" of representative rest-of-the-world areas. It applies the ratios expressing these relationships to projections of the socioeconomic variables that have been exogenously made for at least one of the future years under study, in order to produce projections of the traffic flows likely to occur under the assumptions implicit in the model.

2. *Interzonal Flow Models.* For estimating freight traffic patterns, "who sells to whom" must be calculated. One of the analytic systems (e.g., the I^3 Demand Model) uses the simplest approach to interzonal flow estimation, which some call a "trade model." Here, the assumption is made that the zonal distribution of a good is some fixed proportion of the total amount of the commodity consumed or produced in a region.

The gravity model also is commonly used to estimate interzonal commodity flows. In it, interzonal flows are related proportionately to the total production and consumption in two regions and are assumed to be inversely proportional to the total. A gravity "friction" parameter is estimated from an interzonal trade matrix and by some function of transport time and/or costs. The Northeast Corridor interzonal flow model is an adaptation of this concept (*26*). Still another type of interzonal flow model is the linear programming model which allocates shipments between regions to meet a given regional distribution of demand. A number of simplifying assumptions must be made to use the programming formulation, making it of doubtful utility at this time.

An interesting competitive concept to the gravity concept is that of "intervening opportunities." The models are similar in the sense that they both pose some relationship between the proportion of one zone's originating traffic which is distributed to some other zone acting as a destination, and some operational definitions of that other zone's attraction force and of the resistance offered by some characterization of intervening space. The integration of these two concepts accounts for the attenuation of interaction over space. For example, the gravity model asserts a direct proportionality to the former and an inverse proportionality to the latter. "Intervening opportunities" poses a more complex attenuation function.

3. *Predicting Share of Total Transport for Each Mode ("Modal Split")* (*21, 27*). Urban transport studies have traditionally circumvented the problem of ascertaining freight modal split by starting their various trip generation analyses at the level of goods vehicles, and specifically with trucks—not with the loads.

On the interurban or regional scale, several freight modal split procedures do exist. Their respective effectiveness depends on many factors. In general, however, a freight modal split should accept as input the volume of freight that will travel between specified origin-destination pairs, segregated according to some commodity classification scheme. The output of the model should be the amount of freight that will be shipped by each mode between specified origin-destination pairs. Further, the model must be transport-sensitive; i.e., the modal split must be affected by changes in transport characteristics. Also, modal split is subject to manipulation through policy control. As Plowden (*28*) notes, "modal split must be treated as a planning matter, not merely as a predictive matter."

Roberts (*29*) has done work on modal coordination and transportation costs. Although his major concern was the economic value (to companies providing transportation services) of changing rates and providing multimodal service, the expected use of any new service created was included in the economic evaluation. The market value of the service was described by the shipping rate charged plus a measure of service qualities. If the sum of these variables was lower than that for existing modes, the new service would attract traffic. The service quality includes such things as transit time, reliability of service, and costs associated with the distribution system that are not considered in the rate. The freight modal split model must consider factors that influence modal choice. Therefore, it is important to define the factors that are included in "service quality" quantitatively so that a model for predicting modal split can be developed.

Perle (*30*) also has analyzed the influence of price on the demand for freight transportation. His technique involved an examination of the relationships existing between the consumption of freight services and the price system. The work was confined to motor carriers and railroads. Service quality characteristics were not taken into account. Simple demand models were constructed based on price elasticity relationships recognizing the substitution potentials between highway and railroad use. Perle achieved a high degree of success in explaining global substitution effects. Likewise, he pointed to markedly differing transport demand responses by census region. Statistical difficulties precluded a definitive or conclusive description of substitution effects at a level of analysis smaller than the census region.

It has been suggested that the determination of a dollar value for time would provide a method of evaluating alternative transportation modes. If the value of time is known, time cost and dollar cost can be added to provide one variable.

Church (*31*) shows that modal split between highway and rail can be predicted if one knows the commodity type, the size of shipment, and the distance between origin and destination. The results obtained, using Church's method, are shown to be accurate to within 5 percent as applied to the examples used in the report.

The Northeast Corridor Transportation Project presents three separate models for predicting the demand for transportation by freight shippers. The task assigned to these models is to generate, distribute, and predict the modal split for freight movements. They are relevant to this discussion in that modal split was included as part of the total task; however, it is necessary to delineate the portion of each model that deals with modal split.

One of the Northeast Corridor models is referred to as "the macro-economic approach" and is a system of simultaneous equations that is calibrated using data arranged in various degrees of aggregation. The first equation uses socioeconomic variables to predict the total demand for transportation in the study area during a given time period. The last equation uses results obtained from the previous equations to predict the flow of goods from specified origins to specified destinations by mode.

The Northeast Corridor models handle the generation, distribution, and modal split tasks as one task. Although this approach seems justified in handling passenger demands, the aggregation of these tasks for freight movements appears artificial. A person's decision to make a trip depends, in part, on the perceived cost and time required to make the trip, whereas the generation of goods to be shipped depends on the market value of the goods. Once the decision to produce has been made, the decision to ship has been made. Modal choice enters as a separate decision, connected with the decision to ship only as transportation costs affect prices of products or the associated profits. Therefore, it seems more realistic to handle modal split as a separate task in the procedure of determining the freight traffic volumes that will flow on the transportation network.

FORECAST OF DEMAND FOR INTERCITY MOTOR FREIGHT MOVEMENTS WITHIN A STATE

For this analysis, the economic impact of changes in vehicle sizes and weights must be measured and evaluated at the level of the individual state. The adoption of the state as the basic analysis unit complicates the use of such global parameters as changes in GNP and industrial production that served as a basis for developing estimates of future intercity heavy-vehicle ton-mile output at the national level (*16*).

The difficulties inherent in working at a subarea level relate primarily to the high degree of decomposition or disaggregation involved. The 50 states have widely disparate levels of population and economic output. In addition, the states vary in geographic area and resources and, through a combination of these and other factors, they vary in wealth.

A dominant feature in the U.S. in recent years has been the growth of economic regionalism. Industry has become increasingly "footloose." This locational dispersion has largely been caused by two transportation developments: (1) the cost of transporting goods over long distances is tending to rise relative to the cost of producing them; and (2) the development of commercial air travel has provided efficient and, in terms of executive time, cheap passenger transport. These two factors have combined with the increased mobility of capital, labor, and technology to support and complement the regional dispersion of the population to new urban centers. The purchasing power of the population also has increased, providing regional market strength. As a result, it has been possible to set up regional branches of manufacturing and other businesses at an economically advantageous scale.

The tendencies toward decentralization are presently leading to a relatively high rate of growth of a number of regional metropolitan centers with populations of about one million persons, and the relative decline of the rate of growth of some (though not all) of the greater metropolitan centers, and also of a very large number of smaller centers.

Clark (*32*) notes the following pattern of industrial development:

> Until comparatively recently, transport and other considerations kept most new manufacturing development within a broad axis extending approximately from Chicago to Connecticut. During the 1940's the industrial center in California became firmly established. Since then, many regions in the West, and some in the South, have become centers of growth for industries capable of selling in national and international, not merely local markets.

It is against this background of regional dispersion that the administrative boundaries and economic strengths of the states should be recognized.

Surrogate Measures for GNP

A practical limitation imposed by the global indicators of GNP and industrial production is that statistical descriptions of these aggregates are not available at the subarea level. However, an acceptable substitute measure of economic activity is provided by personal income received by individuals and corporations. This indicator is available for states, metropolitan areas, and counties. Personal income (Table 15) comprises approximately 80 percent of the GNP. This proportion has remained fairly stable over the years.

Table 16 compares personal income, population, rural highway vehicle-miles generated, and "special fuels" consumed by the 50 states and the District of Columbia. On the premise that intercity heavy-vehicle ton-mile generation is closely related to the level of prevailing economic activity, the states have been ranked according to personal income received.

Immediately apparent is the disparity in distribution of wealth among the states. The 11 "industrial" states received 62.0 percent of total personal income. Conversely, the 11 lowest ranking states received only 3.0 percent.

Population shows a similar though slightly more even distribution than personal income. This reflects differences in per capita income among states.

The volume of vehicle-miles generated on the rural highway system has been selected as an indicator because, to a large extent, this is the best available measure of intercity highway use. Also given are vehicle-miles of travel generated on the rural Interstate System. It is reasoned that, because heavy vehicles tend to predominate in the generation of "over-the-road" vehicle-miles on the rural Interstate System, interstate vehicle-miles provide a reasonable indicator of intercity movements by state of "weight- and size-affected" vehicles. This assumption is tested further later herein.

It is notable that the distribution of vehicle-miles on both the total rural and interstate rural systems is more uniform throughout the array of states. Although the "industrial" states accounted for 62.0 percent of total personal income, these states generated 45.3 percent and 46.3 percent, respectively, of total rural and interstate rural vehicle-miles.

Of course, the volume of highway vehicle-miles generated in a state depends on several important factors, some of the most significant being the state's geographic size, and the extent, nature, and location of economic activity generators (including urban places) both within the state and in relation to other states. The relevance of a state's highway system as serving major corridor movements cannot be overemphasized. Although a particular state may have relatively modest economic activity centers within its boundaries, it may nonetheless serve heavy interstate movements on certain segments or corridors of its highway system. The impact of geographic size on the volume of vehicle-miles generated by the "industrialized" states is exemplified by New Jersey. This state, ranked eighth in terms of personal income share, was thirty-eighth in terms of rural vehicle-miles generated. This last was consistent with the fact that the state comprises only 0.4 percent of the U.S. land area or, expressed another way, ranks forty-sixth among all states. This is an isolated and extreme example, but Table 16 indicates that similar conditions exist for several other states that are of disproportionate size.

The proportionate distribution of rural highway vehicle-miles among states is markedly more even than might be expected from the allocation of personal income, and even population. This is particularly the case in the large "middle band" of states. Two hypotheses serve to explain the greater ubiquity of vehicle-mile generation: (1) demand for rural highway system movements is generally inelastic; and (2) as will be demonstrated, many states serve interstate corridor transport demands.

Estimates of future interstate travel demands were allocated to major highway corridors in a PCA study (*33*). The forecasts were essentially indicative, developed to identify the future corridors of intercity movement by highway for the year 2000. The study was designed to include all major transport axes of North America.

Figure 9 shows the "minimum mileage" network tested under the PCA study. Also shown are the principal "industrial" states as defined by personal income received, as given in Table 16. Recognizing continued growth in urban place population, it is estimated that the network shown in Figure 9 would directly serve approximately 80 percent of the U.S. population by the horizon year 2000. Significant to this discussion is the dispersed, reticulated nature of the highway network, with mileage concentration in the industrial states but with important interconnecting links that also serve intermediate states and cities. The result is a

TABLE 15

RELATION OF GNP AND PERSONAL INCOME, 1950–1971

ITEM	1950	1955	1960	1965	1969	1970	1971 [a]
GNP ($ billions)	284.6	397.5	504.5	676.3	931.4	976.5	1,020.7
Personal income ($ billions)	228.5	310.2	402.2	530.7	748.9	801.0	831.5
Personal income as percent of GNP	80.3	78.0	79.7	78.5	80.4	82.0	81.5

[a] First quarter, preliminary.
Source: Department of Commerce, *Survey of Current Business* (various eds.).

TABLE 16

COMPARISON OF DISTRIBUTION BY STATE OF PERSONAL INCOME, POPULATION, RURAL HIGHWAY VEHICLE-MILES, AND CONSUMPTION OF SPECIAL FUELS FOR HIGHWAY PURPOSES

	PERSONAL INCOME (1969)		POPULATION (1970)		RURAL SYSTEM VEHICLE-MILES (1969)				SPECIAL FUELS (1969) [a]	
					INTERSTATE		ALL SYSTEMS			
STATE, BY RANK	$ BILLIONS	% OF TOTAL	THOUSANDS	% OF TOTAL	MILLIONS	% OF TOTAL	MILLIONS	% OF TOTAL	MILLIONS OF GAL	% OF TOTAL
1. California	83.1	11.2	19953	9.8	8883	8.9	39026	7.4	577	9.1
2. New York	81.0	10.9	18191	9.0	3137	3.1	23959	4.6	216	3.4
3. Illinois	47.6	6.4	11114	5.5	4148	4.1	20211	3.8	320	5.1
4. Pennsylvania	43.2	5.8	11794	5.8	5548	5.6	26806	5.1	394	6.2
5. Ohio	40.6	5.6	10652	5.2	4960	5.0	25003	4.8	399	6.3
6. Texas	36.4	4.9	11197	5.5	5557	5.6	30554	5.8	417	6.6
7. Michigan	34.6	4.7	8875	4.4	3732	3.7	23768	4.5	188	2.9
8. New Jersey	30.6	4.1	7168	3.5	677	0.7	11160	2.1	220	3.5
9. Massachusetts	22.6	3.0	5689	2.8	1572	1.6	7266	1.4	99	1.6
10. Florida	21.8	2.9	6789	3.3	3195	3.2	16834	3.2	170	2.7
11. Indiana	18.9	2.5	5194	2.6	3842	3.8	18698	3.6	232	3.7
12. Missouri	16.1	2.2	4677	2.3	3180	3.9	13653	2.6	183	2.9
13. Wisconsin	15.4	2.1	4418	2.2	1894	1.9	12601	2.4	114	1.8
13. Maryland	15.4	2.1	3922	1.9	1196	1.2	9908	1.9	82	1.3
13. Virginia	15.4	2.1	4648	2.3	4030	4.0	16986	3.2	182	3.0
16. North Carolina	15.0	2.0	5082	2.5	2849	2.8	19006	3.6	167	2.6
17. Georgia	14.1	1.9	4590	2.3	3538	3.5	16143	3.1	209	3.3
18. Connecticut	13.6	1.8	3032	1.5	856	0.9	3498	0.7	77	1.2
19. Minnesota	13.4	1.8	3805	1.9	1052	1.1	12000	2.3	115	1.8
20. Washington	13.0	1.7	3409	1.7	1853	1.9	9910	1.9	93	1.5
21. Tennessee	11.2	1.5	3924	1.9	2389	2.4	10866	2.1	147	2.3
22. Louisiana	10.4	1.4	3643	1.8	1879	1.9	10076	1.9	92	1.4
23. Iowa	9.8	1.3	2825	1.4	1834	1.8	10217	1.9	118	1.9
24. Kentucky	9.2	1.2	3219	1.6	2203	2.2	12874	2.4	102	1.6
25. Alabama	9.1	1.2	3444	1.7	1830	1.8	9730	1.8	120	1.9
26. Kansas	8.2	1.1	2249	1.1	1202	1.2	7977	1.5	89	1.4
27. Oklahoma	7.9	1.1	2559	1.2	1635	1.6	9476	1.8	110	1.7
28. Colorado	7.5	1.0	2207	1.1	1634	1.6	6708	1.3	66	1.0
29. Oregon	7.2	1.0	2091	1.0	1577	1.6	7391	1.4	106	1.7
30. South Carolina	6.9	0.9	2591	1.3	1840	1.8	11089	2.1	84	1.3
31. Arizona	5.6	0.7	1771	0.9	2601	2.6	6005	1.1	98	1.6
32. Nebraska	5.3	0.7	1484	0.7	1087	1.1	6456	1.2	64	1.0
33. Mississippi	5.2	0.7	2217	1.1	1225	1.2	7979	1.5	106	1.7
34. Arkansas	5.0	0.7	1923	0.9	1205	1.2	7439	1.4	88	1.4
35. West Virginia	4.7	0.6	1744	0.9	983	1.0	6208	1.2	64	1.0
36. D.C.	3.9	0.5	757	0.4	—	—	—	—	14	0.2
37. Rhode Island	3.4	0.5	947	0.5	236	0.2	846	0.2	18	0.3
38. Utah	3.1	0.4	1059	0.5	1167	1.2	3133	0.6	48	0.8
39. Maine	3.0	0.4	992	0.5	621	0.6	4264	0.8	24	0.4
40. Hawaii	3.0	0.4	769	0.4	175	0.2	1169	0.2	8	0.2
41. New Mexico	2.9	0.4	1016	0.5	1573	1.6	4466	0.8	73	1.2
42. New Hampshire	2.5	0.3	738	0.4	541	0.5	2748	0.5	11	0.2
43. Delaware	2.2	0.3	548	0.3	61	— [b]	1487	0.3	12	0.2
43. Montana	2.2	0.3	694	0.3	828	0.8	3538	0.7	50	0.8
45. Idaho	2.1	0.3	713	0.4	820	0.8	3410	0.6	28	0.4
46. Nevada	2.0	0.3	489	0.2	682	0.7	1811	0.3	35	0.6
47. South Dakota	2.0	0.3	665	0.3	742	0.7	3624	0.7	29	0.5
48. North Dakota	1.9	0.3	618	0.3	492	0.5	2971	0.6	29	0.5
49. Vermont	1.4	0.2	444	0.2	452	0.5	2052	0.4	6	0.1
50. Alaska	1.3	0.2	302	0.2	—	—	662	0.1	5	0.1
51. Wyoming	1.1	0.1	332	0.2	848	0.8	2366	0.5	4	0.1
Total	743.0	100.0	203173	100.0	100061	100.0	526028	100.0	6,331	100.0

Note: Columns may not add to totals owing to rounding.

[a] Motor fuels other than gasoline that are used on the highway, and consist primarily of diesel fuel and liquefied petroleum gases (see text).

[b] Less than 0.1 percent.

Source: Personal income, Department of Commerce, *Survey of Current Business* (various eds.); rural system vehicle-miles, U.S. DOT, *Highway Statistics, 1969;* population, Bureau of the Census, preliminary data.

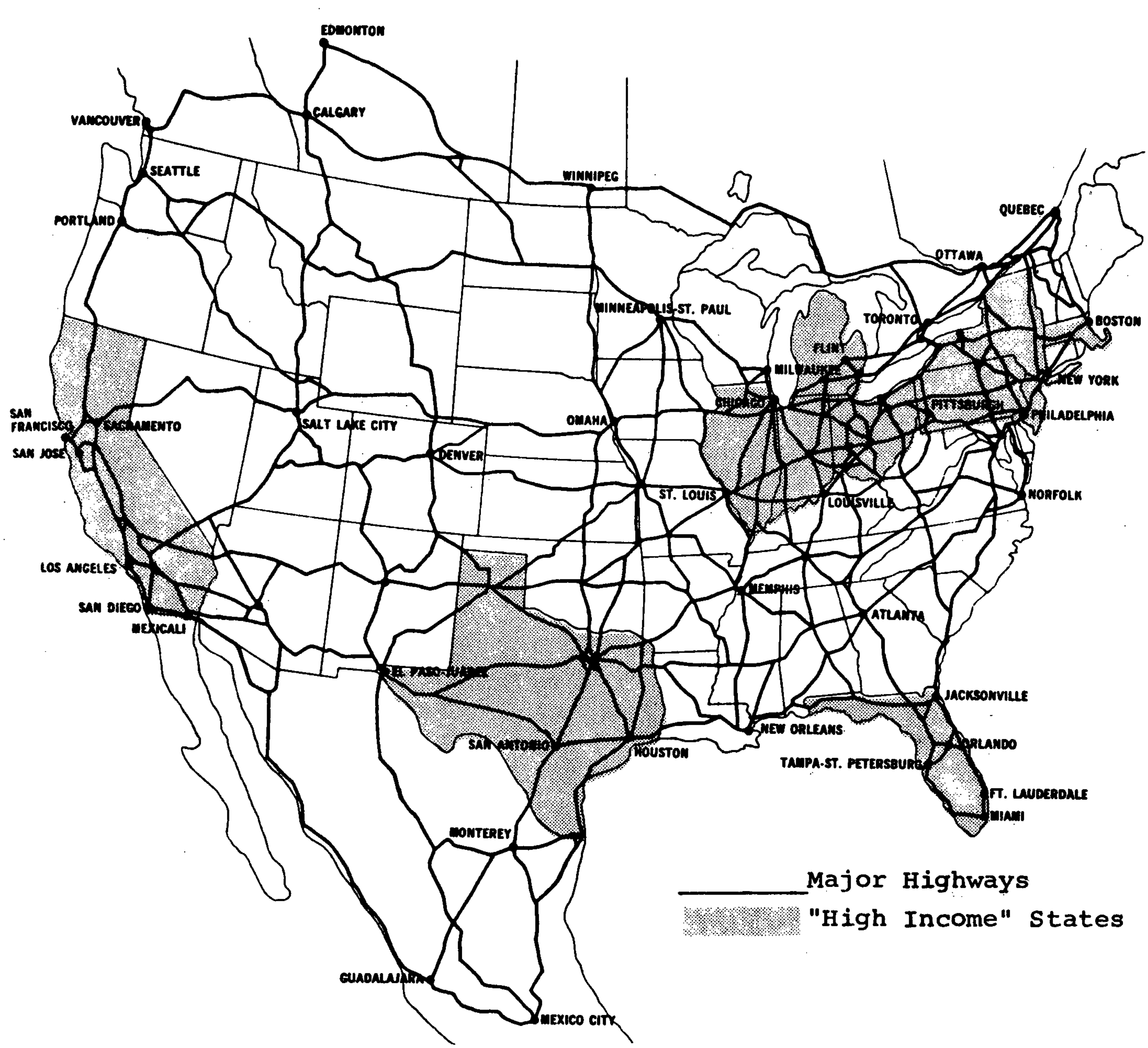


Figure 9. High-income states in relation to the principal highway network, U.S. Source: PCA (33) *and Dept. of Commerce.*

multicenter network with three principal and dispersed points of industrial concentration.

The industrial agglomerations shown in Figure 9 have service areas in the neighboring states that, in accordance with central place theory, become attenuated with distance. An additional feature is intercomplex movement of materials and goods. It is estimated that in addition to the 11 industrial states, as many as 22 intermediately located states are involved in intra- and intercomplex movements over major corridor axes. It is believed that the service region function combined with the corridor role explains the noticeably more even distribution of rural road system and particularly rural Interstate System vehicle-mile output that occurs over the array of states given in Table 16. To this must be added the dispersion of economic activity per se.

Using GNP and Personal Income Alone to Project State Ton-Miles

Most important is the identification of a performance measure that will adequately reflect the dynamics of change in highway ton-mile output over the planning period. A close relationship exists between real-term changes in GNP and freight ton-mile output. The consistent relationship between GNP and personal income received is shown in Table 15. Distribution of personal income among the

TABLE 17

PROPORTIONATE INVOLVEMENT OF THE INDUSTRIAL STATES IN SELECTED INDICATORS, 1970 (% OF TOTAL U.S.)

STATE	PERSONAL INCOME	POP.	RURAL SYSTEM VEHICLE-MILES: INTERSTATE	RURAL SYSTEM VEHICLE-MILES: ALL SYSTEMS	TRUCKS REGISTERED	HIGHWAY USE OF SPECIAL FUELS[a]	LAND AREA[b]	VALUE ADDED BY MANUFACTURES	COMPOSITE ACTIVITY INDICATORS[c]
California	11.2	9.8	8.9	7.4	11.2	9.1	5.2	8.9	10.3
New York	10.9	9.0	3.1	4.6	3.6	3.4	1.6	9.7	10.5
Illinois	6.4	5.5	4.1	3.8	3.6	5.1	1.9	6.6	6.9
Pennsylvania	5.8	5.8	5.6	5.1	4.0	6.2	1.5	7.4	6.3
Ohio	5.6	5.2	5.0	4.8	3.5	6.3	1.4	7.8	6.3
Texas	4.9	5.5	5.6	5.8	8.2	6.6	8.8	4.2	4.7
Michigan	4.7	4.4	3.7	4.5	3.3	2.9	1.9	6.6	5.4
New Jersey	4.1	3.5	0.7	2.1	1.8	3.5	0.3	4.9	4.1
Massachusetts	3.0	2.8	1.6	1.4	1.3	1.6	0.3	3.3	3.1
Florida	2.9	3.3	3.2	3.2	2.9	2.7	1.8	1.4	2.5
Indiana	2.5	2.6	3.8	3.6	1.4	3.7	1.2	3.9	3.1
Total	62.0	57.4	45.3	46.3	44.8	51.1	25.9	64.7	63.2

[a] Motor fuels other than gasoline that are used on the highway, and consist primarily of diesel fuel and liquefied petroleum gases.
[b] Excluding Alaska.
[c] Covers value added by manufactures, retail sales, and service receipts—all for 1967.
Source: Personal income, Department of Commerce, *Survey of Current Business* (Aug. 1970); population, Bureau of the Census; rural system vehicle-miles, trucks registered, and special fuels, U.S. DOT, *Highway Statistics, 1969;* all other data, Department of Commerce.

states is heavily slanted in favor of a few that are well-endowed with both population and economic activity centers—11 "industrial" states received 62 percent of total personal income in 1969 (Table 17).

It follows, therefore, that the national rate of change in personal income is dominated by, and closely related to, the experience of the industrial states. Table 18 gives the relative difference between the personal income growth performance of the foremost "industrial" states and the nation. The change for the listed states as a group was 95.0 percent. Of significance is the index showing the relative difference in state and national growth rates for the periods 1949 to 1959 and 1959 to 1969. It appears that over the former period, the industrial states as a group led the nation; over the latter period the group's performance was slightly below that of the nation. This reflects the leveling off of regional income-per-capita differentials. This is exemplified by the phenomenon of "catch-up" growth that has emerged as a characteristic of income change. For example, over the period 1959 to 1969 the traditionally poorer Southeast and Southwest regions led in rate of income growth.

TABLE 18

RELATIVE DIFFERENCE BETWEEN PERSONAL INCOME GROWTH IN THE INDUSTRIAL STATES AND THE U.S., 1959–1969

STATE	PERSONAL INCOME ($ BILLIONS) 1959	1969	CHANGE (%) 1959–69	RELATIVE DIFF. BETWEEN STATE AND U.S. GROWTH RATES[a] 1949–59	1959–69
California	41.0	83.1	103.4	51.8	9.4
New York	44.3	81.0	83.3	17.2	—11.9
Illinois	25.8	47.6	83.7	—9.6	—11.4
Pennsylvania	24.6	43.2	75.0	—18.0	—20.6
Ohio	22.0	40.6	82.4	9.3	—12.8
Texas	18.0	36.4	102.6	—1.5	8.6
Michigan	17.5	34.6	100.3	—3.2	6.1
New Jersey	15.8	30.6	91.3	12.0	—3.4
Massachusetts	12.1	22.6	87.2	—14.3	—7.7
Florida	9.3	21.8	140.6	122.1	48.9
Indiana	9.8	18.9	93.0	—4.0	—0.3
Subtotal	240.4	460.4	84.5	9.4	—10.6
U.S.	381.0	743.0	95.0	—	—

[a] State growth rate divided by national growth rate less 1.00 times 100.
Source: Personal income, Department of Commerce, *Survey of Current Business* (Aug. 1970).

It may be assumed that the leveling of interregional and interstate growth differentials will continue in the future (*34*). Moreover, the industrial states in their capacity as the prime arbiters of the level of exogenous intercity ton-mile generation may be expected to exhibit a corresponding conformity with the national growth performance; this is the essence of this analysis of the distribution of personal income by state. The leveling out of differential rates of increase points to the usefulness of personal income, and by substitution GNP, as the best global indicator of future growth in freight movement demand. It follows that it is the performance of the industrial states that acts as the dominant instigative force determining the aggregate or global level of economic activity. On the other hand, the state-by-state distribution of personal income appears to bear little relationship to the ton-mile activity among or within states.

Use of Special Fuels Consumption in State Ton-Mile Projections

It is now possible to isolate the significant determinants on which a forecast by state of future highway ton-mile activity may be based. The distribution by state of the consumption of "special fuels" given in Table 19 should be recognized. This distribution series is to be used as the criterion by state of freight ton-mile activity. Of special importance are the average annual rates relating to real-term increase in GNP, in personal income (including that of the "industrial" states), and in intercity ton-miles of freight. The rates of increase applying to these three indicators show a strong similarity to each other. The logic behind this is described by Kanwit et al. Further, on the basis of these relationships, a forecast is provided of the real-term rate of increase in GNP that may be most reasonably expected over the planning period. This rate, 3.8 percent per annum, has been applied equally to personal

TABLE 19

FORECAST OF RATES OF GROWTH OF SELECTED TRANSPORT OUTPUT INDICATORS, INDUSTRIAL STATES AND THE U.S., 1959–1990

INDICATOR	AVERAGE ANNUAL COMPOUND RATE OF INCREASE (%) 1960–65	1959–69	1965–69	1969–90
Personal income:				
Current $'s:				
Industrial states	6.0	6.7	—	—
United States	6.1	6.9	—	—
Constant 1960 $'s:				
Industrial states	4.5	4.2	3.2	3.8
United States	4.5	4.1	3.4	3.8
Rural system vehicle-miles, U.S.:				
All vehicles	3.7	3.4	—	—
Trucks	5.8	5.1	—	—
Single-unit trucks	n.a.	n.a.	6.9	—
Combinations	n.a.	n.a.	3.8	—
ATA volume of Class I and II motor freight carried (1959=100)	7.4	7.3	—	—
Intercity property freight carried (millions of revenue tons)	38.2	23.0	—	—
Intercity highway ton-miles of freight	4.7	3.7	3.0	3.8
Trucks registered:				
Industrial states	4.3	4.2	—	—
United States	4.4	4.4	—	—
Trucks manufactured, U.S.:				
All trucks	11.7[a]	6.8[b]	2.4	—
Trucks over 30,000 lb, GVW	23.8[a]	17.8[b]	11.9	—
Special fuel consumed:				
Industrial states	11.2	10.9	—	—
United States	10.8	11.0	—	—
Industrial production (quantity) (1959=100), U.S.	7.2	7.6	—	—
Value of shipments from manufacturing plants (constant 1960 $'s)	—	4.3	—	—
Gross National Product (constant 1960 $'s)	4.8	3.7	3.6	3.8

n.a. = not available.
[a] 1961 to 1965.
[b] 1961 to 1969.
Note: The forecast rates are keyed to the anticipated real-term increase in GNP.
Source: Personal income, Dept. of Commerce, *Survey of Current Business* (various eds.), (deflator from *Economic Report of the President*, 1971 ed.); rural system vehicle-miles, U.S. DOT, *Highway Statistics* (various eds.); ATA volume of Class I and Class II freight index, Dept. of Research and Transport Economics, American Trucking Assns.; intercity property freight index, Dept. of Commerce, *Survey of Current Business* (various eds.); intercity highway ton-miles of freight, ICC, given in Automobile Manufacturers Assn., *1970 Motor Truck Facts;* trucks registered, U.S. DOT, *Highway Statistics* (various eds.); trucks manufactured, Automobile Manufacturers Assn., *1970 Motor Truck Facts;* special fuel consumed, U.S. DOT, *Highway Statistics* (various eds.); index of industrial production, Federal Reserve Board, given in Dept. of Commerce, *Survey of Current Business* (various eds.); value of shipments, Dept. of Commerce, *Industry Profiles 1958-1968;* GNP, *Economic Report of the President, 1970.*

income and to intercity highway ton-miles of freight to provide global control totals for these parameters.

In short, the procedure requires two steps to arrive at a projection of intercity highway ton-miles of freight for any state for a given forecast year. The first step requires the determination of the total national projection of ton-miles for the forecast year based on the anticipated real-term rate of increase in GNP. The second step allocates the proportion of the total national ton-miles to the state according to its share of special motor fuel consumption.

Special fuels consist primarily of diesel fuel (about 95 percent) and liquified petroleum gases. According to the Automobile Manufacturers Association (*35*) of all trucks produced in 1969 and sold in the U.S., 5.7 percent were diesel-powered. However, of the trucks between 26,000- and 33,000-lb gross vehicle weight (GVW), 34 percent were diesel-powered; of those greater than 33,000-lb GVW, 80 percent were diesel-powered. Table 20 gives the trend in the use of diesel fuel as the main power source for heavy trucks. Parallel to the increasing reliance on diesel fuel, from 1961 to 1969 the percentage of vehicles produced and sold of more than 33,000-lb GVW increased from 3.2 to 6 percent. Special fuel consumption, therefore, appears a sound indicator of the distribution of activity by size and weight of affected vehicles. Moreover, being a direct measure of vehicle use, this indicator is the most significant controlling variable in the derivation of estimates of vehicle-miles of travel.

The combined effects of the trend toward more diesel-powered heavy trucks and the gradual increase in the percentage of heavy trucks within the total vehicle fleet contribute to a steady increase in special fuels as a percentage of the total highway motor fuel consumption. This serves to emphasize the validity of using the sales of diesel fuel by state as a surrogate for heavy-vehicle activity by state. It should be noted, however, that the high rate of increase in diesel fuel sales is heavily influenced by the switch to the diesel power unit. The rates depicted, therefore, are not representative of the natural secular growth patterns in heavy-vehicle activity. The relative distribution of special fuel consumption by subareas, however, appears reasonable.

This conclusion is supported by the relative constant share of special motor fuel consumption found for the 11 industrial states in Table 20. Although a number of definite shifts are apparent, the rank order of the five most important industrial states in terms of special motor fuel consumption did not change between 1961 and 1969.

Therefore, GNP can be used to obtain a national projection of ton-miles of motor freight. This national demand is then assigned to an individual state on the basis of its share of total special fuels consumed in that state.

For example, to project 1980 intercity ton-miles of freight for Tennessee, first project the 1980 total for the U.S. Assuming a 3.8 percent real growth in GNP, the total is obtained by multiplying the latest base-year figure (1969): 404,000 million ton-miles × 1.5072 (1.03811) = 608,900. Tennessee's share then would be 2.3 percent, or 14,000 million ton-miles. For 1985, the procedure would yield 16,800 million ton-miles (733,700 × 0.023); i.e., 2.3 percent of 733,700 (404,000 × 1.8162).

The results of this procedure appear in Table 21, which forecasts intercity highway freight traffic by public and private road carriers by state, 1969 to 1990. By 1990, intercity road freight movements are anticipated to reach 884,000 million ton-miles. (By comparison, Kanwit calculated that 762,000 million ton-miles would occur on the rural highway system by 1990.)

The foregoing analysis is an aggregative approach to establishing intercity road freight transport demands by state. In essence, the line of analysis affirms the validity of a forecasting approach uncomplicated by attempts to establish future transport demands based on the individual characteristics of each state.

Although the objective of the study is to provide each state with a means or method to adequately measure the impact of changes in vehicle sizes and weights, it may be assumed that a state may choose to capture weight and size change impacts within individual travel corridors. This is reasonable and to be expected. However, to be of value to both state and federal policy makers, such impacts sub-

TABLE 20

SPECIAL MOTOR FUEL CONSUMPTION OF THE INDUSTRIAL STATES, 1961–1969 (% OF TOTAL U.S.)

	FUEL CONSUMPTION (%)								
STATE	1961	1962	1963	1964	1965	1966	1967	1968	1969
California	11.6	12.0	11.3	10.5	10.1	9.6	9.0	8.7	9.1
Texas	7.2	6.8	6.7	6.9	6.8	6.8	7.0	6.9	6.5
Ohio	5.4	5.6	6.0	6.2	6.4	6.4	6.1	6.4	6.3
Pennsylvania	5.3	5.3	5.4	5.5	5.8	6.0	6.2	6.2	6.2
Illinois	4.9	5.0	5.1	5.2	5.1	5.1	5.1	5.1	5.1
New Jersey	3.8	3.6	3.8	4.0	3.9	3.8	3.7	3.7	3.5
New York	3.7	3.4	3.3	3.4	3.3	3.3	3.4	3.2	3.4
Indiana	3.1	3.4	3.6	3.6	3.7	3.7	3.7	3.7	3.7
Florida	2.0	2.1	2.2	2.3	2.3	2.4	2.5	2.5	2.7
Michigan	1.9	2.1	2.3	2.5	2.7	2.8	2.8	2.9	3.0
Massachusetts	1.4	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.6
Total	50.3	50.8	51.2	51.7	51.7	51.5	51.1	50.9	51.1

sequently must be aggregated in relation to defined systems, particularly highway systems. Also, the adopted procedures must have sufficient uniformity to enable comparability between states and for aggregation to provide measurements at the national level.

It does not appear reasonable that heavy-vehicle travel demands on major corridors within a particular state can be assessed through resort solely to national performance indicators. Within its boundaries, each state has a unique combination of socioeconomic, transportation, and geographic characteristics. Also of particular relevance is the influence on state travel patterns of the spatial disposition of the population and employment centers. The location of population, in turn, is heavily influenced by the availability of transport facilities. For example, it has been noted that "while the automobile has tended to equalize accessibility over broad areas, it is evident from recent urban expansion that freeway and other transportation corridors are likely to experience more rapid growth than other areas" (*36*).

Two important data sets are necessary for specific corridor impact analyses: (1) "ongoing" traffic counts and processed AADT figures developed under the functional classification procedures for highways, and (2) continuing truck weight surveys. These represent the key building blocks to the demand side necessary for quantifying the impact on size and weight of affected vehicles within specific travel corridors.

On the other hand, it is considered that the estimates of future ton-mile activity given in Table 20 provide the necessary control totals for making ton-mile assignments for specific corridor analyses. These estimates provide an essential and a suitably accurate depiction of future demand for analyses conducted on the basis of the state as delineated both as a geographic and administrative unit.

FORECASTING INFLUENCE OF NEW LIMITS ON VEHICLE CHARACTERISTICS

The degree to which motor vehicle operators may beneficially implement a change in their equipment and operations must be precisely forecast if the impact of proposed changes in legal limits is to be evaluated. In the case of a change in axle weight limits, a straightforward method can be assembled using data from truck weight studies which lead to the assembly of axle load equivalent factors to arrive at a projected total load experience.

Changes in dimensions cannot be forecast with comparable reliability because of the complexity of possible combinations of dimensions and their relationships to geometric design, highway operations, safety, etc.

Any change enacted in legal limits will not always result in immediate movement of large volumes of vehicles making beneficial use of the change. The rate of implementation will depend, for example, on the availability of equipment responsive to the new limits, the conversion lag as operators phase out existing equipment and phase in the new, and the relative advantages to the operators resulting from the change. Interpretation of each forecast must take this into consideration, principally by a judgment factor

TABLE 21

FORECAST OF INTERCITY HIGHWAY FREIGHT TRAFFIC BY PUBLIC AND PRIVATE ROAD CARRIERS, BY STATE, 1969–1990

STATE [a]	HIGHWAY FREIGHT TRAFFIC (MILLIONS OF TON-MILES) 1969	1980	1990	% OF TOTAL [b]
California	36,000	53,000	81,000	9.1
New York	14,000	21,000	30,000	3.4
Illinois	20,000	30,000	46,000	5.1
Pennsylvania	24,000	37,000	56,000	6.2
Ohio	25,000	38,000	57,000	6.3
Texas	27,000	40,000	60,000	6.6
Michigan	12,000	18,000	26,000	2.9
New Jersey	14,000	20,000	31,000	3.5
Massachusetts	6,000	9,000	13,000	1.6
Florida	11,000	17,000	25,000	2.7
Indiana	16,000	23,000	36,000	3.7
Missouri	12,000	18,000	26,000	2.9
Wisconsin	7,000	11,000	15,000	1.8
Maryland	5,000	8,000	11,000	1.3
Virginia	12,000	18,000	26,000	3.0
North Carolina	11,000	17,000	24,000	2.6
Georgia	13,000	19,000	29,000	3.3
Connecticut	5,000	8,000	11,000	1.2
Minnesota	7,000	11,000	15,000	1.8
Washington	6,000	9,000	13,000	1.5
Tennessee	9,000	14,000	20,000	2.3
Louisiana	6,000	9,000	13,000	1.4
Iowa	8,000	12,000	17,000	1.9
Kentucky	6,000	9,000	13,000	1.6
Alabama	8,000	12,000	17,000	1.9
Kansas	6,000	9,000	13,000	1.4
Oklahoma	7,000	11,000	15,000	1.7
Colorado	4,000	6,000	9,000	1.0
Oregon	7,000	11,000	15,000	1.7
South Carolina	5,000	8,000	11,000	1.3
Arizona	5,000	8,000	11,000	1.6
Nebraska	4,000	6,000	9,000	1.0
Mississippi	7,000	11,000	15,000	1.7
Arkansas	6,000	9,000	13,000	1.4
West Virginia	4,000	6,000	9,000	1.0
D.C.	1,000	1,000	2,000	0.2
Rhode Island	1,000	1,000	2,000	0.3
Utah	3,000	5,000	7,000	0.8
Maine	2,000	3,000	4,000	0.4
Hawaii	1,000	1,000	2,000	0.2
New Mexico	5,000	8,000	11,000	1.2
New Hampshire	1,000	1,000	2,000	0.2
Delaware	1,000	1,000	2,000	0.2
Montana	3,000	5,000	7,000	0.8
Idaho	2,000	3,000	4,000	0.4
Nevada	3,000	5,000	7,000	0.6
South Dakota	2,000	3,000	4,000	0.5
North Dakota	2,000	3,000	4,000	0.5
Vermont	1,000	1,000	2,000	0.1
Alaska	1,000	1,000	2,000	0.1
Wyoming	1,000	1,000	2,000	0.1
Total	404,000 [a]	609,000 [d]	884,000 [d]	100.0

Note: Columns may not add to totals due to rounding.
[a] Includes the District of Columbia.
[b] Distribution according to consumption by state of diesel and liquefied petroleum gases for highway use. (See Table 20.)
[c] Base ton-mile data from Bureau of Economics, ICC, *Transport Economics, Monthly Comment* (Aug. 1971).
[d] An average annual global rate of increase of 3.8 percent has been adopted. See Table 4.

applied to a forecast mode which assumes immediate conversion of equipment to the new limits.

Economy of Size

Various modes of transport compete for inland transport of commodities. Their managements are constantly studying means to improve equipment and techniques. The trend in almost all media is toward vehicles of greater payload capacity, up to some maximum limit resulting from a combination of economic, engineering, and operating factors.

Based on tonnage and density of commodities moving over the highways in line-haul operation, it has been said that only about 10 percent of the total highway tonnage could use to advantage a greater cubic capacity of the vehicle (viz., greater dimensions—width, height and/or body length) without a commensurate increase in number of axles or cargo units (*37*). In slight contradiction, another study (*38*) estimated that about one-half of all shipments are closed out due to this volume factor.

The latter study states that today's general truck freight shipment averages about 12.5 lb/cu ft (*9*). At this density, a standard 40-ft semi-trailer body is physically filled before reaching an optimum weight, estimated to be about 30 percent short of its maximum. The optimum density has been estimated at about 17.5 lb/cu ft.

For combinations using 45-ft semi-trailers, a freight density of 12.5 lb/cu ft more nearly approaches the maximum weight. This range of density is one feature favoring twin-trailer combinations. Others are operational efficiency in loading and unloading, the flexibility provided in permitting division of shipments without reloading, and economic factors leading to increased productivity.

TABLE 22

STOWAGE CAPACITY RELATED TO LENGTHS OF VAN TRAILER COMBINATIONS

	LENGTH (FT)		TOTAL CARGO STOWAGE CAPACITY
COMBINATION TYPE	OVER-ALL	TOTAL CARGO BODY	(CU FT)
Tractor and semitrailer	45	35	1,880
	50	40	2,150
Truck and full trailer	60	50	2,660
Tractor semitrailer and full trailer	60	50	2,660
	65	54	2,870
	100	80	4,300

Source: Stevens (*124*).

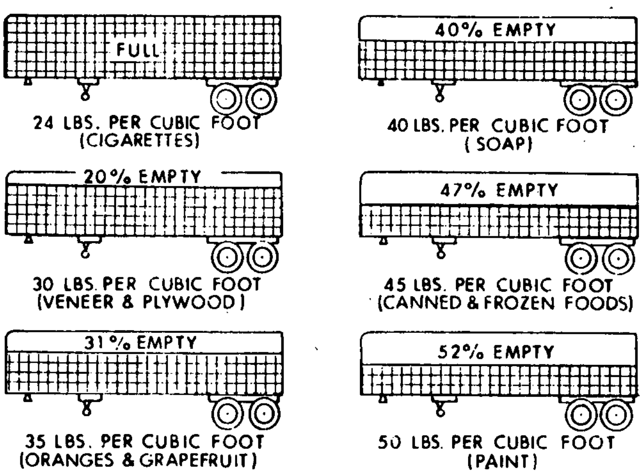


Figure 10. Examples of vehicle space utilization. Source: Kent, M. F., "The Freight's the Weight." Proc. HRB, *Vol. 37 (1958) p. 35.*

Motor Truck Factors Affecting Transport Economy

Table 24 gives the interdependency and no dependency of the various physical truck factors and the operational factors of the transport industry, the highway, the economy of transportation, and defense requirements. This is a good conclusive statement of the results of these considerations.

Vehicle Height

The desirable height limit is primarily the result of loading practice, overhead clearances on the highway, and the effect of vehicle height on traffic.

The transport industry has expressed no particular interest in increases in the height of freight vehicles, with the exception of certain large items of specialized equipment. This is because a particular commodity can be stacked only so high without damage from its own weight, and loading/unloading costs increase with higher stacking. Furthermore, increasing vehicle height would require considerable remodeling of freight depots, warehouses, and docks, and could result in vehicle instability in high crosswinds and at sharp turns.

Hauling of light-density products (say, 10 to 15 lb/cu ft) is limited by interior cubic space and not axle weight limits. Cubage can be increased by an increase in any of the three dimensions. Industry appears to prefer increases in length and slight increases in width, rather than increases in height.

Added vehicle height may increase highway construction costs. Vertical clearances of bridges, underpasses, utility lines, traffic control devices, overhead signs, etc., would be affected by certain magnitudes of limit changes to vehicle heights. A relatively small percentage of goods transported by highway, whether measured on a tonnage basis or on a cubic foot basis, would require an especially high maximum legal limit for vehicle with load.

Vehicle Width

Both transport industry (passenger bus and trucks) and highway officials are concerned about vehicle widths. The bus industry, questioned by the researchers, expressed a desire to seat a minimum of four persons transversely while maintaining a center aisle wide enough to permit passengers to move in and out of the bus safely. To do so would require an increase in over-the-road bus width limits to 102 in. Some bus terminal facilities would require remodeling should an increase in width be adopted.

TABLE 23

PAYLOAD WEIGHTS RELATED TO STOWAGE CAPACITY OF VAN TRAILER COMBINATIONS [a]

COMMOD. DENSITY (PCF)	PAYLOAD WEIGHT (LB) TA-S 35 FT	TA-S 40 FT	TR-F 50 FT	TA-S-F 50 FT	TA-S-F 54 FT	TA-S-F 80 FT
10	18,800	21,500	26,000	26,600	28,700	43,000
15	28,200	32,250	39,900	39,900	43,050	64,500
20	37,600	43,000	53,200	53,200	57,400	86,000
25	47,000	53,750	66,500	66,500	71,750	107,500
30	56,400	64,500	79,800	79,800	86,100	129,000
35	65,800	75,250	93,100	93,100	100,450	150,500
40	75,200	86,000	106,400	106,400	114,800	—
45	84,600	96,750	119,700	119,700	129,150	—
50	94,000	107,500	133,000	133,000	143,500	—

[a] Ta = tractor, Tr = truck, S = semi-trailer, and F = full trailer. Length is approximate length of cargo body.
Source: Stevens (*124*).

The trucking industry desires an increase in vehicle width to accommodate certain commodities having dimensions of modular nature such as 2, 4, and 8 ft. An 8-ft bed width cannot be provided when a 96-in. maximum width is imposed, due to the van wall construction. Increases in truck width would affect terminal facilities' existing driveways, alleys, warehouses, and public alleys.

In the area of vehicle design, industry believes that the present limitation of 96 in. does not provide the necessary width over the rear drive axles for adequate design of differential, braking, and tire equipment. Increased vehicle stability probably would result from a wider tread.

Widths of buses and freight vehicles affect highway design and traffic operations (e.g., horizontal clearances on bridges and in tunnels, lane width, and traffic safety). Highway geometrics must be studied to determine the effect of lateral placement of vehicles within the lane. Overtaking and passing maneuvers are significant on two-lane bi-directional highways. Traffic operations and safety are "closely related" to the placement of trucks with respect to merging, overtaking and passing, design of interchange ramps, low-radius curves, and corners. Extra lane width due to offtracking on sharp curves will have to accompany an increase in vehicle width.

Vehicle Length

The lengths of single-unit vehicles and over-all lengths of combinations require different treatment. Bus operators

TABLE 24

TRANSPORT, HIGHWAY, AND VEHICLE FACTORS [a]

OPERATIONAL FACTOR	HEIGHT	WIDTH	LENGTH: SINGLE UNITS: BUS	TRUCK	TRAILER	COMBINATIONS	NO. OF UNITS	AXLE WEIGHT	GVW
A. Transport industry									
1. Loading and unloading operations	x	x	x	x	x	x	x	x	x
2. Terminal facilities	x	x	x	x	x	x	x	—	—
3. Urban pickup and delivery	—	x	—	x	x	x	x	x	x
4. Line-haul, or intercity operation	x	x	x	x	x	x	x	x	x
B. Highway									
1. Geometrics, transverse	—	x	x	x	x	x	x	—	—
2. Geometrics, longitudinal	x	—	x	x	x	x	x	—	—
3. Structural design of pavement	—	—	—	—	—	—	—	x	x
4. Structural design of bridges	x	x	—	x	x	x	x	x	—
5. Effects on traffic	x	x	x	x	x	x	x	—	x
6. Traffic safety	x	x	x	x	x	x	x	—	x
C. Economy of transportation									
1. Highway costs	x	x	—	—	—	x	—	x	—
2. Vehicle operating cost	—	—	—	—	—	—	—	—	x
D. Defense requirements	x	x	—	—	—	x	—	x	x

[a] X = factors are interdependent; — = no dependency. The factor of ADT is not considered. Pavement structures of terminal areas could be affected by these factors under certain circumstances.

desire increased length for more passengers per vehicle and for additional amenities of rest rooms, food service, etc.

Length of freight vehicles depends on intended type of service. Single units operating in loading/unloading operations in urban pickup and delivery are affected differently by length limits from those in line-haul operations. Vehicle combinations where two or three cargo units operate with one tractor are not generally feasible for urban traffic conditions. These types, where permitted in about 24 states, usually break up into small combinations for urban operations after the long-haul phase of the trip.

Additional length, it was maintained, permits the increase in the number of axles. Total gross weight can thereby be increased without exceeding axle-weight limits. This applies to highway surfaces, but would have to be examined for its impact on bridge structures that may be affected.

Longitudinal geometrics of highway design are affected by the vehicle length through restrictions on sight distances and passing opportunities. Greater time is required for passing clearance.

Highway costs attributable to increased length include extra roadway widths to accommodate the combination offtracking characteristics at curves and ramps, and construction required to lengthen the sight distance on curves and crests.

Basic Concepts Determining Truck Performance

For any vehicle at a specific load condition to travel at a steady speed on level pavement, all external existing forces must be overcome. These external forces are the drawbar pull required to maintain that speed comprised of the following components: rolling resistance, and air resistance. In an accelerating mode, inertial resistance is introduced. In negotiating a grade, the "grade resistance" to overcome gravity is encountered.

These external forces are overcome by torque delivered to the drive wheels through the transmission and differential by the engine. To be effective, this torque must be resisted by the traction of the tire contact point on the roadway surface. The power lost in transmitting engine power to tire-road surface contact area is commonly due to dissipation of energy in the driveline resistance. This loss occurs after the torque developed at the clutch plate; therefore, the drive wheel torque always is lower than the clutch plate torque. The rolling resistance of the driven wheel(s) also is included in the drive train resistance.

TABLE 25

ROLLING RESISTANCE FACTORS

SURFACE TYPE AND CONDITION	ROLL. RESIST. FACTOR
Smooth concrete and asphalt (dry)	0.006
(wet) [a]	0.015
Brushed concrete (dry)	0.012
Asphalt with sand or chip seal (dry)	0.012
Packed snow	0.013
Packed earth or gravel (dry)	0.06
Sand (dry)	0.25

[a] Water depth insufficient to create resistances other than those resulting from tire hysteresis and surface tension.

Rolling Resistance Force

Many variables affect the rolling resistance. If it is assumed that tires are correctly inflated for their load, tire temperature is stabilized at normal operating conditions, there is usual tire wear, and there are hard pavement surfaces, then for a narrow range of tire sizes of similar construction and low speeds, the rolling resistance factor is (*39*):

$$R_r = \nu^N$$

in which N is wheel load normal to pavement, and ν is a constant for a given speed. On a hard surface, rolling resistance is equal to about 50 lb of drawbar pull for each ton of GVW or GCW.

Rolling Resistance Factor

The rolling resistance is a function of the surface type, and whether the surface is wet or dry. It does not remain constant with speed but appears to vary as:

$$r = C_1 + C_2V$$

in which C_1 and C_2 are constants, and V is in mph. Approximate rolling resistance factors of various road surfaces for heavy trucks at normal road speeds are as given in Table 25 (*39*).

In computing rolling resistance, the following is used (*39*):

$$R_r = \frac{(\text{GCW} - N_d')}{375} V_r$$

in which

V = speed, mph;
GCW = gross combination (vehicle) weight;
r = rolling force factor;

and

$N_d' = N_s + T(h/b)$
N_s = scale weight in pounds on drive axle(s);
T = recorded tractive effort, pounds;
h = fifth wheel pivot height, inches;
b = truck-tractor wheelbase, inches.

If it is assumed that the rolling resistance of all wheels (including the drive wheels) is a single external resistance to be overcome by the propulsion system, then (*39*):

$$R_r = [(\text{GCW})\,V_r/375]$$

Rolling resistance is an extremely complex subject. Only the principal aspects are mentioned here.

Air Resistance

Air resistance of a truck is a function of the design and configuration of the vehicle. Laws of aerodynamics are applied in computing this function. At low speeds, air resistance is negligible. At speeds of about 30 mph and more, it becomes a significant factor dependent on the frontal area. Air resistance in still air sometimes is computed by (*39*):

$$R_a = AV^3/C$$

in which

A = frontal area, square feet;
V = speed, mph; and
C = constant.

This simple equation ignores the effects of skin friction (laminar flow friction), design for streamlining, etc., wind, turbulence, and air density. A more detailed expression that includes air drag is (*39*):

$$R_a = KAV^3/375$$

in which

K = air drag coefficient at a given air density;
V = speed, mph;
A = frontal area = $W(H - 1)$;
W = over-all vehicle width, feet; and
H = height, feet.

In the above equation, K is given in one technical publication as 0.00164. However, for longer combinations of van trailers, K has been determined to be 0.0023 (*39*).

Grade Resistance

Grade resistance is that component of the gross vehicle weight acting parallel to the incline surface. Thus, grade resistance can be computed as:

$$R_g = (\text{GVW}) \sin \gamma$$

in which

GVW = gross vehicle or combination weight; and
γ = angle of incline.

For grades of less than 10 percent, the following approximation can be used:

$$R_g = (\text{GVW})\ G$$

in which

G is grade expressed as a decimal.

Thus, for each ton of gross weight, 20 lb of resistance are exerted by each 1 percent of grade.

Gradeability

The ability of a truck or truck combination to negotiate a certain grade is directly related to the tractive capability possessed by the power unit through its drive train to overcome grade resistance. The same power unit, although having constant tractive effort, probably will exhibit different gradeability depending on the configuration of the semi-trailer/trailer combination and the distribution of axle load on the driven axle(s) to the gross vehicle or combination weight. The gear ratio employed affects the gradeability. The low or low-low gear drive has the maximum gradeability at the expense of minimum sustained speed on a limiting grade for most cases, because in this condition the net driving force is maximum.

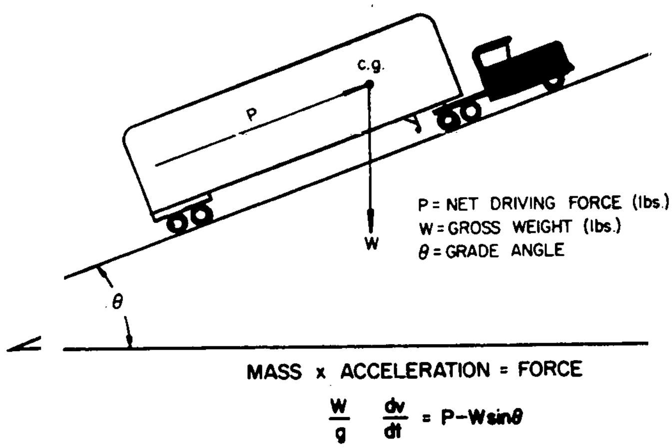


Figure 11. Truck ascending diagram. Source: Glennon and Joyner (113).

A simplified equation for the hill-climbing ability of a vehicle was developed from theory of forces acting on the vehicle on an incline (Fig. 11).

The equation, based simply on $F = ma$, is:

$$(W/g)(dv/dt) = P - W \sin \theta$$

in which

W = gross weight;
g = acceleration of gravity (32.2 ft/sec^2);
dv/dt = vehicle acceleration, ft/sec^2;
θ = angle of grade (+ if an upgrade, — if a downgrade); and
P = net driving force of driven axle(s) (neglecting wind, surface, and inertial resistance of rotating parts).

On the assumption that P/W remains constant, regardless of acceleration, a plot of computations was made using, for the most part, basic velocity data observed for average heavy vehicles operating on mountain grades in Arizona (*41*). Speed points interconnected with straight lines are shown in Figure 12 (*41*).

Each straight line segment could be represented by:

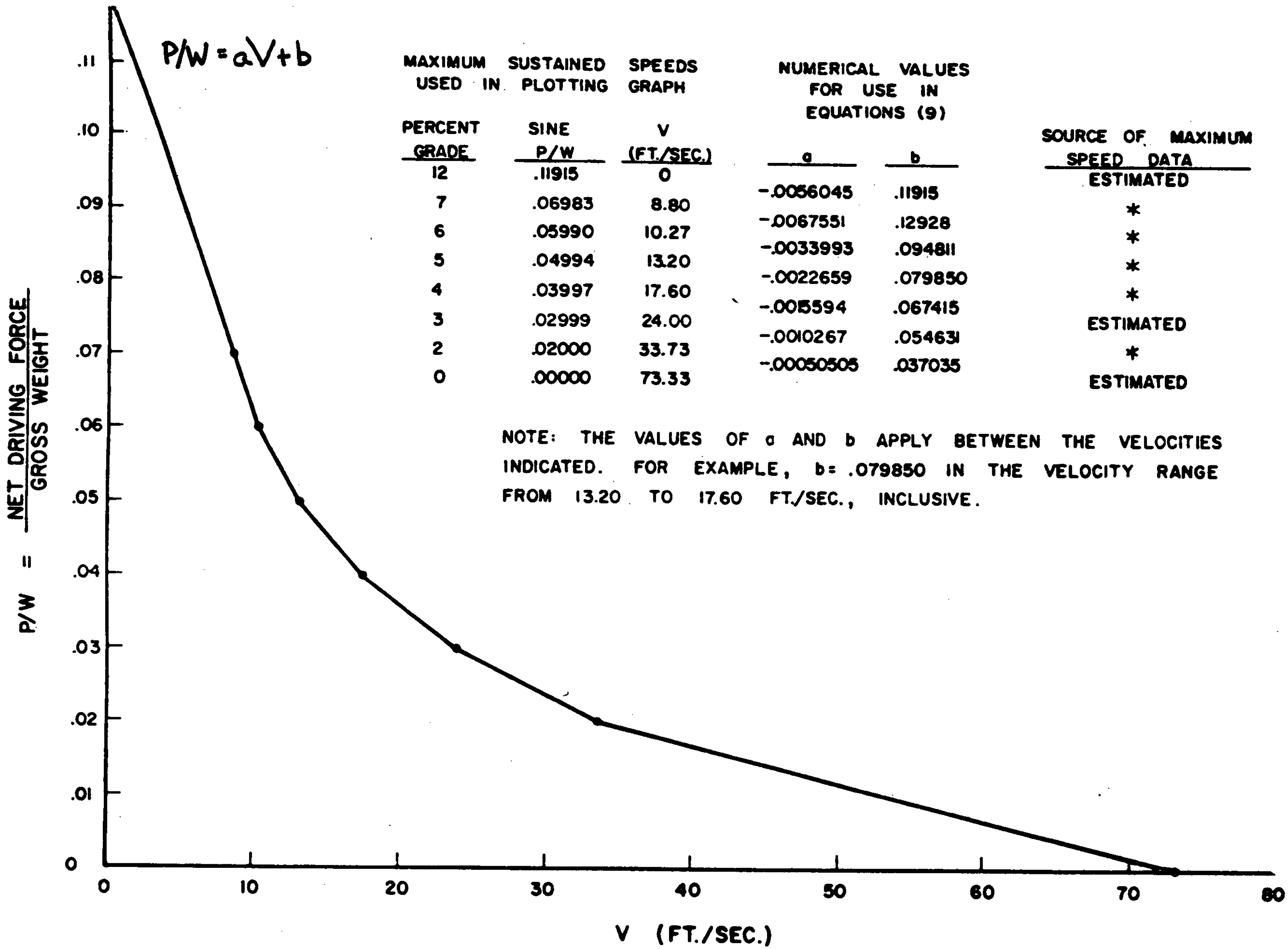
$$P/W = aV + b$$

in which

V = velocity change between V_n and V_{n+1} within interval;
a and b = constants within that interval; and
V_n and V_{n+1} = incremental velocities along a straight line segment of the plot.

A simple expression for speed-distance can be obtained as follows:

$$x = \frac{1}{g}\ \frac{V_0{}^2 - V^2}{a(V_0 - V) - 2(\sin\theta - b)}$$



MAXIMUM SUSTAINED SPEEDS USED IN PLOTTING GRAPH

NUMERICAL VALUES FOR USE IN EQUATIONS (9)

PERCENT GRADE	SINE P/W	V (FT./SEC.)	a	b	SOURCE OF MAXIMUM SPEED DATA
12	.11915	0			ESTIMATED
			-.0056045	.11915	
7	.06983	8.80			*
			-.0067551	.12928	
6	.05990	10.27			*
			-.0033993	.094811	
5	.04994	13.20			*
			-.0022659	.079850	
4	.03997	17.60			*
			-.0015594	.067415	
3	.02999	24.00			ESTIMATED
			-.0010267	.054631	
2	.02000	33.73			*
			-.00050505	.037035	
0	.00000	73.33			ESTIMATED

NOTE: THE VALUES OF a AND b APPLY BETWEEN THE VELOCITIES INDICATED. FOR EXAMPLE, b = .079850 IN THE VELOCITY RANGE FROM 13.20 TO 17.60 FT./SEC., INCLUSIVE.

Figure 12. P/W *vs maximum sustained speed on various grades. Source: Willey* (41, *p.* 52).

Another similar expression has been published (*41*). The following is used to compute vehicle speed-distance relationships for vehicles on grade:

$$x = V_0 t = (F_0 g / W) t^2$$

$$V = V_0 = (F_0 g / W) t$$

in which

V_0 = entering vehicle velocity;
F_0 = net force on drive wheels at wide-open throttle;
g = percent grade in radians;
W = gross vehicle weight; and
t = time after entering grade, seconds.

Computer calculations yield the curves shown in Figures 13, 14, and 15.

Grade Climbing Under Adverse Conditions

The foregoing equations are based on the assumption that the tractive effort (the force applied to the wheels by the engine and drive train) never exceeded the drive traction ability of the tires in contact with the road surface. Should the tractive effort exceed the drive traction ability of the tires and surface, wheel spin-out occurs and the driving force is no longer effective.

The grade at which this spin-out occurs, or traction-limiting grade, varies with type of vehicle, type of tractor, coefficient of traction, certain truck dimensions, the gross vehicle or combination weight, and the ratio of GCW to static weight on the driven wheels. The following equations have been derived for the traction-limiting grades (*43*):

Single-unit vehicle with all-wheel drive

$$\%\ \text{grade} = f$$

Single-unit vehicle with rear-wheel drive

$$\%\ \text{grade} = f(TR) - r(1 - TR)$$

Combination vehicle with all-wheel drive truck-tractor or full truck

$$\%\ \text{grade} = f(TR) - r(1 - TR)$$

Combination vehicle with rear-wheel drive truck-tractor or full truck all rear wheels driven

$$\%\ \text{grade} = \frac{f(TR)}{1 - f\left(\frac{h}{b}\right)} - r(1 - TR)$$

Combination vehicle with tandem-axle truck-tractor or full truck, pusher or tag-axle drive

$$\%\ \text{grade} = \frac{f(TR)}{1 - f\left(\frac{h}{b}\right)\left(\frac{N_R}{N_T}\right)} - r(1 - TR)$$

Combination vehicle with rear-wheel drive truck-tractor and powered dolly

$$\%\ \text{grade} = \frac{f(TR)}{1 - f\left(\frac{h}{b}\right)} - r(1 - TR) + 0.0175\left(\frac{G_{\max}}{G}\right)$$

% grade = traction-limiting grade. Highest negotiable grade where traction is the limiting factor. (Also spin-out grade.)
f = coefficient of traction. Surface friction utilized by tires in the rolling driven mode.
TR = traction ratio. The static weight on all driven wheels divided by the gross combination weight.
r = rolling resistance factor.
h = fifth-wheel pivot height (in the case of truck-tractors).
h = trailer coupling height or center of gravity (in the case of full trucks).
b = tractor wheelbase.
N_R = static weight upon rear driven axle(s).
N_T = total static weight upon a pusher or tag tandem.
G = gross combination weight.

Using these relationships, Tables 26 and 27 give typical traction-limiting grades, assuming a coefficient of traction of 0.4.

In these comprehensive road tests (*43*)

> . . . standard length and longer trucks loaded to maximums in conformance with Bridge Formula "B" standards,* and made up of well-maintained equipment currently in use can go anywhere on the nation's highways with traction to spare on both wet and dry pavements. The only time some trucks will have difficulties is on grades above 5.0 percent when the pavements are covered with ice or snow.

The tests verified that additional traction ability can be obtained by the use of such traction aids as tire chains, load transfer devices, traction equalizers, anti-slip differentials, and sand or grit spread on the surface.

The report contended that low weight-to-horsepower ratios were overrated as performance guarantees, either for climbing speed or for the ability to climb grades where traction is a critical factor. Both traction and adequate horsepower must be present to ensure good performance. Excessive horsepower on low-traction surfaces may be disadvantageous, especially at low speeds where torque may exceed tractive effort capability, resulting in the wheels spinning out. However, high horsepower will permit greater momentum in enabling the truck to enter short grades at a higher velocity before slowing to a critical speed.

In extensive tests of ten triple combinations (*44*) in actual winter conditions on grades up to 6 percent, snow pack was found to have a coefficient of traction of 0.29 to 0.335 for various densities of snow with a sanded surface. The coefficient of traction with dual-tire chains was 0.33. Chained-up duplex tires provided less traction than chained-up standard dual tires. Coefficient of traction was found to improve from 0.33 with ordinary cross-link chains to 0.35 with chains having reinforced cross links. Certain critically loaded combinations could negotiate grades from 0.50 to 2.75 percent steeper if kept rolling. Once they were stopped, traction demand to overcome static inertia

* $W = 500\left(\frac{LN}{N-1} + 12N + 36\right)$.

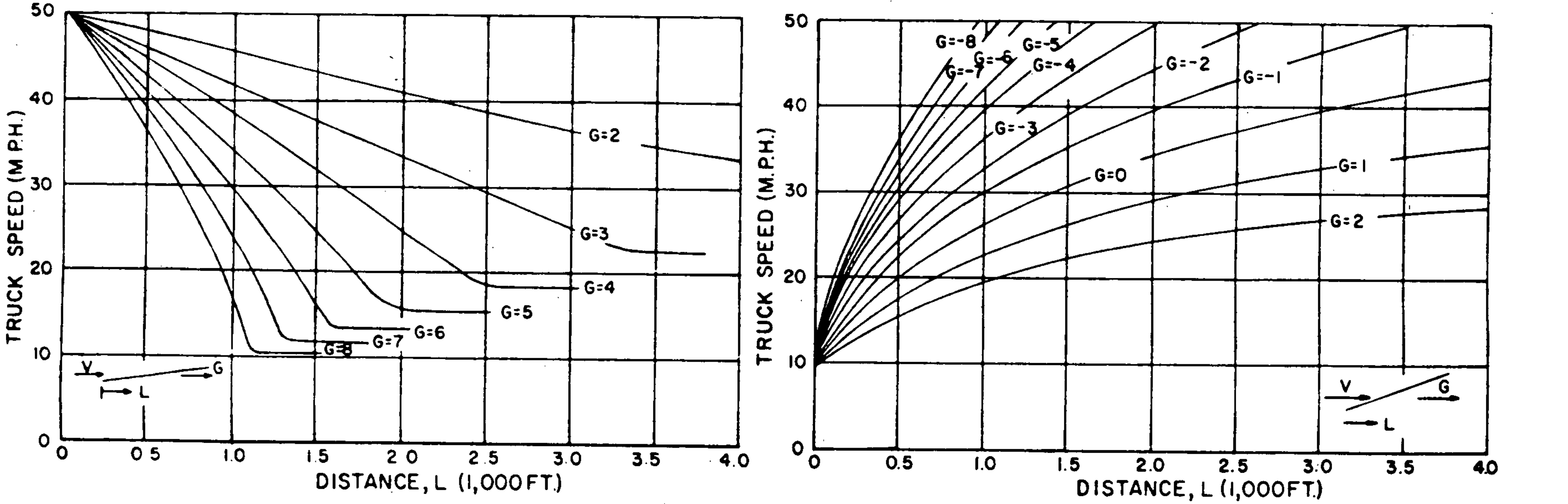


Figure 13. Deceleration and acceleration gradeability curves for trucks with GVW/BHPW ratio=400. Source: Glennon and Joyner (113).

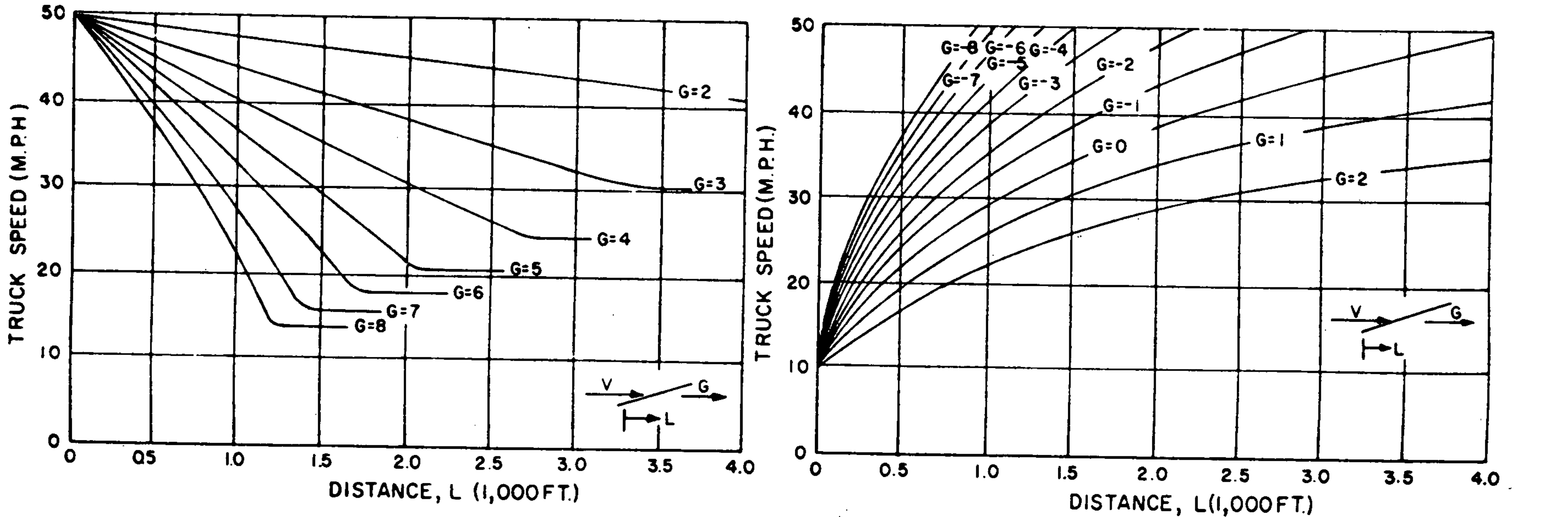


Figure 14. Deceleration and acceleration gradeability curves for trucks with GVW/BHPW ratio=300. Source: Glennon and Joyner (113).

and acceleration prevented some combinations from rolling on these critical grades. Surplus horsepower did not enable a truck to go up a steeper grade than that limited by traction.

No problems with maneuvering, braking, passing, or stability were encountered during these tests.

Inertial Resistance

The force necessary to change a steady state of motion or rest is proportional to the mass of the body multiplied by the rate of change of velocity; i.e., acceleration. In a vehicle the total mass is that of the truck plus that of the engine, drive train, and other rotating components. The latter contributions usually are limited in practical sense to the inertia of the clutch and flywheel. The equation for inertial resistance is sometimes expressed (*39*):

$$R_i = [(GW)a/g] + R_{ir}$$

in which

R_i = inertial resistance, pounds;
GW = gross vehicle or combination weight, pounds;
R_{ir} = inertia of rotating masses;
a = acceleration, fps; and
g = acceleration of gravity, fps.

Engine flywheels and clutches vary in inertia. Flywheels usually fall in the range of 50 to 60 lb-ft^2 and clutches would generally be about 40 lb-ft^2. These inertial components would be multiplied by the square of the over-all gear reduction between their position in the drive train and the axle (transmission and differential).

Other Losses and Influences

Although small in aggregate, other losses occur that reduce the available horsepower to drive the vehicle. Driveline resistance due to mechanical friction of bearing surfaces and viscous friction of oil causes an energy loss which is a function of temperature and speed. Tire inflation pressure influences the rolling resistance of the tire. Tire slip, where driven wheels make more revolutions for the same tire size as undriven wheels, requires a small but usually insignificant energy to overcome.

Power required for such accessories as fuel, oil, and water pumps, fans, alternators, power steering, air conditioners, and other energy-consumption devices reduces the engine gross horsepower. Thus, the net horsepower generally is around 90 percent of the gross horsepower.

Engine Horsepower and Torque

Horsepower is a standard theoretical unit of the rate of work. One horsepower is equal to the energy equivalent of 33,000 foot-pounds per minute. Torque is the rotational force applied at some distance (lever) expressed in foot-pounds. The two are related by

$$HP = M\omega/5{,}252$$

in which M is torque in foot-pounds; and ω is angular velocity in rpm.

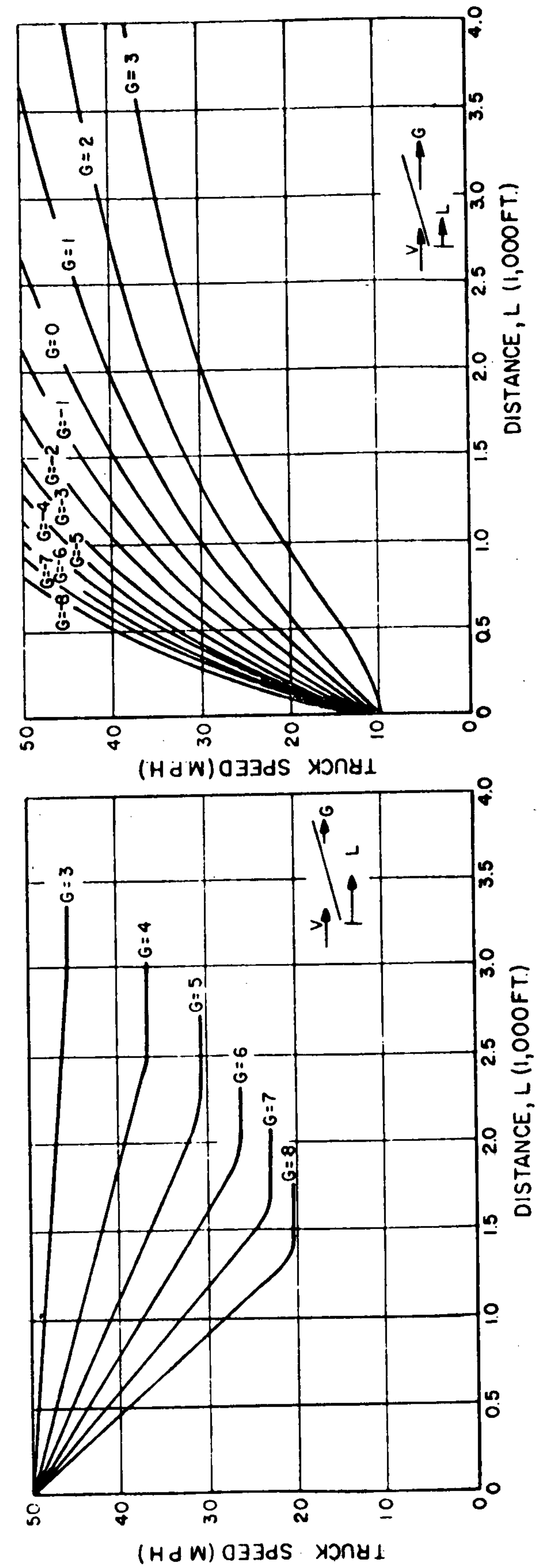


Figure 15. Deceleration and acceleration gradeability curves for trucks with GVW/BHPW ratio=200. Source: Glennon and Joyner (113).

TABLE 26

	California Law	BPR Bridge Formula "B"	Tare	Traction Ratios	GCW	Weight on Driven Axles	Axles	Length Overall	Wheelbase Overall	% Grade @ r = 0.1 f = 0.4
40' Semi										
6 x 4 tractor	x			.437	73,280	32,000	5	50'	45'	16.5
6 x 4 tractor		x		.445	76,125	34,000	5	50'	45'	16.8
6 x 4 tractor			x	.460	25,000	11,500	5	50'	45'	17.4
6 x 2 tractor	x			.246	73,280	18,000	5	50'	45'	8.8
6 x 2 tractor		x		.263	76,125	20,000	5	50'	45'	9.5
6 x 2 tractor			x	.240	25,000	6,000	5	50'	45'	8.6
27' Doubles										
4 x 2 tractor	x			.234	76,800	18,000	5	65'	60'	8.4
4 x 2 tractor		x		.234	85,500	20,000	5	65'	60'	8.4
4 x 2 tractor			x	.251	29,600	7,439	5	65'	60'	9.0
4 x 4 tractor	x			.352	76,800	27,000	5	65'	60'	13.1
4 x 4 tractor		x		.342	85,500	29,225	5	65'	60'	12.7
4 x 4 tractor			x	.514	29,600	15,200	5	65'	60'	19.6
6 x 4 tractor	x			.260	76,800	20,000	6	65'	60'	9.4
6 x 4 tractor		x		.257	85,500	22,000	6	65'	60'	9.3
6 x 4 tractor			x	.324	30,900	10,000	6	65'	60'	12.0
7 Axle 40' Doubles										
4 x 2 tractor	x			.173	104,000	18,000	7	95'	90'	5.9
4 x 2 tractor		x		.178	112,500	20,000	7	95'	90'	6.1
4 x 2 tractor			x	.222	37,600	8,335	7	95'	90'	7.9
4 x 4 tractor	x			.260	104,000	27,000	7	95'	90'	9.4
4 x 4 tractor		x		.260	112,500	29,225	7	95'	90'	9.4
4 x 4 tractor			x	.404	37,600	15,200	7	95'	90'	15.2
4 x 2 + PD1	x			.346	104,000	36,000	7	95'	90'	12.8
4 x 2 + PD1		x		.356	112,500	40,000	7	95'	90'	13.2
4 x 2 + PD1			x	.417	39,200	16,333	7	95'	90'	15.7
9 Axle 40' Doubles										
6 x 4 tractor	x			.308	104,000	32,000	9	95'	90'	11.3
6 x 4 tractor		x		.277	122,625	34,000	9	95'	90'	10.1
6 x 4 tractor			x	.277	41,500	11,500	9	95'	90'	10.1
6 x 6 tractor	x			.404	104,000	42,000	9	95'	90'	15.2
6 x 6 tractor		x		.359	122,625	44,000	9	95'	90'	13.4
6 x 6 tractor			x	.470	41,500	19,500	9	95'	90'	17.8
6 x 4 + PD2	x			.615	104,000	64,000	9	95'	90'	23.6
6 x 4 + PD2		x		.555	122,625	68,000	9	95'	90'	21.2
6 x 4 + PD2			x	.560	43,100	12,600	9	95'	90'	21.4
27' Triples										
4 x 2 tractor	x			.173	104,000	18,000	7	95'	90'	5.9
4 x 2 tractor		x		.178	112,500	20,000	7	95'	90'	6.1
4 x 2 tractor			x	.188	39,500	7,439	7	95'	90'	6.5
4 x 4 tractor	x			.260	104,000	27,000	7	95'	90'	10.0
4 x 4 tractor		x		.260	112,500	29,225	7	95'	90'	10.0
4 x 4 tractor			x	.385	39,500	15,200	7	95'	90'	14.4
4 x 2 + PD1	x			.346	104,000	36,000	7	95'	90'	12.8
4 x 2 + PD1		x		.356	112,500	40,000	7	95'	90'	13.2
4 x 2 + PD1			x	.351	41,100	14,439	7	95'	90'	13.0
6 x 4 tractor	x			.192	104,000	20,000	8	95'	90'	6.7
6 x 4 tractor		x		.187	117,428	22,000	8	95'	90'	6.5
6 x 4 tractor			x	.245	40,800	10,000	8	95'	90'	8.8

Source: WHI (43).

TABLE 27

COMPUTATIONS OF TRACTION LIMITING GRADES WHEN COEFFICIENT OF TRACTION = 0.45

	New York Turnpike	California Law	BPR Bridge Formula "B"	Tare	Traction Ratios	GCW	Weight on Driven Axles	Length Overall	Wheelbase Overall	% Grade @ f = .45 r = .01
40' Semi										
6 x 4 tractor		x			.437	73,280	32,000	50'	45'	22.3
6 x 4 tractor			x		.445	76,125	34,000	50'	45'	22.7
6 x 4 tractor				x	.388	25,000	9,700	50'	45'	19.7
6 x 2 tractor					.246	73,280	18,000	50'	45'	11.2
6 x 2 tractor					.263	76,125	20,000	50'	45'	12.2
6 x 2 tractor					.218	25,000	5,456	50'	45'	10.0
27' Doubles										
4 x 2 tractor		x			.234	76,800	18,000	65'	60'	11.9
4 x 2 tractor			x		.234	85,500	20,000	65'	60'	11.9
4 x 2 tractor				x	.251	29,600	7,439	65'	60'	12.9
4 x 4 tractor		x			.352	76,800	26,991	65'	60'	15.2
4 x 4 tractor			x		.342	85,500	29,225	65'	60'	14.7
4 x 4 tractor				x	.514	29,600	15,200	65'	60'	22.6
6 x 4 tractor		x			.255	76,800	19,566	65'	60'	12.6
6 x 4 tractor		x			.417	76,800	32,000	65'	60'	21.3
6 x 4 tractor			x		.253	85,500	21,608	65'	60'	12.6
6 x 4 tractor			x		.398	85,500	34,000	65'	60'	20.3
6 x 4 tractor				x	.284	30,900	8,786	65'	60'	14.2
7 Axle 40' Doubles										
4 x 2 tractor	x				.204	110,000	22,400	95'	90'	10.3
4 x 2 tractor		x			.173	104,000	18,000	95'	90'	8.6
4 x 2 tractor			x		.178	112,500	20,000	95'	90'	8.9
4 x 2 tractor				x	.222	37,600	8,335	95'	90'	11.3
4 x 4 tractor	x				.290	110,000	31,904	95'	90'	12.4
4 x 4 tractor		x			.260	104,000	26,991	95'	90'	11.0
4 x 4 tractor			x		.260	112,500	29,225	95'	90'	11.0
4 x 4 tractor				x	.431	37,600	16,200	95'	90'	18.8
4 x 2 + PD1	x				.204	110,000	22,400	95'	90'	12.0
4 x 2 + PD1		x			.173	104,000	18,000	95'	90'	10.3
4 x 2 + PD1			x		.178	112,500	20,000	95'	90'	10.6
4 x 2 + PD1				x	.213	39,200	8,335	95'	90'	15.8
9 Axle 40' Doubles										
6 x 4 tractor	x				.283	127,400	36,000	95'	90'	14.1
6 x 4 tractor		x			.308	104,000	32,000	95'	90'	15.4
6 x 4 tractor			x		.277	122,625	34,000	95'	90'	13.8
6 x 4 tractor				x	.234	41,500	9,700	95'	90'	11.5
6 x 6 tractor	x				.360	127,400	45,891	95'	90'	15.6
6 x 6 tractor		x			.403	104,000	41,891	95'	90'	17.5
6 x 6 tractor			x		.359	122,625	44,073	95'	90'	15.6
6 x 6 tractor				x	.422	41,500	17,500	95'	90'	18.4
6 x 4 + PD2	x				.283	127,400	36,000	95'	90'	15.9
6 x 4 + PD2		x			.308	104,000	32,000	95'	90'	17.2
6 x 4 + PD2			x		.277	122,625	34,000	95'	90'	15.6
6 x 4 + PD2				x	.226	43,000	9,700	95'	90'	16.0
27' Triples										
4 x 2 tractor		x			.173	104,000	18,000	95'	90'	8.6
4 x 2 tractor			x		.178	112,500	20,000	95'	90'	8.9
4 x 2 tractor				x	.188	39,500	7,439	95'	90'	9.4
4 x 4 tractor		x			.260	104,000	26,991	95'	90'	11.0
4 x 4 tractor			x		.260	112,500	29,225	95'	90'	11.0
4 x 4 tractor				x	.385	39,500	15,200	95'	90'	16.7
4 x 2 + PD1		x			.173	104,000	18,000	95'	90'	10.4
4 x 2 + PD1			x		.178	112,500	20,000	95'	90'	10.6
4 x 2 + PD1				x	.181	41,100	7,439	95'	90'	13.8
6 x 4 tractor		x			.197	104,000	20,496	95'	90'	9.9
6 x 4 tractor		x			.308	104,000	32,000	95'	90'	15.4
6 x 4 tractor			x		.192	112,500	21,608	95'	90'	9.3
6 x 4 tractor			x		.302	112,500	34,000	95'	90'	15.1
6 x 4 tractor				x	.215	40,800	8,786	95'	90'	10.5

Source: WHI (43).

