ATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT **86**

TENTATIVE SERVICE REQUIREMENTS FOR BRIDGE RAIL SYSTEMS

GHWAY RESEARCH BOARD
ATIONAL RESEARCH COUNCIL
ATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT **86**

TENTATIVE SERVICE REQUIREMENTS FOR BRIDGE RAIL SYSTEMS

ROBERT M. OLSON, EDWARD R. POST,
AND WILLIAM F. McFARLAND
TEXAS A & M UNIVERSITY
COLLEGE STATION, TEXAS

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
OF STATE HIGHWAY OFFICIALS IN COOPERATION
WITH THE BUREAU OF PUBLIC ROADS

SUBJECT CLASSIFICATION:
HIGHWAY DESIGN
BRIDGE DESIGN
HIGHWAY SAFETY

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1970

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 Highway Planning and Research funds from participating member states of the Association and it receives the full cooperation and support of the Bureau of Public Roads, 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, non-profit 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.

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the U. S. Bureau of Public Roads. Individual fiscal agreements are executed annually by the Academy-Research Council, the Bureau of Public Roads, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract. It has been accepted by the Highway Research Board and published in the interest of an effectual dissemination of findings and their application in the formulation of policies, procedures, and practices in the subject problem area.

The opinions and conclusions expressed or implied in these reports are those of the research agencies that performed the research. They are not necessarily those of the Highway Research Board, the National Academy of Sciences, the Bureau of Public Roads, the American Association of State Highway Officials, nor of the individual states participating in the Program.

NCHRP Project 12-8 FY '66
SBN 309-01784-X
Library of Congress Catalog Number: 70-605165

FOREWORD

By Staff

Highway Research Board

This report presents the current state of the knowledge with respect to service requirements for bridge rail systems. It should be of special interest and use to design engineers, safety engineers, and others concerned with effective bridge rail systems. The researchers performed a thorough literature search of the subject and conducted detailed interviews with representatives of highway departments of nine states as a basis for compiling a list of service requirements and formulating specific recommendations for revision of sections of the *AASHO Standard Specifications for Highway Bridges.* Suggestions are also made for the additional research necessary to permit the complete development of design criteria for bridge rail systems.

Highway bridge railing system designs have evolved through need and experience, but have often been based on questionable design information. In recent years, additional information has been provided by the many full-scale crash tests on bridge railings. There is a need, then, to assemble and correlate the information generally accepted as valid for the purpose of outlining bridge railing service requirements. It is of prime importance to delineate the functions that railings are expected to satisfy for various site conditions, with due consideration being given to safety, economy and appearance. Following the achievement of a valid definition of service requirements, existing and new research data can be used to formulate comprehensive design criteria which will include various configurations and materials.

In a one-year study, the objective of the Texas Transportation Institute was to define service requirements for bridge rail systems, as extensively as possible, as a preliminary step toward developing design criteria. This report, then, is on what might be termed a pilot study intended to ascertain the state of the art and pinpoint gaps in the knowledge concerning bridge railings. However, the pilot study resulted in much more than that—in fact, TTI was able to produce a simple mathematical model to predict the reactions of a vehicle-guardrail collision. The investigators were also successful in finding evidence that relates vehicle deceleration rate to occupant safety; vehicle damage to deceleration rate, hence to potential for occupant injury; and the vehicle exit angle and velocity to frequency of departure from the intended travel lanes. Furthermore, they were successful in formulating structural design criteria, including a rational technique for determining design loadings; suggesting revisions to sections of the *AASHO Standard Specifications for Highway Bridges;* and postulating a philosophy and methodology for an engineering economy approach for selection of bridge rail systems.

These research findings indicate that bridge rail technology is reasonably well advanced and that the engineer has at his disposal considerable evidence and many tools for the design of apparently less hazardous systems than are now in common use. The technology can be advanced even further through additional research to develop more information on accident statistics and cost-effectiveness data having to do with the effects of roadway widths, bridge rail designs, roadway and bridge geometrics, and traffic characteristics. Full-scale proof tests of any proposed system(s) will be required to substantiate the theoretical analyses and provide assurance of successful performance in practice. In the meantime, a major question of policy must be resolved by the administrator—that is, how much is it worth to the public to spend additional money on bridge rail systems to prevent death and injury? Until answered, the design engineer is constrained in his approach, regardless of the tools at his disposal.

CONTENTS

ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-8 and was conducted at Texas A&M University by personnel of the Texas Transportation Institute. Dr. Robert M. Olson, Associate Research Engineer, represented the Institute as principal investigator. He was assisted in the research by Edward R. Post, Assistant Research Engineer, and William F. McFarland, Assistant Research Economist.

The authors gratefully acknowledge the patience and counsel of Advisory Panel C 66 during the course of this study. Also acknowledged is the hospitable attitude and graciousness of the personnel of the nine highway departments visited. The candid interchange of ideas at meetings with these groups in California, Illinois, Minnesota, New Jersey, New York, Oregon, Texas, Virginia, and Washington is reflected in this report.

It is not possible to thank each individual who has contributed to this report; however, the help and advice of each is gratefully acknowledged.

TENTATIVE SERVICE REQUIREMENTS FOR BRIDGE RAIL SYSTEMS

SUMMARY

It is evident that many barrier rail installations constitute a hazard to a vehicle that is out of control. Examination of accident information gathered in this study shows that approximately 20 percent of fixed-object fatal accidents involve bridge barrier rail systems. Four hazardous conditions identified in this study are: (1) vehicle penetration of bridge or approach barrier rails, (2) snagging of a vehicle by components of bridge or approach barrier rails, (3) vehicle collisions with the approach end of bridge or approach barrier rails, and (4) collisions in which a vehicle is redirected by a railing system. It has become apparent in this study that it is necessary to consider the approach railing and the bridge railing as an integrated system, and the two subsystems must be compatible.

The first three hazardous conditions can be eliminated by providing adequate strength, by attention to details of design, and by providing a satisfactory transition between approach rails and bridge rails. The fourth hazardous condition thus becomes of major concern to highway engineers, and has received major emphasis in the research effort reported herein. Findings contained in this report indicate that a standard size vehicle would be subjected to an average lateral deceleration (at the center of mass of the vehicle) of 3 G's or less in approximately 85 percent of collisions. At this level of deceleration, it is demonstrated that 85 percent of accidents would be non-fatal, and 60 percent of accidents would not produce injuries to unrestrained vehicle occupants. These are reassuring indications of the adequacy of rigid barrier installations. From the viewpoint of safety, maintenance, and economy it is apparent that a properly designed rigid bridge railing system is not a hazard in the majority of collisions studied. It must be emphasized, however, that such a conclusion is based on accident information on existing installations. It must be further emphasized that such barriers are capable of producing fatalities and injuries in severe collisions. Application of information contained in this report will permit engineers to evaluate the extent to which rigid barriers may be considered safe.

A barrier rail capable of lateral displacement produces a lower impact force than a rigid barrier, with a corresponding reduction in the severity of damage to a colliding vehicle. It is demonstrated herein that barrier rail displacements of 1 to 2 ft produce substantially lower impact forces than those produced by rigid barriers. Continued efforts to develop acceptable bridge railing systems capable of lateral displacement are necessary in many installations. Design criteria for rigid and flexible railing systems can be developed by employing the rational analytical approach and other information presented in this report. For example, careful consideration must be given to providing adequate connections. A criterion needs to be developed to define an adequate connection in both rigid and flexible systems; and, further, a criterion needs to be developed for transitions between these two systems. Applications of concepts presented in this report can be used to clearly define the service requirements on which to base design criteria.

To assist the practicing highway engineer in the design of a bridge barrier rail system, ten service requirements have been defined and listed. Some of these service

requirements are widely accepted, some are controversial; all reflect current understanding of available information. The list provides a starting point for the development of design criteria for bridge rail and approach rail systems. It is recommended that these service requirements and the methods outlined in this report be applied to prepare design criteria and outline specifications to produce railing systems based upon a rational design technique. Such a rational approach will produce safe economical railing systems.

An attempt has been made in this report to provide (1) usable information for immediate application, and (2) a framework on which to construct design criteria based on contemporary knowledge and conditions. Since knowledge and conditions are continually changing, it is apparent that design criteria must be subject to continual review and revision.

CHAPTER ONE

INTRODUCTION

Highway bridge barrier railing systems have evolved through need and experience using design information not fully substantiated by research. Railings were provided on early highway bridges for pedestrians and slowly moving vehicles; collisions were rare, impact forces small, aesthetics of small import, and construction and maintenance were not major items of expense, as shown in Figure 1. However, with the advent of high-speed highways necessary to accommodate the greater volume of heavier and faster vehicles, problems of insignificant importance on early highways have now become problems of major concern. For example, some highway bridge railings in recent years have proved to be decorative, but not structurally sound when subjected to the greater impact forces, and, as a result, vehicle penetration of such railing has occurred, as shown in Figure 2.

During the past two decades, significant results having practical application have evolved from the efforts of highway engineers in designing and conducting full-scale dynamic tests on new and modified barrier railing concepts, but still there is a need for a more comprehensive definition of service requirements for bridge barrier railing systems. The present study is aimed at reviewing existing research literature in an attempt to develop a comprehensive definition of bridge railing system service requirements which include: (1) vehicle parameters, such as physical dimensions, weight, and speed; (2) roadway characteristics, such as width and type of surface; (3) railing performance; and (4) the comparative costs of selected barrier railing systems. It is futher intended to provide in this study a method of evaluating candidate railing systems, and to outline a program for continued research.

During the course of this study, nine state highway departments * were visited and discussions concerning bridge barrier railing systems were held with engineers representing bridge design, highway design, traffic operations, and maintenance divisions. It was anticipated that through such discussions the research engineers of Texas Transportation Institute (TTI) would have a better understanding of the highway engineers' endeavors, and, as a result, that a better presentation of information developed during this study might ensue. At the outset of this research project, twelve state highway departments were selected by NCHRP Advisory Panel C66, Section 12, as being representative of the bridge barrier railing systems in existence; however, because of time and monetary limitations, it later became necessary to reduce the number of highway departments to be visited to nine. The research agency engineers are indeed gratified by the interest shown by the highway engineers of the states visited in an attempt to help resolve and provide a definition of bridge barrier service requirements; in fact, much of the information included in this report has been written with the counsel and aid provided by the highway engineers.

Information involving single-vehicle fixed-object fatal accidents, in which one or more fatalities had occurred, has been received from several of the state highway departments visited. The securing of accident information and its interpretation is an arduous task. No statistical significance has been placed on the results presented in this study based on the accident information examined. It is evident during the period of 1965-68 that: (1) approximately 33 percent of the fatal accidents involve fixed objects on high-speed high-

* California, Illinois, Minnesota, New Jersey, New York, Oregon, Texas, Virginia, and Washington.

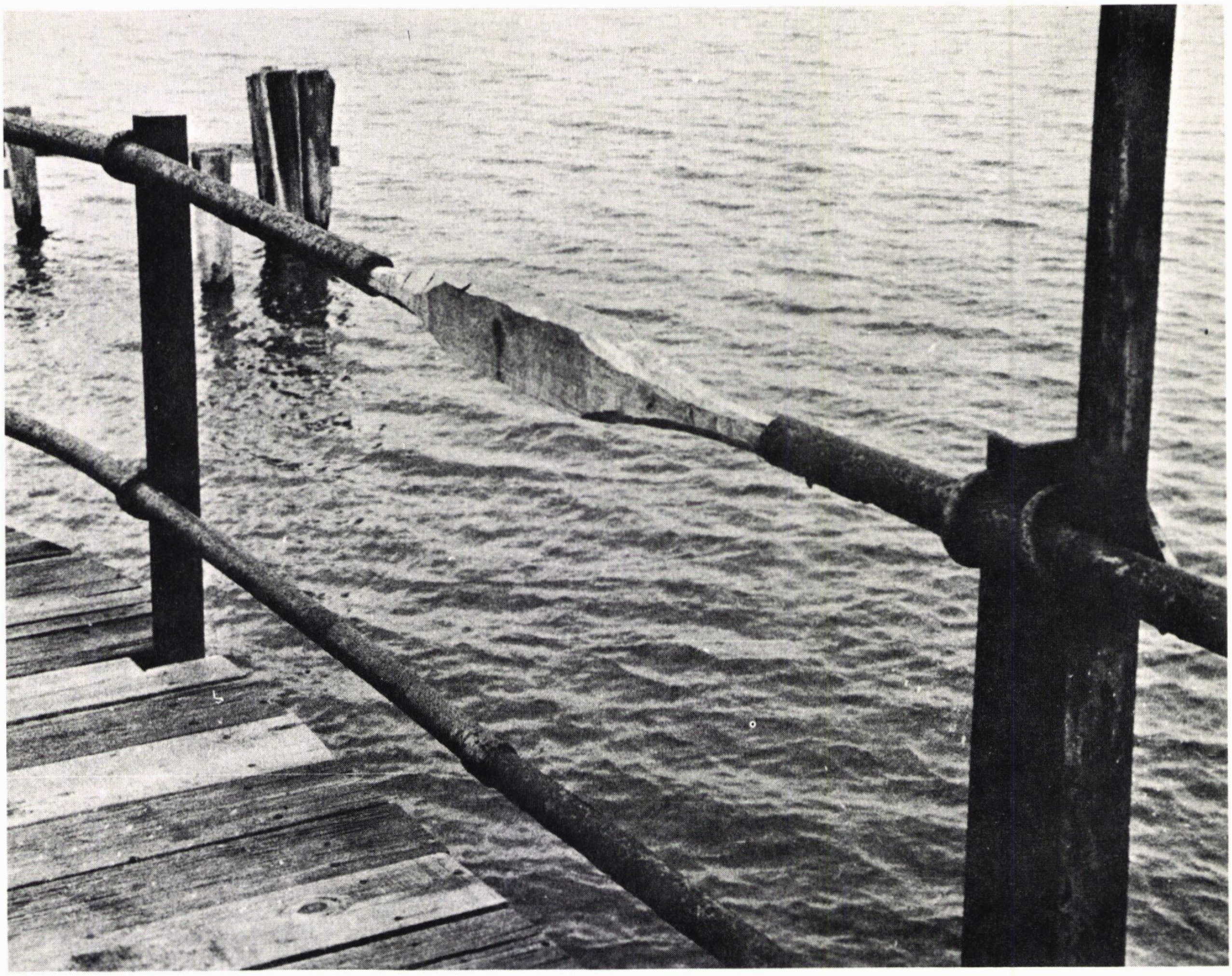


Figure 1. Improvised handrail of the past.

ways, (2) approximately 22 percent of the fixed-object fatal accidents involve bridge barrier railing systems, and (3) certain areas of a bridge barrier railing if not properly designed or adequately protected can constitute a hazardous condition to an out-of-control vehicle.