The effective torque applied to a driven axle is transformed by gears in the transmission and differential and power is lost in this transmission. The effective torque is thus:

$$M_d = M_e y n$$

in which

M_e = engine torque;
y = gear ratio; and
n = transmission efficiency.

The horsepower and torque produced by various engines vary characteristically with engine design. Some engines achieve a higher horsepower at lower rpm than others, with corresponding changes in torque characteristics. Gear selection in the transmission is provided generally so that the developed torque versus fuel consumption approaches an optimum, usually at higher engine rpm.

Fuel Consumption

Fuel consumption is influenced by many factors (e.g., traffic conditions affecting the engine's duty cycle profile, wind force, direction). Tests conducted on engines in road test vehicles indicate that the principal factors affecting fuel consumption are gross weight and design top speed.

A series of runs on a truck with a design top speed of 53.6 mph indicated that the average gallons per mile (gpm) equaled $1.55 \times 10^{-6} \times$ the gross combination weight (GVW) + 0.135. In similar runs with a design top speed of 43.2 mph (40.1 mph average), the gpm was 1.55×10^{-6} (GVW) + 0.127.

The general empirical equation for observed fuel mileage becomes (*39*):

$$\text{gpm} = 0.103 + 1.55 \times 10^{-6} \times \text{GVW} + 14.9 \times 10^{-6}\,(\text{mph})^2$$

Converting fuel consumption to payload ton-miles illustrates better the economic impact of fuel cost to revenue income. These test results indicated that fuel economy greatly increased with GVW. For example, at 50,000 lb, 1 gal was consumed for 40 ton-miles of payload. Doubling GVW to 100,000 lb resulted in 120 payload ton-miles per gallon. Reducing top design speed from 53.6 to 43.2 mph resulted in no change in fuel economy for the vehicle combinations tested. Changing the rated engine output from 280 to 335 mph or to 380 hp did not result in any change in fuel economy for the same GVW.

Fuel economy is also a function of driver characteristics, which are not included in the foregoing model.

DETERMINATION OF PHYSICAL EFFECTS ON PAVEMENTS

In view of the many different methods for pavement design and evaluation being used in various states, the formulation of a single set of procedures that could be used by every highway organization in determining the effects of changes on pavements would be improbable. Instead, efforts were restricted to compiling procedures for pavement design and evaluation that would serve as a guide or as a convenient reference. In the application of these procedures, modifications of some steps and substitutions of certain methods by others may be necessary in individual states. As discussed later herein, the evaluation of the physical effects on pavements is primarily for determining the incremental costs related to pavements due to changes in legal vehicle weights and dimensions.

Design of Pavements

Although the effects of changes should be determined for both new and existing pavements, the method for such studies is based mainly on the principles for the design of new pavements. For this reason, pavement design is discussed in sufficient detail to permit a general appraisal of the methods of analysis.

Design of Flexible Pavements

Because of the complexity of many variables affecting pavement performance, the design of flexible pavements has been conducted on an empirical or semi-empirical basis. In recent years, individuals and organizations have tried to establish pavement design procedures based on some rational criteria. Burmister (*45*), Peattie (*46*), and Schiffman (*47*) reported the application of layered elastic systems for the design and evaluation of flexible pavements. The use of three-layer viscoelastic systems for the analyses of stresses and displacements in a pavement was reported by Elliott and Moavenzadeh (*48*). A significant step toward achieving a rational design procedure was the development of a method for predicting pavement deflection according to laboratory test data, as reported by Seed and others (*49*). The advances in analytical studies of pavement design, the use of a systems approach for the design of pavement structures, and other contributions toward the refinement of pavement design procedures were discussed in 1970 in a workshop sponsored by FHWA in cooperation with the University of Texas (*50*). In spite of recent progress, an analytical or rational method for pavement design has not developed to the point of practical application in this study.

The AASHO Interim Guide procedures for the design of flexible pavement structures (*127*) were developed primarily on the basis of statistical analyses of results obtained from the AASHO Road Test (*51*). The Road Test was conducted at a site having a specific type of subgrade soil and under certain environmental conditions.* Obviously, the Road Test findings may not be applicable to pavements in areas where subgrade soils and environmental factors differ from those at the test site. To account for these effects, the evaluation of the "soil support value" and the selection of a "regional factor" are included in the Interim Guide design procedures. As indicated in the Interim Guide, specific procedures for determining the soil support values of various types of subgrade materials and

* Research investigations related to environmental factors such as the moisture conditions of subgrade soils have been conducted by many individuals and organizations, including the British Road Research Laboratory (*52*).

the criteria for selecting regional factors are to be developed by individual states. Investigations for these purposes were conducted in a number of states such as Massachusetts (*53*), South Carolina (*54*), and Virginia (*55*).

Since completion of the AASHO Road Test in 1960, various states have undertaken efforts to extend the application of the Road Test results. These efforts were directed toward research programs such as the satellite studies of pavement performance (*56, 57*) and investigations related to the basic properties of pavement components and the behavior of flexible pavements. The nature and effect of dynamic pavement loads have also been studied by various investigators. Results of the satellite studies and other investigations have been published in many reports (e.g., *58-68*). Findings from these and future investigations will permit refinements in the Interim Guide procedures for applications in individual states in particular, and for pavement design and evaluation in general.

Design of Rigid Pavements

Insofar as design principles are concerned, the analysis of certain factors related to rigid pavements may be conducted on a more rational basis than in the case of flexible pavements. Nevertheless, similar to flexible pavements, the design of rigid pavements in current practice is often carried out by empirical or semi-empirical procedures. Investigations of the performance of rigid pavements were included in the AASHO Road Test.

As with flexible pavements, the Interim Guide method for design of rigid pavements (*128*) was developed by statistical analysis of Road Test results.

Another method for design of rigid pavements was developed by the Portland Cement Association (PCA). The PCA method as reported in 1966 (*69*) was based on the Westergaard theory (*70*), the studies by Pickett (*71*), model and full-scale tests conducted by the PCA laboratories and other organizations (*72*), and the results from a number of road tests, including the Maryland Road Test (*73*) and the AASHO Road Test. Information concerning the performance of normally constructed pavements subjected to mixed traffic was also used.

In some respects, the design procedures in the PCA method are considered more refined than those in the Interim Guide method. The PCA method, however, also is based on necessary simplifications and assumptions, some of which are debatable. For example, although the effect of repeated loading and fatigue on the concrete pavement is included in the design analysis, the modulus of subgrade reaction is based on nonrepetitive static plate load tests. Furthermore, the modulus of subgrade reaction actually depends not only on the factors related to the subgrade, but also on other variables such as loading area and pavement thickness (*74*).

Selection of Design Methods

In selecting the method for pavement design, the following criteria were used:

1. In cognizance of the variations in current pavement design practice in various states, the selected methods should be suitable for adoption by as many highway agencies as possible. In this respect, information obtained from surveys of current engineering practice will be used to assist in the selection of the design methods.

2. The design procedures of the selected methods should be adequate for determining the physical effects that result from changes in legal vehicle limits. Furthermore, the procedures must be flexible and readily adaptable to regional conditions and prevailing engineering practice in various states.

3. Although it is desirable to select methods that represent the most advanced and refined procedures for pavement design and evaluation, time and effort required to apply the methods must be reasonable. A preferred method for pavement design for actual construction may require excessive time, so that it may not be suitable for providing approximate estimates in incremental cost analysis.

The AASHO Interim Guide methods for the design of pavement structures (*127, 128*)* were found to be accepted by many highway agencies. According to the results of a survey conducted in connection with the evaluation study (*129*), 32 of the 52 highway agencies surveyed made direct use of the Interim Guides, either in their entirety or with some modification. Information obtained from a survey of the current pavement design methods conducted in this study indicates that the Interim Guide methods are most commonly used by state highway departments. For this reason and based on considerations of the second and third criteria described above, the Interim Guide methods were selected for this study. The specific procedures of the Interim Guide methods are discussed herein, and the applications of these methods for determining the effects of changes are shown in a numerical example of Appendix C.

The methods selected for this study may not be the best for all highway agencies. Other methods, such as the California method, may be preferred by certain states for the required analysis. The California method for design of flexible pavements was based on many years of laboratory and field investigations. It is particularly suited to that state's environmental and soil conditions. In the California method, the supporting power of a subgrade soil is evaluated in terms of a "resistance value" determined by the Hveem stabilometer. This method also has been used in a number of states other than California. Another method for design of flexible pavements was developed by The Asphalt Institute. Its design procedures were based on information from the AASHO Road Test, the WASHO Road Test, the British Test Roads, and tests conducted by various state highway agencies. Its general approach is similar to the Interim Guide method for flexible pavements; however, it is not as widely accepted by highway organizations.

For design of rigid pavements, the PCA method is frequently used by highway agencies. It is based on a design concept involving the fatigue failure of the concrete slab

* The Interim Guides were reviewed in "Evaluation of AASHO Interim Guides for Design of Pavement Structures" (*129*). The first draft of "AASHO Interim Guide for Design of Pavement Structures, 1972" (*130*) was available during this study.

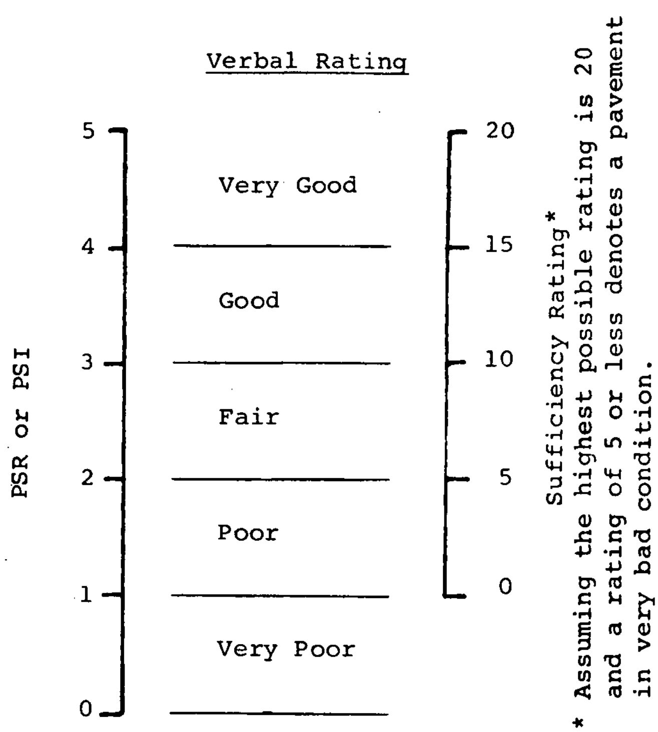


Figure 16. Correlation of verbal ratings with numerical values denoting pavement conditions.

due to repeated axle load applications. Although the procedures used are believed to reflect some refinements in pavement design, in comparison with other methods, the PCA method was not selected for this study. Its application for determining the effects on pavements would require excessive time, in comparison with the use of the Interim Guide method.

AASHO Interim Guide Methods

In the Interim Guide methods for design of either flexible or rigid pavements (*127, 128*), similar design concepts are employed insofar as the evaluation of pavement conditions and the analysis of load factors are concerned. These design concepts are reviewed in the following, before specific design procedures are presented.

Evaluation of Pavement Conditions.—The pavement serviceability-performance concept developed from the AASHO Road Test was adopted in the Interim Guide methods. This concept was evolved on the principle that the primary function of a pavement is to serve the traveling public in providing riding quality. The Interim Guide gives the following definitions for the technical terms related to pavement performance:

Serviceability—the ability at time of observation of a pavement to serve high-speed, high-volume automobile and truck traffic.

Serviceability rating—the mean value of the independent subjective ratings by members of a special panel for the AASHO Road Test as to the serviceability of a section of highway. The members of the panel included highway specialists representing many fields of interest and concern in highways.

Serviceability index—a number derived by formula for estimating the serviceability rating from measurements of certain physical features of the pavement.

Adjectives such as "initial," "present," and "terminal" are often used to describe the serviceability rating or index at a specific time in relation to the service life of a pavement. Figure 16 shows the correlation between verbal rating and the numerical values assigned to the present serviceability rating (PSR) or present serviceability index (PSI). The figure includes a correlation with the sufficiency rating of pavements, as suggested by Corvi and Bullard (*75*).

The serviceability index of a pavement, as defined in the Interim Guides, normally is determined by measurements of certain physical features of the pavement. The specific measurements and the formulas for computing the present serviceability index of pavements were developed during the AASHO Road Test.

The formula for rigid pavements (*51*) is:

$$PSI = 5.41 - 1.78 \log(1 + \overline{SV}) - 0.09\sqrt{C + P}$$

in which

C = length of substantial cracking, whether sealed or not, and is expressed in linear feet of projected length per 1,000 sq ft of pavement area;

P = area of patches per 1,000 sq ft of pavement area; and

$\overline{SV}$ = slope variance measured by a profilometer.

The formula for flexible pavements (*51*) is:

$$PSI = 5.03 - 1.91 \log(1 + \overline{SV}) - 1.38(\overline{RD})^2 - 0.01\sqrt{C + P}$$

in which

$\overline{SV}$ = the mean of slope variance in the two wheelpaths;

$\overline{RD}$ = average rut depth, inches; and

$C + P$ = a measure of cracking and patching in the pavement surface.

Recent research provides quantitative comparisons between the method for determining PSI described here and other procedures of pavement condition evaluation. The serviceability indices of new pavements are normally above 4.0. In the AASHO Road Test, the initial serviceability index (just after construction) of flexible pavements was 4.2; that of rigid pavements was 4.5. When the serviceability index of either flexible or rigid pavements reached a terminal value of 2.0 to 2.5, resurfacing or replacement of the pavement was required.

Equivalent Axle Load Concept.—In the Interim Guide methods for pavement design, the load factor is expressed in terms of a common denominator: namely, the 18-kip single-axle load. The mixed traffic on a highway may be converted to equivalent 18-kip single-axle load applications by using the equivalence factors given in the Interim Guides. This approach, as described by Scrivner and Duzan (*76*), is based on the following assumptions:

1. The effect of any axle load can be expressed in terms of an equivalent number of applications of any selected

load. For convenience, the selected axle load is designated axle load a.

2. The combined effect of all axle loads in mixed traffic can be expressed in terms of the combined number of equivalent applications of axle load a.

3. The effect of mixed traffic can be introduced into a single load equation for load a by means of the combined number of equivalent applications of axle load a.

The analysis reported by Scrivner and Duzan (*76*) concerning equivalence factors is described briefly in the following.

If R_i denotes the ratio of equivalence that will convert each application of axle load i to an equivalent number of applications of each load a, then

$$R_i = \frac{W_a}{W_i} = \frac{\rho_a \, g^{1/\beta_a}}{\rho_i \, g^{1/\beta_i}}$$

in which

ρ_a or ρ_i = number of axle load applications to reduce the serviceability index to a value of 1.5;

g = a function of axle loads, number of axles, and design parameters related to pavement thickness; and

β_a or β_i = a function of design and load variables.

If W is the total number of axle applications in mixed traffic, and C_i is the portion of total applications of weight i, then the number of applications of axle load i is $C_i W$. The number of applications of each axle load is converted to an equivalent number of applications of axle load a (in practice, $a = 18$ kips) by means of the appropriate ratio of equivalence. The sum of the products is the equivalent number of applications of axle load a in mixed traffic. Thus:

$$W_a = W \sum_{i=1}^{k} C_i \, R_i$$

in which k denotes the categories, each containing axles of the same type (single or tandem) and weight. The substitution of this expression for W in the general AASHO Road Test equation, together with the analyses of the effects related to pavement thickness and the terminal serviceability index, will permit formulation of the desired equivalence factors. From their study, Scrivner and Duzan concluded that the difference between values obtained by the "equivalent applications approach" and a "mixed traffic theory" would be negligible for all except relatively thin flexible pavements.

To facilitate the conversion of mixed traffic to equivalent 18-kip single-axle load applications, the AASHO Interim Guide provides data on equivalence factors (see Table 28). In conducting approximate analysis, the required computations may be simplified by using the average equivalence factors given in Table 29.

The effect on pavements of an increase in axle loads may be evaluated by comparing the equivalence factors for the respective axle loads. Table 29 indicates that the equivalence factor for a 32-kip tandem-axle load is 0.83, if the structural number is 2. If the tandem-axle load is increased by 50 percent to 48 kips, the equivalence factor would be increased by approximately 600 percent to 4.98. The same trend of variation in the equivalence factors with respect to changes in axle load is indicated in Table 29. This example illustrates the important effect of changes in axle weights on the equivalent 18-kip single-axle load applications.

The equivalence factors given in the Interim Guides, however, were determined essentially on the basis of AASHO Road Test data. Potential changes in legal vehicle weights and dimensions may introduce a mixed traffic involving axle and wheel loads beyond the range of validity of the equivalence factors in the Interim Guides. Although theoretical analysis may be used to evaluate the influence on equivalence factors of substantial changes in vehicle limits, as well as modifications in wheel and axle configurations, further research is required to provide factual data on how these variables affect pavement performance.

Design of Flexible Pavements.—The Interim Guide method for flexible pavement design is based on the following AASHO Road Test equation, with necessary modifications to include a subgrade soil scale and a regional factor.

$$G_t = \log\left(\frac{c_o - p_t}{c_o - 1.5}\right) = \beta(\log W_t - \log \rho)$$

in which

G_t = a function (the logarithm) of the ratio of loss in serviceability at time t to the total potential loss taken to the point where $p = 1.5$; *

c_o = initial serviceability of pavement (equal to 4.2 on test road);

p_t = serviceability at end of time t;

β = a function of design and load variables that influences the shape of the p versus W_t serviceability curve;

W_t = weighted traffic factor; and

ρ = a function of design and load variables that denotes the expected number of axle load applications to a serviceability index of 1.5.*

To expedite the application of this method for the design of flexible pavements, the Interim Guide provides design charts with terminal serviceability indices of 2.0 and 2.5. Figure 17 shows a typical chart. It is necessary to obtain information concerning (1) soil support values, (2) equivalent 18-kip single-axle load applications, and (3) regional factor. The determination of the equivalent 18-kip single-axle load applications is discussed in the previous section. Procedures for determining the other factors follow.

The soil support value, S, is expressed in an abstract scale that can be related to certain laboratory or field test procedures now in use or to be developed. (Subgrade soils at the AASHO Road Test site have a soil support value of 3.0.) Several individual states have developed specific procedures for determining the soil support value. Figure 18 shows an Interim Guide chart that permits approximate correlation between the soil support value and the CBR, R-value, or Group Index. The conditions that apply to the correlation scales shown are:

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* The point at which pavement sections were removed from test in the AASHO Road Test.

TABLE 28

EQUIVALENCE FACTORS FOR THE DESIGN OF FLEXIBLE PAVEMENT STRUCTURES, AASHO INTERIM GUIDE

Serviceability index expected to be reached at the end of design period: 2.0.

TANDEM AXLE LOAD (KIPS)	STRUCTURAL NUMBER—SN 1	2	3	4	5	6
10	0.01	0.01	0.01	0.01	0.01	0.01
11	0.01	0.01	0.01	0.01	0.01	0.01
12	0.01	0.02	0.02	0.01	0.01	0.01
13	0.02	0.02	0.02	0.02	0.02	0.02
14	0.02	0.03	0.03	0.03	0.02	0.02
15	0.03	0.04	0.04	0.03	0.03	0.03
16	0.04	0.05	0.05	0.05	0.04	0.04
17	0.05	0.06	0.06	0.06	0.05	0.05
18	0.07	0.08	0.08	0.07	0.07	0.07
19	0.08	0.09	0.10	0.09	0.09	0.08
20	0.10	0.12	0.12	0.12	0.11	0.10
21	0.13	0.14	0.15	0.14	0.13	0.13
22	0.16	0.17	0.18	0.17	0.16	0.16
23	0.19	0.20	0.22	0.21	0.20	0.19
24	0.23	0.24	0.26	0.25	0.24	0.23
25	0.27	0.29	0.31	0.30	0.29	0.28
26	0.32	0.34	0.36	0.35	0.34	0.33
27	0.38	0.40	0.42	0.41	0.40	0.39
28	0.45	0.46	0.49	0.48	0.47	0.46
29	0.52	0.54	0.56	0.56	0.54	0.53
30	0.61	0.62	0.65	0.64	0.63	0.62
31	0.70	0.72	0.74	0.74	0.72	0.71
32	0.81	0.82	0.84	0.84	0.83	0.82
33	0.93	0.94	0.96	0.96	0.95	0.94
34	1.06	1.07	1.08	1.08	1.08	1.07
35	1.21	1.22	1.22	1.22	1.22	1.22
36	1.38	1.38	1.38	1.38	1.38	1.38
37	1.56	1.55	1.54	1.54	1.55	1.55
38	1.76	1.75	1.73	1.72	1.73	1.74
39	1.98	1.96	1.93	1.92	1.94	1.95
40	2.22	2.19	2.15	2.13	2.16	2.18
41	2.48	2.45	2.39	2.37	2.40	2.43
42	2.77	2.73	2.64	2.62	2.66	2.70
43	3.08	3.03	2.93	2.89	2.94	3.00
44	3.42	3.36	3.23	3.18	3.24	3.31
45	3.80	3.72	3.56	3.49	3.56	3.65
46	4.20	4.11	3.92	3.83	3.91	4.02
47	4.63	4.53	4.30	4.19	4.28	4.41
48	5.10	4.98	4.72	4.58	4.68	4.83

Source: AASHO (*127*, p. F4).

1. *Scale A:* *R*-value (California)—The correlation is with the design curves used by California using AASHO Test Designation T-173. The correlation is for a 240-psi exudation pressure (*77*).

2. *Scale B:* *R*-value (Washington)—The correlation is with the design curves used by Washington. The correlation is for a 300-psi exudation pressure (*78*).

3. *Scale C:* CBR (Kentucky)—The correlation is with the CBR design curves developed by Kentucky (*79*). The following conditions apply to the test procedure:

(a) The test specimen is molded at or near the optimum moisture content as determined by AASHO Test Designation T-99.

(b) Dynamic compaction is used in preparing the test specimen, with a hammer weighing 10 lb dropped from a height of 18 in.

(c) The test specimen is compacted in five equal layers, with each layer receiving 10 blows from the hammer.

(d) The test specimen is soaked for 4 days.

This is valid for crushed-rock base courses, using the Kentucky CBR curves.

4. *Scale D:* CBR (Kentucky)—The correlation is with the Kentucky CBR design curves when bituminous-stabilized base courses are used. The preceding conditions apply, except that the test specimen is compacted in five equal layers, with each layer receiving 25 blows with the hammer.

5. *Scale E:* Group Index—This scale has been established by comparative testing between the *R*-value as run by California and the Group Index (*80*). It does not agree

with cooperative testing performed in connection with the AASHO Road Test.

The regional factor (R) is provided in the design charts to allow for an adjustment in the structural number because of climatic and environmental conditions. The Interim Guide gives a method of approximating the regional factor.

Figure 19 shows an example for applying the Interim Guide method for pavement design. In this example, for a soil support value of 6.0 and the expected traffic of 45 equivalent daily 18-kip single-axle load applications during the 20-year analysis period, a structural number (SN) of 2.1 is required. With a regional factor of 3.0, the original SN of 2.1 is increased to a weighted SN of 2.5. The required thickness of the pavement structure can then be determined according to this weighted SN together with the coefficients of relative strength of paving materials. Table 30 gives the coefficients given in the Interim Guide. The determination of the thickness of each pavement component is illustrated in the numerical example in Appendix C.

Design of Rigid Pavements.—The Interim Guide method for rigid pavement design is based on the AASHO Road Test equation similar to that for the design of flexible pavements. The general procedure for design of rigid pavements is, therefore, essentially the same as that just described. Design charts for terminal serviceability indices of 2.0 and 2.5 appear in the Interim Guide; Figure 19 shows one of them. As the design chart shows, traffic data are also expressed in terms of equivalent 18-kip single-axle load applications. However, the supporting power of subgrade is evaluated by a method different from that used in the design of flexible pavements. It is usually determined by field plate bearing tests and expressed as a modulus of subgrade reaction. Although no regional factor is included in the design chart, the effect of variations in climatic and environmental conditions on subgrade support power may be accounted for in the evaluation of the modulus of subgrade reaction for the project area.

The use of the design chart for determining the required thickness of a rigid pavement is illustrated by the dashed lines. For the traffic, working stress in concrete, and modulus of subgrade reaction indicated by the dashed lines, the required slab thickness is 10 in.

Appraisal of AASHO Design Methods

The AASHO Interim Guide methods for the design of pavement structures were based primarily on the findings of the AASHO Road Test. These methods are subject to certain limitations or errors, especially in the application for pavement design and evaluation under physical and environmental conditions that differ from those of the Road Test. Because many variables affect pavement performance, it is difficult to predict the probable errors on a percentage basis. A general appraisal of the reliability or limitations of the Interim Guide design methods follows.

In the AASHO Road Test, the pavements were subjected to test traffic for approximately two years. Because of the relatively short time in which pavement performance was evaluated, the influence of environmental factors may not be fully developed. For this reason, the Road Test was considered by some investigators as a traffic-dominant project. In view of the substantial difference between the normal service life of a highway pavement and the short time the Road Test was conducted, the thickness of pavements determined by the Interim Guide method may not

TABLE 29

AVERAGE EQUIVALENCE FACTORS FOR THE DESIGN OF FLEXIBLE PAVEMENT STRUCTURES, AASHO INTERIM GUIDE

Serviceability index expected to be reached at the end of design period: 2.0.

AXLE LOAD (LB)	EQUIVALENCE FACTORS: SINGLE AXLES	EQUIVALENCE FACTORS: TANDEM AXLES
2000–8000	0.006	—
8000–16000	0.18	0.02
16000–20000	1.00	0.08
20000–24000	2.35	0.17
24000–30000	5.80	0.42
30000–34000	12.00	0.83
34000–38000	20.00	1.38
38000–44000	33.00	2.40
44000–48000	—	3.90
Pass. cars	0.0002	

Source: AASHO (*127*, p. 19a).

TABLE 30

COEFFICIENTS OF RELATIVE STRENGTH OF PAVEMENT COMPONENTS

PAVEMENT COMPONENT	COEFFICIENT [c] a_1	a_2	a_3
Surface course:			
Roadmix (low stability)	0.20		
Plantmix (high stability)	0.44 *		
Sand asphalt	0.40		
Base course:			
Sandy gravel		0.07 [b]	
Crushed stone		0.14 *	
Cement treated (no-soil-cement):			
650 psi or more [a]		0.23 [b]	
400 psi to 650 psi		0.20	
400 psi or less		0.15	
Bituminous treated:			
Coarse graded		0.34 [b]	
Sand asphalt		0.30	
Lime treated		0.15–0.30	
Subbase:			
Sandy gravel			0.11 *
Sand or sandy-clay			0.05–0.10

[a] Compressive strength at 7 days.
[b] This value has been estimated from AASHO Road Test data, but not to the accuracy of those factors marked with an asterisk.
[c] It is expected that each State will study these coefficients and make such changes as their experience indicates necessary.
Source: AASHO (*127*, p. 22).

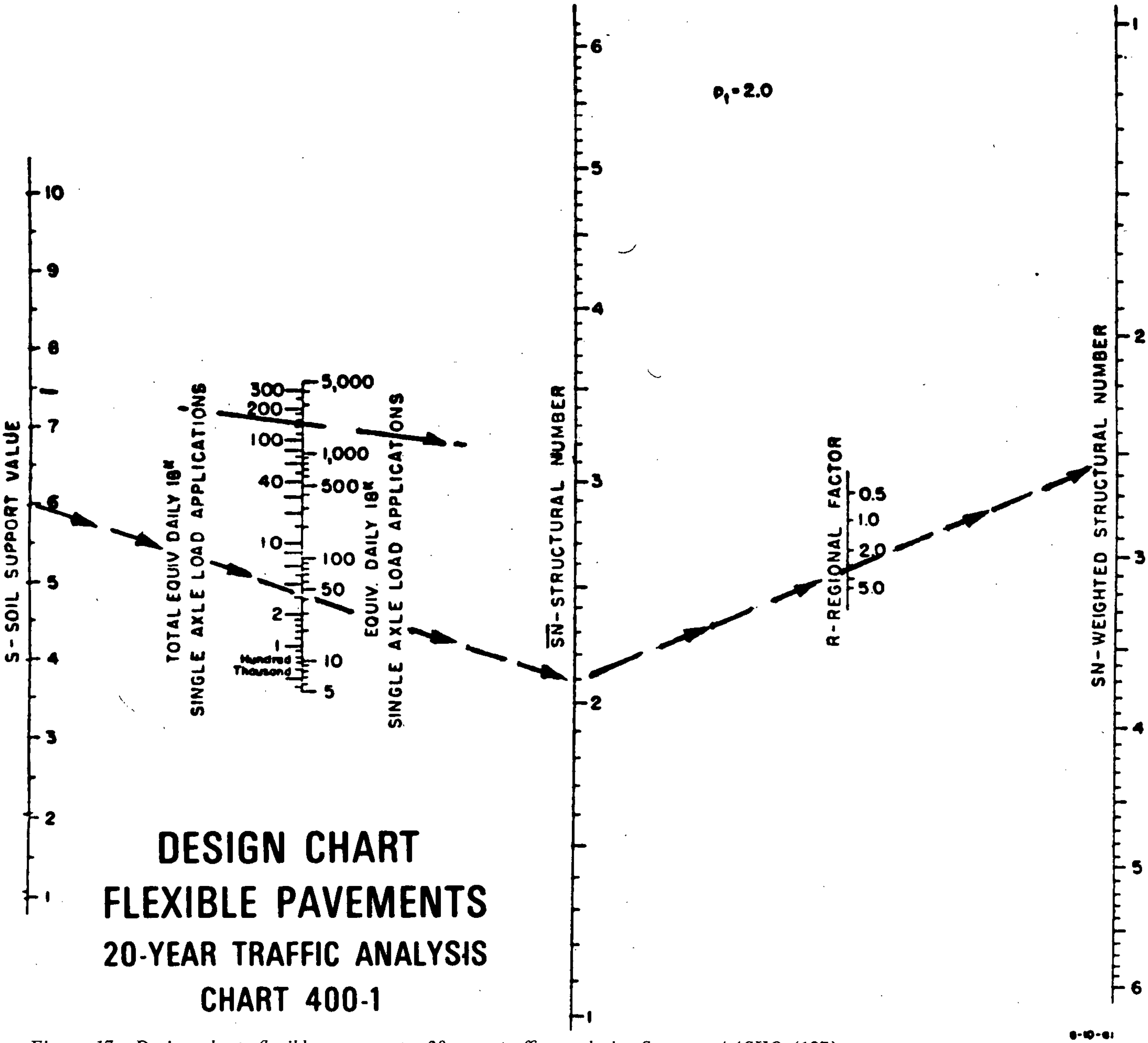


Figure 17. Design chart, flexible pavements, 20-year traffic analysis. Source: AASHO (127).

be adequate for highways carrying relatively light traffic and located in areas where unfavorable environmental conditions are likely to have adverse effects on pavement performance.

The planning of the AASHO Road Test in regard to the composition, distribution, and speed of test traffic, and the formulation of the load equivalence factors presented in the Interim Guide, are controversial subjects. For example, the American Trucking Associations (*81*) made the following criticism:

> As stated earlier, the trucking industry, along with many other groups, assumes that the current AASHO Road Test will be helpful in providing reasonable answers to some of the problems involved in highway engineering and construction. Translation of the test results into conclusions as to cost responsibility requires, however, extreme care and recognition of certain important limitations of the results. For example, the trucking industry believes the test will not fully indicate what is required in the way of a minimum highway facility. All of the conditions necessary for testing such a facility are not being met in the so-called light-axle loop. The industry has specific reference to the lack of sufficient time fully to reflect the bearing of climatic conditions and to the absence of traffic comparable in volume and speed with that anticipated in many instances of highway use.

The time available to measure the influence of climatic conditions in the Road Test was insufficient. In addition, Road Test operations may not have been extensive enough

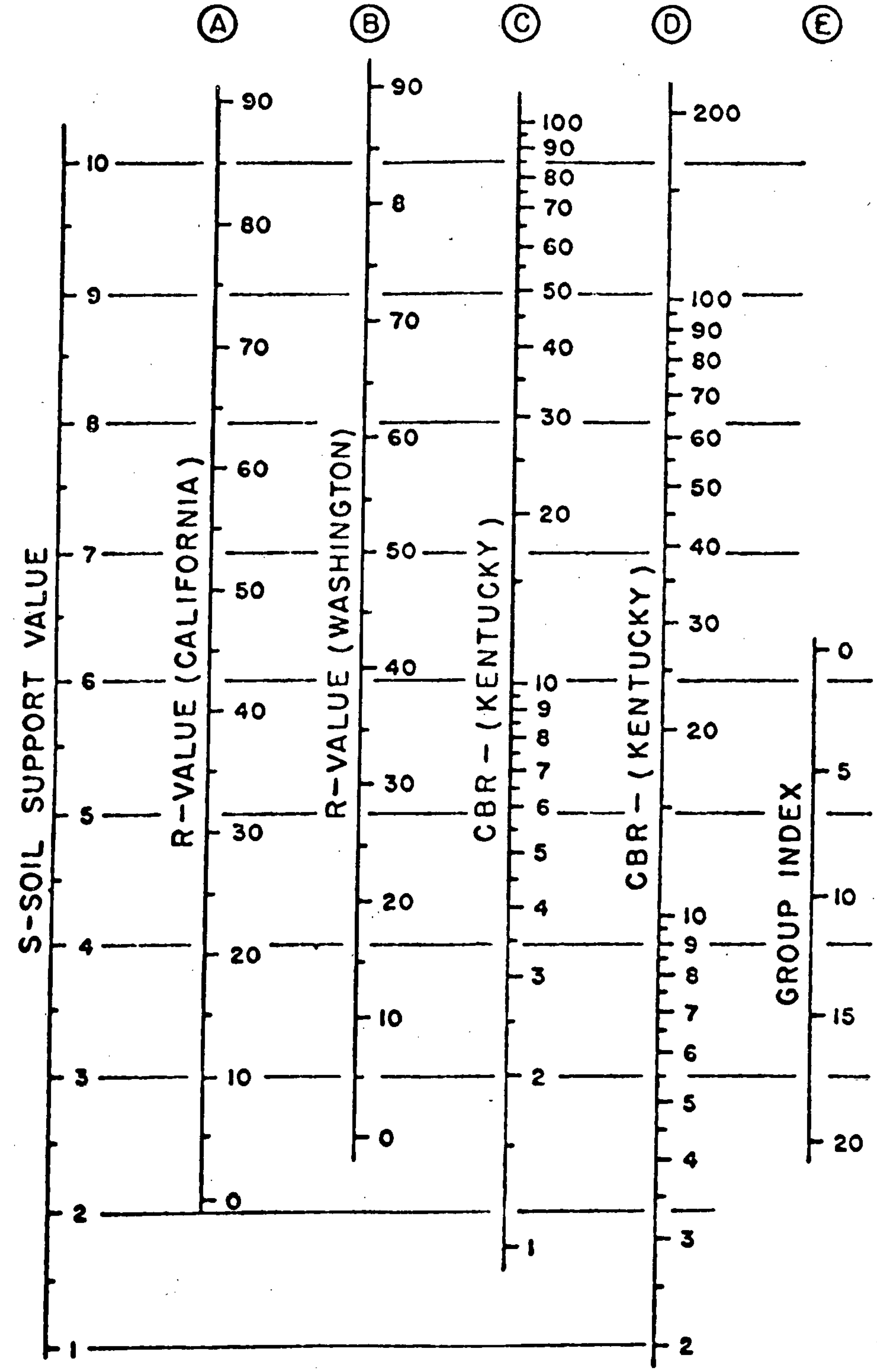


Figure 18. Correlation chart for estimating soil support value. Source: AASHO (127).

to represent all combinations of traffic pertinent to pavement design and evaluation. The time and funds available for road tests usually are limited, and the actual investigations may not be carried out to the desired extent. In formulating the AASHO equivalence factors, therefore, interpolations and extrapolations of the Road Test results are required to provide sufficient information for the entire range of wheel or axle loads expected on highways. Consequently, some errors necessarily will be involved when the equivalence factors are applied in pavement design and evaluation. These errors could be minimized if an analytical or rational method is employed. Such a method is

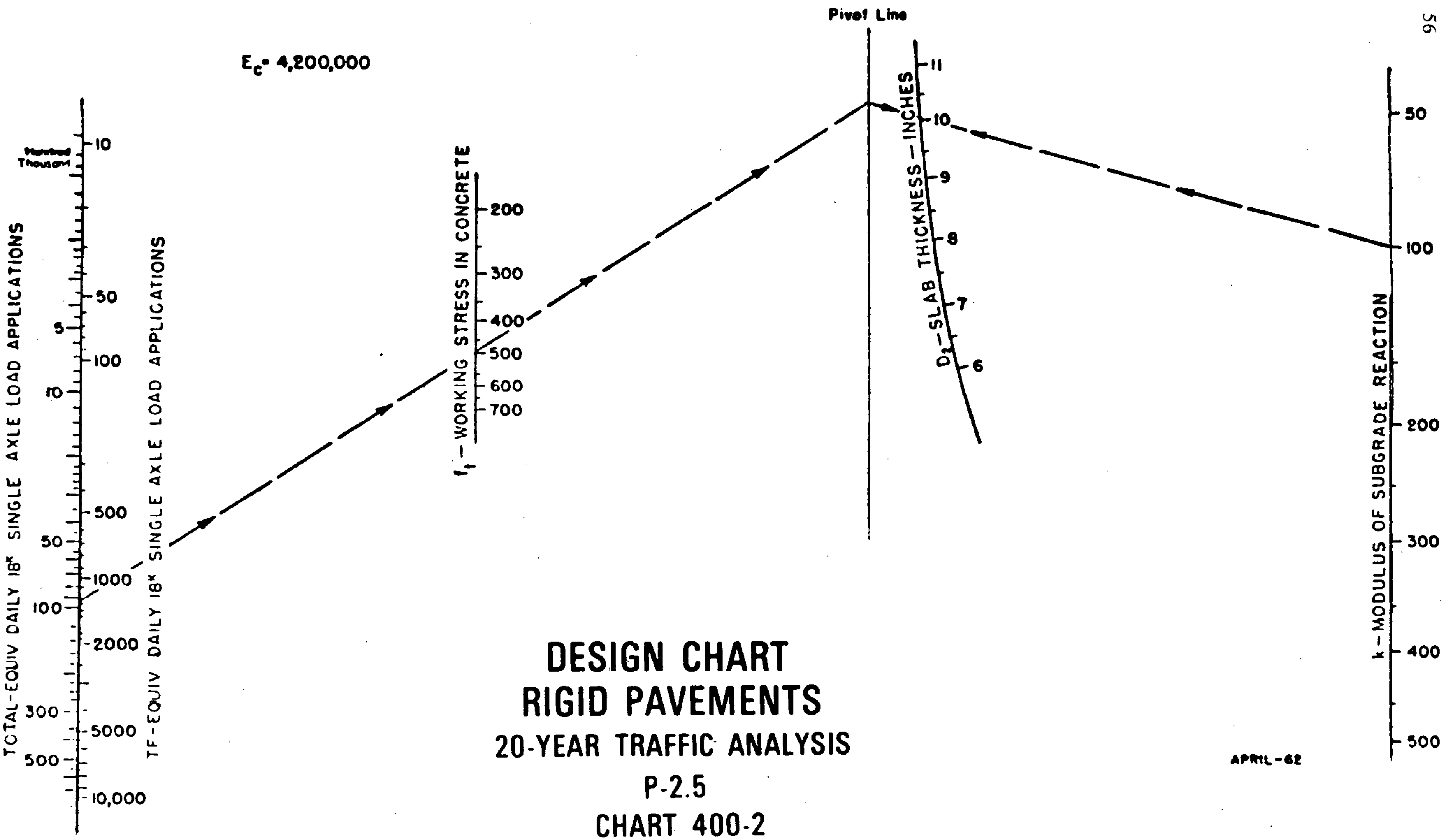


Figure 19. Design chart, rigid pavements, 20-year traffic analysis. Source: AASHO (128).

urgently needed, especially for the investigation of the effects on pavement costs due to increases in legal vehicle limits.

A report of the International Road Transport Union (*82*) emphasized that the effects of changes in legal vehicle weights should be studied by "scientific" analyses instead of by using the AASHO equivalence factors. Nevertheless, as discussed herein and pointed out in the Union report, the development of an analytical or a rational method is currently at such a stage that its application in engineering practice would be impracticable.

Regardless of these limitations concerning the findings of the AASHO Road Test, the Interim Guides' equivalence factors have been widely used by highway agencies (*129*). This is due apparently to the general belief that, despite certain limitations in the accuracy or validity of these factors, their application in pavement design represents a reasonable approach in engineering practice and an improvement of methods used previously for evaluating the effect of traffic on pavement structures.

Design of Overlays

Flexible overlays usually are constructed over either flexible or rigid pavements. The required thickness of overlays above flexible pavements may be determined by a procedure similar to that for the design of flexible pavements for new construction, except that the coefficients of relative strength of the layers in the existing pavement structure should be reexamined and revised, if necessary, to account for the deterioration of the compacted paving materials in the period that the pavement has been opened to traffic. Flexible overlays above rigid pavements may be designed by similar procedures.

In recent years, extensive investigations related to the design of overlays and pavement rehabilitation have been conducted in various states. Their findings led to the development of more refined procedures for the design of overlays (*83, 84*). One approach is to perform deflection measurements of the existing pavement and then determine the required overlay thickness on the basis of established criteria regarding the permissible deflection of the resurfaced pavement. Although this approach appears to be preferable for the design of overlays in actual construction, its application for determining the effect of changes in statewide incremental cost analysis is debatable because of the excessive time and effort required. In the numerical example in Appendix C the effect of changes on overlay design is illustrated by the application of the Interim Guide method for the design of flexible pavement structures.

Estimating Remaining Service Life of Existing Pavements

The service life of a pavement is related to traffic, pavement structure, subgrade materials, and environmental conditions. Insofar as traffic is concerned, it is usually represented by the number of equivalent 18-kip single-axle load applications. Any modification in legal vehicle limits will result in a change in the equivalent 18-kip single-axle load applications. Consequently, the effect on pavements of modifications in legal limits may be evaluated on the basis of the corresponding changes in equivalent load applications.

Several methods have been developed to estimate the service life of existing pavements. One involves the application of "survivor curves"—graphic representations of the relationship between the age of different types of pavement and the percentage surviving. This method may be applied in a state where sufficient data are available for analysis. Its sensitivity in regard to the evaluation of the effect of changes would depend on the nature of available data. Results from a survey conducted in this project indicate that many states do not have sufficient information to permit the use of this method.

"Manual B" of the *National Transportation Planning Manual* (*36*) provides a procedure for estimating the remaining service life of a group of existing pavements: for example, those of all primary highways in a county. Application of this method requires the following information:

1. Present pavement condition (PSR, PSI, or equivalent).
2. Pavement structure or thickness.
3. Soil support value (S) in the case of flexible pavements.
4. Number of present equivalent annual 18-kip single-axle load applications (EALA).
5. Average annual rate of traffic growth.

From this information, together with a prescribed minimum tolerable condition of the pavements, a range of the remaining service life can be estimated. "Manual B" provides tables for estimating the remaining service life of flexible or rigid pavements with a minimum tolerable pavement condition of PSR = 2.1 or 2.6. Tables 31 and 32 (from "Manual B") are for PSR = 2.1. The remaining service life of flexible pavements also depends on the soil support value. To account for this influence the pavement structure value should be adjusted, if necessary, according to the data given in Table 33 (from "Manual B"). Application of the data in the tables is illustrated in Appendix C.

The use of the foregoing procedure provides very approximate estimates of remaining service life as required in transportation planning studies. For a more precise analysis of a specific pavement section it is necessary to use other methods, such as the one developed by Corvi and Bullard (*75*). This method is based primarily on the findings from the AASHO Road Test and the procedures in the AASHO Interim Guides. The remaining service life of a pavement section is estimated by using nomographs relating the traffic factor with other variables affecting pavement performance. Figures 20, 21, 22, and 23, show 4 of the 23 nomographs from the report by Corvi and Bullard (*75*). The application of the nomographs is illustrated in Appendix C.

Although the term "soil support value" is used in both the Interim Guide method for the design of flexible pavements (see Fig. 17) and the method reported by Corvi and Bullard for estimating the remaining service life of existing flexible pavements, there are significant differences between the applications of this term in the two methods. These differences are explained by Corvi and Bullard:

TABLE 31

RIGID PAVEMENT—REMAINING SERVICE LIFE (MINIMUM TOLERABLE CONDITION—PSR = 2.1)

Pavement thickness	Years of remaining life	Annual traffic growth								
		1 to 3 percent			4 to 6 percent			7 percent and over		
		Pavement condition			Pavement condition			Pavement condition		
		Very good	Good	Fair	Very good	Good	Fair	Very good	Good	Fair
Present equivalent annual 18-kip single-axle load applications (EALA)										
Light (D=6.0-7.0)	Over 20	Less than 17,999	Less than 10,999	Less than 3,999	Less than 12,999	Less than 7,999	Less than 2,999	Less than 8,999	Less than 4,999	Less than 1,999
	16-20	18,000 to 26,999	11,000 to 14,999	4,000 to 4,999	13,000 to 20,999	8,000 to 11,999	3,000 to 3,999	9,000 to 15,999	5,000 to 8,999	2,000 to 2,999
	11-15	27,000 to 42,999	15,000 to 24,999	5,000 to 7,999	21,000 to 36,999	12,000 to 21,999	4,000 to 6,999	16,000 to 30,999	9,000 to 17,999	3,000 to 5,999
	6-10	43,000 to 92,999	25,000 to 53,999	8,000 to 17,999	37,000 to 87,999	22,000 to 50,999	7,000 to 16,999	31,000 to 80,999	18,000 to 46,999	6,000 to 14,999
	1-5	93,000 or more	54,000 or more	18,000 or more	88,000 or more	51,000 or more	17,000 or more	81,000 or more	47,000 or more	15,000 or more
Medium (D=7.1-9.0)	Over 20	Less than 103,999	Less than 70,999	Less than 21,999	Less than 75,999	Less than 51,999	Less than 15,999	Less than 48,999	Less than 32,999	Less than 9,999
	16-20	104,000 to 150,999	71,000 to 101,999	22,000 to 31,999	76,000 to 119,999	52,000 to 81,999	16,000 to 25,999	49,000 to 87,999	33,000 to 59,999	10,000 to 18,999
	11-15	151,000 to 242,999	102,000 to 164,999	32,000 to 51,999	120,000 to 211,999	82,000 to 143,999	26,000 to 44,999	88,000 to 174,999	60,000 to 118,999	19,000 to 37,999
	6-10	243,000 to 527,999	165,000 to 357,999	52,000 to 112,999	212,000 to 499,999	144,000 to 338,999	45,000 to 106,999	175,000 to 458,999	119,000 to 310,999	38,000 to 97,999
	1-5	528,000 or more	358,000 or more	113,000 or more	500,000 or more	339,000 or more	107,000 or more	459,000 or more	311,000 or more	98,000 or more
Heavy (D=9.1-11.0)	Over 20	Less than 571,999	Less than 382,999	Less than 125,999	Less than 417,999	Less than 279,999	Less than 91,999	Less than 268,999	Less than 179,999	Less than 58,999
	16-20	572,000 to 827,999	383,000 to 553,999	126,000 to 182,999	418,000 to 660,999	280,000 to 441,999	92,000 to 145,999	269,000 to 483,999	180,000 to 323,999	59,000 to 106,999
	11-15	828,000 to 1,338,999	554,000 to 895,999	183,000 to 295,999	661,000 to 1,166,999	442,000 to 779,999	146,000 to 257,999	484,000 to 961,999	324,000 to 643,999	107,000 to 212,999
	6-10	1,339,000 to 2,905,999	896,000 to 1,942,999	296,000 to 641,999	1,167,000 to 2,749,999	780,000 to 1,838,999	258,000 to 606,999	962,000 to 2,524,999	644,000 to 1,687,999	213,000 to 556,999
	1-5	2,906,000 or more	1,943,000 or more	642,000 or more	2,750,000 or more	1,839,000 or more	607,000 or more	2,525,000 or more	1,688,000 or more	557,000 or more

Source: "Manual B" (36).

TABLE 32

FLEXIBLE PAVEMENT—REMAINING SERVICE LIFE (MINIMUM TOLERABLE CONDITION—PSR = 2.1)

Pavement structure	Years of remaining life	Annual traffic growth								
		1 to 3 percent			4 to 6 percent			7 percent and over		
		Pavement condition			Pavement condition			Pavement condition		
		Very good	Good	Fair	Very good	Good	Fair	Very good	Good	Fair
		Present equivalent annual 18-kip single-axle load applications (EALA)								
Light (SN=1.0-3.0)	Over 20	Less than 699	Less than 499	Less than 99	Less than 499	Less than 299	Less than 99	Less than 299	Less than 199	Less than 59
	16-20	700 to 999	500 to 699	100 to 199	500 to 899	300 to 499	100 to 199	300 to 599	200 to 399	60 to 99
	11-15	1,000 to 1,999	700 to 999	200 to 299	900 to 1,499	500 to 999	200 to 299	600 to 999	400 to 799	100 to 199
	6-10	2,000 to 3,999	1,000 to 1,999	300 to 699	1,500 to 3,999	1,000 to 1,999	300 to 599	1,000 to 2,999	800 to 1,999	200 to 599
	1-5	4,000 or more	2,000 or more	700 or more	4,000 or more	2,000 or more	600 or more	3,000 or more	2,000 or more	600 or more
Medium (SN=3.1-4.5)	Over 20	Less than 30,999	Less than 23,999	Less than 8,999	Less than 22,999	Less than 16,999	Less than 6,999	Less than 13,999	Less than 10,999	Less than 3,999
	16-20	31,000 to 44,999	24,000 to 33,999	9,000 to 12,999	23,000 to 35,999	17,000 to 26,999	7,000 to 9,999	14,000 to 25,999	11,000 to 19,999	4,000 to 7,999
	11-15	45,000 to 71,999	34,000 to 55,999	13,000 to 20,999	36,000 to 62,999	27,000 to 47,999	10,000 to 17,999	26,000 to 51,999	20,000 to 39,999	8,000 to 14,999
	6-10	72,000 to 156,999	56,000 to 120,999	21,000 to 44,999	63,000 to 147,999	48,000 to 113,999	18,000 to 42,999	52,000 to 135,999	40,000 to 104,999	15,000 to 38,999
	1-5	157,000 or more	121,000 or more	45,000 or more	148,000 or more	114,000 or more	43,000 or more	136,000 or more	105,000 or more	39,000 or more
Heavy (SN=4.6-6.0)	Over 20	Less than 356,999	Less than 311,999	Less than 151,999	Less than 260,999	Less than 227,999	Less than 110,999	Less than 167,999	Less than 146,999	Less than 71,999
	16-20	357,000 to 515,999	312,000 to 451,999	152,000 to 219,999	261,000 to 411,999	228,000 to 360,999	111,000 to 175,999	168,000 to 301,999	147,000 to 263,999	72,000 to 128,999
	11-15	516,000 to 834,999	452,000 to 729,999	220,000 to 356,999	412,000 to 726,999	361,000 to 635,999	176,000 to 310,999	302,000 to 599,999	264,000 to 524,999	129,000 to 255,999
	6-10	835,000 to 1,810,999	730,000 to 1,584,999	357,000 to 773,999	727,000 to 1,713,999	636,000 to 1,499,999	311,000 to 731,999	600,000 to 1,573,999	525,000 to 1,376,999	256,000 to 671,999
	1-5	1,811,000 or more	1,585,000 or more	774,000 or more	1,714,000 or more	1,500,000 or more	732,000 or more	1,574,000 or more	1,377,000 or more	672,000 or more

Source: "Manual B" (36).

TABLE 33

ADJUSTMENT TO PAVEMENT STRUCTURE VALUE TO ACCOUNT FOR DIFFERENCE IN SOIL VALUE

	PAVEMENT STRUCTURE VALUE		
SOIL SUPPORT VALUE	LIGHT SN=1.0–3.0	MEDIUM SN=3.1–4.5	HEAVY SN=4.6–6.0
1.5 or less	No change	Decrease to light	Decrease to medium
1.6–5.9	No change	No change	No change
6.0 or more	Increase to medium	Increase to heavy	No change

Source: "Manual B" (*36*).

In plotting the soil support scale for the flexible pavement nomographs, the authors followed the procedure recommended by the AASHO Committee on Design to obtain the first point, $S=3$, and the maximum point. However, they did not, as suggested, assign a value of 10 to the maximum point and assume a linear scale between points 3 and 10 and extend the scale to 1. Rather, they assigned a value of 110 to the maximum point and assumed a logarithmic scale between the value of 3 and 110 and extended the scale to 1.

A logarithmic scale, rather than the linear scale suggested by the AASHO Committee on Design, was used simply because good correlation between nomographs at the different serviceability levels could not be obtained with a linear scale.

The soil support values used in the Interim Guide method may be correlated with the data from laboratory or field tests on the subgrade materials. One such test is the method for determining the modulus of deformation of the subgrade soil, developed in South Carolina (*54*). For a number of years, this method has been used by the South Carolina State Highway Department for the design of flexible pavement structures according to the Interim Guide procedures. The correlation between the modulus of deformation and the soil support value in the Interim Guide is shown on the right side of Figure 24. The dashed lines show that for a modulus of deformation of 4,000 lb/sq in. the soil support value is 7. As the figure shows, this correlation is independent of the regional factor.

It is difficult to correlate the soil support value used in the method reported by Corvi and Bullard with the modulus of deformation determined by the South Carolina method. This is because, unlike the Interim Guide method, in the Corvi and Bullard method no regional factor is involved. For this reason, any attempted correlation will depend on the regional factor of the area where the pavement section is located. An approximate correlation, however, is shown on the left side of Figure 24. The dashed lines indicate that for a modulus of deformation of 4,000 lb/sq in. the soil support value is approximately 23. In applying the Corvi and Bullard method for estimating the remaining service life (see Appendix C), equivalent or hypothetical soil support values are used to represent the subgrade factor.

Computing Effects of Changes on Pavement Elements

Methods for the design of pavements and overlays and for estimating the remaining service life of existing pavements described previously permit the determination of the effects of changes in legal limits on costs related to pavements. Table 34 gives the required analyses for estimating these incremental pavement costs. The effect on pavement maintenance cost is discussed in the following.

Substantial changes in legal limits may affect the maintenance cost of existing pavements, especially if they were designed according to the original legal limits. How the changes influence pavement costs cannot be determined by methods comparable to those discussed previously. In a survey conducted in this study, many states replied that there had been some indication that pavement maintenance costs had been affected by changes in legal vehicle weight limits. Few states, however, have documented evidence in this respect. In essence, if the effect of changes on pavement maintenance cost is to be included in the cost analysis, it would have to be estimated primarily on the basis of experience and engineering judgment, unless traffic and pavement maintenance cost data related to changes in legal limits are available.

Figure 25 shows general procedures for determining the effects of legal axle weight changes in an incremental cost analysis. Details regarding the determination of the remaining service life of existing pavements and the design of pavements and overlays appear in Appendix C.

FORECASTING EXTENT OF LOAD APPLICATIONS ON HIGHWAYS UNDER NEW LIMITS

Traffic forecasts must be made for each class of vehicle that might be affected by vehicle dimension and weight limits. If it is assumed that the number of applications of equivalent 18,000-lb axles determines the pavement design, then a forecast of the traffic stream by class of vehicle according to axle arrangement is required. Further, it is necessary to determine the weight distribution on each of the axles. Thus, besides finding the ADT of each class of vehicle, the investigator must decide whether a change will occur over the forecast period in the distribution of axle weight and in the total weights on the axles.

Load Applications Due to Trucks

The truck traffic forecast by class of vehicle can be approached by a study of transport by commodity class and by the vehicle class most likely to be used in hauling each commodity group. In making such a forecast, attention must be given to the ton-miles of freight and the highway's share of the total tonnage that will be moved by all transport modes. In recent years, the highway share of the nation's total freight haulage has gradually increased. This increase may or may not continue, depending on many factors now undergoing change.

Over the years the highway trucking industry has been successful in increasing the average number of pounds of payload per trip by improving management, dispatching, and ordering, and by using vehicles with lighter tare weights. Any probable future increase in payload per

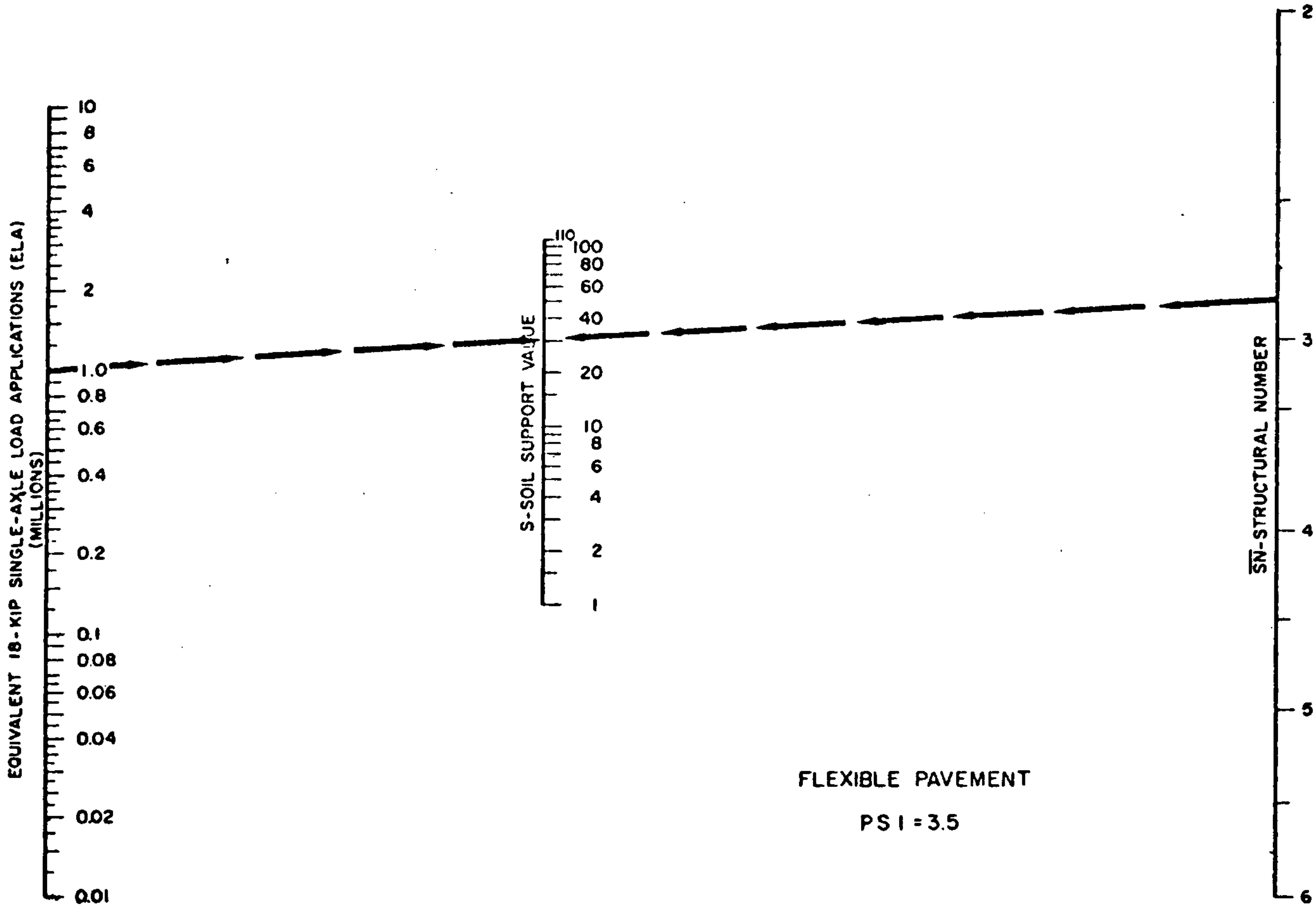


Figure 20. Flexible pavement service life nomograph for PSI=3.5. Source: Corvi and Bullard (75).

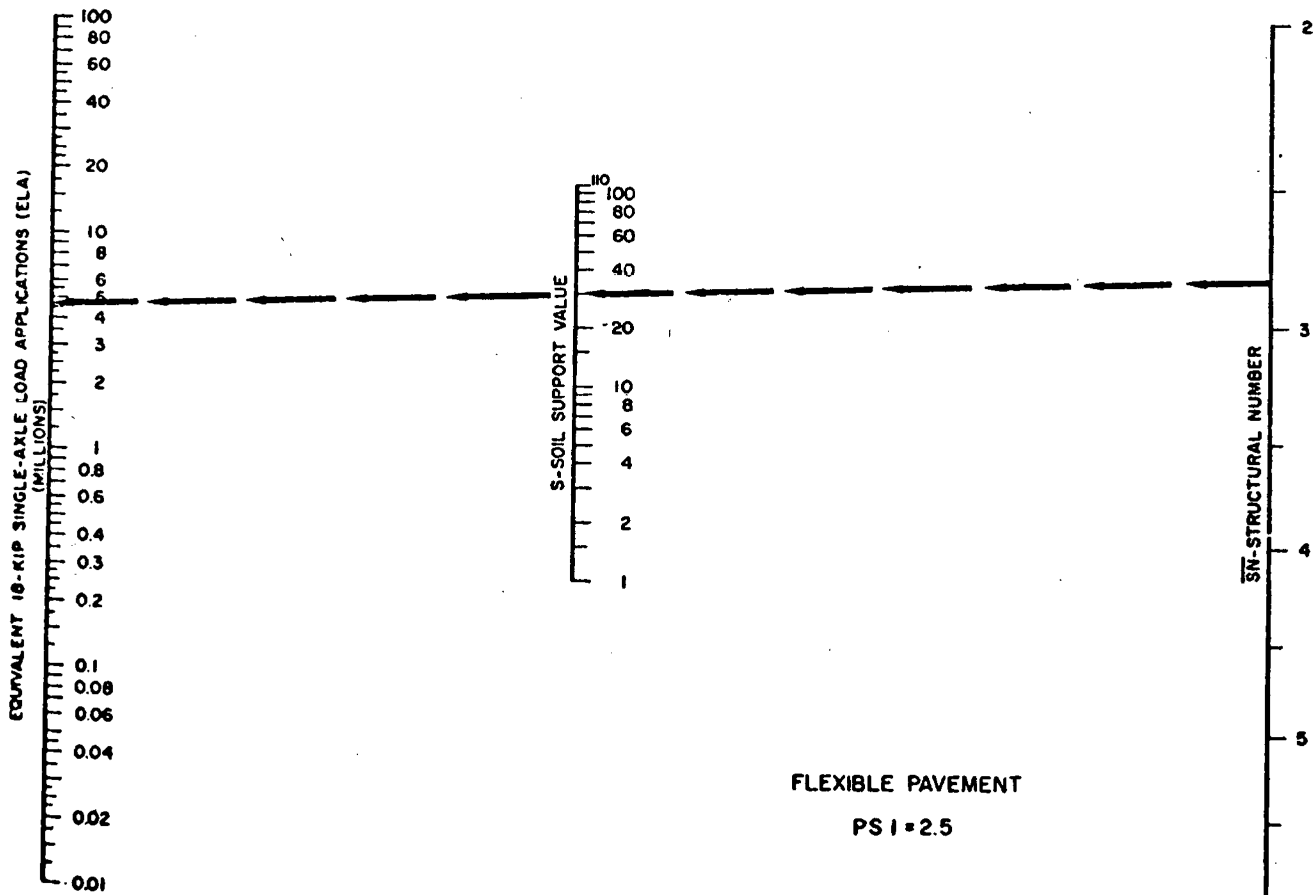


Figure 21. Flexible pavement service life nomograph for PSI=2.5. Source: Corvi and Bullard (75).

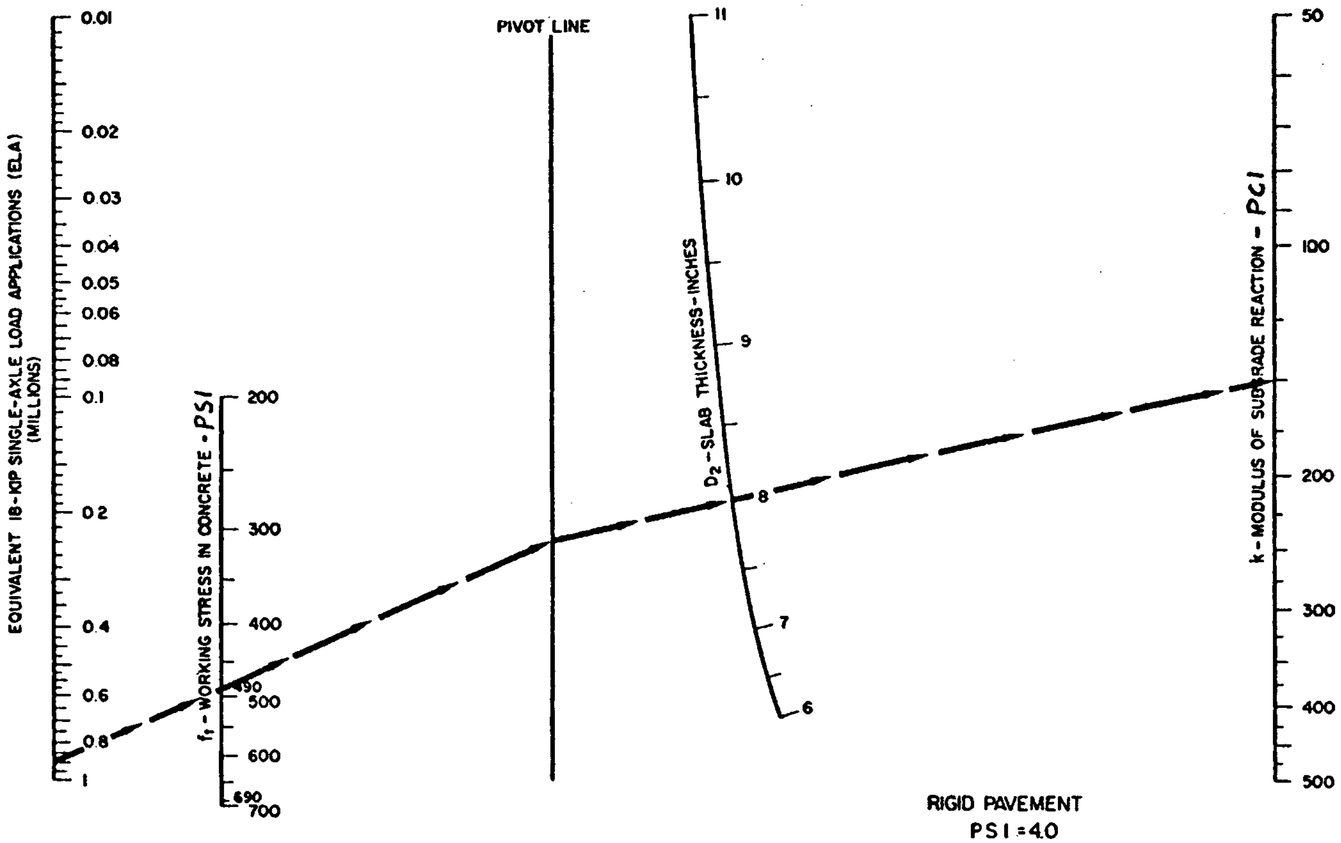


Figure 22. Rigid pavement service life nomograph for PSI=4.0. Source: Corvi and Bullard (75).

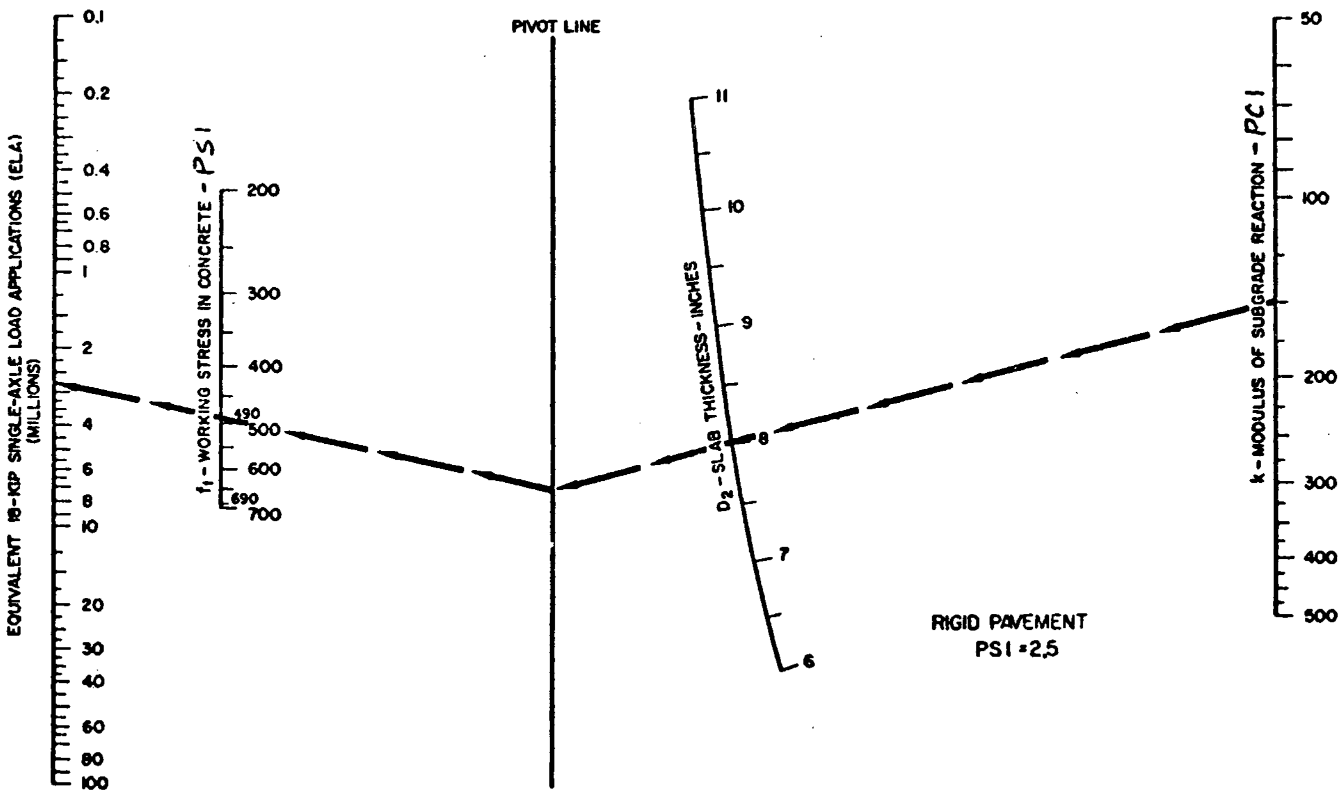


Figure 23. Rigid pavement service life nomograph for PSI=2.5. Source: Corvi and Bullard (75).

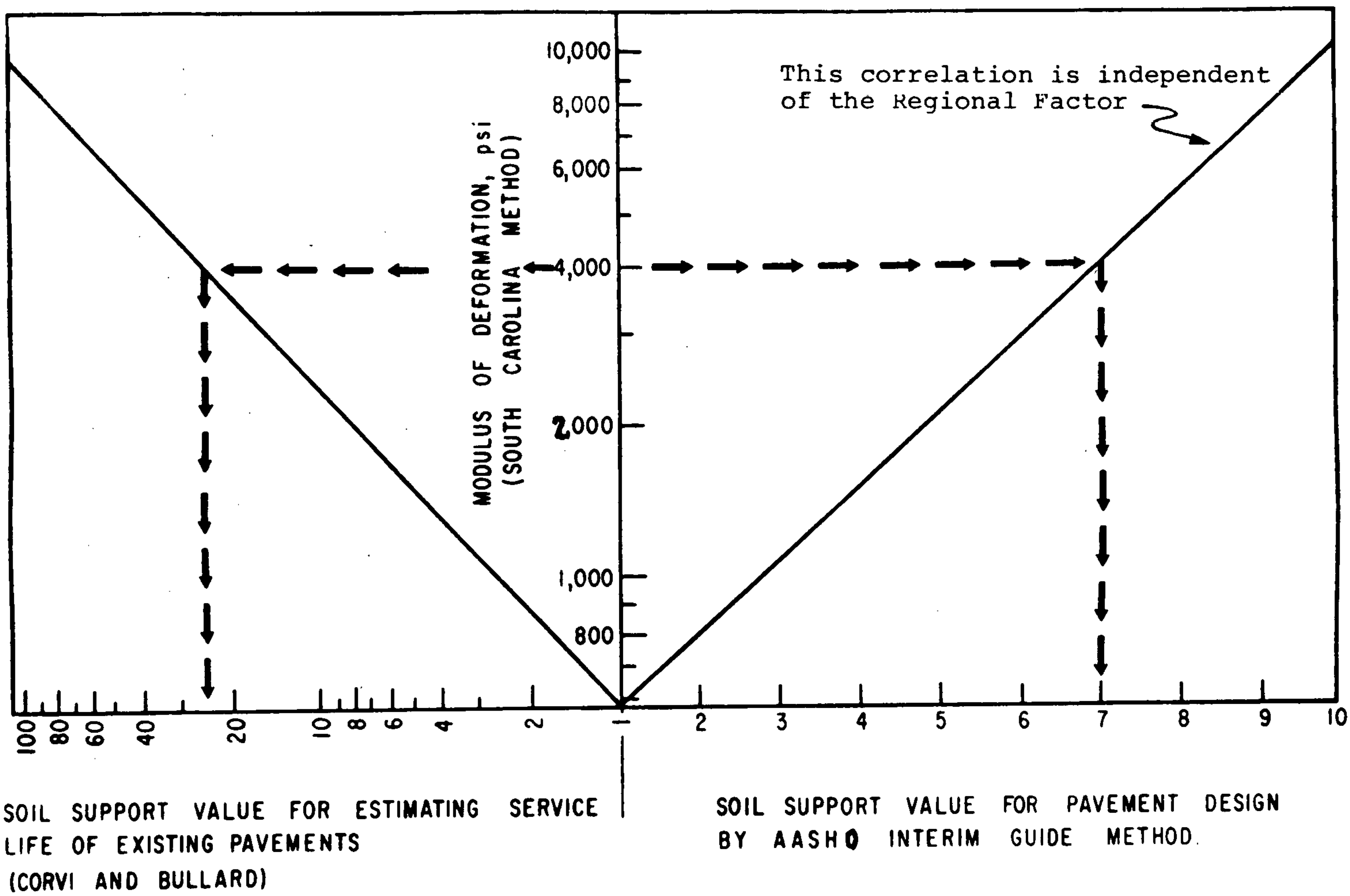


Figure 24. Correlation of modulus of deformation of subgrade with soil support values.

vehicle should be accounted for in the forecast of truck traffic. The payload increase per trip, in effect, will reduce the number of trucks on the highway. An examination of past annual truck weight data gives some indication of the trend.

Truck use in a particular state or political jurisdiction is greatly affected by the legal limits on dimensions and weights in surrounding states. This is an important consideration in forecasting both traffic and truck practice under higher limits of dimensions and weights. Interstate trucking is so general that all truck movements must be planned in the light of the requirements imposed by the laws of each state to be entered or crossed. The laws of the state with the minimum limits usually govern trucking practice on any given trip.

Load Applications Due to Bus Operations

Table 35 gives the modal distribution of intercity passenger demands, 1950 to 1969, expressed in passenger-miles. Although total passenger movements increased at an annual average rate of 4.2 percent, the private automobile served most of this expansion. Domestic aviation also grew dramatically, although from a small base. Rail and motor coach passenger movements declined. Service provided by inland waterways, although increasing, remained comparatively unimportant.

Allocation of passenger traffic to modes shows the automobile predominant with, in 1969, 86.6 percent of all intercity movements. Domestic airlines served 9.8 percent of demands; motor coaches generated 2.2 percent of national passenger-miles.

Of prime interest to this analysis is the bus. The passenger-mile performance by intercity buses declined sharply from 26.4 billion in 1950 to 19.9 billion in 1960 (Table 35). Thereafter, a recovery was achieved over the 1960's to 24.9 billion by 1969, to give a modest decline for the total period. The pattern in modal share for buses is more alarming. Owing, it is surmised, to the rapid advances made by domestic air travel, the bus share of total passenger-miles declined progressively from 5.2 percent in 1950 to 2.2 percent in 1969. This decline should not be interpreted as indicating that intercity bus travel will eventually become insignificant. Important price differentials between buses and the principal alternative mode, the airlines, dictate that buses will continue to serve low-cost demands.

TABLE 34

MAJOR ITEMS IN EVALUATING EFFECTS OF CHANGES IN LEGAL VEHICLE LIMITS ON PAVEMENT COSTS

	ANALYSIS FOR ESTIMATING INCREMENTAL COST DUE TO INCREASES IN LEGAL LIMITS		
COST ITEM (1)	ASSUMING NO CHANGE IN LEGAL LIMITS (2)	ASSUMING CERTAIN INCREASES IN LEGAL LIMITS (3)	BASIS FOR ESTIMATING INCREMENTAL COST RELATED TO PAVEMENTS (4)
Construction for new pavements or reconstruction of existing pavements	Determine required thickness of pavements.	Same as in Col. (2), except that increased thickness may be required due to change in legal limits.	Difference in pavement or overlay thickness referred to in Cols. (2) and (3).
Construction for overlays above existing pavements	Determine required thickness of overlays.		
Cost of pavements concerning remaining service	Estimate remaining service life of existing pavements.	Same as in Col. (2), except that increase in legal limits may result in a reduction in remaining service life.	Difference in the remaining service life referred to in Cols. (2) and (3).
Pavement maintenance	Estimate cost to meet all maintenance requirements.	Same as in Col. (2), except that increase in legal limits may result in an increase in maintenance cost.	Difference between the values from Cols. (2) and (3).

However, buses do not appear to generate a significant 18-kip equivalent axle load. The principal interest of bus operators is in increased vehicle width, as cited in Chapter One. The influences of this parameter on highway geometric design and traffic operations are discussed later.

Projection of Highway Load Experience—Truck Weight Studies

Each year, as part of their annual collection of information for the federal-aid highway planning projects, the states weigh vehicles at permanent and temporary weighing stations. These studies are part of engineering and economic investigations financed in accordance with Section 307 (C), Title 23, U.S. Code "Highways," and are conducted in cooperation with the FHWA (*86*):

> Truck weight data collected annually by the States are the bases for estimating annual travel by each type of truck, the ton-miles of cargo hauled via highway, year-to-year changes in axle and gross weight frequencies and comparison of the characteristics of actual usage with administrative policies. The results are used at the State and National levels in the consideration of transportation policy, allocation of highway costs and revenue, size and weight regulations, establishment of geometric design criteria related to the size and weight of vehicles, in pavement design for the establishment of procedures and design criteria, and for a variety of special administrative, planning, design and research studies. At the State level, truck weight data are used in calculating pavement loading in 18-kip equivalents or other comparable procedure, and in bridge loading analysis in terms of both bending moment and fatigue. Safety studies require data relating class of operation, vehicle type, time, highway type, and State registration to provide exposure rates related to available accident data. Planning, program budgeting, and administrative studies require axle and total weight distribution data which can be related to operational characteristics, taxation rates, incremental construction and maintenance responsibility, and enforcement effectiveness.
>
> The continuity of the trends beginning in 1936 provides important indications of changing patterns in transportation by highway compared to rail and other modes, and provides a measure of the effect of changing policies and regulations, changes in economic activity, and technological advances. The annual reporting by each State of consistent reliable data which is representative of truck usage of the various highway and street systems is essential to the continuation of reliable output from these studies and analyses.
>
> Data from these annual trucking characteristics studies are summarized in a series of tables. Prior to 1970 it was the responsibility of each State highway department to analyze the data collected and prepare a report containing these tables. Arrangements have now been made for the data to be transmitted to the Program Management Division, Federal Highway Administration, in the form of data processing cards or card images on magnetic tape. Appropriate summary tables are then prepared by computer and returned to the States. In order to satisfy the need for these data at the State level, it is desirable that each highway department prepare an annual report which includes basic tables and a suitable narrative which can be made available to all users.