At the present time, a hazardous condition of major concern, which is responsible for approximately 50 percent of the fatal accidents, is the end of the bridge railing. Examination of information reported by California revealed that 34 percent of the fatal accidents involve bridge ends not protected by an approach guardrail, whereas 18 percent of the fatal accidents involve bridge ends protected by an approach guardrail. It is surmised that many of the fatal accidents at bridge ends had occurred on narrow bridges having a width less than the adjacent highway shoulder width. The current practice in highway design is to construct shoulder-width bridges on high-speed highways. This additional width provides an area for a driver to maneuver a vehicle in the event of an emergency.

It has been demonstrated by researchers conducting full-scale dynamic tests that the approach guardrail-bridge rail juncture must possess structural compatibility as well as alignment compatibility. A satisfactory collision with an approach guardrail progressing to a fatal collision with a bridge end, due to the lateral displacement of the guardrail, or inadequate connection of the guardrail, is not a safe collision. To put it another way, a strength and alignment transition from a flexible structural system (approach guardrail) to a rigid structural system (bridge rail) must be provided. Current design practice to achieve a strength transition adjacent to the bridge railing has resulted in decreased guardrail post spacing, larger size posts, adequate bolting of the approach guardrail to the bridge structure, and the anchoring of the guardrail end.

If an out-of-control vehicle is redirected by an approach guardrail, a secondary, often high-angle, impact with the bridge railing may result in a fatal accident. Thus, the ultimate point of a fatality has been relocated. Certainly, the elimination of the hazardous exposed bridge rail end is desirable, but it must be borne in mind that this is an amelioration and not a panacea.

Methods of eliminating or reducing other well-identified

Figure 2. Example of railing penetration.

hazardous conditions associated with bridge railing systems, such as penetration, vaulting, snagging, and pocketing, are discussed in this report. Highway research programs appear to be moving in the direction of safer installations—if the reduction in fatal accidents is accepted as a criterion.

To evaluate the performance of a bridge railing system, it is necessary to have an estimate of (1) the magnitude of impact forces, and (2) vehicle decelerations which will be tolerable to vehicle occupants. To accomplish this task, the research agency engineers developed a mathematical model to simulate the dynamic behavior of a vehicle-barrier railing collision. By use of the mathematical model, it is now possible to estimate impact forces which take into consideration vehicle parameters as well as roadway characteristics. An extensive amount of research has been conducted in an attempt to determine human tolerance to longitudinal decelerations, such as would occur in front-end type vehicle impacts; however, research regarding human tolerance to simultaneous lateral and longitudinal vehicle deceleration has been limited. An occupant restrained by a seat belt and shoulder harness would most likely experience decelerations similar to the vehicle compartment area, whereas an unrestrained occupant might experience decelerations completely different from that of the vehicle. In any event, the severity of damage to the vehicle would appear to be a good indication of the vehicle decelerations and incidence of injury to unrestrained occupants.

From a 1967 field study conducted in Oregon involving 950 traffic accidents, the National Safety Council (NSC) demonstrated that the proportion of damaged vehicles in which occupant (unrestrained) injuries occurred was proportional to the square of the severity of damage to a vehicle as rated on an arbitrary 7-point photographic scale by police officers and others at the scene of an accident. Employing the mathematical model and extending the work of the NSC to include average vehicle decelerations, a first approximation has been made of estimated injuries or fatalities in relation to vehicle decelerations and impact forces as determined from full-scale dynamic barrier tests.

To summarize, this study has produced two concepts: (1) a method for estimating the magnitude of impact forces, and (2) an insight into establishing the limits of tolerable deceleration. These concepts have been useful in preparing a comprehensive definition of service requirements for bridge barrier railing systems, proposing a rational analytical approach for estimating impact forces, and evaluating selected barrier rails which have been subjected to full-scale dynamic tests. These results are discussed in the following sections.

CHAPTER TWO

RESULTS OF STUDY

ACCIDENT INFORMATION

Information involving single-vehicle fixed-object fatal accidents, in which one or more fatalities had occurred, has been received from seven state highway departments (California, Illinois, New York, Oregon, Texas, Virginia, and Washington). It must be emphasized that no statistical significance has been placed on the results and conclusions presented in this study regarding fatal accidents. Also, after a review of available research literature and the accident information received, it was concluded that fatal accidents involving: (1) the condition of the driver, (2) the condition of the vehicle, (3) climatic conditions, and (4) ejection of the vehicle occupants, which occurred in 75 percent of the fatal accidents involving bridge barrier railing systems in California, were beyond the scope of this investigation.

During the period 1965-66, the New York Department of Transportation indicates that 1,671 fatal accidents occurred of which 481 (28.8 percent) involved fixed objects. The Virginia Highway Research Council indicates that, during the period 1965-67, an average 41.4 percent of the 216 fatal accidents involved fixed objects on the Interstate System. Accident information compiled by the California Division of Highways Traffic Department, as given in Table 1 for the period 1965-67, indicates that an average of 33.8 percent of the freeway fatal accidents involve single vehicles hitting off-road fixed objects, which was higher than any other type of classification. A reduction of this information as to the type of vehicle and object struck is given in Table 2.

The fixed objects in Table 2 that would pertain to this study are: (1) guardrail at fixed objects, (2) bridge rails, and (3) bridge end post at gore. The reason for installing guardrail adjacent to fixed objects, which include bridge rail ends, bridge abutments and piers, lightpoles, steel sign posts adjacent to the highway shoulder, and steel sign posts in the off-ramp gore areas, is to reduce the accident severity. Glennon (*1, p. 184*) indicates that guardrail reduces accident severity only for those conditions where the over-all severity of striking the guardrail is less than the over-all severity of striking the fixed object.

From a field inventory conducted by Glennon (*1, p. 201*) on single-vehicle fatal accidents involving fixed objects on 95 percent of the 1,157 miles of California freeway as of Jan. 1, 1963, it was evident that 39 percent of the guardrail adjacent to fixed objects was located in the vicinity of a bridge structure. Based on this information, it was possible to estimate that 6.9 percent of the fatal accidents in Table 2 involved guardrail at bridge structures. Combining the three fixed objects in Table 2 pertinent to this study, it is evident that bridge barrier railing systems constitute approximately

TABLE 1

640 SINGLE-VEHICLE FATAL ACCIDENTS COMPILED BY CALIFORNIA HIGHWAY TRAFFIC DEPARTMENT (1965-1967)

	NUMBER OF FATAL ACCIDENTS						NUMBER OF PERSONS KILLED					
	1965		1966		1967		1965		1966		1967	
ACCIDENT TYPE	NO.	%	NO.	%	NO.	%	NO.	%	NO.	%	NO.	%
Ran off road hit fixed object	197	33.6	240	37.6	204	30.3	221	31.6	274	37.8	235	28.5
Ran off road did not hit fixed object	113	19.3	163	25.6	167	24.8	124	17.7	172	23.8	189	22.9
Rear-end	98	16.7	80	12.5	107	15.9	111	15.8	90	12.4	124	15.0
Wrong-way	35	6.0	23	3.6	32	4.7	69	9.9	31	4.3	58	7.0
X-median	33	5.6	35	5.5	43	6.4	51	7.3	54	7.5	85	10.3
Pedestrian	69	11.7	78	12.2	74	11.0	72	10.3	80	11.0	79	9.6
Sideswipe	17	2.9	2	0.3	27	4.0	21	3.0	2	0.3	28	3.4
Construction zone	19	3.2	8	1.3	1	0.1	25	3.6	11	1.5	1	0.1
Miscellaneous	6	1.0	9	1.4	19	2.8	6	0.8	10	1.4	26	3.2
All	587	100.0	638	100.0	674	100.0	700	100.0	724	100.0	825	100.0

	1965	1966	1967
Travel (MVM)	23,000	25,970	28,870
Fatality rates per 100 MVM	3.04	2.79	2.86
Fatality accident rates per 100 MVM	(2.55)	(2.46)	(2.33)

TABLE 2

640 SINGLE-VEHICLE FIXED-OBJECT FATAL ACCIDENTS (%) COMPILED BY CALIFORNIA HIGHWAY TRAFFIC DEPARTMENT (1965-1967)

	DISTRIBUTION OF ACCIDENTS (%)													
VEHICLE TYPE	ABUTMENTS/PIERS	GUARDRAIL AT FIXED OBJECTS	BRIDGE RAILS	STEEL SIGN POLES	LIGHT POLES	CABLE BARRIER	OTHER GUARDRAIL	RIGHT-OF-WAY-FENCE	BEAM BARRIER	TREES	BRIDGE END POST AT GORE	WALL	MISCELLANEOUS	ALL
Standard cars	15.2	11.1	7.5	3.9	3.9	4.4	2.2	1.7	1.6	1.6	1.6	0.8	3.4	58.9
Intermediate cars	0.2	0.6	0	0.2	0	0	0	0	0	0.2	0.2	0	0.2	1.6
Compacts and foreign cars	3.1	3.0	2.7	2.2	3.1	3.6	1.6	0.5	0.3	0.8	0.6	0.2	1.9	23.6
Station wagons and vans	1.4	0.9	0.6	0.3	0.6	0.5	0.3	0.3	0	0.3	0.3	0	0.8	6.3
Pick-ups	0.8	0.9	0.3	0.1	0.5	0	0.5	0	0.3	0	0.1	0.1	0	3.6
Trucks	0.6	0.6	0.8	0.5	0.1	0.5	0	0.3	0	0.3	0	0	0.1	3.8
Motorcycles	0.1	0.5	0.3	0.1	0	0.3	0.8	0	0.1	0	0	0	0	2.2
All	21.4	17.6	12.2	7.3	8.2	9.3	5.4	2.8	2.3	3.2	2.8	1.1	6.4	100.0

22 percent of the freeway fixed-object fatal accidents in California. Other states indicate similar results. The Illinois Division of Highways Bureau of Traffic indicated that, during the period of 1967, 281 fatal accidents involved fixed objects of which 63 (22.4 percent) involved bridge barrier railing systems. Also, it was possible to estimate that, during the period of 1967, 22 percent of the fixed-object fatal accidents on the Virginia Interstate System involved bridge barrier railing systems.

Estimates of the vehicle speeds and types involved in fixed-object fatal accidents, as compiled by California and the Texas Highway Department, are given in Tables 3 and 4. It is evident that passenger vehicles constitute between 83 and 90 percent of the fatal accidents, whereas trucks constitute between 4 and 6 percent of the fatal accidents. Because trucks constitute such a small percentage of the fatal accidents, it would seem reasonable to eliminate trucks from design considerations except in unusual circumstances.

It is also evident, from Tables 3 and 4, that the estimated speed range in which the largest percentage of the fatal

TABLE 3

640 SINGLE-VEHICLE FIXED-OBJECT FATAL ACCIDENTS (%) COMPILED BY CALIFORNIA HIGHWAY TRAFFIC DEPARTMENT (1965-1967)

	DISTRIBUTION OF ACCIDENTS[a] (%)								
VEHICLE TYPE	75+ MPH	71-75 MPH	61-70 MPH	51-60 MPH	41-50 MPH	31-40 MPH	21-30 MPH	UNKNOWN MPH	ALL[b]
Standard cars	11.1	6.9	20.9	13.3	4.4	0.6	0	1.7	58.9
Intermediate cars	0.3	0.3	0.5	0.2	0	0	0	0	1.3
Compacts and foreign cars	4.2	1.4	8.3	6.9	1.2	0.2	0.2	0.6	23.0
Station wagons and vans	0.7	0.6	2.8	1.3	0.8	0.2	0	0.1	6.5
Pick-ups	0.2	0.6	2.2	0.5	0.2	0.2	0	0	3.9
Trucks	0	0	0.3	0.8	2.2	0.3	0.2	0.2	4.0
Motorcycles	0.5	0.2	0.3	0.6	0.5	0	0.2	0.1	2.4
All	17.0	10.0	35.3	23.6	9.3	1.5	0.6	2.7	100.0

[a] Speed is estimate of reporting officer.

[b] Passenger vehicles, which include standard cars, intermediate cars, compacts and foreign cars, and station wagons and vans, constitute 89.7 percent of the fatal accidents.

TABLE 4

204 SINGLE-VEHICLE FATAL ACCIDENTS (%) INVOLVING BRIDGE BARRIER RAILING SYSTEMS COMPILED BY TEXAS HIGHWAY DEPARTMENT (1967-1968)

	DISTRIBUTION OF ACCIDENTS[a] (%)								
VEHICLE TYPE	80+ MPH	71-80 MPH	61-70 MPH	51-60 MPH	41-50 MPH	31-40 MPH	21-30 MPH	UN-KNOWN MPH	ALL
Passenger	15.7	11.8	23.5	16.2	9.3	1.9	0.5	4.4	83.3
Truck[b]	1.0	0	2.5	3.4	2.9	0	0	0.5	10.3
Truck & trailer	0	0	1.0	2.9	1.5	0	0	0	5.4
Bus	0	0	0	0	0	0	0	0	0
School bus	0	0	0	0	0	0	0	0	0
Farm tractor or similar	0	0	0	0	0	0	0	0	0
Motorcycle or similar	0	0.5	0	0	0.5	0	0	0	1.0
All	16.7	12.3	27.0	22.5	14.2	1.9	0.5	4.9	100.0

[a] Speed is estimate of reporting officer.
[b] Pickup trucks included.

accidents occurred is 61 to 70 mph. A barrier rail design based on a speed of 70 mph would therefore include approximately 75 percent of the standard and smaller size passenger vehicles.

Only a limited amount of information is available at the present time on vehicle impact angles. Information reported by Deleys (*2, p. 8*) on guardrail accidents, as given in Table 5, indicates that 63.0 percent of the accidents were the result of a right turn into the barrier rail, whereas 28.7 percent of the accidents were the result of a cross one lane left into the barrier rail.

From this information, and making certain assumptions, it is possible to obtain a good estimate of the maximum vehicle impact angle by use of the widely accepted mathematical equation (*10, p. 90; 2, p. 7; 28*):

$$\theta = \cos^{-1}\left[1 - \frac{fgd}{V_I^2}\right] \quad (1)$$

in which

θ = impact angle (deg);
d = initial lateral distance from barrier (ft);
V_I^2 = vehicle impact speed (fps);
g = acceleration of gravity (ft/sec.^{2}); and
f = coefficient of friction between vehicle tires and road.

This equation is based on the assumption that the vehicle is initially traveling parallel to the barrier railing on a straight, level road and is suddenly turned into the barrier on a constant minimum radius determined by dynamic equilibrium of the lateral forces on the vehicle for incipient skidding. During this maneuver the velocity of the vehicle is assumed constant.

Assuming now: (1) a vehicle velocity of 65 mph at which most of the fatal accidents occur, (2) a coefficient of friction between vehicle tires and roadway of 0.7, which is a reasonable value for a dry concrete surface, (3) lane widths of 12 ft, and (4) a shoulder width on each side of 10 ft, a vehicle impact angle of 16° was obtained for a right turn into the barrier railing, and a vehicle impact angle of 22° was obtained for a cross one lane left turn into the barrier railing. Using the same value for the coefficient of friction, Deleys (*2, p. 6*) has shown that the mathematical equation fairly well defines the envelope of injury accidents involving guardrail on two-lane highways and four-lane divided highways.

The information presented fairly well substantiates the full-scale impact test conditions suggested by the Highway Research Board Committee in 1962 on Guardrails and Guide Posts (*3*); that is, the test vehicle shall be of standard design, weighing 4,000 ± 200 lb, with load, have a center of gravity approximately 21 in. above the pavement, and the tests shall be conducted at a speed of 60 mph and at impact angles of 7 and 25°. However, based on the estimated speeds obtained for passenger vehicles involved in fixed-

TABLE 5

1964 NEW YORK STATE THRUWAY REPORTED GUARDRAIL ACCIDENT

LANES CROSSED IN APPROACH TO BARRIER	NO. ACCIDENTS
Right turn into barrier	167 (63.0%)
Cross one lane right	5 (1.9%)
Cross two or more lanes right	3 (1.1%)
Left turn into barrier	3 (1.1%)
Cross one lane left	76 (28.7%)
Cross two or more lanes left	11 (4.2%)

object fatal accidents as shown in Tables 3 and 4 for the period 1965-68, it is recommended that full-scale dynamic tests be conducted at an increased speed of 65 to 70 mph. In a low-angle vehicle impact with a bridge barrier railing system having a standard 10-in. curb extending horizontally 4 in. or more, Nordlin (*4, p. 140*) has indicated that there is a good possibility of the vehicle mounting the curb and vaulting a low type railing. This would appear to be the reason for suggesting that full-scale tests be conducted at an impact angle of 7° in order to assure that the barrier railing is of adequate height.

More information is presented on the encroachment angles of an out-of-control vehicle striking a bridge barrier railing system in "Limits of Tolerable Deceleration," Chapter Two.

A summary breakdown of the single-vehicle fatal accidents which involve only bridge barrier railing systems indicates, as given in Table 6, that certain areas of a barrier rail if not properly designed or adequately protected can constitute a hazardous condition to an out-of-control vehicle. The item in Table 6 of most concern, accounting for more than 50 percent of the fatal accidents, is the collision involving the end of the bridge barrier railing. In California, during the period 1966-67, bridge ends not protected by an approach guardrail accounted for 34 percent of the fatal accidents, whereas bridge ends protected by an approach guardrail accounted for 18 percent of the fatal accidents. During the 1967—90th Congressional Hearing before the Special Subcommittee on the Federal-Aid Highway Program (*5*) inquiring into the design and operational efficiency of recently completed Interstate Highway sections, it was illustrated by numerous photographs that structural continuity and a strength transition were not maintained between the semi-flexible approach guardrail and the more rigid bridge railing. Thus, for impacts near the bridge, the probability of the vehicle hitting the end of the bridge railing due to lateral displacement of the guardrail would be quite high, as indicated in the accident information furnished by California. Current practice to achieve a strength transition adjacent to the bridge railing has resulted in decreased guardrail post spacing, large size posts, adequate bolting of the approach guardrail to the bridge structure, and the anchoring of the guardrail end (*29, p. 9*).

An excellent example to emphasize the importance of a structurally adequate guardrail-bridge rail bolted connection was demonstrated by a full-scale dynamic test conducted by California (*29, p. 16*) as shown in the photographs of Figure 3. Primarily because of a splitting failure of the unreinforced concrete bridge rail through one of the bolt holes, the guardrail was displaced laterally a sufficient distance to allow the vehicle to impact the end of the bridge rail.

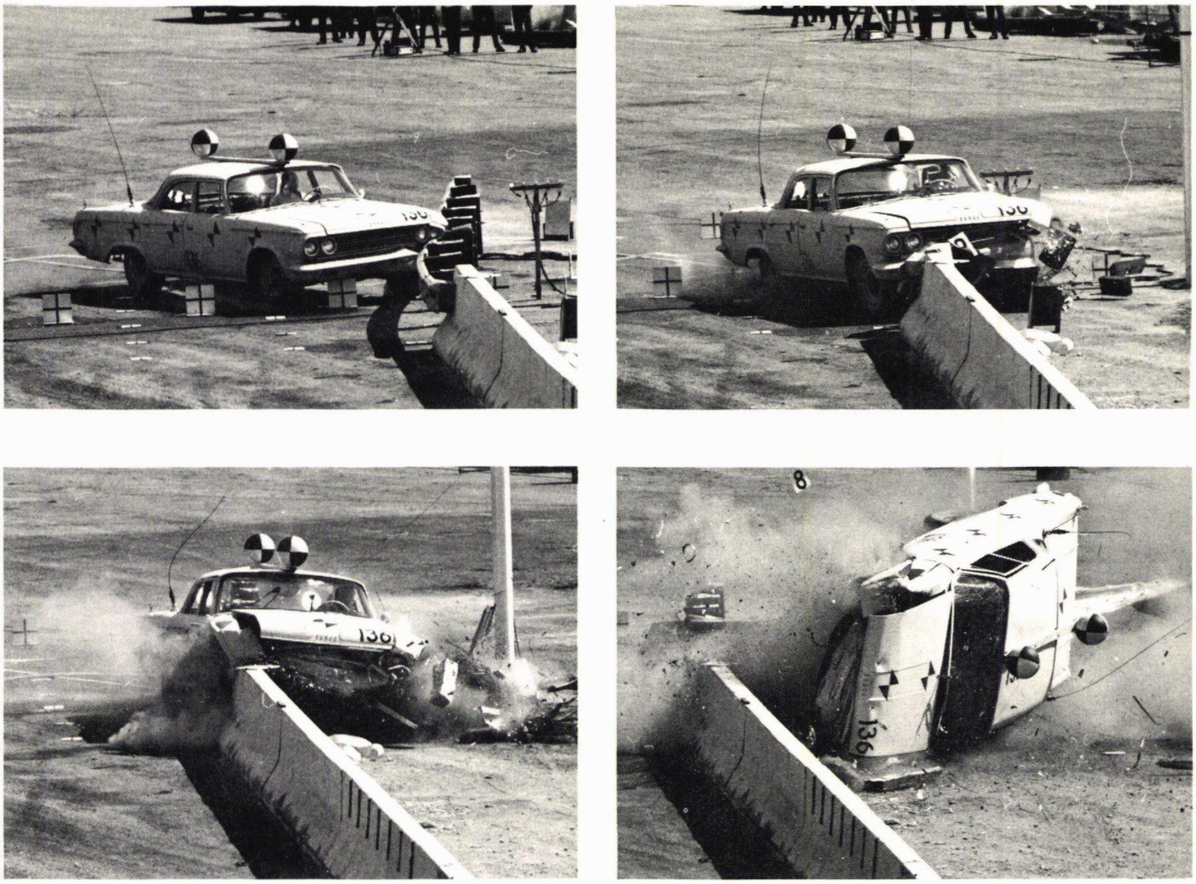
During the period 1963-64, Glennon (*1, p. 202*) has

TABLE 6

SINGLE-VEHICLE FATAL ACCIDENTS (FA) INVOLVING BRIDGE BARRIER RAILING SYSTEMS (PERCENT)

ITEM	CALIFORNIA [a] (77 FA) 1966-67	ILLINOIS [b] (63 FA) 1967	OREGON (9 FA) 1966-67	TEXAS (204 FA) 1967-68	WASHINGTON (5 FA) 1968
TYPE OF FATAL COLLISION ACCIDENT					
Collision with approach guardrail	10	21	0	15	0
Collision with bridge end	52	59	22	57	80
No approach guardrail	34	NR	NR	NR	NR
Protected by approach guardrail	18	NR	NR	NR	NR
Collision with approach guardrail, redirected into collision with bridge rail	7	NR	NR	NR	NR
Collision with bridge rail	31	20	78	21	20
BARRIER RAIL PERFORMANCE					
Vehicle penetrated	14	6	44	22	20
Vehicle vaulted	2	5	34	21	0
Vehicle pocketed or snagged (includes end type impacts)	52	41	11	51	80
Vehicle redirected	32	48	11	NR	NR
Unknown	0	0	0	6	0

NR=Information not requested in accident form letter.
[a] Ejection of the vehicle occupants occurred in 72 percent of the fatal accidents.
[b] Illinois had a total of 281 fatal accidents involving fixed objects of which 63 (22.4%) involved bridge barrier railing systems.
[c] From the findings of the Blatnik Committee (5), it can be assumed that structural continuity was not maintained between approach guardrail and the bridge railing.



This test emphasizes the importance of a structurally adequate guardrail-bridge rail bolted connection. Due primarily to a splitting failure of the unreinforced concrete bridge rail through one of the bolt holes, the guardrail was displaced laterally a sufficient distance to allow the vehicle to impact the end of the bridge rail. The test installation consisted of a standard 27-in.-high "W" section guardrail connected to a simulated bridge parapet (New Jersey Barrier) with two 1-in.-diameter high-strength bolts. The opposite end of the flared 53-ft length of guardrail was anchored. Using a standard size vehicle (4,540 lb), the test was conducted at a speed of 60 mph and 25° relative to bridge rail or approximately 30° relative to impact point of guardrail.

Figure 3. Full-scale dynamic test 136 conducted by California Division of Highways Materials and Research Laboratory (29, p. 16)*—Sept. 28, 1967.*

determined, based on 158 fixed-object fatal accidents,* that 19 (12 percent) of the accidents involved bridge railing ends not protected by an approach guardrail, whereas 16 (10.1 percent) of the accidents involved bridge railing ends protected by an approach guardrail. (From the findings of the Blatnik Committee (*5*), it can be assumed that structural continuity was not maintained.) If guardrail were installed adjacent to all bridge rail ends according to a Collision Index (Eq. 2) analysis, Glennon (*1, p. 204*) also determined that a reduction in fatal accidents of 37 percent would be expected.

$$CI = SI \times PI = \frac{25\,F + 6\,I + P}{V} \tag{2}$$

in which

SI = severity index;
PI = probability index;
CI = collision index;
F = number of fatal accidents for condition;
I = number of injury accidents for condition;
P = number of PDO accidents for condition;
N = number of total accidents for condition; and
V = number of vehicles exposed to condition during accident study period.

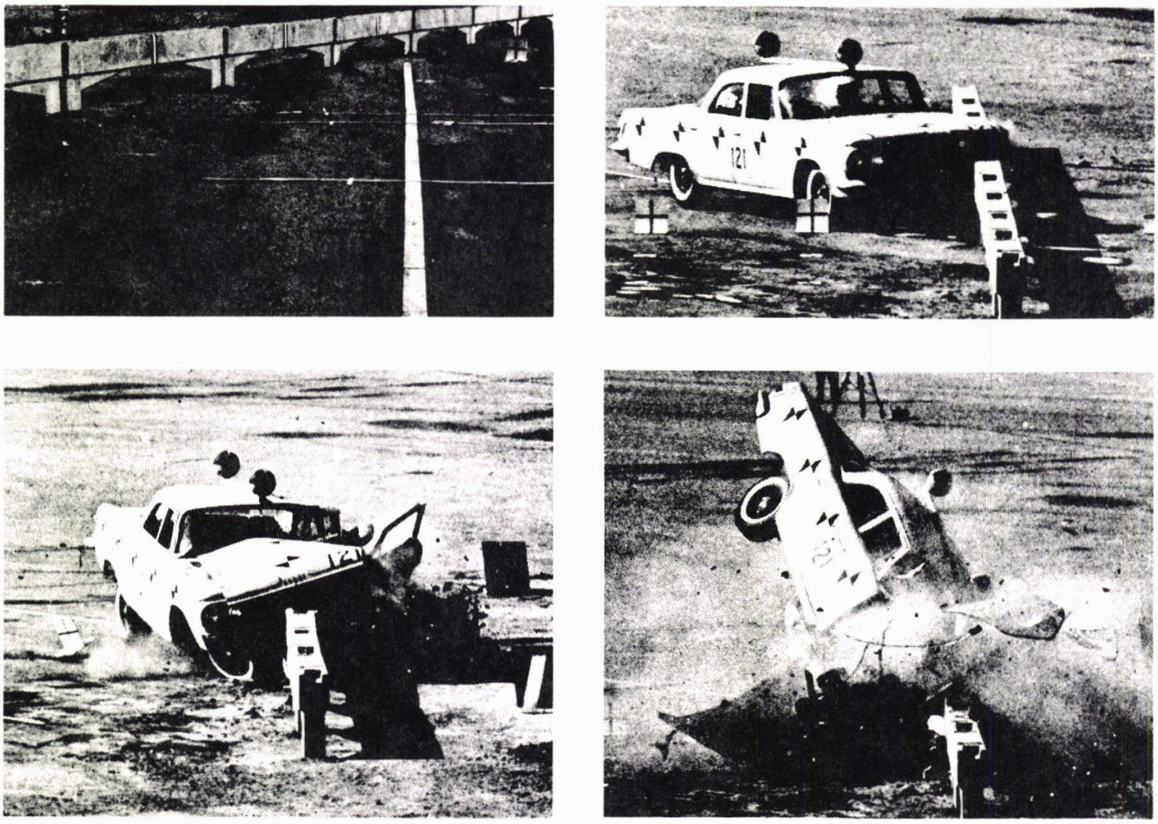
It is reasonable to assume that if, in addition, the approach guardrail were adequately connected to the bridge railing, the reduction in fatal accidents involving the end of the bridge railing would be higher than 37 percent.