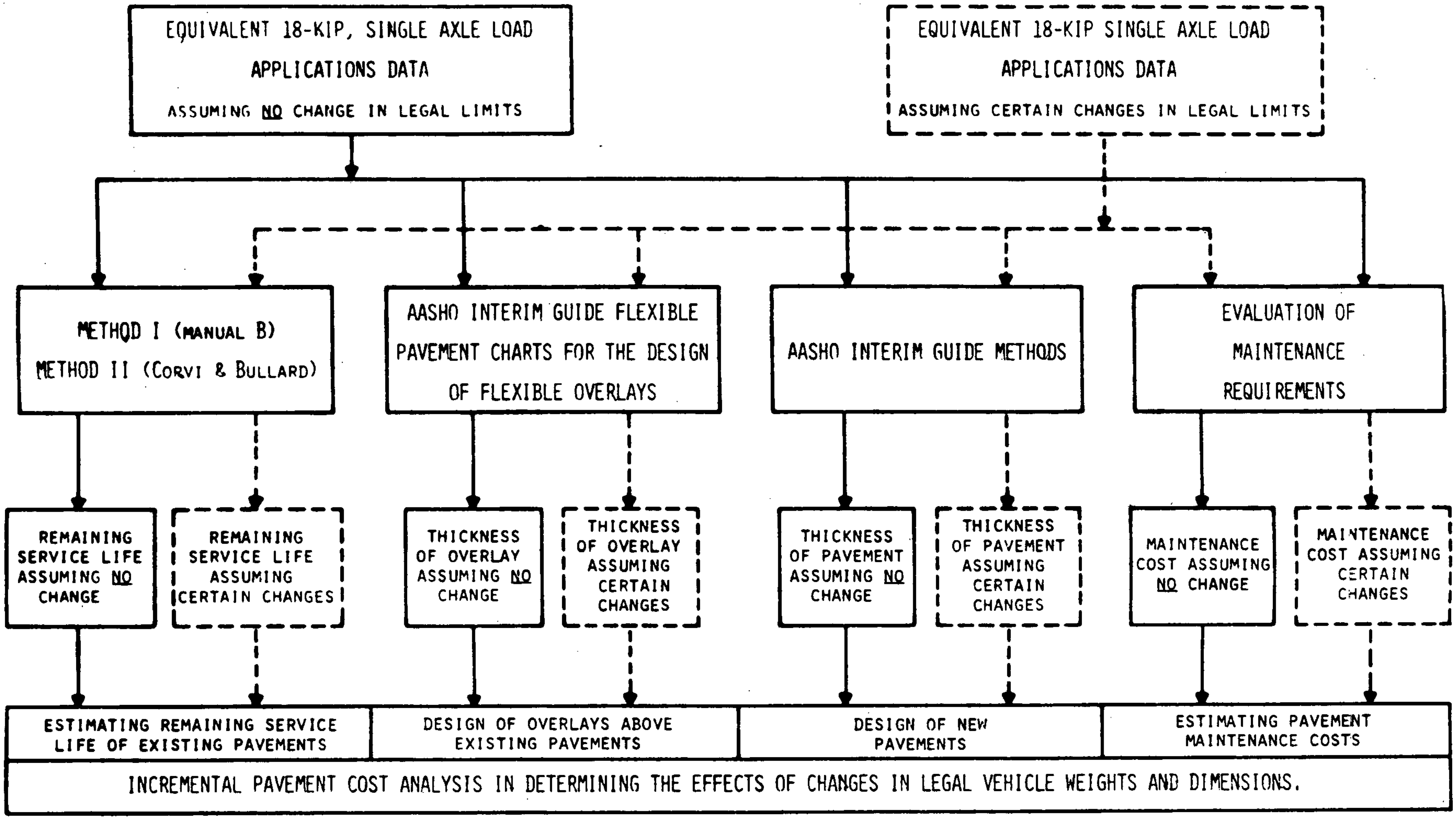


Figure 25. Procedures for determining effects of changes in incremental cost analysis.

TABLE 35

INTERCITY PASSENGER-MILES, BY TRANSPORT MODE, 1950–1969

	PASSENGER TRAFFIC, BY TRANSPORT MODE						
	MOTOR VEHICLES			RAIL REVENUE PASS.	INLAND WATER-WAYS	DOMESTIC AIR	TOTAL
YEAR	AUTOS	COACHES	TOTAL				
	(*a*) Billions of passenger-miles						
1950	438.3	26.4	464.7	32.5	1.2	10.1	508.5
1955	637.4	25.5	662.9	28.7	1.7	22.7	716.0
1960	706.1	19.9	726.0	21.6	2.7	34.0	784.3
1967	889.8	24.9	914.7	15.3	3.4	87.2	1,020.6
1968	936.4	24.5	960.9	13.3	3.5	101.2	1,078.9
1969	977.0	24.9	1,001.9	12.0	3.8	111.0	1,128.7
	(*b*) Percent passenger-miles, by mode						
1950	86.19	5.20	91.39	6.39	0.23	1.99	100
1955	89.02	3.56	92.58	4.01	0.24	3.17	100
1960	90.03	2.54	92.57	2.75	0.34	4.34	100
1967	87.18	2.44	89.62	1.50	0.33	8.54	100
1968	86.79	2.27	89.06	1.23	0.32	9.39	100
1969	86.56	2.21	88.77	1.06	0.34	9.83	100

Note: Data subsequent to 1960 include Alaska and Hawaii.
Source: 1950-1967—Automobile Manufacturers Assn. 1970—*Automobile Facts and Figures*, p. 54. 1968, 1969—ICC, *Transport Economics, Monthly Comment* (Aug.-Sept. 1970, p. 14).

Reliability and accuracy—The success and value of all uses of the truck weight data depend on the reliability and accuracy of the data collected in the field. The field procedures must be directed toward reliability of data, while at the same time giving full consideration to efficiency of operation and the safety of the traveling public and the field staff. There must be a continuing effort to develop citizen understanding and appreciation for the State and Federal governments' efforts to provide more efficient and convenient transportation. Each of these considerations must be weighed in selecting each station location, scheduling the work and assigning personnel to each task, sampling from the traffic stream, interviewing, and obtaining weights and dimensions.

Studies made by individual states of the adequacy of truck weight data for some purposes indicate that problems arise in the size of the sample if the data are aggregated into many categories. Alexander and Bowling (*87*) point out that determining the weight breakdown of trucks into 18-kip axle equivalents causes some difficulty. Field weight studies cannot be conducted on each design situation without excessive cost.

One method, used in the past in highway design in Georgia, consists of using statewide averages for 18-kip equivalents based on data from all weight stations, broken down into averages for each truck type and for rural and urban road systems. Because of the many different types of roads and design conditions, this method was not successful.

A second method sought to use data from a weight station on a road similar to the road being designed. Although this method provided a better estimate than statewide averages, it did not provide the accuracy needed.

A third method relates traffic classification counts to truck weight data. Classification data are used to determine average percentages of different truck types (cars, single-unit trucks, and combinations) that might be expected. With these estimated percentages applied to the design road, and using average equivalents for each vehicle type, an estimate of equivalent design loading is made. This method provides more flexibility, although errors are introduced in using average vehicle percentages and equivalents.

Buffington, Schafer, and Adkins (*88*) found significant differences between most station and highway system averages within vehicle types. Grouping stations according to highway system, as well as geographically, failed to produce homogeneous weight distributions. Much of this variation was attributed to changes in the proportion of loaded to empty tandem-axle vehicles. Part of the variation between station averages of vehicle and axle weights was due to differences in weighing schedules and to small samples that produce chance differences.

Combining all vehicles weighed at all stations produced reliable averages of vehicle 18-kip axle equivalents. The analysis concluded that considerably more vehicles must be weighed to obtain average *vehicle* weights in 18-kip axle equivalents than to obtain accurate average *axle* weights in 18-kip axle equivalents. The study also concluded that combining multiyear loadometer data produced more accurate estimates of total 18-kip axle equivalents at each station than did a single year of data, and removed some of the differences due to sample size and weighing schedule.

Estimating New Total Highway Load Under New Limits

Fundamental to the analysis of highway department construction and maintenance costs that would probably prevail under a change in the legal maximum limits (normally to higher limits) of gross weight or axle weight is the estimation of the probable distribution of (1) the gross weight of each class of vehicle, and (2) the axle weight of each class of vehicle by single and tandem axles, separately. Further, these estimates would be made for each road system to be considered.

The probable expected gross weight distribution and the probable axle weight distribution are requirements in the design of structures and pavements to accommodate the new legal limits. The data also are necessary to estimate the number of vehicles that would be used to transport the projected total tons of cargo. These same estimates of weight distribution are required in any analysis of the relative transportation economy to be achieved with the new weight legal limits, as compared to existing legal limits.

Also essential to the over-all analysis is a forecast of future highway use, with special attention to the effects of the change in legal maximum weight limits. This discussion is confined to estimating the two sets of weight distributions on the basis of 100 vehicles in each vehicle class; i.e., on a percentage basis of the total number for each vehicle class in the traffic stream.

Gross weight and axle weight distributions under new legal weight limits are estimated from truck weight data collected yearly by state highway departments, or data that could be collected by direct weighing of vehicles at the roadside. In general, these weighings in the past have supplied the following field data:

1. Average daily traffic volume of cars, buses, and each class of truck.
2. Number of vehicles weighed in each vehicle class.
3. Designation of whether the vehicle weighed was empty or with payload.
4. Weight of each axle on each vehicle weighed and the total, or gross, vehicle weight.
5. Type of vehicle design (body), commodity carried, axle spacing distance, and whether common, contract, agricultural exempt, or private carrier. These items are shown only for specific years.

From available data it is possible to calculate the following for each class of vehicle:

1. Average empty weight.
2. Average loaded weight.
3. Average gross weight.
4. Weight distributions—percentages of weighed vehicles by weight intervals:
   (a) Empty vehicles.
   (b) Loaded vehicles.
   (c) Loaded and empty vehicles combined, gross weight.
   (d) Distribution of weight of each axle.
5. Payload weight per vehicle.
6. Total payload per vehicle class, and for all classes.
7. Percentage of gross weight carried by each axle for each gross weight interval; say, 1,000-lb increments.

There has been no established procedure of using actual field weighings and counts of trucks operating on the highway under current effective legal limits (including permit operation of overload and oversize vehicles) to estimate operating weights and frequencies by vehicle class under changed legal limits. In fact, such estimates were made only once previously—by the BPR in its research of the transportation economy of the legal maximums of vehicle dimensions and weights (*89*).

In the current study, the attempt is made to devise one or more procedures of estimating the gross weight distribution and axle weight distribution. One of the constraints imposed is that the procedure is to be based on use of data available within a given state and not taken from other states that may be operating under legal limits that are higher or lower than those in states under study.

Estimating Gross and Axle Weight Distribution for a Specific Class of Truck Under New Limits *

The logic of the procedures is based on having available the axle weight and gross weight distributions and other data under existing legal limits, such data being taken by weighing the trucks at roadside stations for a sufficient time and number of locations to get a reliable sample.

The basic assumptions or logic used in the following procedures are that, under new legal weight limits, (1) the empty weight of the trucks will increase, assuming legal weights are increased, to provide for the strength and durability of the vehicle in use under heavier payloads; (2) trucks will carry greater payloads per trip, and, therefore, operate with higher axle weights and higher gross weights; and (3) operation under the new limits will change somewhat in proportion to the change in the practical maximum gross weight of each vehicle class, which is defined as the sum of the individual axle legal weights, with the front or steering axle weights set at a reasonable amount, consistent with that class of vehicle and what past roadside weighing has shown as being normal practice.

It is assumed that, under the new legal limits, the change in axle weight distribution will be generally consistent with the increase in gross weight. The new distribution in axle weight for each type of axle—single or tandem (all singles may be added together and all tandems may be added together for the same vehicle class)—may be assumed to retain the same ratio to the gross weight under the new limits as was found in the roadside weighings.

To calculate a series of ratios it is first necessary to plot two curves: (1) the accumulated percentage of the vehicles that weigh within each of a series of gross weight intervals of, say, 1 kip; and (2) the accumulated percentages of each axle type that weigh within each of a series of intervals of axle weights of, say, 1 kip. From reading the weight on each curve at a chosen series of percentages (say, each 5 percent) the data are obtained for computation of ratios of axle weight to gross weight for the series of percentages.

Examples of curves are shown in Figure 26, indicating typical distributions of axle weights by vehicle class and by states having axle limits of 18-kip single axle/32-kip tandem axle, 20-kip single axle/35-kip tandem axle, and 22-kip single axle/38-kip tandem axle. Distributions also are grouped by weights on single axles and tandem axles. Dotted curves were extrapolated by judgment to indicate

* See Appendix B for a step-by-step example.

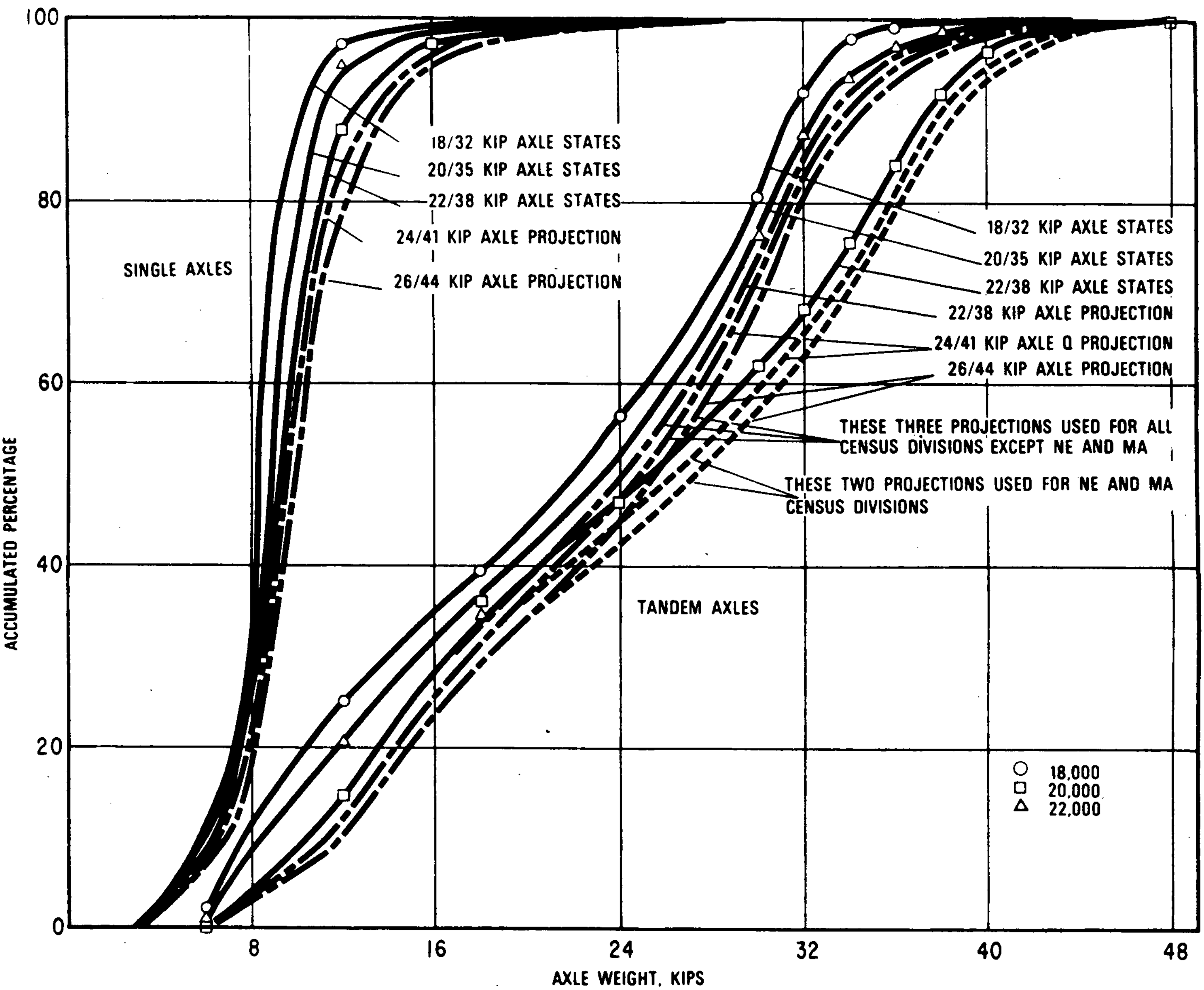


Figure 26. Examples of curves showing distribution of axle weight by vehicle class from a truck weight study (solid curves).

trend distributions should 24/41 or 26/44 limits apply. These curves do not reflect the results of the application of the method assembled here.

Weight Interval Ratio Method

The method assembled and recommended to obtain axle weight distributions by truck classification for all trucks operating under a proposed new axle weight limit has six phases. This concept differs somewhat from previous attempts, and its execution is considered much simpler. Using truck weight data, the first two phases are to calculate adjusted average empty weight, adjusted gross weight, and total payload carried by each vehicle class at the base (present) limits.

From these distributions the third phase is to calculate gross weight and total payload carried by each vehicle class under the proposed new axle weight limit. These projections permit the fourth phase: estimation of the number of vehicles required to carry the original payload under proposed limits.

The number of trucks in each vehicle class is then redistributed in the fourth phase to produce a series of axle weight distributions that can be anticipated under proposed limits. This distribution is for a total payload related to each class.

These procedures are based on the following assumptions:

1. The lowest gross weight interval in the truck weight data represents the lightest tare weight encountered.

2. At the gross weight interval equal to the original practical maximum gross weight (PMGW), the percentage of vehicles at this interval will remain the same for the new PMGW.

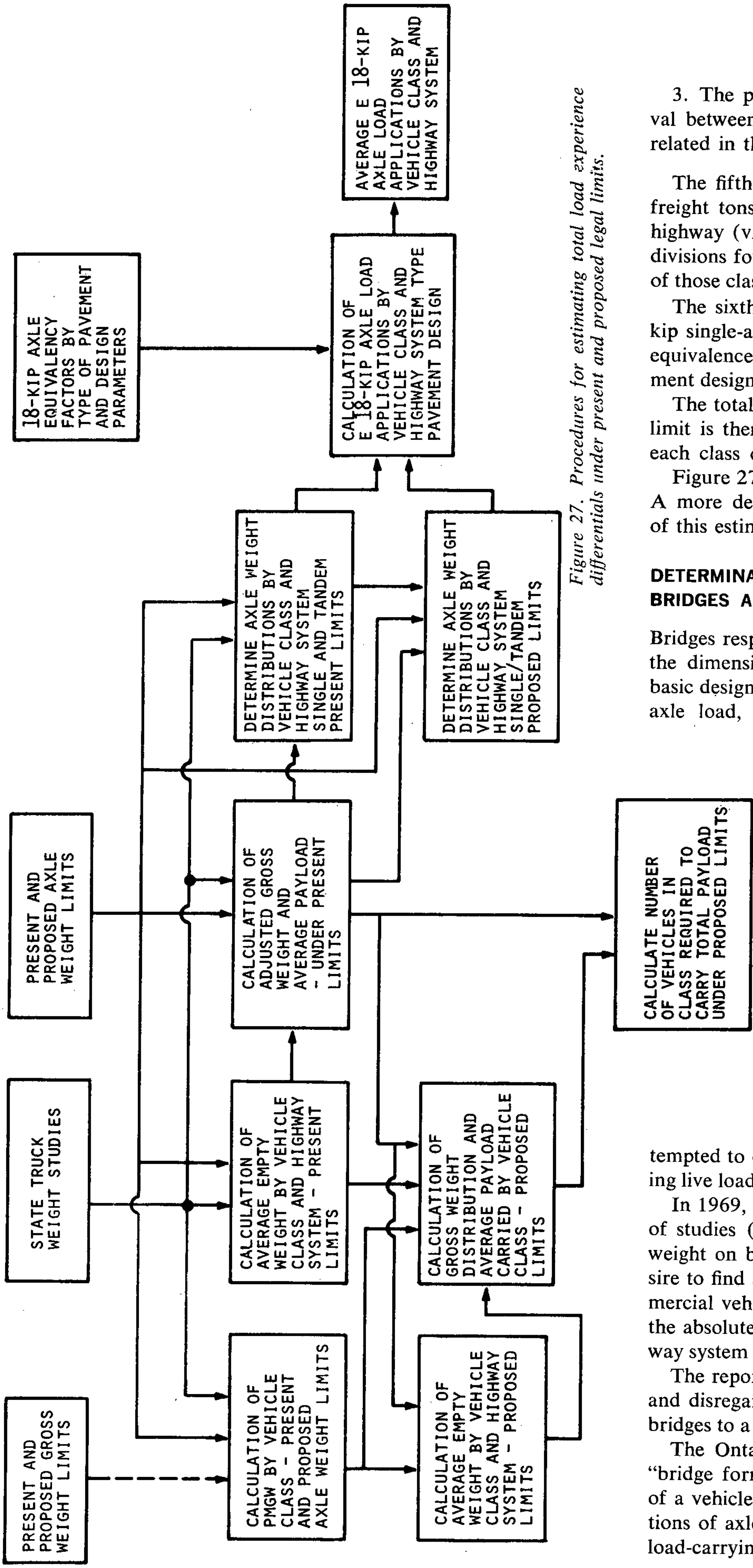


Figure 27. Procedures for estimating total load experience differentials under present and proposed legal limits.

3. The percentage of gross weights at any weight interval between the lightest and the original PMGW will be related in the same fashion to the new PMGW.

The fifth phase assigns the projections of annual motor freight tons to each class of vehicle and to each class of highway (viz., rural Interstate, rural primary) based on the divisions found in the truck weight studies in the ADT mix of those classes of highways.

The sixth phase converts these data into equivalent 18-kip single-axle loads, using AASHO Road Test single-axle equivalence factors for the various flexible and rigid pavement designs.

The total highway load due to a proposed new axle load limit is then the summation of the 18-kip equivalents for each class of vehicle by highway classification.

Figure 27 shows a summary flow diagram of this process. A more detailed flow diagram and a numerical example of this estimation process appear in Appendix B.

DETERMINATION OF PHYSICAL EFFECTS ON BRIDGES AND STRUCTURES

Bridges respond differently to loads, depending, mainly, on the dimension, the material, the type of bridge, and the basic design. The vehicle load is a function of gross weight, axle load, and axle spacing. Several authors have attempted to explain various aspects of the effects of increasing live loads on bridges.

In 1969, the Ontario Highway Department made a series of studies (*90, 91, 92*) to determine the effect of vehicle weight on bridges. These studies were motivated by a desire to find a reliable method of determining how the commercial vehicles could be licensed for normal operation at the absolute maximum loads and still not subject the highway system to abnormal damage.

The report revealed that regulating only the gross weight and disregarding axle-weight restrictions may overload the bridges to a large extent.

The Ontario report gives a new method, as well as a "bridge formula," to calculate the maximum gross weight of a vehicle that may be permitted with different combinations of axle spacing and axle weight. In this method, the load-carrying capacity of a bridge is allowed to exceed design loads by the application of the following criteria:

Unwelded steel beams . . 85 percent of yield stress.
Welded steel beams . . . 75-80 percent of yield stress.
Prestressed concrete . . . Modulus of rupture.
Reinforced concrete . . . 73 percent of cylinder strength.

Many studies (*93, 94, 95, 96*) have been conducted on the fatigue strength and life of bridges, collecting data on load spectrum and stress range through field observation and tests. These tests reveal that, under current traffic conditions, the bridges examined have fatigue lives well over 1,000 years, and the bridges that have less traffic with high-damage range would have shorter lives than the ones that have more volume with low-damage range. The heaviest traffic in these studies had an annual commercial traffic volume of 860,000, which is about 40 percent of the annual total traffic volume of 2,150,000.

Bridges are the principal highway structures of concern in evaluating the impact of a contemplated change in legal vehicle limits. However, other minor structures, such as retaining walls and culverts, may be affected by some form of change and should be included when physical impacts on the highway system are considered.

It was evident that any evaluation technique or method selected would involve some compromise between accuracy and detailed analysis, and the economics of the application of the method in resources required to perform the evaluation.

It was assumed in this approach that the method assembled would be suitable for the computation of the structural cost of elements for most types of bridge structures in current highway system inventories. It was recognized that there would be an occasional highly specialized design type, such as a suspension bridge, an arch bridge, or a truss-type bridge, that would require specific structural analysis rather than some general analysis technique. Therefore, these specialized structures were not included in the general methodology. More specific forms of analysis would be applied to these specialized structures, which represent only a fraction of the total in any state.

Bridges are individual and respond to loads differently. Because of the complexity of the variables involved, a study of each bridge under each live load condition is impractical (even though possibly desirable) because of the extensive time and effort required. The method must be simple in application. It should be office-procedure oriented instead of purely theoretical—statistical rather than specific. However, simplicity must not sacrifice mathematical rigor. Sound engineering assumptions must balance formidable mathematical complexity. For these reasons, the method cannot be expected to substitute for the detailed engineering analysis of an individual structure to determine its specific, individual structural integrity.

The method should be capable of evaluating the five most commonly used general bridge types: flat slab, reinforced concrete T beam, reinforced concrete box girder, prestressed concrete girder, and steel girder. In addition to the girder system, other major structural parts of a bridge—deck, pier, and foundation—must be included in the analysis. Each is affected potentially by a change in load limits.

The change in live load, resulting from an increase in legal vehicle weight limit, probably will have an adverse structural effect on existing bridges. Qualitatively, increases in live load may be expected to decrease serviceability, accelerate deterioration, shorten structural life span, and conceivably cause failure of some structures. Ideally, any increase in live load beyond the original design should require immediate new construction to the new standard. Obviously, such upgrading requirements applied to an entire highway system cannot be so easily accomplished, practically or economically.

Fortunately, the safe live-load-carrying capacities of existing highway bridge structures generally are higher than the original design loads. This suggests that only a portion of the existing bridges would require immediate replacement under certain ranges of possible new load limits. Frequent overload, within reasonable limits on some bridges with certain materials and span ranges, may reduce the life span considerably, but may not cause immediate failure. Bridges in this category may be serviceable for a sufficient time to defer immediate reconstruction. Further, some structures may be capable of being strengthened to accommodate the increased loading.

The method therefore should permit the determination of the aggregate structural behavior of groups of bridges under the proposed new load limits. The structures in the inventory would be apportioned into three groups:

1. Unsafe and cannot be reasonably strengthened—requires immediate replacement.
2. Overstressed in part or total, but can be reasonably strengthened—continue in service with modification.
3. New load is within or reasonably near the allowable design limits—continue in service without change.

From these categories and a complete inventory of the bridges in the system, the approximate or relative cost impact of the new loading can be estimated. Obviously, this introduces another essential condition for any evaluation technique—the existence or availability of a detailed, comprehensive systemwide bridge inventory. Without such data as input, it is inconceivable how any rational general evaluation can be made and still have reasonably accurate results.

The problem of evaluating the cost impact of the new limits to new construction appears more direct and simpler. Methods for this category of bridge cost are to be treated separately from that required to evaluate existing structures.

The method proposed here is believed to be effective and systematic. Numerical examples and other data given will permit highway departments to apply the technique and perform the cost impact study of their bridge structures. The derivation of a step-by-step computational method for structures, and numerical examples illustrating its application, appear in Appendix D.

Ontario Method

The Ontario reports (*91, 92, 93*) offer a relatively sound mathematical and engineering concept for examining the effects of different types of axle group combinations on bridge structures. Various factors that influence the bridges, such as materials and design type, were taken into consideration. The method focuses on the regulation of vehicle weights. Cost impact was not the purpose of that study. The "bridge formula" derived therein related only to the effect of loads on superstructures and disregarded the effect on substructure elements of the bridges, which is acceptable because the substructure has a higher dead load to live load ratio than the superstructure.

In general, most of the concepts presented in the Ontario reports may be adopted and tailored to suit some of the needs of this study.

Logical Development of Proposed Method

Figure 28 shows the logic flow of the proposed method. It shows the approximate relationships of the various elements of the method involved in estimating the total cost, by increments, assignable to proposed new legal vehicle limits on structures.

Inputs

Inputs are the inventory of structures, the estimated average unit costs, and a summary statement of vehicle weight/size and configuration, including the frequency and traffic mix of the vehicles using the new limits.

Classification

From the inventory, existing structures are classified by bridge type, original design load, span group, and the recorded material quantities of the original design.

The summary of vehicle weight/size and configuration is used to calculate and classify the overstress factors under the new load limits. Upon establishment of criteria for permissible overstress factor, which may be adjusted as desired by the user, the serviceability of the bridge is estimated and the bridges are assigned to one of the following categories:

1. *Needs Immediate Replacement:* For these bridges, the next step is to determine the material quantity ratio and to perform a cost analysis.
2. *Post Bridge to Original Limit:* Here, the decision to permit the posting of a bridge to the original limit is an operational rather than a technical problem. Some states see no difficulty in operating their systems under a multiple load classification; others deem this type of operation undesirable or unenforceable.
3. *Strengthen to Serve New Load Limit:* Some bridges, because of their design and construction, cannot be strengthened economically. These bridges may be automatically excluded from this category. Other bridges which, by nature of their design, can be economically strengthened are examined for overstress factor. If the overstress factor falls within an acceptable range, an approximate cost analysis for strengthening is then conducted.
4. *Upgrade Load Rating to Serve New Load Limit Without Modification:* Structures in this category should undergo a fatigue life analysis to determine the extent of shortened life of the structure due to the new loads. A cost analysis is performed on bridges whose useful life is appreciably affected by new load limits.

Secondary Structures Cost Studies

Cost studies are made of secondary highway structures, such as culverts and retaining walls, that might be affected by a change in limits.

Compute New Construction Requirements

New construction of highway structures should have been identified in the planning included in the Fiscal and Needs Studies. The planned construction is based on retirements and projected new facilities. Using these programs as a base, compute the cost of new construction if designed and built in conformance with the new load requirements. When these new costs have been estimated, the cost differential for the new construction to the new limits, and therefore that amount assignable to the changes, would be the cost difference between the planned and the newly designed bridge structures. The difference is then included in the total incremental cost study.

Maintenance Cost Increment

Certain maintenance costs for structures will have been identified from existing cost records. The probable impacts on these costs are discussed and certain judgment factors are applied to determine maintenance cost increments, if any, due to new load limits.

The literature search disclosed no authoritative references on the relative importance of the several factors that contribute to structure maintenance costs. Some of the factors, such as painting costs, obviously bear little or no relationship to loads. Other factors, such as bridge deck maintenance costs, probably are jointly related to both weather conditions (deicing chemicals) and load frequency and magnitude. Still other maintenance costs may be related primarily to load. Load is more properly dealt with under the general heading of fatigue life, and is so considered in this report.

Assuming that relatively small changes (in the order of ± 20 percent) in current load limits are within the probable scope of consideration under this study, either of the two methods of estimating changes in structure maintenance costs will yield acceptable results for over-all evaluation purposes. It can be assumed (1) that structure maintenance costs are independent of load limits. Application of this assumption will of course show no change in costs from current experience records. Or, it can be assumed (2) that structure maintenance costs are linearly related to maximum permitted gross vehicle weights.* It is believed that the application of this method will show a probable boundary limit of change in maintenance costs.

* New maintenance cost equals the ratio of proposed gross vehicle weight to present maximum gross vehicle weight times current average maintenance costs.

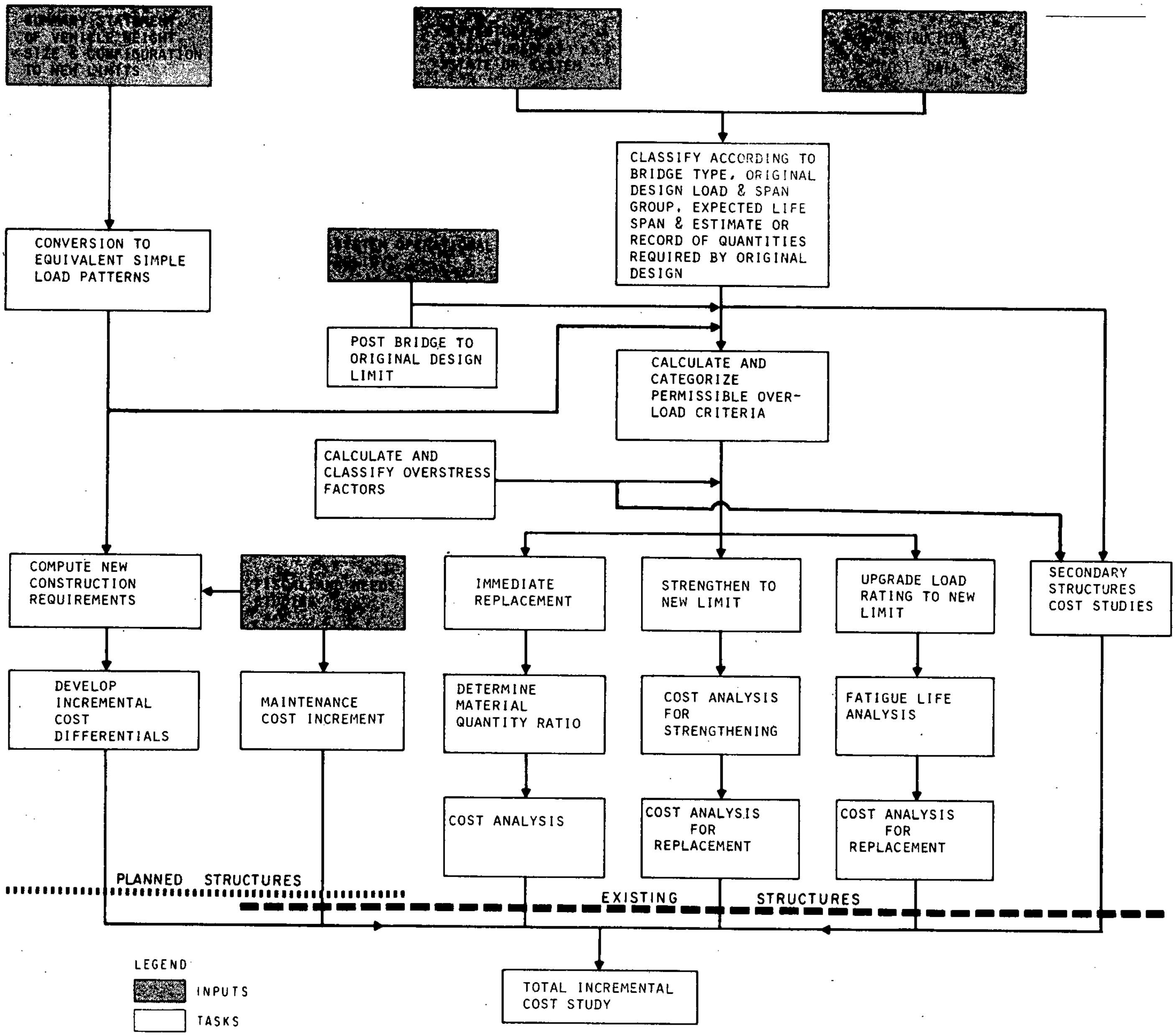


Figure 28. Logic flow for determining effects of changes in incremental cost analysis.

Secondary Structure Cost Analysis

Box Culverts.—The overstress ratio for box culverts without earthfill is similar to that of flat slab bridges. The effect of live loads on box culverts becomes less significant as the depth of earthfill increases. When this depth is equal to or greater than the width of the culvert, the effect of live loads may be negligible. For depths of earthfill between these two limit points, the overstress factor is interpolated between the overstress without earthfill and zero overstress on the basis of the ratio of earthfill depth to culvert width. Other types of culverts are neglected for the purposes of this study.

Retaining Walls.—Except for very low walls, the effect of additional live loads applied as surcharge usually is small compared to the effect of lateral soil pressures involved in the design of retaining walls. In the design of retaining walls, high safety factors are used to account for sliding, overturning, etc. The overstress factor, and hence the increase in cost chargeable to legal limit changes, are so small that they can be neglected for the purposes of this study.

Impact of Increased Legal Limits on Bridge Rails.—The criteria in AASHO specifications for designing bridge rails aim at the protection of average passenger cars but not heavy trucks or truck trailers. For heavy trucks, a safe design may involve unjustifiable high cost. If the same viewpoint is held, an increase in legal loads generally will not alter the design of handrails.

However, one could not totally disregard the importance of bridge rails to trucks and truck trailers in some urban areas. In those cases where an accident could mean a more severe economic damage and social impact, stronger rails become increasingly important. To the researchers' knowledge, no study is available on this topic.

IMPACTS OF VEHICLE DIMENSIONS AND PERFORMANCE ON GEOMETRIC DESIGN

Highway geometric design practices and policies usually are based on critical design vehicles whose characteristics are assumed to be representative of most of the vehicles using the facility. In the manner discussed for bridge design, the parameters of the critical vehicles are chosen so that the limiting case can be explored and properly accommodated in that particular design element. From these investigations, design policies and standards are established and applied to the highway system.

So that a new limit of weight or size can be properly evaluated, the new relationships between the inherent characteristics and performance of vehicles and these limits must be developed and compared with existing relationships. Changes in geometric design criteria can be determined, and assessed for economic impact on reconstruction and new construction of the highway system to accommodate the advanced vehicle. Likewise, the impact of performance of these vehicles operating in mixed traffic can be determined.

Sizes and weights play an important role in determining elements of geometric design. Legal limits are imposed on these parameters by all states. Truck performance also influences geometric design. The weight/horsepower ratios of existing heavier vehicles impose physical limitations on the speeds of these vehicles on level and on grade and limit their passing performance. Only two states appear to legally control the maximum weight/horsepower ratio, even though legal regulation of this ratio has been recommended by both AASHO and FHWA (BPR) at various times.

Truck braking capability, as it affects stopping-sight distances, passing operations, performance on downgrades, etc., influences highway geometrics. In view of the intimate relationship between truck performance and highway geometric design and operations, it appears judicious to consider some legal limits on the performance capabilities of heavier vehicles, because they do restrict operation of vehicles in mixed traffic and the flow of traffic, particularly on grades.

Horizontal and vertical alignments are two principal design elements that appear to be influenced by truck dimensions and performance. Although existing data are inadequate for making conclusions as to the ultimate effects of truck characteristics on highway geometrics, some tentative conclusions may be drawn.

Geometric Design—Horizontal Alignment

The offtracking characteristics of trucks and truck combinations should be considered in determining the roadway design criteria for pavement widening of horizontal curves on rural highways, pavement width and radius of curvature of on- and off-ramps of freeways and Interstates, design of intersections, and minimum median widths of divided highways where truck turns are to be permitted.

Sight-distance requirements dictate criteria for vertical and horizontal curves as well as passing zones, and are directly related to braking distance of both trucks and private automobiles.

Lane widths may be influenced if the legal width of trucks and buses is increased. Studies of the transverse positions of truck and car mixed traffic have been conducted to establish present pavement width standards, but these arrived at no judgment factors as to how an increase in vehicle width would affect these standards.

Offtracking Characteristics of Trucks and Truck Combinations

The definition of offtracking has many variations; for example:

- "In general, offtracking is defined as the difference in the path of the first inside front wheel and of the last inside front wheel as a vehicle negotiates a curve" (*97*).
- "Offtracking is the difference in the path of the inside front wheel and of the inside rear wheel as a vehicle or combination vehicle negotiates a curve" (*98*).
- "The difference in radii from the turning center to the vehicle centerline at the foremost and at the rearmost axles of a vehicle or combination is called offtracking" (*37*).
- "Offtracking is the difference in radii from the turning center to the vehicle centerline at the foremost and rearmost axles of a vehicle or combination, and represents the increase beyond the tangent track occasioned by a turn" (*99*).
- "Offtracking is the path of the outside of the outer tire on a rear or trailing axle that deviates inward toward the center of a turn from the circular path of the outside of the outer front tire, while the vehicle or trailer combination is making a turn" (*100*).

Figure 29 shows the equivalence of these various definitions. The basic three dimensions are essentially the same value, and differ only in the end reference points.

The turning track width is the radial distance between the turning paths of the outer front tire and the outside of the rear tire nearest the center of the turn. This path is the sum of the maximum offtracking distance and the outside width of the tires on the same axle. (In this definition, as well as those for offtracking, it is assumed that this tread is the same for all axles.)

Offtracking varies directly with wheelbase of a unit and inversely with the radius of turn. The magnitude of offtracking is affected in combinations by the number and location of articulation points, by the length of the arc and the type of curve, and by the speed and turnability of the wheels. Variations of driver skills, inflation and condition of tires, loads on steering axles, amount of superelevation, velocity and direction of wind, speed of approach of vehicle, and pavement condition also affect offtracking.

Offtracking is determined by two general approaches:

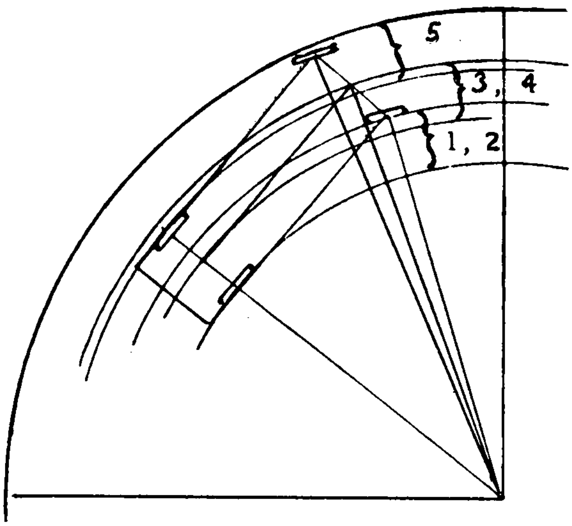


Figure 29. Equivalence of various definitions of offtracking. Source: WHI (101).

(1) analytically by models, graphs, and equations, and (2) by road tests using actual vehicle combinations. Only in road tests can some of the factors mentioned in the previous paragraph be accommodated.

Offtracking determination by models, such as the tractrix integrator, uses scaled devices to trace the wheel paths and centerlines on scaled drawings. This means is most flexible in determining offtracking associated with compound curves. The resulting product is useful in analyzing the effects of offtracking on certain geometric designs such as intersections.

Mathematical determination of offtracking generally is simple and unsophisticated. It is based on the solution of the triangle formed between the turning radius of the front and rear wheels and the vehicle wheelbase when the combination is in a circular turn (*99*):

$$OT = \sqrt{WB^2 + (\sqrt{TR^2 - WB^2} - HT)^2} - \sqrt{TR^2 - WB^2} + HT$$

in which

OT = offtracking;
WB = wheelbase;
HT = ½ front wheel track; and
TR = turning radius.

In addition to the difficulty of applying this simple equation to compound curves, the method does not easily indicate the point where maximum offtracking occurs. It also does not apply to very restrictive turns where the center of the turning radius is less than the combination wheelbase.

A short form of the equation, which drops the consideration of one-half of the front wheel track, has been suggested (*101*):

$$MOT = R_1 - R_1^2 - \sqrt{(L_1^2 + L_2^2 L_3^2 + L_4^2 + L_5^2 + \ldots)}$$

in which

MOT = maximum offtracking;
R_1 = turning radius of outside front wheel;
L_1 = wheelbase of tractor;
L_2 = wheelbase of first trailer or semitrailer;
L_3 = distance between rear axle and articulation point (pintle hook);
L_4 = distance between articulation point and front axle of next trailer; and
L_5 = wheelbase of trailer.

A further simplification is:

$$MOT = R_1 - \sqrt{R_1^2 - \Sigma(L)^2}$$

Figure 30 shows the general relation between maximum offtracking, and the sum of the squares of the vehicle wheelbases and turning radii using the simplified equation. Table 36 indicates maximum offtracking of typical truck combinations with indicated dimensions.

These data relate to wheel tracks only. Not accounted for in these calculations is the projected swept area due to vehicle overhangs between wheels and bumpers and projections outside of the wheel tread. These projected areas obviously affect the clearances required on the roadway between passing vehicles on turns and fixed objects along the roadway.

Pavement Widening of Curves

On earlier highways with narrow pavements and sharp curves it was common practice to consider widening the pavements so that operating conditions on curves approached those on tangents. This was considered desirable because any vehicle occupies a wider path in negotiating the curve because of offtracking and because of the influence of "slip angle" of tires with respect to the direction of travel. "Slip angle" depends on speed, tire tread design, and the amount of friction developed to counteract the centrifugal force not compensated by superelevation. In widening curve pavements a further consideration is the driver's feeling of security in steering the vehicle through the curve at various speeds.

AASHO Policy (*102*) regarding pavement widening on curves on rural highways suggests that a minimum widening of 2 ft be used on restrictive curves. No widening is recommended for 24-ft-wide pavements where curves are 10° or flatter. Table 38 gives calculated and design values for pavement widening on open highway curves for one- or two-way two-lane pavements. This table contains the note: "Where semitrailers are significant, increase tabular values of widening by 0.5 [ft] for curves of 10 to 16 degrees, and by 1.0 [ft] for curves 17 degrees and sharper." Thus, offtracking characteristics of other combinations, as well as their widths, will have a similar influence if the total path (track) width is correspondingly greater than the SU design vehicle used to arrive at the values in the table. For curves less than 10° (greater than 400-ft radius), differences in track widths among the SU, WB-40, and WB-50 design trucks are considered insignificant. (For design vehicle dimensions, see Table 37.)

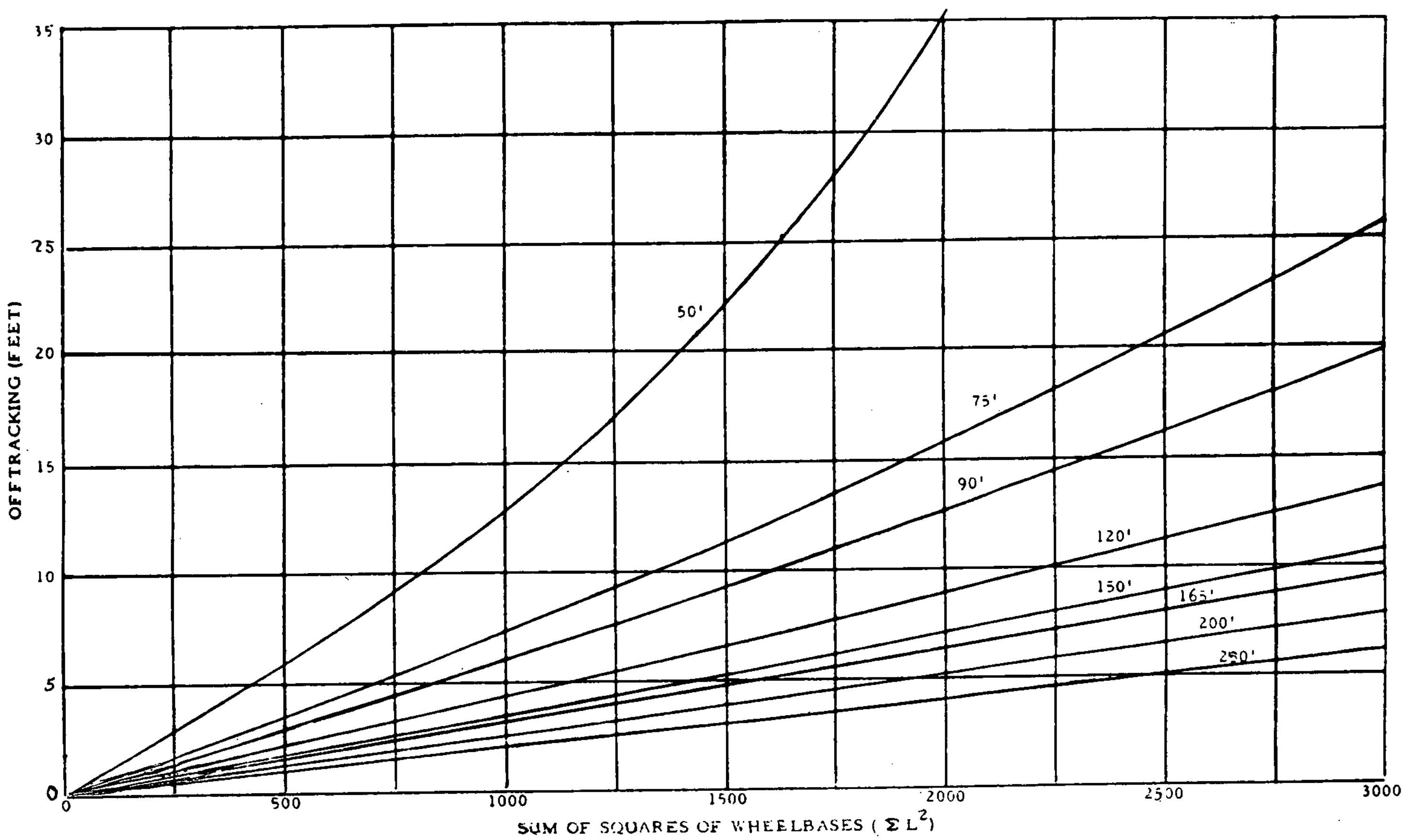


Figure 30. Maximum offtracking of vehicles according to the squares of their wheelbases and radius of curvature. Source: WHI (101).

Figure 31 shows the derivation of Table 38 values. It can be concluded that an increase in legal vehicle width of 0.5 ft would increase the track width by at least that amount, and possibly more if front and rear overhang is of the same proportion as in present vehicles. Thus, two-lane pavement widening on curves would be increased by at least 1 ft. With curves greater than 17°, the increase would approach 2 ft. The single-unit design truck assumed by the AASHO Policy has a wheelbase of 20 ft, front overhang of 4 ft, rear overhang of 6 ft, over-all length of 30 ft, and a width of 8 ft (96 in.).

The AASHO Policy also provides for doubling these design width values for curves on four-lane undivided highway pavements.

Vehicle Roll-Over

Curvature of most rural highways is limited by the amount of lateral load factor permissible in excess of that reacted by superelevation of the roadway. For almost any passenger car, any passenger-limited lateral acceleration produces roll moments far below the critical value for the vehicle (*103*).

However, loaded trucks generally have a higher center-of-gravity (CG) to tread width ratio. Such a factor deserves special consideration, for the lateral forces could produce a near-to-critical roll moment for certain speeds and curve radii. Measurement of CG height is impractical in normal operations. Loading practices of the vehicle operator vary, as does the nature of the load. Regulation of this factor appears remote, but speeds of such vehicles should be carefully controlled to a value below this near-to-critical point.

Pavement Widths at Intersections

Pavement widths of turning roadways at intersections are governed by volumes of turning traffic, types of vehicles to be accommodated, and whether one- or two-way operation is permitted. Widths also depend on whether passing of stalled vehicles on the turning section is permitted.

Table 39 gives basic design policy recommended by AASHO for pavement widths of turning roadways. Three traffic conditions and three cases of traffic operation are cited. Full clearances assumed for the design vehicle combinations are:

	DESIGN TRAFFIC CONDITION		
	A	B	C
Case I	P	SU	WB-40
Case II	P-P	P-SU	SU-SU
Case III	P-SU	SU-SU	WB-40-WB-40

TABLE 36

MAXIMUM OFFTRACKING OF VARIOUS TRUCK COMBINATIONS

TYPE	PROFILES	SYMBOL	OVERALL LENGTH FT.	LENGTH EACH TRAILER FT.	WHEELBASE AND HITCH DISTANCE FT. AB	BP_1	P_1C	CP_2	P_2D	DE	EP_3	P_3F	FG	MAXIMUM OFFTRACKING 165' CURVE RADIUS FT
Single Unit Truck		3	40		33									3.4
3-Axle Tractor-Semitrailer		2-S1	40	27	10	25								2.3
4-Axle Tractor-Semitrailer		2-S2	50	40	11	33								4.0
5-Axle Tractor-Semitrailer		3-S2	50	40	11	33								4.0
5-Axle Tractor-Semitrailer		3-S2	55	40	16	33								4.2
5-Axle Tractor-Semitrailer		3-S2	60	45	16	38								5.4
5-Axle Tractor-Semitrailer		3-S2 Stinger	65	40	17	7	29							3.4
5-Axle Truck and Trailer		3-2	60	27	19	4	13	20						2.4
5-Axle Truck and Trailer		3-2	65	30	21	4	13	23						3.5
5-Axle Doubles		2-S1-2	65	27	10	20		3	6	21				3.0
7-Axle Triplets		2-S1-2-2	95	27	10	20		3	6	21	3	6	21	4.5
9-Axle Doubles		3-S2-4	100	40	16	30		3	6	34				8.1

Source: WHI (101, pp.46-47).

TABLE 37

DESIGN VEHICLE DIMENSIONS

Design vehicle		Dimensions in feet					
Type	Symbol	Wheel-base	Front over-hang	Rear over-hang	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

Source: AASHO (*102*, p. 86).

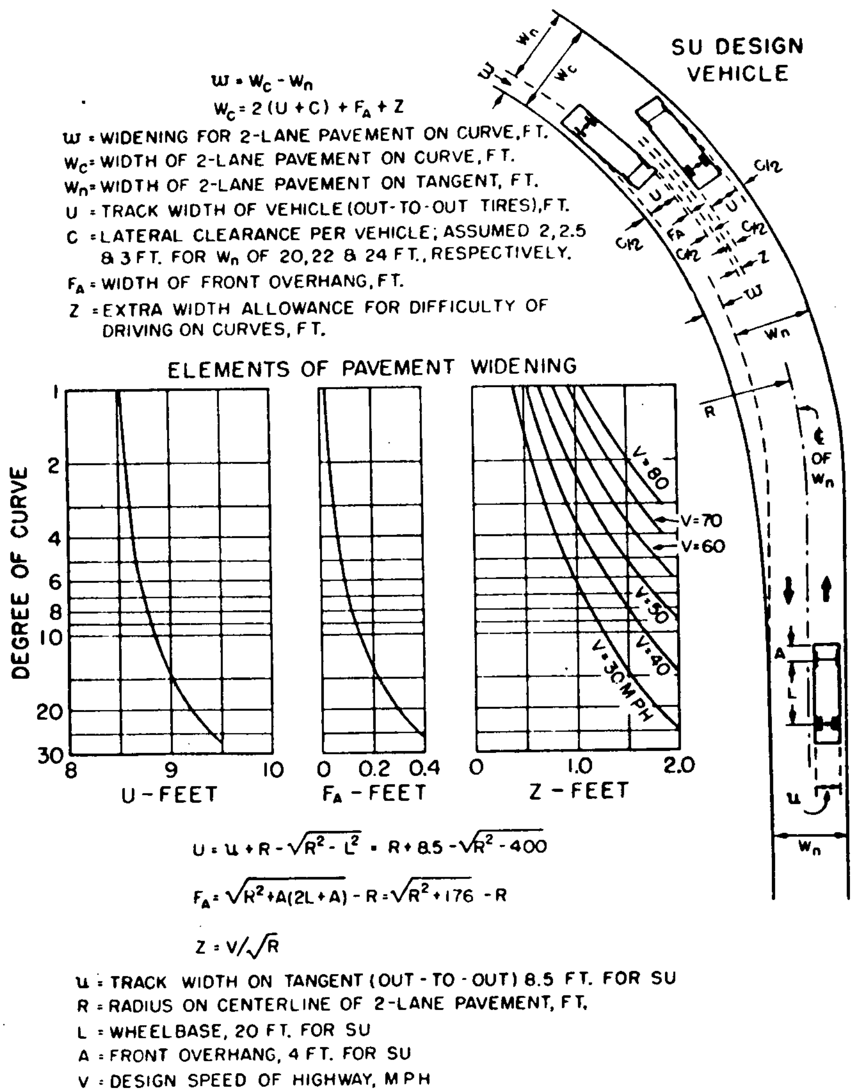


Figure 31. Pavement widening on curves: basis of derivation. Source: AASHO (102, p. *184*).

TABLE 38

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.

Source: AASHO (*102*, p. 186).

This indicates partial clearance for the following traffic operational conditions: full clearance for either one-way traffic movement with no provision for passing (Case I); one-way traffic movement with provision for passing stalled vehicle (Case II); or two-lane operation either one- or two-way (Case III). For instance, width specified for Case II, Condition A, for a radius on inner edge of pavement of 150 ft is 14 ft, a width that will allow a passenger car to pass another stalled passenger car, as indicated by P-P symbol. An additional 2-ft width in the turning roadway, or 16 ft, would be required to permit a passenger car (P) to pass the single-unit (SU) design truck.

Partial clearance, defined as that operating condition requiring shorter lateral clearance between vehicles (such as lower speeds, and more caution and skill by the drivers) is given in the AASHO Policy as:

	DESIGN TRAFFIC CONDITION		
	A	B	C
Case I	WB-40	WB-40	WB-50
Case II	P-SU	P-WB-40	SU-WB-40
Case III	SU-WB-40	WB-40-WB-40	WB-50-WB-50

These design widths are derived from the relationships shown in Figure 32.

The roadway width for turning roadways, as distinct from pavement widths given previously, includes shoulders or equivalent lateral clearance outside the edge of traveled pavement. These vary as a function of whether the curve is on level terrain or in a cut or on fill. Generally, at ground level, for short right or left turns within a channelized intersection, a lateral clearance for the outside pavement edge of at least 2 ft is essential, and 4 ft is desirable. If the turn is at ground level, of intermediate or long length, or is in a cut or on fill, the recommended minimum clearance is 4 ft for the left edge and 6 ft for the right edge. Desirable clearances are 6 to 10 ft at left and 8 to 12 ft at right.

Clearances recommended at underpasses and at overpasses follow those shown for typical situations in Figures 33 and 34, respectively.

This discussion of pavement widths for both long curves and for turning roadways uses AASHO Policy figures, which may or may not conform to a specific state's design standard. Differences may occur where the critical design vehicle assumed by the state differs from that used in developing the AASHO Policy.

Measurement of the impacts of curves and turns on design features must begin by computing and comparing proposed vehicle widths and offtracking characteristics with those of the critical design vehicle. Changes in turning widths can be computed as a result of this comparison. Costs to meet the additional width could then be assigned to those vehicles benefitting from the legal limit changes.

Such changes not only affect additional pavement, but also generate requirements for additional bridge deck widths, and additional cuts and fills required to maintain adequate lateral clearances. Such costs must be included.

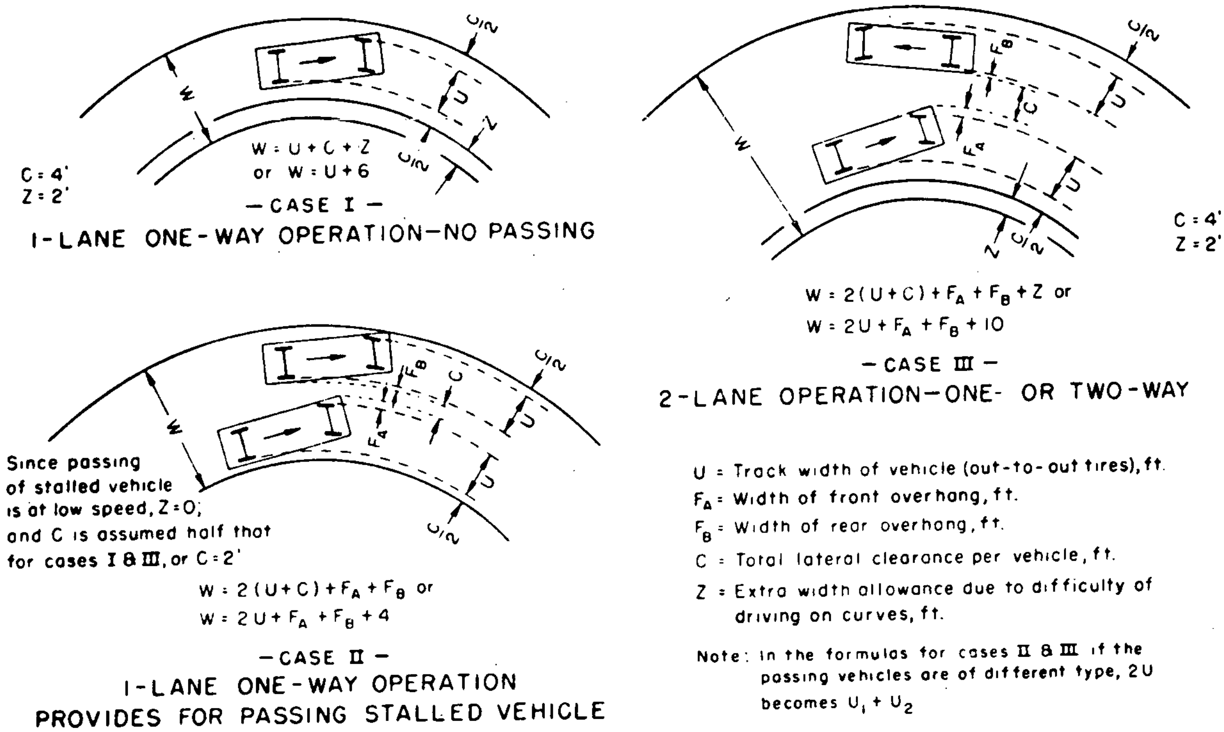


Figure 32. Pavement widths on curves at intersections: basis of derivation. Source: AASHO (102, *p. 333*).

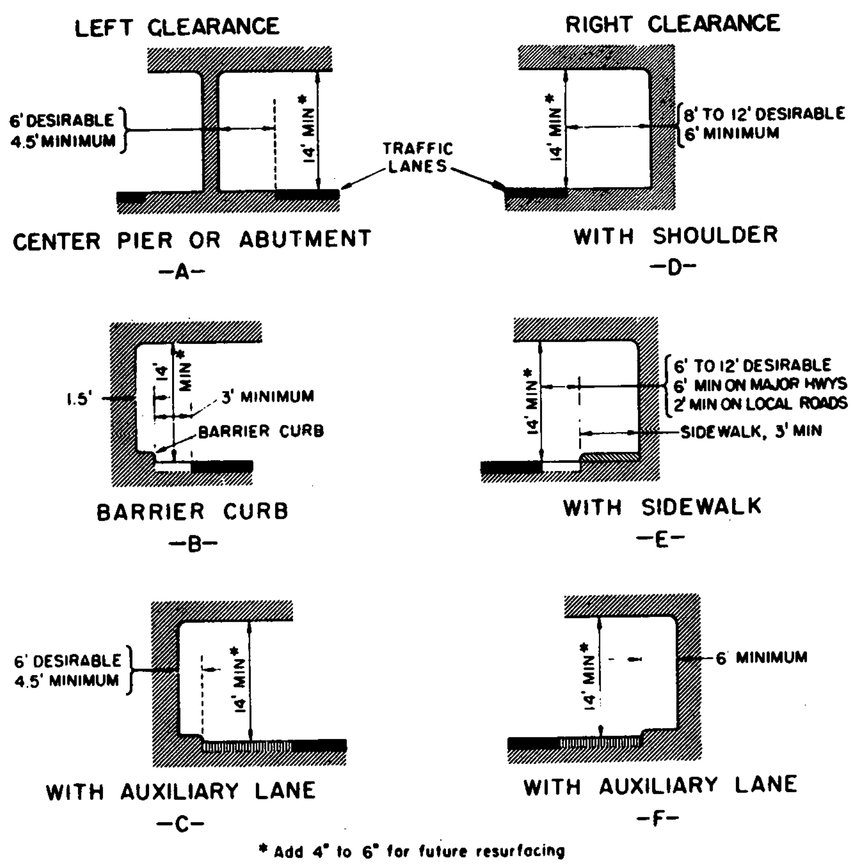


Figure 33. Clearance at underpasses. Source: AASHO (102, *p. 511*).

TABLE 39

DESIGN WIDTHS OF PAVEMENTS FOR TURNING ROADWAYS

R Radius on inner edge of pavement, feet	Pavement width in feet for: Case I 1-lane, one-way operation—no provision for passing			Case II 1-lane, one-way operation—with provision for passing a stalled vehicle			Case III 2-lane operation—either one-way or two-way		
	Design traffic condition								
	A	B	C	A	B	C	A	B	C
50	18	18	23	23	25	29	31	35	42
75	16	17	19	21	23	27	29	33	37
100	15	16	18	20	22	25	28	31	35
150	14	16	17	19	21	24	27	30	33
200	13	16	16	19	21	23	27	29	31
300	13	15	16	18	20	22	26	28	30
400	13	15	16	18	20	22	26	28	29
500	12	15	15	18	20	22	26	28	29
Tangent	12	15	15	17	19	21	25	27	27

Width modification regarding edge of pavement treatment:

	Case I	Case II	Case III
No stabilized shoulder	None	None	None
Mountable curb	None	None	None
Barrier curb: one side	add 1′	None	add 1′
Barrier curb: two sides	add 2′	add 1′	add 2′
Stabilized shoulder, one or both sides	None	Deduct shoulder width; minimum pavement width as under Case I	Deduct 2′ where shoulder is 4′ or wider

Traffic Condition A—Predominantly P vehicles, but some consideration for SU trucks; values in table VII-7 are somewhat above those for P in table VII-6.

Traffic Condition B—Sufficient SU vehicles to govern design, but some consideration for semitrailer vehicles; values in table VII-7 for cases I and III are those for SU in table VII-6. For case II, values are reduced as explained below.

Traffic Condition C—Sufficient semitrailer, WB-40 or WB-50 vehicles to govern design; values in table VII-7 for cases I and III are those for WB-40 in table VII-6. For case II, values are reduced.

Source: AASHO (*102*, p. 338).

Lane Widths

Studies have shown that, from ideal standards of driver convenience, and ease of operation and safety, it is desirable to construct all highways with lanes 12 ft wide and usable shoulders of 6 to 10 ft (*104*). But to establish this as an absolute minimum lane width is impractical with limited resources committed to highway construction and maintenance. Marginal lane widths less than 12 ft can be operationally acceptable if traffic volumes are low, if only moderate truck volumes are in the traffic stream, and if relatively low design speeds are feasible. Pavement widths of 20 ft, and under some circumstances 18 ft, can be acceptable for some two-lane roads.

The report of a basic study performed in 1945 (*105*) is still a prominently referenced work on the transverse placement of vehicles as influenced by roadway width. Extensive speed-placement studies performed on instrumented sections of highways of various widths in 10 states were reported therein. At that time, 95 percent of the 256,000 miles constituting the "dustless" primary rural state highway system were two-lane roads, and 48 percent of these were less than 20 ft wide. Therefore, the availability of 24-ft two-lane highways limited the study to one sample site for measuring traffic conditions related to that width. This is not to detract from the value of the study, for later experience and studies have confirmed its conclusions.

One of the main purposes of the study was to observe, record, and analyze driver behavior as reflected in the choice of speeds, positions in lanes under free-moving traffic conditions, and placement in the lane when meeting opposing vehicles and passing vehicles traveling in the same direction.

Results indicated that free-moving vehicles travel closer to the centerline than to the edge of the pavement, regardless of lane width. Except when traffic volumes were large,

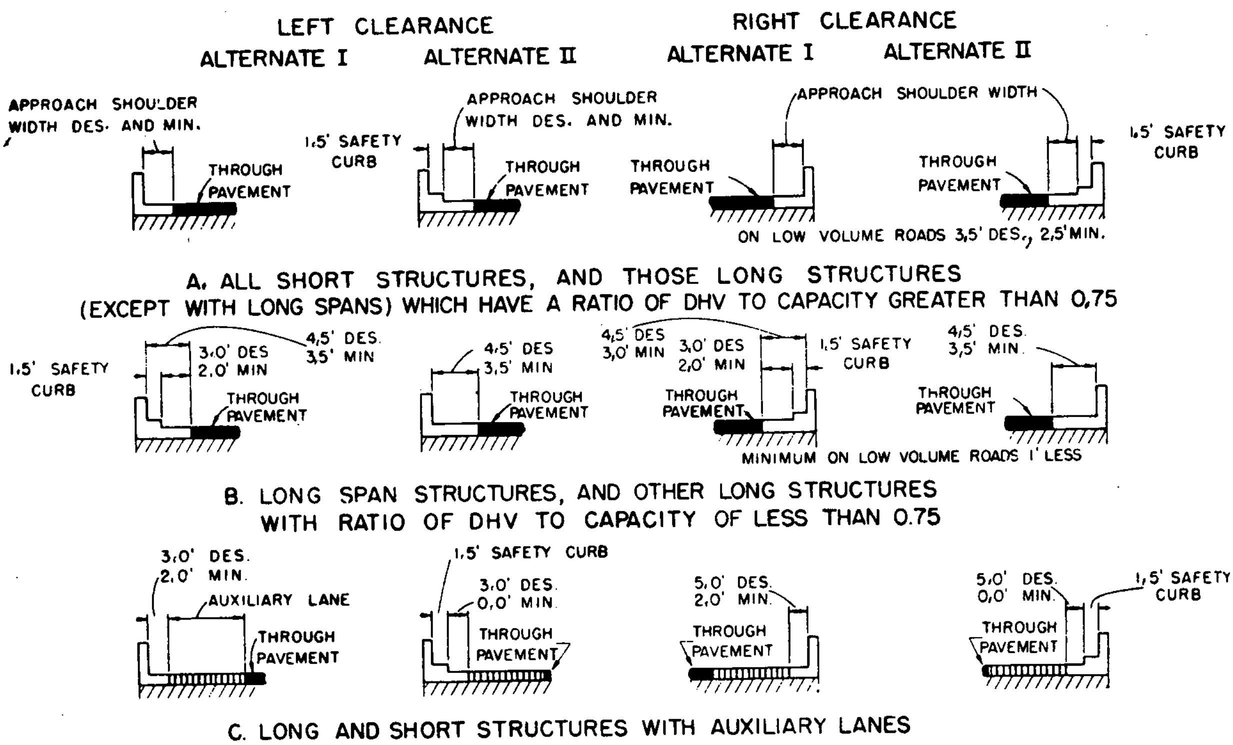


Figure 34. Clearance at overpasses. Source: AASHO (102, *p. 516*).

there was always a definite difference in lane placement of free-moving vehicles when compared with placement of vehicles meeting opposing traffic. Body and edge clearances for meeting or passing vehicles were identified as critical factors in judging the adequacy of pavement width. It was found that drivers of commercial vehicles traveled closer to the centerline when meeting other commercial vehicles on 18-ft pavements than on 20-ft pavements. Only on 24-ft pavements were drivers apparently satisfied with both edge and center clearance, as indicated by their choice of a placement near the center of their lanes.

Drivers of passenger cars preferred a body clearance of about 5 ft when meeting other passenger cars, or a clearance of about 3 ft between their left front tire and the centerline. Such clearances cannot be achieved until pavement reaches 22 ft (11-ft lanes).

It was concluded that pavement widths adequate to accommodate meeting-vehicle clearances were more than adequate for passing maneuvers. The report also concluded that desired clearances for mixed traffic required a 24-ft pavement (12-ft lanes).

The FHWA conducted a test track investigation in daylight and under free-flowing conditions to consider whether lateral clearance is related to steering control behavior. The criterion for selective judgment in lane placement in this human factors study (*106, 107*) was basically the same as that in the 1945 study.

The driver uses laterally angular velocity to determine the placement of his vehicle when he meets another vehicle or object. The study measured the threshold of a number of drivers to detect or perceive this angular velocity by a change in lane placement. From test data, the probability of a deviation from present lane position (lane placement change) was determined using 10 experienced drivers. Various approach speeds and apparent vehicular widths were incorporated into the test conditions. Lateral body clearances of 4.6 ft at the relative velocity of 154 fps were required to exceed the threshold of angular velocity detection. With 4.6-ft clearance on 11-ft lanes, the frequency of change in the lane placement was 84 percent of the time. On 12-ft lanes, the frequency was 42 percent.

It was found that small changes in apparent approaching vehicle width caused large changes in the frequency and magnitude of lateral lane placements. Pavement markings did not significantly reduce lateral displacement of meeting vehicles.

Capacity of a highway for uninterrupted flow conditions is affected adversely by a decrease in lane width. Using a 12-ft lane as a standard, an effective lane width of 11 ft on two-lane highways reduces the capacity to 88 percent; for a 10-ft lane, capacity drops to 81 percent; and a 9-ft lane has only 76 percent of a highway with 12-ft lanes. On multilane highways, the reductions are, respectively, 97, 91, and 81 percent (*104*).

The incidence rate of accidents before and after pavement widening was shown to improve from 21.5 to 46.6 percent, depending on original accident rate and original traffic volume (*108*).

On four-lane divided highways, AASHO Policy recommends a 12-ft lane, with 13-ft lanes considered for highways that accommodate many large combinations. However, with a wider-than-12-ft lane, drivers tend to use a single lane as two effective lanes in some situations. That accident incidence rates tend to increase on 13-ft lanes, as compared to 12-ft lanes, is an indication of this practice.

Shoulders and Lateral Side Clearance

Shoulders provide an emergency escape for vehicles and a place to stop out of traffic when the vehicle fails. They structurally support the main pavement edge and control pavement drainage. On narrow-lane roadways, trucks (and, to some extent, passenger cars) tend to use shoulders as the traveled way when meeting other vehicles. Adequate shoulders give the driver a sense of security and thereby enhance traffic flow.

On highways with 12-ft lanes, however, the relationship of shoulder width to truck dimensions appears to be governed by the width required for truck emergency stops clear of traffic. Wide trucks on adequate shoulders aggravate the situation. But any vehicle or grouping of vehicles that appears in distress or in an accident situation slows traffic because of the "rubbernecking" of curious drivers.

Signing

Signing is a means of communication with all highway users. Its effectiveness is reduced by vehicles that block the sign from view of all drivers. Trucks contribute significantly to this blockage, particularly on multilane highways with more than one lane of travel in each direction.

The probability of blockage of signs has been analyzed from the standpoint of the geometry of the problem (*109*). A driver's vision will be blocked if a vehicle comes between him and the roadside sign. The shape and speed of this "shadow" is a function of truck and other vehicle speed, truck size and position, and sign size. Lane widths, road length and geometry, and position of driver's line of sight also influence the problem. The solution of this problem appears very general because of the several random variables involved. The relationship between the probability of blockage, ADT, and percentage truck mix was not apparent in the study; more work was suggested by the investigator.

The severity of consequences of sign blockage would depend on the relative value and importance of the communication being attempted, and could be an additional basis for warrants for overhead placement of signs and increasing the number of signs carrying the same message. It is recommended that research be extended toward that objective.

Driver Visibility

Sight distance is a critical design factor in determining distances required for passing on two-lane highways and for stopping. The distance traveled between the point an object is visible to the driver, whose vehicle is proceeding at or near design speed, and the point at which he can stop his vehicle is often crucial. Conventionally, the time is broken down into components of perception and brake reaction time and braking distance.

Sight distance is a function of geometry of roadway. On vertical curves and in some horizontal alignments, eye-level height limits the sight distance. The configuration and elevation of trucks usually provide an increased sight distance because of higher driver eye level.

Perception and brake reaction time varies with skill and attentiveness of drivers. The usual average value for this combined time is 2.5 sec, a conservative figure from results of limited experimental data.

Vehicle Braking Distance

A standard formula, used for computing braking distance in the AASHO Policy, is:

$$d = V^2/30F$$

in which

d = stopping distance, feet;
V = initial speed, mph; and
F = coefficient of friction.

To account for gradient of roadway, this equation may be modified (*103*):

$$d = \frac{V^2}{30(F + g)}$$

in which g is percent of grade.

The coefficient of friction varies with tire tread design and wear, and types and conditions of roadway surfaces, including whether wet or dry. The AASHO Policy assumes conservative coefficients in computing data given in Table 40.

Braking Performance of Trucks and Buses

Figure 35 compares braking distance of truck combinations from actual road tests of controlled trucks with the braking distance in the AASHO Policy. Tests were conducted on trucks in excellent state of maintenance. Skilled mechanics constantly checked and adjusted brakes. Tires and equipment were new, or relatively so. Drivers were picked carefully. As a result, the data shown probably represent optimum values. Deterioration in any of these factors is known to occur in normal operations. The brake tests conducted by BPR were panic stops from only 20 mph, which falls below the lowest speed in the AASHO Policy. Combining these factors, it appears that some reconsideration of these distances is in order. AASHO Policy states that the values given reflect passenger-car operation. It points out that the eye levels of truck drivers are higher than those of passenger-car drivers. Trucks are conceded to be generally slower than passenger cars. The two factors are claimed to offset the braking distance differential between the two types of vehicles. Greater stopping distances were desirable.

Relationship Between Braking Performance and Accident Incidence.—The complex relationship between braking capabilities of current commercial vehicles and the frequency of accidents involving these vehicles is neither completely understood nor statistically documented. Based on the hypothesis that such a relationship does exist, the National Highway Traffic Safety Administration, DOT,

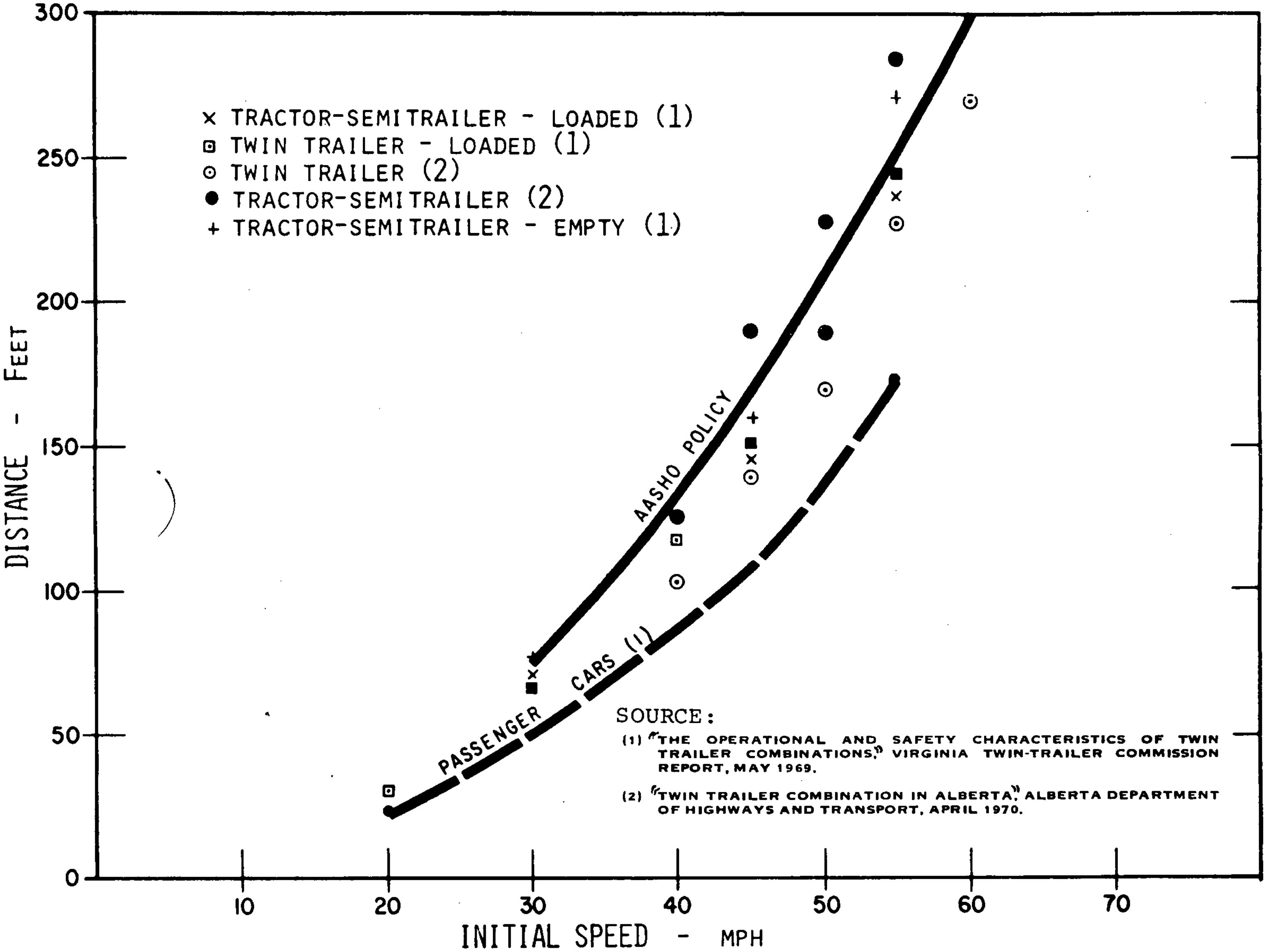


Figure 35. Comparison of braking distance of truck combinations from tests with braking distance in AASHO Policy.

initiated the "Bus, Truck and Tractor-Trailer Braking System Performance Study" (*110*) to: (1) determine, by vehicle testing, the range of braking performance currently exhibited by these vehicles; (2) establish their maximum braking performance, based on existing brake system technology; and (3) recommend a national braking standard.

Ten baseline vehicles equipped with standard braking systems were tested for minimum stopping distance, and maximum deceleration, loaded and empty. The results of these effectiveness tests are given in Table 41 and shown in Figure 36.

The vehicles also were tested for various modes of brake failure, where deceleration performance and stopping distance was determined (Table 42).

Fade and recovery tests were conducted where all vehicles were decelerated from test speed at 15 ft/sec^2 to 10 mph and then accelerated back to test speed. These cycles were repeated until the specified deceleration rate could no longer be achieved. The city bus and the tractor-trailers failed to fade after 10 cycles, and tests were discontinued after this series of runs.

Brake response time of tractor-trailer combinations was measured by using pressure transducers at various points in the system. Response time for brake application was determined as the time from the beginning of pressure rise at the treadle valve to the time the pressure at a given axle reached 60 psi. Release time was measured from the time the pressure began to drop at the output of the treadle valve to the time pressure at a given axle dropped to 5 psi.

Brake rating tests were conducted by towing the test vehicle on a flat surface at a constant velocity with braking force equivalent to that required to maintain constant velocity on a 7 percent descending grade. Towing force, determined by a towbar dynamometer, was averaged and corrected for rolling resistance, and used to calculate

TABLE 40

MINIMUM STOPPING SIGHT DISTANCE

Design speed	Assumed speed for condition	Perception and brake reaction: Time	Perception and brake reaction: Distance	Coefficient of friction	Braking distance on level	Stopping sight distance: Computed	Stopping sight distance: 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.
Source: AASHO (*102*, p. 136).

average braking force, energy (horsepower) absorption rate of the brake, and total energy absorption.

Trucks and tractor-trailer components were tested with various advanced systems. These included trucks with disc brakes and tractor-trailers equipped with proportioning valves, adaptive brakes to prevent lock-up, and a trailer

TABLE 41

EFFECTIVENESS TEST SUMMARY, BASELINE VEHICLES

VEHICLE	NOM. V_o (MPH)	MIN. STOPPING DISTANCE (FT): EMPTY: NO [a]	EMPTY: SOME [a]	LOADED: NO	LOADED: SOME	MAX. DECELERATION (FT/SEC^2): EMPTY: NO	EMPTY: SOME	LOADED: NO	LOADED: SOME
Light truck	60	238	150	219	191	20.0	28.0	23.0	25.0
Medium truck	60	322	282	307	—	13.0	13.7	12.6	—
Heavy truck	60	316	248	263	262	13.2	15.8	15.5	15.5
School bus	40	108	84	119	—	19.0	22.0	20.0	—
Intercity bus	60	290	221	202	—	16.3	21.5	19.5	—
City bus	40	93	72	89	—	20.5	24.0	20.3	—
Tractor-trailer 2-S1	60	291	258	292	—	15.5	17.0	15.2	—
Tractor-trailer 2-S2	60	328	320	322	—	15.0	20.0	20.0	—
Tractor-trailer 3-S2	60	366	247	376	299	14.0	16.0	14.0	16.1
Tractor-double trailer	60	395	294	309	—	15.0	18.8	16.6	—

[a] Indicates no wheels locked, or some wheels locked.
Source: Murphy et al. (*110*).

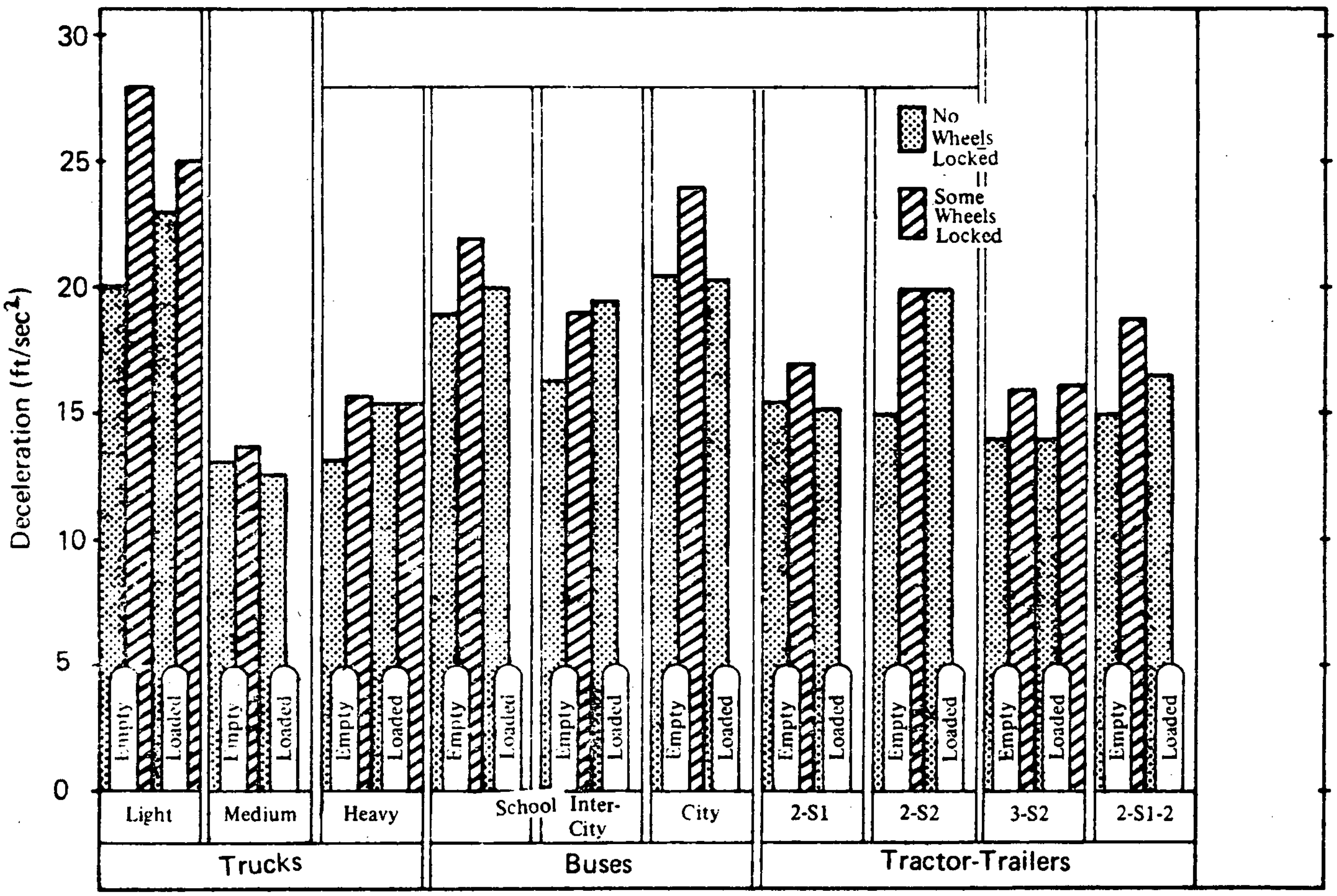


Figure 36. Maximum deceleration capability, baseline vehicles. Source: Murphy et al. (110).

brake synchronizing system. Results of these tests are shown in Figure 37.