* Fatal accidents relative to the fixed objects—bridge rail ends, abutments and bridge piers, lightpoles, steel sign posts adjacent to the shoulder, and steel sign posts in gore area.

A substantial amount of accident information is also available in regard to guardrail ends. For example, Deleys (*2, p. 8*) has shown that approximately 50 percent of the accidents in 1964 involving guardrail on two-lane highways and four-lane highways, in which the guardrail performance was classified as a failure, were the result of end impacts.

It is generally agreed that a safe bridge barrier railing system should restrain a colliding vehicle, prevent a vehicle from vaulting, and minimize vehicle decelerations to a level which will be tolerable to the vehicle occupants assumed to be unrestrained. Penetration of a barrier railing, which constituted approximately 20 percent of the fatal type accidents as given in Table 6, can be eliminated by proper design for strength. Recommendations as to the determination of a lateral impact design force are discussed in this chapter under section "Rational Analytical Approach." Vaulting of a barrier railing can be eliminated by proper attention to providing adequate railing height in addition to structural strength, and the elimination of discontinuities such as curbs, safety walks, and sidewalks. To minimize vehicle decelerations to a level which will be tolerable to unrestrained occupants, snagging and pocketing of a barrier railing must be eliminated. An example of undesirably high vehicle decelerations caused by the pocketing of a section of barrier railing and then snagging on the exposed end of an adjacent section of railing was demonstrated by a full-scale dynamic test conducted by California (*30*) as shown in the photographs of Figure 4.

If an out-of-control vehicle is redirected by an approach guardrail, a secondary usually high-angle impact with the bridge railing may result in a fatal accident, as given in Table 6. Thus, the ultimate point of a fatality has been relocated. Certainly, the elimination of the hazardous exposed bridge rail end is desirable, but it must be borne in mind that this is an amelioration and not a panacea. It has been observed that accident frequency increases as length



This test demonstrates the undesirably high vehicle decelerations caused by pocketing of a section of barrier railing and then snagging on the exposed end of an adjacent section of railing. The impact was so severe that the engine was ejected from the vehicle. The anthropomorphic dummy's head was thrown 23 ft from impact when the three $\frac{1}{4}$-in.-diameter high-strength bolts in the neck joint were sheared as the shoulder struck the windshield post. The test installation consisted of 10 precast reinforced concrete units with an over-all length of approximately 80 ft. The units, which were connected by a double angle steel bar, were set in drilled holes back-filled by soil. Using a standard size vehicle (4,540 lb), the test was conducted at a speed of 66 mph and an angle of impact of 25°.

Figure 4. Full-scale dynamic test 121 conducted by California Division of Highways Materials and Research Laboratory (30)*—Oct. 28, 1965.*

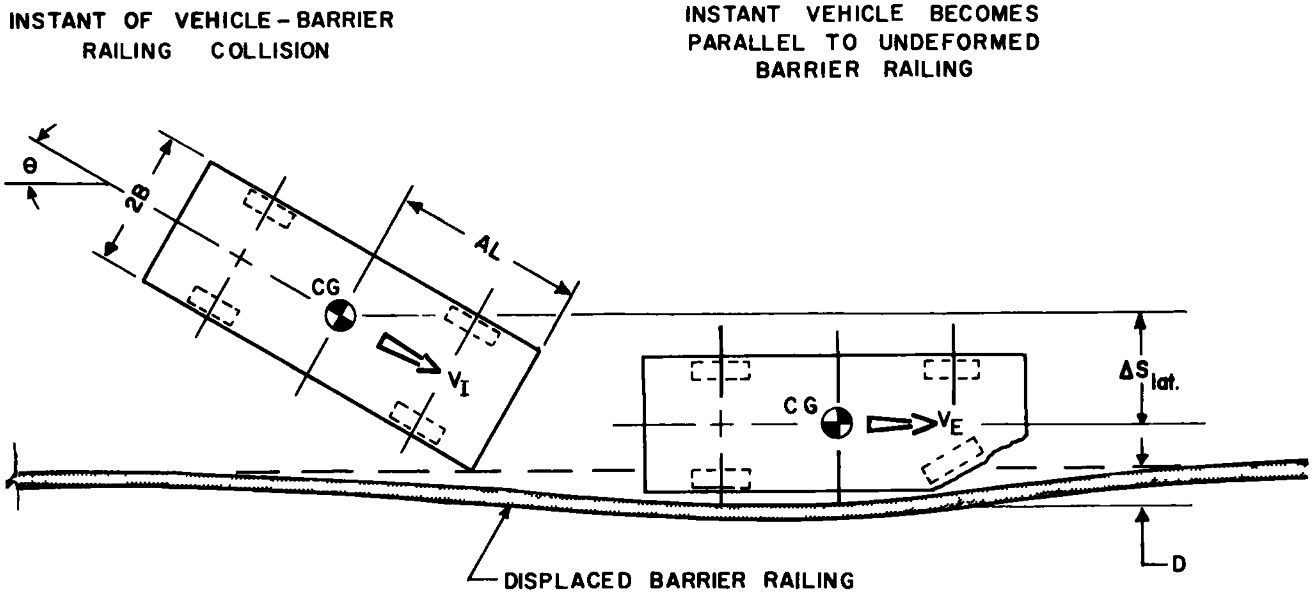


Figure 5. Mathematical model of vehicle-barrier railing collision.

of guardrail increases. Beaton (*6*) and Sachs (*7*) concluded that a median barrier virtually eliminates cross-median fatal accidents, but the frequency of injury accidents increases. These findings lead one to conclude that installation of approach railing adjacent to barrier railing has a salutary effect. Highway safety programs appear to be moving in the direction of safer installations, if reduction in fatal accidents is accepted as a criterion.

In conclusion, it is evident that many barrier rail configurations constitute a hazardous condition to an out-of-control vehicle, and, as a result, approximately 22 percent of the fixed-object fatal accidents involve bridge barrier rail systems. The hazardous conditions that have been well identified in research literature, through discussions with highway engineers in several states, and in this study are: (1) penetration of the bridge or the approach barrier rail by a standard or smaller size vehicle, (2) snagging of a vehicle by the bridge or the approach barrier rail, and (3) a vehicle impact at the approach end of the bridge or the approach barrier rail.

MATHEMATICAL MODEL OF A VEHICLE-BARRIER RAILING COLLISION

An extensive review of available research literature revealed that many full-scale dynamic tests of barrier railing have been conducted by several research organizations, and much useful information has been made available in written reports and on high-speed film records. As study of this information progressed, it was evident that a comprehensive definition of service requirements and a rational analytical approach as a basis for the design of a bridge barrier railing system have not been available to the highway engineer.

To help achieve these objectives, a simple mathematical model of a vehicle-barrier railing collision evolved. Observations of high-speed films and sequence photographs led to the development of the equations presented in this report. These equations assume that, at the instant of impact, the vehicle motion can be defined by an angle, θ, and a velocity, V_I, as shown in Figure 5. That is to say, the vehicle is out of control and should the driver be able to reduce his speed or turn the vehicle, a less severe collision, or none at all might result. The following assumptions have also been made:

(1) The lateral and longitudinal vehicle decelerations are constant during the time interval required for the vehicle to become parallel to the undeformed barrier.
(2) Vertical and rotational accelerations of the vehicle are neglected.
(3) The lateral component of velocity is zero after the vehicle is redirected parallel to the barrier railing.
(4) The vehicle is not snagged by the barrier railing as it is being redirected.
(5) Deformation of the vehicle occurs in the area of impact, but the center of mass (C.G.) of the vehicle is not thereby changed appreciably.
(6) The mass center of the vehicle moves as if the entire mass were concentrated at that point.
(7) A barrier may be rigid ($D = 0$) or it may be flexible ($D > 0$).
(8) The friction forces developed between the vehicle tires and roadway surface are neglected.
(9) The barrier railing system does not contain discontinuities (jutting curbs, etc.) which might produce abrupt vertical movement of the vehicle.

Based on these assumptions, the motion of the vehicle, that occurs during the time interval required for the vehicle to become parallel to the undeformed barrier railing, can be determined from the basic principles of dynamics.

Referring to Figure 5, the lateral movement of the vehicle, $\Delta S_{lat.}$, is expressed by the equation:

$$\Delta S_{lat.} = AL \sin(\theta) - B[1 - \cos(\theta)] + D \qquad (3)$$

This lateral movement of the vehicle occurs during the time interval, Δt, expressed by the equation:

$$\Delta t = \frac{\Delta S_{\text{lat.}}}{\text{Average Lateral Velocity}}$$

in which

Average Lateral Velocity $= \frac{1}{2}\,[V_I \sin(\theta) + 0]$

$$\Delta t = \frac{AL \sin(\theta) - B[1 - \cos(\theta)] + D}{\frac{1}{2} V_I \sin(\theta)} \tag{4}$$

Having an expression for the time interval, the average vehicle decelerations, G's, are expressed by the equations:

Average Lateral Vehicle Deceleration ($G_{\text{lat.}}$)

$$G_{\text{lat}} = \frac{a_{lat}}{g} = \frac{(\Delta V)_{\text{lat.}}}{g(\Delta t)}$$

in which

Change in Lateral Velocity $(\Delta V) = V_I \sin(\theta) - 0$

$$G_{\text{lat.}} = \frac{V_I \sin(\theta)}{g(\Delta t)}$$

$$G_{\text{lat.}} = \frac{V_I^2 \sin^2(\theta)}{2g\{AL \sin(\theta) - B[1 - \cos(\theta)] + D\}} \tag{5}$$

Average Longitudinal Deceleration (G_{long})

$$G_{\text{long}} = \frac{a_{\text{long.}}}{g} = \frac{(\Delta V)_{\text{long.}}}{g(\Delta t)}$$

$$= \frac{V_I \cos(\theta) - V_E}{g(\Delta t)}$$

$$= \frac{G_{\text{lat}}\,[V_I \cos(\theta) - V_E]}{V_I \sin(\theta)}$$

$$G_{\text{long.}} = G_{\text{lat}}\left[\cot(\theta) - \frac{V_E}{V_I}\csc(\theta)\right] \tag{6}$$

The magnitude of the average lateral and longitudinal impact force developed between the vehicle and barrier railing can be determined from Newton's basic laws of motion.

Average Lateral Impact Force (F_{lat})

$$F_{\text{lat.}} = m(a_{\text{lat.}})$$

$$F_{\text{lat.}} = W(G_{\text{lat.}}) \tag{7}$$

Average Longitudinal Impact Force ($F_{\text{long.}}$)

$$F_{\text{long}} = \mu(F_{\text{lat.}})$$

$$F_{\text{long.}} = \mu W(G_{\text{lat.}}) \tag{8}$$

From Eq. 8, an expression can be obtained between the average lateral and longitudinal vehicle decelerations.

$$F_{\text{long.}} = m(a_{\text{long}})$$

$$F_{\text{long.}} = W(G_{\text{long.}})$$

Thus,

$$G_{\text{long.}} = \mu(G_{\text{lat.}}) \tag{9}$$

in which

$L =$ vehicle length (ft);
$2B =$ vehicle width (ft);
$D =$ lateral displacement of barrier railing (ft);
$AL =$ distance from vehicle's front end to center of mass (ft);
$V_I =$ vehicle impact velocity (fps);
$V_E =$ vehicle exit velocity (fps);
$\theta =$ vehicle impact angle (deg);
$\mu =$ coefficient of friction between vehicle body and barrier railing;
$a =$ vehicle deceleration (ft/sec^2);
$g =$ acceleration due to gravity (ft/sec^2);
$m =$ vehicle mass (lb-sec^2/ft); and
$W =$ vehicle weight (lb).

These equations express the average vehicle decelerations as a function of: (1) type of barrier railing—rigid or flexible, (2) dimensions of the vehicle, (3) location of the center of mass of the vehicle, (4) impact speed of vehicle, (5) impact angle of the vehicle, and (6) coefficient of friction between the vehicle body and barrier railing. Review of published reports of full-scale crash tests revealed that the investigators had recorded impact speeds, impact angles, and other information such as measured or computed decelerations. Some of this information could be used for input to the mathematical model and other information could be used to compare with output from the model. For example, substitution of impact speed and impact angle reported for a selected crash test of a rigid barrier as input into the model produces a computed deceleration value. This value can be compared with the value reported by the investigator. Such computations were made for many reported crash tests to establish confidence in the mathematical model. The results of these computations are contained in Appendix A.

It is demonstrated in Appendix A that the mathematical model predicts the behavior of a standard size passenger vehicle to an accuracy of ±20 percent. Such a comparison is remarkable when one considers the simplicity of the model and the difficulties involved in acquiring and reducing data obtained from full-scale dynamic tests (*8, p. 18*). An identical equation was developed independently in the Netherlands and reported by Jehu (*9, p. 46*); however, the author gave no indication of how well the model simulated an actual vehicle-barrier impact.

LIMITS OF TOLERABLE DECELERATION

During a collision between an out-of-control vehicle and a bridge barrier railing system in which the vehicle is smoothly redirected, the vehicle is subjected simultaneously to lateral and longitudinal decelerations. An insight into the dynamic behavior of the vehicle and bridge barrier railing system has been provided by the mathematical model.

To evaluate the performance of a bridge barrier railing system, the mathematical model is used in conjunction with available research information to determine average vehicle deceleration levels which would produce injury to the occupants.

An extensive amount of research has been conducted in an attempt to determine human tolerance to longitudinal vehicle decelerations (*31*), such as would occur in front-end-type vehicle impacts; however, research regarding human tolerance to simultaneous lateral and longitudinal vehicle decelerations has been limited.

To evaluate barrier performance, Graham (*10, p. 92*) indicated, after a review of available literature on tolerable deceleration levels, that a total deceleration of 10 *G*'s for more than 50 milliseconds at the vehicle center of gravity was considered as probably capable of producing serious injuries and perhaps even fatal injuries to an unrestrained occupant. Therefore, an effort was made by Graham to keep the vehicle decelerations below this level during the full-scale impact tests conducted on the barrier railing developed in New York.

Investigators at Cornell Aeronautical Laboratory (*10, p. 91*) suggested the limits given in Table 7 of tolerable longitudinal and lateral decelerations where the duration did not exceed 200 milliseconds and the rate of onset did not exceed 500 *G*'s per sec. Cornell also reported that further work with anthropomorphic dummies subjected to known decelerations is required to better establish these limits. It is shown subsequently that the results obtained by the research agency engineers are in good agreement with the deceleration limits established by both Cornell and Graham.

During full-scale dynamic tests on various highway barrier railing designs, Nordlin (*11*) recorded acceleration data by means of a triaxial mechanical stylus accelerometer mounted in the chest cavity of a Sierra Engineering Company, Model 157, anthropomorphic dummy and another accelerometer mounted on the rear floor of the vehicle. The dummy was placed in the driver's seat and restrained by a lap belt and/or shoulder harness. Due to the effects of "ringing" caused by transient vibrations through the vehicle frame, the recordings from the vehicle accelerometer were not considered representative of the actual decelerations sustained by the vehicle and were therefore not reported. Accelerometer recordings of the dummy are given in Table 8. An approximation of the *G*-level can be obtained by multiplying tabulated values by 6. No attempt was made to relate this accelerometer information or dummy injuries to actual injuries that would have been sustained by a human counterpart. The primary function of the dummy was to evaluate the relative efficiency of the various restraint systems in the prevention of partial ejection. However, the accelerometer recordings were of value for a comparative study of the various barrier railing de-

TABLE 7

LIMITS OF TOLERABLE DECELERATION SUGGESTED BY CORNELL AERONAUTICAL LABORATORY

RESTRAINT	MAXIMUM DECLERATION (G'S): LATERAL	LONGITUDINAL	ALL
Unrestrained occupant	3	5	6
Occupant restrained by lap belt	5	10	12
Occupant restrained by lap belt and shoulder harness	15	25	25

TABLE 8

ANTHROPOMORPHIC DUMMY DECELERATIONS REPORTED BY CALIFORNIA DIVISION OF HIGHWAYS (*11*)

			DUMMY IMPACT[b]						VEHICLE TRAJECTORY					
			TRANS.		LONG.		VERT.		ANGLE		SPEED (MPH)		ROLL	
BARRIER	TEST NO.	DUMMY RESTRAINT	L	R	FWD.	BK.	UP	DN.	ENT.	EXIT[a]	ENT.	EXIT[a]	L	R
New Jersey median	161-A	lap belt	1.0		0.2	0.1		0.2	7°		38	41		
	161-B	lap belt	1.2		0.7		0.7	0.2	7°		65	61		14°
	162	lap belt	4.3		0.8	0.7	1.0	1.7	25°	12°	63	55		25°
W-beam median	101	lap belt & harness	3.5		2.3			1.5	25°	15°	69	41		5°
W-beam guardrail	107	lab belt	2.5		1.4		0.9		25°	17°	60	37		Flat
New York median	142	lap belt	2.0		1.0			1.0	25°	6°	64	46	18°	
Concrete Type 1 bridge rail	B-5	lap belt & harness	4.0		2.0			1.3	25°	5°	78	62		2°

[a] Exit angle and speed measured 25 ft to 50 ft from point of impact and prior to cutting ignition and applying brakes.
[b] Readings indicate relative impact intensities as recorded on mechanical stylus impactograph. The magnitudes are not to be construed as actual G forces.

signs tested. Barrier rail systems that deflected laterally were effective, as noted by the lower transverse readings. It is interesting to note that there was no significant difference between the vertical readings for the New Jersey barrier, which has a lower sloping face, and the W-beam guardrail when tested under the same impact conditions. Also of interest, the longitudinal decelerations of the anthropomorphic dummy in the majority of tests were approximately or equal to one-half of the lateral decelerations. As shown in Appendix A, a similar relationship was found to exist between the average lateral and longitudinal decelerations of vehicles involved in full-scale dynamic tests.

Thus, an occupant restrained by a seat belt and shoulder harness would most likely experience decelerations similar to the vehicle compartment area, whereas an unrestrained occupant might experience decelerations completely different from that of the vehicle. In any event, the severity of damage to the vehicle would appear to be a good indication of the vehicle decelerations and incidence of injury to unrestrained occupants. Based on the work of Michalski (*12*) and employing the mathematical model discussed previously, this hypothesis has been confirmed.

From the results of a 1967 field study conducted in Oregon involving 951 vehicles in traffic accidents of which there were 184 personal injuries and 7 fatalities, Michalski demonstrated, as shown in Figure 6, that the proportion of damaged vehicles in which injuries occurred was proportional to the square of the severity of damage to a vehicle as rated on a 7-point photographic scale (*17*) by police officers and others at the scene of an accident. Michalski

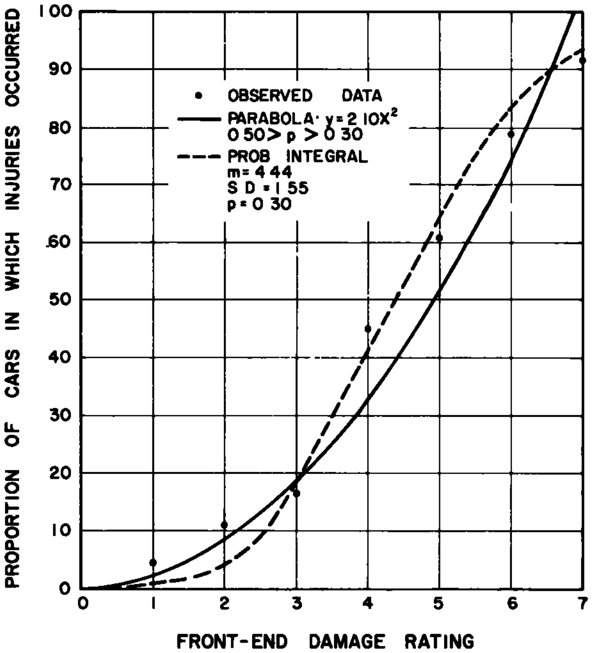


Figure 6. Occurrence of personal injuries in relation to front-end damage rating (12, p. 38).

indicates that the probability integral and parabolic curves in Figure 6 may be used with equal facility to predict incidence of injuries in relation to vehicle damage rating. Michalski has indicated verbally that less than 5 percent of the vehicle occupants were restrained by a seat belt and/or shoulder belt.

To apply and extend the work of Michalski to include average vehicle decelerations, vehicles damaged in full-scale dynamic tests by various research agencies were selected for evaluation. As shown in Figures 7 and 8, the results of this study, which are developed in depth in Appendix B, indicate that the average vehicle decelerations are directly proportional to: (1) the proportion of damaged vehicles involved in traffic accidents in which occupant injuries occurred, and (2) the square of the vehicle damage rating. In mathematical notation, this relationship would be represented by the equations:

Type Vehicle Impact	*Mathematical Equation*	
Front	$G_{long} = 0.280\,R^2 = 13.7\,P$	(10)
Angle	$G_{lat} = 0.204\,R^2 = 10.0\,P$	(11)

in which

G = average vehicle deceleration;
R = vehicle damage rating; and
P = proportion of vehicles in which injuries occurred.

It must be noted that the average lateral vehicle decelerations are based on the assumption that the vehicle is smoothly redirected by the barrier rail.

In addition to demonstrating that the proportion of vehicles in which personal injuries occurred in relation to damage rating was nearly parabolic in form, Michalski determined that at mean damage ratings * of: (1) 1.99—vehicles are drivable, (2) 4.08—vehicles are non-drivable, (3) 4.45 and 4.73—injuries occurred in front-end and side vehicle impacts, respectively, and (4) 2.32 and 2.49—no injuries occurred in front-end and side vehicle impacts, respectively. Based on the mathematical relationship established, Eqs. 10 and 11, the average vehicle decelerations and the percent of vehicles involved in an accident in which injuries would occur that correspond to the conditions for which mean damage ratings were determined by Michalski are given in Table 9.

Before attempting to predict the severity of a barrier rail accident, a study of vehicle encroachments on a barrier rail must be made. From accident data on the Ohio Turnpike for a period of five months during the summer and fall of 1967, Garrett (*13*) reported vehicle speeds and the departure angles as given in Table 10. For purposes of this study, it is assumed that the mean departure angles for various speeds as reported by Garrett would also be representative for an out-of-control vehicle striking a barrier railing.

From injury accident data involving guardrail on two-lane highways and four-lane divided highways, it was possible to estimate from the graph presented by Deleys (*2, p. 6*) that approximately 85 to 90 percent of the accidents (excluding inappropriate data) occurred at an angle of 20° and less.

* Unless noted, the mean damage ratings include all types of impacts.

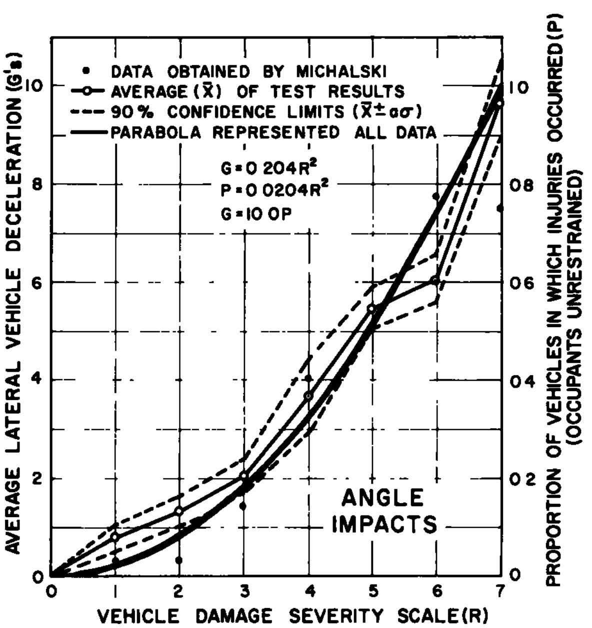


Figure 7. Curve relating lateral deceleration, proportion of injuries, and damage rating scale.

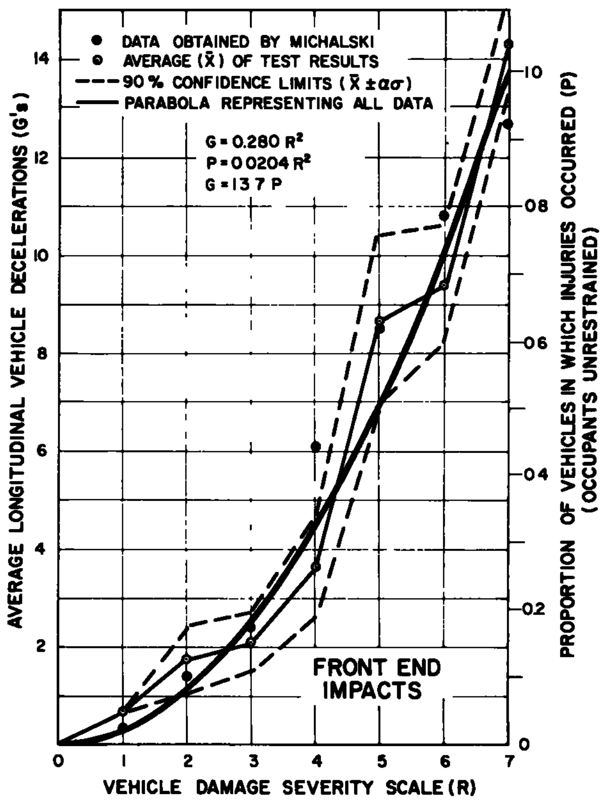


Figure 8. Curve relating longitudinal deceleration, proportion of injuries, and damage rating scale.

When the average lateral decelerations of the mathematical model, Eq. 5, are graphically plotted using the vehicle impact speed as the ordinate and the impact angle as the abscissa, it is evident as shown in Figure 9 that approximately 80 to 85 percent of the accident data reported by Deleys and the curve representing the information obtained by Garrett fall to the left (less than) of a 2-*G* level deceleration curve; whereas 85 to 90 percent of the accident data fall to the left of a 3-*G* deceleration curve.

Based on the information presented, and assuming that the hazardous conditions discussed in section "Accident Information" are eliminated, this agency-conducted study indicates that for approximately 85 percent of the accidents, a standard-size vehicle would be subjected to an average lateral deceleration (at the center of mass of the vehicle) of 3 *G*'s or less for various combinations of impact speed and angle. At this deceleration level, this study also indicates that 85 percent of the accidents would be non-fatal, and 60 percent of the accidents would not produce injuries to unrestrained occupants. As is evident from Table 9, a 3-*G* average deceleration level for angle impacts would correspond to a slightly lower rating than that at which vehicles are non-drivable.

TABLE 9

AVERAGE VEHICLE DECELERATIONS AS FUNCTION OF NSC MEAN DAMAGE VEHICLE RATINGS AND INJURY LEVELS

CONDITIONS FOR WHICH MEAN DAMAGE RATINGS WERE DETERMINED BY MICHALSKI (*12*)	AVERAGE VEHICLE DECELERATIONS (G'S)		% OF VEHICLES IN WHICH INJURIES WOULD OCCUR	
	FRONT IMPACTS	ANGLE IMPACTS	FRONT IMPACTS	ANGLE IMPACTS
Vehicles drivable	1.1	0.8	8	8
Vehicles non-drivable	4.7	3.4	34	34
No injuries	1.5	1.3	11	13
Injuries	5.5	4.6	40	46

TABLE 10

SPEED-MEAN DEPARTURE ANGLE

Speed range (mph)	10-19	20-29	30-39	40-49	50-59	60-69
Mean departure angle[a] (degrees)	48.5	8.8	7.9	7.1	2.0	3.7
No. of observations	2	5	8	30	78	126

[a] Departure angle was defined as the angle of the path of the vehicle as it left the paved roadway.