Based on these test results, a three-stage program of safety standards for braking systems was recommended (Table 43). Step 1 represents standards that are achievable by current design practice as demonstrated by baseline vehicle performance. Before Step 2 can be implemented, further development and testing of advanced systems, such

TABLE 42

PERFORMANCE OF TRUCKS AND BUSES UNDER FAILURE CONDITIONS

VEHICLE	TYPE OF FAILURE	BRAKES USED	V_o (MPH)	MAX. DECEL. (FT/SEC^2) EMPTY	LOADED	MIN. STOPPING DISTANCE (FT) EMPTY	LOADED
Trucks:							
Light	Front brakes	Rear only	60	11.5[a]	7.3	380	620
	Rear brakes	Front only	60	15.5	9.0	281	484
	Power boost	Service	60	26.0[a]	17.0	160	238
Medium	Power boost	Service	60	13.0[a]	5.5	352	744
Heavy	Service brakes	Hand	60	1.5	1.0	1594	2200
Buses:							
School	Service brakes	Emergency	40	9.2	6.0	188	287
Intercity	Service brakes	Emergency	60	6.8	5.8	571	710
City	Service system	Hand	40	8.1	5.0	212	437
	Rear door opening	Rear	40	8.0	5.1	235	350

[a] Lock-up of one or more wheels.
Source: Murphy, et al. (*110*).

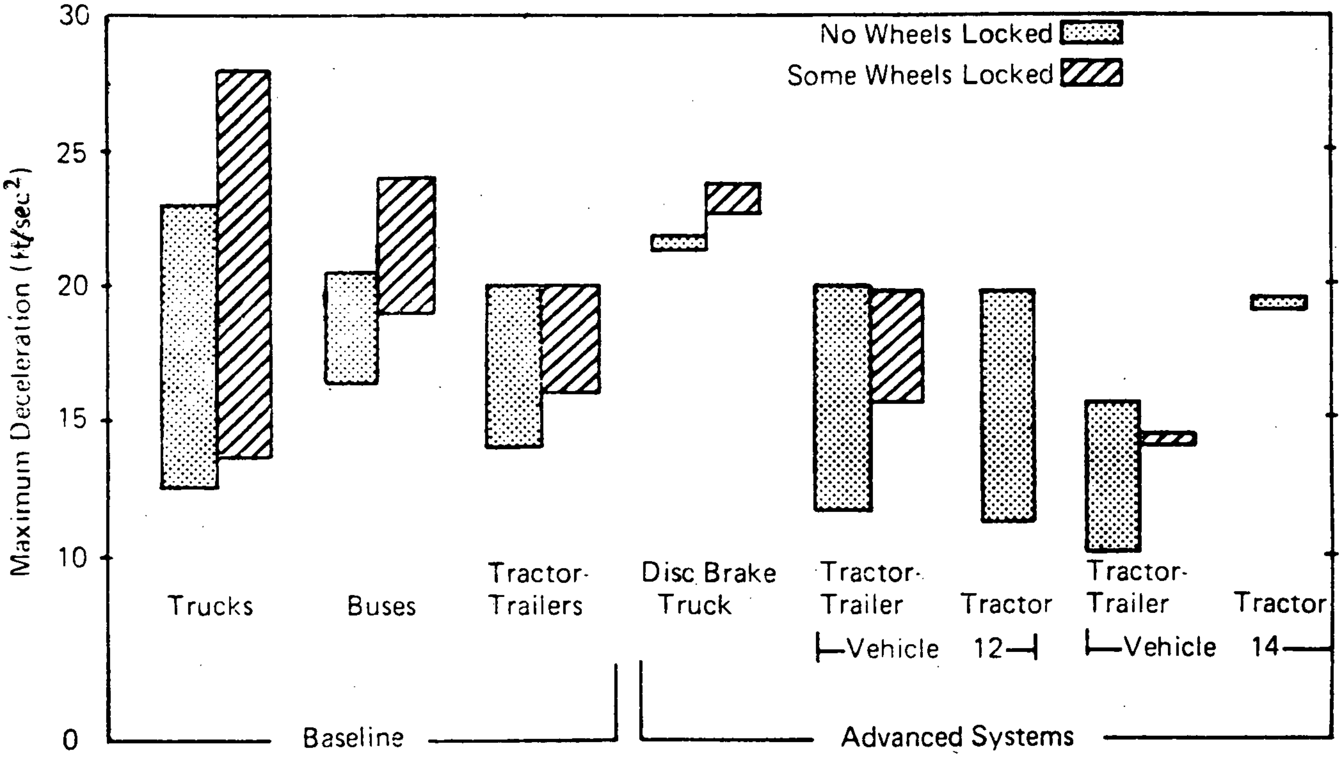


Figure 37. Maximum deceleration performance ranges, baseline and advanced system. Source: Murphy et al. (110).

as load-sensitive proportioning systems for certain vehicles, are required to attain the limit of tire-road interface tractive capabilities of truck tires now available, with due regard to realistic braking efficiencies.

The report also identified certain other factors for serious consideration. More effective brakes will necessitate stronger suspensions and adjacent vehicle structures. Larger brakes on front axles may require new front axles and steering system designs, including, perhaps, power steering. Increasing deceleration capability may require reevaluation of cargo restraint methods for trucks and passenger restraints for buses. Stability of vehicles with high center of gravity, particularly straight trucks, may be critical at higher decelerations, and may require further study and evaluation. Maintainability and reliability of advanced systems in actual service environments require particular attention.

During the testing program, it was observed that tractor-trailer combinations presented a special challenge because of brake balance. A tractor may perform well with one trailer and not with another. Based on this experience, it was recommended that tractors be certified to pull only trailers which, by calculation and test, have been proved compatible.

Passing Maneuvers.—The classic analysis of passing maneuvers on two-lane highways divides the maneuver period into three phases: (1) perception and reaction time; (2) time the passing vehicle occupies the left lane in passing the slower vehicle; (3) time required to return to the right lane. Distances covered during these phases, and the distance and placement of an opposing vehicle, determine the passing sight distance to be allowed in highway design.

The distance traveled during the initial maneuver, d_1, is:

$$d_1 = 1.47t_1\left(v - m + \frac{at_1}{2}\right)$$

in which

t_1 = time of initial maneuver;
a = average acceleration, mphps;
v = average speed of passing vehicle, mph; and
m = difference in speed of passing vehicle and passed vehicle, mph.

The distance passing vehicle occupies the left lane, d_2, is:

$$d_2 = 1.47vt_2$$

in which

t_2 = time passing vehicle occupies left lane, seconds; and
v = average speed of passing vehicle.

The clearance length, d_3, is the distance the passing vehicle must proceed beyond the passed vehicle when the passing vehicle pulls back into the right lane. Passing studies indicate this distance varies between 110 and 300 ft.

The distance traversed by the opposing vehicle, d_4, is assumed to be ⅔ d_2 if the opposing vehicle is traveling at the same speed as the passing vehicle. Figure 38 shows the elements of the passing maneuver and the total passing sight distance for a two-lane highway as a function of passing vehicle speed. Table 44 gives these elements and total for various speed groups and average passing speeds.

As a result of acceleration tests of trucks having weight-to-horsepower ratios of 100 to 400 lb/ghp, the passing time

TABLE 43
THREE-STAGE IMPLEMENTATION OF BUS, TRUCK, TRACTOR-TRAILER BRAKING SYSEM SAFETY STANDARDS

STAGE	MAX DECEL. RATE (FT/SEC^2)	MIN. BRAKING EFFICIENCY (%)	THERMAL CAPACITY	AIR BRAKE RESPONSE TIME (SEC)	SPECIAL SYSTEMS REQUIRED
Step 1: Immediate action program	16	65 [a]	Per SAE J786 specs. @ 15 ft/sec^2 min.	Application: Tractor–0.25. Trailer–0.35. Release: Tractor–0.50. Trailer–0.70.	None
Step 2: Intermediate program	20	75 [a]	Test upgraded to heaviest duty cycles experienced in class of service.	Application: Tractor–0.25. Trailer–0.30. Release: Tractor–0.30. Trailer–0.40.	Static load proportioning system.
Step 3: Ultimate program (performance equaling or approaching passenger cars)	24 [b]	85 [c]	No change from step 2.	No change from Step 2.	Improved tires anti-lock system.

[a] For surfaces having peak truck tire-road friction coefficients between 0.2 and 0.8.
[b] With upgraded tires such that peak truck tire-road friction coefficient is at least 0.85.
[c] For surfaces having peak truck tire-road friction coefficients between 0.2 and 0.9.
Source: Murphy et al. (*110*).

and distance have been graphically determined (see Fig. 39). In this example, it was assumed that a 55-ft tractor-semitrailer traveling at 20 mph was being passed by a 65-ft twin-trailer combination accelerating from 20 to 35 mph. Forty-foot clearance was assumed to apply before and after the passing maneuver.

From acceleration performance data, the passing distance vs weight/horsepower ratios were plotted in Figure 40. The passing time vs weight/horsepower ratios were plotted in Figure 41. The solid curves show the variance of test data and are used to develop the passing performance envelopes shown in Figure 42. For example: a vehicle having 300 lb/ghp ratio probably would require 19.5 sec to pass and a passing distance of 770 ft. However, passing time could vary from 17 to 22 sec, and passing distance could vary from 690 to 830 ft, depending on vehicle condition, environmental conditions, and driver performance.

The passing distance of a twin-trailer combination of 65-ft length and that of a semi-trailer combination of 55-ft length were compared in an American Trucking Associations report (*44*). Using the following relationship for two-lane roads, the passing time is:

$$T_p = \frac{L_f + L_s + 150'}{1.47\ V}$$

in which

L_f = length of faster vehicle (car = 18 ft);
L_s = length of slower vehicle (combination truck: 55 or 65 ft);
$150'$ = 75-ft allowance for pull-out distance and 75-ft allowance for return-to-lane;
1.47 = conversion factor (mph to fps); and
V = speed difference between vehicles (mph).

For four-lane roads, the expression omits the 150-ft allowance for pull-out and return-to-lane.

The expression assumes a constant speed throughout the maneuver, and therefore is maximum time. Acceleration, which is normally encountered by the passing vehicle during the maneuver, will reduce the time.

Application of these equations results in the time differences given in Table 45.

Geometric Design—Vertical Alignment

The impact of unique truck performance characteristics that contrast with similar performance characteristics of other vehicles has been most obvious on the vertical alignment of highways, particularly in regard to grades and truck climbing lanes.

Truck Performance on Grades

Grades, weight/horsepower ratio of trucks, and the tractive effort developed by them on grades must be considered in the establishment of geometric design standards. The speeds at which trucks negotiate grades depend, for example, on the speed at which they enter the grade, and the steepness, length, and condition of the grade.

Huff and Scrivner conducted road tests to develop a curve similar to Figure 12 and comparisons were made

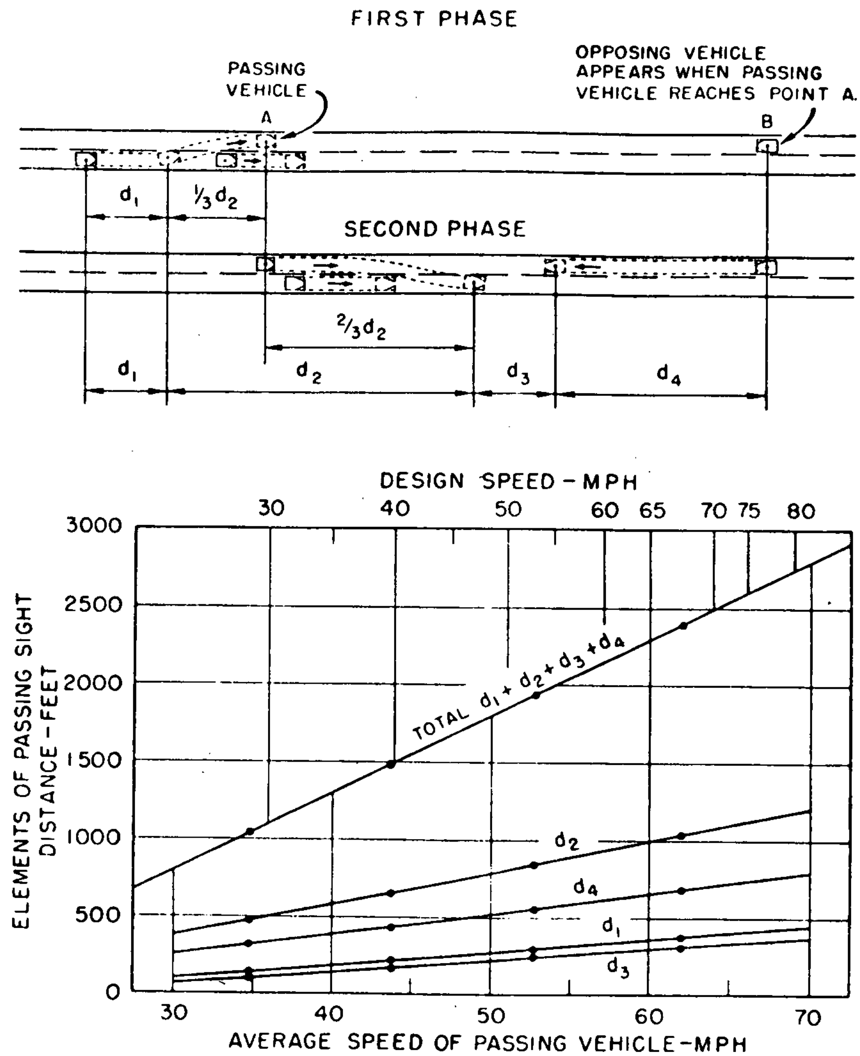


Figure 38. Elements of and total passing sight distance, two-lane highway. Source: AASHO (102, *p. 143*).

TABLE 44

ELEMENTS OF SAFE PASSING SIGHT DISTANCE—
2-LANE HIGHWAYS

Speed group, mph Average passing speed, mph	30-40 34.9	40-50 43.8	50-60 52.6	60-70 62.0
Initial maneuver:				
a=average acceleration, mphps*	1.40	1.43	1.47	1.50
t_1=time, seconds*	3.6	4.0	4.3	4.5
d_1=distance traveled, feet	145	215	290	370
Occupation of left lane:				
t_2=time, seconds*	9.3	10.0	10.7	11.3
d_2=distance traveled, feet	475	640	825	1030
Clearance length:				
d_3=distance traveled, feet*	100	180	250	300
Opposing vehicle:				
d_4=distance traveled, feet	315	425	550	680
Total distance, $d_1+d_2+d_3+d_4$, feet	1035	1460	1915	2380

* For consistent speed relation, observed values adjusted slightly.

Source: AASHO (*102*, p. 144).

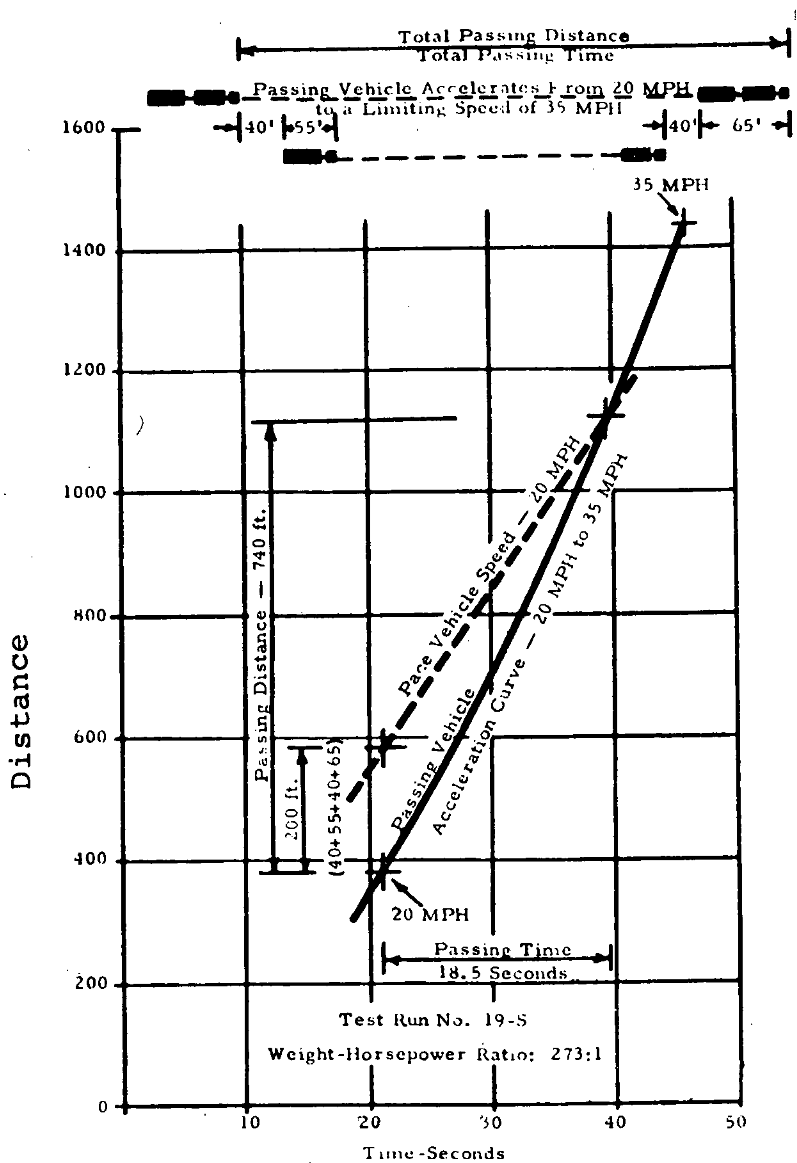


Figure 39. Graphical solution of low-speed pass.

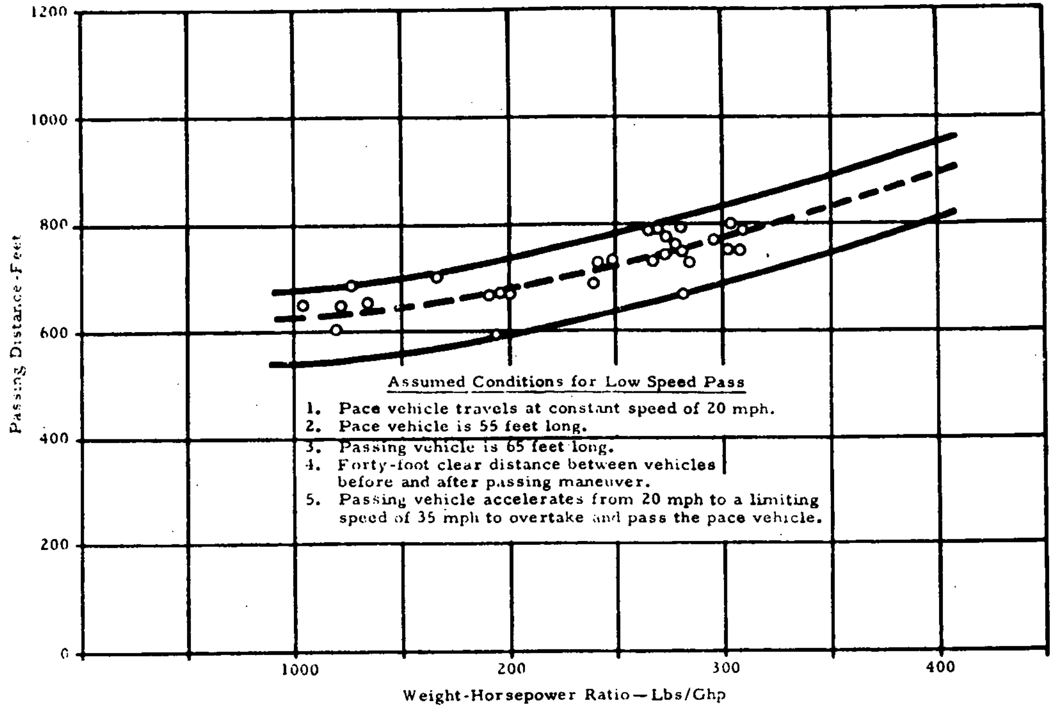


Figure 40. Passing distance for low-speed pass vs various weight/horsepower ratios. Source: WHI (39).

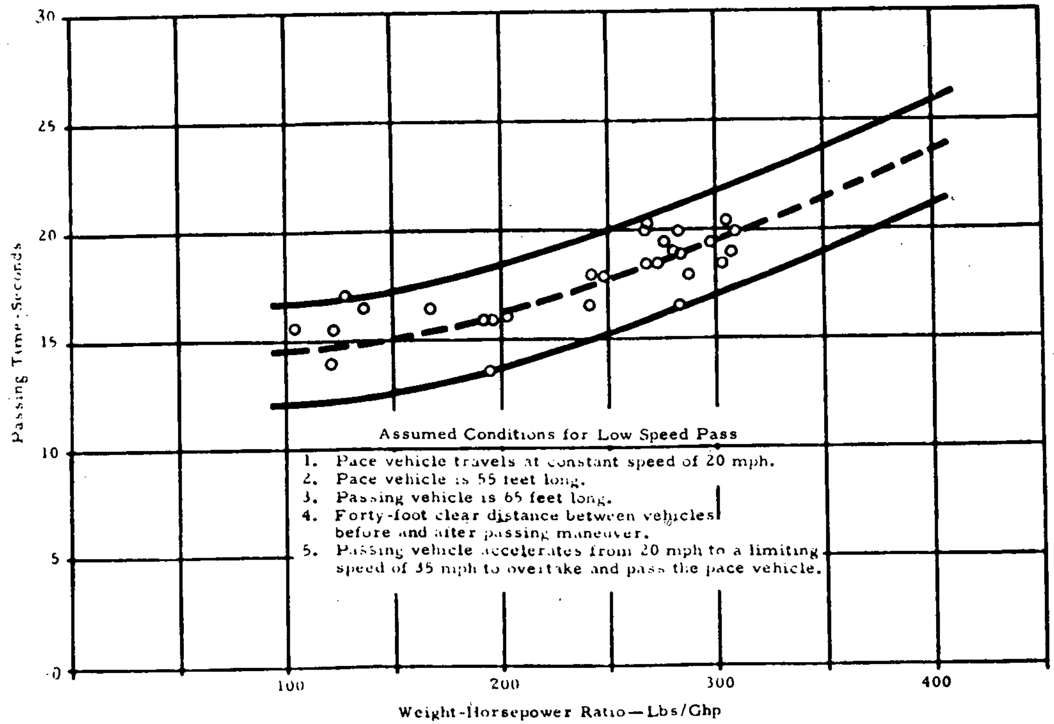


Figure 41. Passing time for low-speed pass vs various weight/horsepower ratios. Source: WHI (39).

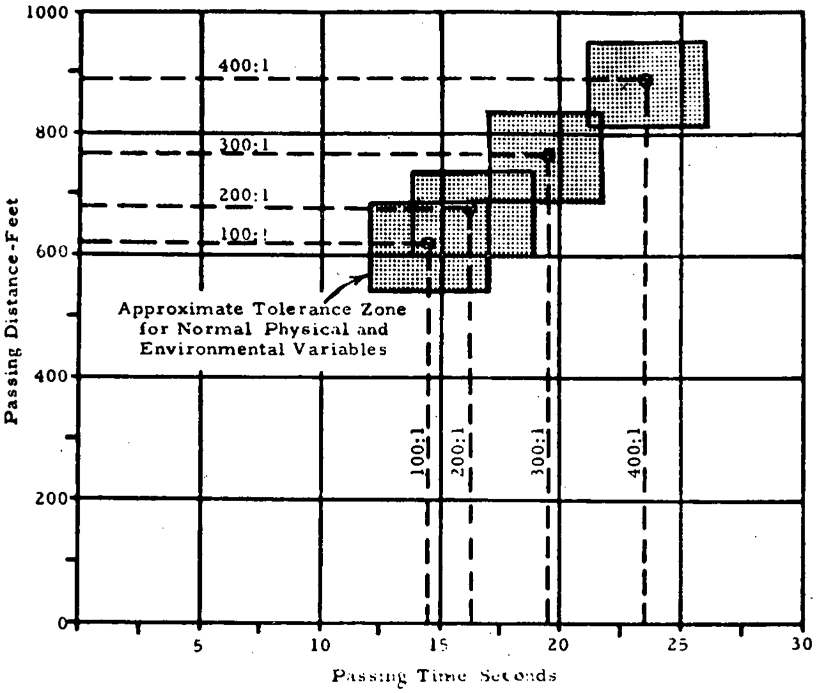


Figure 42. Passing time and distance for low-speed pass by various weight/ horsepower ratios. Source: WHI (39).

TABLE 45

COMPARISONS OF TIME TO PASS A 55-FT AND A 65-FT COMBINATION [a]

SPEED DIFF. (MPH)	2-LANE TIME TO PASS (SEC) 55-FT COMB.	65-FT COMB.	ADDED TIME	4-LANE TIME TO PASS (SEC) 55-FT COMB.	65-FT COMB.	ADDED TIME
5	30.34	31.70	1.36	9.93	11.29	1.36
10	15.17	15.85	0.68	4.97	5.65	0.68
15	10.11	10.57	0.46	3.31	3.76	0.46

[a] Within the range of normal passing speeds (5 to 15 mph difference), the added 10 ft in length requires no more than 1⅓ extra sec, and often less than ½ sec. Further calculations show that, at an auto speed of 60 mph and a combination speed of 50 mph, only a total of 60 additional ft of travel are required to pass the longer 65-ft combinations—less than a 5 percent increase on a two-lane road.

with SAE procedures (*111*). By using these values of the ratio of net driving force of driven axles, *P*, to gross weight, *W*, in their equations, similar speed-distance curves were developed, as shown in Figures 43 and 44.

A comparison of the computed curves with the measured gradeability curves showed a reasonable degree of consistency. Measured speeds were from 1 to 3 mph greater than computed speeds. Net driving force acting at any sustained speed was assumed greater, on the average, than when the truck was in the process of accelerating or decelerating through the same speed. Test curves also contained irregularities in motion which were explained in some part by gear shifting.

Although these discrepancies were noted, it was concluded that the curves were representative of the performance of the test truck on grades. These curves are those employed in the AASHO Policy (*102*).

In a study and reevaluation of warrants for climbing lanes, Williston (*112*) determined the effects of upgrades on mixed traffic actually encountering three magnitudes of grades at three separate sites. Vehicle classifications in this study were: regular passenger cars; compact passenger cars; panel and pickup trucks; single-unit truck; and tractor-trailer combinations. Vehicle speed was measured under normal conditions with radar speed-meter equipment.

The data gathered for each site were processed by the use of a BPR Speed Check Analysis Computer Program. The mean speeds of the various vehicle classifications on the three grades are shown in Figures 45, 46, and 47, and compared there with existing warrants.

The study considered the results as conclusively supporting the fact that truck deceleration speeds have increased substantially since the establishment of existing deceleration curve warrants, and concluded: "If trucks in the future continue to increase in speed as much as they have in the past the need for truck climbing lanes where approach speed is in the category of 50-60 mph, such as those found on almost all 4-, 6-, and 8-lane expressways, will be significantly reduced."

However, it pointed out that more research is needed on various grades where a lower average approach speed exists. Such data are needed to establish warrants for truck climbing lanes at lower speed limits, and secondary highway systems. Accident history on the grades studied was missing; the report recommended additional research.

Truck Climbing Lanes.—Climbing lanes on long, sustained upgrades usually are provided where they may be useful in maintaining traffic service levels approaching that realized on flat sections of the highway. On these grades, the difference in speeds between passenger cars and trucks increases and passing opportunities are less frequent. When significant truck volumes are encountered, the trucks take up more space (i.e., have higher passenger-car equivalents) than on level sections. Drivers of passenger cars tend to attempt to pass without adequate sight distances. Incidence of accidents increases as a result of this willingness to risk safety for travel progress.

The AASHO Policy states maximum grades for various design speeds for main highways (Fig. 48). For short grades (less than 500 ft and one-way downgrades), the maximum grade can be about 1 percent steeper. For low-volume rural highways, grades may be about 2 percent steeper.

This discussion indicates that the maximum grade is not

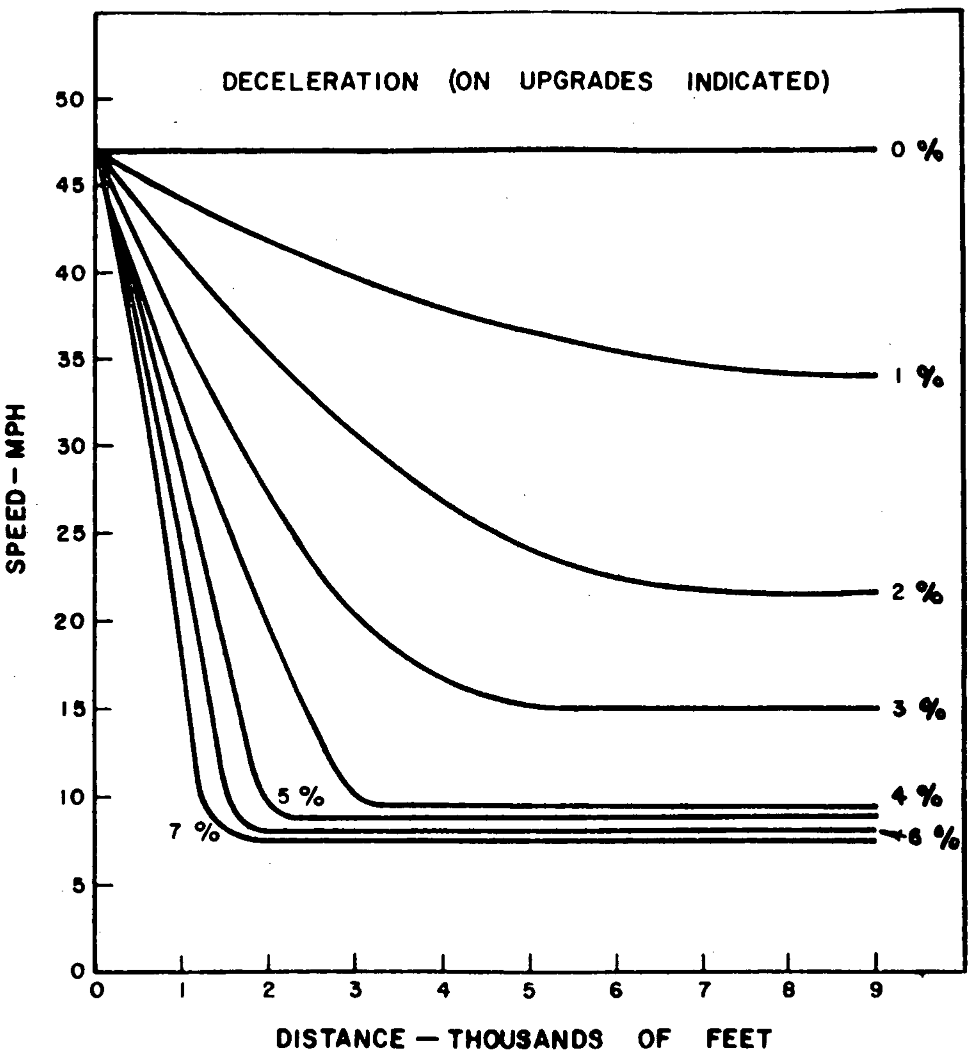


Figure 43. Speed-distance curves from Road Test of a typical heavy truck operating on various grades. Source: Huff and Scrivner (40).

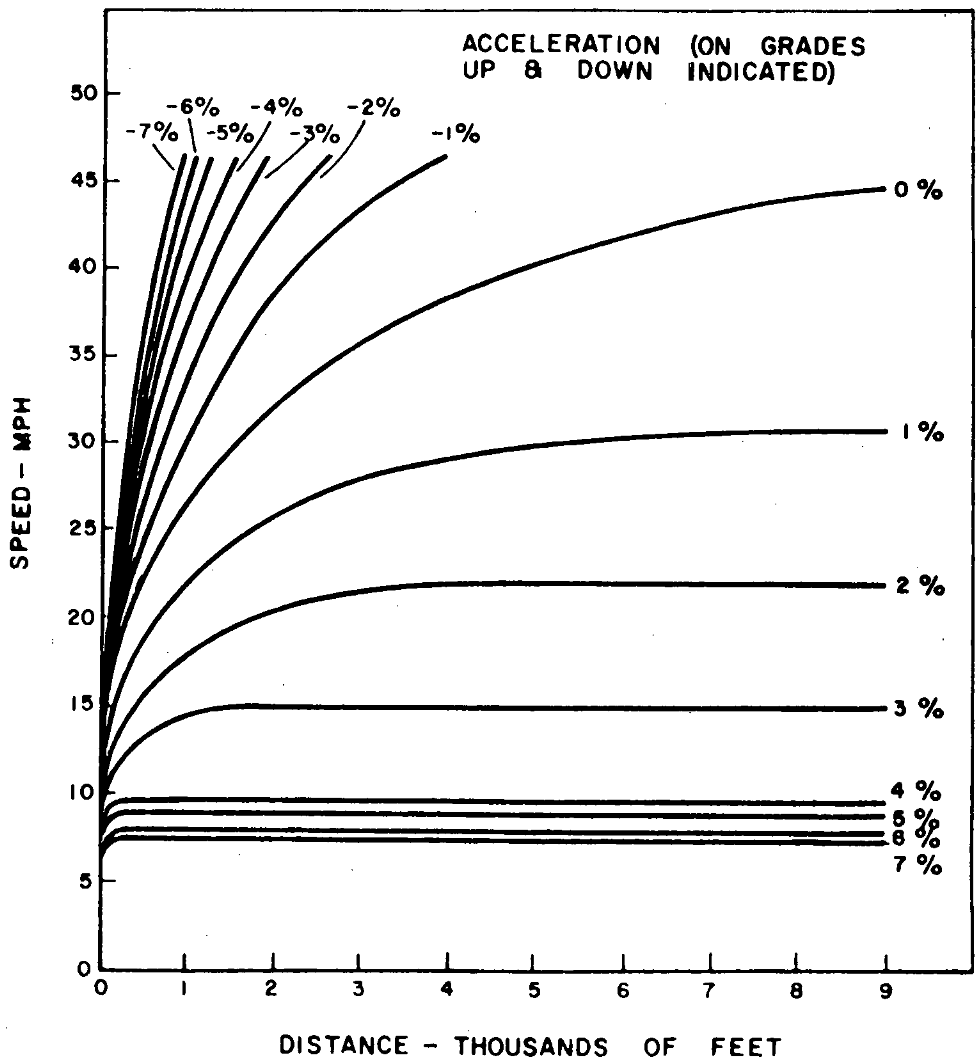


Figure 44. Speed-distance curves from Road Test of a typical heavy truck operating on various grades. Source: Huff and Scrivner (40).

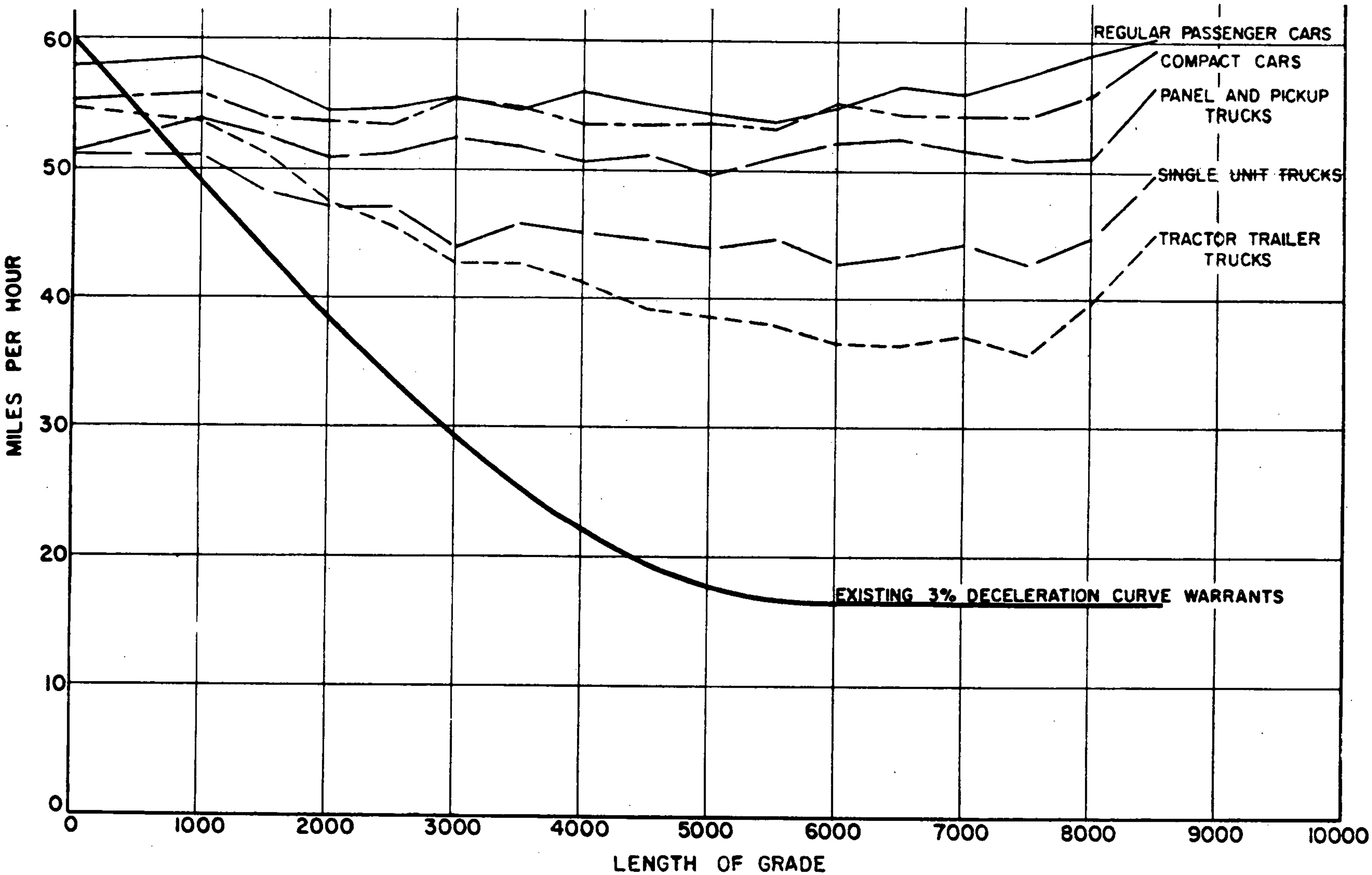


Figure 45. Deceleration comparison curve (3 percent grade). Source: Williston (112).

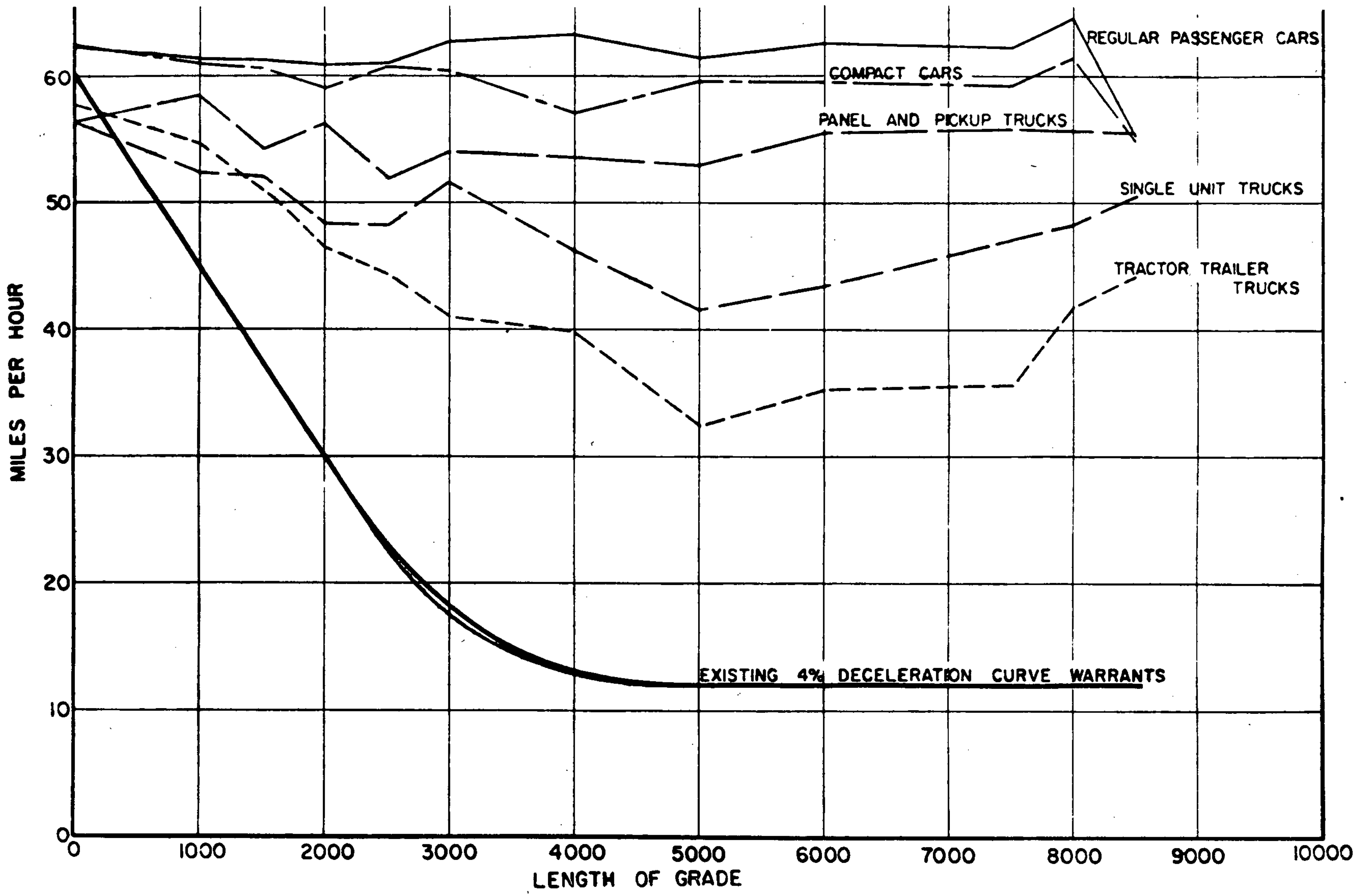


Figure 46. Deceleration comparison curve (4 percent grade). Source: Williston (112).

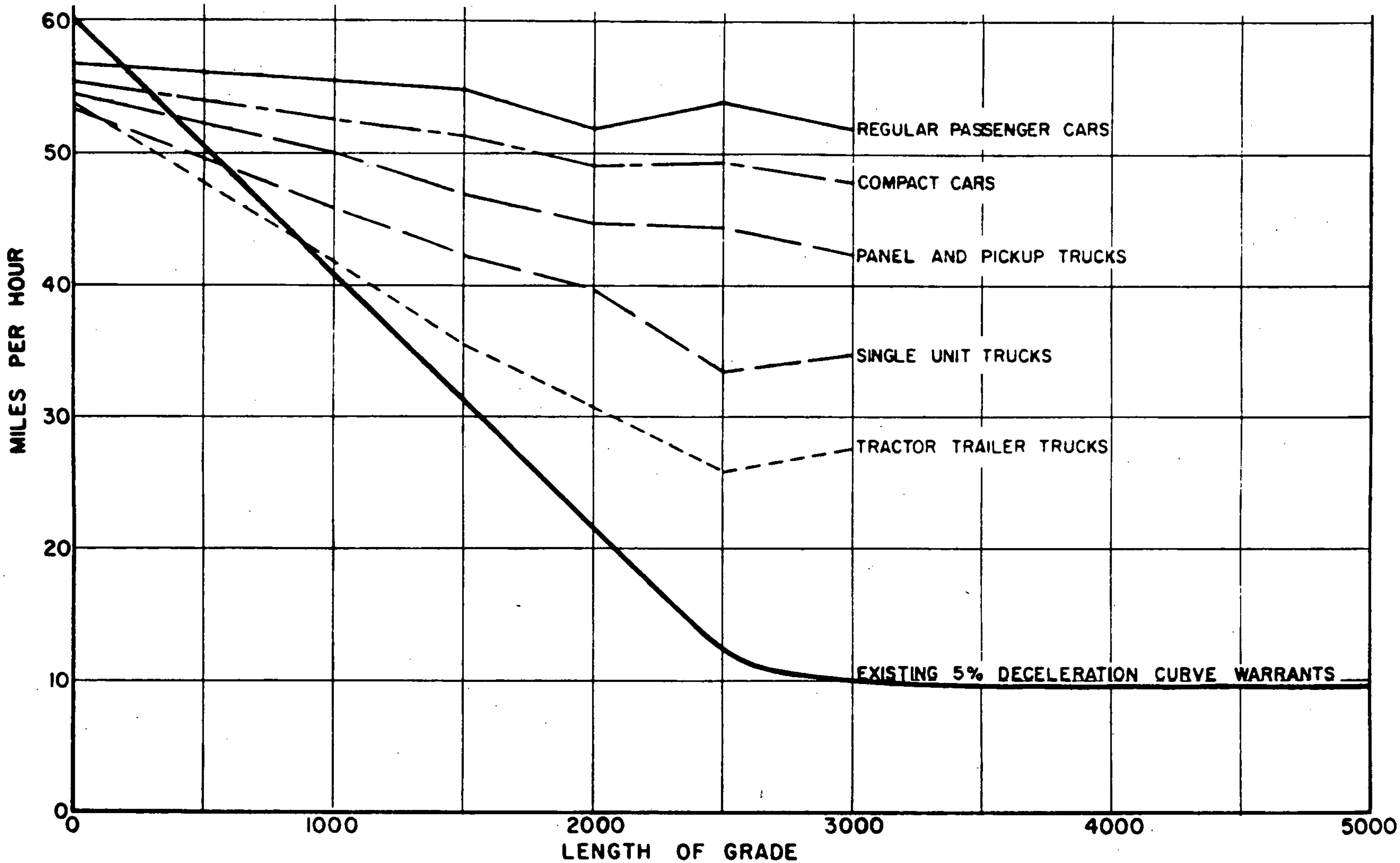


Figure 47. Deceleration comparison curve (5 percent grade). Source: Williston (112).

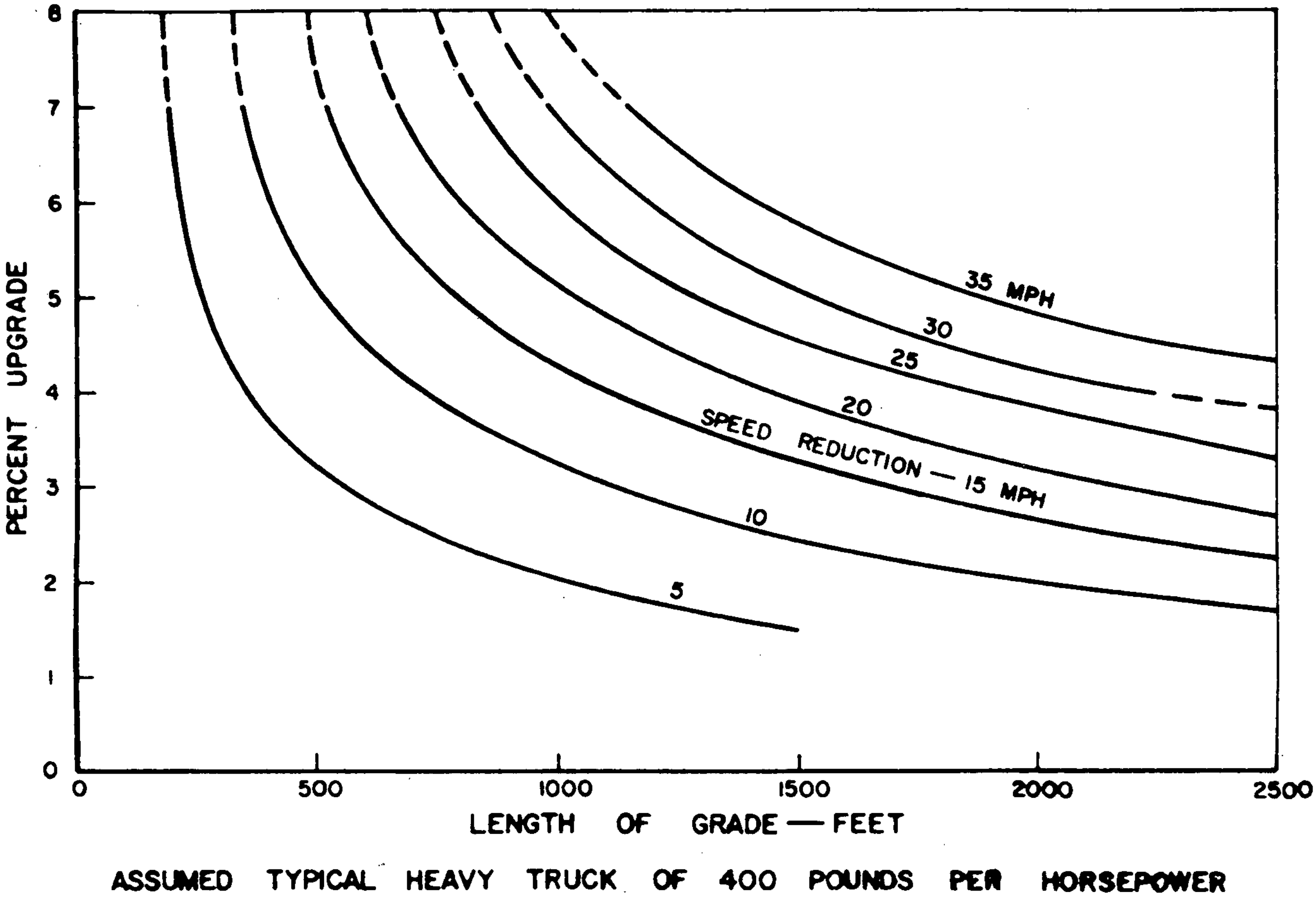


Figure 48. Critical lengths of grade for design. Source: AASHO (102).

a complete design control. Length of grade affects vehicle speeds, particularly those of trucks. "Critical length of grade" indicates the maximum length that a loaded truck can operate without an "unreasonable" reduction in speed. AASHO Policy, in establishing the warrant for climbing lanes, defines this speed reduction as 15 mph, provided traffic volume and heavy truck mix also justify the added cost of the climbing lane. Therefore, truck gradeability, and its effect on the level of traffic service, will determine the critical length of grade.

To establish design values for critical lengths of grades for which gradeability of trucks is the limiting factor, AASHO Policy recommends that the following data be assembled:

1. Size and power of a representative truck or truck combination to be used as the design vehicle.
2. Gradeability data for this vehicle.
3. Entrance speed at entrance to critical grade.
4. Minimum speed on grade for reasonable performance.

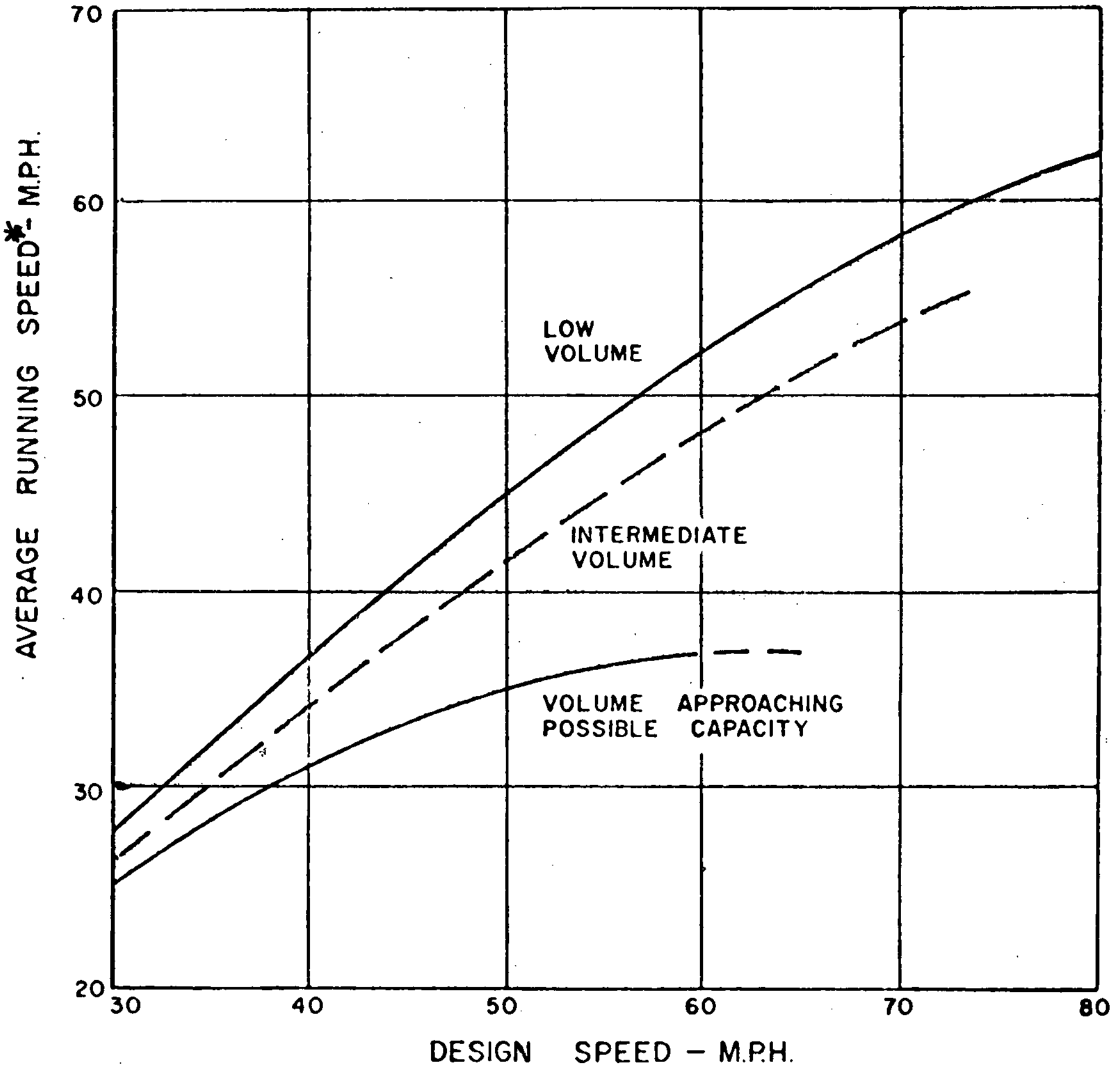
AASHO Policy recognized that a weight/horsepower ratio of 400 is a representative value for design. As such, the speed-distance curves (Fig. 43) determine the gradeability norm.

Figure 49 shows the AASHO ratio of average running speed to design speed. Average running speeds on the level section preceding the grade can be used as the entrance speed to the grade. No specific data are available to define explicitly the minimum tolerable speed. Assuming that unreasonable annoyance is not created for drivers unable to pass on two-lane roads of 40- to 60-mph design speed, minimum truck speeds of "20 to 35 mph" may be reasonable, provided the time interval is not excessive. As volume approaches capacity, however, actual traffic operations may require reconsideration of these running speed values.

AASHO Policy also provides for the justification of climbing lanes on two-lane roads from the standpoint of highway capacity. Table 46 gives these standards.

This calls for the assignment of a passenger-car equivalent for trucks in the traffic mix according to their speed on the grade. In so doing, the Policy standards given in Table 46 indicate percent grade and grade length for two-way design-hour volumes, including trucks (not passenger-car equivalents).

A recent study (*113*) introduced an interesting evaluation technique that relates speed variation of trucks on grades to accident involvement rate. This method is discussed more fully later. By this method, the accident involvement rate for a 15-mph speed differential was computed to be 2,193. This is almost nine times the rate for zero speed differential, where the involvement rate dropped to 913, which is more than one-half of that for a 15-mph



RUNNING SPEED IS THE SPEED (OF AN INDIVIDUAL VEHICLE) OVER A SPECIFIED SECTION OF HIGHWAY, BEING DIVIDED BY RUNNING TIME.

*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 IT IS APPROXIMATELY EQUAL TO THE AVERAGE OF THE RUNNING SPEEDS OF ALL VEHICLES BEING CONSIDERED.

Figure 49. Relationship between average running speed and design speed, AASHO Policy. Source: Glennon (131).

differential. On this basis, the report recommended a 10-mph speed differential criterion rather than a 15-mph differential. Results of the application of this methodology are given in Table 47 and shown in Figure 50.

This section summarizes basic principles relating truck size, weight, and performance characteristics to geometric design. No specific conclusions are made because of the lack of detailed methods that permit scaling these influences on an incremental basis to changes in vehicle weights or dimensions. The decision-making process, therefore, must rely on judgment rather than on computed values. To assist in this process, Table 48 gives geometric design impacts that could result from changes in legal limits regulating length, gross weight, and width.

IMPACTS OF CHANGES IN LEGAL LIMITS ON TRAFFIC OPERATIONS

Trucks operate on highway systems mixed with other classes of vehicles. The comparative performance capabilities and characteristics of these classes of vehicles are related to the highway capacity and the safety of that mixed

TABLE 46

MINIMUM TRAFFIC VOLUMES FOR CONSIDERATION OF CLIMBING LANES ON GRADES ON TYPICAL TWO-LANE ROADS, AASHO POLICY [a]

GRADIENT (%)	LENGTH OF GRADE (MI)	MINIMUM TWO-WAY DHV, INCLUDING TRUCKS (NOT PASSENGER-CAR EQUIVALENTS), FOR CONSIDERATION OF CLIMBING LANE FOR VARIOUS PERCENTAGES OF DUAL-TIRED TRUCKS			
		3% TRUCKS	5% TRUCKS	10% TRUCKS	15% TRUCKS
		4 lanes	4 lanes	4 lanes	4 lanes
4	⅓	warranted	for DHV	over 600	over 525
	½	for DHV	over 700	550	450
	¾	over 750	670	500	390
	1	750	640	470	370
	1½	730	610	440	340
	2	710	590	420	340
		4 lanes	4 lanes	4 lanes	4 lanes
5	⅓	for DHV	over 640	over 550	over 480
	½	over 690	620	460	370
	¾	650	540	380	300
	1	630	510	360	270
	1½	600	490	340	260
	2	600	480	330	250
		4 lanes	4 lanes		
6	⅓	over 625	over 580	480	390
	½	570	470	330	250
	¾	540	430	290	220
	1	530	420	280	210
	1½	520	410	270	200
	2	510	410	270	200
7	⅓	470	410	310	240
	½	400	320	210	160
	¾	380	300	200	150
	1	360	280	180	140
	1½	350	270	170	130
	2	340	260	160	120

Detailed analysis of each grade is recommended in lieu of tabular values.
[a] Source: AASHO (*102*).

TABLE 47

ACCIDENT INVOLVEMENT RATES ON GRADES COMPARED TO VARIATION FROM AVERAGE SPEED OF ALL VEHICLES ON A HIGHWAY

SPEED REDUCTION (MPH)	ACCIDENT INVOLVEMENT RATE	INVOLVEMENT RATE RATIO RELATED TO 0 SPEED REDUCTION
0	247	1.00
5	481	1.95
10	913	3.70
15	2193	8.90
20	3825	15.90

Source: Glennon and Joyner (*113*).

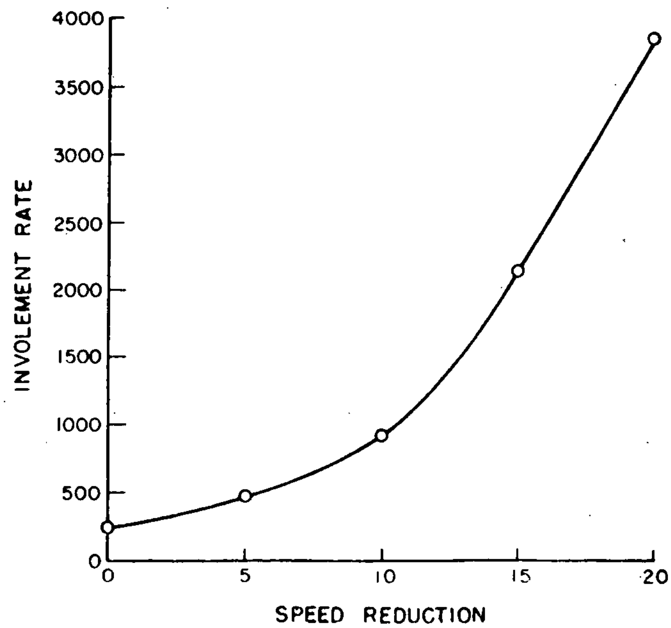


Figure 50. Accident involvement rate vs speed reduction, average of all vehicles on a highway. Source: Glennon and Joyner (113).

traffic, the travel time of highway users, and the economics of vehicle operation.

Criteria generally accepted in highway design assume that the operating characteristics of trucks and passenger cars are not similar, nor can they be designed in the near future to similar performance standards. Therefore, the public interest would be served if highways were designed and constructed for this mixed traffic so that these vehicles could move without unduly restricting the mobility of traffic (*114*).

The ability of trucks to maintain speeds on level highways, in comparison to the cruise speeds of other vehicles, is directly related to the number of passing maneuvers or encounters experienced by the highway users.

The performance of trucks on grades influences traffic flow and capacity. Trucks reach terminal velocities on plus grades which are slow and depend on: length of grade; entering speed; the weight/horsepower ratio of the truck; the traction capability, a function of drive wheel loads; tire treads; surface type and condition, and the presence of snow, ice, or moisture thereon; etc.

Some claim that the braking performance of trucks, compared with that of other vehicles, is offset by the increase in sighting distances. Others claim this is not so. Braking performance and stability have obvious impacts on safety. Braking distance is a function of brake design, number of tires and their tread design, traction between tires and road surfaces, truck configuration, etc.

Speed Trends

Annual studies performed under FHWA auspices indicate that the average speed of all types of vehicles on main rural highways has steadily increased since World War II

TABLE 48

SUMMARY OF POSSIBLE GEOMETRIC DESIGN IMPACTS

NATURE OF LIMIT CHANGE	POSSIBLE EFFECT	POSSIBLE IMPACT	ANALYSIS PROCEDURE
Length	a. Offtracking	1. Pavement widening of curves.	1. Compare new limit against present critical design vehicle.
		2. Pavement widths at intersections.	2. Compare new limit against present critical design vehicle.
	b. Passing maneuvers	1. Increased sight distance for passing zones.	1. Compare sight distance required for safe passing maneuver—existing and required for proposed limit.
Gross weight limit	a. Weight/horsepower ratio	1. Performance of truck on positive grades.	1. Compare limiting grade of present critical vehicle against proposed vehicle performance.
		2. Cruise speed on level tangents.	2. Compare cruise speed of present critical vehicle against proposed vehicle performance—check capacity influence.
		3. Acceleration limits on trucks passing other vehicles.	3. Compare acceleration limits of present critical vehicle against that of proposed vehicle.
	b. Braking	1. Stopping distance.	1. Compare stopping distances of present critical vehicle against proposed vehicle performance—analyze impact of results to passing sight distances, stopping sight distances on vertical curves.
Width	a. Path width	1. Increase lane widths. 2. Decrease in capacity. 3. Safety.	1. Compare width limit of present vehicles against proposed widths for influences on impact areas.
	b. Vehicle-induced aerodynamic disturbances	1. Geometric design. 2. Safety.	
	c. Increased shadow area	1. Sign visibility.	1. Study now under way—no conclusive results published. 2. Study extent of vehicle shadow increases due to new dimensional change.

(Fig. 51). The average speed * of passenger cars and buses has drifted together, averaging just over 60 mph in 1968. Over the same period, the average speed of trucks has experienced the same trend; however, it has been approximately 10 mph slower than that of cars and buses.

The distribution of truck speeds in the percentage of vehicles exceeding 50 mph on main rural highways has improved considerably. In 1958, about 61 percent of the passenger cars exceeded 50 mph, whereas only 32 percent of the trucks achieved this speed—a difference of about 29 percent. Ten years later, about 85 percent of the cars and 69 percent of the trucks exceeded 50 mph, or a difference of only 16 percent.

These trends indicate that, commensurate with improvements in highway design and construction, trucking design has progressed toward larger engines for heavier loads.

Weight/Horsepower Ratio

Frequency distribution of weight/horsepower among vehicles in use confirmed this trend in a series of brake tests (Figs. 52 and 53) (*114*). Net horsepower is shown in these series. Other studies have considered net brake horsepower, advertised gross or nameplate horsepower, and wheel horsepower for application in this ratio. Standards for the control of truck performance have been recommended by AASHO and FHWA. Only Michigan and Pennsylvania have enacted legislation universally regulating a maximum weight/horsepower ratio for trucks. Idaho requires a maximum of 400 lb/net bhp for 98-ft multiple combinations. Oregon requires that combinations of three trailers or semi-trailers on designated highways have a maximum of 400 lb/net horsepower. Nevada stipulates that longer truck combinations either have a 350 to 1 weight/horsepower ratio or be able to maintain a minimum speed of 30 mph.

Gross weights can be either actual gross vehicle or gross combination scale weights, maximum gross weight for which the vehicle is licensed, or legal maximum gross weights.

As in any regulatory procedure, a standard for both horsepower and weight should be uniform, easy to determine, and regulated and established under standard environmental conditions (e.g., temperature and altitude).

* "Average speed" in these data corresponds to the free-running speeds of vehicles, as opposed to average daily or hourly average speeds.

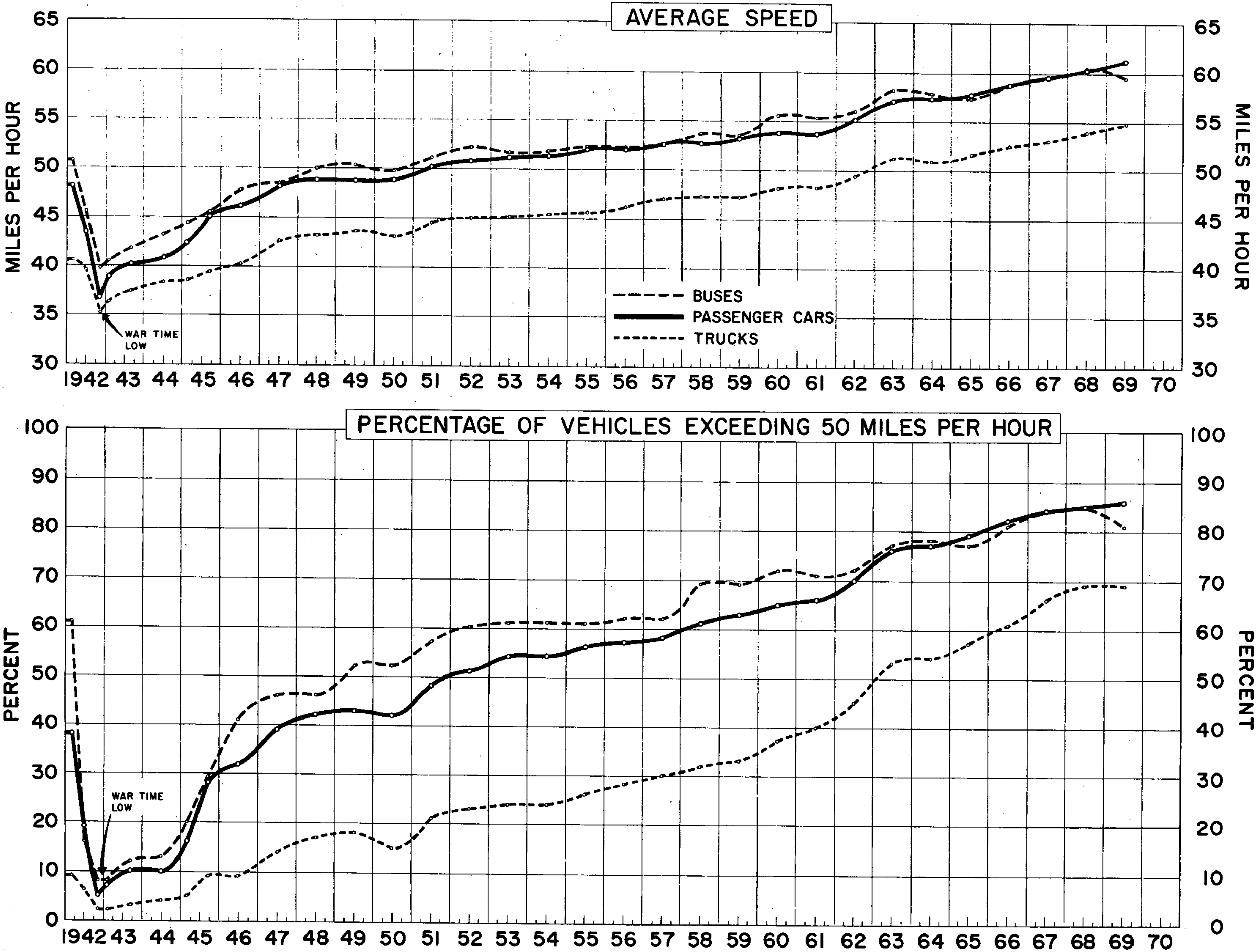


Figure 51. Speed trends on main rural highways, by vehicle type. Source: FHWA, Highway Statistics (1969).

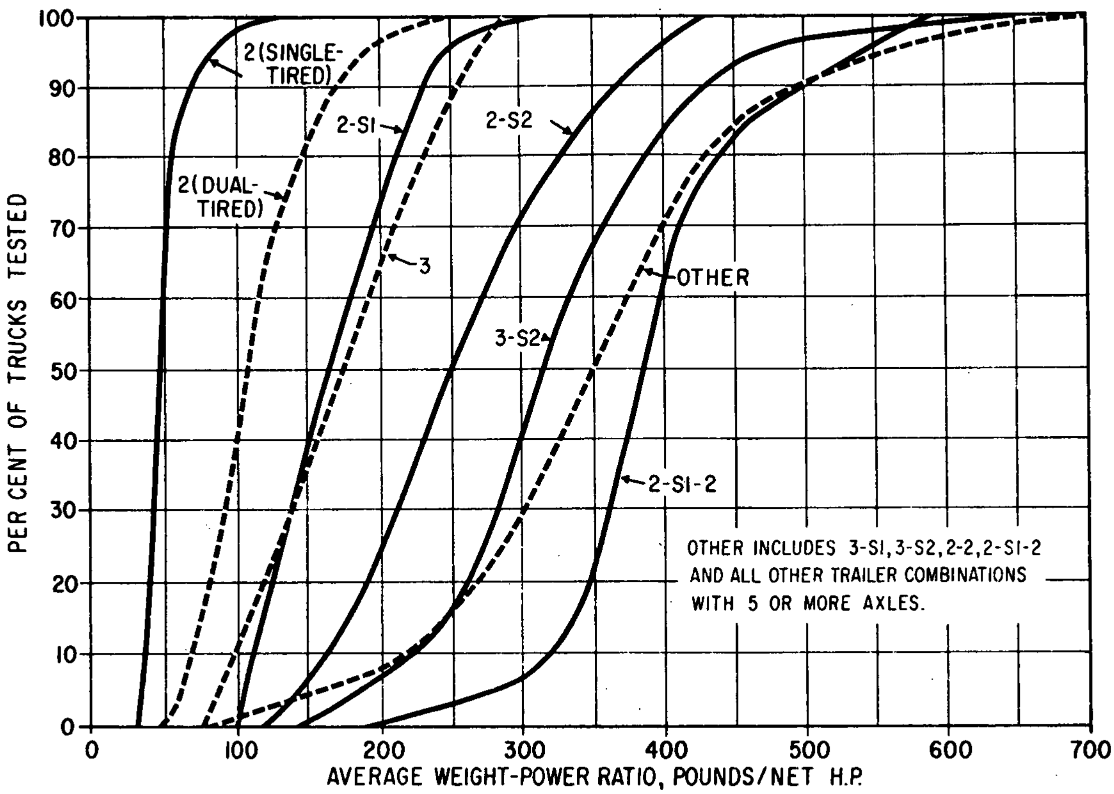


Figure 52. Cumulative frequency distributions of weight/horsepower ratios for loaded trucks, 1963 brake test. Source: Wright and Tignor (114).

Regulation also opens the question of what constitutes adequate performance. The performance standard/horsepower requirement problem can be stated in four parts (*39*):

1. Amount of horsepower needed to obtain *x* performance under *y* conditions.
2. Determination of desired performance standards on *x* roads under *y* conditions.
3. Availability of engines with sufficient horsepower to meet 1 and/or 2.
4. Regulations and enforcement of 1 and/or 2.

Effects of Trucks on Traffic Flow

The capacity of a highway system is defined as:

> . . . the maximum number of vehicles which has a reasonable expectation of passing over a given section of a lane or a roadway (or in both directions for a two-lane or a three-lane highway) during a given time period under prevailing roadway and traffic conditions (*104*).

Thus, the capacity of a roadway depends on a number of conditions (e.g., composition of traffic, roadway alignments, number and width of lanes). Those attributes that relate to highway design generally are termed roadway factors; those that relate to traffic are traffic factors.

Level of service is (*104*):

> . . . a term which, broadly interpreted, denotes any one of an infinite number of differing combinations of operating conditions that may occur on a given lane or roadway when it is accommodating various traffic volumes. Level of service is a qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs. In practice, selected specific levels are defined in terms of particular limiting values of certain of these factors.

The restrictive physical features incorporated into the design of a section of roadway adversely affect its capacity and service volumes. These roadway factors as related to trucks include lane width, lateral clearance, shoulders, auxiliary lanes, surface conditions, alignment, and grades.

Traffic factors as they appear to relate to trucks are summarized here.

Passenger-Car Equivalence

Previous studies have found that trucks reduce the capacity of a highway in terms of the total vehicles carried per hour. Under specific conditions, this reduction is thought of as the equivalent number of cars that the truck displaces in the traffic flow.

Values of car equivalents for truck vary with type of highway and level of service. On sections of two-lane highways that are flat and where speed differences between trucks and cars are small, trucks usually are considered equivalent to two or three passenger cars. On multilane highways, research indicates that this equivalence can vary

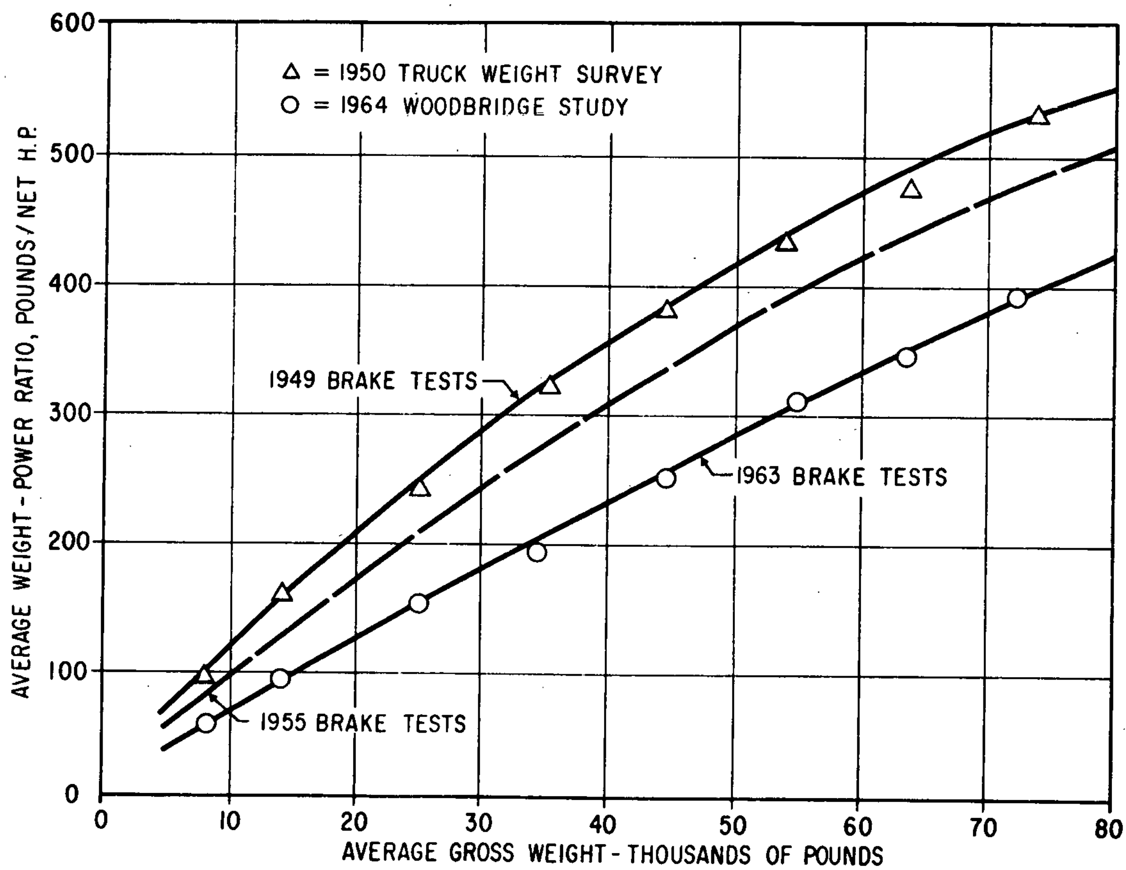


Figure 53. Trend in weight/horsepower ratios. Source: Wright and Tignor (114).

from 0.8 to 2. On upgrades, there is a wide variation in car equivalents, depending on steepness and length of grade, and on the speed differences on grades between passenger cars and trucks. Published tables give car equivalence factors for trucks on various grades that might be encountered. These tables assume a weight/horsepower ratio of 400, with no means given for compensating for higher performance trucks. As stated previously, truck performance has been improved.

Recent studies indicate the degree of improvement to be substantial. One of these studied the influence of trucks on traffic in four locations: (1) downstream from a traffic signal, (2) downstream from an entrance roadway, (3) level, tangent roadway, and (4) grade. In location (1) it was found that as the number of trucks in a stream increases, the average headway of the stream increases, but the truck equivalence appears to be far less than 2. For location (2), on a volume basis only, truck equivalence generally ranged from 0.9 to 1.3. On the level tangent roadway of location (3), trucks approached the equivalent of two cars as the number of trucks in the stream approached 100 percent.

Without a program of data collection there apparently is no means of relating truck equivalence factor to truck variables such as weight/horsepower ratio, dimensions, and configurations.

IMPACTS OF CHANGES IN LEGAL LIMITS ON SOCIAL COSTS

The impacts of changes in legal limits of motor vehicles on the physical structures of the highway were well documented in the literature. This is interpreted as an awareness of highway officials of the observed deterioration of the highway system caused by heavily loaded vehicles, and their responsibilities provide facilities to accommodate all traffic within the permitted limits.

In recent years, the relationships between highway system construction, design, and operation and the social aspects of life have become increasingly important. These aspects of life (e.g., safety, clean environment, aesthetic appearance of highway structures and rights-of-way, noise, division of neighborhoods, displacement of persons due to right-of-way acquisition) are related to social costs that must be considered in evaluating the highway system.

Social costs, in this study, were viewed as all costs related to both users and nonusers of the highway system, which can or should be quantified in relation to changes in legal limits. Topically, these include:

1. *Safety*—The influence of changes in legal limits on accident incidence, accident severity, fatalities, injuries, and property loss.
2. *Noise*—The influence on traffic noise level and spectrum of the new trucks, annoyance created by this addition

to the traffic stream, the loss of property value to owners adjacent to highway facilities, and the loss of efficiency in performing tasks in offices, homes, etc.

3. *Traffic Operations*—The influence of new trucks on new criteria for traffic operations, the effect of traffic flow on travel time of automobile travelers, and the estimate of value of time lost or gained.

4. *Air Pollution*—The increase in air pollution levels, nature and composition of pollutants, and associated costs.

5. *Vibration*—The increase in ground-conducted vibration from traffic consisting of trucks operating under the new legal limits.

Safety Aspects of Legal Limits

Safety of highway operation is an immediate concern that is intimately related to proposed changes in legal vehicle limits. The records of accidents involving trucks have been examined by a number of investigators. In reviewing these studies, it is important to ascertain whether the approach is an incremental analysis, or would lead to that analysis format.

As Baker (*115*) points out, the terms "safe" and "unsafe" frequently are used in an absolute sense. Rarely is the distinction precise. Although one may reject his principle initially, further reflection leads to acceptance of the viewpoint that safety and the willingness to take risks are interrelated.

> To live is to be in danger of not living, and in that sense one can never be "safe." It is necessary, therefore, to operate on the basis that "perfectly safe" is not a description that is applicable in a literal sense to any of our activities, let alone the job of transportation in the real world.

Safety must be judged in a relative sense. Therefore, in evaluating the safety aspects of a proposed change in legal limits, one must determine whether the safety aspects of the highway system are enhanced or degraded as a result of the changes. To merely quote safety indices of present safety records does not answer the question adequately. This incremental analysis method has been applied in other value judgments related to legal limit changes and appears appropriate in regard to safety.

Such an approach requires an understanding of the highway system function as a means of moving people and goods from one place to another so as to enhance people's economic, social, and cultural lives. The resulting mobility that must be provided is defined by the desires of many different individuals, groups, and institutions. "Safety is therefore a measure of how well individuals are being served rather than the end point or objective of the transportation system."

Although there is need for a suitable measure of safety, there has been no generally accepted approach to an indicator. The number of fatalities, severity of accidents, severity of collision, suffering and emotional impacts, and economic losses in income and property have all been used as indicators of highway safety.

Indiscriminate use of rates used in these indicators can often be misleading. Investigating highway safety, May, Chief Counsel of the Special House Subcommittee, said (*116*):

> The reason the staff is having dificulty with rates is that we are not quite so sure that the rate is a real indication of whether or not a highway was designed and built safely.
>
> We are concerned because the Beltway right here in Washington had 30 people killed last year, and we have checked and during rush hours from 6 to 9 in the morning and 4:30 to 6:30 at night, no motorist was killed. If we knew the rate, it would probably run about 2.5.
>
> We just challenged the fact that somebody suggested a rate would indicate whether or not the highway is safe. In rush hour, nobody was killed; on the same highway, and other hours, 30 people were killed.

Baker continues:

> From this example, one can conclude that the risk of a fatal accident was less during the times of greatest accumulation of miles of travel. Gross rates of fatalities can obscure the true nature of the risk.