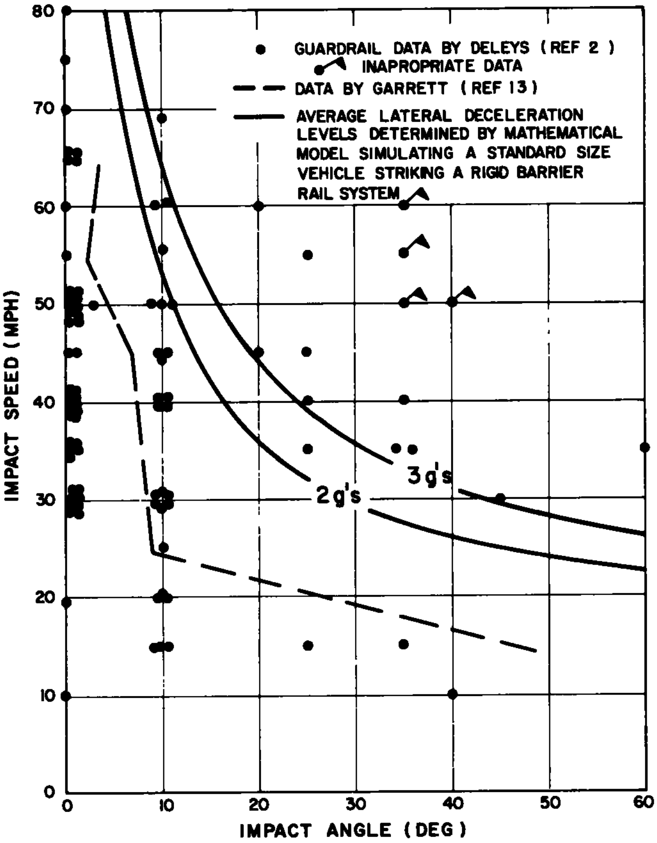


Figure 9. Impact speed vs impact angle with computed G-levels superimposed.

It was noted earlier that Cornell Aeronautical Laboratory (Table 7) suggested that a lateral vehicle deceleration of 3 *G*'s would be tolerable for an unrestrained occupant in which the time duration did not exceed 200 milliseconds. This value is the same as that just established in this study. In the 20 full-scale dynamic tests conducted by California on various highway barrier railing systems (Appendix A, Table A-2), it was determined by use of Eq. 4 that the average time duration, required for the vehicle to become parallel to the center line of the undeformed barrier railing, was 201 milliseconds. This average was over a time interval ranging from 126 to 333 milliseconds.

For front-end-type impacts, an average longitudinal vehicle deceleration of 4.1 *G*'s would produce the same severity rating as that established for angle impacts. This value is in good agreement with the 5-*G* longitudinal deceleration level suggested by Cornell (Table 7). The average time duration of the 27 full-scale dynamic tests conducted by Texas Transportation Institute was also 201 milliseconds. This averge was over a time interval range of 126 to 476 milliseconds.

The findings of this study also support the tolerable deceleration levels established by Graham. As noted earlier, Graham indicated that a total deceleration of 10 G's for more than 50 milliseconds at the vehicle center of gravity was considered as probably capable of producing serious injuries and perhaps even fatal injuries to an unrestrained occupant. Referring to Figure 7, it is also apparent that at an average vehicle deceleration level of 10 G's the proportion of vehicles in which injuries of unrestrained occupants occurred was 100 percent. The average time duration of the collisions in Figure 7 was 201 milliseconds. From the severity of damage to a vehicle at 10 G's, which corresponds to a National Safety Council Rating of 7, it is estimated that fatalities would have occurred in the majority of the accidents at this level of deceleration.

RATIONAL ANALYTICAL APPROACH

From the accident information presented in a foregoing section, it is evident that many bridge barrier railing systems in existence are not structurally adequate to restrain or smoothly redirect an out-of-control standard size passenger vehicle. An apparent explanation for this structural inadequacy of a barrier railing has been the need for a method to determine the magnitude of a realistic impact design force as a function of: (1) the vehicle characteristics, such as dimensions, weight, speed; and (2) the roadway characteristics, such as width, and type of surface. The mathematical equations presented in this report are responsive to this need. The average lateral deceleration of the vehicle and the magnitude of the impact force, as a function of the vehicle characteristics, are expressed by Eqs. 5 and 7, repeated here.

Average Lateral Vehicle Deceleration ($G_{lat.}$)

$$G_{lat.} = \frac{V_I^2 \sin^2(\theta)}{2g\{AL \sin(\theta) - B[1 - \cos(\theta)] + D\}} \tag{5}$$

Average Lateral Impact Force ($F_{lat.}$)

$$F_{lat} = W(G_{lat.}) \tag{7}$$

The lateral decelerations of a vehicle, as a function of the impact angle, are readily determinable when presented graphically as shown in Figure 10, in which the vehicle dimensions are constants, vehicle impact speeds are parameters, and the barrier railing is assumed to be rigid.

To express the average lateral deceleration of a vehicle and the magnitude of the impact force as a function of the roadway characteristics, as well as of a function of the vehicle characteristics, Eq. 1 is now employed. Combining Eqs. 1 and 5, the lateral decelerations of a vehicle, as a function of the width of roadway traversed by an out-of-control vehicle traveling at some constant speed in advance of striking a rigid barrier railing, are readily determinable when presented graphically as shown in Figures 11 and 12, in which the vehicle dimensions are constants, and the coefficient of friction between the vehicle tires and roadway are parameters. Maintaining a fixed value for the coefficient of friction, the lateral deceleration of a vehicle can also be expressed as a function of the width of roadway traversed by the vehicle and the displacements of a semi-rigid barrier railing as shown in Figure 13. It is to be noted that, in Figures 11 through 13, the dimensions of the vehicle are identical to the vehicle dimensions used in Figure 10.

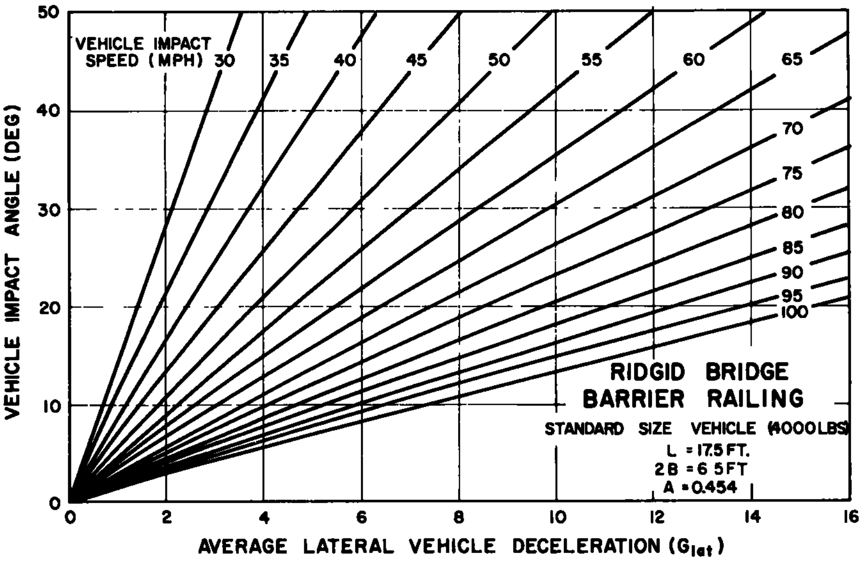


Figure 10. Relationship between impact angle, average lateral deceleration, and impact speed.

In Figure 13, it is evident that the lateral decelerations of a vehicle are reduced as a semi-rigid-type barrier railing is displaced laterally. A reduction in vehicle decelerations is certainly desirable; however, displacements of the barrier railing greater than 2 ft would not be particularly advantageous. This is quite evident from Figure 13, which illustrates that as the displacement increments of the barrier railing increase for any one fixed width of roadway traversed by an out-of-control vehicle, the corresponding lateral deceleration increments of the vehicle decrease. For example, assuming that the impact conditions are as shown in Figure 13, the lateral decelerations of a vehicle, which traversed a 30-ft width of highway, would decrease by 2.6 *G*'s during the first 2 ft of displacement; whereas, during the displacement from 2 to 3 ft, the deceleration would decrease by only 0.6 *G*'s.

Although other parameter studies * of Eqs. 1, 5, and 7 could have been presented, it will be shown that the graphs in Figures 10 through 13 are sufficient to demonstrate the practicality and advantages of predicting a lateral impact force which is required for a bridge design engineer to achieve a structurally sound barrier railing design under any type of estimated impact conditions.

The *1965 AASHO Standard Specifications for Highway Bridges* (*24*) require that a bridge railing shall be designed for a transverse load of 10 kips by elastic methods using working stresses for the appropriate material used, or the bridge railing configuration shall be successfully tested by full-scale dynamic tests. Research agencies conducting full-scale bridge railing tests have more or less followed the guidelines suggested by *Highway Research Board Circular 482* (*3*) for guardrail; that is, a test shall be conducted at a speed of 60 mph at an angle of 25° using a standard size 4,000-lb (±200 lb) passenger vehicle. Under these test conditions, referring to Figure 10, an average lateral vehicle deceleration of 7 *G*'s is obtained. Thus, for a vehicle weight of 4,000 lb, a lateral impact force of 28 kips is predicted by Eq. 7. It should be noted that a vehicle weighing 5,000 lb, and having identical dimensions and location of center of gravity, would produce a lateral impact force of 35 kips.

Maintaining an impact angle of 25°, and referring to Table A-2, California has conducted the majority of their full-scale dynamic tests on rigid-type bridge barrier railing in the speed range of 65 to 75 mph. These severe impact conditions would correspond to the speed range in which the largest percentage of fatal accidents had occurred on high-speed highways as given in Tables 3 and 4. However, it is of interest to note that, at impact speeds of 60 mph and 70 mph and an impact angle of 25°, an out-of-control standard-size passenger vehicle must traverse the widths of roadway in advance of striking a rigid barrier railing given in Table 11. Thus, for a coefficient of friction of 0.7 between the vehicle tires and roadway, which is a reasonable value for a dry concrete surface, the impact conditions of 60 mph and 25° would be representative of a collision that could occur on a four-lane divided highway or a two-lane highway in which the traveled lane widths are 12 ft and the shoulder width is 10 ft, whereas an increase in speed to 70 mph would be representative of a collision that could occur on a six-lane divided highway. It is also evident, in Table 11, that as the coefficient of friction values decrease the width of roadway traversed by a vehicle would increase. These findings indicate that the selection of full-scale dynamic test conditions, such as the dimensions, weight, and speed of a vehicle, should be based on a representative width of highway in an area in which the barrier railing is to be located. This is basically the approach that Graham (*10, p. 90*) used in selecting the impact conditions of 60 mph and 25°, and 45 mph and 35° in conducting

* This topic of parameter studies is discussed in Chapter Five as possible future research.

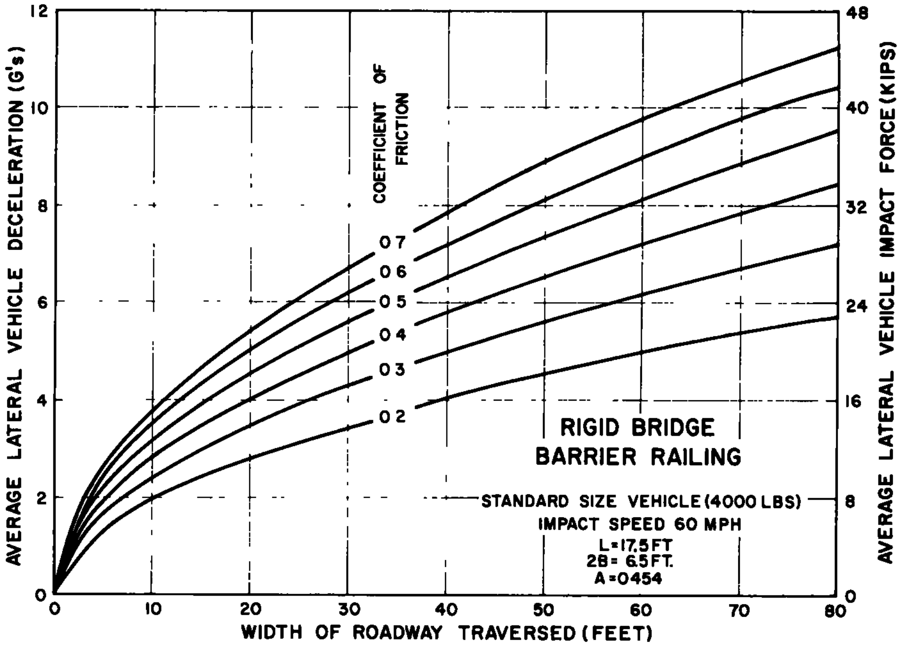


Figure 11. Rigid barrier impact forces at 60 mph.

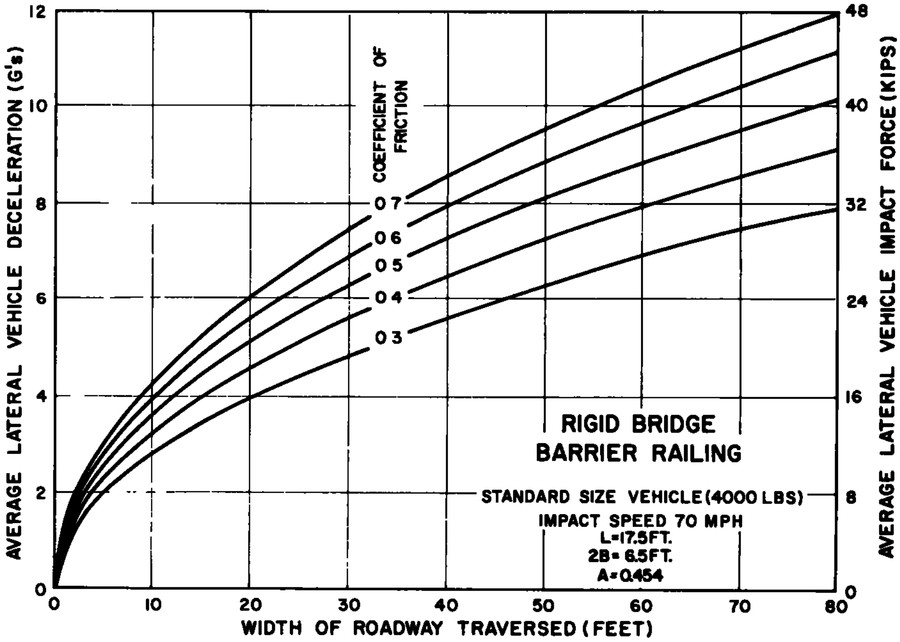


Figure 12. Rigid barrier impact forces at 70 mph.

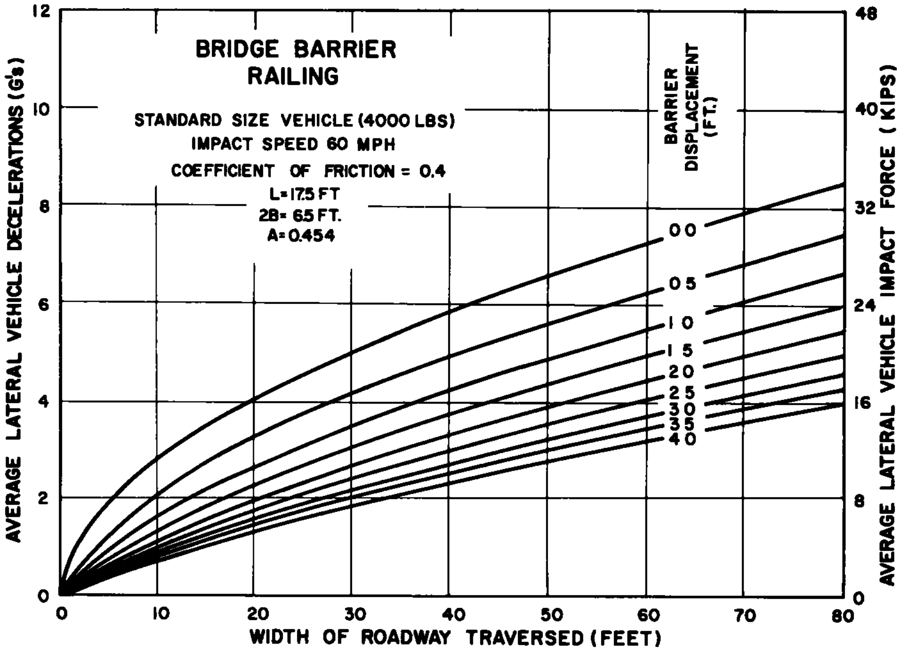


Figure 13. Displaceable barrier impact forces.

48 full-scale tests on various barrier railing design configurations which would be located on a two-lane highway.

In 1962, the Bureau of Public Roads (BPR) prepared a bridge barrier railing specification as a proposed substitute for the *1957 AASHO Standard Specifications for Highway Bridges*, which had been considered inadequate for the protection of traffic. The *BPR Proposed Specification* required that a bridge barrier railing shall be designed using plastic theory for a transverse load of 30 kips. In selecting the 30-kip load, BPR states that reference was made to designs developed by these specifications which full-scale tests and experience indicated would be adequate to resist the usual anticipated forces of impact. To take advantage of the increase in strength of steel and concrete when subjected to dynamic loads, dynamic stress coefficients were also specified.

Under the test conditions of 60 mph and 25°, the 30-kip design force contained in the *BPR Proposed Specification* is predicted by Figure 10 for a vehicle weighing 4,280 lb. Thus, it is evident that for a vehicle having the dimensions shown in Figure 10 and weighing 4,280 lb, the 1962 BPR specification, the lateral impact force predicted by Eq. 7, and the conditions of *HRB Circular 482* are equivalent.

Written comments received from and discussions held with highway bridge design engineers in several of the states visited suggested that designing in accordance with *1965 AASHO Standard Specifications for Highway Bridges* would result in a barrier railing design of adequate strength to restrain the majority of passenger vehicles in operation on high-speed highways. Furthermore, it appeared that a bridge barrier railing designed for a transverse force of 10 kips by elastic methods using allowable working stresses was equivalent to a plastic design, using a dynamic stress coefficient, based on the 30-kip transverse force suggested in 1962 by BPR. In Appendix C, the following mathematical relationships are determined between the transverse design force of AASHO and BPR.

$$P_A = \begin{bmatrix} 0.345\,P_B \ldots \textit{Pipe Rail} \\ 0.382\,P_B \ldots \textit{Box Rail} \\ 0.212\,P_B \ldots \textit{Post} \\ 0.333\,P_B \ldots \textit{Specified for Rail and Post Members} \end{bmatrix} \qquad (12)$$

in which

P_A = *AASHO transverse force (10 kips)*

P_B = *BPR transverse force (30 kips)*

It is now apparent, from Eq. 12, that the loading criteria of AASHO are for all practical purposes equivalent to BPR with regard to the railing members, whereas the AASHO loading criteria are more severe than that of BPR for the post members. To re-emphasize: the 1962 BPR Proposed Specification *is based on plastic design methods using a* dynamic *stress coefficient, whereas the* 1965 AASHO Specification *applies a reduction factor of 3 to the BPR transverse force of 30 kips and then specifies that a design be based on allowable working stresses for the appropriate material. Thus, in an indirect manner, the* 1965 AASHO Specification *in reality results in a design in which permanent deformations occur.*

TABLE 11

LATERAL DECELERATIONS OF VEHICLE AS A FUNCTION OF ROADWAY CHARACTERISTICS

RIGID BRIDGE BARRIER RAILING (IMPACT ANGLE 25°)

		ROADWAY CHARACTERISTICS	
IMPACT SPEED (MPH)	LATERAL DECELERATION [a] (G'S)	COEFFICIENT OF FRICTION	WIDTH OF ROADWAY TRAVERSED [b] (FT)
60	7.0	0.7	33
60	7.0	0.6	37
60	7.0	0.5	45
60	7.0	0.4	56
70	9.5	0.7	48
70	9.5	0.6	57
70	9.5	0.5	69
70	9.5	0.4	80+

[a] Lateral deceleration of vehicle as predicted by Eq. 3, which is graphically presented in Figure 10.

[b] Width of roadway traversed as predicted by Eqs. 1 and 5, which are graphically presented in Figures 11 and 12.

Railing and posts properly designed according to the specification of AASHO or BPR would appear to be of adequate strength to restrain an out-of-control standard size vehicle traveling at 60 mph and striking a rigid barrier railing at an angle of 25° or less; however, it will be demonstrated later that connections, such as a fillet weld post-base plate connection, designed by either of these two methods may not possess adequate strength to prevent localized or complete failures.

As stated earlier, the *1965 AASHO Standard Specifications for Highway Bridges* require that a barrier railing shall be designed based on elastic theory for a transverse load of 10 kips, or the barrier railing configuration shall have been successfully tested by full-scale dynamic tests. Employing the mathematical model equations, and based on the findings of a critical evaluation of the California Type 8 bridge barrier railing, presented in Appendix D, an attempt will now be made to establish the relation between the two design alternatives presented by AASHO. The California Type 8 bridge barrier railing design was selected for evaluation because it had been tested by full-scale dynamic tests in accordance with *Highway Research Board Circular 482*, and the investigators had provided information concerning the behavior of the barrier railing and the crash vehicle under full-scale crash test conditions. Using a lateral impact force computed by Eqs. 5 and 7 for the conditions under which the tests were conducted,* it was demonstrated by a stress analysis that the structural tube rail members designed according to the *1965 AASHO Standard Specifications for Highway Bridges* were approximately

* In the two California tests evaluated, Test 112 was conducted at an impact speed and angle of 58.5 mph and 25°, whereas Test 113 was conducted at an impact speed and angle of 61.5 mph and 23°.

at the yield strength of the material used. Plastic deformations of the rail members, continuous over two intermediate posts, were also visible in the sequence photographs presented in the report by the California investigators. If plastic deformations are desirable, these findings suggest that a design based on the AASHO 10-kip force would perform satisfactorily when subjected to the *HRB Circular 482* full-scale dynamic test conditions of 60 mph and 25°. However, the rail components of the barrier would require considerable repairs subsequent to a high-speed collision.

As long as structural continuity of the rail members is maintained by adequately designed splice connections, it is evident from photographic observations of actual failures of bridge railing that the weak link in most designs is usually located in an area of a post connection. Typical failures observed include: (1) weld failures between the post and base plate, and (2) anchorage failures between the base plate and the concrete parapet wall, or bridge deck. In evaluation of the California Type 8 barrier railing, it was observed that localized fillet-weld failures had occurred in the area of the fabricated two-steel rectangular post plates and the base-plate connection; also, excessive plastic deformations of the base plate had occurred. A stress analysis indicated that the fillet-welds were at approximately the allowable working stress level when designed in accordance with AASHO; however, recalling the relationship established between AASHO and BPR, Eq. 12, in which it was determined that the AASHO loading on the posts was more severe than that of BPR, it could have been predicted prior to conducting the full-scale tests that the stress level in the area of the fillet-welds would be in the plastic range. Because of the complex nature of the stress distribution in an area of a connection, a theoretical analysis would at best be a rough approximation; therefore, if these localized connection failures are undesirable because of safety and/or maintenance considerations, it is suggested that an adjustment in the design load of AASHO be made when designing a connection to restrain a vehicle under the impact conditions of 60 mph and 25°. Further discussion of this topic will be presented in Chapter Four, "Interpretation, Appraisal, and Application." Analyzing the situation now from a different viewpoint, it is to be emphasized that localized failures, which would have essentially the same effect as a plastic hinge (mechanism) forming, would appear to be highly desirable in order to reduce the lateral decelerations of the colliding vehicle. It would also appear that if one attempts to achieve a plastic hinge in a barrier railing design, one must have a method, such as the mathematical equations presented in this report, to determine the magnitude of the actual impact force, which takes into consideration the vehicle and roadway characteristics.

At the present time, the specifications of AASHO (1965) or BPR (1962) are considered by the research agency engineers not sufficient to provide the design engineer the assurance that localized failures will or will not occur in areas of a connection unless, however, full-scale dynamic tests are conducted.

BRIDGE RAIL SERVICE REQUIREMENTS AS A BASIS FOR DESIGN CRITERIA

The primary objective of this research study was to prepare a list of service requirements for bridge rail systems to serve as a basis for design criteria, so one of the first tasks undertaken was the preparation of such a list; and during the course of the study this list has been continuously reviewed and revised. The revisions have been made to correspond with the level of understanding of information obtained from available literature, discussions with engineers, and development of the rational design approach discussed in another section of this report. Some of the service requirements are widely accepted, whereas others are controversial.

It is recognized that any attempt to prepare such a list implies a presumptive attitude on the part of those preparing the list. The researchers have presumed to prepare a list of service requirements, and present them to the reader as a definition to stimulate discussion and provoke evaluations of present day designs. The list can serve as a guide for examining existing rail designs, and can serve as a basis for preparing design criteria for current needs. A sincere attempt has been made to avoid presumptiousness in preparing this list.

A list of ten service requirements for bridge rails follows, and a brief commentary on each of the ten requirements is presented in Chapter Four.

Bridge Rail Service Requirements

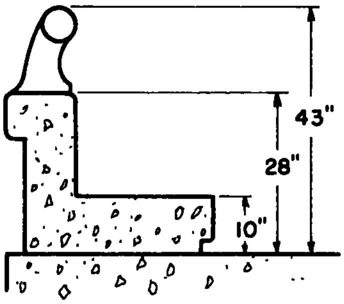
1. A bridge rail system must laterally restrain a selected vehicle.
2. A bridge rail system must minimize vehicle decelerations.
3. A bridge rail system must smoothly redirect a colliding vehicle.
4. A bridge rail system must remain intact following a collision.
5. A bridge rail system which serves vehicles and pedestrians must provide protection for vehicle occupants and pedestrians.
6. A bridge rail system must have a compatible approach rail or other device to prevent collisions with the end of the bridge rail system.
7. A bridge rail system must define yet permit adequate visibility.
8. A bridge rail must project inside the face of any required curb.
9. A bridge rail system must be susceptible of quick repair.
10. The foregoing requirements must be met by giving emphasis first to safety, second to economics, and third to aesthetics.

EVALUATION OF FULL-SCALE CRASH TEST RESULTS

An objective of this study was to attempt a comparison of selected bridge barrier railing systems. During the past twenty years, full-scale crash tests have advanced engineering technology concerning the dynamic behavior of vehicle-barrier impacts. Many crash tests have been conducted on

Reference Nordlin, E.F., Field, R.N., and Hackett, R.P., "Dynamic Full-Scale Impact Tests Of Bridge Barrier Rails," Highway Research Record, No. 83, 1965, pp 132-68.

Sketch of Barrier and Dimensions



Type Barrier Rigid Type 2

Material
Posts Cast Alum.
Rails Alum.

Post Spacing 10 (ft)

Test No. B-1
Test Date 9-27-62

Parameters

Vehicle ..... Type 1960 Dodge 4-Dr. Sedan Wt. 4,300 (lbs)
Vehicle Impact Speed 76 (mph) Impact Angle 25 (deg)

Decelerations (G's)

Dummy (As Reported)	Lat. Not available	Long. Not available
Vehicle (As Reported)	Lat. N. A.	Long. N. A.
Mathematical Model (Avg)	Lat. 11.2 (.13s)	Long. 3.3 (μ=0.3)

Damage

Vehicle NSC Damage Rating 7 (See photo in HRR 83, p. 151).