The need for a technical measurement of how the level of mobility is affected by safety measures must be combined with the judgment of the level of safety achieved. Baker cites an example. If no level of mobility impact is introduced in the evaluation, accident rates can be reduced by merely limiting travel.

Baker (*115*) continues:

> If safety measures do not improve transport efficiency, there also will be national and international economic impacts related to the cost increase produced in transportation. Corrective measures which tend to increase transport productivity as well as to improve safety are therefore preferred. For example, increasing truck performance requirements for such behavior as acceleration capability might be a costly way to reduce accident involvement, but increased productivity would accompany the safety advance. Safety correctives directed toward reducing the severity of accidents, do not increase transport productivity and are therefore pure increases in cost.

The general lack of methods to relate changes in legal limits to any of the safety indicators was substantiated in the literature search of truck accident studies. None applied the visibility of trucks, weight, width, length, or performance characteristics to the safety experience of trucks operating in mixed traffic. This serious gap in knowledge is probably due to the difficulty in specifying as the cause of the accident any one of the many factors and situations involved. Accident report procedures generally class the vehicle, if it is a truck, in gross terms, such as light, medium, or heavy, without documenting the parameter.

Robinson et al. (*117*) performed a safety study for the National Highway Safety Bureau, DOT, that analyzed data on truck accidents in two states. Although data and conclusions in that report no doubt are useful—particularly with regard to driver ejection frequencies, need for driver and passenger restraints, types and nature of impacts in truck-related accidents—they did not provide methods or relationships useful in evaluating the size or weight relationships in accidents involving trucks.

Numerous data were presented in Congressional hearings in consideration of the various bills, as discussed earlier. Evaluation of these data likewise revealed no method in their preparation in line with the incremental evaluation and analysis techniques.

Safety on the highways requires the comprehension of all issues. Emotionalism, introduced by those for and against changes in legal limits, is no substitute for intelligent and realistic analyses. However, these issues apparently have not been addressed in research programs at this time. Further research, data collection, and evaluation are necessary if analyses of safety, particularly with regard to truck size and weight, are to be accomplished in an objective and meaningful manner.

Accident Incidence

Since there is speed differential, the involvement rate of a vehicle in accidents has been related to the variation of that vehicle from the average speed of the traffic stream. Figure 54 shows this relationship for daytime and nighttime travel (*118*).

The relationship has been used to relate accident incidence potential to trucks of various classes in evaluating climbing lane warrants (*113*). In these computations, the average speed of all vehicles on level grade is used as a basic reference. A distribution of speeds within a given grouping of speed categories is compared with the average speed to develop the difference from average. The distribution of trucks within each speed category is determined from a survey. The speed difference from average for each speed category is converted to involvement rate by using Figure 54. The involvement rate for each category is the product of the percentage of trucks within that speed category and the involvement rate for that category. The summation of each of these prorated involvements divided by 100 equals the involvement rate prediction for trucks with a given speed below average for that grade.

The application of this method is illustrated in Appendix F.

Although the foregoing study applied the technique to climbing lanes only, a similar procedure might be applied to evaluate other geometric configurations and distributions of truck characteristics where trucks and vehicles operate at speeds different from average speed of traffic flow.

The principal data input required for this computation is the speed distribution in discrete speed intervals of trucks in each visual truck classification. This distribution should also be related to the variable of the legal limit being considered (e.g., length, axle weight, weight/horsepower).

Speed distribution would then have to be projected to develop the new distribution should the new limit apply. Comparisons of the accident incidence under present and proposed limits would then be possible.

The development of such a technique of projection is only conceptually given here. To develop the complete method would require data collection, analysis, and verification not within the scope of this study. Further research is required.

Highway Truck Noise

Basic to the evaluation of the effects of truck size, horsepower, type of engine, tread-roadway interaction noise, and other sources of noise emitted by vehicles on highways, are the factors of this composite noise, which is perceived as annoying to the individual. The mechanisms for the generation of traffic noise are complex, and the responses to noise are influenced by many factors. These include expressions of annoyance, difficulties in speech comprehension, degradation of task performances, noise-induced stress, loss of or interference with sleep, and, although not apparently involved in traffic noise, noise-induced temporary and permanent loss of hearing.

Loudness, temporary variations of loudness, the frequency spectrum of the noise, the location and activity of the individuals, their interpretation of who is responsible for the noise, its necessity and resulting value, and even the psychological perceptive aspects of the facility generating the noise all have important relationships to the annoyance one feels to noise exposure.

Among the conclusions of a Road Research Laboratory report on traffic noise (*119*), the following are considered pertinent here.

Present levels of noise from road traffic have an adverse effect on living conditions in the United Kingdom, but have not been shown to be a significant hazard to health. There are virtually no reliable data on the economic value of the loss of amenity in homes and the loss of efficiency in offices, etc.

There is a fair knowledge of sources and levels of noise generated in individual vehicles, the manner of noise transmission from the source, and possible methods of lessening noise by building design. Less is known about how individual vehicle-generated noises combine into traffic noise.

Some general, fundamental relationships are given to relate sound level and vehicle speed, engine noises, the effect of engine size, fan, scavenging blower, inlet and exhaust noise, road surface, and tire reaction noise. The most effective manner of traffic noise reduction should be the attenuation of these components at their source.

Means are given of attenuating traffic noise by highway geometric design and construction of noise barriers and planting in strategic locations along the right-of-way.

Costs and cost-effectiveness of noise reduction are complex. No comprehensive costs are available for these various methods of noise reduction. Valuation of noise reduction benefits presents major difficulties in view of "the intangible and often unquantifiable nature of the benefit." Possible techniques include a study to determine the influence of environmental factors on house prices.

Plowden (*120*) sought to establish, in London, "the amount of money a person would have to be given, upon the imposition of a noise nuisance alone, if he was, in his own estimation, to be as well off after as before the nuisance arose."

In the survey, alternative courses of action were open to him—stay and endure the noise, spend money for partial protection, or move. If he chose the first, "endurance" costs, borne by the owner/occupants, should be related only to the effects of a change in noise level, as distinct from other amenity costs such as the transport advantages of a "motorway" nearby.

The expense of exercising the moving alternative in-

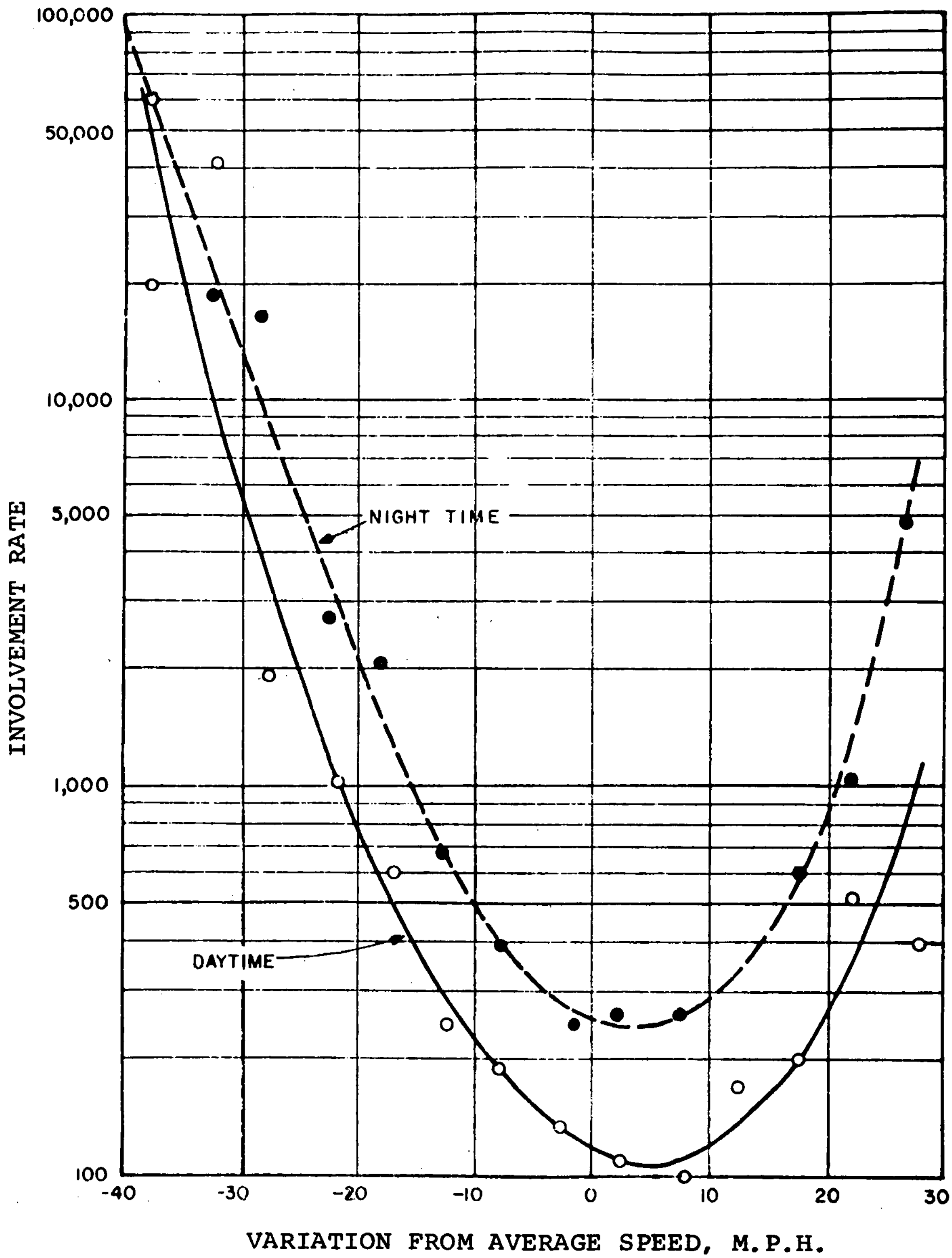


Figure 54. Involvement rate by variation from average speed on study section, night and day. Source: Solomon (118).

cluded the decrease in sales price of one's house, and moving expenses.

In this study, some caution was applied in interpreting the results, because of problems involving "actuality" and "purity" of survey data gathered.

Galloway et al. (*121*) presented guidelines for use by highway design engineers in creating new facilities; showed how noise abatement might be achieved through the design of less noisy tires and automotive units; and provided data for building design as related, considering traffic noise attenuation. A standard technique for roadside noise level measurement also is given.

The study presented "ample" evidence that people can reliably judge the relative loudness and noisiness of sounds. Within limits, an absolute scale of subjective noisiness can be constructed for noises in some context. "The data failed to show a simple, strong relationship between objectively measured noise and reactions to that noise by freeway-proximal residents in their natural settings." To illustrate, the report pointed to the fact that 51 percent of residents in the relatively noisiest area complained of noise, while 67 percent of residents in the relatively quiet area complained. Actual measurements showed that the perceived noisiness of the noisier area was about four times as great as that of the quiet area.

> Peoples' general attitude toward freeways and toward living near a freeway were related to their attitudes of freeway noise specifically, and complaints about noise may be a socially acceptable way to complain about freeways in general.
>
> . . .
>
> It seems probable that by also taking into account a composite socioeconomic or demographic factor, these predictions (of annnoyance) could be improved but at present there is not such a single factor constructed.

Although the state of the art does provide models that predict the contribution to traffic noise of individual vehicles, based on speed, horsepower, load, engine design, muffler design and other physical factors, which do relate to legal limits, the application of these data to annoyance, and therefore to economic values, is not well enough understood to be useful at this time. Much more research and study are required before methods can be assembled in a meaningful manner.

The situation was well summarized in a study by the National Bureau of Standards (*122*) of the economic impact of noise:

> Because the data at the present time are, at best, fragmentary, the findings of this study should be considered suggestive rather than exhaustive. A number of reasons can be cited for this lack of data. One factor is the nature of noise itself. In contrast to water or air pollution, which can have long-lasting effects on the environment, noise pollution "decays" rapidly in both time and distance. As soon as the source of the noise is silenced, the unwanted sound disappears almost instantaneously. Moreover, the intensity of sound diminishes rapidly with distance—a loud roar will be reduced to a muffled rumble by a short distance. A second factor is that the effects of noise are not as "dramatic" or immediate as the consequences of other pollutants. The hearing damage caused by noise generally occurs after exposure over extended periods of time. Also, many of the consequences of noise can be attributed to annoyance caused by noise rather than the threat of imminent hearing loss.* Thirdly, different individuals exhibit varying levels of tolerance to noise levels. Finally, one of the reasons that noise has not been viewed as a form of pollution is the attitude of the public toward noise as the "price of progress." The noise produced by a product is often associated with efficiency and the ability of a product to perform its designated function; e.g., a "quiet" vacuum cleaner was rejected by a test group because it was perceived to clean less effectively than a noisier model of equal power.
>
> Because many kinds of noise are primarily a source of "nuisance" or annoyance rather than a danger to health, it must be recognized that it might not ever be possible to obtain precise estimates of either the cost of noise or the benefits derived from noise abatement. This is true because nuisance and annoyance are psychological states, which to date have defied adequate quantification by social scientists.

* Because so many aspects of noise are psychological, researchers encounter the same problems as those found in the theory of consumer behavior. For example, economists and other social scientists have not been able to estimate or to compare the satisfaction or utility that one might derive from consuming three dry martinis and the annoyance or disutility of one's spouse from watching the consumption of three martinis.

OVERSIZE/OVERWEIGHT PERMIT OPERATIONS

Oversize/overweight loads on highways fulfill special and unique demands for the movement of large prefabricated items from point of assembly to point of use. Large turbine and generator assemblies, tanks, and blowers that cannot be effectively and economically fabricated on-site characterize this type of load. In addition to private industry, the Department of Defense and NASA use this type of movement to transport missile assemblies and other defense-related items.

These movements are made under special permits issued by each state authority. There is a great diversity among the states as to authority for issuance, to financial responsibility of the applicant, and in the collection of special permit fees.

Aside from the basic problem of administration of permit operations, the assessment of special permit fees to transport oversize vehicles and gross weights and axle loads in excess of those allowed in regular operation is not without precedent.

To establish uniformity in the proposed schedule of fees, the AASHO Committee on Highway Transport (*123*) proposed a method based on the following assumptions:

> (a) The permittee should pay a special I&D permit issuance fee sufficient in amount to cover the administrative cost of handling and enforcing the permit.
>
> (b) The permittee receiving a special I&D permit should pay for the privilege of transporting gross weight in excess of the limitations—in proportion to the costs borne by operators for transporting gross weights in regular operation.
>
> (c) The permittee receiving a special I&D permit when the weight of one or more axles exceeds the limits for axles in regular operation, should pay for the added wear and tear to the highway in proportion to the reduction in service life caused by the excessive loading.

Basic Issuance Fee

The assessment of the fee is a matter of determining the administration and enforcement costs involved. This may also be affected if the movement is through a number of jurisdictions with differing enforcement procedures. Existing practices indicate that a fee of $5 to $10 should be assessed for this portion.

Time Extension Fee

For processing an application for a time extension, a cost in the range of $5 to $10, exclusive of all communication costs involved, is suggested.

Supplemental Permit Fee

A fee in the range of $5 to $10 should be charged for each supplemental special permit issued.

Special Permit Overload Fee

Determination of an equitable charge for movement over the National System of Interstate and Defense Highways or other designated highways of a vehicle exceeding the established limits for gross weight or axle weight may be based on the following considerations:

(a) The approximate investment in the portion of the Interstate System affected wholly or partially by vehicular weight.

(b) The relationship of pavement life to weight and frequency of axle loadings as established by equations developed in the AASHO Road Test.

(c) The relationship between excess gross weight and the established maximum gross weight for vehicles, or vehicles and loads in regular operation.

Investment in Interstate System Related to Vehicle Weight

The cost of the Interstate System was estimated at $860,027 per mile. This figure includes a number of cost items not affected by vehicle weight. Allocations of these cost items are given in Table 49. Based on this division of costs, about 50 percent of the total per-mile cost, or $430,000, is affected by vehicle weight.

Overweight-Axle Fees

Based on total lifetime equivalent 18-kip single-axle applications (estimated at 6,600,000), the average investment charge per 18-kip equivalent axle application is (cost/18 kip-mile) = (investment cost/average 18-kip application lifetime). The calculation resulted in a cost of $0.0652 per 18-kip equivalent axle load per mile.

TABLE 49

DIVISION OF CONSTRUCTION COSTS

CONSTRUCTION ITEM	% OF TOTAL COST [a]	INVESTMENT COST INFLUENCED BY VEHICLE WEIGHT [b] AMOUNT	%
Clearing and grubbing	0.93	None	None
Utilities	2.03	None	None
Grading and drainage	21.05	½	10.53
Base, surface, and shoulders	21.74	All	21.74
RR grade separations	2.74	¼	0.69
Highway grade separations	6.13	¼	1.53
Highway interchanges	15.59	¼	3.90
Bridges	15.21	¼	3.80
Walls	0.60	None	None
Guardrails, etc.	2.78	All	2.78
Roadside improvements	1.52	None	None
Other	0.76	½	0.38
Construction engineering	8.92	½	4.46
Total	100.00		49.81 (Use 50%)

[a] Report to Congress by BPR.
[b] Estimate developed by W. E. Chastain, Illinois Division of Highways.

Overweight-Gross Fee

The method used here was to estimate the total gross tonnage carried on the system during its life, based on ADT, percentage of trucks in mix of traffic, and average pavement life. Computations on the assumed average conditions of: ADT = 6,000, percent trucks = 25 percent and average pavement life = 20 years, yielded a total estimated gross tonnage during life of pavement of 226,446,000 tons.

The cost per gross ton is:

$$\frac{\text{Investment cost of weight-related items (\$/mile)}}{\text{Total gross weight for pavement life (tons)}}$$

This cost was estimated as $0.0234 per ton-mile.

Special Permit Oversize Fees

For an oversize vehicle, or vehicle and indivisible load, a charge should be sufficient to pay for the special privilege and to compensate for the economic loss to operators of vehicles in regular operation who suffer delays and inconveniences due to movement.

The following estimates, without disclosure of means of computation or determination, were considered reasonable:

1. *Width*—A charge per inch-mile for each inch or fraction thereof of $0.005 to $0.02 per mile.
2. *Length*—A charge per foot-mile for each foot or fraction thereof of $0.02 to $0.05 per mile.
3. *Height*—A charge per foot-mile for each foot or fraction thereof of $0.02 to $0.05.

A change in legal limits might result in a reduction of expenses for the administration of oversize-overweight permit operations.

LINE-HAUL TRUCKING COSTS

Line-haul trucking costs for various vehicle gross weights were updated by using the procedure used by Stevens when he updated his original costs from 1956 to 1964 (*124, 125, 126*). He referred to the ICC *Transport Statistics in the United States, Part 7, Motor Carriers.* His scheme was to update the original cost for the same factors by the ratio of 1964 costs to 1956 costs. For overhead and indirect costs, he found the increase for the period 1956 to 1964, then added this uniform amount to each 1956 cost for each loaded gross weight. The change was a uniform amount for the full-load gross-weight range, rather than a variable amount that would have resulted by using the same index number for the full range of gross weight, as he had done in the other items.

The form of some of the statistics on motor carriers as published by ICC has changed. The carrier operations have changed. Therefore, the method Stevens originally used (*125*) was not applicable for the period 1964 to 1970. The carriers have materially increased their use of leased trucks with and without driver. It is most difficult to arrive at comparative costs in cents per mile for Stevens' items (*124, 125*) for the span of years. Because of the time required to even try to work out Stevens' procedure (*125*), other attempts were made.

With some modifications by judgments, the following schemes were used to update Stevens' line-haul trucking costs (*125*) to 1970.

Repairs, Servicing, and Lubricants

The updating index was taken as the 1970/1964 ratio given in the ICC reports for annual wage for mechanics ($9,069/ $7,084 = 1.341). The 1970 annual wage was estimated because the ICC reports were available only through 1969.

Tires and Tubes

Tires and tubes were updated by use of the *Consumers Wholesale Price Index*, published by the Department of Labor. The index used was that arrived at by the ratio of the indexes 1970/1964 (107.2/87.7 = 1.222). The base year for the WPI is 1957-1959 = 100.0.

Fuel

For fuel the 1970/1964 *Consumers Price Index* was used (119.0/102.1 = 1.166). This procedure neglects to account for any change in the mileage rate of consumption of fuel.

Driver Wage and Subsistence

For updating the driver wage and subsistence, the ICC motor carrier statistics showing the average annual wage income of truck drivers were used. The ratio was developed for 1970/1964 ($12,500/9,695 = 1.289). The wage for 1970 was estimated.

Overhead and Direct Costs

Overhead costs from 1964 to 1970 were updated by using Stevens' (*125*) percentage of overhead costs to total per-mile cost. Thus, for 1970, the total of the five items not including overhead and indirect costs were found first. This total was expanded to the total of all six items by the same ratio as in Stevens' report. This expansion was applied to the cents-per-mile cost at each of the loaded gross weights in 5-kip increments from 25 to 180 kips.

Depreciation and Interest

Because depreciation essentially is directly related to the cost new of the vehicle equipment, the ratio of market prices of trucks for 1970/1964 was used. The ratio used in the wholesale price index for trucks was 111.6/98.7 = 1.181. To the 1.181 was added a factor for the increase in market interest rates of 6.5 to 8.5 percent, or 0.096. The total index used is 1.277.

An examination of the original curves for gasoline and diesel fuels (*124*) indicates that the combined curve does not represent a reasonable combination of the original data or for the relative proportion of the two types of fuel. Then the original data (*124*) were reevaluated and a new curve for combined fuels was developed. The new curve was updated to 1964 in accordance with the index of 0.760 (*125*), then updated from 1964 to 1970 with the factor 1.166.

Table 50 gives the costs updated to 1970 for all trucks, a combination of gasoline and diesel fuels. The table was derived through the use of the following:

TABLE 50

PAYLOAD AND GROSS TON-MILE COSTS, BY LOADED GROSS WEIGHT, ALL TRAILER COMBINATIONS

	TON-MILE COSTS ($)		
	PAYLOAD		
LOADED GROSS WEIGHT (LB)	LOADED, BOTH DIRECTIONS	LOADED, ONE DIRECTION	GROSS
27,500	0.05868	0.1174	0.0325
44,000	0.0328	0.0656	0.0221
58,000	0.0262	0.0524	0.0182
65,000	0.0244	0.0488	0.0169
73,000	0.0228	0.0456	0.0158
82,000	0.0217	0.0434	0.0149
91,000	0.0209	0.0418	0.0142
100,600	0.0202	0.0404	0.0137
123,000	0.0189	0.0378	0.0129
137,000	0.0182	0.0364	0.0127
171,000	0.0174	0.0358	0.0126

1. *Repairs, Service, Lubrication*
   $y = 7.14965 - 0.04799x + 0.00120x^2$
2. *Tires, Tubes*
   $y = 1.73999 - 0.01004x + 0.00040x^2$
3. *Fuel*
   $y = 2.76861 - 0.00405x + 0.00012x^2$
4. *Driver's Wages, Subsistence*
   $y = 14.10490 + 0.02741x + 0.00005x^2$
5. *Overhead, Indirect*
   $y = 13.87990 + 0.00982x + 0.00006x^2$
6. *Depreciation, Interest*
   $y = 0.28048 + 0.15466x - 0.00027x^2$
7. *Total*
   $y = 39.92790 + 0.12976x + 0.00156x^2$

The results of the computation were carried to five decimal places. Tabulated data are rounded off to four decimal places. The results of these computations are shown in Figure 55.

Studies indicate the Stevens' "loaded gross weight" was about 80 percent of the vehicle's practical maximum gross weight. Therefore, in applying this figure, this relationship is to be used.

Boundaries of Trucking Cost Study

Attention (*124*) was devoted primarily to the costs related to the gross weight of trailer combinations, and secondarily to variables of axle weight, vehicle type, and cargo capacity. However:

> . . . vehicle sizes cannot be evaluated costwise, either with regard to vehicular or highway costs, the committee's task is somewhat simplified as only the cost effects of axle and gross weights on pavements and bridges need be considered.

Because a decade has passed since this conclusion was

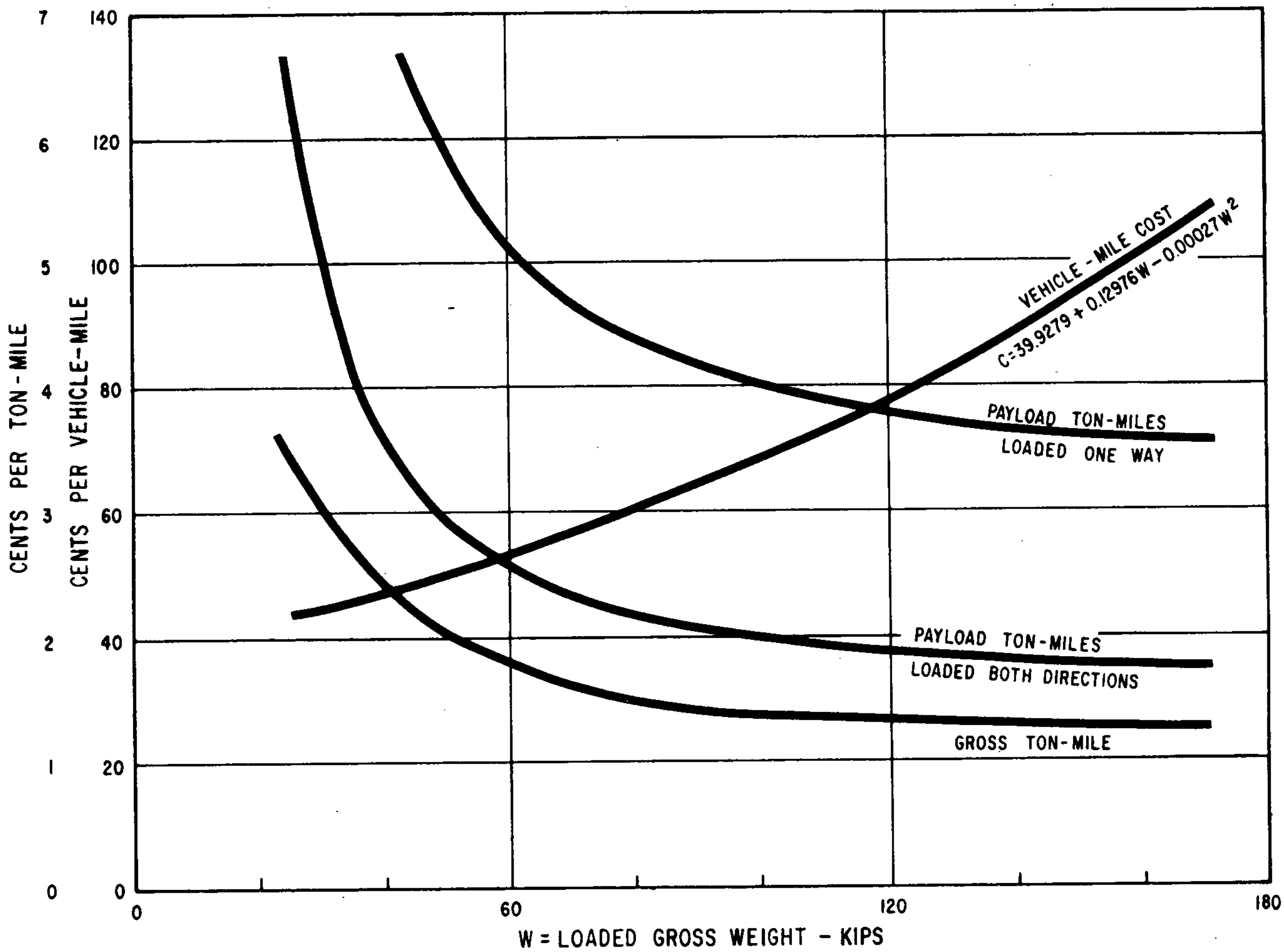


Figure 55. Line-haul truck costs by gross ton-miles and payload ton-miles (loaded one way, return empty, and loaded both ways).

reached, it is recommended that this position be reviewed, and that appropriate studies be implemented, should methodology techniques now available appear promising.

COST/BENEFIT ANALYSIS

Cost/benefit analysis refers to the several procedures of economic analysis that can be applied to a proposed change to relate initial and subsequent costs of implementing a new process, operation, or physical facility to the gross of net benefits, or gains, expected to be realized therefrom. The purpose of the cost/benefit analysis is to furnish information on a systems analysis basis that can be used by decision-makers together with other decision aids and criteria in the authorization or rejection of a proposed change.

These analysis techniques can be applied to a range of possible changes in legal vehicle limits as a means of comparing the resulting highway costs of capital improvements and annual maintenance expense with the changes in motor vehicle operating costs in the transport of goods and people and other economic effects imposed on nonusers.

Meaning of "Benefit"

With specific reference to the consequences of highway improvements, Winfrey (*85*) says that "the resulting benefits are generally considered to include those consequences that are desired and those that reduce transportation cost, other social or economic cost, or which produce satisfactions not otherwise enjoyed." He emphasized that this definition is not that generally used by economists to whom a benefit is the total cost one pays for a service or satisfaction. Thus, a benefit relates to the pricing of a commodity or service.

The context intended in this study closely relates savings with benefits, which infer a before-and-after type of comparison. A monetary saving is defined as the reduction in the cost of a product or service effected by a change to a process, procedure, or device, thereby accomplishing the same objective as before but at less cost.

A benefit may be realized without having gained a true monetary saving by reason of not having the before-after relationship. Thus, the saving can be a hypothetical one that does not accrue in a manner that could be applied toward paying for the facility.

Any action that produces an increase in the cost of operation or use of a facility over that which was formerly maintained is a "disbenefit" to the segment of society that suffers this increase. In this context, disbenefits can be considered as a cost element, additive to other costs for implementing the change.

In this analysis technique, the measurement of benefits is required. Practical measures of benefits are difficult, and no simple method is available that will result in unique and unambiguous results. Essentially, the difficulty is that the normal analytical and empirical tools typified by supply and demand curves of conventional, static economic analysis do not convey all information required to perform this evaluation.

Benefit/Cost Ratio

On the assumption that all benefits and disbenefits derived from a proposed change in legal vehicle limits can be quantified in monetary terms, the application of the benefit/cost ratio method could be a powerful means of evaluating the wisdom of the proposed change. In fact, this method has been used extensively in the economic analysis of highway improvements since about 1952.

The benefit/cost ratio method provides an abstract dimensionless number representing the ratio of net benefits to net costs. As an index, the ratio calculated using benefits and costs from a prior or existing situation may be compared with the ratio projected on the basis of the proposed alternative.

In such a comparison, the emphasis is on benefits that may accrue to different groups of users and nonusers. To apply the method, it is not necessary, as in the case of cost allocation studies, to identify separately benefactors and beneficiaries. Factors such as taxation and finance are not a part of the evaluation process.

Application of Assembled Method to B/C Ratio Method

One objective of this study was to identify the benefits and disbenefits, and to assemble methods that might be applied to the benefit/cost ratio method of evaluation. To some degree, this objective has been achieved.

In the consideration of a proposed change in legal limits, the objective should be that of determining the incremental change in costs and benefits, rather than absolute total values. Incremental costs are the cost differentials or increments obtained by computing elemental costs, assuming (1) no change in legal limits ("before"), and (2) the proposed change is in effect ("after"). The difference between these "before" and "after" costs is termed the "incremental costs" attributable to the cost element resulting from the proposed change.

Table 51 gives the more obvious general consequences that could result from a change in legal maximum limits. Code letters indicate the present capabilities or maturity of methods as an index of conclusions of this study. This table indicates that gaps in knowledge and developed methods exist in the present capabilities. Research to overcome these deficiencies (identified after a literature search) is beyond the scope of this project.

The numerical example included here deals only with those transportation costs that can be quantified in terms of cost. The environmental factors, general economic factors of the community, and social consequences are not presently quantifiable in these terms. The effects on traffic operations and their influence on safety, congestion, and accident costs are likewise not amenable to quantification at this time.

The study did uncover methods permitting the estimation of costs involved in constructing new pavements and bridges to both present and proposed axle weight and gross weight limits. A method of predicting possible axle configurations resulting from changes in legal limits is suggested so that they might be examined and compared with present critical design vehicles used in bridge design. Thereby, the likely new vehicles whose characteristics reflect these proposed limit changes can be judged as being within present critical design loads or as being new critical loads. With these methods it is possible to develop equivalent uniform annual capital costs required for these highway facilities.

Similarly, the line-haul trucking costs, before and after, and for both base and period end years, can be estimated to provide the measure of benefits to the motor freight industry. The benefits are then reduced to equivalent uniform annual benefits.

The cost/benefit analysis then determines a ratio of equivalent uniform annual benefits to equivalent uniform annual capital costs.

Acceptance or rejection of proposed changes in legal limits should not be decided solely on the strength of this cost/benefit analysis. As is true with the evaluation techniques of most highway improvements, the decisions require tradeoffs and evaluations also, often based on subjective judgment of other factors not quantifiable in terms of costs.

In the numerical example (Appendix D) the concept determines the change in highway costs and in vehicle operating costs for one mile of new highway construction, including a pro rata cost of constructing the average number of feet of bridges per mile of typical bridges used on the highway system being considered. Vehicle operating costs are derived by assuming that the same total tons of cargo are hauled over that one mile under the "before" and "after" conditions. This concept produces a relative measure of transportation economy of the proposed legal limit changes because the "before" and "after" conditions carry the same payload tonnage for the same one mile of highway.

If this analysis indicates a gain in economy, it follows that there also would be a gain in economy of transportation from reconstructing, strengthening, upgrading, and maintaining existing highways.

Other highway cost items, such as annual pavement and bridge maintenance under present and proposed load limits, properly belong in a cost/benefit analysis. The study failed to uncover a method sensitive to size and weight of trucks that could be employed to determine how these changes would affect highway maintenance costs. Maintenance cost accounting procedures vary among the states; but, more importantly, these procedures are not capable of differ-

TABLE 51

CONSEQUENCES OF CHANGING LEGAL MAXIMUM LIMITS OF VEHICLES AND INDEX TOWARD QUANTIFYING THESE CONSEQUENCES

ITEM OF CONSEQUENTIAL CHANGE (BENEFIT OR COST)	DIMENSION OR WEIGHT ITEM TO BE INCREASED OR DECREASED IN LEGAL MAXIMUM LIMIT				
	Axle Weight	Gross Weight	Length of Vehicle	Width of Vehicle	Height of Vehicle
A	B	C	D	E	F
TRANSPORTATION FACTORS RELATED TO THE HIGHWAY AND TO THE VEHICLE					
1. Changes in the cost of operating vehicles.	A	A	B	B	B
2. Changes in the total per ton-mile, cost of transport of payload.	A	A	B	B	B
3. Changes in loading practices of vehicles.	B	B	B	B	B
4. Changes in distribution of cargo afforded by multiple unit conditions.	F	F	B	F	F
5. Changes in packaging and loading of vehicles (stowage, containers, modular dimensions) as affecting manufacturing, size of lot purchases, and warehousing.	B	B	B	B	B
6. Number of vehicle-trips on the highway to transport the same total tons of cargo or cubic feet of cargo.	A	A	B	B	B
7. Incremental change in future date when added capacity will be needed to accommodate truck ADT under "before" and "after" conditions.	C	C	C	C	C
8. Change in number of special permits to be issued for oversize and overweight vehicles.	B	B	B	B	B
9. Incremental cost to traffic because of posting existing bridges against use by vehicle weighing more than a safe load.	C	C	C	F	F
10. Impact on other transport modes due to changes in modal split resulting from truck transportation factors.	C	C	C	C	C
HIGHWAY PAVEMENTS					
1. Incremental change in cost of new pavement construction designed for traffic under a change in legal limits.	A	A	C	C	C
2. Incremental cost of constructing or not constructing overlays to existing pavements.	A	A	B	F	F
3. Incremental change in years of probable remaining usefulness of existing pavements as compared to expectancy under existing legal limits.	A	F	F	F	F
4. Incremental change in cost of pavement designed for a change in legal limits.	A	A	B	B	F
5. Incremental change in cost to maintain existing pavements.	C	C	C	C	F
HIGHWAY BRIDGES					
1. Incremental change in cost of constructing new bridges designed for the change in legal limits.	A	A	A	C	C
2. Incremental changes in cost of strengthening existing bridges.	A	A	A	F	F
3. Incremental change in cost to maintain bridges designed for a change in legal limits.	C	C	C	C	F
4. Incremental change in cost to maintain existing bridges.	C	C	C	C	F

entiating the maintenance costs by size and weight of vehicles.

Although no universal method is included here, it may be possible under some applications to develop incremental maintenance costs of pavements and bridges. In these cases, their inclusion is encouraged, but these items are omitted in the numerical example.

Figure 56 shows the logic flow in performing a cost/benefit analysis. Should maintenance costs be determinable in a specific application, it would be preferable to subtract these cost increments, reduced to equivalent uniform annual operating costs, from the benefit prior to calculating the benefit/cost ratio. The method used in the numerical example in Appendix D follows this general flow.

ITEM OF CONSEQUENTIAL CHANGE (BENEFIT OR COST)	DIMENSION OR WEIGHT ITEM TO BE INCREASED OR DECREASED IN LEGAL MAXIMUM LIMIT				
	Axle Weight	Gross Weight	Length of Vehicle	Width of Vehicle	Height of Vehicle
A	B	C	D	E	F
HIGHWAY GEOMETRICS					
1. Incremental change in cost of widening traffic lanes of existing pavements, earlier or later in time than they would be under existing legal limits.	D	D	D	D	F
2. Incremental change in cost to increase or decrease vertical clearance on existing highways and future highway construction.	F	F	F	F	D
3. Incremental change in cost to construct horizontal curves and access ramps to accommodate change in off-tracking and lane clearance.	F	F	D	D	F
4. Incremental cost to construct intersections to accommodate changes in off-tracking and lane clearance.	F	F	D	D	F
5. Incremental change in cost to construct highways at different vertical gradients and/or to construct up-hill truck lanes.	C	C	C	C	F
HIGHWAY TRAFFIC OPERATIONS					
1. Incremental impacts on vehicle running costs and travel time, including the effects of changed ADT of trucks under the changed legal limits.	A	A	B	B	B
2. Incremental impact of changes in driver sight distances affecting traffic operations.	F	F	C	C	C
3. Incremental impacts upon incidence and severity of traffic accidents.	C	C	C	C	C
SOCIAL, COMMUNITY, AND ENVIRONMENTAL					
1. Change in consumer prices for goods resulting from change in cost per ton-mile to haul cargo, or deferred price changes from inflation because of change in cost of transportation and distribution.	D	D	D	D	D
2. Incremental cost due to a change in traffic noise levels.	E	E	E	E	E
3. Incremental costs due to a change in air pollution levels.	E	E	E	E	E
4. Incremental costs due to a change in land and building vibration resulting from moving vehicles.	E	E	E	E	E
5. Incremental change in the availability of goods, either in quantity or in timing of delivery.	D	D	D	D	D
6. Incremental change in the type, quality, class, or form of goods available to a given community.	E	E	E	E	E
7. Incremental costs in travel time due to impact of trucks on traffic operations.	C	C	C	C	C

KEY TO LETTER MATURITY CODES USED IN TABLE 51

A. Methodology included in report from computing influence of an increase or decrease in legal limits.

B. Methodology is not included in report because data not available; adaptation of existing methodology and data collection thought feasible within state-of-the-art.

C. Methodology is not included in report because of deficiency in state-of-the-art in one or more significant influence areas.

D. No general methodology is included; incremental values cannot be quantified or method lacks sensitivity; some indication of magnitude of influence can be obtained from gross computations.

E. Methodology does not exist in present state-of-the-art; values can neither be quantified nor influences grossly estimated; present evaluation solely by subjective judgment.

F. Item appears to have little or no influence on legal limit parameter.

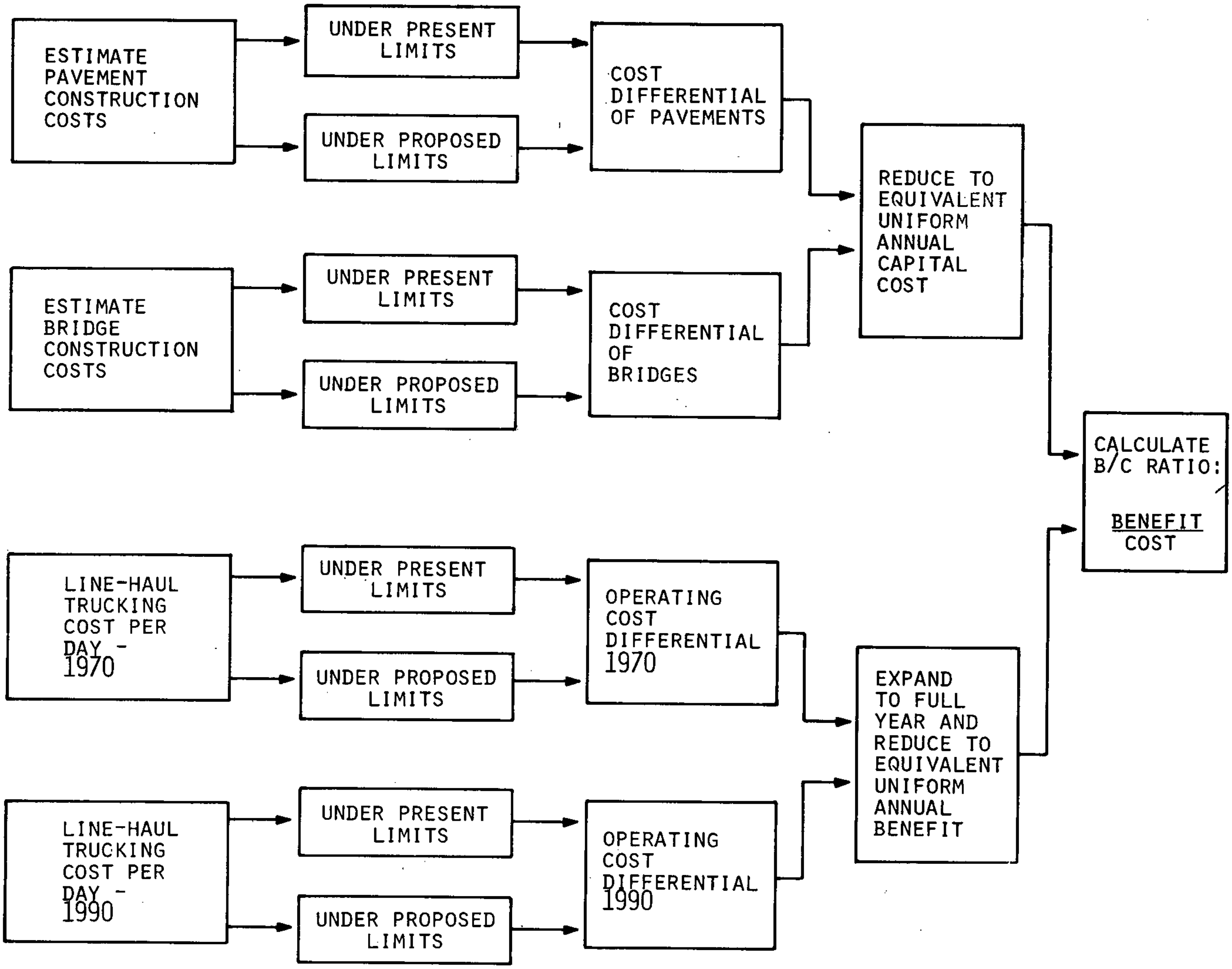


Figure 56. Logic flow in performing cost/benefit analysis.

CHAPTER THREE

INTERPRETATION, APPRAISAL, APPLICATION

In scaling economic impacts of changes in legal vehicle weights and dimensions, the approach should be that of comparing "before and after" impacts. Such an approach is termed an incremental method here—similar, but not restricted to that interpretation of the incremental method of cost allocation.

Any valid method used to estimate the impact should be capable of estimating the economic effects of decreases as well as increases in existing limits. The methods assembled in the study appear to meet this requirement.

In making these no-change/with-change comparisons, an estimate should be made of benefits and costs that would accrue if the highway system were to operate without a change in existing legal limits. Using the same inputs and assumptions, another estimate is made of the effect should the changes be implemented.

The nature of the change should be analyzed for possible impacts on the physical properties of the highway system, on beneficiaries of the change, on other highway users, to the socioeconomic climate, and on nonusers of the highway facility. The extent of these analyses is presently limited by existing methods.

ESTIMATE OF MOTOR FREIGHT TONS AND TON-MILES

The method assembled in this study provides an estimate of tons of motor freight and ton-miles generated in its transport, based on the relationship between GNP and the tonnage and ton-miles of motor freight. Extrapolation of these factors generally has been accepted and applied to many economic forecasts.

The national figure is then prorated to each state in a proportion of the consumption of special fuels in that state to the total national special fuel consumption. Thus, national projections are related to the states in a manner that reflects present and projected freight movements in each state. The method reflects unique state differences in motor transport due to industrial, agricultural, and other economic activity. An illustrative example of such a project appears in Chapter Two.

The simplified method was chosen after an evaluation of more sophisticated econometric modeling techniques. Although these more complex methods are valuable for other purposes, they appeared to require extensive data, calibration, and data processing. Suitable econometric modeling admittedly would be more flexible, and possibly more sensitive, but did not appear in the literature. Should a simpler model emerge that provides this desirable flexibility, it should be substituted for the rather simple form included here.

FORECASTING EXTENT OF APPLICATION OF A NEW LIMIT

The extent of application of new limits depends on the nature of the cargo being carried within the jurisdiction and the future nature of that cargo should new limits be enacted. Comparisons should be made of legal limits of surrounding states and the present and proposed limits of the state being considered. Differences in these limits could affect future highway loadings. Through-state movements of vehicles and loads not now permitted under existing limits obviously will not be reflected in present data. Therefore, the increase in through-state truck traffic cannot be predicted from present state truck-weight study data.

Changes in dimensional limits without changes in axle-load limits may result in changes in the distributions of axle loads. Such trends will affect pavement wear and average performance of the vehicle in the traffic stream, but cannot be predicted from present truck-weight study data.

The study method relies on truck-weight study data to obtain axle weight distributions, empty weights, gross weights, payloads carried, and frequency of truck classes on various types of highways. The practical maximum gross weight (PMGW) of present vehicles is compared with the PMGW of vehicles expected to be employed under the proposed limits. From these comparisons, new axle weight distributions, gross weights, and payloads for each class of vehicle on each type of highway are projected.

Axle weight distributions under present limits and under proposed limits are converted into equivalent 18-kip single-axle load applications for each class of truck and type of highway. The number of vehicles required to carry the projected motor freight over the planning period is computed assuming (1) no change in legal limits, and (2) the proposed change is in effect.

The number of vehicles by types is associated with the equivalent 18-kip single-axle load applications and is summed to obtained the total load experience anticipated under present and proposed load limits. These computations are made for all types of highways in the system. An example of this method is given in Appendix D.

The procedure necessarily assumes that the truck-weight studies adequately reflect the existing truck mix and loading. Care should be taken to ensure that the weight samples represent reasonable samples of the actual truck population.

An analysis must be made of how changes in legal limits of gross weight may affect vehicle axle configuration. This is a vehicle design problem that can best be determined by consultation with automotive engineers, manufacturers, and vehicle operators. Any new configuration anticipated under new size and weight limits must be compared with existing critical vehicles used in present bridge design. The application of such a method is described and illustrated in Appendix D.

Changes in length that would permit double- or triple-bottom combinations on the highway can affect the numbers of trucks and their axle weight distributions. Adjustments must be made to the computations of axle weight distribution to account for these new vehicle classes. Judgment based on distributions of comparable truck types in truck weight study data of other states is the recommended approach. The format of this analysis follows that illustrated in Appendix D.

The results of changes in length affecting truck combinations and vehicle designs also must be analyzed for vehicle-handling and performance characteristics. This is an automotive design problem also. Likely dimensions of these new vehicles can best be determined by automotive engineers and vehicle operators.

ESTIMATING PHYSICAL IMPACTS ON PAVEMENTS, AND COSTS

Projections of total highway load experience under existing and proposed limits permit an estimate of the costs that would result from a change in legal limits.

The method employed here assumes that the principles relating highway design and wear to axle load applications as found in the AASHO Road Test are valid; viz., highways deteriorate in a predictable fashion as a function of the accumulation of equivalent 18-kip single-axle load applications. It further assumes that remaining service life of highway pavements, both flexible and rigid, can be evaluated by first determining present serviceability index (PSI) through measurement, or present serviceability rating (PSR) through visual inspection by a rating panel. PSR is assumed to be convertible to PSI.

By applying appropriate factors in accordance with the type of pavement and its design and composition, including thickness, and, in the case of flexible pavements, the base, subbase, and soil types, PSI can be used to compute the number of equivalent 18-kip axle load applications corresponding to the remaining service life of existing pavements. Further, maintenance costs are assumed to be a function of these equivalent 18-kip axle load applications. Maintenance costs may be adjusted in proportion to the magnitude of this total load experience. New pavements may be designed to these anticipated equivalent single-axle load applications to provide satisfactory service for a given expected "life.'

It further assumes that the results of the AASHO Road Test (which was for one type of meteorological environment, soil condition, and type of load application) can be translated to meet local conditions and traffic mix prevailing in the state being considered.

Finally, it assumes that highway classification and need studies provide an inventory of highway types, design factors, lengths, serviceability ratings, and costs that can be used as input to the method. This inventory should not only reflect the present condition of existing highways, but also indicate the type and length of highways planned to be constructed over the planning period.

With these data inputs, the cost analysis can be made as illustrated in Appendix C. Two alternative methods are illustrated in this example, with preferences for their application given. Costs are estimated for highway load experiences projected on the no-change/with-change basis. The difference between these two estimates is the pavement cost that can be anticipated if the new change is enacted.

ESTIMATING PHYSICAL IMPACTS ON STRUCTURES, AND COSTS

Changes in axle spacing and axle load distribution of vehicles will have the greatest effect on structures. While this is indirectly related to gross vehicle weight, it must be translated to axle load configuration.

In the assembled methods, a compromise was required between accuracy and detailed analysis, and the economics of application of the method. An underlying hypothesis in assembling the methods was that structural cost elements could be estimated for most bridge types found in highway system inventories. Some specialized structural types would require specific rather than general cost analysis.

The method is statistical rather than specific—oriented to office-type procedures. Sound engineering assumptions were balanced against mathematical rigor to minimize mathematical complexity. The method cannot be expected to substitute for detailed engineering analysis of an individual structure to determine its specific, individual structural integrity.

The method covers flat slab, reinforced concrete T beam, reinforced concrete box girder, prestressed concrete girder, and steel girder type bridges. The deck, pier, and foundation of these bridge types also are considered.

Structures in the inventory are analyzed and placed into one of three categories: requires immediate replacement; can continue in service with modification; or can continue in service without change.

The summary of vehicle weight/size and anticipated new axle configurations is used to calculate the overstress factor. Based on the establishment of a permissible overstress factor, the designs for existing bridges are compared with these criteria to permit the previous categorization.

Bridges planned for construction are assumed to be included in the needs studies. Their designs are similarly analyzed to determine the need for increased load ratings commensurate with projected loads. The cost of upgrading the load ratings must then be estimated. The construction costs of the planned bridges can then be related to the proposed limit change.

Secondary structures are analyzed. Items such as culverts and retaining walls may be affected by changes in legal limits.

Maintenance cost increments must be identified, and judgment factors must be applied to determine increases in these costs if the legal limits are changed.

Appendix B gives the procedure for analysis and estimating structural cost impacts.

IMPACTS OF CHANGES IN LEGAL LIMITS ON GEOMETRIC DESIGN

Methods for determining the impacts of legal limits on geometric design are not as specific as those related to physical properties of pavements and bridges. The complexities of this analysis are caused by interactions between possible combinations of changes in legal limits and geometric design principles. Specific step-by-step methodology must be replaced by engineering evaluation. Many of the geometric design standards were based on studies that did not evaluate the influence of truck parameters specifically. The review of geometric design parameters was included in this report to furnish the basis of this engineering judgment.

DECISION FACTORS DEVELOPED BY RECOMMENDED METHOD

The study results in an assembly of methods for conservatively estimating the incremental cost impact on physical highway facilities. The magnitude of these differences thereby becomes the primary decision factor to be used in the judgment of the economic impact of changes in legal vehicle weights and dimensions.

CHAPTER FOUR

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

1. Any measure of economic impacts of changes in legal vehicle limits must be based on an incremental computational procedure—one that permits a calculation of impacts over the planning period with no change in limits, and one that calculates the impacts, assuming legal limits are changed. This no-change/with-change procedure would provide the cost increment attributable to the proposed legal limit change.

2. Methods are assembled here that relate to the cost impacts on physical facilities of highways of proposed changes in legal limits.

3. Economic impacts on highway operations cannot be calculated by present methods because they are not sufficiently sensitive to changes in vehicle dimensions or weights. Engineering judgment based on established design parameters must be substituted for formal methods in judging most geometric design standards.

4. Benefits and disbenefits relating to motor vehicle freight operations, safety, noise, property values, and other socioeconomic environments cannot be accurately measured and the cost impacts cannot be estimated by existing means because of their lack of sensitivity to vehicle weights and dimensions.

5. It follows, therefore, that the necessary benefit/cost method of analysis related to changes in legal vehicle limits and reflecting all areas of possible benefits and disbenefits cannot be formulated within the present knowledge and art.

6. Decision factors can be developed for evaluating the economic impact of changes in legal limits based on cost differentials of the pavements and bridges that must be reconstructed, constructed, replaced, and maintained to accommodate new limit loads. These methods are presented herein.

7. Extensive research is required in a number of disciplines in order to realize an improved total comprehensive method of evaluation.

RECOMMENDED RESEARCH

Research is required in a number of disciplines so that a more sophisticated and sensitive evaluation of total economic impacts of changes in legal vehicle limits can be made. These areas are as follows.

Transport Economics

1. Research is required to develop a simplified means of determining the economic benefits accruing to the motor freight carrier that result from increases in legal vehicle limits.

2. A program needs to be established that permits modeling, in a simple form and with minimum data requirements, of the motor freight tonnage, ton-miles and routes over which the cargo is carried, sensitivity to changes in economic activity, commodities carried, percent loaded, and other realistic variables.

3. Studies should be continued of the effects of increased motor freight operations resulting from liberalized legal limits on other freight transport modes.

Determining Total Highway Load Experience

1. A research program is recommended to determine the feasibility of applying statistical analysis of axle weight distributions, performed by computer processing that uses existing statistical programs rather than manual plotting and curve fitting required in the proposed method.

2. Present truck weight studies gather data related to rural and urban highway traffic. The scarcity of comparable data on truck movements on urban streets inhibits the evaluation of economic impacts of changes in legal limits on those facilities. Therefore, a study should be made to develop a practical means of gathering these truck data.

Pavement Performance

The method for determining the effects on pavements is based primarily on the application of the axle load equivalence factors developed from the AASHO Road Test. The limitations and possible errors in these factors are discussed elsewhere. It is recommended that research be conducted to verify the reliability of the AASHO equivalence factors and, if necessary, to modify these factors or to develop a more precise method for representing the applied loads in pavement design and evaluation. Both theoretical and experimental investigations are required to establish the basic relationships among all variables affecting pavement performance. Specifically, the effects on pavements of variations in wheel and axle configurations and weights are to be quantified. Findings from the suggested research should permit refinements in pavement design methods in general, and improvements in the procedures for determining the effects of changes in particular.

Bridge Structure Evaluation

1. The methods described here could be beneficially reduced to a computer program to perform the necessary calculations and data processing. Bridge inventories and new axle configurations would be the input to such a program. The output would be the cost estimates and summaries required for the structural cost analysis.

2. A research study should be implemented on a statistical analysis of the proposed method to confirm the validity of this mathematically derived method.

3. Continued research into structural fatigue life of

bridges is recommended. Although estimate procedures are included in this report, substantial refinement in the state of the art is required to improve this accuracy of the method.

Economic Impacts of Highway Noise

1. Continued research is recommended into highway noise components and levels as they relate to annoyance complaints attributed to highway noise. Although acoustic values can be adequately measured, the physiological and psychological aspects of the composite noise are not adequately identified and correlated to annoyance and complaints.

2. Continued research is encouraged into the economic impacts on property values, apartment rents, and fatigue and working inefficiencies induced by highway noise. Such research probably will be achievable only after problem (1) is better defined.

3. Research is required to establish and determine the relationship between truck and bus size and/or weight and their contribution to the composite highway noise discovered as a result of problem (1).

Safety Research

1. Research studies are recommended to relate the impacts of discrete changes in vehicle sizes and weights to highway safety. Accident incidence rates alone are not considered sufficient indices of highway safety. Combinations of other indices (e.g., severity, property damage, property loss, and nature and direction of collision) must be interpreted with skill. Pre-crash, crash, and post-crash phases of the accident should be considered as a possible approach to define the contributions of vehicle size and/or weight to the occurrence. Comparative studies of accidents in states with different legal limits might be beneficial.

Traffic Operations and Geometric Design

1. The relationship between vehicle dimensions and weights and traffic operations should be the subject of a series of comprehensive traffic operation studies. The concept of car equivalency factors of trucks in mixed traffic has been useful, but does not relate to vehicle weights and dimensions. The influences of increased vehicle dimensions on traffic flow, capacity, and other traffic operations require better definition. The consideration of the value of time to other highway users, a concept often used in urban transportation and toll road feasibility studies, could be related to new dimensions of vehicles operating under new legal limits, if better insight into the operational aspects can be developed.

2. Continued research is required in the area of geometric design principles to more specifically relate vehicle size and weight changes to such geometric considerations as lane widths, curve widths, and shoulders.

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APPENDIX A

UPDATED 1970 TRUCK OPERATING COSTS FROM REF. 125

The equations at the end of the section on "Line-Haul Trucking Costs" in Chapter Two were extended by computer to produce Table A-1.

TABLE A-1

UPDATED 1970 TRUCK OPERATING COSTS

LOADED GROSS VEHICLE WEIGHT (1) KIPS	Repair, Servicing & Lubri-cants	Tires and Tubes	Fuel	Driver Wage and Subsistence	Overhead and Indirect	Depreciation and Interest	TOTAL
	LINE-HAUL OPERATING COSTS--CENTS PER VEHICLE-MILE FOR GASOLINE AND DIESEL FUEL TRUCKS COMBINED						
	(Factor) 1.341	(Factor) 1.222	(Factor) 1.166	(Factor) 1.289	-	(Factor) 1.277	
20	6.67	1.70	2.74	14.67	14.10	3.27	43.15
21	6.67	1.71	2.74	14.70	14.11	3.41	43.34
22	6.67	1.71	2.74	14.73	14.12	3.55	43.52
23	6.68	1.72	2.74	14.76	14.14	3.69	43.73
24	6.69	1.73	2.74	14.79	14.15	3.84	43.94
25	6.70	1.74	2.74	14.82	14.16	3.98	44.14
26	6.71	1.75	2.74	14.85	14.18	4.12	44.35
27	6.73	1.76	2.75	14.88	14.19	4.26	44.57
28	6.75	1.77	2.75	14.91	14.20	4.40	44.78
29	6.77	1.79	2.75	14.94	14.21	4.54	45.00
30	6.79	1.80	2.76	14.97	14.23	4.68	45.23
31	6.82	1.81	2.76	15.00	14.24	4.82	45.45
32	6.84	1.83	2.76	15.03	14.26	4.95	45.67
33	6.87	1.85	2.77	15.06	14.27	5.09	45.91
34	6.91	1.86	2.77	15.09	14.28	5.23	46.14
35	6.94	1.88	2.77	15.13	14.30	5.36	46.38
36	6.98	1.90	2.78	15.16	14.31	5.50	46.63
37	7.02	1.92	2.78	15.19	14.33	5.63	46.87
38	7.05	1.94	2.79	15.22	14.34	5.77	47.11
39	7.10	1.96	2.79	15.25	14.35	5.90	47.35
40	7.15	1.98	2.80	15.28	14.37	6.03	47.62
41	7.20	2.00	2.80	15.31	14.38	6.17	47.86
42	7.25	2.03	2.81	15.34	14.40	6.30	48.13
43	7.30	2.05	2.82	15.38	14.41	6.43	48.39
44	7.36	2.07	2.82	15.41	14.43	6.56	48.64
45	7.42	2.10	2.83	15.44	14.44	6.69	48.92
46	7.48	2.13	2.84	15.47	14.46	6.82	49.20
47	7.55	2.15	2.84	15.50	14.47	6.95	49.46
48	7.61	2.18	2.85	15.54	14.49	7.08	49.75
49	7.68	2.21	2.86	15.57	14.51	7.21	50.04
50	7.75	2.24	2.87	15.60	14.52	7.34	50.32
51	7.82	2.27	2.87	15.63	14.54	7.47	50.60
52	7.90	2.30	2.88	15.67	14.55	7.59	50.89
53	7.98	2.33	2.89	15.70	14.57	7.72	51.19
54	8.06	2.37	2.90	15.73	14.59	7.84	51.49
55	8.14	2.40	2.91	15.76	14.60	7.97	51.78
56	8.23	2.43	2.92	15.80	14.62	8.09	52.09
57	8.31	2.47	2.93	15.83	14.63	8.22	52.39
58	8.40	2.51	2.94	15.86	14.65	8.34	52.70
59	8.49	2.54	2.95	15.90	14.67	8.47	53.02
60	8.59	2.58	2.96	15.93	14.68	8.59	53.33
61	8.69	2.62	2.97	15.96	14.70	8.71	53.65
62	8.79	2.66	2.98	16.00	14.72	8.83	53.98
63	8.89	2.70	2.99	16.03	14.74	8.95	54.30
64	8.99	2.74	3.00	16.06	14.75	9.07	54.61
65	9.10	2.78	3.01	16.10	14.77	9.19	54.95
66	9.21	2.82	3.02	16.13	14.79	9.31	55.28
67	9.32	2.87	3.04	16.17	14.81	9.43	55.64
68	9.44	2.91	3.05	16.20	14.82	9.55	55.97
69	9.55	2.95	3.06	16.23	14.84	9.67	56.30
70	9.67	3.00	3.07	16.27	14.86	9.78	56.65
71	9.79	3.05	3.09	16.30	14.88	9.90	57.01
72	9.91	3.09	3.10	16.34	14.90	10.02	57.36
73	10.04	3.14	3.11	16.37	14.92	10.13	57.71
74	10.17	3.19	3.13	16.41	14.93	10.25	58.08
75	10.30	3.24	3.14	16.44	14.95	10.36	58.43
76	10.43	3.29	3.15	16.48	14.97	10.48	58.80
77	10.57	3.34	3.17	16.51	14.99	10.59	59.17
78	10.71	3.39	3.18	16.55	15.01	10.70	59.54
79	10.85	3.45	3.20	16.58	15.03	10.81	59.92
80	10.99	3.50	3.21	16.62	15.05	10.93	60.30
81	11.14	3.55	3.23	16.65	15.07	11.04	60.68
82	11.28	3.61	3.24	16.69	15.09	11.15	61.06
83	11.43	3.67	3.26	16.72	15.11	11.26	61.45
84	11.59	3.72	3.27	16.76	15.13	11.37	61.84
85	11.74	3.78	3.29	16.80	15.15	11.47	62.23
86	11.90	3.84	3.31	16.83	15.17	11.58	62.63
87	12.06	3.90	3.32	16.87	15.19	11.69	63.03
88	12.22	3.96	3.34	16.90	15.21	11.80	63.43
89	12.38	4.02	3.36	16.94	15.23	11.91	63.84
90	12.55	4.08	3.37	16.98	15.25	12.01	64.24
91	12.72	4.14	3.39	17.01	15.27	12.12	64.65
92	12.89	4.21	3.41	17.05	15.29	12.22	65.07
93	13.07	4.27	3.43	17.09	15.31	12.33	65.50
94	13.24	4.33	3.45	17.12	15.33	12.43	65.90
95	13.42	4.40	3.47	17.16	15.35	12.54	66.34
96	13.60	4.47	3.48	17.20	15.38	12.64	66.77
97	13.79	4.53	3.50	17.23	15.40	12.74	67.19
98	13.97	4.60	3.52	17.27	15.42	12.84	67.62
99	14.16	4.67	3.54	17.31	15.44	12.95	68.07
100	14.35	4.74	3.56	17.35	15.46	13.05	68.51
101	14.54	4.81	3.58	17.38	15.48	13.15	68.94
102	14.74	4.88	3.60	17.42	15.51	13.25	69.40

TABLE A-1 (*Continued*)

LOADED GROSS VEHICLE WEIGHT KIPS (1)	Repair, Servicing & Lubricants	Tires and Tubes	Fuel	Driver Wage and Subsistence	Overhead and Indirect	Depreciation and Interest	TOTAL
	LINE-HAUL OPERATING COSTS--CENTS PER VEHICLE-MILE FOR GASOLINE AND DIESEL FUEL TRUCKS COMBINED						
103	14.94	4.95	3.62	17.46	15.53	13.35	69.85
104	15.14	5.03	3.64	17.50	15.55	13.44	70.30
105	15.34	5.10	3.67	17.53	15.57	13.54	70.75
106	15.55	5.17	3.69	17.57	15.59	13.64	71.21
107	15.75	5.25	3.71	17.61	15.62	13.74	71.68
108	15.96	5.33	3.73	17.65	15.64	13.83	72.14
109	16.18	5.40	3.75	17.69	15.66	13.93	72.61
110	16.39	5.48	3.77	17.72	15.69	14.03	73.08
111	16.61	5.56	3.80	17.76	15.71	14.12	73.56
112	16.83	5.64	3.82	17.80	15.73	14.21	74.03
113	17.05	5.72	3.84	17.84	15.76	14.31	74.52
114	17.27	5.80	3.87	17.88	15.78	14.40	75.00
115	17.50	5.88	3.89	17.92	15.80	14.50	75.49
116	17.73	5.96	3.91	17.96	15.83	14.59	75.98
117	17.96	6.04	3.94	18.00	15.85	14.68	76.47
118	18.20	6.13	3.96	18.04	15.87	14.77	76.97
119	18.43	6.21	3.99	18.07	15.90	14.86	77.46
120	18.67	6.30	4.01	18.11	15.92	14.95	77.96
121	18.91	6.39	4.03	18.15	15.95	15.04	78.47
122	19.16	6.47	4.06	18.19	15.97	15.13	78.98
123	19.40	6.56	4.08	18.23	16.00	15.22	79.49
124	19.65	6.65	4.11	18.27	16.02	15.31	80.01
125	19.90	6.74	4.14	18.31	16.04	15.39	80.52
126	20.15	6.83	4.16	18.35	16.07	15.48	81.04
127	20.41	6.92	4.19	18.39	16.09	15.57	81.56
128	20.66	7.01	4.22	18.43	16.12	15.65	82.09
129	20.93	7.11	4.24	18.47	16.14	15.74	82.63
130	21.19	7.20	4.27	18.51	16.17	15.82	83.16
131	21.46	7.29	4.30	18.55	16.19	15.91	83.70
132	21.72	7.39	4.33	18.59	19.22	15.99	84.24
133	21.99	7.49	4.35	18.63	16.25	16.07	84.78
134	22.27	7.58	4.38	18.67	16.27	16.16	85.33
135	22.54	7.68	4.41	18.71	16.30	16.24	85.88
136	22.82	7.78	4.44	18.76	16.32	16.32	86.44
137	23.10	7.98	4.47	18.80	16.35	16.40	87.00
138	23.38	8.08	4.49	18.84	16.38	16.48	87.55
139	23.66	8.18	4.52	18.88	16.40	16.56	88.10
140	23.95	8.28	4.55	18.92	16.43	16.64	88.67
141	24.24	8.39	4.58	18.96	16.46	16.72	89.24
142	24.53	8.49	4.61	19.00	16.48	16.80	89.81
143	24.83	8.59	4.64	19.05	16.51	16.87	90.39
144	25.12	8.70	4.67	19.09	16.54	16.95	90.96
145	25.42	8.81	4.70	19.13	16.57	17.03	91.55
146	25.72	8.91	4.73	19.17	16.59	17.11	92.13
147	26.03	9.02	4.77	19.21	16.62	17.18	92.72
148	26.33	9.13	4.80	19.25	16.65	17.26	93.31
149	26.64	9.24	4.83	19.30	16.67	17.33	93.90
150	26.95	9.35	4.86	19.34	16.70	17.40	94.49
151	27.26	9.46	4.89	19.38	16.73	17.48	95.09
152	27.58	9.57	4.92	19.43	16.76	17.55	95.70
153	27.90	9.69	4.96	19.47	16.79	17.62	96.31
154	28.22	9.80	4.99	19.51	16.81	17.69	96.91
155	28.54	9.91	5.02	19.55	16.84	17.77	97.52
156	28.86	10.03	5.06	19.60	16.87	17.84	98.14
157	29.19	10.15	5.09	19.64	16.90	17.91	98.76
158	29.52	10.26	5.12	19.68	16.93	17.98	99.38
159	29.85	10.38	5.16	19.73	16.96	18.04	100.00
160	30.19	10.38	5.19	19.77	16.99	18.11	100.63
161	30.52	10.50	5.23	19.81	17.02	18.18	101.26
162	30.87	10.62	5.26	19.85	17.04	18.25	101.89
163	31.21	10.74	5.29	19.90	17.07	18.32	102.53
164	31.55	10.86	5.33	19.94	17.10	18.38	103.16
165	31.90	10.98	5.37	19.99	17.13	18.44	103.81
166	32.25	11.10	5.40	20.03	17.16	18.51	104.45
167	32.60	11.22	5.44	20.07	17.19	18.58	105.10
168	32.96	11.35	5.47	20.12	17.22	18.64	105.76
169	33.31	11.47	5.51	20.17	17.25	18.71	106.42
170	33.67	11.60	5.55	20.21	17.28	18.77	107.08
171	34.03	11.73	5.58	20.25	17.31	18.83	107.73
172	34.40	11.85	5.62	20.30	17.34	18.89	108.40
173	34.76	11.98	5.66	20.34	17.37	18.96	109.07
174	35.13	12.11	5.70	20.39	17.40	19.01	109.74
175	35.50	12.24	5.73	20.43	17.43	19.08	110.41
176	35.87	12.37	5.77	20.48	17.46	19.14	111.09
177	36.25	12.50	5.81	20.52	17.50	19.19	111.77
178	36.63	12.63	5.85	20.57	17.53	19.25	112.46
179	37.01	12.76	5.89	20.61	17.56	19.31	113.14
180	37.39	12.90	5.92	20.66	17.59	19.37	113.83

(1) See HRB Bulletin 301 and HRR 127 for definition of Loaded Gross Weight which is about 80 per cent of Practical Loaded Gross Weight.

APPENDIX B

STRUCTURAL DESIGN AND COST ANALYSIS METHODOLOGY

The evolved methodology for evaluation of highway structural design was based on computation of structural cost elements for most types of bridge structures found in current highway system inventories. The five most commonly used general types of bridges included in the analysis methodology are:

1. Flat slab.
2. Reinforced concrete T Beam.
3. Reinforced concrete box girder.
4. Prestressed concrete girder.
5. Steel girder.

In addition to the girder system, other major structural parts must be included in the analysis; i.e., deck, pier, and foundation.

The change in live load, resulting from an increase in legal vehicle weight limits or a change in axle configuration such as might result from changes in maximum vehicle length limits, probably will have an adverse structural effect on existing bridges.

Following the logic flow diagram of the proposed method (Fig. 28) it is required that present structures be classified by bridge type, original design load, span group, and the recorded material quantities of the original bridge design in the inventory. Overstress factors must be calculated for each applicable type, depending on new load limits. These must be compared with a permissible overstress factor, as established by the methodology user. These comparisons then permit bridges to be assigned to one of four general categories:

1. Need immediate replacement.
2. Post bridge to original limit.
3. Strengthen to serve new limit.
4. Upgrade load rating to serve new load limit without modification.

Cost elements for each of these categories are then computed and employed in the incremental cost study.

This appendix is divided into two major parts: Derivation of Computation Method, and Numerical Example of Method Application.

DERIVATION OF COMPUTATION METHOD

Table B-1 gives definitions of symbols used.

Live Load Conversion

Major Properties of an Axle Group (90)

Figure B-1 shows a general axle group. To describe the effects of such an axle group on simply supported beams, the following parameters can be used:

n = number of axles in an axle group;

$$W = \sum_{i=1}^{n} P_i;$$

$$\xi = \frac{2\sum_{i=1}^{n} P_i|x_i|}{Wb};$$

$$\mu = \frac{\sum_{i=1}^{n} P_i x_i}{Wb} = \frac{c}{b}; \text{ and}$$

$$\delta = \frac{\sum_{i=1}^{n} P_i z_i}{Wb} = \frac{r_{\min}}{b}.$$

Conversion of Given Axle Group to Equivalent Load Pattern (Fig. B-2)

Let

B_m = equivalent base length for moment, over which length the total weight must be uniformly distributed in order to generate the same maximum bending moment in a simply supported beam as the axle load itself;
B_v = equivalent base length for shear;
L = span length of the beam;
M_m = maximum bending moment; and
V_n = maximum end shear.

A given axle group may be converted into an equivalent load pattern through the following:

Truck load:

$$B_m = 2[\xi - (b/L)\mu^2]b$$

$$B_v = 2\delta b$$

$$M_m = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right)$$

$$V_n = W\left(1 - \frac{B_v}{2L}\right)$$

Lane load:

$$M_m = \frac{W/b}{2.75}(0.08L^2 + 4.5L)$$

$$V_n = \frac{W/b}{2.75}(26 + 0.32L)$$

Maximum moment, $\overline{M}$, and maximum shear, $\overline{V}$, associated with the new load group may be defined as:

$\overline{M}_L$ = maximum of either M_m of truck load or M_m of lane load times $(1 + I)$.
$\overline{V}_L$ = maximum of either V_n of truck load or V_n of lane load times $(1 + I)$.

in which I (impact factor) $= 50/(125 + L)$.

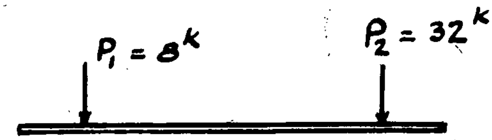
TABLE B-1

DEFINITIONS OF SYMBOLS USED IN STRUCTURAL ANALYSIS COMPUTATION METHOD

A = plan area of bridge (sq ft).
b = vehicle wheelbase.
B_m = equivalent base length for moment.
B_v = equivalent base length for shear.
C = total cost of new construction for a bridge.
C_f = cost attributable to new load limits.
C_m = miscellaneous costs of a bridge, for parapet, railing, curb, etc.
C_o = original cost of bridge.
C_s = cost for strengthening.
F = fatigue life of bridge.
$\bar{f}_c$ = concrete stress associated with new load limits.
f_c = concrete stress associated with original design load.
f_c' = 28-day concrete cylinder strength.
f_{ci} = compression strength of concrete at initial prestress.
f_y = yield stress of reinforcing or structural steel.
K_{cm} = permissible flexural overstress factor for reinforced concrete.
K_{cv} = permissible shearing overstress factor for reinforced concrete.
K_{sm} = permissible flexural overstress factor for structural steel.
K_{sv} = permissible shearing overstress factor for structural steel.
K_{pm} = permissible flexural overstress factor for prestressed concrete.
K_{pv} = permissible shearing overstress factor for prestressed concrete.
L = span length of beam.
M_D = maximum moment due to dead load.
M_L = live load moment plus impact due to original design load (AASHO).
$\overline{M}_L$ = live load moment plus impact due to new loading under new limit.
M_m = maximum bending moment.
M_T = total design moment due to original design load.
n = number of axles.
$\bar{P}$ = wheel loads under new limits.
P = wheel loads assumed in original design.
Q_s = average material quantity ratio for all structural elements.
r_{min} = distance between C.G. and reference (see Fig. B-1).
T = time elapsed between date of construction and date new limits are implemented.
U = unit material cost.
U_a = original unit cost of construction (\$/sq ft).
V_n = maximum end shear.
V_D = maximum end dead load shear.
V_L = original maximum end live load shear.
$\bar{V}_L$ = live load shear ratio of new load group under new limit.
W = gross vehicle weight.
α_m = ratio of maximum new live load moment to original maximum live load moment.
α_v = ratio of maximum new live load end shear to original maximum live load end shear.
β_m = ratio of dead load to live load moments.
β_v = ratio of dead load to live load shear.
γ = material quantity ratio of a structural element.
γ_a = average material quantity ratio of all structural elements.
γ_{sg} = material quantity ratio of a structural steel girder.
θ_b = sectional modulus ratio for bottom fiber of composite section to noncomposite section of girder.
ξ = ratio of C.G. location to half of wheelbase.
Ω = overstress factor.
$\Omega_m{}^S$ = flexural moment overstress factor for slab.
$\Omega_m{}^G$ = flexural moment overstress factor for girder.
Ω_f = overstress factor for foundation.
Ω_p = overstress factor for pier.

Conversion of AASHO Loadings to Equivalent Simple Load Patterns

HS20-44.—



in which $B_m \cong 11.2$ ft; $B_v \cong 5.6$ ft.

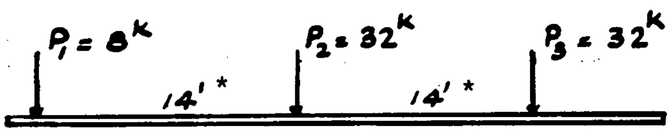
Truck load for $L \geqq 28$ ft:

$$M_m = (10L - 56)$$
$$V_n = 40\left(1 - \frac{2.8}{L}\right)$$

Lane load:

$$M_m = (0.08L^2 + 4.5L)$$
$$V_n = (26 + 0.32L)$$

HS20-44.—



Truck load:

$$M_m = \left(18L - 280 + 14\frac{b}{L}\right)$$
$$V_n = \left(72 - \frac{670}{L}\right)$$

Lane load:

$$M_m = \frac{72}{(28)(2.75)}(0.08L^2 + 4.5L)$$
$$V_n = \frac{72}{(28)(2.75)}(26 + 0.32L)$$

HS15-44.—

M_m = 75% of M_m for HS20-44 loading
V_n = 75% of V_n for HS20-44 loading

H15.—

M_m = 75% of M_m for H20-44 loading
V_n = 75% of V_n for H20-44 loading

Live Load Ratio

AASHO considers impact as a percentage of the total live load. Assume that this percentage applies also to the new load. In addition, assume that the lateral load-distribution criteria set forth by AASHO apply to the new loads as well.

Consequently, the live load moment and shear ratio of the new loads to AASHO loadings may be defined as:

$\alpha_m = \overline{M}_L / M_L$ = live load moment ratio of new load group against AASHO design load.

* This spacing is taken to be 14 ft to produce critical effect.

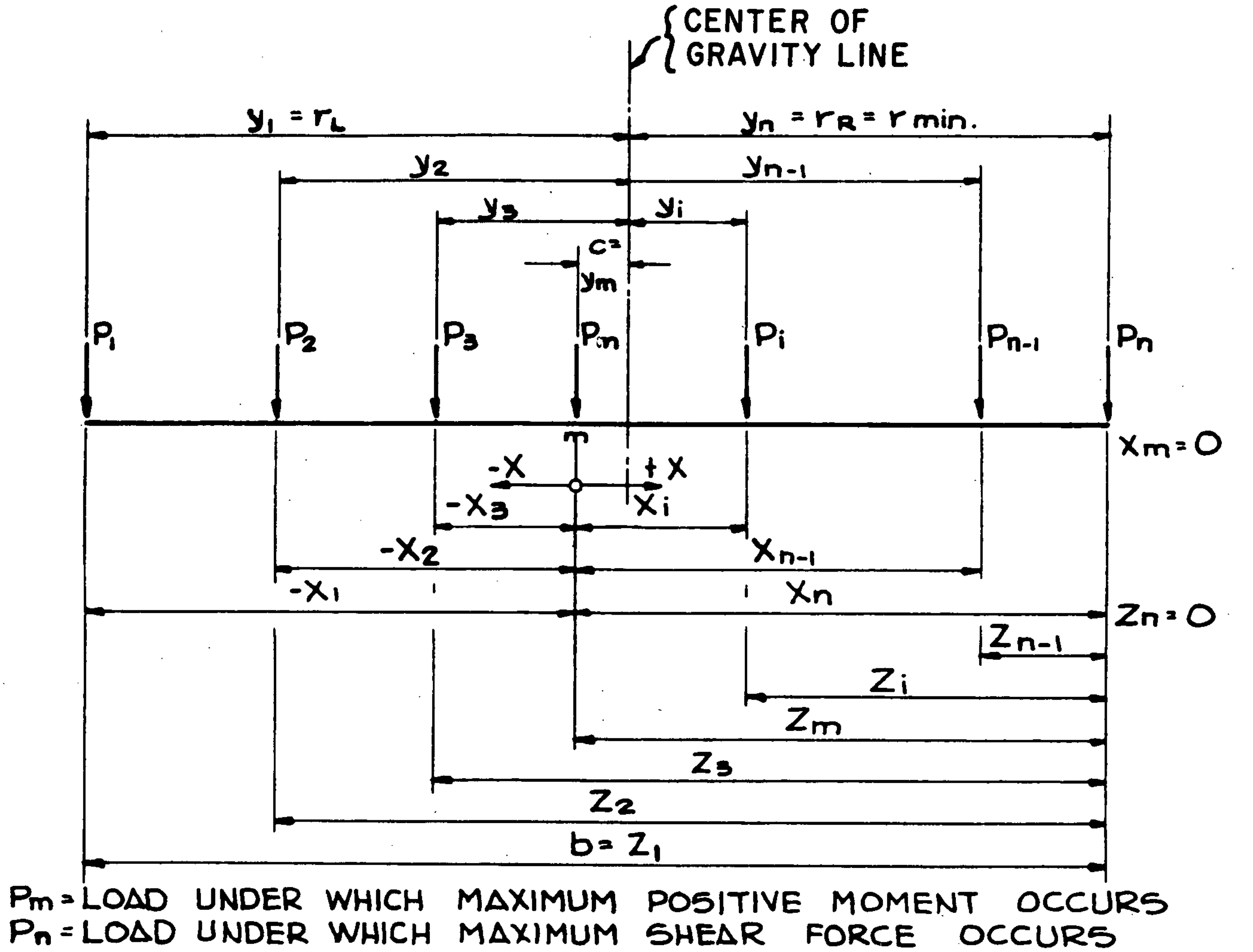


Figure B-1. Loads of general axle group.

$\alpha_v = \overline{V}_L / V_L =$ live load shear ratio of new load group against AASHO design load.

After several investigations, the load ratio for continuous bridges with end spans 75 percent of interior spans was found to be fairly close to the load ratio for the simple span case. Because this type is fairly typical, these ratios will be used for both simple span bridges and continuous bridges.

Determine Overstress Factor

Dead Load to Live Load Stress Ratio for AASHO Bridges

$$\beta_m = M_D / M_L$$
$$\beta_v = V_D / V_L$$

Typical dead load to live load ratios for various types of bridges are shown in Figures B-3 through B-11.

Overstress Factor Under New Load Limits

Overstress factor, Ω, is used here to denote the ratio of incremental stress caused by higher live load limits in excess of the original design live load to the original total design stress of a structural element.

Deck Slab.—For the same girder spacing, the moment due to wheel load on slab is linearly proportional to maximum wheel load. Because the stress is proportional to moment, flexural overstress factor for deck slabs can be represented as:

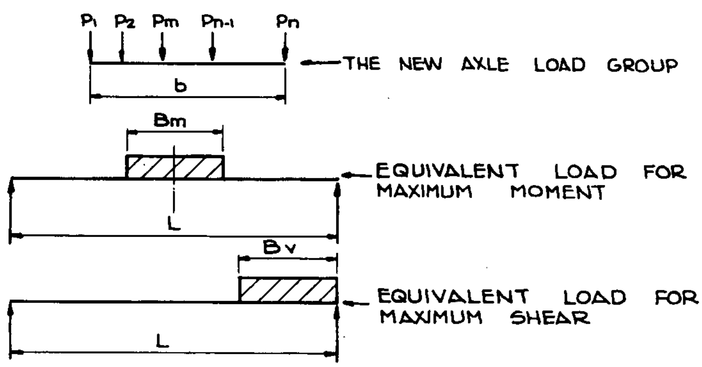


Figure B-2. Equivalent simple load patterns.

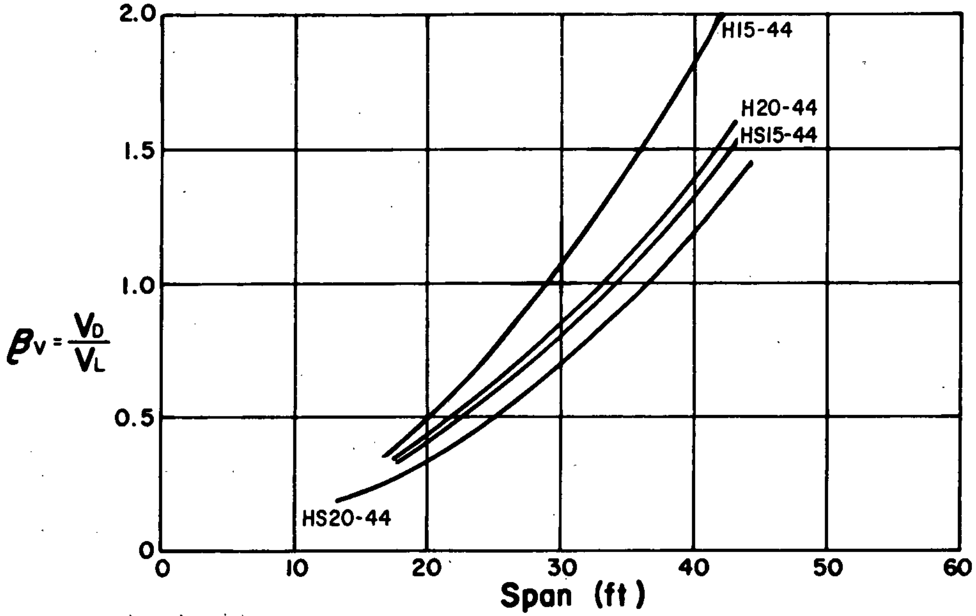


Figure B-3. Flat slab bridge, dead load to live load shear ratio vs span.

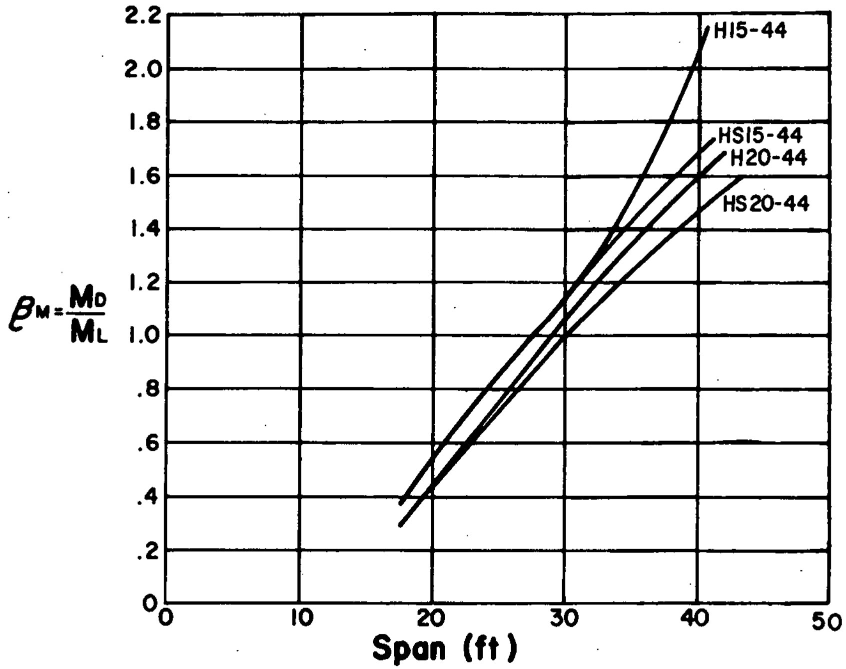


Figure B-4. Flat slab bridge, dead load to live load moment ratio vs span.

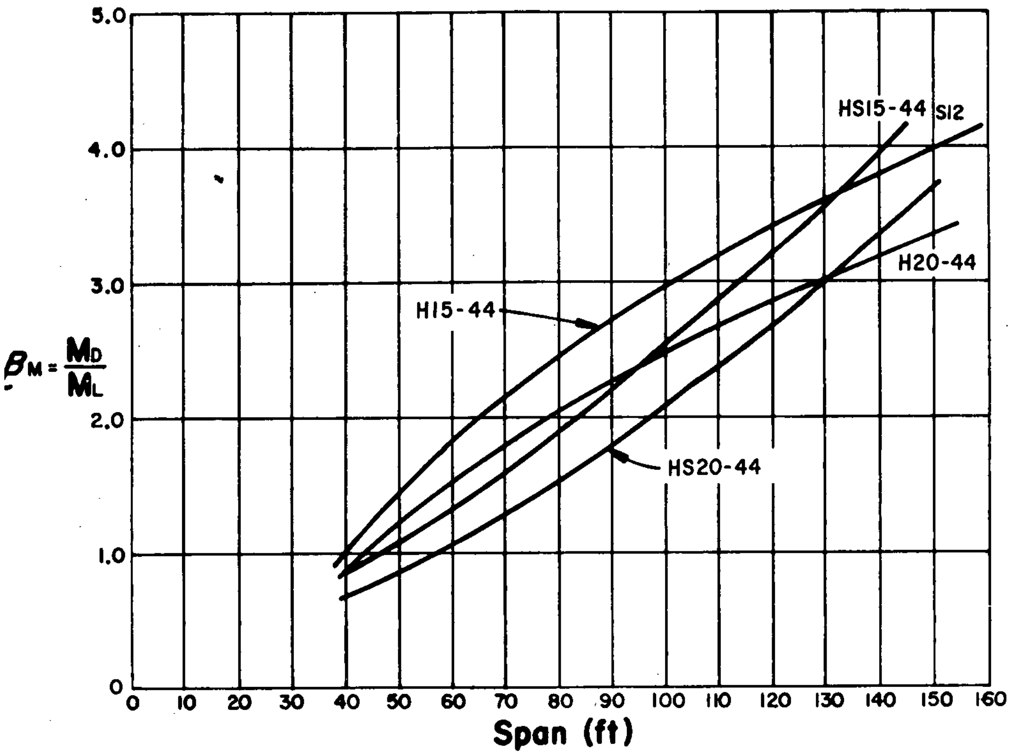


Figure B-5. Concrete box girder, dead load to live load moment ratio vs span.

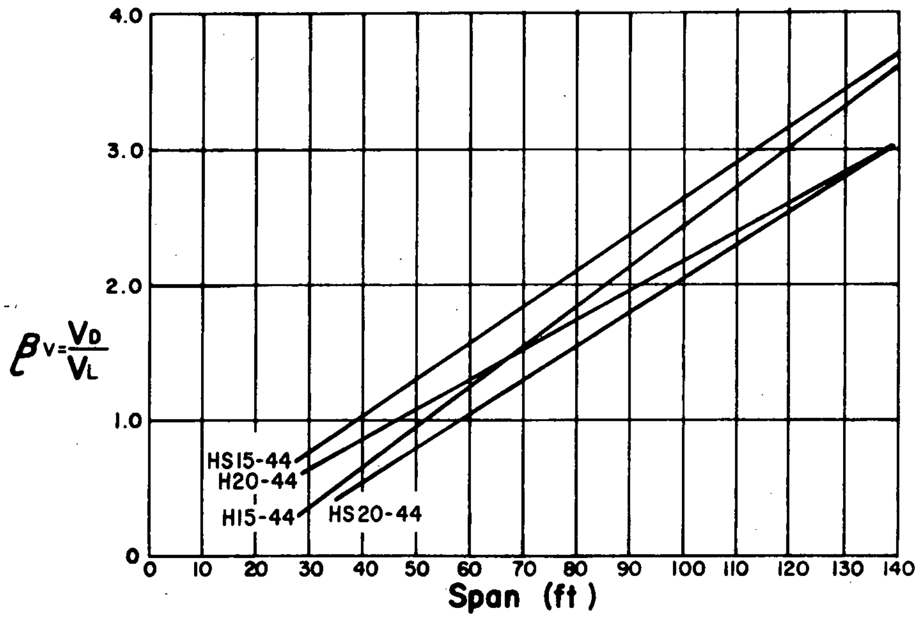


Figure B-6. Concrete box girder, dead load to live load shear ratio vs span.

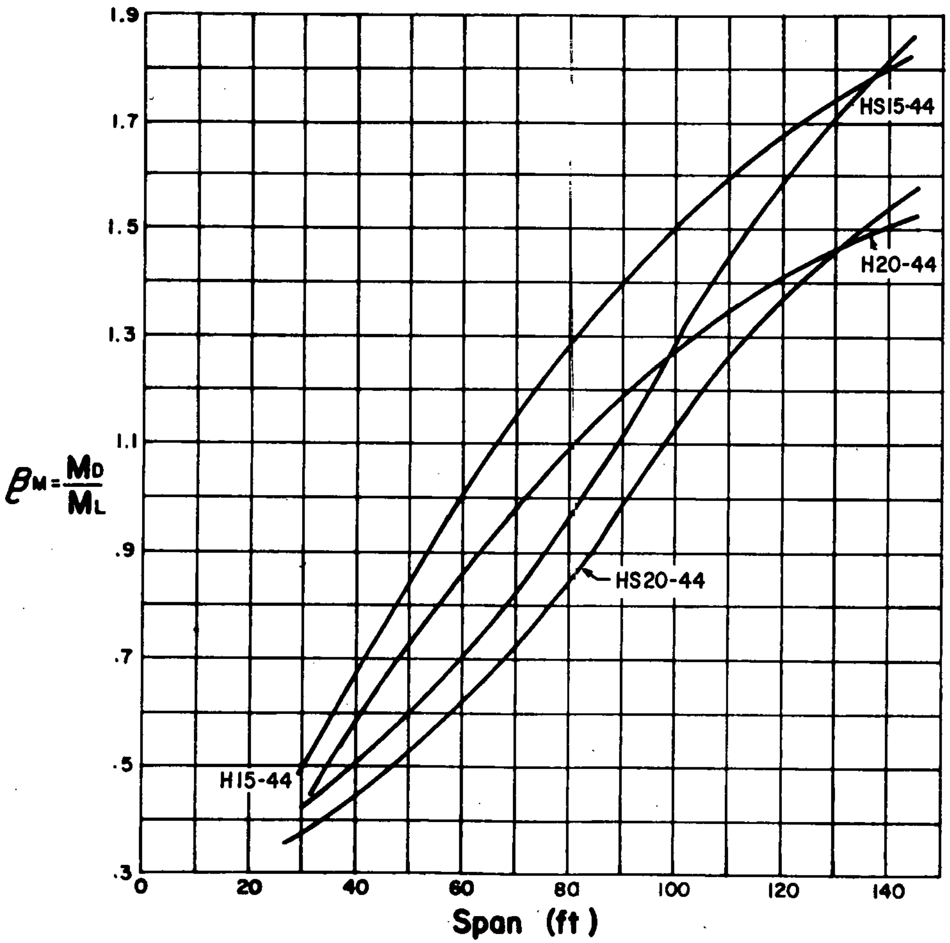


Figure B-7. Steel girder, dead load to live load moment ratio vs span.

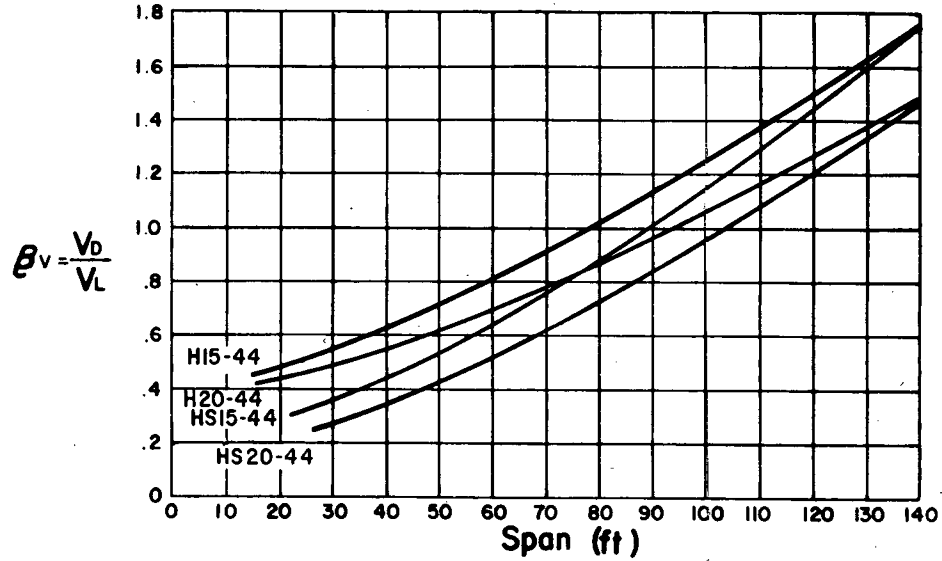


Figure B-8. Steel girder, dead load to live load shear ratio vs span.

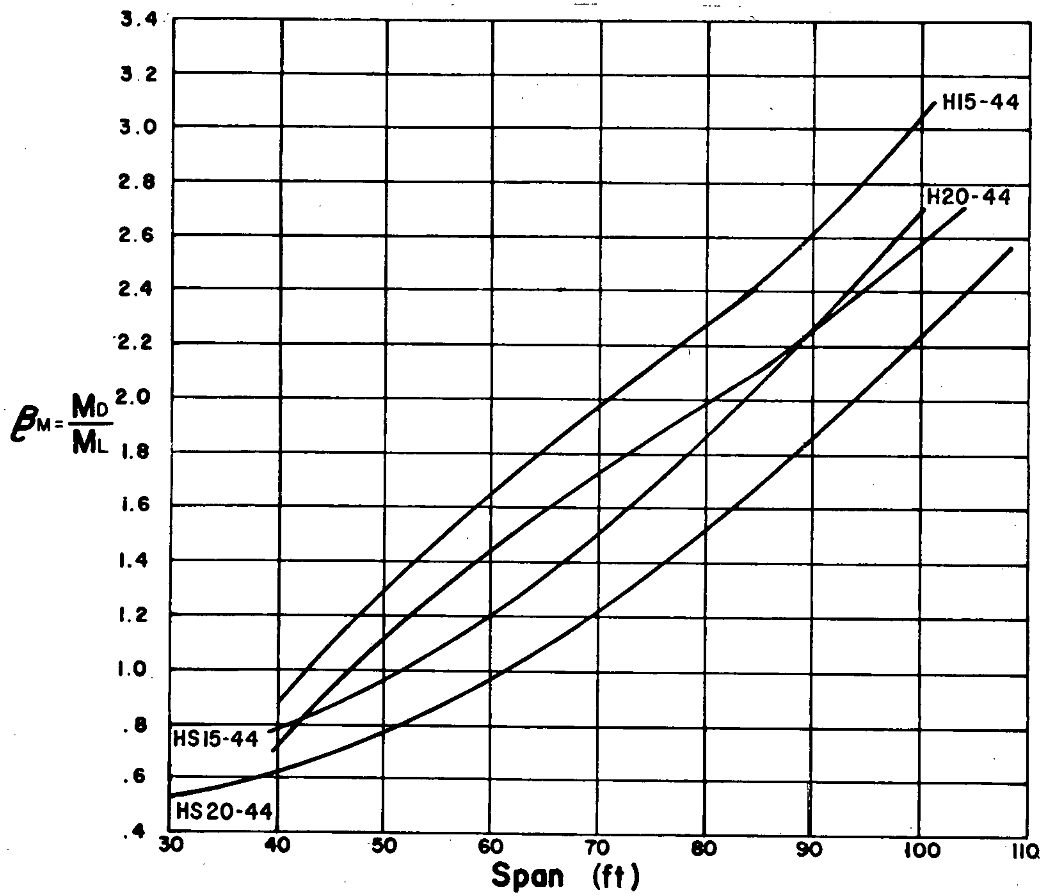


Figure B-9. Concrete T beam, dead load to live load moment ratio vs span.

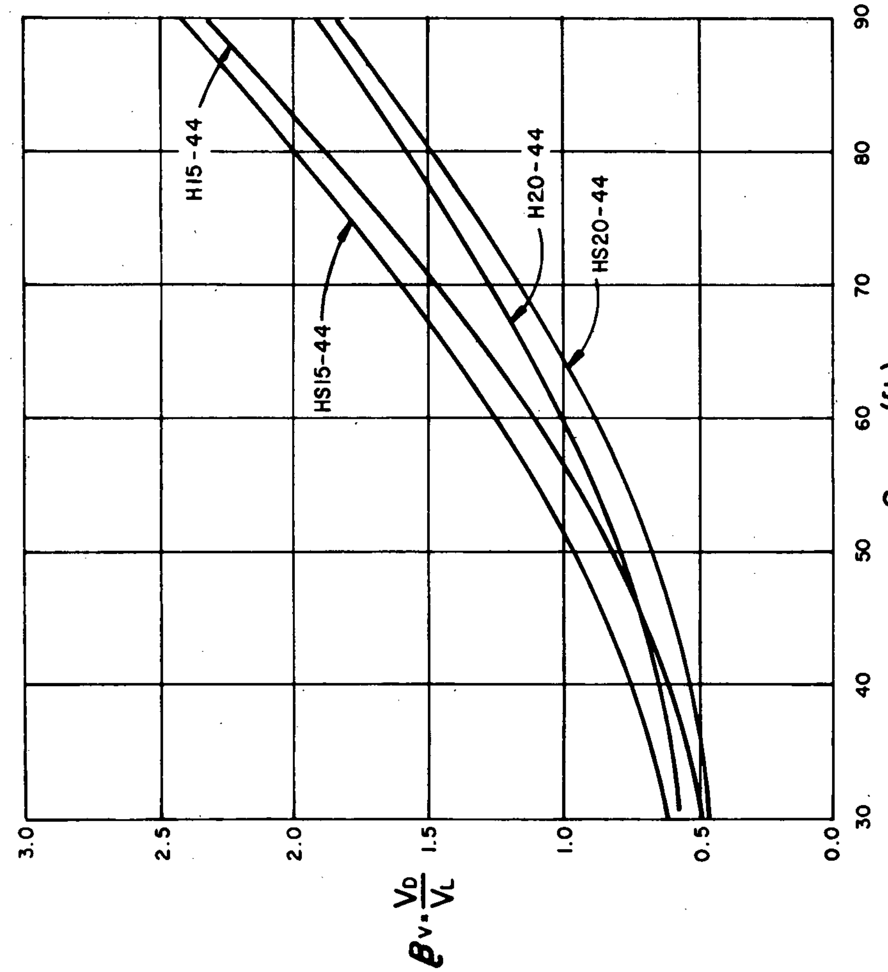


Figure B-11. Prestressed concrete girders, dead load to live load shear ratio vs span.

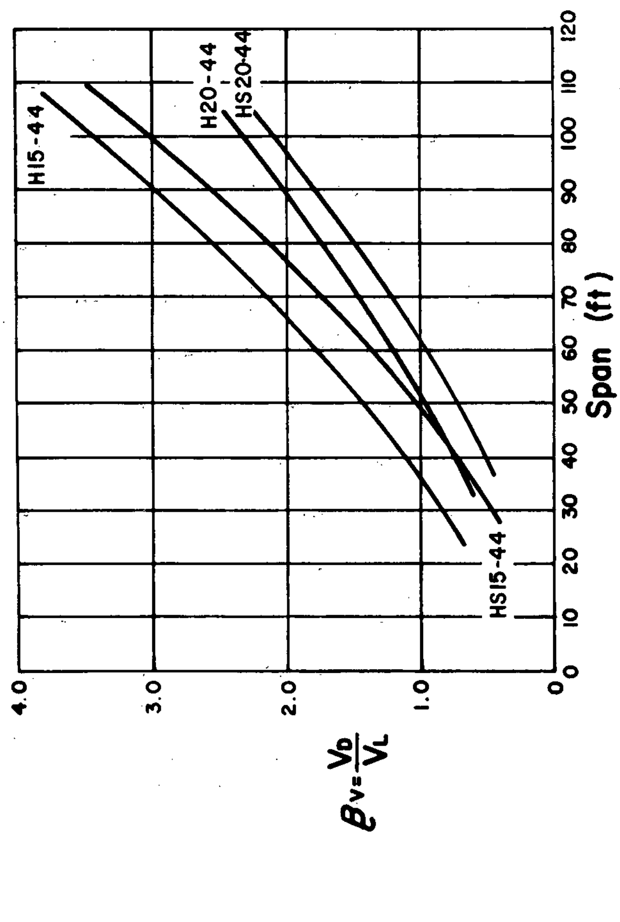


Figure B-10. Concrete T-beam, dead load to live load shear ratio vs span.

$$\Omega_m{}^S = \frac{\overline{M}_L - M_L}{M_T} = \frac{\overline{M}_L - M_L}{M_L + M_D} = \frac{(\overline{M}_L/M_L) - 1}{1 + (M_D/M_L)} = \frac{(\overline{P}/P) - 1}{1 + \beta_m}$$

Note: Shear in slab seldom governs the design and hence is not considered.

Girder Systems.—It is assumed that the original design stresses in flexure and in shear are the maximum allowable stresses set forth in AASHO specifications.

1. *Steel Girders with Noncomposite Design:*

Case I: Flexural overstress factor of steel beam. For the listed type of girders this may be found as:

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \beta_m}$$

Case II: Shear overstress factor of steel beam. For the listed type of girders this may be found as:

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + \beta_v}$$

2. *Steel Girders with Composite Design:*

Case I: Flexural overstress factor of steel beam. For the listed types of girders this may be found as:

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \theta_b \beta_m}$$

in which θ_b is the sectional modulus ratio for bottom fiber of composite section versus noncomposite section of the girder.

For average conditions, the value of θ_b may be assumed to equal 1.30, or may be refined for a particular family of spans.

Case II: Shear overstress factor of steel beam. This may be found in the same way as for the case of noncomposite design.

3. *Flat Slab—T Beam and Box Girders:*

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \beta_m}$$

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + \beta_v}$$

4. *Prestressed Concrete Girders of Composite Design:*

Case I. Flexural overstress factor. This usually is governed by the stress condition at the bottom fibers.

AASHO girders are designed generally to develop a stress of 0.4 $f_c' = 2{,}000$ psi due to dead and live load only, of which 1,200 psi is due to live load. This corresponds to:

$$M_T = M_D + M_L = 2{,}000S \text{ in.-lb}$$
$$= 0.167S \text{ in.-lb}$$

The overstress factor thus is:

$$\Omega_m{}^G = \frac{\overline{M}_L - M_L}{M_T} = \frac{M_L(\alpha_m - 1)}{0.167S}$$

Generally, M_L corresponds to

$$M_L \approx 0.24 f_c' \approx 1{,}200S = 0.10S$$

Therefore,

$$\Omega_m{}^G \approx 0.6(\alpha_m - 1)$$

Case II. Shear overstress factor.

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + 0.6\beta_v}$$

Piers.—The dead load to live load ratio for piers will be greater than that of girders because the former includes also the weight of the pier. Except possibly in the case of AASHO load Group III, the overstress factor in the pier under new proposed load limits for all loading combinations will be less than the overstress factor in girders. Even when AASHO Group III loading governs the design of the pier, the overstress factor in the pier under normal circumstances would be less than that in the girder for the same new load limits.

For all practical purposes, the overstress factor of the pier may be safely taken as the overstress factor of the bridge girder.

Foundations.—The overstress ratio in foundations will be assumed to be less than the one in piers because the dead load on foundations is more than the dead load on piers. For all practical purposes, it will also be taken as the overstress factor of the bridge girder.

Permissible Overload Criteria

Permissible Overstress Factors *

The actual load-carrying capacity of a bridge depends on the sectional properties and the stress that the material of the structural member can endure. If these allowable stresses are frequently exceeded, the accumulation of damage will significantly reduce its service life. The AASHO Road Test on bridges, the American Concrete Institute recommendations, and the DIN standards prevailing in Europe suggest stress limits that vary in some small magnitude. A conservative limitation may be concluded therefrom for purposes here.

1. *Reinforced concrete:*
Concrete: 70 percent of cylinder strength $\times \Phi$.
Re-bar: 75 percent of yield stress $\times \Phi$.
2. *Structural steel:* 75 percent of yield stress $\times \Phi$.
3. *Prestressed concrete:* 75 percent of modulus of rupture $\times \Phi$ or $5.5\Phi\sqrt{f_c'}$, in which Φ is taken to be 0.9 to account for the allowance of reasonable frequent overloading.

With the limiting stresses decided, the permissible overstress factors may be determined by comparing the limiting service stresses given here to the working stresses used in the original design criteria.

1. *Reinforced Concrete:* Design working stress of AASHO:
Concrete: 40 percent of cylinder strength.
Re-bar: 50 percent of yield stress.

The permissible overstress factor for moment, K_{cm}, may be calculated by:

$$K_{cm} = \text{min.}\left[\text{of either } \frac{\bar{f}_c}{f_c} \text{ or } \frac{\bar{f}_s}{f_s}\right] - 1$$
$$= \text{min.}\left[\text{either } \frac{0.7 \times 0.9 f_c'}{0.4 f_c'} \text{ or } \frac{0.9 \times 0.75 f_y}{0.5 f_y}\right] - 1$$

The permissible overstress factor for shear, K_{cv}, may be safely taken as 0.3.

2. *Structural Steel:* Because design working stress of structural steel given in the AASHO specifications is 0.55 f_y, the permissible overstress factor may be found as:

$$K_{sm} = \frac{0.75 f_y \times 0.9}{0.55 f_y} - 1 = 0.23$$

Permissible shear overstress factor, K_{sv} also may be taken as 0.23.

3. *Prestressed Concrete:* It is estimated that this bridge type can endure an overstress of 12 percent ($K_{pm} = 0.12$) with little or no relevant cracking of the tensile zone of the girders. The permissible overstress factor for shear, K_{pv}, may be taken to be the same as that for reinforced concrete; i.e., equal to 0.30.

* Suggested allowable overstress factors are shown in this report where necessary. These suggested allowable overstress values may be adjusted to conform with local practice or the judgment of the user.

Bridge Serviceability Under New Load Limits

In new or good condition, any bridge with indicated overstress factors less than the permissible overstress factors under new load limits is considered to be serviceable without any modification to the structure. Bridges not meeting this criterion are considered to require either immediate replacement or strengthening to serve the new load limits.

Strengthening Bridges to Serve New Load Limits

The problem of developing a precise criterion that governs the strengthening of bridges to serve new load limits involves many subjective factors that vary greatly, such as labor costs, material costs, and, above all, engineering practice. However, necessary assumptions are made to develop the methodology:

1. Strengthening of bridges with concrete superstructure is not considered feasible because of the difficulties involved in revising the concrete structure.
2. Strengthening of steel girder bridges with stress factors, given below for the major elements of the structure, is considered feasible.
   (a) Deck slab: $\Omega_m{}^S \leq 0.35$.
   (b) Steel girder: $0.23 < \Omega_m{}^G \leq 0.30$.
   (c) Piers and foundations: $\Omega_p = \Omega_f \leq 0.35$.
3. Therefore, the contribution to incremental cost due to new loads is essentially from steel girders. From the stress factors for girders, the material quantity ratio (ratio of material quantities required by new loads to those required by AASHO design loads)

$$\gamma_{sg} = \tfrac{1}{2}[(1 + \Omega_v{}^G) + (1 + \Omega_m{}^G)^{\frac{1}{2}}]$$

4. The cost of strengthening is then expressed as:

$$C_s = \gamma_{sg} Q_s U$$

in which

γ_{sg} = material quantity ratio for structural steel of girder;
Q_s = original structural steel quantity; and
U = unit cost of structural steel.

The estimate of costs for strengthening a bridge should be compared to the estimate of costs for replacement, and the lesser cost should be used in the final summation.

Cost of New Construction

For new construction, the precise cost evaluation in general terms is a complex problem involving subjective factors such as choice of type of bridge structure and arrangement of its structural elements. The costs of elements of construction vary from state to state, and from region to region within a state. Therefore, compromise is necessary to reduce the problem to manageable dimensions and to develop a method that could give meaningful, although approximate, results.

A useful relationship between material quantities of new construction and of the bridge found unserviceable with new loads can be established with reasonable assumptions that (1) the new construction will be of the same type and arrangement of structural elements as the original, and (2) the state has the information regarding construction quantities of the original bridge. After this relationship is established, the cost of new construction is obtained in terms of increased quantities required and the present cost of construction in the respective regions.

Material Quantity Ratio

The material quantity ratio is defined as the ratio of material quantities required by the new load limits to those required by AASHO design loads. The ratio is a comparative measure of construction costs between the two loading systems. To relate the material quantity ratio to the overstress factors is a complex matter. A semi-rational approach is adopted here in which certain assumptions are made regarding each of the four fundamental structural elements of a bridge.

1. *Deck:*

Case I—Slab: The material quantity ratio for the deck slab may be expressed in terms of wheel loads:

$$\gamma_s = (\overline{P}/P)^{\frac{1}{2}} \quad \text{but not less than 1}$$

Case II—Flat slabs:

$$\gamma_s = (1 + \Omega_m{}^S)^{\frac{1}{2}}$$

2. *Girders:*

Case I—T beams and box girders:

$$\gamma_g = \tfrac{1}{2}[(1 + \Omega_v{}^G) + (1 + \Omega_m{}^G)^{\frac{1}{2}}]$$

Case II—Steel girders: A realistic estimate of the material quantity ratio may be expressed as:

$$\gamma_{sg} = \tfrac{1}{2}[(1 + \Omega_v{}^G) + (1 + \Omega_m{}^G)^{\frac{1}{2}}]$$

Case III—Prestressed concrete girders: There are four standard types of AASHO prestressed concrete girders, with service spans overlapping between any two consecutive types. For purposes of this study, upgrading the capacity of a prestressed concrete girder is not considered practical. In practical terms, the overstress can be relieved either by increasing the number of girders or by changing the girder type. Strengthening prestressed concrete bridges is considered to be impractical—replacement is necessary if overstress ratio is more than permissible. An approximate expression for material quantity is:

$$\gamma_{sg} = \tfrac{1}{2}[1 + (1 + \Omega_m{}^G)^{\frac{1}{2}}]$$

3. *Piers:* The material quantity ratio for piers is conservatively expressed in shearing overstress factor as:

$$\gamma_p = 1 + \Omega_v{}^G$$

4. *Foundations:* The conservative expression for piers applies also to foundations, as:

$$\gamma_f = 1 + \Omega_v{}^G$$

Summary

The cost of new construction is a function of original material quantity ratios of the major structural elements and the unit material cost at the time new construction is to be implemented. A general expression for the cost may be:

$$C = C_m + \Sigma\chi Q\gamma$$

in which

C_m = cost of miscellaneous items (rails, parapet) not affected by new load limits;
U = unit material cost of a structural element;
Q = original quantity of a structural element;
γ = material quantity ratio of a structural element; and
χ = inflation factor of material cost.

Incremental cost due to new loads may be expressed as:

$$\Delta C = \Sigma(\gamma - 1)QU$$

The summation covers all the structural elements of a bridge—namely; deck, girder, pier, and foundation.

If material quantity data for structural elements cannot be obtained from past records, a lump-sum cost estimate will have to suffice. In this case the cost estimate is based on:

$$C = \chi_a(C_m + \gamma_a U_a A)$$

in which

χ_a = average inflation factor of materials;
U_a = original cost of construction, $/sq ft;
A = plan area of bridge, sq ft; and
γ_a = average material quantity ratio of all structural elements.

Fatigue Life of Bridges

The fatigue life of a structure is affected by a combination of many factors, the most important of which are load spectrum, nature and condition of structural member, and environment of the structure.

A relatively precise estimation of fatigue life of a bridge requires: collecting the design data of the bridge; obtaining the loading and stress range history through field observation and testing; projecting past and present traffic data to obtain future expected traffic volume; and making theoretical analysis.

Several studies (*128, 129, 130*) reveal that under current traffic conditions the bridges examined have fatigue lives well above 1,000 years. The heaviest traffic in these studies had an annual commercial traffic volume of 860,000. Assuming commercial traffic is 40 percent of total traffic, total traffic volume amounts to 2,150,000 vehicles per year, or an ADT of approximately 6,000 vehicles. Because of the difference in the load spectrum of the bridges, fatigue life has no direct relationship to truck volume alone. Bridges that have less traffic volume but with high damage range would have shorter lives than the ones that have more traffic volume with low damage range.

In the cited studies, the bridges investigated include:

1. Simple span, rolled beam with welded cover plate.
2. Suspended center span, rolled beam with welded cover plate.
3. Simple span, prestressed concrete I beam.
4. Welded plate girder with flange thickness transition.
5. Continuous spans, wide-flange beam with cover plate.

Each bridge was designed for AASHO HS20-44 loading.

Assuming that the relationship between the logarithmic value of the fatigue cycles and the stress range is linear and the stress range value at 2×10^8 cycles for zero-to-tension stresses is equal to one-third the value at 2×10^6 cycles, an overload at a stress level of 30 percent will reduce the fatigue life of a bridge to about one-fifth of its original value. Thus, if the original fatigue life of a bridge under current traffic conditions is 1,000 years, an overload of 30 percent at all vehicle load levels with the same traffic volume will reduce the expected life to 200 years.

It may be concluded that bridges with a normal distribution of trucks loaded to new permissible load limits (say, 10 percent of the truck traffic or 4 percent of the total traffic, and with an ADT under 20,000) should have a fatigue life of 50 years or more.

The fatigue life of a bridge with an ADT exceeding 20,000 and yet classified as serviceable without modification under new load limits may be estimated by the following, regardless of the original design load:

$$\text{Fatigue life} = F = (20{,}000/\text{ADT})\,50\text{ yr}$$

Assuming that the expected life span of the bridge is 50 years from the date of construction, and the cost of construction is spread uniformly over the expected life span of 50 years, the cost apportioned to the shortened life due to accommodating new loads can be approximated by

$$C_f = (50 - F - T)C_o/50$$

in which

F = fatigue life of the bridge;
T = time elapsed between date of construction and date new legal load limits are implemented;
C_o = original cost of the bridge; and
C_f = cost attributable to new load limits.

NUMERICAL EXAMPLE OF METHOD APPLICATION

Numerical examples are given to illustrate the application of the assembled methods to arrive at a rational cost analysis of bridge structures for arbitrarily chosen new vehicle load limits.

Two new loads are used throughout the examples. The Type I new load, although improbable, serves to demonstrate the method's flexibility.

Bridges are chosen with different types of construction such that they will yield different conclusions as to whether the bridge:

1. May be considered serviceable for the new loads.
2. May be considered to need strengthening to serve the new loads.
3. Needs new construction to serve the new loads.

For the last two cases, computations of costs also are shown.

Although all the bridges given in the examples are assumed to have been originally designed for standard AASHO loadings, this condition is not necessary for the method to apply. Should a highway department have bridges designed with load standards different from AASHO loads, the method is still applicable. In this case the overstress factors are calculated using the original design load as an input parameter, in lieu of the AASHO design load.

In Example II, fatigue life is estimated by using the formula given. The current traffic conditions are assumed for use of the formula. If the user has developed a better method than the one illustrated, that method may be used instead.

Example I

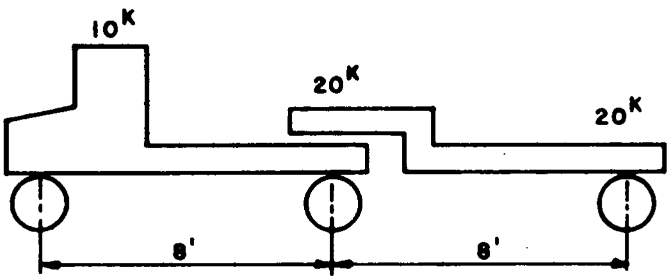
1. *Bridge type:* Simple-span WF composite girder with concrete slab, bents, and spread footings. Girder span length = 90 ft.

2. *Load data:* Bridge designed for HS20-44 loading, as per AASHO specification. Present ADT = 2,000.

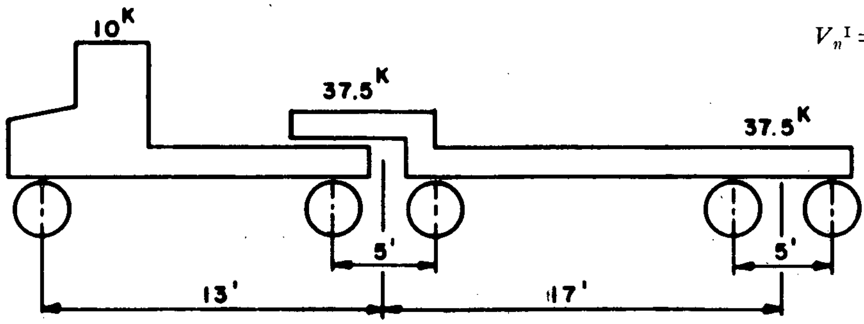
3. *Present condition:* Good.

4. *Assumed new loads:*

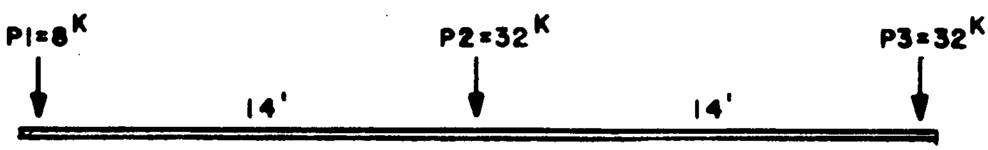
Type I new load (this improbable truck configuration is selected to demonstrate the method's flexibility):



Type II new load:



Step 1: Determine load parameters for original design load: Under HS20-44 AASHO loading; with 90-ft span, truck load controls:



$$\xi = \frac{2\sum_{i=1}^{3} P_i|x_i|}{Wb} = 0.555$$

$$\mu = \frac{\sum_{i=1}^{3} P_i x_i}{Wb} = 0.167$$

$$\delta = \frac{\sum_{i=1}^{3} P_i z_i}{Wb} = 0.333$$

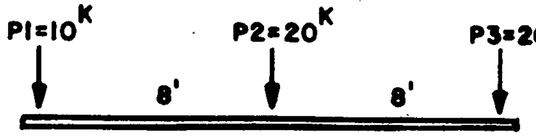
$$B_m = 2[\xi - (b/L)\mu^2]b = 30.62 \text{ ft}$$

$$B_v = 2\delta b = 18.67 \text{ ft}$$

$$M_m = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = 1{,}342 \text{ ft-kips}$$

$$V_n = W\left(1 - \frac{B_v}{2L}\right) = 64.5 \text{ kips}$$

Step 2: Determine load parameters for Type I new load:



TYPE I NEW LOAD

$$\xi = \frac{2\sum_{i=1}^{3} P_i|x_i|}{Wb} = 0.6$$

$$\mu = \frac{\sum_{i=1}^{3} P_i x_i}{Wb} = 0.1$$

$$\delta = \frac{\sum_{i=1}^{3} P_i z_i}{Wb} = 0.4$$

$$B_m = 2[\xi - (b/L)\mu^2]b \approx 18.6 \text{ ft}$$

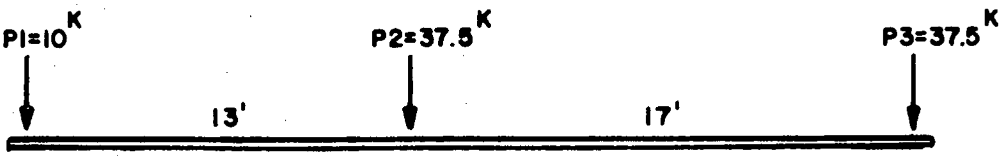
$$B_v = 2\delta b = 12.8 \text{ ft}$$

$$M_m{}^{\mathrm{I}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = 1{,}010 \text{ ft-kips}$$

$$V_n{}^{\mathrm{I}} = W\left(1 - \frac{B_v}{2L}\right) = 46.5 \text{ kips}$$

Because $M_m^{\text{I}} < M_m$ and $V_n^{\text{I}} < V_n$, it is concluded that the Type I new load is less severe than the original designed load HS20-44. Therefore, the bridge is safer under Type I new load.

Step 3: Determine load parameters for Type II new load:



TYPE II NEW LOAD

$$\xi = \frac{2\sum_{i=1}^{3} P_i|x_i|}{Wb} = 0.602$$

$$\mu = \frac{\sum_{i=1}^{3} P_i x_i}{Wb} = 0.199$$

$$\delta = \frac{\sum_{i=1}^{3} P_i z_i}{Wb} = 0.368$$

$$B_m = 2[\xi - (b/L)\mu^2]b = 35.4 \text{ ft}$$

$$B_v = 2\delta b = 22.1 \text{ ft}$$

$$M_m^{\text{II}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = 1{,}530 \text{ ft-kips}$$

$$V_n^{\text{II}} = W\left(1 - \frac{B_v}{2L}\right) = 74.5 \text{ kips}$$

Because $M_m^{\text{II}} > M_m$ and $V_n^{\text{II}} > V_n$, the bridge designed for HS20-44 will be overstressed under Type II new load.

Step 4: Determine live load ratios:

Live load moment ratio of Type II new load to HS20-44 loading:

$$\alpha_m = M_m^{\text{II}}/M_m = 1.14$$

Live load shear ratio of Type II new load to HS20-44 loading:

$$\alpha_v = V_n^{\text{II}}/V_n = 1.15$$

Step 5: Determine overstress factors:

1. *Girder:*

β_m = ratio of dead load moment to live load moment (including impact) for HS20-44 loading for a 90-ft-span steel composite bridge girder = 0.98 (from Fig. B-7).

θ_b = ratio of sectional modulus of composite section to noncomposite section at bottom flange = 1.30.

The flexural overstress factor may be found as

$$\Omega_m^G = \frac{\alpha_m - 1}{1 + \beta_m\theta_b} = \frac{1.14 - 1}{1 + 1.3(0.98)} = 0.0616$$

Similarly, the shearing overstress factor may be found as

$$\Omega_v^G = \frac{\alpha_v - 1}{1 + \beta_v}$$

but is not critical because the shear stress in WF beam generally is well below the maximum allowable stress.

2. *Deck Slab:* Because the single wheel load in either type of new loading under consideration is less than the 16-kip design load, the slab will not be subjected to overstress, and hence is safe.

3. *Piers and Foundations:* Overstress in piers and foundations generally is less severe than the shearing stress in superstructure and can be taken conservatively as:

$$\Omega_p = \Omega_f < \Omega_v^G = \frac{\alpha_v - 1}{1 + \beta_v} = \frac{0.15}{1 + 0.84} = 0.082$$

Step 6: Determine serviceability of bridge:

1. *Deck Slab:* Because it is not overstressed as a result of new proposed loading, the slab certainly is serviceable under new loads.

2. *Girder:* The overstress factor for girder in flexure is less than the permissible overstress factor:

$$\Omega_m^G = 0.0616 < K_{sm} = 0.23$$

Therefore, the girder also is serviceable under new loads.

3. *Piers and Foundations:* The overstress for piers and foundations is less than the overstress factor for girder: Use Ω_p or $\Omega_f < 0.082$, which is $< K_{cm} = 0.35$; therefore, the bridge is serviceable.

Step 7: Fatigue life estimation: Because the ADT of the bridge is fairly low, fatigue life need not be examined.

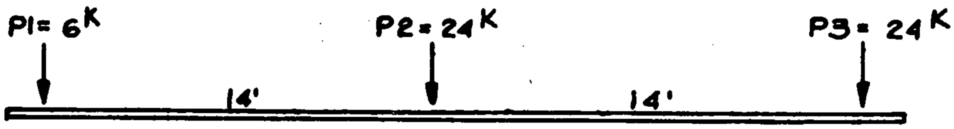
Conclusion: The bridge under consideration for the assumed two types of new loads is found to be safe and serviceable.

Example II

Consider a bridge that has the same features as the bridge in Example I, except for the following:

1. Bridge span is 60 ft instead of 90 ft.
2. Noncomposite between girder and slab.
3. Bridge was designed for HS15-44 loading and has been in service for 10 years. Present ADT = 40,000, of which 40 percent is commercial traffic and 5 percent is multi-axle trucks with gross weight between 80 and 130 percent of original design load.

Step 1: Determine load parameters for HS15-44 loading:



$$\xi = 0.4;\ \delta = 0.2;\ \mu = 0.2$$

$$B_m \approx 30.6 \text{ ft}$$

$$B_v \approx 18.7 \text{ ft}$$

$$M_m = \frac{(54)(60)}{4}\left(1 - \frac{30.6}{120}\right) = 610 \text{ ft-kips}$$

$$V_n = 54\left(1 - \frac{18.7}{120}\right) = 45.5 \text{ kips}$$

Step 2: Determine load parameters for Type I loading:

$$\xi = 0.6;\ \mu = 0.1;\ \delta = 0.4$$

$$B_m = 19 \text{ ft};\ B_v = 12.8 \text{ ft}$$

$$\mathrm{M}_m{}^{\mathrm{I}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = 630 \text{ ft-kips}$$

$$V_n{}^{\mathrm{I}} = W\left(1 - \frac{B_v}{2L}\right) = 45.3 \text{ kips}$$

Step 3: Determine load parameters for Type II loading:

$$\xi = 0.602;\ \mu = 0.2;\ \delta = 0.368$$

$$B_m = 34.3 \text{ ft};\ \mathrm{B}_v = 22.1 \text{ ft}$$

$$M_m{}^{\mathrm{II}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = 486 \text{ ft-kips}$$

$$V_n{}^{\mathrm{II}} = W\left(1 - \frac{B_v}{2L}\right) = 69.5 \text{ kips}$$

Step 4: Determine live load ratio:

$$\alpha_m{}^{\mathrm{I}} = \frac{M_m{}^{\mathrm{I}}}{M_m} = 1.04;\ \alpha_v{}^{\mathrm{I}} = \frac{V_n{}^{\mathrm{I}}}{V_n} = 1.0$$

$$\alpha_m{}^{\mathrm{II}} = \frac{M_m{}^{\mathrm{II}}}{M_m} = 1.44;\ \alpha_v{}^{\mathrm{II}} = \frac{V_n{}^{\mathrm{II}}}{V_n} = 1.53$$

Step 5: Determine overstress factors:

1. *Girder:* Overstress factor for noncomposite steel girder in flexure is given by:

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \beta_m} = \frac{1.44 - 1.0}{1 + 0.7} = 0.26$$

Overstress factor in shear is given by:

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + \beta_v} = \frac{1.53 - 1}{1 + 0.65} = 0.32$$

in which

β_m = ratio of dead to live load moment (including impact) for 60-ft-span noncomposite WF girder (from Fig. B-7 ≅0.70); and

β_v = ratio of dead to live load shear (including impact) for 60-ft-span noncomposite WF girder (from Fig. B-8) ≅0.65.

2. *Deck Slab:* Because the single wheel load in either loading is less than 16 kips, for which the slab is designed, the overstress factor need not be considered.

3. *Piers and Foundations:*

$$\Omega_p = \Omega_f < \Omega^G = 0.32$$

Step 6: Determine serviceability of bridge: Because the flexural overstress factor computed is between 0.23 and 0.30 and the shearing overstress factor is slightly greater than 0.30, it can be concluded that strengthening the bridge is feasible.

Step 7: Predict fatigue life of bridge: Because the ADT exceeds 20,000 and the traffic has a normal mix of heavy trucks, fatigue life may be estimated by:

$$\text{Fatigue life} = F = (20{,}000/\text{ADT})\,50 \text{ yr}$$
$$= (20{,}000/40{,}000)\,50 = 25 \text{ yr}$$

The bridge thus will have 15 years or more of life left; it is considered feasible to strengthen the bridge to serve the new load limits.

Step 8: Compute material quantity ratio for steel girder:

$$\gamma_{sg} = ½[(1 + \Omega_v{}^G) + (1 + \Omega_m{}^G)^{\frac{1}{2}}]$$
$$= ½[1.32 + (1.26)^{\frac{1}{2}}] = 1.22$$

Step 9: Compute cost of strengthening:

$$C_s = (\gamma_{sg} - 1)QU = 0.22QU$$

Therefore, 22 percent of the original cost of the bridge is chargeable to the legal limit change.

Example III

Substitute composite prestressed concrete girder for steel girder; assume same design data as in Example II.

Step 1: Determine load parameters for H20 loading (same as Example II):

$$M_m = 610 \text{ ft-kips}$$

$$V_n = 45.5 \text{ kips}$$

Step 2: Determine load parameters for Type I loading (same as Example II):

$$M_m{}^{\mathrm{I}} = 630 \text{ ft-kips}$$

$$V_n{}^{\mathrm{I}} = 45.3 \text{ kips}$$

Step 3: Determine load parameters for Type II loading (same as Example II):

$$M_m{}^{\mathrm{II}} = 905 \text{ ft-kips}$$

$$V_n{}^{\mathrm{II}} = 69.5 \text{ kips}$$

Step 4: Determine live load ratios (same as Example II):

$$\alpha_m{}^I = 1.03; \alpha_v{}^I = 1.00$$

$$\alpha_m{}^{II} = 1.48; \alpha_v{}^{II} = 1.53$$

Step 5: Determine overstress factors (same as Example II):

For 60-ft span:

1. *Girder:*

$$\beta_v = 1.30 \text{ (from Fig. B-11)}$$

Thus, the overstress factors may be found as:

$$\Omega_m{}^G = 0.60(1.03 - 1) = 0.018;$$

$$\Omega_m{}^{GII} = 0.6(1.48 - 1) = 0.268$$

$$\Omega_v{}^{II} = 0.41; \Omega_v{}^I = 0.081$$

Therefore: Type II load governs.

2. *Deck Slab:* As in Example II, slab is safe.

3. *Piers and Foundations:* As in Example II, the overstress factor may be safely assumed as that of the prestressed girder.

Step 6: Determine serviceability of bridge: Because the permissible overstress factor in flexure, $K_{pm} = 0.12$, is less than the actual flexural overstress, $\Omega_m{}^{II} = 0.268$, and the permissible overstress factor in shear, $K_{pv} = 0.20$, is less than the actual shearing overstress, $\Omega_m{}^{II} = 0.41$, it is concluded that the bridge needs new construction to serve the new loads.

Step 7: Determine cost of new construction:

Let χ = cost index in material and labor based on original construction cost = 1.2.

For girder:

$$\gamma_{cg} = \tfrac{1}{2}[1 + (1 + 0.268)^{\frac{1}{2}}] = 1.065$$

$$C^G = \chi\gamma_{cg}C_o{}^G = 1.28C_o{}^G$$

For slab:

$$C^S = \chi C_o{}^S = 1.5C_o{}^S$$

For substructure:

$$\gamma_p = 1.065$$

$$C^{SS} = \chi\gamma_p C_o{}^{SS} = 1.2 \times 1.065C_o{}^{SS}$$

$$= 1.28C_o{}^{SS}$$

Therefore, the total cost of new construction = 1.5 × original cost of slab + 1.28 × original cost of girder + 1.28 × original cost of pier and foundation.

These three examples demonstrate the sensitivity of the proposed method.

The serviceability of an existing bridge under new legal load limits is affected by many factors (e.g., configuration and axle arrangement of the new loads; original design load of the bridge; span, type, and composition of construction; bridge's present condition; and traffic spectrum). The examples are designed to reflect explicitly or implicitly all these factors, and hence their impact on bridge costs.

Total Structural Cost Impact for the System

To determine the total cost impact of a proposed change in legal limits on structures, the inventory of highway structures within the system must be divided into types, span ranges, and original design load classes, etc. The foregoing cost analysis procedures then would be applied to each structure in these groups. The costs derived from each of these categories are then summed to obtain the total system cost related to highway structures.

The procedure outlined herein is intended to illustrate a technique that can be applied with reasonable accuracy. Additional refinements and expanded applicability can be implemented to suit particular needs.

APPENDIX C

NUMERICAL EXAMPLE—PROCEDURES FOR DETERMINING EFFECTS OF CHANGES ON PAVEMENTS

The methods of analysis illustrated in this example are discussed in Chapter Two. All numerical values given here are primarily for illustration. They are not necessarily the typical values for a specific analysis.

EFFECT OF CHANGES ON REMAINING SERVICE LIFE OF EXISTING PAVEMENTS

Two methods for estimating the remaining service life of existing pavements are illustrated. The first method is applicable to the group of highways of a specific category in a jurisdiction or cost analysis area. The second method is suitable for an individual pavement section.

The first method was developed for highway needs studies; consequently, its use provides very approximate estimates for transportation planning purposes. The second method is preferable if more precise analyses are desired. For each method, the procedures for determining the remaining service life of flexible as well as rigid pavements are described.

Method I—Flexible Pavements

For a given cost analysis area, such as a county, the following information is assumed to be available from the inventory in highway planning studies:

- Flexible pavements with "medium" pavement structures (SN = 3.1 to 4.5).
- The soil support values of the subgrade soils are substantially greater than that of the AASHO Road Test subgrade soil. In the AASHO Road Test, $S = 3$. In this example, $S = 6$ or more.
- Present pavement condition is rated as "fair" (PSR or PSI = 2.1 to 3.0).
- Minimum tolerable pavement condition is represented by PSR = 2.1.
- Number of present equivalent annual 18-kip single-axle applications (EALA) = 189,216 * if there is no change in legal limits.
- Annual traffic growth rate = 4 percent.

The procedure for estimating the remaining service life of the group of flexible pavements is:

1. According to the data in Table 33, it is necessary to increase the medium pavement structure (SN = 3.1 to 4.5) with an S value of 6 or more to a heavy pavement structure (SN = 4.6 to 6.0).
2. From Table 32, the remaining service life is found to be 11 to 15 years in the general section identified as "Heavy Pavement Structure" and under the column "Annual Traffic Growth—4 to 6 Percent" and "Pavement Condition—Fair." The remaining service life given here is based on the assumption of no change in legal limits.
3. To estimate the remaining service life of the pavements, provided there would be changes in legal limits, it is necessary to determine the effect of the changes on the equivalent 18-kip single-axle load applications during the remaining service life of the pavements. In this example, assume that all data given here remain unchanged, except that the equivalent 18-kip single-axle load applications during the remaining service life of the pavements would be increased by 65 percent due to increases in legal limits. To make it possible to use Table 32 for the desired analysis, this 65 percent increase may be applied to the EALA value. In this example, EALA = 189,216(1 + 0.65) = 312,206.

With the increased value of EALA, Table 32 shows that the remaining service life of the pavements will be reduced to the range of 6 to 10 years. If the influence on the equivalent 18-kip single-axle load applications due to changes in legal limits would become effective gradually in a number of years, the specific effect (65 percent increase in this example) is necessarily dependent on the years of remaining service life of the pavements. In this case, a trial-and-error procedure is required to determine the increase in EALA during the remaining service life.

* The procedure for determining the EALA value is explained in a numerical example in "Manual B" (*36*, p. H-10). The EALA and other values given in the above example are the same as those in the example in "Manual B." Another method is illustrated in the numerical example of Appendix D.

Use of this procedure provides a very approximate estimate of the effect of changes on the remaining service life of existing pavements. For instance, if the increase is 50 percent instead of 65 percent, then EALA = 189,216 (1 + 0.50) = 283,824.

With this value of EALA, Table 32 shows that the remaining service life will be 11 to 15 years, instead of 6 to 10 years. Obviously, this substantial difference is due to the fact that in one case the estimated remaining service life is near the lowest value in the 11 to 15 years range, whereas in the other case it is very close to the highest value in the 6 to 10 years range. When a large number of cost analysis areas or counties are analyzed in a statewide study, these errors tend to compensate each other somewhat. One way to minimize this type of error is to use the method of interpolation for determining a specific value of the remaining service life within the range obtained from Table 32. The method for estimating remaining service life is approximate, using Table 32. If a more precise approximation is desired, it is preferable to use Method II (described later) to determine the effects of limit changes on the remaining service life of existing pavements.

Method I—Rigid Pavements

A similar procedure is followed in estimating the remaining service life of rigid pavements, except that no soil support value data are needed. In this example, assume that all data concerning pavement conditions, EALA, and traffic growth rate are the same as those for flexible pavements. Assume also that the highways have heavy pavement structures ($D = 9.1$ to 11.0 in.). From Table 31, the remaining service life will be in the range of 11 to 15 years, provided there is no change in legal limits. If the equivalent 18-kip single-axle load applications were increased by either 50 or 65 percent as a result of the changes in legal limits, the expected remaining service life, as indicated by the information in Table 31, would be in the range of 6 to 10 years.

Similar to the evaluation of flexible pavements, the use of Method II, which follows, provides a more precise analysis of the effect of changes on the remaining service life of rigid pavements.

Method II—Flexible Pavements

In the method reported by Corvi and Bullard (*75*) a series of charts is available for the analysis of flexible pavements with various present serviceability indices (PSI). This method is applicable for determining the remaining service life of individual pavement sections. In the evaluation of the effect of changes on the remaining service life of all existing pavements in a state, it is necessary to repeat the same procedure for each pavement section having the same thickness, characteristics, and traffic. To minimize the work required in this type of study, consideration may be given to the alternative of conducting the analysis only for the typical pavement sections in a state, instead of for all existing pavements.

The following procedure of analysis refers to a particular section of an existing highway, such as a two-lane highway one mile in length.

Given

- Accumulated number of equivalent 18-kip single-axle load applications from time pavement was opened to traffic to the present = 1,000,000.
- Present serviceability index = 3.5.

Solution

1. Determine the structural number (SN) of the existing pavements. In this example, the SN is assumed to be 2.85.

2. Find the soil support value of the subgrade by using Figure 20, for flexible pavements with a PSI of 3.5. The dashed line in Figure 20 indicates that, based on the traffic data and structural number given, the soil support value is 30. The soil support value used in the Corvi and Bullard method is different from that in the "AASHO Interim Guide for the Design of Flexible Pavement Structures." Furthermore, soil support values required for the design of flexible pavement structures usually are determined by field or laboratory tests on the subgrade materials. For this reason the soil support value determined by the foregoing procedure may be considered as an equivalent or hypothetical value because it is not based on experimental data obtained from tests on the actual subgrade material.

3. Find the total equivalent 18-kip single-axle load applications from the time the pavement was opened to traffic to the end of service life when a prescribed terminal serviceability index will have been reached. In this example, the terminal serviceability index is assumed to be 2.5. The dashed line in Figure 21 shows that, based on the SN and the soil support value given, total equivalent 18-kip single-axle load applications are 4,700,000.

4. Compute the number of equivalent 18-kip single-axle load applications remaining (before the pavement section requires reconstruction or resurfacing). This is done by subtracting the accumulated number up to the present time from the total number determined in Step 3: Number of equivalent 18-kip single-axle load applications remaining = 4,700,000 − 1,000,000 = 3,700,000.

5. Determine the difference in the remaining service life of the pavement section between the following cases:

Case A. No change in legal vehicle limits.

Case B. With specific changes in legal vehicle limits.

In each case, the remaining service life in years may be computed on the basis of the projected annual equivalent 18-kip single-axle load applications. An increase in legal limits in Case B will result in a shortening of the remaining service life of the pavement section because of the increase in the projected annual equivalent 18-kip single-axle load applications, as illustrated elsewhere in this example. The difference in the remaining service life of the pavement section between Cases A and B is one of the factors related to pavements that should be included in the incremental cost analysis for determining the effects of changes in vehicle weights and dimensions.

For illustration, the following projected traffic data are assumed:

Case A. Annual equivalent 18-kip single-axle load applications = 370,000.

Case B. Annual equivalent 18-kip single-axle load applications = 740,000.

Therefore,

Case A. Remaining service life $= \frac{3{,}700{,}000}{370{,}000} = 10$ years.

Case B. Remaining service life $= \frac{3{,}700{,}000}{740{,}000} = 5$ years.

The use of this information to determine the effect of changes on the cost related to pavements in a 20-year analysis period is illustrated later in this example.

Method II—Rigid Pavements

For rigid pavements, the procedure for the desired analysis is similar to that for flexible pavements, except the parameters related to subgrade and pavement structure are different. The following data are assumed to have been obtained from the studies of a particular pavement section:

- Accumulated number of equivalent 18-kip single-axle load applications from time pavement was opened to traffic to the present = 900,000.
- Present serviceability index = 4.0.
- Thickness of concrete slab = 8 in.
- Working stress in concrete = 490 psi.

Solution

1. Find the *k* value (modulus of subgrade reaction) by using Figure 22. The dashed line shows a *k* value of 150 pci. Similar to the soil support value in connection with flexible pavement analysis, the *k* value determined in this manner may be considered as an equivalent or hypothetical value because it is not based on experimental data obtained from tests on the actual subgrade material.

2. Find the total equivalent 18-kip single-axle load applications from the time the pavement was opened to traffic to the end of service life when a prescribed terminal serviceability index will have been reached. A terminal serviceability index of 2.5 is assumed in this example. The dashed line in Figure 23 shows that, based on the information available, the total equivalent 18-kip single-axle load applications are 2,750,000.

3. Compute the number of equivalent 18-kip single-axle load applications remaining (before the pavement section requires reconstruction or resurfacing). This is done by subtracting the accumulated number up to the present time from the total number determined in Step 2: Number of equivalent 18-kip single axle-load applications remaining = 2,750,000 − 900,000 = 1,850,000.

4. Determine the difference in the remaining service life of the pavement section between the following cases:

Case A. No change in legal vehicle limits.

Case B. With specific changes in legal vehicle limits.

The computations in this step are similar to those in Step 5 for the analysis of flexible pavements.

EFFECT OF CHANGES ON DESIGN OF NEW PAVEMENTS

The procedures described herein for determining the effect of changes are applicable to the design of pavements either for new highways or for reconstruction of existing highways. Again, the procedures for flexible and rigid pavements are presented separately. For either type of pavement, the method of analysis is based on the AAHSO Interim Guides for the design of pavement structures (*127, 128*).*

Flexible Pavements

For a particular highway section, the following data are assumed to have been obtained from pertinent studies described elsewhere in this report:

- Estimated total number of equivalent 18-kip single-axle load applications in a particular analysis period (a 20-year analysis period used in this example), provided there would be no change in legal vehicle limits = 328,500.
- Soil support value in the particular highway section = 6.0.
- Regional factor = 3.0.
- Terminal serviceability index = 2.0.

With the assumption of no change in legal limits, the design of the flexible pavement is carried out by the following steps:

1. Convert total equivalent load applications in the 20-year analysis period to equivalent daily 18-kip single-axle load applications.† That is: $328{,}500/(20 \times 365) = 45$.
2. Find the weighted structural number (SN) by using Figure 17, which is for the design of flexible pavements with a terminal serviceability index of 2.0. The dashed lines in the figure show that, based on the traffic data, soil support value, and regional factor given previously, the weighted SN is 2.50.
3. Determine the thickness of each pavement component. The weighted SN given here represents the combination of all components in the flexible pavement structure. For each pavement component, the thickness may be determined according to the coefficients of relative strength, such as those given in Table 30. In this example, a possible combination of three pavement components (surface course, base course, and subbase) is given:

PAVEMENT COMPONENT	THICKNESS (IN.)	COEFFICIENT OF RELATIVE STRENGTH	COMPUTATION OF SN
Asphaltic concrete surface	3	0.44	$3 \times 0.44 = 1.32$
Crushed stone base	5	0.14	$5 \times 0.14 = 0.70$
Sandy clay subbase (avg. quality)	6	0.08	$6 \times 0.08 = 0.48$
Total			2.50

This procedure also may be applied for the design of the flexible pavement for the same highway section with the assumption that there would be certain changes in legal vehicle limits. In this case, the equivalent 18-kip single-axle load applications would be increased if the changes are to allow increases in legal limits. The difference in the required thicknesses of pavements with or without changes in legal limits should be included in the incremental cost analysis concerning flexible pavements.

Rigid Pavements

The following data are assumed for a particular section of highway:

- Estimated total number of equivalent 18-kip single-axle load applications in a 20-year analysis period, provided there would be no change in legal vehicle limits = 8,979,000.
- Modulus of subgrade reaction (k) = 100 psi.
- Working stress in concrete = 490 psi.
- Terminal serviceability index = 2.5.

The following design procedure for rigid pavements is similar to that for flexible pavements.

1. For convenience in applying the design chart (Fig. C-1), convert the total equivalent load applications to equivalent daily applications: $8{,}979{,}000/(20 \times 365) = 1{,}230$.
2. Find the required thickness of the concrete slab for the pavement section. The dashed lines in Figure C-1 show that, based on the traffic data, modulus of subgrade reaction, and the working stress in concrete given above, the slab thickness is 10 in. (The design chart in Figure C-1 is for a terminal serviceability index of 2.5. The AASHO Interim Guide provides a separate design chart for a terminal serviceability index of 2.0.)

The design of rigid pavements with anticipated changes in legal vehicle limits may be carried out in a similar manner. In this case, a thickness greater than 10 in. would be required if the changes result in an increase in the equivalent 18-kip single-axle load applications.

* The Interim Guides were reviewed in "Evaluation of AASHO Interim Guides for Design of Pavement Structures" (*129*).

† Although the total number of applications is shown in a scale in Figure 17, it is more convenient to use the scale in equivalent daily applications.

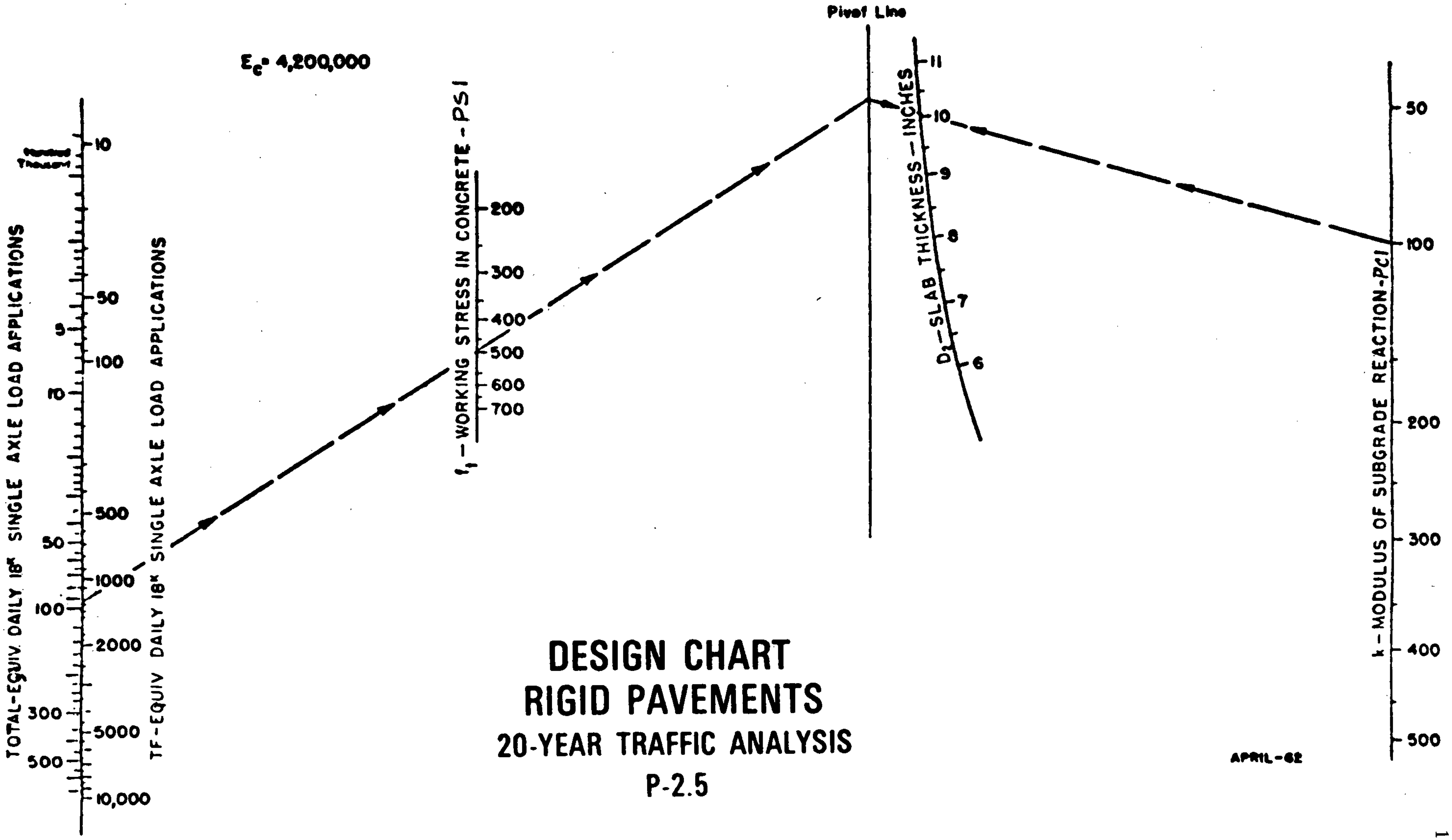


Figure C-1. Design chart, rigid pavements.

EFFECT OF CHANGES ON DESIGN OF OVERLAYS ABOVE EXISTING PAVEMENTS

Flexible overlays usually are constructed above existing pavements of either the flexible or the rigid type, after the pavements have reached the end of their service life. The thickness of a flexible overlay above either type of pavement may be determined by AASHO Interim Guide design charts. The procedure is similar to that for the design of flexible pavements described previously, except that relatively low coefficients of relative strength are used because the pavement components of the existing highway at the end of the service life are normally inferior to those at the time of construction. The following data illustrate the necessary analysis for determining the effect of changes on the required thickness of the overlay above an existing pavement (e.g., a two-lane one-mile section of flexible pavement).

Given

- Estimated total number of equivalent 18-kip single-axle load applications in a 20-year period after completion of the overlay, provided there would be no change in legal limits = 10,000,000.
- Same as above, except that certain increases in legal limits are assumed = 22,000,000.
- Soil support value = 7.5.
- Regional factor = 1.0.
- Terminal serviceability index = 2.5.
- SN * of existing pavement structure before resurfacing = 1.92.

Solution:

1. From the AASHO design chart (Fig. C-2) the SN is found to be 3.24, as indicated by a dashed line.

2. The required overlay should provide a portion of this

* In this case, the SN and the weighted SN are the same because the regional factor is 1.0.

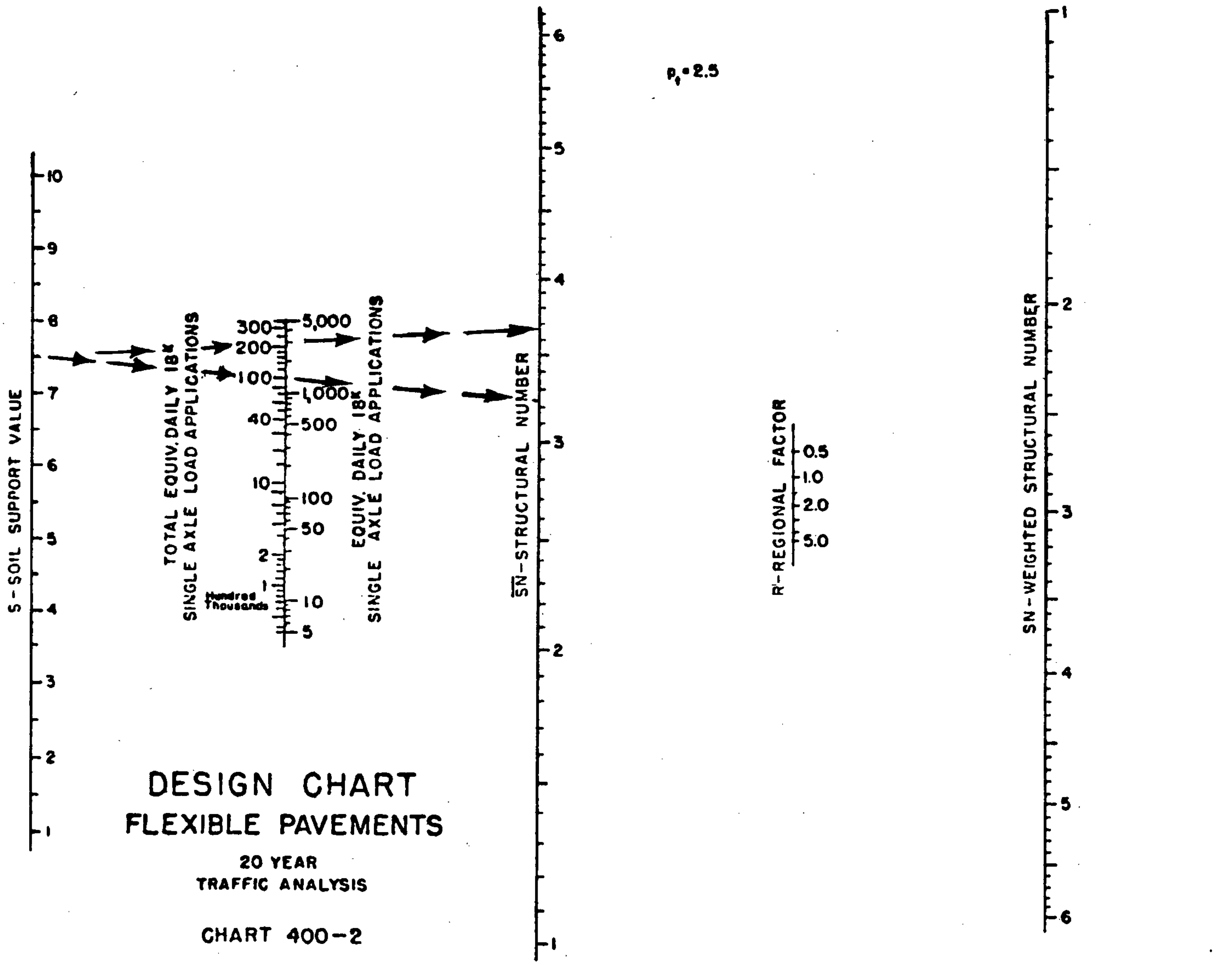


Figure C-2. Design chart, flexible pavements.

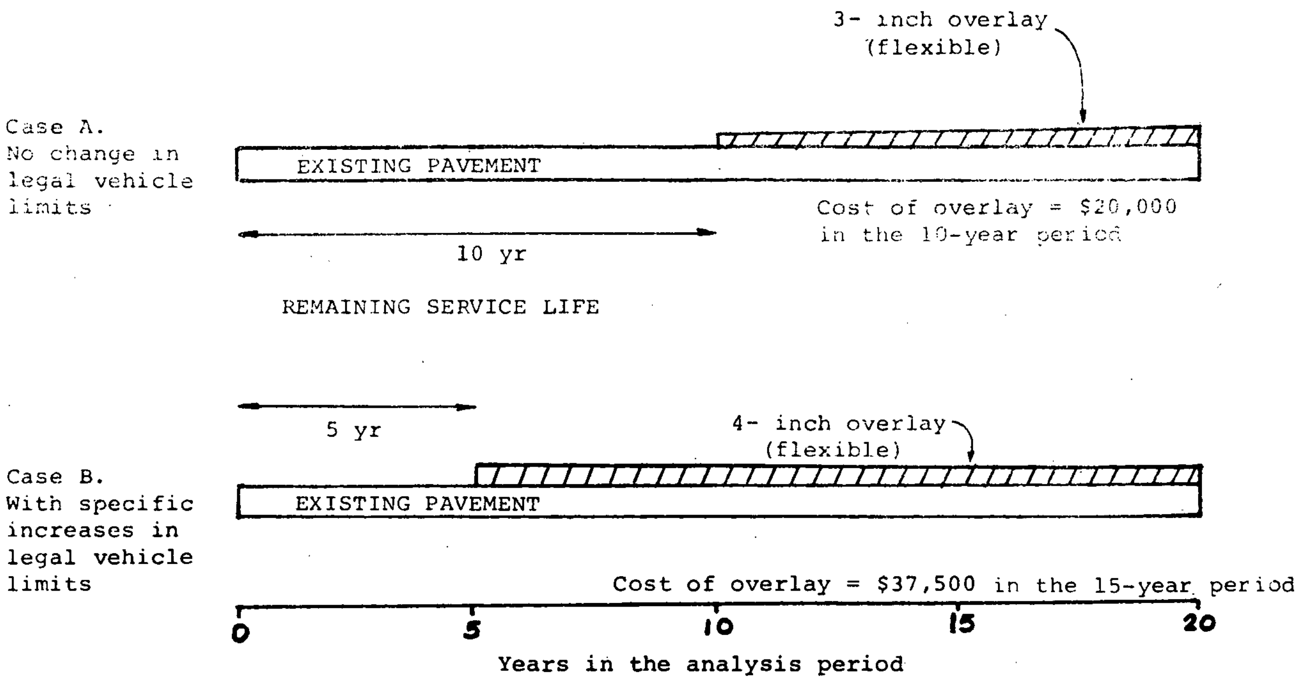


Figure C-3. Comparisons of overlay construction costs, Cases A and B.

SN: 3.4 (total SN) − 1.92 (SN of existing pavement) = 1.32.

3. If asphaltic concrete with a coefficient of relative strength of 0.44 is used for the construction of the overlay, the required thickness is 3 in.: $3 \times 0.44 = 1.32$ (SN of the overlay). The 3-in. asphaltic concrete overlay is based on the assumption of no change in legal limits.

4. The thickness of the required asphaltic concrete overlay with the assumption of certain increases in legal limits may be determined by repeating Steps 1, 2, and 3. In this case, the required SN of the overlay is computed as follows: 3.68 (total SN) − 1.92 (SN existing pavement) = 1.76. Therefore, 4-in. asphaltic concrete overlay is required: $4 \times 0.44 = 1.76$ (SN of the overlay).

The application of this information for incremental cost analysis is illustrated in the following section.

DETERMINATION OF INCREMENTAL COST RELATED TO PAVEMENTS

In this example, a 20-year analysis period is assumed. The total cost related to pavements in the 20-year period is to be computed for each of the following cases:

Case A. No change in legal vehicle limits.

Case B. With specific increases in legal vehicle limits.

The difference between the total pavement costs in these cases is the incremental cost in regard to the changes in legal limits.

Figure C-3 shows the computation of some of the costs related to the pavement section in a two-lane highway, one mile in length. The remaining service life of 10 or 5 years and the overlay thickness of 3 or 4 in. shown in the figure are in accordance with the values obtained from the analyses described previously. The portion of the original construction cost of the existing pavement to be charged against the 20-year period may be considered to be the same for Cases A and B. The incremental cost related to the overlay may be determined as follows:

- Assumed service life of pavement section after construction of overlay . . . . 20 years.
- Assumed total construction cost of overlay and related work for Case A . . . . $40,000.
- Assumed total construction cost of overlay and related work for Case B . . . . $50,000.
- If cost related to the interest of the capital investment is not included in this analysis, annual cost of the overlay within the 20-year service life in Case A = 40,000/20 = $2,000.
- Similarly, annual cost of the overlay within the 20-year service life in Case B = 50,000/20 = $2,500.
- In the 20-year period shown in Figure C-3, cost of the overlay in Case A = 10 (yr) × 2,000 = $20,000.
- Similarly, cost of the overlay in Case B in the 20-year period = 15 (yr) × 2,500 = $37,500.
- Therefore, incremental cost related to overlay construction in the 20-year period = 37,500 − 20,000 = $17,500.

If the existing pavement is reconstructed, instead of resurfaced, at the end of its service life, the incremental cost related to reconstruction may be determined in a similar manner.

In addition to the incremental cost analysis related to the resurfacing or reconstruction of existing pavements, the evaluation of the effects of changes on pavements also should include the incremental costs related to the construction of new pavements and the maintenance of existing pavements, as discussed elsewhere herein.

APPENDIX D

NUMERICAL EXAMPLE—APPLICATION OF COST/BENEFIT ANALYSIS METHODOLOGY

PROBLEM STATEMENT

This appendix sets forth in detail the application of methods described in Chapter Two, and illustrated in principle in Appendices B and C. Data tabulated in Appendix A are used to estimate benefits and costs that might result from an increase in axle weight limits from the present 18-kip single-axle and 32-kip tandem-axle legal limits to 20-kip single-axle and 34-kip tandem-axle limits.

To estimate the cost impacts of this change, as discussed in Chapter Two, one mile of a typical roadway in a specific highway system is used as the input to the problem. Using the incremental approach, the highway costs for the construction of this one mile of roadway to handle present and proposed limits must be estimated. The cost differentials include pavement construction and the average cost of bridges per mile, based on the types of bridges on the entire mileage of that highway system.

Other costs in highway construction may be incurred as a result of changes in legal limits. These costs might be for additional earthwork, drainage, shoulders, increased curve width at ramps and intersections, etc. Although these items do not appear in the specific example chosen, they are included here.

In this example, benefits are restricted to the operational cost differentials to truck operators under proposed legal limits as compared to present limits.

Both of these cost differentials are then reduced to equivalent uniform annual capital costs and benefits. The ratio of these two elements provides a benefit/cost ratio resulting from the proposed change for that highway system. This procedure follows the logic flow shown in Figure 56.

The resulting benefit/cost ratio will be applicable to the specific highway system used. For a complete cost/benefit analysis, the process should be recycled for all highway systems in the state or region being studied. Judgmental factors, not quantified in this analysis, are then melded with these data to arrive at a decision.

NUMERICAL EXAMPLE

In this numerical example, several qualifications and statements are first appropriate:

1. In compiling inputs, such as those related to the truck weight study, data from several states were combined to provide a reasonable sample data base, for no single state had what was considered a sufficient sampling of all truck classifications, empty and loaded, to be used alone. Other data, such as costs for materials, bridges in the inventory, and paving costs, were generated hypothetically for the example and were checked against experiences and records available to approach what might reasonably be expected in a specific case. However, the results of the sample calculations should not be assumed to be representative of a real-world situation, and therefore should not be used in justifying a specific proposal for change.

2. The numerical example demonstrates the application of a process only once. For a complete computation, many recycles of the same process (using other inputs but following the general steps shown) may be required. Values summarized in the example given that would result from these operations were estimated but, for brevity, were not included.

3. The example estimates only that situation relating to the primary rural system. The entire process must be repeated for all highway systems, following the same general steps used here.

4. The example given uses 1970 and 1990 as initial and terminal years of a 20-year planning period. Flexibility in both years and period is provided in the method, with appropriate changes in the constants used in arriving at the equivalent uniform annual capital costs and benefits.

Figure D-1 shows general concepts to be used in the example to estimate the incremental cost of new pavement construction, due to the proposed limit change, and the incremental transportation benefit. Figure D-2 shows general concepts for estimating the average bridge cost per mile by highway system.

PROBABLE GROSS VEHICLE WEIGHT AND AXLE WEIGHT DISTRIBUTIONS RESULTING FROM A CHANGE IN LEGAL AXLE WEIGHT LIMITS

In this example, the procedure for estimating the effects of a change in legal axle weight limits from 18/32 kips to 20/34 kips is applied to vehicle class 2-S2. This class was selected because it has each of the three types of axle configuration normally found in typical trucks: steering axle, single load axle, and tandem axle. In actual application, the procedures would be repeated for all vehicle classes to provide the estimates for probable gross weight and axle weight distributions for the entire vehicle spectrum. The objective is to present the basic concepts, assumptions, and procedures to be employed for these computations.

The procedures have not been fully and completely tested; however, calculations, concepts, and procedures indicate that the results are reasonable and should yield reliable results.

Adjusted Average Empty Weight

As Figure D-3 shows, the input to the estimation of adjusted average empty weight stems basically from truck weight study data that have been combined by highway

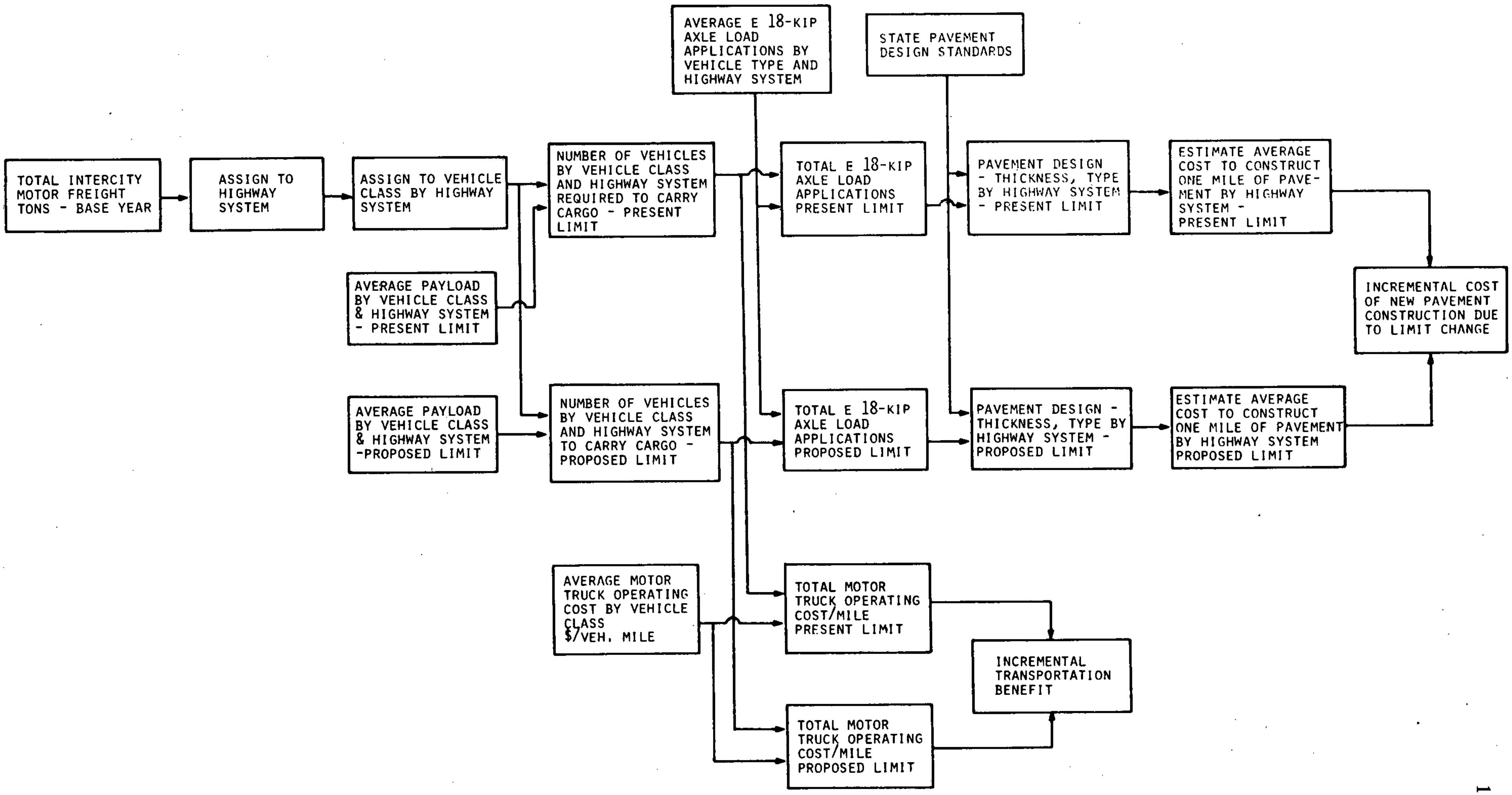


Figure D-1. General concepts used in estimating the incremental cost of new pavement and incremental transportation benefits.

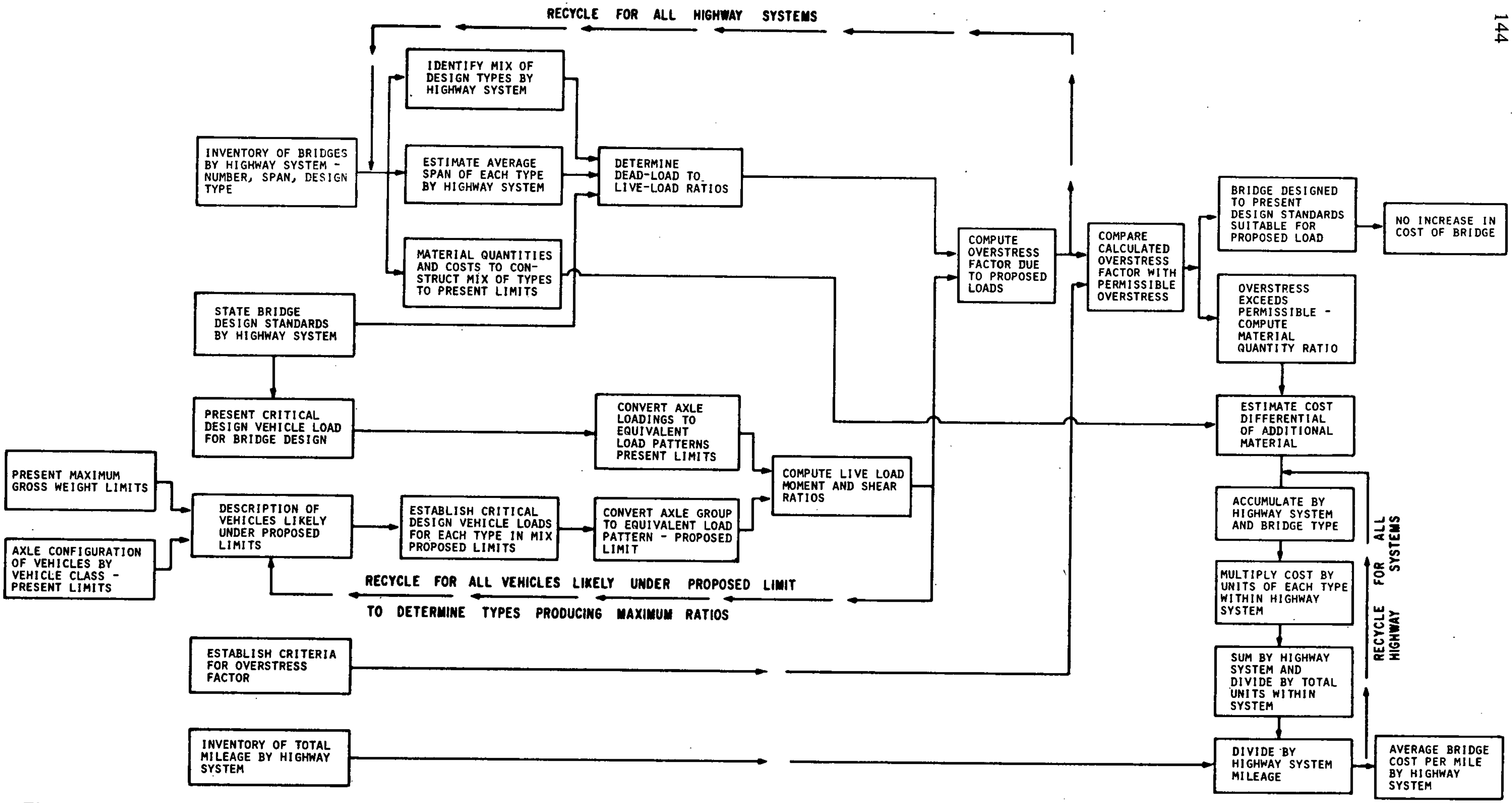


Figure D-2. General concepts used in estimating the average bridge cost per mile by highway system.

system and aggregated by vehicle class. In this example, the estimations are for the 2-S2 vehicle on Highway System 3 (primary rural). This calculation uses Table D-1. The entries in each column of the table are shown in the figure.

Step 1. Record in Table D-1, Col. B, the number of empty vehicles weighed in each 1-kip gross empty weight interval.

Step 2. Calculate the percentage of total number of empty vehicles weighed for the vehicles in Col. B for each 1-kip interval and record in Col. C.

Step 3. Calculate the cumulative percentages in Col. C and record in Col. D.

Step 4. Plot the accumulated percentages in Col. D as the ordinate, and the end-of-interval weight as the abscissa. See Figure D-4.

Step 5. Smooth the curve of Step 4 to eliminate any roughness resulting from inadequate samples of roadside weighings.

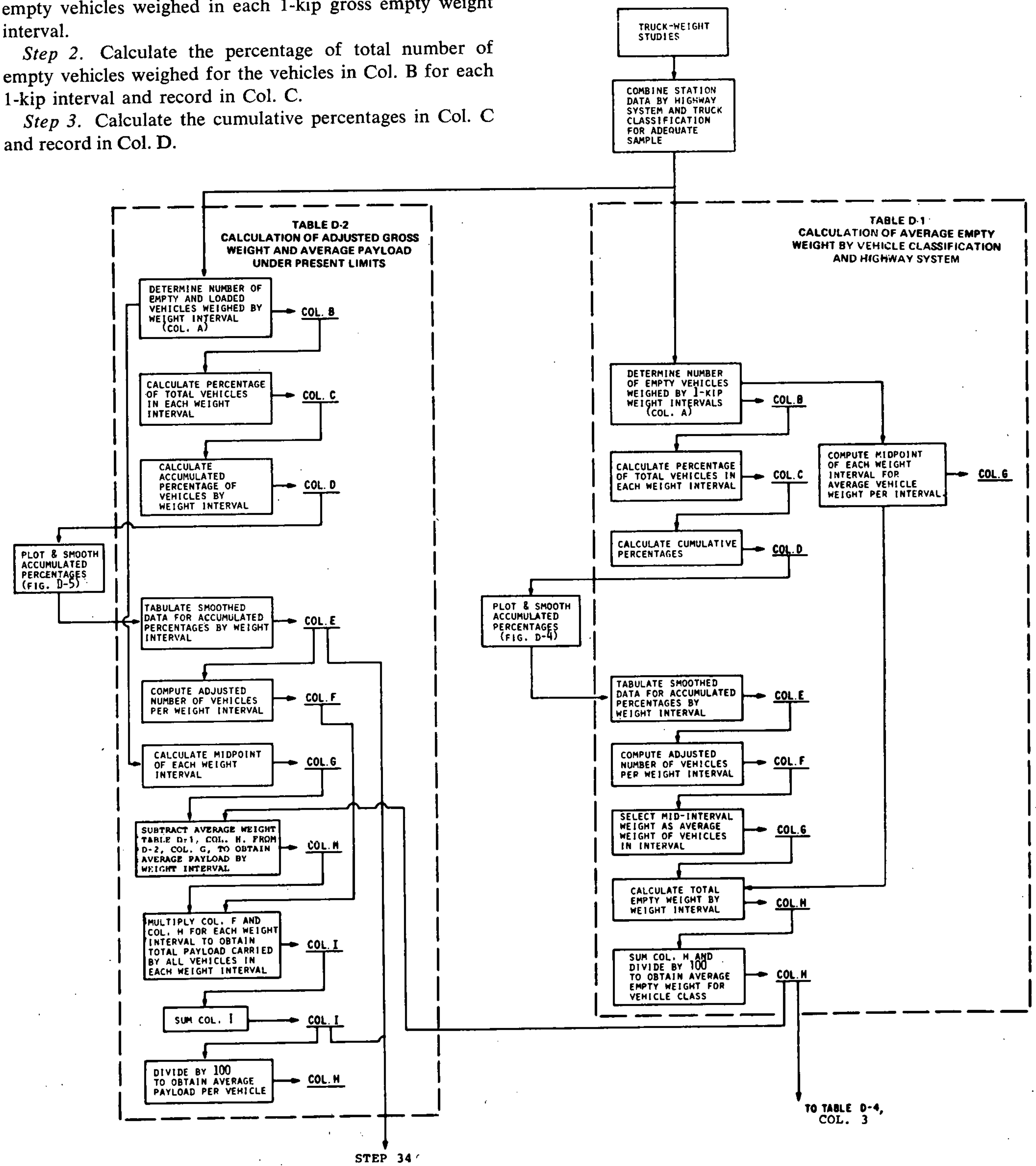


Figure D-3. Procedures for calculating average empty weight and average payload by highway system and vehicle class—present limits.

TABLE D-1

CALCULATION OF ADJUSTED AVERAGE EMPTY WEIGHT

GROSS VEHICLE WEIGHT INTERVAL (KIPS)	VEHICLES WEIGHED EMPTY							TOTAL EMPTY WEIGHT FOR ALL VEHICLES (LB)
	NO.	% OF TOTAL	ACCUMU-LATED PERCENT	ADJUSTED ACCUMU-LATED PERCENT	ADJUSTED NO. OF VEHICLES	AVG. GROSS EMPTY WEIGHT (LB)		
A	B	C	D	E	F	G		H
13–13.9				0.00	0.00			
14–14.9	1	0.86	0.86	0.90	0.90	14,500		13,050
15–15.9	1	0.86	1.72	1.00	0.10	15,500		1,550
16–16.9	0	0.00	1.72	1.50	0.50	16,500		8,250
17–17.9	0	0.00	1.72	2.00	0.50	17,500		8,750
18–18.9	1	0.86	2.58	3.00	1.00	18,500		18,500
19–19.9	3	2.59	5.17	5.00	2.00	19,500		39,000
20–20.9	5	4.31	9.48	14.00	9.00	20,500		184,500
21–21.9	9	7.76	17.24	24.00	10.00	21,500		215,000
22–22.9	18	15.52	32.76	35.00	11.00	22,500		247,500
23–23.9	13	11.21	43.97	46.00	11.00	23,500		258,500
24–24.9	15	12.93	56.90	58.00	12.00	24,500		294,000
25–25.9	13	11.21	68.11	68.00	10.00	25,500		255,000
26–26.9	6	5.17	73.28	75.00	7.00	26,500		185,500
27–27.9	7	6.04	79.32	79.00	4.00	27,500		110,000
28–28.9	4	3.45	82.77	83.00	4.00	28,500		114,000
29–29.9	3	2.59	85.36	86.00	3.00	29,500		88,500
30–30.9	3	2.59	87.95	89.00	3.00	30,500		91,500
31–31.9	4	3.45	91.40	91.00	2.00	31,500		63,000
32–32.9	2	1.72	93.12	93.00	2.00	32,500		65,000
33–33.9	2	1.72	94.84	95.00	2.00	33,500		67,000
34–34.9	2	1.72	96.56	96.00	1.00	34,500		34,500
35–35.9	0	0.00	96.56	97.00	1.00	35,500		35,500
36–36.9	2	1.72	98.28	98.00	1.00	36,500		36,500
37–37.9	1	0.86	99.14	99.00	1.00	37,500		37,500
38–38.9	1	0.86	100.00	100.00	1.00	38,500		38,500
39–39.9								
40–40.9								
Total	116	100.00						2,510,600
Average						25,106		

Note: The main purpose of Table D-1 is to calculate the average empty weight of the vehicle. If the roadside weighings are adequate for this purpose, the average empty weight may be calculated directly by adding the actual field empty weights of the vehicles weighed and dividing by the same number weighed.

Step 6. Read the smoothed curve of Figure D-4 at the interval ends and record in Col. E, Table D-1, as the adjusted accumulated percentage.

Step 7. Successively subtract the entries in Col. E to get the number (percentage) of vehicles weighed in each 1-kip interval. Record these differences in Col. F, Table D-1, as the adjusted number weighed empty in each 1-kip interval.

Step 8. Record in Col. G the midpoint of the weight interval in Col. A, Table D-1, and assume that this midpoint is the average weight of the vehicles in the interval.

Step 9. Multiply the adjusted number weighed in Col. F by the average interval weight in Col. G and record the product in Col. H, Table D-1.

Step 10. Sum Col. H and divide the total by 100 to get the average empty weight of the vehicles weighed empty, 25,106 lb.

Note: It is assumed that the loaded vehicles will have an empty weight on the average equal to the average weight of the empty vehicles weighed.

Adjusted Gross Weight and Total Payload Carried—Present Limits

Together with Figure D-3, Table D-2 is used in these calculations of adjusted gross weight distributions.

Step 11. Record in Col. B, Table D-2, the number of vehicles weighed (empty plus loaded) in each of the 1-kip intervals of Col. A.

Step 12. Calculate the percentage of total number of vehicles weighed (sum of Col. B = 388) in each weight interval and record in Col. C, Table D-2. Accumulate these interval percentages, top to bottom, and record in Col. D.

Step 13. Plot the accumulated percentages in Col. D, with the percentage as the ordinate and the gross weight at the end of the interval as the abscissa. See Figure D-5.

TABLE D-2

CALCULATION OF ADJUSTED GROSS WEIGHT (EMPTY PLUS LOADED VEHICLES) AND TOTAL PAYLOAD CARRIED—PRESENT LIMITS

GROSS VEHICLE WEIGHT INTERVAL (KIPS)	VEHICLES WEIGHED, EMPTY + LOADED				AD-JUSTED NO. OF VEHICLES	AVG. GROSS WEIGHT (LB)	AVG. PAYLOAD PER VEHICLE (LB)	TOTAL PAYLOAD FOR ALL VEHICLES AT PMGW (LB)
	NO.	% OF TOTAL	ACCUMU-LATED PERCENT	ADJUSTED ACCUMU-LATED PERCENT				
A	B	C	D	E	F	G	H	I
13–13.9	1	0.26	0.26	0.3	0.3	13,500	0	
14–14.9	1	0.26	0.52	0.5	0.2	14,500	0	
15–15.9	1	0.26	0.78	0.6	0.1	15,500	0	
16–16.9	0	0.00	0.78	0.7	0.1	16,500	0	
17–17.9	0	0.00	0.78	0.8	0.1	17,500	0	
18–18.9	2	0.52	1.30	1.5	0.7	18,500	0	
19–19.9	4	1.03	2.33	2.4	0.9	19,500	0	
20–20.9	6	1.55	3.88	3.9	1.5	20,500	0	
21–21.9	10	2.58	6.46	6.5	2.6	21,500	0	
22–22.9	19	4.90	11.36	11.4	4.9	22,500	0	
23–23.9	14	3.61	14.97	15.2	3.8	23,500	0	
24–24.9	16	4.12	19.09	19.0	3.8	24,500	0	
25–25.9	16	4.12	23.21	23.2	4.2	25,500	394	1,655
26–26.9	11	2.83	26.04	26.1	2.9	26,500	1,394	4,043
27–27.9	12	3.09	29.13	29.1	3.0	27,500	2,394	7,182
28–28.9	10	2.58	31.71	32.0	2.9	28,500	3,394	9,843
29–29.9	9	2.32	34.03	34.7	2.7	29,500	4,394	11,864
30–30.9	14	3.61	37.64	37.0	2.3	30,500	5,394	12,406
31–31.9	10	2.58	40.22	39.9	2.9	31,500	6,394	18,543
32–32.9	8	2.06	42.28	42.1	2.2	32,500	7,394	16,267
33–33.9	6	1.55	43.83	44.6	2.5	33,500	8,394	20,985
34–34.9	10	2.58	46.41	47.1	2.5	34,500	9,394	23,485
35–35.9	11	2.83	49.24	49.7	2.6	35,500	10,394	27,024
36–36.9	16	4.12	53.36	52.2	2.5	36,500	11,394	28,485
37–37.9	8	2.06	55.42	55.0	2.8	37,500	12,394	34,703
38–38.9	6	1.55	56.97	57.0	2.0	38,500	13,394	26,788
39–39.9	8	2.06	59.03	60.0	3.0	39,500	14,394	43,182
40–40.9	8	2.06	61.09	62.2	2.2	40,500	15,394	33,867
41–41.9	7	1.80	62.89	65.0	2.8	41,500	16,394	45,903
42–42.9	13	3.35	66.24	67.2	2.2	42,500	17,394	38,267
43–43.9	14	3.61	69.85	69.9	2.7	43,500	18,394	49,664
44–44.9	9	2.32	72.17	72.0	2.3	44,500	19,394	44,606
45–45.9	8	2.06	74.23	74.0	2.0	45,500	20,394	40,788
46–46.9	6	1.55	75.78	75.9	1.9	46,500	21,394	40,649
47–47.9	4	1.03	76.81	77.5	1.6	47,500	22,394	35,830
48–48.9	7	1.80	78.61	79.4	1.9	48,500	23,394	44,449
49–49.9	4	1.03	79.64	81.2	1.8	49,500	24,394	43,909
50–50.9	2	0.51	80.15	83.0	1.8	50,500	25,394	45,709
51–51.9	5	1.29	81.44	84.9	1.9	51,500	26,394	50,149
52–52.9	11	2.83	84.27	86.5	1.6	52,500	27,394	43,830
53–53.9	5	1.29	85.56	88.0	1.5	53,500	28,394	42,591
54–54.9	5	1.29	86.85	89.8	1.8	54,500	29,394	52,909
55–55.9	12	3.09	89.94	91.0	1.2	55,500	30,394	36,473
56–56.9	9	2.32	92.26	92.3	1.3	56,500	31,394	40,812
57–57.9	7	1.80	94.06	93.8	1.5	57,500	32,394	48,591
58–58.9	8	2.06	96.12	94.9	1.1	58,500	33,394	36,733
59–59.9	6	1.55	97.67	96.0	1.1	59,500	34,394	37,833
60–60.9	4	1.03	98.70	97.0	1.0	60,500	35,394	35,394
61–61.9	0	0.00	98.70	97.8	0.8	61,500	36,394	29,115
62–62.9	0	0.00	98.70	98.2	0.4	62,500	37,394	14,958
63–63.9	1	0.26	98.96	98.9	0.7	63,500	38,394	26,876
64–64.9	2	0.52	99.48	99.2	0.3	64,500	39,394	11,818
65–65.9	1	0.26	99.74	99.6	0.4	65,500	40,394	16,158
66–66.9	0	0.00	99.74	99.8	0.2	66,500	41,394	8,279
67–67.9	0	0.00	99.74	99.9	0.1	67,500	42,394	4,239
68–68.9	1	0.26	100.00	100.0	0.1	68,500	43,394	4,339
Total	388							1,291,193
Average							12,912	

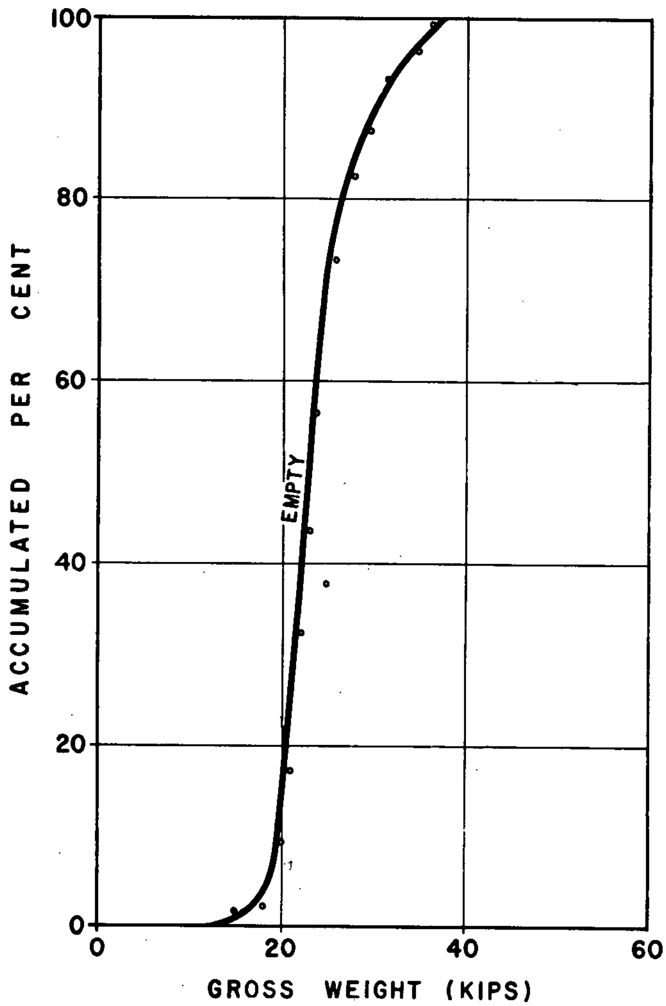


Figure D-4. Smoothed plot of empty weights—vehicle class 2-S2.

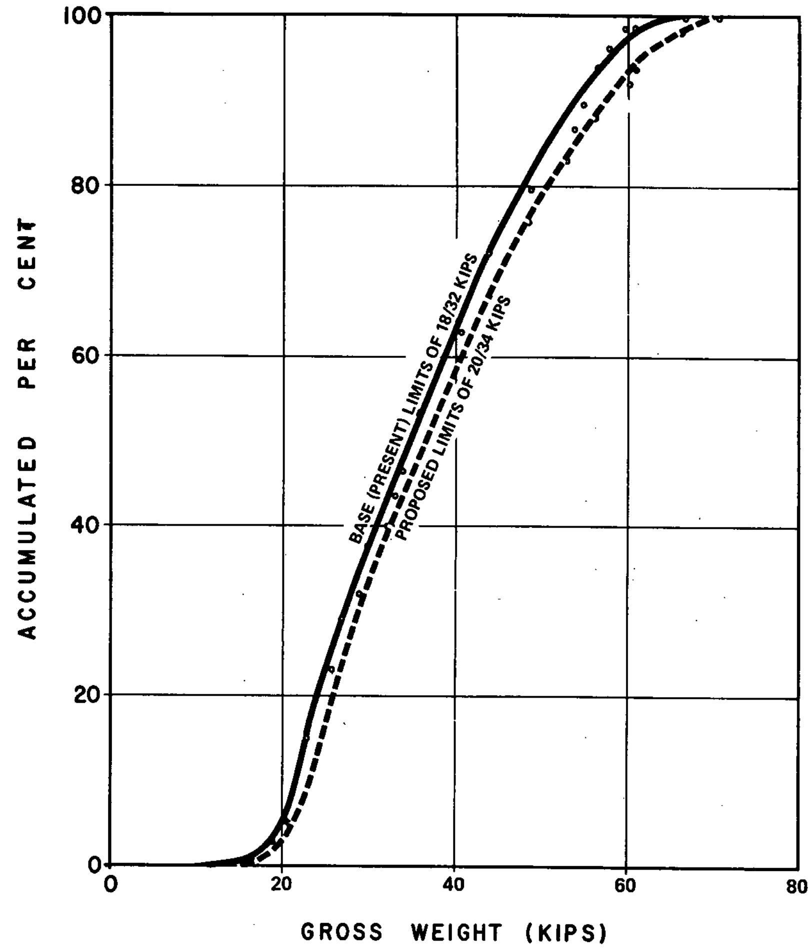


Figure D-5. Smoothed data for empty plus loaded weights—vehicle class 2-S2.

Step 14. Smooth the curve of Step 13, Figure D-5, to adjust for the errors in the sample of weighings, assuming that the curve will follow a smooth regression.

Step 15. Read the smoothed curve at the end of the 1-kip intervals and record the percentages in Col. E, Table D-2.

Step 16. Take successive differences in the entries in Col. E and record in Col. F. These interval percentages, on the basis of weighing 100 vehicles, become the adjusted number of vehicles whose gross weight falls into each of the 1-kip weight intervals of Col. A.

Step 17. In Col. G, Table D-2, record the midpoint of the interval in Col. A as being the average gross weight of the vehicles in each of the 1-kip weight intervals.

Note: This assumption introduces a slight error because the curve across an interval is not strictly a straight line in every case.

Step 18. Subtract the average empty weight (Table D-1, Step 10) from each of the average gross weights in Col. G, Table D-2, to produce the average weight of payload in each of the 1-kip weight intervals and record in Col. H. This step assumes that the loaded vehicles have an empty weight equal to the average empty weight of the empty vehicles weighed (Table D-2, Col. B).

Step 19. Multiply the number of vehicles in Col. F by the payload weight carried (Col. H) to produce the total payload carried by each set of vehicles in each gross weight interval. Record the product in Col. I, Table D-2.

Step 20. Sum Col. I to get the grand total of pounds of payload (cargo) carried by the 100 vehicles given in Col. F and divide by 100 to get the average payload per vehicle, 12,912 lb, Col. H, Table D-2.

Adjusted Gross Weight and Total Payload Carried—Proposed Limits

The procedures for calculation of gross weight and total payload carried follow those shown in Figure D-6 and use Table D-3.

Step 21. From Table D-2, Col. E, pick up the accumulated percentage and record in Col. B, Table D-3.

Step 22. By reference to manufacturers' specifications, analysis of empty weight of vehicles, practical maximum gross weight (PMGW), payloads, and other information and judgment, determine an average empty weight for the 2-S2 vehicle under a legal axle weight limit of 20-kip single

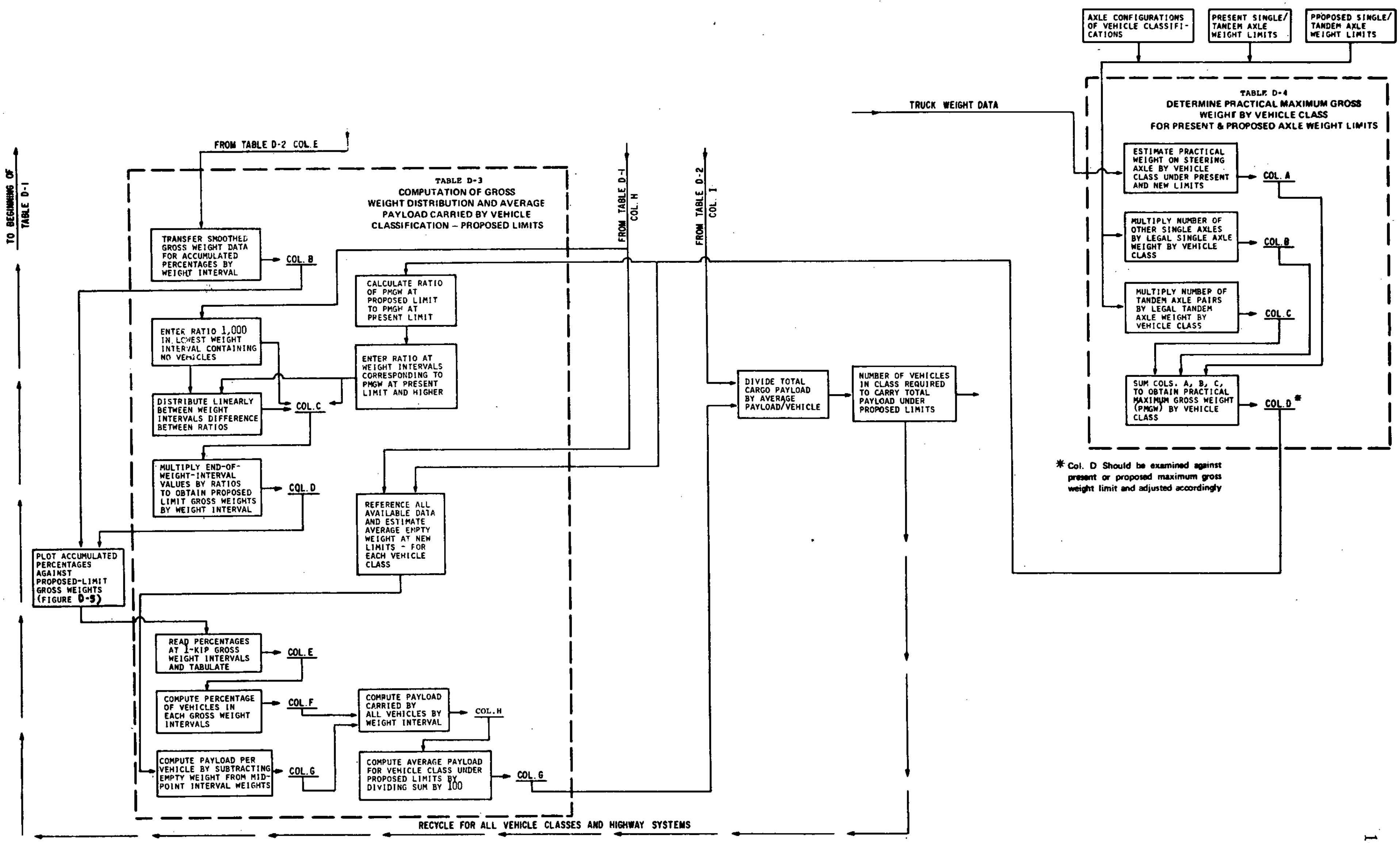


Figure D-6. Procedures for calculating PMGW, payload, and number of vehicles required—present and proposed limits.

TABLE D-3

CALCULATION OF GROSS WEIGHT AND TOTAL PAYLOAD CARRIED—PROPOSED LIMITS

GROSS VEHICLE WEIGHT INTERVAL (KIPS)	ADJUSTED ACCUMULATED PERCENT OF VEHICLES, AT END OF INTERVAL	MULTIPLYING FACTOR TO GET NEW END-OF-INTERVAL (KIPS)	GR. WT. AT END OF NEW INTERVAL FOR % IN COL. B, 20/34 LIMITS (KIPS)	ACCUMULATED % OF GROSS WEIGHT, END OF INTERVAL IN COL. A	VEHICLES IN INTERVAL (%)	PAYLOAD PER VEHICLE (KIPS)	TOTAL PAYLOAD FOR ALL VEHICLES (KIPS)
A	B	C	D	E	F	G	H
13–13.9	0.3	1.0016	14.022	0.2	0.2	0	0
14–14.9	0.5	1.0032	15.048	0.3	0.1	0	0
15–15.9	0.6	1.0048	16.077	0.4	0.1	0	0
16–16.9	0.7	1.0064	17.109	0.5	0.1	0	0
17–17.9	0.8	1.0080	18.144	0.7	0.2	0	0
18–18.9	1.5	1.0096	19.182	1.4	0.7	0	0
19–19.9	2.4	1.0112	20.224	2.3	0.9	0	0
20–20.9	3.9	1.0128	21.269	3.8	1.5	0	0
21–21.9	6.5	1.0144	22.317	6.1	2.3	0	0
22–22.9	11.4	1.0160	23.368	9.2	3.1	0	0
23–23.9	15.2	1.0176	24.422	13.9	4.7	0	0
24–24.9	19.0	1.0192	25.480	18.0	4.1	0	0
25–25.9	23.2	1.0208	26.541	21.2	3.2	0	0
26–26.9	26.1	1.0224	27.605	25.0	3.8	0.500	1.900
27–27.9	29.1	1.0240	28,672	27.9	2.9	1.500	4.350
28–28.9	32.0	1.0256	29.742	30.6	2.7	2.500	6.750
29–29.9	34.7	1.0272	30.816	32.9	2.3	3.500	8.050
30–30.9	37.0	1.0288	31.893	35.0	2.1	4.500	9.450
31–31.9	39.9	1.0304	32.973	37.2	2.2	5.500	12.100
32–32.9	42.1	1.0320	33.997	39.8	2.6	6.500	16.900
33–33.9	44.6	1.0336	35.142	42.0	2.4	7.500	18.000
34–34.9	47.1	1.0352	36.232	44.3	2.3	8.500	19.550
35–35.9	49.7	1.0368	37.325	46.7	2.4	9.500	22.800
36–36.9	52.2	1.0384	38.421	48.9	2.2	10.500	23.100
37–37.9	55.0	1.0400	39.520	51.1	2.2	11.500	25.300
38–38.9	57.0	1.0416	40.622	53.4	2.3	12.500	28.750
39–39.9	60.0	1.0432	41.728	55.7	2.3	13.500	31.050
40–40.8	62.2	1.0448	42.837	58.4	2.7	14.500	39.150
41–41.9	65.0	1.0464	43.949	60.9	2.5	15.500	38.750
42–42.9	67.2	1.0480	45.064	63.2	2.3	16.500	37.950
43–43.9	69.9	1.0496	46.182	65.3	2.1	17.500	36.750
44–44.9	72.0	1.0512	47.304	67.3	2.0	18.500	37.000
45–45.9	74.0	1.0528	48.429	69.5	2.2	19.500	42.900
46–46.9	75.9	1.0544	49.557	71.2	1.7	20.500	34.850
47–47.9	77.5	1.0560	50.688	73.0	1.8	21.500	38.700
48–48.9	79.4	1.0576	51.822	75.0	2.0	22.500	45.000
49–49.9	81.2	1.0592	52.960	76.7	1.7	23.500	39.950
50–50.9	83.0	1.0608	54.101	78.2	1.5	24.500	36.750
51–51.9	84.9	1.0624	55.245	80.0	1.8	25.500	45.900
52–52.9	86.5	1.0640	56.392	81.4	1.4	26.500	37.100
53–53.9	88.0	1.0656	57.542	83.2	1.8	27.500	49.500
54–54.9	89.8	1.0672	58.696	84.8	1.6	28.500	45.600
55–55.9	91.0	1.0688	59.853	86.1	1.3	29.500	38.350
56–56.9	92.3	1.0704	61.013	87.8	1.7	30.500	51.850
57–57.9	93.8	1.0720	62.176	89.0	1.2	31.500	37.800
58–58.9	94.9	1.0736	63.342	90.1	1.1	32.500	35.750
59–59.9	96.0	1.0736	64.416	91.2	1.1	33.500	36.850
60–60.9	97.0	1.0736	65.490	92.5	1.3	34.500	44.850
61–61.9	97.8	1.0736	66.563	93.6	1.1	35.500	39.050
62–62.9	98.2	1.0736	67.637	94.5	0.9	36.500	32.850
63–63.9	98.9	1.0736	68.710	95.4	0.9	37.500	33.750
64–64.9	99.2	1.0736	69.784	96.2	0.8	38.500	30.800
65–65.9	99.6	1.0736	70.858	97.0	0.8	39.500	31.600
66–66.9	99.8	1.0736	71.931	97.8	0.8	40.500	32.400
67–67.9	99.9	1.0736	73.005	98.2	0.4	41.500	16.600
68–68.9	100.0	1.0736	74.078	98.8	0.6	42.500	25.500
69–69.9				99.2	0.4	43.500	17.400
70–70.9				99.5	0.3	44.500	13.350
71–71.9				99.8	0.3	45.500	13.650
72–72.9				99.9	0.1	46.500	4.650
73–73.9				100.0	0.1	47.500	4.750
Total							1,375.700
Average						13.757	

axle and 34-kip tandem. In this sample, 26,000 lb (Table D-4, Col. E) was selected. The increase would be a marginal increase rather than an increase proportional to the PMGW increase.

Step 23. At the present limits (18/32 kips) average gross empty weight as calculated in Table D-1 is 25,106 lb. Enter in Col. E, Table D-4.

Step 24. By reference to the axle weight and gross weight data in the truck weight study data, the 2-S2 was determined to have a front axle weight of 8,420 lb when that vehicle class was loaded to maximum limits on the other axles. Enter this in Col. A, Table D-4. The single-axle weight limit under present limits is 18 kips, which is entered in Col. B. The present tandem-axle weight limit is 32 kips, and is entered in Col. C. The PMGW is the sum of Cols. A, B, and C, and is entered in Col. D.

Step 25. Repeat Step 24 but use instead the axle weight limits under the proposed limit. The front axle weight was estimated by a similar procedure to be 8,720 lb. The average empty weight of the 2-S2 vehicle under present limits was estimated at 25,106 lb. Under the new limits of 20/34 kips, the average empty weight was estimated at 26,000 lb.

Using the PMGW under present limits of 58,420 lb (Col. D, Table D-4) and under proposed limits of 62,720 lb, the ratio of these two PMGW's was calculated as 1.0736. The ratio of the empty gross weights was calculated as 1.0356.

Step 26. For the 1-kip intervals between 14 and 58 kips, linearly distribute the ratio (1.0736) of the two PMGW's of Step 25 and record in Col. C, Table D-3. This step assumes that the PMGW ratio will increase linearly from 1.0000 to 1.0736 in the 46 1-kip intervals between 13 and 59 kips. After the 59-kip interval (the PMGW of present limits), the ratio is assumed to hold constant.

Step 27. Multiply the end-of-interval kips in Col. A, Table D-3, by the multiplying factor in Col. C to produce the end-of-interval weight in Col. D for the new limits of 20/34 kips.

Step 28. Plot a curve with the accumulated percentages in Col. B as the ordinate and the end-of-interval weight in Col. D as the abscissa. See Figure D-5.

Step 29. Read the curve plotted in Step 28, Figure D-5, at integral 1-kip intervals and record the percentage in Col. E, Table D-3.

Step 30. Take successive differences in the interval percentage in Col. E and record in Col. F.

Step 31. Subtract from the midpoint of each interval in Col. A, Table D-3, the 26 kips of empty weight at 20/34-kip limits (see Table D-4) and record the result (payload per vehicle) in Col. G.

Step 32. Multiply Col. F by Col. G to get the total kips of payload carried by the vehicles in each weight interval. Record in Col. H.

Step 33. Sum Col. H and divide by 100 vehicles to get the average payload carried per vehicle (13,757 lb) under the new 20/34-kip limits, a gain of 845 lb per vehicle.

TABLE D-4
PRACTICAL MAXIMUM GROSS VEHICLE WEIGHT—PRESENT AND PROPOSED LIMITS

VEHICLE CLASS	PRACTICAL WEIGHT ON STEERING AXLE A (KIPS)	TOTAL LEGAL AXLE WEIGHT (KIPS): ALL OTHER SINGLE AXLES	TOTAL LEGAL AXLE WEIGHT (KIPS): ALL TANDEM PAIRS	PMGW (KIPS)	EMPTY WEIGHT (KIPS)
	A	B	C	D	E
Present axle weight limits of 18/32 kips					
2D	6.50	18.00	None	24.60	9.699
3A	10.26	None	32.00	42.26	17.684
2–S1	8.62	36.00	None	44.62	20.082
2–S2	8.42	18.00	32.00	58.42	25.106
3–S2	9.54	None	64.00	73.54	28.173
Proposed axle weight limits of 20/34 kips					
2D	7.22	20.00	None	27.22	10.000
3A	10.90	None	34.00	44.90	18.400
2–S1	9.08	40.00	None	49.08	20.900
2–S1	8.72	20.00	34.00	62.72	26.000
3–S2	10.12	None	68.00	78.12	29.500

Number of Vehicles Required to Carry Total Payload (Cargo)—Proposed Limits

Step 34. Refer to Table D-2 for the calculated pounds of payload per vehicle and the total of 1,291,193 lb of payload transported by the 100 class 2-S2 vehicles at present axle weight limits of 18/32 kips.

Step 35. Refer to Table D-3 for the calculation of pounds of payload per vehicle (13,757) at the new limits of 20/34 kips. At the new limits, the number of vehicle trips necessary to transport the 1,291,193 lb is 1,291,193/13,757 = 93.86 vehicles.

Distribution of Axle Weights—Present Limits

Figure D-9 shows the procedures to be used in Table D-5 to determine the axle weight distributions under present limits.

Step 36. Enter in Cols. B, C, and E, Table D-5, the axles weighed at the roadside, according to 1-kip intervals of weight, for the 2-S2 vehicle. The "A" axle is the front of the vehicle, etc.

Step 37. Enter in Col. D the total number of single axles weighed by the 1-kip weight intervals (Col. D is the sum of Cols. B and C).

Step 38. Calculate the percentage of the total number of single axles weighed that fall into each 1-kip interval as shown in Col. D and enter in Col. F. Likewise, compute percentages in Col. E for tandem axles and enter in Col. G.

Step 39. Accumulate the percentages in Cols. F and G and record in Cols. H and I, respectively.

Step 40. Plot the accumulated percentages of Cols. H and I and draw in a smooth curve to eliminate any sampling errors. See Figures D-7 and D-8.

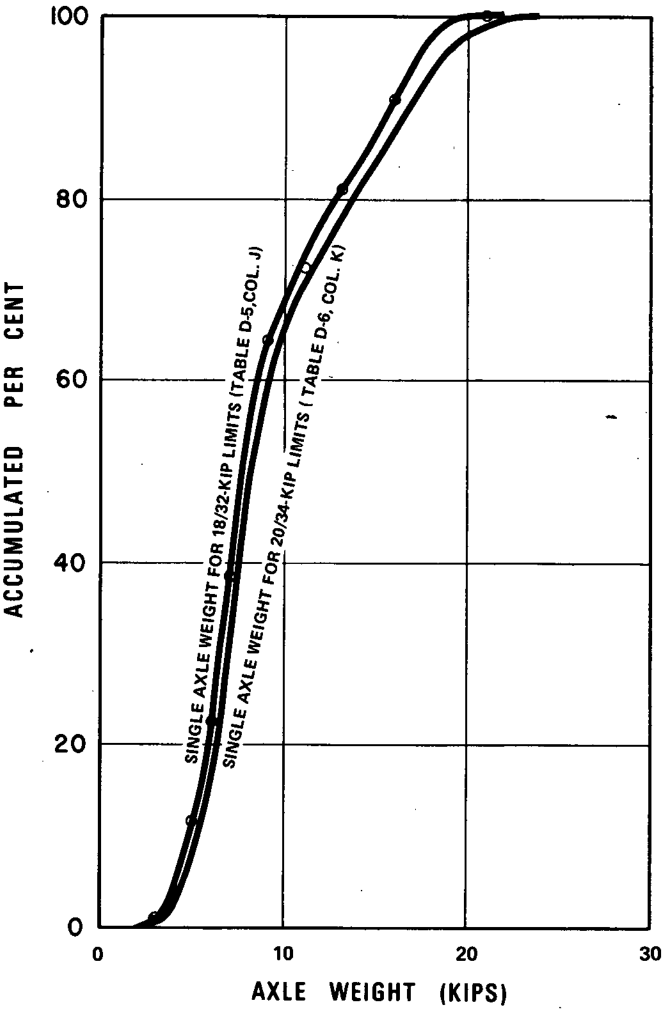


Figure D-7. Smoothed plot of single axle weights—vehicle class 2-S2.

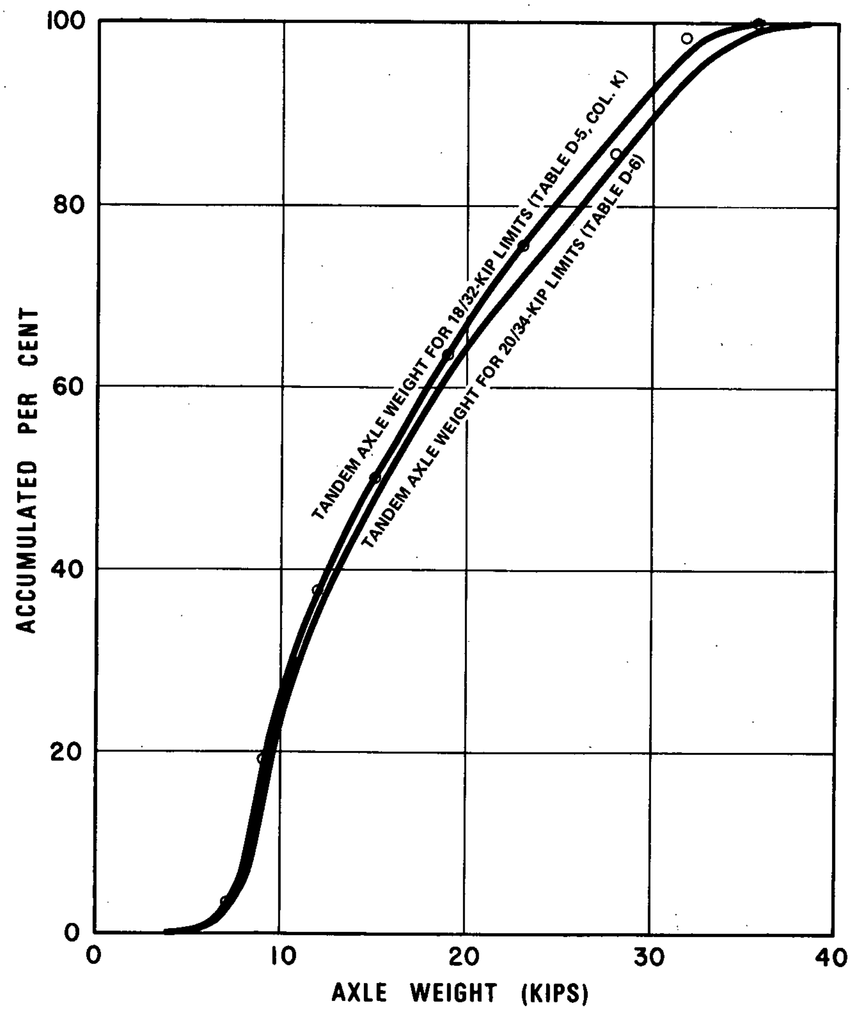


Figure D-8. Smoothed plot of tandem axle weights—vehicle class 2-S2.

Step 41. From Figures D-7 and D-8 read the smooth curve at each 1-kip interval and record the percentages in Cols. J and K, Table D-5.

Step 42. Take the successive differences in Col. J and record in Col. L. Col. L is the number (based on 100 axles) of single axles to be found in each weight interval for the 2-S2 vehicle at present limits of 18/32 kips. Repeat this procedure for tandem axles, recording the differences in Col. M, Table D-5.

Distribution of Single/Tandem Axle Weights—Proposed Limits

Procedures to be used are shown in Figure D-10; Table D-6 is used.

Step 43. Enter in Col. A, Table D-6, a series of selected accumulated percentages so chosen that the gross weight and axle weight curves are well-defined. A 5 percent interval series is sufficient, except at the beginning and for the percentages upward from about 85 percent.

Step 44. For the percentages recorded in Col. A, read from the 18/32-kip base gross weight curve of Figure D-5 the corresponding gross weights and record in Col. B. The Figure D-5 curve is the one from which the smoothed percentages were read and recorded in Table D-2, Col. E.

Step 45. Read from Figures D-7 and D-8, for the 18/32-kip limits, the single- and tandem-axle weights corresponding to the percentages in Col. A and record in Cols. D and E, Table D-6.

Step 46. Calculate the ratios of the axle weights in Cols. D and E to the gross weights in Col. B and record in Cols. G and H, Table D-6.

Step 47. From Table D-3, Col. E, plot the gross weight curve for the 20/34-kip limits on Figure D-5. Read this curve for the percentages in Col. A, Table D-6, and record in Col. I.

Step 48. Multiply Cols. G and I to produce the single-axle weights in Col. K, and similarly multiply Cols. H and I to produce the tandem-axle weight distribution in Col. L at the proposed limits of 20/34 kips.

Axle Weight Distributions by Vehicle Classification—Proposed Limits

Continue with flow of procedures shown in Figure D-10. Table D-7 is used to obtain the axle weight distributions under proposed limits.

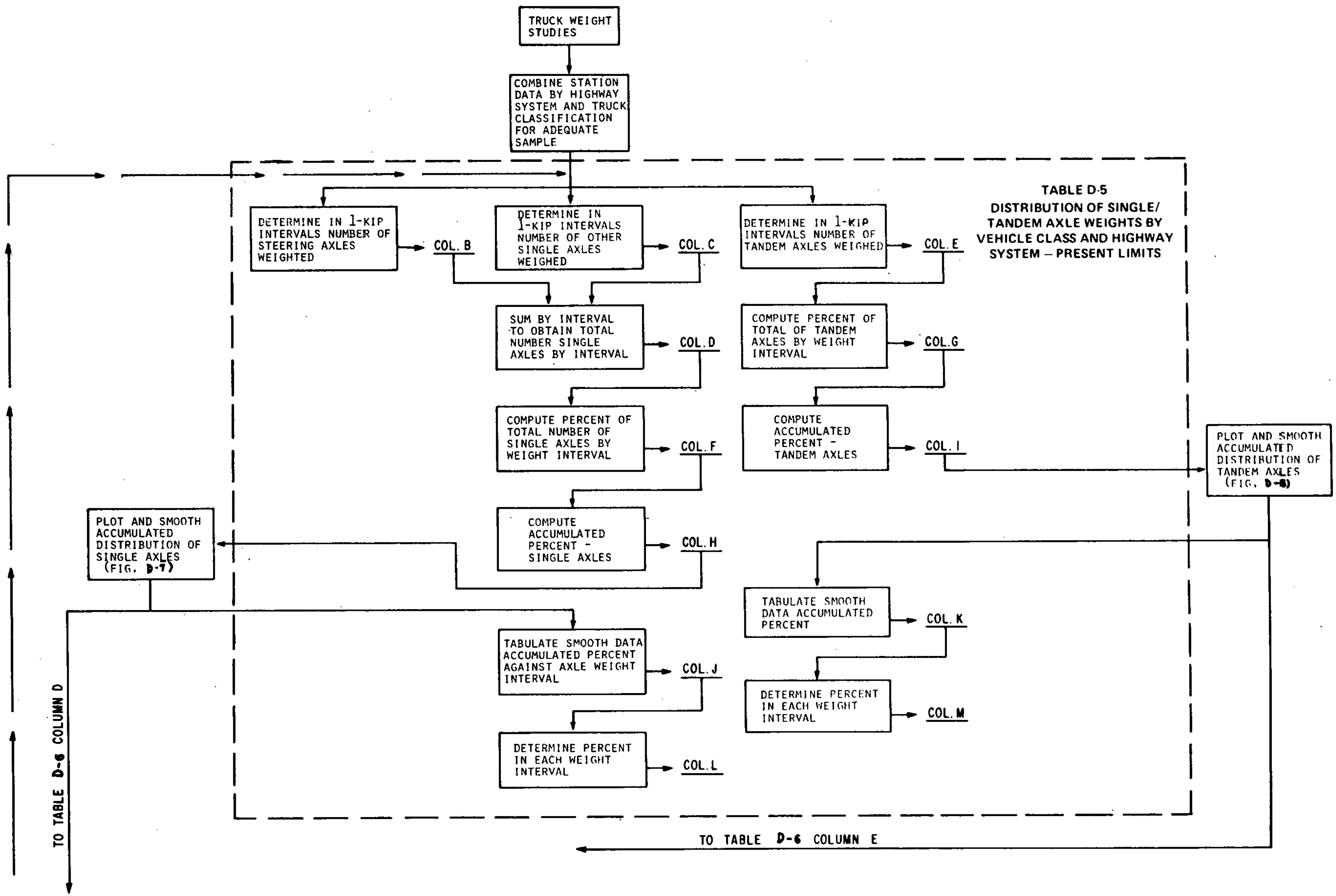


Figure D-9. Procedure for calculating distribution of axle weights by vehicle class and highway system—present limits.

TABLE D-5

DISTRIBUTION OF SINGLE/TANDEM AXLE WEIGHTS—PRESENT LIMITS

AXLE WEIGHT INTERVAL (KIPS)	AXLES WEIGHED — NUMBER: STEERING AXLES	SINGLE AXLES	TOTAL SINGLE AXLES	TANDEM AXLES	COMPUTED PERCENT OF TOTAL: SINGLE	TANDEM	COMPUTED ACCUMULATED PERCENT OF TOTAL AXLES: SINGLE	TANDEM	SMOOTHED ACCUMULATED PERCENT OF TOTAL AXLES: SINGLE	TANDEM	SMOOTHED PERCENT OF TOTAL AXLES: SINGLE	TANDEM
A	B	C	D	E	F	G	H	I	J	K	L	M
2– 2.9												
3– 3.9	5		5		0.65		0.65		0.6		0.6	
4– 4.9	22	1	23	1	2.96	0.26	3.61	0.26	3.6	0.2	3.0	0.2
5– 5.9	56	6	62	1	7.99	0.26	11.60	0.52	11.5	0.4	7.9	0.2
6– 6.9	62	21	83	2	10.70	0.52	22.30	1.04	22.3	1.0	10.8	0.6
7– 7.9	77	48	125	10	16.11	2.58	38.41	3.62	38.4	3.5	16.1	2.5
8– 8.9	91	42	133	19	17.14	4.90	55.55	8.52	55.5	8.4	17.1	4.9
9– 9.9	49	18	67	43	8.63	11.08	64.18	19.60	64.2	19.5	8.7	11.1
10–10.9	20	21	41	29	5.28	7.47	69.46	27.07	69.4	27.0	5.2	7.5
11–11.9	3	23	26	24	3.35	6.19	72.81	33.26	73.4	33.2	4.0	6.2
12–12.9	2	31	33	18	4.25	4.64	77.06	37.90	77.1	37.8	3.7	4.6
13–13.9	1	28	29	16	3.74	4.12	80.80	42.02	80.8	42.0	3.7	4.2
14–14.9		20	20	19	2.58	4.90	83.38	46.92	83.4	46.5	2.6	4.5
15–15.9		23	23	12	2.96	3.09	86.34	50.01	86.4	50.0	3.0	3.5
16–16.9		33	33	13	4.25	3.35	90.59	53.36	90.7	53.3	4.3	3.3
17–17.9		30	30	16	3.87	4.12	94.46	57.48	94.6	57.6	3.9	4.3
18–18.9		28	28	15	3.61	3.86	98.07	61.34	97.5	61.2	2.9	3.6
19–19.9		11	11	9	1.41	2.32	99.48	63.66	99.4	63.8	1.9	2.6
20–20.9		2	2	13	0.26	3.35	99.94	67.01	99.8	67.0	0.4	3.2
21–21.9		1	1	13	0.13	3.35	99.87	70.36	99.9	70.4	0.1	3.4
22–22.9		1	1	9	0.13	2.32	100.00	72.68	100.0	72.7	0.1	2.3
23–23.9				10		2.58		75.26		75.3		2.6
24–24.9				11		2.84		78.10		78.1		2.8
25–25.9				7		1.80		79.90		80.4		2.3
26–26.9				6		1.55		81.45		83.5		3.1
27–27.9				7		1.80		83.25		86.2		2.7
28–28.9				10		2.58		85.83		89.0		2.8
29–29.9				15		3.86		89.69		92.0		3.0
30–30.9				16		4.12		93.81		94.3		2.3
31–31.9				13		3.35		97.16		96.7		2.4
32–32.9				4		1.03		98.19		97.9		1.2
33–33.9				0		0.00		98.19		98.5		0.6
34–34.9				3		0.77		98.96		99.1		0.6
35–35.9				2		0.52		99.48		99.6		0.5
36–36.9				3		0.52		100.00		100.0		0.4
Total	388	388	776	389	100.00	100.00					100.0	100.0

Step 49. Plot the axle weight distribution of Cols. K and L, Table D-6, for the 20/34-kip limits on Figures D-7 and D-8, together with the axle weight distribution for the 18/32-kip limits.

Step 50. Read the percentage for each 1-kip interval of weight on Figures D-7 and D-8 for the 20/34-kip distribution and record in Cols. B and D, Table D-7.

Step 51. Take successive differences in Cols. B and D, Table D-7, and record in Cols. C and E. The frequency distributions in Cols. C and E provide the basis of calculating the equivalent 18-kip axle weight applications for use in pavement design for the new limits of 20/34 kips.

EQUIVALENT 18-KIP AXLE LOAD APPLICATIONS FOR 2-S2 VEHICLE

The computation of equivalent 18-kip axle load applications relates to Table D-8, using Figure D-11 procedures.

Step 52. Enter in Table D-8, Cols. B and C, the equivalent 18-kip single- and tandem-axle factors for the midpoint of the axle weight intervals in Col. A, by 1-kip intervals. These factors are chosen for the specific pavement type, soil conditions, and PSI factors applicable. A good source of these factors is: Federal Highway Administration, *Highway Planning Program Manual* (Transmittal 86), Vol. 4, Chap. 7, Appendix A, "Guide for Forecasting Traffic on the Interstate System for the 1970 Cost Estimate" (Jan.

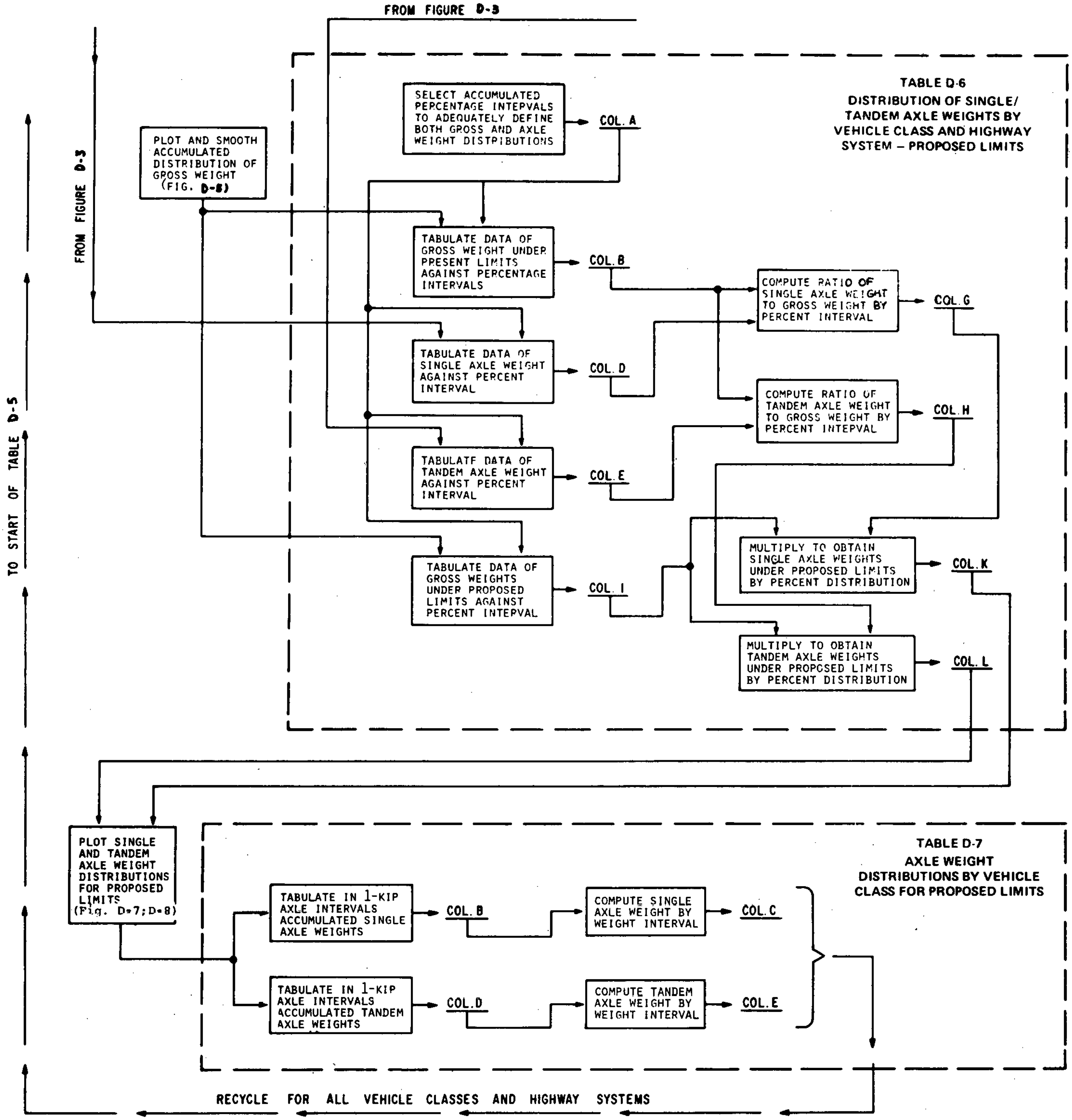


Figure D-10. Procedure for calculating single/tandem axle weights and distributions—proposed limits.

1969). The values of the equivalence factors in this reference are for integral values of the axle weight, usually to three or four significant digits. Computer programs are available for calculating the factors for decimal weights (e.g., 2.5, 3.5, 4.5) as needed for the procedure given herein. In the specific example shown, a flexible pavement is to be constructed with a structural number (SN = 3.0) and with a terminal PSI of 2.0. The 18-kip equivalent factors for these conditions are entered in Cols. B and C.

Step 53. For the present limits of 18/32 kips, bring forward from Table D-5, Cols. L and M, the percentage distribution of axle weights for each of the 1-kip weight

TABLE D-6

DISTRIBUTION OF SINGLE/TANDEM AXLE WEIGHTS—PROPOSED LIMITS

		PRESENT AXLE WEIGHT LIMITS OF 18/32 KIPS						PROPOSED AXLE WEIGHT LIMITS OF 20/34 KIPS			
		AXLE WEIGHT (KIPS)			RATIO TO GROSS WEIGHT				AXLE WEIGHT (KIPS)		
ACCUMULATED PERCENT OF VEHICLES	GROSS WEIGHT (KIPS)	STEERING AXLE	SINGLE LOAD AXLE	TANDEM AXLE	STEERING AXLE	SINGLE LOAD AXLE	TANDEM AXLE	GROSS WEIGHT (KIPS)	STEERING AXLE	SINGLE LOAD AXLE	TANDEM AXLE
A	B	C	D	E	F	G	H	I	J	K	L
0	12.0	INCLUDED	2.5	3.6	INCLUDED	0.208	0.300	12.5	INCLUDED	2.60	3.75
2	19.4	IN COL.	3.6	6.5	IN COL.	0.186	0.335	19.7	IN COL.	3.66	6.60
5	21.2	D	4.3	7.4	G	0.203	0.349	21.6	K	4.33	7.54
10	22.8		4.9	8.3		0.215	0.364	23.3		5.01	8.48
15	23.8		5.6	8.9		0.225	0.374	24.3		5.71	9.09
20	25.1		6.0	9.2		0.239	0.367	25.5		6.09	9.36
25	26.5		6.2	9.8		0.234	0.370	27.1		6.34	10.03
30	28.2		6.5	10.5		0.230	0.372	28.9		6.65	10.75
35	30.1		6.8	11.4		0.226	0.379	31.0		7.01	11.75
40	32.1		7.1	12.5		0.221	0.389	33.2		7.34	12.91
45	34.1		7.4	13.7		0.217	0.402	35.3		7.66	14.19
50	36.0		7.7	15.0		0.214	0.417	37.6		8.05	15.68
55	38.0		8.0	16.5		0.211	0.434	39.7		8.38	17.23
60	40.0		8.5	17.8		0.212	0.445	41.7		8.84	18.56
65	42.0		9.1	19.3		0.217	0.460	43.9		9.53	20.19
70	44.1		10.2	20.9		0.231	0.474	46.3		10.70	21.95
75	46.6		11.5	22.9		0.247	0.491	49.0		12.10	24.06
80	49.3		12.7	25.8		0.258	0.523	52.0		13.42	27.20
85	52.0		14.4	26.5		0.277	0.510	55.2		15.29	28.15
88	54.0		15.3	27.6		0.283	0.511	57.4		16.24	29.33
90	55.2		15.8	28.4		0.286	0.514	59.0		16.87	30.33
92	56.8		16.4	29.0		0.289	0.511	60.7		17.54	31.02
94	58.1		16.9	29.8		0.290	0.512	62.5		18.13	32.00
95	58.2		17.2	30.3		0.291	0.513	63.6		18.51	32.63
96	60.0		17.5	30.6		0.292	0.510	64.8		18.92	33.05
97	61.0		18.0	31.2		0.295	0.511	66.0		19.47	33.73
98	62.5		18.4	32.2		0.294	0.515	69.6		19.87	34.81
99	64.6		18.9	34.6		0.293	0.536	69.5		20.36	37.25
100	69.0		22.0	36.0		0.319	0.522	74.0		23.61	38.63

intervals in Col. A, and enter these percentages in Table D-8, Cols. D and F, respectively.

Step 53A. For the proposed limits of 20/34 kips, bring forward from Cols. C and E, Table D-7, the percentage distributions, and enter in Cols. H and J, respectively.

Step 54. Multiply Col. B by Col. D and enter the product in Col. E; multiply Col. C by Col. F and enter the product in Col. G. Likewise, for the new limits of 20/34 kips, multiply Cols. B and H, and enter product in Col. I; and Cols. C and J, and enter product in Col. K.

Step 55. Sum Cols. E and G and enter the totals at the bottom of the columns. The total is the number of equivalent 18-kip single and tandem axles corresponding to 100 single and tandem axles. The single-axle totals multiplied by two and added to the tandem-axle total give the total equivalent 18-kip axles for 100 vehicles of the 2-S2 class for axle weight limits of 18/32 and 20/34 kips, respectively.

Step 56. Prepare Table D-9 as a summary of the results of the foregoing steps and tables. Note that, for this example, the hauling of 1,291,193 lb of cargo under the limits of 18/32 kips applies 61.37 equivalent 18-kip single-axle loadings to the pavement. Hauling the identical pounds of payload under the limits of 20/34 kips would apply 80.95 equivalent 18-kip single-axle loadings.

Note: This numerical example provides the information necessary to calculate the operating cost of the 2-S2 vehicle under the 18/32-kip limits and the 20/34-kip limits. The applicable vehicle operating costs are given in Table A-1 for gross weights above 20 kips.

The foregoing calculations are to be repeated for each class of vehicle that would be affected by a change in legal limits of axle or gross weight or dimension.

ADT AND PAYLOAD HAULED IN 1970 AND 1990

Step 57. From all sources of ADT and classification by vehicle type, determine the present ADT (1970) by vehicle class for an average mile of the highway system under study. Prepare Table D-10, and record in Col. B the present ADT for each vehicle type, not just trucks.

Step 58. Fill in Table D-10, Col. C, from the payload analysis results in Table D-2, and multiply Cols. B and C to get Col. D.

Step 59. From roadside truck weighings for past years, reports on the trucking industry, and all other available information, estimate for 1990 the percentage productivity increase in payload per vehicle over the 20-year period. This increase comes about through increased productivity of the trucking industry through a higher percentage of fully loaded vehicles, better packaging and loading techniques, and job lot purchasing and shipping practices, and vehicle design. Enter in Table D-10, Col. E, these estimates of percentage increased payload per vehicle trip.

Step 60. Multiply the average payload per vehicle in 1970 of Col. C by unity plus the percentage in Col. E, and enter the product in Col. F.

Step 61. Develop the daily tons or ton-miles of intercity payload to be hauled in 1990 on an average mile of the highway system under study. Possible useful estimates to make are:

1. For all modes of goods transport, estimate the total tons of payload (or ton-miles) intercity (in this case for the primary rural system) movement 20 years hence.
2. Estimate the percentage of the total tons arrived at in Item 1 that will be hauled by highway vehicles.
3. Estimate the percentage (or tons arrived at in Item 2) that will be hauled on the highway system under study (primary rural in this case).
4. Estimate the total daily payload tons for 20 years hence that will be hauled over an average mile of the highway system under study.

Step 62. Following estimating the total daily tons of payload that are to be transported over an average mile of the highway system under study as arrived at in Step 61, the next step is to estimate the percentage of the total tons that will be hauled by each line-haul vehicle type. Guides for this step can be found in the percentages carried in past years by vehicle type, trucking industry trends, trends in the number of each vehicle type found in the ADT trends, and other available information. When these percentages are arrived at, multiply them by the total tons of payload for the average mile and enter the results in Table D-10, Col. G.

Step 63. From the usual practices applied to forecasting ADT, estimate the 1990 ADT by each type of vehicle, passenger carriers and trucks, and enter the results in Table D-10, Col. H.

Step 64. For the cargo-carrying vehicles, divide the total tons (or pounds) of payload to be carried daily by each class of vehicle in Col. F into the estimated total daily cargo to be hauled by each truck class in Col. G. Compare these totals with the 1990 ADT already entered in Col. H from Step 63, from the traffic forecast of ADT. Adjust the two forecasts for 1990 to give a final forecast that seems to be reasonable, and enter in Col. H (adjusting as necessary the entries already recorded there from Step 63).

Note: Steps 63 and 64 are two different approaches to estimating the line-haul ADT vehicles in Col. H.

Step 65. Prepare Table D-11 for recording the calculations of payload and ADT for 1990 with the increases in axle weight limits. For 1970 bring forward from Table D-3, Col. H, payload pounds per vehicle, and enter in Col. B. From Col. D, Table D-10, bring forward to Table D-11, Col. C, the daily payload pounds for each vehicle class in 1970.

Step 65A. Divide Col. C by Col. B to get the number of vehicle trips required to haul the total payload pounds. Enter results in Col. D.

Step 66. Estimate the percentage increase in payload per vehicle because of increased productivity 1970 to 1990 for each vehicle class and enter in Col. E. These percentages probably would be less than the similar percentages in Table D-10, Col. E, because the increased payload made possible by the increase in maximum legal axle weights would absorb some of the increase that otherwise might be achieved through better management. In Table D-11, Col. E, only one-half of the percentage estimated for the existing legal limits was estimated for the proposed limits of 20/34 kips.

TABLE D-7

AXLE WEIGHT DISTRIBUTION—PROPOSED LIMITS

	SINGLE AXLE		TANDEM AXLE	
AXLE WEIGHT INTERVAL (KIPS)	ACCUMULATED PERCENT OF VEHICLES	NO. OF VEHICLES IN INTERVAL	ACCUMULATED PERCENT OF VEHICLES	NO. OF VEHICLES IN INTERVAL
A	B	C	D	E
2– 2.9				
3– 3.9	0.5	0.5		
4– 4.9	3.0	2.5	0.2	0.2
5– 5.9	9.0	6.0	0.5	0.3
6– 6.9	18.1	9.1	0.9	0.4
7– 7.9	35.2	17.1	2.4	1.5
8– 8.9	50.2	15.0	6.6	4.2
9– 9.9	60.0	9.8	14.8	8.2
10–10.9	66.2	6.2	24.1	9.3
11–11.9	70.6	4.4	31.2	7.1
12–12.9	74.5	3.9	36.2	5.0
13–13.9	78.0	3.5	40.5	4.3
14–14.9	81.3	3.3	44.5	4.0
15–15.9	84.4	3.1	47.8	3.3
16–16.9	87.3	2.9	51.9	4.1
17–17.9	90.6	3.3	54.7	2.8
18–18.9	93.5	2.9	58.3	3.6
19–19.9	96.1	2.6	62.8	4.5
20–20.9	97.8	1.7	64.9	2.1
21–21.9	98.8	1.0	67.6	2.7
22–22.9	99.5	0.7	70.0	2.4
23–23.9	99.9	0.4	72.5	2.5
24–24.9	100.0	0.1	74.9	2.4
25–25.9			77.3	2.4
26–26.9			79.8	2.5
27–27.9			82.1	2.3
28–28.9			84.4	2.3
29–29.9			87.2	2.8
30–30.9			89.8	2.6
31–31.9			91.9	2.1
32–32.9			94.0	2.1
33–33.9			95.7	1.7
34–34.9			96.9	1.2
35–35.9			98.0	1.1
36–36.9			98.9	0.9
37–37.9			99.6	0.7
38–38.9			99.9	0.3
39–39.9			100.0	0.1

TABLE D-8

CALCULATION OF TOTAL EQUIVALENT 18-KIP AXLE LOAD APPLICATIONS—PRESENT AND PROPOSED LIMITS

	18-KIP EQUIVALENCY FACTOR AT MID-INTERVAL WEIGHT		PRESENT LIMITS				PROPOSED LIMITS			
AXLE WEIGHT INTERVAL (KIPS)	SINGLE AXLES	TANDEM AXLES	DISTRIB. OF SINGLE AXLE WEIGHTS (100 AXLES)	NO. OF E18-KIP AXLES FOR 100 SINGLE AXLES	DISTRIB. OF TANDEM AXLE WEIGHTS (100 AXLE PAIRS)	NO. OF E18-KIP AXLES FOR 100 TANDEM AXLE PAIRS	DISTRIB. OF SINGLE AXLE WEIGHTS (100 AXLES)	NO. OF E18-KIP AXLES FOR 100 SINGLE AXLES	DISTRIB. OF TANDEM AXLE WEIGHTS (100 AXLE PAIRS)	NO. OF E18-KIP AXLES FOR 100 TANDEM AXLE PAIRS
A	B	C	D	E	F	G	H	I	J	K
2– 2.9	0.00044									
3– 3.9	0.00142		0.6	0.00			0.5	0.00		
4– 4.9	0.00365	0.003407	3.0	0.01	0.2	0.00	2.5	0.01	0.2	0.00
5– 5.9	0.00804	0.004164	7.9	0.06	0.2	0.00	6.0	0.05	0.3	0.00
6– 6.9	0.01563	0.004921	10.8	0.17	0.6	0.00	9.1	0.14	0.4	0.00
7– 7.9	0.02802	0.005678	16.1	0.45	2.5	0.01	17.1	0.48	1.5	0.01
8– 8.9	0.04662	0.006435	17.1	0.80	4.9	0.03	15.0	0.70	4.2	0.03
9– 9.9	0.07320	0.007192	8.7	0.64	11.1	0.08	9.8	0.72	8.2	0.06
10–10.9	0.1097	0.009305	5.2	0.57	7.5	0.07	6.2	0.68	9.3	0.09
11–11.9	0.1589	0.01332	4.0	0.64	6.2	0.08	4.4	0.70	7.1	0.09
12–12.9	0.2233	0.01860	3.7	0.83	4.6	0.09	3.9	0.87	5.0	0.09
13–13.9	0.3045	0.02535	3.7	1.13	4.2	0.11	3.5	1.07	4.3	0.11
14–14.9	0.4087	0.03380	2.6	1.06	4.5	0.15	3.3	1.35	4.0	0.14
15–15.9	0.5373	0.04425	3.0	1.61	3.5	0.15	3.1	1.67	3.3	0.15
16–16.9	0.6966	0.05700	4.3	3.00	3.3	0.19	2.9	2.02	4.1	0.23
17–17.9	0.8882	0.07235	3.9	3.46	4.3	0.31	3.3	2.93	2.8	0.20
18–18.9	1.122	0.09065	2.9	3.25	3.6	0.33	2.9	3.25	3.6	0.33
19–19.9	1.404	0.1123	1.9	2.67	2.6	0.29	2.6	3.65	4.5	0.51
20–20.9	1.731	0.1375	0.4	0.69	3.2	0.44	1.7	2.94	2.1	0.29
21–21.9	2.129	0.1665	0.1	0.21	3.4	0.57	1.0	2.13	2.7	0.45
22–22.9	2.591	0.2005	0.1	0.26	2.3	0.46	0.7	1.81	2.4	0.48
23–23.9	3.129	0.2395	0.4		2.6	0.62	0.4	1.25	2.5	0.60
24–24.9	3.765	0.2835	0.1		2.8	0.79	0.1	0.38	2.4	0.68
25–25.9	4.490	0.3335			2.3	0.77			2.4	0.80
26–26.9		0.3900			3.1	1.21			2.5	0.98
27–27.9		0.4535			2.7	1.22			2.3	1.04
28–28.9		0.5245			2.8	1.47			2.3	1.21
29–29.9		0.6040			3.0	1.81			2.8	1.69
30–30.9		0.6925			2.3	1.59			2.6	1.80
31–31.9		0.7910			2.4	1.90			2.1	1.66
32–32.9		0.9000			1.2	1.08			2.1	1.89
33–33.9		1.020			0.6	0.61			1.7	1.73
34–34.9		1.152			0.6	0.69			1.2	1.38
35–35.9		1.300			0.5	0.65			1.1	1.43
36–36.9		1.460			0.4	0.58			0.9	1.31
37–37.9		1.635							0.7	1.14
38–38.9		1.830							0.3	0.55
39–39.9		2.040							0.1	0.20
Total				21.51		18.35		28.80		23.35
E18-kip axles for 100 vehicles (200 single axles+100 tandem axles) =						61.37				80.95

Step 67. Multiply Col. B by unity plus the percentage in Col. E and enter the result in Col. F, which will be the pounds of payload per vehicle trip in 1990, including the increase resulting from use of limits of 20/34 kips.

Step 68. From Table D-10, Col. G, bring forward to Table D-11, Col. G, the total number of daily payload pounds to be carried by each vehicle class.

Note: These pounds should be adjusted as necessary to agree with any adjustment made in estimates by vehicle type ADT forecast and the direct estimate of payload tons to be hauled by each class of vehicle in 1990. See Step 64.

Step 69. Divide Col. G by Col. F to get the required daily number of vehicles of each class to haul the given total tonnage of payload in 1990. Enter this quotient (109 vehicles) in Col. H, Table D-11.

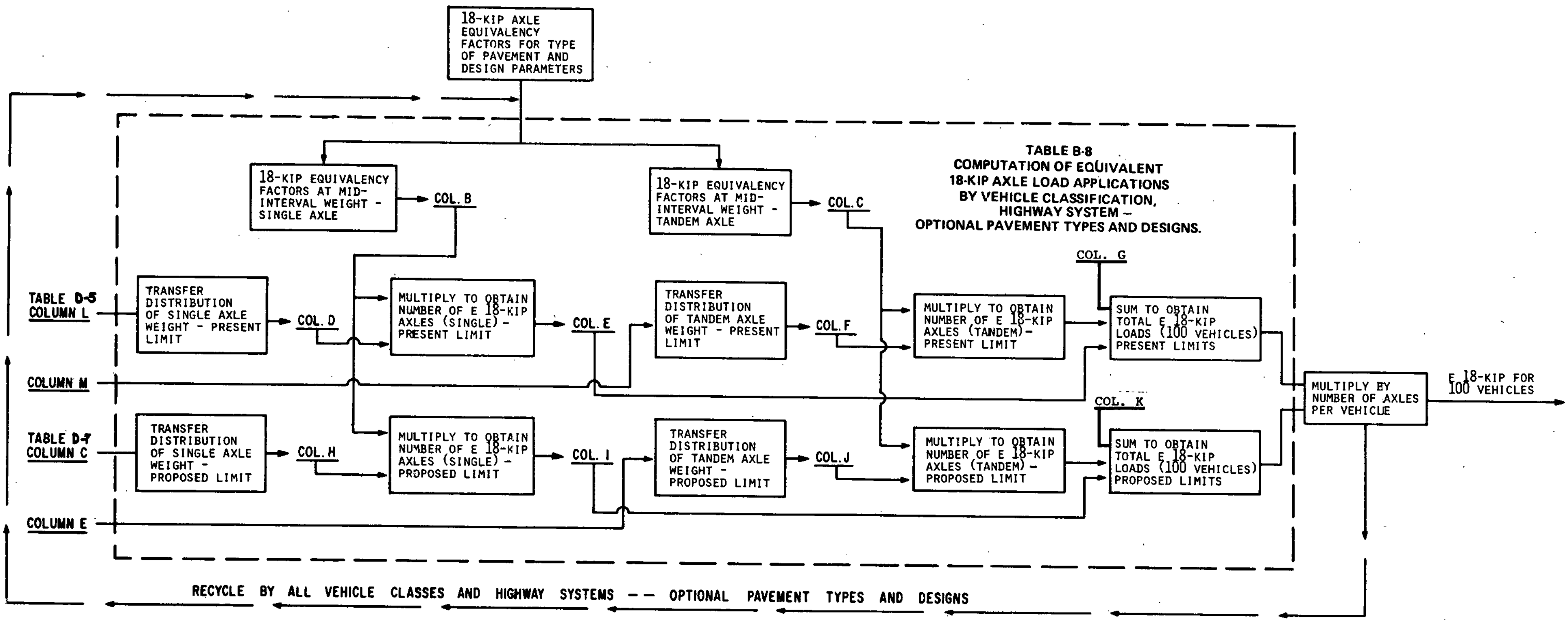


Figure D-11. Procedure for calculating equivalent 18-kip load.

TABLE D-9

SUMMARY COMPARISON: EFFECTS ON 2–S2 VEHICLE OF CHANGING LEGAL AXLE WEIGHT LIMITS

ITEM	PRESENT LIMITS, 18/32 KIPS	PROPOSED LIMITS, 20/34 KIPS
1. Total payload to be hauled (lb)	1,291,193	1,291,193
2. Payload per vehicle-trip (lb)	12,912	13,757
3. Vehicle-trips required to haul total poundage	100.00	93.86
4. Equivalent 18-kip axle applications from 100 vehicles	61.37	80.95
5. Equivalent 18-kip axle applications to haul total poundage	61.37	75.98

MOTOR TRUCK OPERATING COST

Step 69A. Bring forward from Table D-4, Col. D, the PMGW and enter in Col. B, Table D-12, for both 18/32- and 20/34-kip limits.

Step 70. For each of the vehicle classes carrying payload, multiply the practical maximum gross weight (PMGW) by 80 percent for both present and proposed legal weight limits. Enter in Table D-12, Col. C.

Note: The original work in *HRB Bulletin 301* * used the "loaded gross weight" as the independent variable. By further analysis by vehicle type the "loaded gross weight" was found to be about 80 percent of the PMGW of each vehicle class. Loaded gross weight was defined as the gross vehicle weight when the truck left the loading dock.

* STEVENS, H., "Line-Haul Trucking Costs in Relation to Vehicle Gross Weights." *HRB Bull. 301* (1961) 136 pp.

TABLE D-10

ADT AND PAYLOAD FOR PRESENT (1970) AND 1990—PRESENT LIMITS

	PRESENT (1970)			FORECAST FOR 1990			
VEHICLE CLASS	ADT	PAYLOAD PER VEHICLE (LB)	TOTAL DAILY PAYLOAD FOR ADT (LB)	INCREASE IN PAYLOAD PER VEH.-TRIP 1970–1990 (%)	PAYLOAD PER VEH.-TRIP (LB)	TOTAL DAILY PAYLOAD (LB)	NO. OF DAILY VEHICLES REQUIRED TO HAUL TOTAL PAYLOAD
A	B	C	D	E	F	G	H
1. *Passenger vehicles:*							
Cars	1,850						3,900
Motorcycles	10						50
Buses, comm.	12						25
Buses, school	8						10
Subtotal	1,880						3,985
2. *Single-unit trucks:*							
Panel, pickups (4-tire)	140						300
Other 4-tire	10						25
2D, two axle, 6-tire	120	4,010	481,200	8	4,331	952,820	220
3A, three axle	30	9,886	296,580	6	10,479	576,345	55
Subtotal	300			—			600
3. *Tractor semitrailer combs.:*							
2-S1	25	6,524	163,100	10	7,176	287,040	40
2-S2	56	12,912	723,072	12	14,461	1,590,710	110
3-S2	104	22,611	2,351,544	14	25,777	9,021,950	350
3-S1-2	10			—			—[a]
Subtotal	195						500
4. *Truck and full trailer combs:*[a]	8						—[a]
Total trucks	503						1,100
Total	2,383						5,085

[a] Included with 3-S2 combinations.

TABLE D-11

ADT AND PAYLOAD FOR PRESENT (1970) AND 1990—PROPOSED LIMITS

VEHICLE CLASS	PRESENT (1970) PAYLOAD PER VEHICLE (LB)	PRESENT (1970) TOTAL DAILY PAYLOAD FOR ALL VEHICLES IN CLASS (LB)	PRESENT (1970) ADT	FORECAST FOR 1990 INCREASE IN PAYLOAD PER VEH.-TRIP, 1970–1990 (%)	FORECAST FOR 1990 PAYLOAD PER VEH.-TRIP (LB)	FORECAST FOR 1990 TOTAL DAILY PAYLOAD (LB)	FORECAST FOR 1990 NO. OF DAILY VEHICLES REQUIRED TO CARRY TOTAL PAYLOAD
A	B	C	D	E	F	G	H
1. *Single-unit trucks:*							
Panel, pickups (4-tire)							
Other 4-tire 2D, two axle, 6-tire	4,482	481,200	108	4	4,661	952,820	205
3A, three axle	10,358	296,580	29	3	10,669	576,345	54
Subtotal		777,780	137			1,529,165	259
2. *Tractor semitrailer combs.:*							
2-S1	7,897	163,100	21	5	8,292	287,040	35
2-S2	13,757	723,072	53	6	14,582	1,590,710	109
3-S2 [a]	24,560	2,351,544	96	7	26,279	9,021,950	344
Subtotal		3,237,716	170			10,899,700	488
Total trucks		4,015,696	307			12,428,865	747

[a] Includes all combinations of 5 and more axles, but should not when such vehicles are in significant numbers.

Loaded gross weight is not average road weight; hence the 80 percent adjusting factor.

Step 71. From Table A-1 in Appendix A, interpolate the total operating cost for integral "loaded gross weights" to the 80 percent of PMGW, Col. C, Table D-12, for each vehicle class and the two legal limits. Enter results in Table D-12, Col. D, for both axle weight limits.

Step 72. Bring forward from Table D-10, Col. B, the ADT for 1970 (18/32-kip limits) and enter in Table D-12, upper section of Col. E. Bring forward from Table D-11, Col. D, the ADT for 1970 (20/34-kip limits) and enter in Table D-12, lower section of Col. E.

Step 73. Multiply Cols. D and E, Table D-12, and enter the product in Col. F, the total 1970 line-haul truck operating daily cost for each vehicle class.

Step 74. Add the Columns of Table D-11, where appropriate, and enter totals at bottom line of each section.

Step 75. Prepare Table D-13 for calculation of truck operating cost for 1990 for both axle weight limits. Enter in Col. A the classes of vehicles as needed.

Step 76. In Table D-13, Col. B, upper section, enter from Table D-10 the product of Cols. C and E. In the lower section of Col. B enter the product of Table D-11, Cols. B and E. Thus, Col. B gives the pounds of increase in payload per vehicle (under weight limits of both 18/32

TABLE D-12

TOTAL LINE-HAUL TRUCK OPERATING COST FOR PRESENT (1970)—PRESENT AND PROPOSED LIMITS

VEHICLE CLASS	PMGW (LB)	80% OF PMGW (LB)	OPER. COST (CENTS/ VEH.-MILE)	ADT OF VEHICLE CLASS	TOTAL OPER. COST ($/DAY/ MILE)
A	B	C	D	E	F
Present axle weight limits of 18/32 kips					
2D	24,600	19,680	43.02	120	51.62
3A	42,260	33,808	46.10	30	13.83
2-S1	44,620	35,696	46.55	25	11.64
2-S2	58,420	46,736	49.39	56	27.66
3-S2	73,540	58,832	52.97	104	55.09
Total					159.84
Proposed axle weight limits of 20/34 kips					
2D	27,220	21,776	43.48	108	46.96
3A	44,900	35,920	46.61	29	13.52
2-S1	49,080	39,264	47.42	21	9.96
2-S2	62,720	50,176	50.37	53	26.70
3-S2	78,120	62,496	54.14	96	51.97
Total					149.11

TABLE D-13

TOTAL LINE-HAUL TRUCK OPERATING COST FOR 1990—PRESENT AND PROPOSED LIMITS

VEHICLE CLASS	PRODUCTIVITY INCREASE, 1970-1990 (LB)	80% OF PMGW PLUS PRODUCTIVITY INCREASE (LB)	OPER. COST FOR COL. C (CENTS/ VEH.-MILE)	INCREMENT OF OPER. COST INCREASE DUE TO DRIVER, OVERHEADS, DEPRECIATION	NET OPER. COST (CENTS/ VEH.-MILE)	ADT OF VEHICLE CLASS	TOTAL OPER. COST ($/DAY/ MILE)
A	B	C	D	E	F	G	H
Present axle weight limits of 18/32 kips							
2D	230	20,000	43.15	0.05	43.10	220	94.82
3A	593	34,401	46.24	0.10	46.14	55	25.38
2-S1	652	36,348	46.71	0.12	46.59	40	18.64
2-S2	1,549	48,285	49.83	0.33	49.50	110	54.45
3-S2	3,166	61,998	53.98	0.74	53.24	350	186.34
Total							379.63
Proposed axle weight limits of 20/34 kips							
2D	179	21,995	43.52	0.03	43.49	204	89.15
3A	311	36,231	46.69	0.06	46.63	54	25.18
2-S1	395	39,659	47.53	0.08	47.45	35	16.61
2-S2	825	51,001	50.60	0.17	50.43	109	54.97
3-S2	1,719	64,215	54.68	0.39	54.29	344	186.76
Total							372.67

and 20/34 kips) due solely to management betterment in productivity, 1970 to 1990.

Step 77. To the pounds in Table D-12, Col. C, add the productivity increase from Table D-13, Col. B. Enter these sums in Col. C, Table D-13.

Step 78. From the line-haul operating cost Table A-1, interpolate the integral loaded gross weight values for the pounds of gross weight in Table D-13, Col. C, and enter the results in Col. D.

Step 79. Determine the incremental increase in line-haul operating cost in Table D-13, Col. D, as compared to the operating cost at the PMGW without the productivity increase. From the breakdown of costs in Table A-1 determine the incremental cents-per-mile costs attributable to the items of driver wage and subsistence, overhead and indirect, and depreciation and interest. Enter the total of these three cost items in Col. E, Table D-13.

Step 80. In Table D-13, subtract Col. E from Col. D to get the net operating cost to be entered in Col. F.

Step 81. To Table D-13, Col. G, bring forward the ADT for 1990 for both axle weight limits from Table D-10, Col. H, and from Table D-11, Col. H.

Step 82. Multiply Table D-13, Cols. F and G, and enter products in Col. H, total line-haul trucking cost in 1990 for the 18/32-kip limits and for the 20/34-kip limits.

INCREMENTAL COST ANALYSIS FOR NEW PAVEMENT CONSTRUCTION

This numerical example shows the incremental costs of constructing one mile of new, flexible pavement for a design life of 20 years. This pavement forms a roadway 24-ft wide, consistent with the general nature of one mile of primary rural highway in the state. The terminal present serviceability index (PSI) is chosen as 2.0. The procedure given here shows the method for flexible pavement: the procedure for rigid pavement design is given in Chapter Two and illustrated in Appendix C. These may be substituted for the example given. Also, the numerical example illustrates the method as applied to primary rural highways only. For a complete analysis, the procedure should be repeated for all highway systems in the state or region to be analyzed.

Equivalent Load Applications (ELA) and Total Load Experience

This procedure relates to Table D-14.

Step 83. From Table D-10, Col. B, transfer ADT (1970) under present limits to Table D-14, Col. B. From Table D-10, Col. H, transfer ADT (1990) under present limits to Table D-14, Col. J.

Step 84. Because passenger cars and single-unit four-tired trucks are not affected by the proposed limits, trans-

TABLE D-14

CALCULATION OF DAILY EQUIVALENT LOAD APPLICATIONS

	1970								1990							
	PRESENT LIMITS				PROPOSED LIMITS				PRESENT LIMITS				PROPOSED LIMITS			
VEHICLE CLASS	ADT	DAILY PAYLOAD (LB)	AVG. E18-KIP LOAD APPLI- CATIONS	DAILY ELA	ADT	DAILY PAYLOAD (LB)	AVG. E18-KIP LOAD APPLI- CATIONS	DAILY ELA	ADT	DAILY PAYLOAD (LB)	AVG. E18-KIP LOAD APPLI- CATIONS	DAILY ELA	ADT	DAILY PAYLOAD (LB)	AVG. E18-KIP LOAD APPLI- CATIONS	DAILY ELA
A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
1. *Passenger vehicles:*																
Cars	1,850		0.0008	1.480	1,850		0.0008	1.480	3,900		0.0008	3.120	3,900		0.0008	3.120
Motorcycles	10		—[a]		10		—[a]	1.480	3,900		—[a]				—[a]	
Buses, comm.	12		0.2000	2.400	12		0.2000	2.400	25		0.2000	5.000	25		0.2000	5.000
Buses, school	8		0.3000	2.400	8		0.3000	2.400	10		0.3000	3.000	10		0.3000	3.000
Subtotal	1,880			6.280	1,880			6.280	3,985			11.120	3,985			11.120
2. *Single-unit trucks:*																
Panel, pickups	140		0.0020	0.280	140		0.0020	0.280	300		0.0020	0.600	300		0.0020	0.600
Other 4-tired	10		0.0100	0.100	10		0.0100	0.100	25		0.0100	0.250	25		0.0100	0.250
2D	120	481,200	0.1494	17.928	108	481,200	0.1990	21.691	220	952,820	0.1494	32.868	205	952,280	0.1990	40.795
3A	30	296,580	0.3062	9.186	29	296,580	0.4052	11.751	55	576,345	0.3062	16.841	54	576,345	0.4052	21.881
Subtotal	300			27.494	288			33.822	600			50.659	584			63.526
3. *Tractor semi-trailer combs.:*																
2-S1	25	163,100	0.3744	9.360	21	163,100	0.5049	10.603	40	287,040	0.3744	14.976	35	287,040	0.5049	17.672
2-S2	56	723,072	0.6137	34.367	53	723,072	0.8095	42.904	110	1,590,710	0.6137	67.507	109	1,590,710	0.8095	88.236
3-S2	104	2,351,544	0.7178	74.651	96	2,351,544	0.9546	91.742	350	9,021,950	0.7178	251.230	344	9,021,950	0.9546	328.382
3-S1-2	10															
Subtotal	195			118.378	170			145.249	500			333.713	488			434.290
4. *Truck and full trailer combs.:*	8															
Total	2,383			152.152	2,338			185.351	5,085			395.492	5,057			508.936

[a] Negligible.

fer in Table D-14 the corresponding data for these vehicles to Cols. F and N, respectively.

Step 85. From Table D-10, Col. D, transfer total daily payload corresponding to present ADT (1970) to Table D-14, Cols. C and G.

Step 86. From Table D-10, transfer 1990 Col. G data to Table D-14, Cols. K and O.

Note: These daily payloads for the respective years should agree with motor freight tonnage assigned to primary rural highways for base year (1970) and end year (1990). These data would be obtained from an economic analysis and projection of motor freight.

Step 87. From Table D-8 compute the average 18-kip equivalent axle load per vehicle for each vehicle classification and enter into corresponding lines in Cols. D, H, L, and P.

Note: Table D-8 relates to the 2-S2 only; these data in Table D-14 for that vehicle come from Table D-8. Because the procedure is to be repeated for all vehicles, tables corresponding to those calculations (Table D-8) provide the inputs to the remaining lines in Table D-14 for the other vehicles. Further, the equivalencies for passenger cars and single-unit trucks are representative of the factors normally assigned to them. Other factors, representative of those used by the states to account for these loads may be substituted here.

Step 88. Transfer from Table D-11, Cols. D and H, the ADT computed for the proposed limits for 1970 and 1990 to the respective line in Cols. F and N.

Step 89. Compute the daily ELA by multiplying the average E-18 kip load for each vehicle type by the ADT; viz: (Col. B) × (Col. D), etc. Enter product in appropriate columns (E, I, M, and Q).

Step 90. Sum each of the daily ELA's (Cols. E, I, M, and Q) and enter result under Total.

Note: Table D-14 thus supplies the daily ELA for both present and proposed limits required to carry their assigned payloads for 1970 and 1990. In computing the average equivalent 18-kip single-axle load applications for affected vehicles (2D, 3A, 2-S1, 2-S2, and 3-S2), it was assumed that the structural number (SN) applicable to the equivalency factors would be 3.0. The need for assuming this number is in accordance with pavement design practices. The SN may have to be revised in a later step in this procedure, as is discussed. The number of significant figures in the "Average E18-kip Load" and "Daily ELA" do not represent the accuracy of these computations but are carried to that extent to prevent rounding error later. After a summation of these data, rounding the annual ELA computed will be sufficient.

Step 91. In applying the AASHO Interim Guide method for flexible pavement design (*127*), it is necessary to establish the soil support value of the subgrade and to select a proper regional factor. If a mile-long section of highway is used in an analysis of the general conditions in a particular state, a typical soil support value and a representative regional factor should be formulated in accordance with the actual conditions within the state. The following values are used in this example for illustration: soil support value = 3.0; regional factor = 1.0.

Step 92. In using the design chart of Figure 17 to find the SN, the traffic information is to be expressed in terms of either the total equivalent 18-kip single-axle load applications or average daily ELA in the 20-year traffic analysis period. In this example, the average daily ELA values were used for the two legal limits being compared in this analysis. Based on the data given in Table D-14, and assuming a uniform rate of traffic growth in the 20-year period, the average daily ELA values are:

1. Present limit: Average daily ELA = (152 + 396)/2 = 274.
2. Proposed limit: Average daily ELA = (185 + 509)/2 = 347.

These computations are based on rounded figures that are adequate in the application of the AASHO Interim Guide design charts. Based on the assumption of equal amounts of traffic on the highway in the opposite directions, the design ELA values are 50 percent of those given here.

According to the soil support value, regional factor, and traffic data given previously, the SN determined from Figure 17 is approximately as follows:

1. Present limit: SN = 3.7.
2. Proposed limit: SN = 3.9.

The structural numbers thus shown are different from that indicated in Steps 83 through 87. To correct for this difference, the computation of equivalent 18-kip axle load application beginning with Step 52 should be revised accordingly to provide consistent results in various steps as required in the design of pavement structures. This revision, however, would result in minor effects in this cost analysis. For this reason it is omitted in this example.

Step 93. This step will convert the SN to type and thickness of pavement components.

The selection of a specific type of material for each pavement component and the determination of the coefficients of relative strength of pavement components depend on local practice, information available, experience, and engineering judgment. The following information is for illustration only:

1. Present limit:

3-in. asphaltic concrete surface with a coefficient of 0.44	3 × 0.44 = 1.32
5-in. sand asphalt (hot-mix) base with a coefficient of 0.30	5 × 0.30 = 1.50
8-in. sandy gravel subbase with a coefficient of 0.11	8 × 0.11 = 0.88
Total	3.70

2. Proposed limit: The pavement structure is similar to that shown previously, except that 3½-in. asphaltic concrete surface is required to provide a total of approximately 3.9 for the required structural number.

The coefficients of relative strength of the pavement components indicated for Items 1 and 2 are based on the information given in Table 30. The foregoing analysis shows that an additional ½-in. asphaltic concrete surface is required for the pavement structure as a result of the assumed change in legal limit.

Step 94. Because the only difference between the two cases is in the thickness of asphaltic concrete surface, the cost analysis is presented in a simplified manner for the purpose of determining the incremental cost related to the assumed change in legal limit.

1. Present limit:

Cost of construction of 3-in. asphaltic concrete surface for one mile of pavement 24 ft wide	$30,000
Other costs for pavement construction (including the base and subbase)	$60,000
Total	$90,000

2. Proposed limit:

Cost of construction of 3½-in. asphaltic concrete surface for one mile of pavement 24 ft wide	$34,000
Other costs for pavement construction (including the base and subbase)	$60,000
Total	$94,000

These calculations indicate that the increase in the cost for pavement construction due to the assumed change in legal limit is $4,000 for the 1-mile highway used in this analysis. This incremental cost is due to the need of an additional ½-in. thickness of asphaltic concrete surface. In this example, the incremental cost is estimated on the basis of the additional quantity of asphaltic concrete required for the construction of the surface course. Other costs for pavement construction are assumed to be the same for the two cases being compared in this example.

INCREMENTAL COST ANALYSIS OF PROPOSED CHANGE OF LIMITS ON BRIDGE STRUCTURES

Step 95. A numerical example is worked out for a hypothetical rural primary highway system to illustrate the application of the assembled methods to arrive at an approximate change in bridge structure costs to accommodate the chosen new load limits, as shown in Figure D-9 and compared to AASHO design loading HS15.

A typical mix of common types of bridges and the bridge data for each type of bridge per mile of highway system are assumed. The user may generate the same type of actual data for the highway system under consideration from his bridge inventory data, such as that given in Table D-15. If there are any special bridge types besides these common types discussed, the user should either make a special study or assign them to the closest type that is discussed.

The unit costs of material shown are assumed to be the cost of materials in place, and are based on representative average values. The user should identify these costs for each highway system under consideration from existing record unit costs in the region in which the highway system is located. Although AASHO loading HS15 is considered in the example for comparative cost estimate purposes, the method can be applied to bridges designed for load standards different from AASHO loading. In this case the stress factors are calculated using the design load standard in lieu of AASHO loadings.

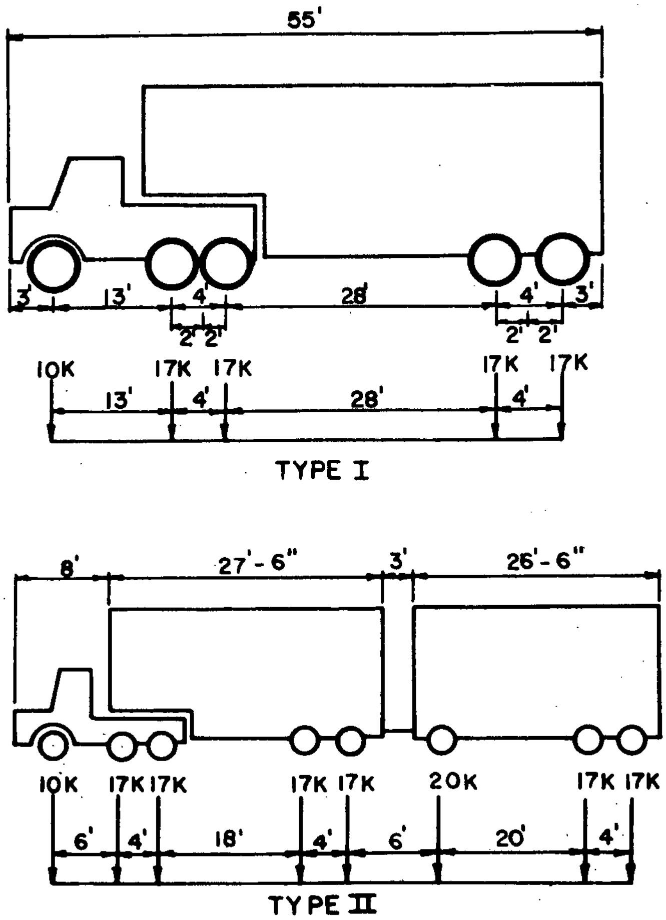


Figure D-12. Description of new loads.

The change in cost due to new load limits is computed on the basis of the material quantities required for the AASHO loading. In the following examples, required material quantities are estimated from representative design data at hand. The user may generate these quantities for each type of bridge from his bridge inventory.

The cost change in this example is estimated for new construction under proposed load limit as compared to the assumed design load. Strengthening and serviceability of bridges are not considered. The user may adapt the method developed for such bridges, as is outlined elsewhere herein.

For bridges with span lengths less than the length of wheel base of the loadings under consideration, the user should investigate, or by judgment decide, which axle group governs for each type of loading, and then use the assembled methods. This is demonstrated in the example in the cases of flat-slab and T-beam bridge types.

Assumed costs of materials are:

Concrete	$79/cu yd
Re-bar steel	$0.14/lb
Structural steel	$0.27/lb

TABLE D-15

ASSUMED DATA FROM BRIDGE INVENTORY OF A RURAL PRIMARY HIGHWAY SYSTEM [a]

ITEM \ BRIDGE TYPE	R.C. BOX GIRDER	COMPOSITE STEEL WELDED GIRDER	NONCOMPOSITE ROLLED STEEL GIRDER	PRESTRESSED CONCRETE GIRDER	T BEAM	FLAT SLABS
Distribution (%)	15	30	20	10	20	5
Area of bridge/mile of highway (sq ft)	240	480	320	160	320	80
Average span (ft)	100	90	75	70	50	26
Quantities:						
Concrete in deck slab (cu yd)	4.8	11.0	7.6	3.8	7.6	3.57
Re-bar steel in deck slab (lb)	1,840	3,680	2,450	1,225	2,450	954
Concrete in beams (cu yd)	9.5	—	—	3.33	6.9	—
Re-bar steel in beams (lb)	3,120	—	—	360	2,210	—
Concrete in substructure (cu yd)	5.3	11.3	8.3	4.32	9.5	—
Re-bar steel in substructure (lb)	876	1,872	1,370	712	1,890	—
Prestressing steel (lb)	—	—	—	360	—	—
Structural steel in beams (lb)	—	13,420	14,100	—	—	—
Piling (lin ft)	—	—	—	—	—	25.2

[a] Total bridge area/mile of highway system = 1,600 sq ft.

18-in.-square prestressed concrete piling . . . . $15/L.F.

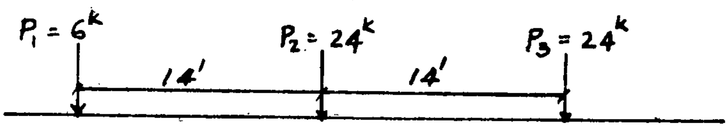
For a 48-in. prestressed concrete girder the costs would be:

Concrete	$90/cu yd
Re-bar steel	$0.18/lb
Prestressing steel	$0.55/lb

1. Reinforced Concrete Box Girder Bridges

Average span assumed = 100 ft c/c.

Step 1: Determine load parameters for HS15-44 AASHO loading.



$\bar{x}$ = distance of center of gravity of loads from load P_3

$$= \frac{6 \times 28 + 24 \times 14}{6 + 24 + 24} = 9.34 \text{ ft}$$

Therefore,

$x_1 = -14$ ft; $x_2 = 0$ ft; $x_3 = 14$ ft

$z_1 = 28$ ft; $z_2 = 14$ ft; $z_3 = 0$ ft

$b = 28$ ft

$$W = \sum_1^3 P_i = 6 + 24 + 24 = 54 \text{ kips}$$

$$\epsilon = 2\left(\frac{\sum_1^3 P_i|x_i|}{Wb}\right) = 2\left(\frac{6 \times 14 + 14 \times 24}{54 \times 28}\right) = 0.556$$

$$\mu = \frac{\sum_1^3 P_i x_i}{Wb} = \frac{-14 \times 6 + 14 \times 24}{54 \times 28} = 0.167$$

$$\delta = \frac{\sum_1^3 P_i z_i}{Wb} = \frac{6 \times 28 + 24 \times 14}{54 \times 28} = 0.334$$

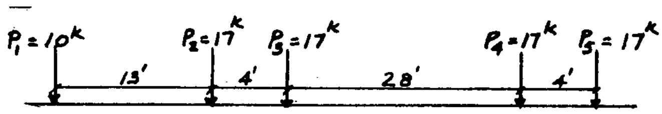
$$B_m = 2[\epsilon - (b/L)\mu^2]b = 2 \times 0.54 \times 28 = 30.2 \text{ ft}$$

$$B_v = 2\delta b = 2 \times 0.334 \times 28 = 18.7 \text{ ft}$$

$$M_L = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{54 \times 100}{4}\left(1 - \frac{30.2}{2 \times 100}\right)$$
$$= 1{,}145 \text{ ft-kips}$$

$$V_L = W\left(1 - \frac{B_v}{2L}\right) = 54\left(1 - \frac{18.7}{2 \times 100}\right) = 49.0 \text{ kips}$$

Step 2: Determine load parameters for Type I loading.



$\bar{x}$ = distance of center of gravity of loads from load P_5

$$= \frac{17(4 + 32 + 36) + 10 \times 49}{78} = 22.0 \text{ ft}$$

Therefore,

$x_1 = -17$ ft; $x_2 = -4$ ft; $x_3 = 0$; $x_4 = 28$ ft; $x_5 = 32$ ft

$z_1 = 49$ ft; $z_2 = 36$ ft; $z_3 = 32$ ft; $z_4 = 4$ ft; $z_5 = 0$

$b = 49$ft

$$W = \sum_1^5 P_i = 10 + 17 + 17 + 17 + 17 = 78 \text{ kips}$$

$$\epsilon = 2\left(\frac{\sum_1^5 P_i|x_i|}{Wb}\right) = 2\left[\frac{10 \times 17 + (4 + 28 + 32)17}{78 \times 49}\right]$$

$$= 0.66$$

$$\mu = \frac{\sum_1^5 P_i x_i}{Wb} = \frac{-10 \times 17 + 17(-4 + 28 + 32)}{78 \times 49}$$

$$= 0.204$$

$$\delta = \frac{\sum_1^5 P_i z_i}{Wb} = \frac{10 \times 49 + 17(36 + 32 + 4)}{78 \times 49} = 0.450$$

$$B_m = 2[\epsilon - (b/L)\mu^2]b = 2 \times 0.62 \times 49 = 60.6 \text{ ft}$$

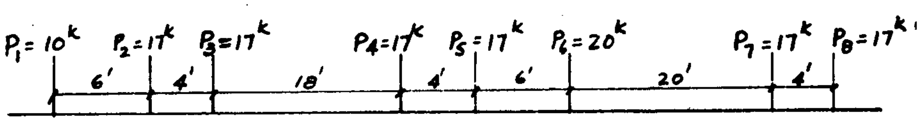
$$B_v = 2\delta b = 2 \times 0.45 \times 49 = 44.1 \text{ ft}$$

$$\overline{M}_L{}^I = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{78 \times 100}{4}\left(1 - \frac{60.6}{2 \times 100}\right)$$

$$= 1{,}360 \text{ ft-kips}$$

$$\overline{V}_L{}^I = W\left(1 - \frac{B_v}{2L}\right) = 78\left(1 - \frac{44.1}{2 \times 100}\right) = 60.8 \text{ kips}$$

Step 3: Determine load parameters for Type II loading.



$\bar{x}$ = distance of center of gravity of loads from load P_8

$$= \frac{10 \times 62 + 17(56 + 52 + 34 + 30 + 4) + 20 \times 24}{10 + 17 \times 6 + 20}$$

$$= 31.0 \text{ ft}$$

Therefore,

$x_1 = -32$ ft; $x_2 = -26$ ft; $x_3 = -22$ ft; $x_4 = -4$ ft; $x_5 = 0$ ft; $x_6 = 6$ ft; $x_7 = 26$ ft; $x_8 = 30$ ft

$z_1 = 62$ ft; $z_2 = 56$ ft; $z_3 = 52$ ft; $z_4 = 34$ ft; $z_5 = 30$ ft; $z_6 = 24$ ft; $z_7 = 4$ ft; $z_8 = 0$ ft

$b = 62$ ft

$$W = \sum_1^8 P_i = 10 + 17 + 17 + 17 + 17 + 20 + 17 + 17 = 132 \text{ kips}$$

$$\epsilon = 2\left(\frac{\sum_1^8 P_i|x_i|}{Wb}\right)$$

$$= 2\left[\frac{10 \times 32 + 17(26 + 22 + 4 + 26 + 30) + 20 \times 6}{132 \times 62}\right]$$

$$= 0.557$$

$$\mu = \frac{\sum_1^8 P_i x_i}{Wb}$$

$$= \frac{-10 \times 32 - 17(-26 - 22 - 4 + 26 + 30) + 20 \times 6}{132 \times 62}$$

$$= -0.016$$

$$\delta = \frac{\sum_1^8 P_i z_i}{Wb}$$

$$= \frac{10 \times 62 + 17(56 + 52 + 34 + 30 + 4) + 20 \times 24}{132 \times 62}$$

$$= 0.50$$

$$B_m = 2[\epsilon - (b/L)\mu^2]b = 2 \times 0.557 \times 62 = 69.1 \text{ ft}$$

$$B_v = 2\delta b = 2 \times 0.50 \times 62 = 62.0 \text{ ft}$$

$$\overline{M}_L{}^{II} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{132 \times 100}{4}\left(1 - \frac{69.1}{2 \times 100}\right)$$

$$= 2{,}160 \text{ ft-kips}$$

$$\overline{V}_L{}^{II} = W\left(1 - \frac{B_v}{2L}\right) = 132\left(1 - \frac{62.0}{2 \times 100}\right) = 91.1 \text{ kips}$$

Because $\overline{M}_L{}^{II} > \overline{M}_L{}^I$ and $\overline{V}_L{}^{II} > \overline{V}_L{}^I$, Type II loading is more severe. Hence, Type II loading should be considered.

Step 4: Determine live load ratios. Live load moment ratio of Type II new load to HS15-44 loading:

$$\alpha_m = \frac{\overline{M}_L{}^{II}}{M_L} = \frac{2{,}160}{1{,}145} = 1.88$$

Live load shear ratio of Type II new load to HS15-44 loading:

$$\alpha_v = \frac{\overline{V}_L{}^{II}}{V_L} = \frac{91.1}{49.0} = 1.86$$

Step 5: Determine stress factor.

A. *Deck Slab:* Flexural stress factor for deck slabs:

$$\Omega_m{}^S = \frac{\overline{P}}{P} - 1 = \frac{20}{24} - 1 = 0.167$$

B. *Beams:* Flexural stress factor:

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \beta_m} = \frac{1.88 - 1}{1 + 2.06} = 0.288$$

Shear stress factor:

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + \beta_v} = \frac{1.86 - 1}{1 + 2.65} = 0.236$$

in which

$\beta_v = 2.65$ (from Fig. B-6)

β_m = ratio of dead to live load HS15-44 moment ratio = 2.06 (from Fig. B-5)

C. *Piers and Foundations (Substructure):* Stress factor:

$$\Omega_p = \Omega_f < \Omega_m{}^G$$
$$\text{and} < \Omega_v{}^G$$

Step 6: Determine material quantity ratio.

A. *Deck Slab:*

$$\gamma_s = (\overline{P}/P)^{\frac{1}{2}} = (0.833)^{\frac{1}{2}} = 0.914 < 1.0$$

Use 1.0 for material quantity ratio of deck slab. Therefore, there is no change in material quantities of deck slab.

B. *Beams:*

$$\gamma_{sg} = \tfrac{1}{2}[(1+\Omega_v{}^G) + (1+\Omega_m{}^G)^{\frac{1}{2}}]$$
$$= \tfrac{1}{2}[(1+0.236) + (1+0.288)^{\frac{1}{2}}]$$
$$= 1.18$$

C. *Piers and Foundations:* Same as material quantity ratio for beams.

$$\gamma_p = \gamma_f = \gamma_{sg} = 1.18$$

Step 7: Determine increase in cost due to new loading.

A. *Deck Slab:* No change in material quantities.

B. *Beams:* Cost of increase in concrete quantity:

$$\Delta C = (\gamma_{sg} - 1)QU$$

in which

Q = concrete quantity for HS15-44 loading = 9.5 cu yd;

U = unit cost of concrete = \$79/cu yd;

γ_{sg} = material quantity ratio = 1.18.

Therefore, $\Delta C = 0.18 \times 9.5 \times 79 = \$139$

Cost of increase in steel quantity:

$$\Delta C = (\gamma_{sg} - 1)QU$$
$$= 0.18 \times 3{,}120 \times 0.14$$
$$= \$78.60$$

C. *Piers and Foundations (Substructure):* Cost of increase in quantity of concrete:

$$\Delta C = (\gamma_p - 1)QU$$
$$= 0.18 \times 5.3 \times 79 = \$75.50$$

Cost of increase in quantity of steel:

$$\Delta C = (\gamma_p - 1)QU$$
$$= 0.18 \times 876 \times 0.14$$
$$= \$22.10$$

Step 8: Total increase in cost due to new loading.

$$\Sigma\Delta C = 139.00 + 78.60 + 75.50 + 22.10 = \$316.20$$

2. Steel Welded Girder Bridges Composite Design

Average span assumed = 90 ft c/c.

Step 1: Determine load parameters for HS15-44 loading.

$$\left.\begin{aligned} \epsilon &= 0.556 \\ \mu &= 0.167 \\ \delta &= 0.334 \\ B_m &= 30.2 \text{ ft} \\ B_v &= 18.7 \text{ ft} \end{aligned}\right\} \text{Same as box girder.}$$

$$M_L = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right)$$
$$= \frac{54 \times 90}{4}\left(1 - \frac{30.2}{2 \times 90}\right) = 1{,}010 \text{ ft-kips}$$

$$V_L = W\left(1 - \frac{B_v}{2L}\right) = 54\left(1 - \frac{18.7}{2 \times 90}\right) = 48.4 \text{ kips}$$

Step 2: Because the span is larger than the length of axle base, the Type II loading is more severe than Type I loading. Therefore, Step 2 need not be computed.

Step 3: Determine load parameters for Type II loading.

$$\left.\begin{aligned} \epsilon &= 0.557 \\ \mu &= -0.016 \\ \delta &= 0.500 \\ B_m &= [\epsilon - (b/L)\mu^2]b = 69.1 \text{ ft} \\ B_v &= 2\delta b = 62.0 \text{ ft} \end{aligned}\right\} \text{Same as box girder.}$$

$$\overline{M}_L{}^{\text{II}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right)$$
$$= \frac{132 \times 90}{4}\left(1 - \frac{69.1}{2 \times 90}\right) = 1{,}830 \text{ ft-kips}$$

$$\overline{V}_L{}^{\text{II}} = W\left(1 - \frac{B_v}{2L}\right) = 132\left(1 - \frac{62.0}{2 \times 90}\right) = 86.5 \text{ kips}$$

Step 4: Determine live load ratios.

$$\alpha_m = \frac{\overline{M}_L{}^{\text{II}}}{M_L} = \frac{1{,}830}{1{,}010} = 1.81$$

$$\alpha_v = \frac{\overline{V}_L{}^{II}}{V_L} = \frac{86.5}{48.4} = 1.79$$

Step 5: Determine stress factor.

A. *Deck Slab:*

$$\Omega_m{}^S = \frac{\overline{P}}{P} - 1 = \frac{20}{24} - 1 = 0.167$$

B. *Beams:*

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \beta_m\theta_b} = \frac{1.81 - 1}{1 + 1.1 \times 1.3} = 0.333$$

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + \beta_v} = \frac{1.79 - 1}{1 + 1} = 0.395$$

in which

β_m = 1.1 (from Fig. B-7)
β_v = 1.0 (from Fig. B-8)
θ_b = sectional modular ratio for bottom fiber = 1.3 for average conditions.

C. *Piers and Foundations:*

$$\Omega_p = \Omega_f < \Omega_m{}^G$$
$$\text{and} \quad < \Omega_v{}^G$$

Step 6: Material quantity ratio.

A. *Deck Slab:* Use 1.0 as in box girder. No change in material quantity.

B. *Beams:*

$$\gamma_{sg} = \tfrac{1}{2}[(1+\Omega_v{}^G) + (1+\Omega_m{}^G)^{\frac{1}{2}}]$$
$$= \tfrac{1}{2}[(1+0.395) + (1+0.333)^{\frac{1}{2}}]$$
$$= 1.27$$

C. *Piers and Foundations (Substructure):*

$$\gamma_p = \gamma_f = \gamma_{sg} = 1.27$$

Step 7: Determine increase in cost.

A. *Deck Slab:* No change in cost.

B. *Beams:* Cost of increase in structural steel, etc., in superstructure:

$$\Delta C = (\gamma_{sg} - 1)QU$$
$$= 0.27 \times 13{,}420 \times 0.27$$
$$= \$980$$

C. *Piers and Foundations:*

$$\Delta C = (\gamma_p - 1)QU$$

Cost of increase in quantity of concrete:

$$\Delta C = 0.27 \times 11.3 \times 79 = \$241$$

Cost of increase in quantity of steel:

$$\Delta C = 0.27 \times 1{,}872 \times 0.14 = \$71$$

Step 8: Total increase in cost due to new loading.

$$\Sigma\Delta C = 980 + 241 + 71 = \$1{,}292$$

3. Rolled Steel Girder Bridges, Noncomposite Design

Average span assumed = 75 ft c/c.

Step 1: Load parameters for HS15-44 loading.

$$\left.\begin{aligned} \epsilon &= 0.556 \\ \mu &= 0.167 \\ \delta &= 0.334 \\ B_m &= 30.2 \text{ ft} \\ B_v &= 18.7 \text{ ft} \end{aligned}\right\} \text{Same as box girder.}$$

$$M_L = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right)$$
$$= \frac{54 \times 75}{4}\left(1 - \frac{30.2}{2 \times 75}\right) = 809 \text{ ft-kips}$$

$$V_L = W\left(1 - \frac{B_v}{2_L}\right) = 54\left(1 - \frac{18.7}{2 \times 75}\right) = 47.2 \text{ kips}$$

Step 2: Same as in steel welded composite girder.

Step 3: Load parameters for Type II loading.

$$\left.\begin{aligned} \epsilon &= 0.557 \\ \mu &= -0.016 \\ \delta &= 0.500 \\ B_m &= 69.1 \text{ ft} \\ B_v &= 62.0 \text{ ft} \end{aligned}\right\} \text{Same as welded steel girder (composite).}$$

$$\overline{M}_L^{\text{II}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right)$$
$$= \frac{132 \times 75}{4}\left(1 - \frac{69.1}{2 \times 75}\right) = 1{,}335 \text{ ft-kips}$$

$$\overline{V}_L^{\text{II}} = W\left(1 - \frac{B_v}{2L}\right) = 132\left(1 - \frac{62.0}{2 \times 75}\right) = 77.5 \text{ kips}$$

Step 4: Live load ratios.

$$\alpha_m = \frac{\overline{M}_L^{\text{II}}}{M_L} = \frac{1{,}335}{809} = 1.65$$

$$\alpha_v = \frac{\overline{V}_L^{\text{II}}}{V_L} = \frac{77.5}{47.2} = 1.64$$

Step 5: Stress factor.

A. *Deck Slab:*

$$\Omega_m{}^S = 0.167$$

B. *Beams:*

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \beta_m} = \frac{1.65 - 1}{1 + 0.89} = 0.344$$

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + \beta_v} = \frac{1.64 - 1}{1 + 0.81} = 0.354$$

in which

$\beta_m = 0.89$ (from Fig. B-7)

$\beta_v = 0.81$ (from Fig. B-8)

C. *Piers and Foundations (Substructure):*

$$\Omega_p = \Omega_f < \Omega_m{}^G \quad \text{and} \quad < \Omega_v{}^G$$

Step 6: Material quantity ratio.

A. *Deck Slab:* No change in material quantity.

B. *Beams:*

$$\gamma_{sg} = ½[(1 + \Omega_v{}^G) + (1 + \Omega_m{}^G)^{\frac{1}{2}}]$$
$$= ½[1.354 + (1.344)^{\frac{1}{2}}]$$
$$= 1.26$$

C. *Piers and Foundations (Substructure):*

$$\gamma_p = \gamma_f = \gamma_{sg} = 1.26$$

Step 7: Increase in cost.

A. *Deck Slab:* No change in cost.

B. *Beams:* Cost of increase in structural steel in superstructure:

$$\Delta C = (\gamma_{sg} - 1)QU$$
$$= 0.26 \times 14{,}100 \times 0.27$$
$$= \$992$$

C. *Piers and Foundations (Substructure):* Cost of increase in quantity of concrete:

$$\Delta C = (\gamma_p - 1)QU$$
$$= 0.26 \times 8.3 \times 79.0$$
$$= \$170$$

Cost of increase in quantity of steel:

$$\Delta C = (\gamma_p - 1)QU$$
$$= 0.26 \times 1{,}370 \times 0.14$$
$$= \$50$$

Step 8: Total increase in cost due to new loading.

$$\Sigma\Delta C = 992 + 170 + 50 = \$1{,}212$$

4. Prestressed Concrete Girder Bridges

Average span assumed = 70 ft c/c.

Step 1: Load parameters for HS15-44 loading.

$$\left.\begin{aligned} \epsilon &= 0.556 \\ \mu &= 0.167 \\ \delta &= 0.334 \\ B_m &= 30.2 \text{ ft} \\ B_v &= 18.7 \text{ ft} \end{aligned}\right\} \text{Same as box girder.}$$

$$M_L = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{54 \times 70}{4}\left(1 - \frac{30.2}{2 \times 70}\right) = 741 \text{ ft-kips}$$

$$V_L = W\left(1 - \frac{B_v}{2L}\right) = 54\left(1 - \frac{18.7}{2 \times 70}\right) = 46.8 \text{ kips}$$

Step 2: Same as in steel welded composite girder.

Step 3: Load parameters for Type II loading.

$$\left.\begin{aligned} \delta &= 0.557 \\ \mu &= -0.016 \\ \delta &= 0.500 \\ B_m &= 69.1 \text{ ft} \\ B_v &= 62.0 \text{ ft} \end{aligned}\right\} \text{Same as welded steel girder (composite).}$$

$$\overline{M}_L{}^{\text{II}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right)$$

$$= \frac{132 \times 70}{4}\left(1 - \frac{69.1}{2 \times 70}\right) = 1{,}170 \text{ ft-kips}$$

$$\overline{V}_L{}^{\text{II}} = W\left(1 - \frac{B_v}{2L}\right) = 132\left(1 - \frac{62.0}{2 \times 70}\right) = 73.6 \text{ kips}$$

Step 4: Live load ratios.

$$\alpha_m = \frac{\overline{M}_L{}^{\text{II}}}{M_L} = \frac{1{,}170}{741} = 1.58$$

$$\alpha_v = \frac{\overline{V}_L{}^{\text{II}}}{V_L} = \frac{73.6}{46.8} = 1.57$$

Step 5: Stress factor.

A. *Deck Slab:*

$$\Omega_m{}^S = 0.167$$

B. *Beams:*

$$\Omega_m{}^G = 0.60(\alpha_m - 1) = 0.60(1.58 - 1) = 0.348$$

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + 0.6\beta_v} = \frac{1.57 - 1.0}{1 + 0.6 \times 1.7} = 0.282$$

in which $\beta_v = 1.7$ (from Fig. B-11)

C. *Piers and Foundations (Substructure):*

$$\Omega_p = \Omega_f < \Omega_m{}^G$$
$$\text{and} \quad < \Omega_v{}^G$$

Step 6: Material quantity ratio.

A. *Deck Slab:* No change in material quantity.

B. *Beams:*

$$\gamma_{sg} = \tfrac{1}{2}[1.0 + (1 + \Omega_m{}^G)^{\frac{1}{2}}]$$
$$= \tfrac{1}{2}[1.0 + (1.348)^{\frac{1}{2}}]$$
$$= 1.08$$

C. *Piers and Foundations (Substructure):*

$$\gamma_p = \gamma_f = \gamma_{sg} = 1.08$$

Step 7: Increase in cost.

A. *Deck Slab:* No change in cost.

B. *Beams:* Cost of increase in concrete:

$$\Delta C = (\gamma_{sg} - 1)QU$$
$$= 0.08 \times 3.33 \times 90.0$$
$$= \$24.00$$

Cost of increase in re-bar steel:

$$\Delta C = (\gamma_{sg} - 1)QU$$
$$= 0.08 \times 360 \times 0.18$$
$$= \$5.19$$

Cost of increase in prestressed steel:

$$\Delta C = (\gamma_{sg} - 1)QU$$
$$= 0.08 \times 360 \times 0.55$$
$$= \$15.85$$

C. *Piers and Foundations (Substructure):* Cost of increase in concrete:

$$\Delta C = (\gamma_p - 1)QU$$
$$= 0.08 \times 4.32 \times 79$$
$$= \$27.30$$

Cost of increase in re-bar steel:

$$\Delta C = (\gamma_p - 1)QU$$
$$= 0.08 \times 712 \times 014$$
$$= \$7.96$$

Step 8: Total increase in cost of bridge due to new loading:

$$\Sigma\Delta C = 24.00 + 5.19 + 15.85 + 27.30 + 7.96$$
$$= \$80.30$$

5. T-Beam Bridges

Average span assumed = 50 ft c/c.

Step 1: Load parameters for HS15-44 loading.

$$\left.\begin{aligned} \epsilon &= 0.556 \\ \mu &= 0.167 \\ \delta &= 0.334 \\ B_m &= 30.2 \text{ ft} \\ B_v &= 18.7 \text{ ft} \end{aligned}\right\} \text{Same as box girder.}$$

$$M_L = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{54 \times 50}{4}\left(1 - \frac{30.2}{2 \times 50}\right) = 471 \text{ ft-kips}$$

$$V_L = W\left(1 - \frac{B_v}{2L}\right) = 54\left(1 - \frac{18.7}{2 \times 50}\right) = 49.4 \text{ kips}$$

Step 2: Because the length of the axle base is less than the span length, the load parameters are the same as in box girder for Type I new loading.

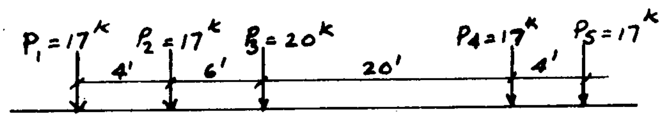
$$\epsilon = 0.66$$

$$\mu = 0.204$$
$$\delta = 0.450$$
$$B_m = 60.6 \text{ ft}$$
$$B_v = 44.1 \text{ ft}$$

$$\overline{M}_L{}^{\text{I}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{78 \times 50}{4}\left(1 - \frac{60.6}{2 \times 50}\right) = 384 \text{ ft-kips}$$

$$\overline{V}_L{}^{\text{I}} = W\left(1 - \frac{B_v}{2L}\right) = 78\left(1 - \frac{44.1}{2 \times 50}\right) = 43.6 \text{ kips}$$

Step 3: Load parameters for Type II loading: Because the length of axle base is greater than the span length, the partial axle configuration that causes maximum moment must be identified. In the problem the partial axle configuration that causes maximum action is as shown.



$\bar{x}$ = distance of center of gravity of loads from load P_5

$$= \frac{17(34 + 30 + 24) + 20 \times 24}{(17 + 17 + 20 + 17 + 17)} = 18.6 \text{ ft}$$

Therefore,

$x_1 = -10$ ft; $x_2 = -6$ ft; $x_3 = 0$ ft; $x_4 = +20$ ft; $x_5 = +24$ ft

$z_1 = 34$ ft; $z_2 = 30$ ft; $z_3 = 24$ ft; $z_4 = 4.0$ ft; $z_5 = 0$ ft

$b = 34$ ft

$$W = \sum_1^5 P_i = 88 \text{ kips}$$

$$\epsilon = 2\left(\frac{\sum_1^5 P_i|x_i|}{Wb}\right)$$

$$= 2\left[\frac{17(10 + 6 + 20 + 24) + 20 \times 0}{88 \times 34}\right] = 0.681$$

$$\mu = \frac{\sum_1^5 P_i x_i}{Wb} = \frac{17(20 + 24 - 10 - 6)}{88 \times 34} = 0.159$$

$$\delta = \frac{\sum_1^5 P_i z_i}{Wb} = \frac{17(34 + 30 + 4) + 20 \times 24.0}{88 \times 34} = 0.546$$

$$B_m = 2[\epsilon - (b/L)\mu^2]b = 2 \times 0.664 \times 34 = 45.1 \text{ ft}$$

$$B_v = 2\delta b = 2 \times 0.546 \times 34 = 37.2 \text{ ft}$$

$$\overline{M}_L{}^{\text{II}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{88 \times 50}{4}\left(1 - \frac{45.1}{2 \times 50}\right) = 605 \text{ ft-kips}$$

$$\overline{V}_L{}^{\text{II}} = W\left(1 - \frac{B_v}{2L}\right) = 88\left(1 - \frac{37.2}{2 \times 50}\right) = 55.3 \text{ kips}$$

Because $\overline{M}_L{}^{\text{II}} > \overline{M}_L{}^{\text{I}}$ and $\overline{V}_L{}^{\text{II}} > \overline{V}_L{}^{\text{I}}$, Type II loading is more severe. Therefore, Type II loading is considered.

Step 4: Live load ratios.

$$\alpha_m = \frac{\overline{M}_L{}^{\text{II}}}{M_L} = \frac{605}{471} = 1.28$$

$$\alpha_v = \frac{\overline{V}_L{}^{\text{II}}}{V_L} = \frac{55.3}{49.4} = 1.12$$

Step 5: Stress factor.

A. *Deck Slab:*

$$\Omega_s = 0.167$$

B. *Beams:*

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \beta_m} = \frac{1.28 - 1}{1 + 0.97} = 0.142$$

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + \beta_v} = \frac{1.12 - 1}{1 + 1.05} = 0.059$$

in which

$\beta_m = 0.97$ (from Fig. B-9)

$\beta_v = 1.05$ (from Fig. B-10)

C. *Piers and Foundations (Substructure):*

$$\Omega_p = \Omega_f < \Omega_m{}^G \text{ and } < \Omega_v{}^G$$

Step 6: Material quantity ratio.

A. *Deck Slab:*

$$\gamma_s = 0.914; \text{ use } 1.0$$

No change in material quantities.

B. *Beams:*

$$\gamma_{sg} = \tfrac{1}{2}[(1 + \Omega_v{}^G) + (1 + \Omega_m{}^G)^{\frac{1}{2}}]$$
$$= \tfrac{1}{2}[1.059 + (1.142)^{\frac{1}{2}}]$$
$$= 1.06$$

C. *Piers and Foundations (Substructure):*

$$\gamma_p = \gamma_f = \gamma_{sg} = 1.06$$

Step 7: Increase in cost.

A. *Deck Slab:* No change in cost.

B. *Beams:* Cost of increase in concrete:

$$\Delta C = (\gamma_{sg} - 1)QU$$
$$= 0.06 \times 6.9 \times 79$$
$$= \$32.70$$

Cost of increase in steel:

$$\Delta C = 0.06 \times 2{,}210 \times 0.14$$
$$= \$18.60$$

C. *Piers and Foundations (Substructure):* Cost of increase in concrete:

$$\Delta C = 0.06 \times 9.5 \times 79$$
$$= \$45.00$$

Cost of increase in steel:

$$\Delta C = 0.06 \times 1{,}890 \times 0.14$$
$$= \$15.90$$

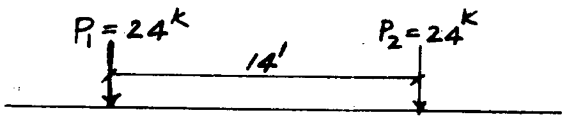
Step 8: Total cost increase due to new loading.

$$\Sigma\Delta C = 32.70 + 18.60 + 45.00 + 15.90 = \$112.20$$

6. Flat Slab Bridges

With 18-in.-square prestressed concrete piling as substructure. Average span assumed = 26 ft c/c.

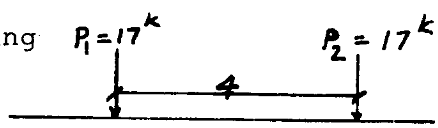
Step 1: Load parameters for HS15-44 loading. Because the length of axle base is longer than the span length, the axle configuration that causes maximum action is as shown.



$$x_1 = 0 \text{ ft};\ x_2 = 14 \text{ ft}$$
$$z_1 = 14 \text{ ft};\ z_2 = 0 \text{ ft}$$
$$b = 14 \text{ ft}$$
$$W = \sum_1^2 P_i = 24 + 24 = 48 \text{ kips}$$
$$\epsilon = 2\left(\frac{\sum_1^2 P_i|x_i|}{Wb}\right) = 2\left(\frac{24 \times 14}{48 \times 14}\right) = 1.0$$
$$\mu = \frac{\sum_1^2 P_i x_i}{Wb} = \frac{24 \times 14}{48 \times 14} = 0.5$$
$$\delta = \frac{\sum_1^2 P_i z_i}{Wb} = \frac{24 \times 14}{48 \times 14} = 0.5$$
$$B_m = 2[\epsilon - (b/L)\mu^2]b = 2 \times 0.825 \times 14 = 23.1 \text{ ft}$$
$$B_v = 2\delta b = 2 \times 0.5 \times 14 = 14.0 \text{ ft}$$
$$M_L = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{48 \times 26}{4}\left(1 - \frac{23.1}{2 \times 26}\right) = 174.0 \text{ ft-kips}$$
$$V_L = W\left(1 - \frac{B_v}{2L}\right) = 48\left(1 - \frac{14.0}{2 \times 26}\right) = 35.0 \text{ kips}$$

Step 2: Because the span length is shorter than the axle base length, the critical axle configuration is as shown.

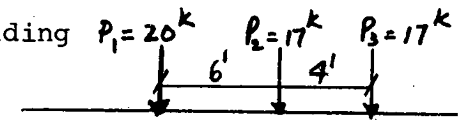
Type I loading



$$x_1 = 0 \text{ ft};\ x_2 = 4 \text{ ft}$$
$$z_1 = 4 \text{ ft};\ z_2 = 0 \text{ ft}$$
$$b = 4 \text{ ft}$$
$$W = \sum_1^2 P_i = 17 + 17 = 34 \text{ kips}$$
$$\epsilon = 2\left(\frac{\sum_1^2 P_i|x_i|}{Wb}\right) = 2\left(\frac{17 \times 4}{34 \times 4}\right) = 1.0$$
$$\mu = \frac{\sum_1^2 P_i x_i}{Wb} = \frac{4 \times 17}{34 \times 4} = 0.5$$
$$\delta = \frac{\sum_1^2 P_i z_i}{Wb} = \frac{17 \times 4}{34 \times 4} = 0.5$$
$$B_m = 2[\epsilon - (b/L)\mu^2]b = 7.7 \text{ ft}$$
$$B_v = 2\delta b = 4.0 \text{ ft}$$
$$\overline{M}_L{}^{\text{I}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{34 \times 26}{4}\left(1 - \frac{7.7}{2 \times 26}\right) = 188 \text{ ft-kips}$$
$$\overline{V}_L{}^{\text{I}} = W\left(1 - \frac{B_v}{2L}\right) = 34\left(1 - \frac{4}{2 \times 26}\right) = 31.4 \text{ kips}$$

Step 3: Because the span length is shorter than the axle base length, the critical axle configuration is as shown.

Type II loading



$$\bar{x} = \text{distance of center of gravity of loads from load } P_3$$
$$= \frac{20 \times 10 + 17 \times 4}{20 + 17 + 17} = 4.96 \text{ ft}$$

Therefore,

$$x_1 = -6 \text{ ft};\ x_2 = 0 \text{ ft};\ x_3 = 4 \text{ ft}$$
$$z_1 = 10 \text{ ft};\ z_2 = 4 \text{ ft};\ z_3 = 0 \text{ ft}$$
$$b = 10 \text{ ft}$$
$$W = \sum_1^3 P_i = 54 \text{ kips}$$
$$\epsilon = 2\left(\frac{\sum_1^3 P_i|x_i|}{Wb}\right) = 2\left(\frac{20 \times 6 + 17 \times 4}{54 \times 10}\right) = 0.696$$
$$\mu = \frac{\sum_1^3 P_i x_i}{Wb} = \frac{-20 \times 6 + 17 \times 4}{54 \times 10} = -0.096$$
$$\delta = \frac{\sum_1^3 P_i z_i}{W_b} = \frac{10 \times 20 + 17 \times 4}{54 \times 10} = 0.496$$
$$B_m = 2[\epsilon - (b/L)\mu^2]b = 2 \times 0.696 \times 10 = 13.92 \text{ ft}$$
$$B_v = 2\delta b = 2 \times 0.496 \times 10 = 9.92 \text{ ft}$$
$$\overline{M}_L{}^{\text{II}} = \frac{WL}{4}\left(1 - \frac{B_m}{2L}\right) = \frac{54 \times 26}{4}\left(1 - \frac{13.92}{2 \times 26}\right) = 257 \text{ ft-kips}$$
$$\overline{V}_L{}^{\text{II}} = W\left(1 - \frac{B_v}{2L}\right) = 54\left(1 - \frac{9.92}{2 \times 26}\right) = 43.6 \text{ ft-kips}$$

Because $\overline{M}_L{}^{\text{II}} > \overline{M}_L{}^{\text{I}}$ and $\overline{V}_L{}^{\text{II}} > \overline{V}_L{}^{\text{I}}$, Type II loading is more severe. Therefore, Type II loading is considered.

Step 4: Live load ratios.

$$\alpha_m = \frac{\overline{M}_L{}^{\text{II}}}{M_L} = \frac{257}{174} = 1.47$$
$$\alpha_v = \frac{\overline{V}_L{}^{\text{II}}}{V_L} = \frac{43.6}{35.0} = 1.248$$

Step 5: Stress factor.

A. *Deck Slab:*

$$\Omega_m{}^G = \frac{\alpha_m - 1}{1 + \beta_m} = \frac{1.47 - 1}{1 + 0.91} = 0.246$$

$$\Omega_v{}^G = \frac{\alpha_v - 1}{1 + \beta_v} = \frac{1.248 - 1}{1 + 0.6} = 0.155$$

in which

$\beta_m = 0.91$ (from Fig. B-3)

$\beta_v = 0.60$ (from Fig. B-4)

B. *Piers and Foundations (Substructure)*:

$$\Omega_p = \Omega_f < \Omega_m{}^G \quad \text{and} \quad < \Omega_v{}^G$$

Step 6: Material quantity ratio.

A. *Deck Slab:*

$$\gamma_s = (1 + \Omega_m{}^G)^{\frac{1}{2}} = (1 + 0.246)^{\frac{1}{2}} = 1.11$$

B. *Piers and Foundations (Substructure)*:

$$\gamma_p = \gamma_f = \gamma_s = 1.11$$

Step 7: Increase in cost.

A. *Deck Slab:*

$$\Delta C = (\gamma_s - 1)QU$$
$$= 0.11 \times 3.57 \times 79$$
$$= \$31.00$$

Cost of increase in steel quantity:

$$\Delta C = (\gamma_s - 1)QU$$
$$= 0.11 \times 954 \times 0.14$$
$$= \$14.70$$

B. *Substructure:* Cost of increase in piles on which the flat slab is supported:

$$\Delta C = (\gamma_p - 1)QU$$
$$= 0.11 \times 25.2 \times 15.00$$
$$= \$41.40$$

Step 8: Total cost increase due to new loading.

$$\Sigma\Delta C = 31.00 + 14.70 + 41.40$$
$$= \$117.10$$

Summary

In this example it is assumed that maintenance costs of new structures designed to accommodate new loads will be insignificantly different from current structure maintenance costs. Therefore, no cost increment is shown. See Chapter Two.

Total cost increase due to new loads per mile of highway assumed:

$$C = 316.20 + 1{,}292.00 + 1{,}212.00 + 80.30 + 112.20 + 117.10$$
$$= 3{,}129.80; \text{ say } \$3{,}130/\text{mile of highway.}$$

CALCULATION OF TRANSPORTATION ECONOMY OF PROPOSED LEGAL AXLE WEIGHT LIMITS RELATED TO EXISTING LEGAL LIMITS

This calculation is applied to one mile of new highway, taking into account the incremental highway and trucking costs and expenses that are attributable to the new legal limits. The process is applied to a two-lane rural highway serving traffic in both directions. The highway costs and trucking costs that are not affected by the change in legal axle weight limits are not considered. This analysis is strictly for the purpose of determining the total transportation costs (highway costs plus motor trucking costs) under the two levels of weight limits to see which limit results in the lower total cost. This analysis does not involve future cash disbursements for remodeling, reconstructing, and maintaining the highway system, with or without any change in legal limits of dimensions and weights of vehicles.

Both the pavement and the bridge structures for the 20/34-kip designs probably will cost slightly more to maintain, year to year, than would the 18/32-kip design. This probable incremental annual expense is omitted in the following calculation of the benefit/cost ratio because supportable methods of estimating the amount of difference between the two designs for pavement and for bridges were not discovered. A comprehensive study of the problem might lead to the development of a rational method for estimating the differences in the annual maintenance expense. Research toward this end is recommended.

The procedure follows the flow indicated in Figure D-13, using the form of Table D-16.

Step 96. Transfer pavement construction costs for present and proposed limits from Step 94 and enter in line 1, Cols. B and C, Table D-16. Compute and enter difference in Col. D.

Step 97. Transfer incremental bridge construction cost per mile of highway to line 2, Col. D, Table D-16 from end of Step 95.

Step 98. Transfer total daily line-haul trucking costs for all vehicle classes in 1970 from Table D-12 to line 4, Cols. B and C, respectively. Enter difference of costs (B minus C) in Col. D.

Step 99. Transfer total daily line-haul trucking costs for all vehicle classes in 1990 from Table D-13 to line 5, Cols. B and C, respectively. Enter difference of costs in Col. D.

Step 100. In Col. D, enter total of lines 1 and 2 in line 3.

Step 101. Convert daily line-haul trucking costs to yearly costs by multiplying lines 4 and 5, Col. D, by 365, and enter results in corresponding lines 6 and 7, Col. D.

Step 102. Multiply line 3, Col. D, by the capital recovery factor of 0.1019 for 8 percent discount rate and 20 years to get the equivalent uniform annual capital cost; enter on line 8.

Step 103. Line 9 corresponds to the step required to convert a straight-line increase over time of the trucking costs 1970 to 1990 to an equivalent uniform annual series over the 20 years when applying the 8 percent time discount factor to the yearly increases.

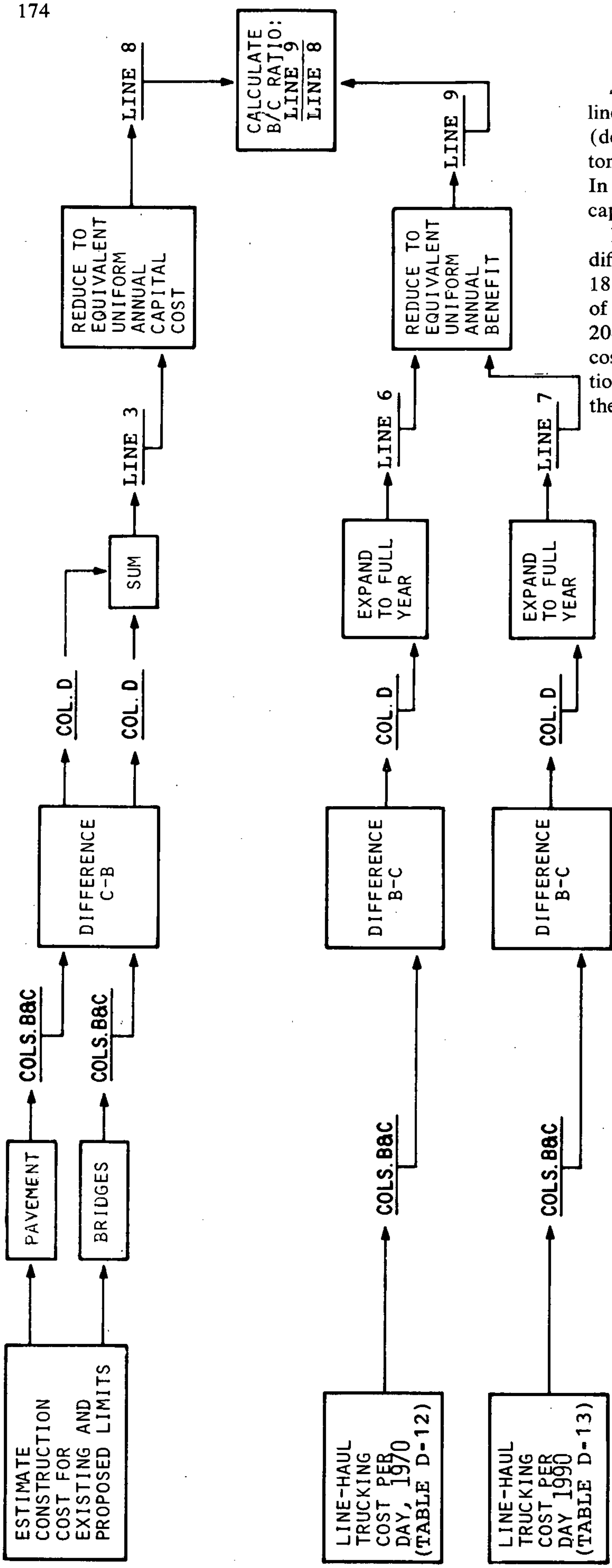


Figure D-13. Flow of procedures to perform the cost/benefit analysis.

Step 104. Calculate the benefit/cost ratio; enter on line 10. In the numerator are the benefits to trucking (decrease in truck operating cost for hauling the same total tons of cargo as before the change in legal weight limits). In the denominator is the equivalent uniform highway capital cost from line 8.

Note: This cost-benefit analysis is based solely on the differences in highway costs and trucking costs for the 18/32-kip and the 20/34-kip limits for an analysis period of 20 years. This is the incremental analysis. Over the 20-year period, the full incremental highway construction costs are absorbed by the benefits to trucking (cost reduction), although many years of service still will remain in the pavement structure, earthwork, and bridges.

TABLE D-16

COST-BENEFIT ANALYSIS BASED ON ONE MILE OF NEW HIGHWAY CONSTRUCTION—PRIMARY RURAL, TWO-LANE, BIDIRECTIONAL HIGHWAY

CONSTRUCTION OR MAINTENANCE ITEM	COST ($)		
	AXLE WEIGHT LIMIT (KIPS)		
	18/32	20/34	DIFFERENCE
A	B	C	D
1. Pavement construction cost, Step 94	90,000	94,000	4,000
2. Bridge construction cost per highway mile, Step 95			3,130
3. Total increase in construction cost			7,130
4. Line-haul trucking cost per day for 1970, Table D-12	159.84	149.11	10.73
5. Line-haul trucking cost per day for 1990, Table D-13	379.63	372.67	6.96
6. Per day 1970 trucking cost, Col. D, extended to full year (line 4×365)			3,916.45
7. Per day 1990 trucking cost, Col. D, extended to full year (line 5×365)			2,540.40
8. Equivalent uniform annual capital cost, based on a discount rate of 8 percent per annum and 20-year analysis period: (line 3, Col. D) (capital recovery factor of 0.1019)			726.55
9. Equivalent uniform annual trucking cost decrease (benefits), assuming a straight-line decrease 1970 to 1990 *			3,363.49
10. Benefit/cost ratio calculation: (line 9 divided by line 8)			4.61

* $6D + (\text{gradient series factor } 8.03695)\,\frac{(7D) - (6D)}{20}$

The factor of 8.03695 is the gradient uniform factor to convert an arithmetical (gradient) increasing factor quantity to an equivalent uniform annual amount at a discount rate of 8 percent for a time period of 20 years. See Winfrey (*85*, p. 776).

APPENDIX E

SUMMARY OF RESPONSES TO QUESTIONNAIRE TO HIGHWAY DEPARTMENTS

A questionnaire was mailed to all state highway departments or state departments of transportation to solicit certain information regarding their experiences and practices in relation to this study topic. Thirty-three replies were received. A summary of these replies follows, correlated with the original questions:

1(a) Do you have definitive evidence that an increase in axle weight legal limits would affect pavement maintenance expense?

Yes—2.

Qualified yes—15.

No—16.

Only Arkansas and Virginia have documented evidence that this is true.

Of the 15 that gave a qualified yes, only Idaho and South Dakota attempted to give written documentation of their beliefs. The rest said that they are assured that it would increase the maintenance cost, but do not have documented evidence. These are based primarily on observations in areas where much hauling is done (e.g., mining areas, rock quarries). Two states reminded the researchers that the AASHO tests in themselves are ample proof.

(*b*) *As an extension of 1(a), do you have similar data regarding bridge structure maintenance?*

Yes—4.
Qualified yes—15.
No—14.

Arkansas and Virginia answered yes, as in Ques. 1(a). Connecticut and South Dakota said they have one bridge each that shows this evidence. Connecticut presently is conducting a study on a single bridge on I-95. The qualified reasons are the same as in Ques. 1(a).

(*c*) *If requested, could these data be made available to the study group?*

Arkansas and Virginia would make their information available. Connecticut's data are not ready. South Dakota and Idaho would give limited data.

2(*a*) *What procedure and concept do you use in the structural design of pavements for new construction?*

(*b*) *If you do not use the equivalent of 18-kip axle applications to represent the traffic, as indicated in the AASHO Interim Guide for Pavement Design, please include a description of your procedure in connection with the traffic factor.*

Nine states use the AASHO Interim Guide (Arkansas, Delaware, Ohio, South Carolina, Vermont, North Dakota, South Dakota, Wisconsin, and New Mexico). The others are:

Colorado —Have own manual, but say it is in accord with AASHO.
Michigan —Experience, checked by AASHO.
Mississippi —Own charts, based on CBR.
Missouri —Flexible pavement—charts. Rigid pavement—load distribution theory.
Nevada —Flexible pavement—AASHO Interim Guide. Rigid pavement—PCA method using *k* values of California.
Oklahoma —Own method and manual, based on ADT and average overloads per day.
Texas —Flexible pavement—Texas triaxial method. Rigid pavement—Westergaard and repetitive load method.
Virginia —Modified AASHO Interim Guide method using deflection data.
Connecticut —Experience only.
California —Own method.
Florida —AASHO Interim Guide for Interstate, and own methods for primary and secondary roads.
Illinois —Based on the AASHO Road Test using the 18-kip EWL.
Minnesota —Rigid pavement—PCA procedure. Flexible pavement—own system.
Tennessee —Experience and AASHO Interim Guide.
Idaho —Own manual using 5-kip equivalent wheel load developed by California.
Iowa —AASHO Interim Guide for flexible pavement; PCA method for rigid pavement.
New York —All available information, including AASHO Interim Guide.
Washington —California method using the 5-kip wheel load.
Oregon —California method modified for Oregon's soil, traffic, and climate.
New Jersey —Flexible pavement—AASHO Interim Guide. Rigid pavement—experience only.
Maine —AASHO Interim Guide, with modifications.
Montana —Flexible pavement—18-kip EWL and traffic index. Rigid pavement—PCA method and AASHO Interim Guide.
Nebraska —Modified AASHO Interim Guide.
Utah —AASHO Interim Guide with 18-kip EWL.

3(*a*) *If there were an appreciable increase in legal limits of axle weights or gross vehicle weights, would your procedure, described in 2(b) above, still be applicable for pavement design?*

Yes—10.

Six did not give an answer. Seventeen said that they would have to modify their procedures or reevaluate their entire concept.

4(*a*) *If there were an appreciable increase in legal limits of axle weights or gross vehicle weights, how would you design the overlay (especially its thickness) for strengthening existing pavements?*

Arkansas —Use traffic analyses to determine a new SN minus the old SN equals the overlay needed.
Colorado —A series of graphs that assumes 1 in. of bituminous surface is equal to 3 in. of gravel.
Delaware —Experience and existing conditions.
Michigan —Experience and judgment.
Mississippi —Method by DOT that resulted from AASHO Road Tests.
Nevada —Reevaluate entire concept.
Ohio —AASHO Interim Guide, using appropriate SN.
Oklahoma —Modify *Hwy. Res. Record No. 254.*
South Carolina —Use AASHO Interim Guide and depreciate pavement by age.
Texas —PCA using 18-kip single-axle equivalents.
Vermont —AASHO and Benkelman Beam method.
Virginia —Experience, funds, and visual need.
North Dakota —Determine increase in SN by equivalent axle load method.
Connecticut —Experience only.
California —Revise own method after loadometer surveys.
Florida —AASHO design modified by NCHRP Project 1-11.
Illinois —Determine need, determine the existing SN; the differences; and the overlay need.

Minnesota	—For rigid pavement, they use a minimum 3-in. overlay; flexible pavement by Benkelman Beam.
South Dakota	—AASHO Interim Guide method.
Tennessee	—Experience, guided by procedures in *Public Roads* (Dec. 1970) and *Hwy. Res. Record No. 327.*
Wisconsin	—Experience only.
Idaho	—Use increase of 5-kip wheel loads and new traffic index.
Iowa	—Modify AASHO Interim Guide.
New York	—Qualified AASHO Interim Guide and experience.
Washington	—Similar to The Asphalt Institute—reduce value of existing pavement.
Oregon	—Deflection method of California.
New Jersey	—Modified AASHO Interim Guide.
Maine	—Use AASHO Interim Guide, but increase 18-kip equivalent axle load factor.
Montana	—SN deficiency and Benkelman Beam analysis and subsoil analysis.
Nebraska	—SN reduced for age, maintenance, and present condition.
New Mexico	—AASHO Interim Guide.
Utah	—Analysis of traffic loadings and deflection equations from AASHO Road Tests.

5(*a*) *In regard to traffic data, do you perform routine truck weight (loadometer) studies?*

Yes—33.
No—0.

(*b*) *Are these data available, and for what years?*

The years that these data are available vary greatly between states. Ohio began in the 1930's; Minnesota, only in the last two years.

(*c*) *Do these data indicate stratification (percentage of trucks in mix, frequency distribution, or other criteria) of vehicles by weight, dimension, configuration, and by highway classification?*

Yes—33.
No—0.

(*d*) *Do these data determine percentage of rated load and commodity carried?*

Yes—10.
No—6.

Seven states showed the commodity but did not show the percentage of rated load. One state carried percentage of rated load but not the commodity. The rest showed only averages or the commodities on those loads in excess of state law.

6(*a*) *Are vehicle weight and dimension limitations in your state applicable to all highway routes and systems uniformly? (Exclude from consideration posted bridges and temporary or seasonal restrictions such as during spring thaw.)*

Yes—18.
No—15.

(*b*) *If "No," please indicate elements which differ, by systems or classes of routes.*

Arkansas	—73,280 lb on Interstate and most of primary routes. All other routes, 64,000 lb.
Michigan	—65-ft length on designated routes only.
Mississippi	—Designated routes.
Missouri	—64,650 lb on supplementary roads and posted bridges; 73,280 lb on other roads.
Nevada	—Oversize or overweight use designated routes.
South Carolina	—Heavy loads allowed on primary routes and on Interstate.
Texas	—Secondary routes less weight.
Vermont	—State system higher than town system.
Connecticut	—Parkways prohibit commercial vehicles.
Wisconsin	—Size uniform, weights on "B" roads allow only 60 percent of weights allowed on "A" roads.
New York	—Throughway and streets in New York City differ from rest of system.
Oregon	—Semi-trucks on Class 1 highways allow 60-ft length and 76,000 lb. Semi-trucks on Class 2 highways allow 50-ft length and 73,280 lb. 18 kip and 32 kip allowed on Interstate; 20 kip and 34 kip on other roads.
Maine	—Interstate allows 8-ft width; all other roads allow 8-ft 6-in. width.
Montana	—Restricted permits on all roads other than Interstate.
Nebraska	—Interstate allows 73,280 lb, other roads allow 95,000 lb on 7 axles.
New Mexico	—Designated routes for oversize and overweight permits.

(*c*) *If "Yes," do you feel that it would be practicable, from an administration and enforcement standpoint, to have different limits on different systems or routes, such as Interstate, primary, or secondary?*

Yes—3.
No—20.

The rest gave no answer.

7(*a*) *Has your state conducted so-called "road life" studies, and developed survivor curves for various surface types for use in planning future programs based on related retirements and replacement requirements?*

Yes—18.
No—15.

Four of the 18 states have not developed survivor curves.

(*b*) *If "Yes," are road life data (construction and retirements) up-to-date?*

Yes—7.
No—11.

The rest did not answer.

8(a) Has your state conducted pavement evaluation surveys for determination of the present serviceability index (PSI) section by section?

Yes—21.
No—12.

Only Michigan and North Dakota have completed all systems. Six states have done all of the Interstate System; four states, all of the primary system; and the rest, only parts of the systems.

(b) If "Yes," in what year was last updating completed?

Of those answering "Yes," six states are not up-to-date.

Only Nevada, South Carolina, New Jersey, and Mississippi have made any special studies during the past 15 years relative to the possible increase in legal weights and dimensions of vehicles and their economic and administrative impact on highway administration.

APPENDIX F

COMPUTATION OF ACCIDENT INVOLVEMENT RATE VS TRUCK SPEED DIFFERENTIAL

The format of the computation of accident involvement rate vs truck speed differential is given in Table F-1. Col. 1 gives the average speed of all vehicles on highways in the state obtained from planning study data. The percentage of trucks, Col. 5, relates to the truck speed categories, Col. 2, from speed studies for all highways. Col. 3 gives the midpoint speed within the speed categories; it is used to develop the speed difference, Col. 4, by subtracting the midpoint speed from the average speed.

From the involvement/speed differential curve, given in Chapter Two, the involvement rate from the curve is entered in Col. 6 for each speed differential in Col. 4. The product of data in Cols. 5 and 6 is entered in Col. 7. Col. 7 is summed to produce the total accident involvement rate

TABLE F-1

INVOLVEMENT RATE OF 4-AXLE TRUCKS ON LEVEL GRADES

1 * AVERAGE SPEED	2 * TRUCK SPEED CATEGORIES	3 * MID-POINT	4 * DIFFERENCE FROM AVERAGE	5 * PERCENT OF TOTAL 4-AXLE TRUCKS	6 * INVOLVEMENT RATE	7 * PRODUCT 5×6
59.4	30–34.9	32.5	−26.9	0.9	2,270	2,493
	35–39.9	37.5	−21.9	3.9	1,080	4,212
	40–44.9	42.5	−16.9	6.1	480	2,928
	45–49.9	47.5	−11.9	18.3	270	4,941
	50–54.9	52.5	−6.9	19.8	180	3,564
	55–59.9	57.5	−1.9	37.4	135	5,049
	60–64.9	62.5	+3.1	10.0	110	1,100
	65–69.9	67.5	+8.1	3.4	118	401
	70–75.0	72.5	+13.1	0.2	148	30
				100.0		24,718

$$\text{Involvement Rate} = \frac{24,718}{100} = 247$$

1 * 1968 average speed of all vehicles on highways in Texas; obtained from the Texas Highway Department's Planning Survey Division.
2 * Truck speed categories as established by the THD's Planning Survey Division.
3 * Midpoint of each truck speed category.
4 * Difference of average truck speed from average speed, 1 minus 3.
5 * Percentage of total 4-axle trucks in each speed category as determined by the THD's Planning Survey Division.
6 * Involvement rate taken from Figure 15 (of *113*).
7 * Product of the percentage of total 4-axle trucks and the involvement rate for the speed differential for each speed category, 5 times 6.
Source: Glennon and Joyner (*113*).

TABLE F-2

INVOLVEMENT RATE OF 4-AXLE TRUCKS WITH AN ASSUMED SPEED REDUCTION ON GRADES OF 5 MPH BELOW THE SPEED ON LEVEL GRADES

1 * AVERAGE SPEED	2 * TRUCK SPEED CATEGORIES	3 * MIDPOINT	4 * DIFFERENCE FROM AVERAGE	5 * PERCENT OF TOTAL 4-AXLE TRUCKS	6 * INVOLVE-MENT RATE	7 * PRODUCT 5×6
57.9	22–27	24.5	−33.4	0.9	13,000	11,700
	28–33	30.5	−27.4	3.9	3,100	12,090
	34–39	36.5	−21.4	6.1	950	5,795
	40–45	42.5	−15.4	18.3	400	7,320
	46–51	48.5	−9.4	19.8	215	4,257
	52–57	54.5	−3.4	37.4	145	5,423
	58–63	60.5	+2.6	10.0	110	1,110
	64–69	66.5	+8.6	3.4	120	408
	70–75	72.5	+14.6	0.2	160	32
				100.0		48,135

$$\text{Involvement Rate} = \frac{48{,}135}{100} = 481$$

1 * Average speed of all vehicles on level grades less 30% of assumed reduction in truck speed on grades; 59.4 — (0.3) (5) = 57.9.
2 * Truck speed categories determined by subtracting the assumed truck speed reduction found in Table C-2 (of *113*) from the speed categories established by the THD's Planning Survey Division.
3 * Midpoint of each truck speed category.
4 * Difference of average truck speed from average speed, 1 minus 3.
5 * Percentage of total 4-axle trucks in each speed category as determined by the THD's Planning Survey Division.
6 * Involvement rate taken from Figure 15 (of *113*).
7 * Product of the percentage of total 4-axle trucks and the involvement rate for the speed differential for each speed category, 5 times 6.
Source: Glennon and Joyer (*113*).

TABLE F-3

INVOLVEMENT RATE OF 4-AXLE TRUCKS WITH AN ASSUMED SPEED REDUCTION ON GRADES OF 10 MPH BELOW THE SPEED ON LEVEL GRADES

1 * AVERAGE SPEED	2 * TRUCK SPEED CATEGORIES	3 * MID-POINT	4 * DIFFERENCE FROM AVERAGE	5 * PERCENT OF TOTAL 4-AXLE TRUCKS	6 * INVOLVE-MENT RATE	7 * PRODUCT 5×6
56.4	17–22	19.5	−36.9	0.9	32,000	28,800
	23–28	25.5	−30.9	3.9	6,800	26,520
	29–34	31.5	−24.9	6.1	1,850	11,285
	35–40	37.5	−18.9	18.3	640	11,712
	42–47	44.5	−11.9	19.8	270	5,346
	49–54	51.5	−4.9	37.4	160	5,984
	56–61	58.5	+2.1	10.0	115	1,150
	63–68	65.5	+10.1	3.4	125	425
	70–75	72.5	+16.1	0.2	180	36
				100.0		91,258

$$\text{Involvement Rate} = \frac{91{,}258}{100} = 913$$

1 * Average speed of all vehicles on level grades less 30% of assumed reduction in truck speed on grades; 59.4 — (0.3) (10) = 56.4.
2 * Truck speed categories determined by subtracting the assumed truck speed reduction found in Table C-2 (of *113*) from the speed categories established by the THD's Planning Survey Division.
3 * Midpoint of each truck category.
4 * Difference of truck speed from average speed, 1 minus 3.
5 * Percentage of total 4-axle trucks in each speed category as determined by the THD's Planning Survey Division.
6 * Involvement rate taken from Figure 15 (of *113*).
7 * Product of the percentage of total 4-axle trucks and the involvement rate for the speed differential for each speed category, 5 times 6.
Source: Glennon and Joyner (*113*).

for all trucks under the assumed condition. The involvement rate is therefore this value divided by 100 (the total percentage of trucks).

This procedure is repeated in Tables F-2 and F-3 for other incremental reductions in truck and average vehicle speeds. Adjustments in truck speed categories are made by subtracting from the level grades, Col. 2, Table F-2, the assumed truck speed reductions.

APPENDIX G

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