Barrier ... Parapet Minor cracking-spalling
Posts Minor (one-post)
Rails Minor (one-section)

Barrier

Retain Vehicle	Yes	Vehicle Exit Speed	N. A. (mph)
Elements Dislodged	No	Vehicle Exit Angle	N. A. (deg)
Lateral Movement	None	Appearance	Good
Vehicle Progression	Smooth	Visibility	Good
Vehicle Rise/Roll	Slight	Maintenance	Low

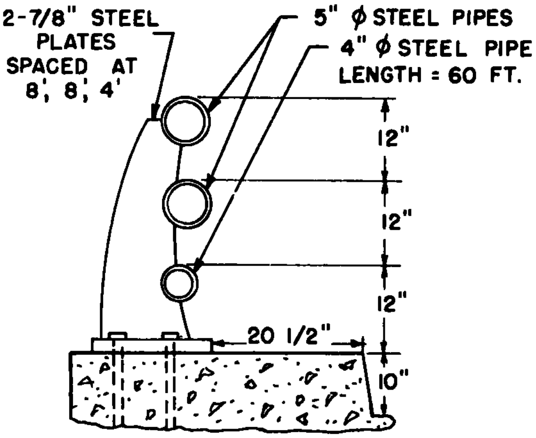
Comments

Equation (1) indicates vehicle would cross 52 feet of pavement at 76 mph to strike barrier at 25 degrees (f = 0.7). NSC Damage Rating of 7 and deceleration of 11.2 g's indicate injuries would occur in 100% of accidents (see Figure 7, Chapter 2). It is estimated that fatalities would occur in most accidents under these conditions. Barrier has adequate strength characteristics.

Figure 14. Evaluation and reporting of full-scale tests (California Test B-1).

Reference Graham, M.D., "The Practical Application of Theoretical Design" Highway Research Record No. 174, 1967, p 139

Sketch of Barrier and Dimensions



Type Barrier Rigid

Material
- Posts: Steel
- Rails: Steel

Post Spacing 8, 8, 4(ft)

Test No. 8
Test Date -------

Parameters

Vehicle ..... Type 1957 4-Dr. Ford Sedan Wt. 3,800 (lbs)
Vehicle Impact Speed 58 (mph) Impact Angle 27 (deg)

Decelerations (G's)

Dummy (As Reported)	Lat. Not available	Long. Not available
Vehicle (As Reported)	Lat. 10.1 (.05s)	Long. 4.7 (.30s)
Mathematical Model (Avg)	Lat. 6.8 (.18s)	Long. 2.3 (μ=.34)

Damage

Vehicle Damage Rating 7 (See photo. in HRR 83, p 170)

Barrier ... Parapet -----
Posts None
Rails Slight

Barrier

Retain Vehicle	Yes	Vehicle Exit Speed	46 (mph)
Elements Dislodged	No	Vehicle Exit Angle	10 (deg)
Lateral Movement	0.2 ft.	Appearance	Good
Vehicle Progression	Fair	Visibility	Good
Vehicle Rise/Roll	Slight	Maintenance	Low if components galv.

Comments

Test conditions were typical for two-lane highway with 10 foot shoulder (vehicle crosses approximately 35 feet). An NSC Damage Rating of 7 and a deceleration of 8-10 g's indicate injuries would occur in 75% of the accidents (see Figure 7, Chapter 2). Barrier railing is strong enough to restrain vehicles on most highways. The curb aggravated vehicle damage. The four inch lower rail offset from the post was insufficient to prevent contact of vehicle with posts.

Figure 15. Evaluation and reporting of full-scale tests (New York Test 8).

vehicle-barrier rail configurations; some on guardrails and some on bridge rails. The results of full-scale crash tests have been reported in such publications as the *Highway Research Record, Highway Research Board Proceedings,* and departmental reports prepared by highway engineers. The researchers have had access to departmental reports which were furnished by the California Division of Highways and the New York Bureau of Physical Research. Other departmental reports of crash test results may be available, but the researchers are unaware of them.

An evaluation form was developed to permit systematic examination of available information, to summarize the results of full-scale crash tests, and, where possible, to make comparisons with predictions of the rational analytical approach discussed earlier in this report. Evaluation forms were prepared on more than 20 selected railing systems; two examples are shown in Figures 14 and 15. These are typical examples to illustrate the evaluation concept, and are not intended to establish relative merits of the two systems. Comments presented in the two figures are based on the researchers' present understanding of bridge rail performance. In a brief study, such as the present one, it has not been possible to reach uncontroversial conclusions; but the comments can serve as a guide for future evaluations. A starting point has been established.

Several highway departments have furnished detailed drawings of their bridge railing systems, and an attempt has been made to evaluate some of these. On systems which have not been crash tested, it would be unfair to publish evaluations at this time. In fact, present knowledge leads to the belief that proof tests will be required in the foreseeable future in order to make impartial evaluations of selected systems. Judicious application of the bridge rail service requirements presented herein, in conjunction with the rational analytical approach can serve as a method of eliminating unpromising design concepts, and in selecting testing conditions consistent with roadway characteristics. So an immediate usefulness of the evaluation technique is apparent.

To illustrate the usefulness of the evaluation technique, one might consider the bridge rail system evaluated in Figure 14. Figure 13 shows that a vehicle traveling at a legal speed of 70 mph would be restrained by the rigid barrier system ($D = 0$). However, if the vehicle traversed three lanes and a shoulder (approximately 42 ft) it is apparent that the average lateral deceleration would be 6 G's. Figure 7 shows that injuries could be incurred in 60 percent of such collisions. The service requirement that vehicle decelerations be minimized has been satisfied, but with the understanding that some injuries can be expected. Smooth redirection of the vehicle was observed in photos from the crash test, so another service requirement has been satisfied.

To further minimize decelerations, it will be necessary to develop an impact attenuation device to provide lateral displacement of the barrier system. Further study of Figure 13 indicates that a lateral displacement of 2 ft would result in a much more tolerable deceleration level, other conditions remaining unchanged.

The previous discussion illustrates the intended use of the evaluation forms. It also convincingly demonstrates that more study is needed to establish quantitative evaluation of selected systems, and this observation is responsive to another objective of this study: to recommend a program for needed research. Recommendations for additional research are contained in Chapter Five.

CHAPTER THREE

ENGINEERING-ECONOMY ANALYSIS OF BRIDGE RAILS

There are two principal methods of combining safety, economics, and aesthetics in a comparison of bridge rail designs. Under one method the benefits to motorists of different designs are calculated in dollar terms. These dollar benefits, together with the costs of different designs, are stated in terms of a benefit-cost ratio or a rate of return. Under the other method the benefits to motorists of different rail designs are stated in terms of measures of effectiveness; and the over-all analysis is called a cost-effectiveness analysis. In general, the measures of effectiveness used in a cost-effectiveness analysis would be the same as those used in a benefit-cost or rate-of-return analysis, but no attempt would be made to state the effectiveness in dollar terms. Before further discussing these methods of analysis, discussions of measures of effectiveness and of cost are given.

MEASURES OF EFFECTIVENESS

Some measures of effectiveness which may be used to compare bridge rails are:

(1) The severity of vehicle accidents with different bridge rail designs.
(2) The inconvenience, loss of time, and increase in operating costs, to motorists, which are related to the

amount of time which is spent repairing or maintaining bridge rails.

(3) The "feelings" of comfort and safety which motorists have with different designs.

(4) The aesthetic appeal of different designs, i.e., how they appear to the motorist and other people who see the bridge rail including not only the appearance when initially installed but also the expected appearance over the entire life of the bridge rail.

(5) The ability of motorist and passengers to see over, and possibly through, the bridge rail so that they can see whatever there is to see while traveling on the bridge.

These measures of effectiveness vary not only with the design of the bridge rail but also with the amount and type of traffic which uses the bridge, other design characteristics of the bridge such as whether the bridge is crown width, and the "environment" in the vicinity of the bridge.

It is not possible with the present state of knowledge to assign dollar values to items (3), (4), and (5) in the listing. Item (3) can be partially estimated by watching the path of vehicles as they approach and cross bridges; if the motorists steer away from the bridge rail, then they evidently feel uncomfortable or unsafe, because of the proximity and/or the design of the bridge rail. It is also possible to quantitatively measure one aspect of item (5), that being the amount of landscape that passengers can see when sitting in a particular position in a specific vehicle. Simply knowing the amount or percent of landscape that a vehicle passenger can see, however, is not sufficient for evaluating this item. There is the further problem of determining the value of seeing the landscape from a particular bridge. This value is probably not only related to what is seen. If it is a stream-crossing in a drab countryside, the motorist wants to see it. If it is a cross-over structure, he wants a view from the elevated position. If it blocks his view of the city dump, he wants to see what he is not permitted to see. In general then, the motorist should be permitted to see from the bridge, whether or not the landscape is "beautiful." This factor probably is more important the longer the bridge and is interrelated with item (3).

Item (4), the aesthetic appeal of different bridge rail designs, probably is most important in how it affects item (3), the feelings of safety and comfort which motorists have with the different designs. Other than that, there are two factors which might be considered under the aesthetics of the bridge rail: the shape of the rail, and the material from which it is made. The material from which the rail is made is important not only in how it looks initially but also in how it changes and whether its appearance can be maintained over time. Rail which is galvanized or which has some coating such as the bituminous undercoat with colored ceramic topcoat described by Mallot (*32*) probably is preferable to painted rail. Mallot maintains that this new coating system used in Indiana is aesthetically superior and also increases safety.

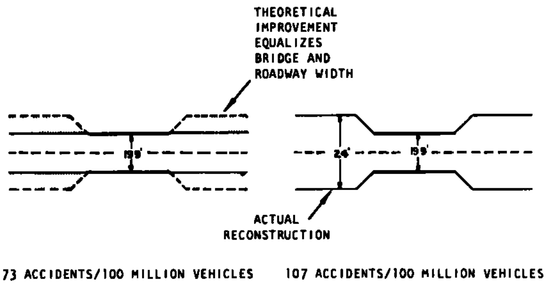
To fully evaluate the effects of bridge rail design on accidents, it is necessary to predict the number of accidents of different types which are expected with, or as a result of, bridge rails and also to predict the severity of accident which is expected for each type of accident. The type of accident would be different for (1) different points at which the bridge rail is hit, (2) different angles of impact, (3) different velocities of impact, (4) different vehicle characteristics, (5) different numbers of persons in the vehicles, and (6) different types of restraining devices used by vehicle occupants.

The number of bridge rail accidents would be higher: (1) the larger the volume of traffic, (2) the more slippery the pavement near and on the bridge, and (3) the closer the bridge rail to the traffic lanes. The number of accidents probably would be higher also for certain roadway geometrics and weather and lighting conditions.

Laughland, Dietz, et al., (*33*) reported two studies, one by Gunnerson (*34*) and the other by Williams and Fritts (*35*), which show that bridge widening decreases the total number of accidents and also that increasing the roadway width to 24 ft while leaving bridge width at about 20 ft leads to a considerable increase in accidents (see Figures 16 and 17). From the two studies, they developed curves relating total accidents, injuries, and property damage, per million vehicles, to the difference between bridge width and roadway width; these are shown in Figures 18, 19, and 20. From these figures they developed Figures 21 and 22 showing the percent reduction in accidents, property damage, and injuries which might be expected from widening bridges. It should be stressed that these figures are based on limited information and are mainly for narrow bridges. The accident, injury, and property damage rates do not take into account the type of bridge rail or the length of the bridge. If figures similar to Figures 18, 19, and 20 could be developed for different roadway widths, bridge rail designs, roadway geometrics, bridge geometrics, and traffic characteristics, then a thorough comparison of different bridge rail designs could be made.

The fact that lower accident rates might be expected the farther the distance from the pavement of the bridge rail is also evident in studies which have shown the maximum lateral distance from the traffic lane traveled by vehicles which run off the road (*36, 37*). It might be possible to use the information in such studies to predict the proportion of vehicles which will hit bridge rails placed at different distances from the travel lanes. Hutchinson and Kennedy (*37*) also give information which might be used to predict the angles at which vehicles will hit bridge rails. It may be found that, although total number of accidents and total accident costs are lower the wider the bridge, the average cost per accident with the wider bridges is higher; at least this may be the case for the angular bridge rail accidents. This is not meant to imply that bridges should not be widened but instead is meant to indicate that even after bridges are widened considerably there will remain some of the more hazardous accidents.

For evaluating bridge rails which are currently being used, useful information may be available from accident records. Glennon and Tamburri (*1, p. 202*) give information on the severity of bridge rail and guardrail accidents in California. Many of the accidents were with spring-mounted curved metal plate bridge rails but also included



PROJECT	CASUALTY ITEM	AVERAGE REDUCTION
Equalize bridge-roadway relative width	Total accidents	22%
	Injuries	43%
	"Bridge-hit" accidents *	36%
	Amount of property damage	47%

* Accidents at bridge "ends"

Figure 16. Results of equalizing bridge and roadway widths (33, p. 197).

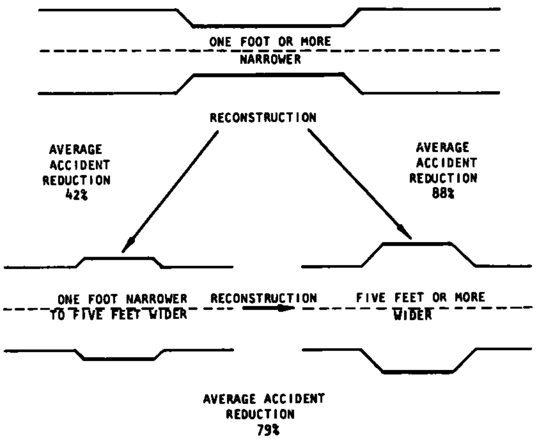


Figure 17. Widening of bridge reduces average accident experience (33, p. 197).

were accidents with a W-section corrugated beam bridge rail. Table 12 gives the number of fatal, injury and property-damage-only accidents. Also given in the table are accident costs calculated using $400 as the average cost of a PDO accident and $2,000 as the cost of an injury accident; two costs are used for fatal accidents, $6,000 and $40,000. The higher value is an estimate of the cost of a fatal accident including as a cost the present worth of future net earnings of the deceased, whereas the lower, $6,000 value includes only direct costs. The variation in cost by type of guardrail accident resulted from some vehicles hitting not only the guardrail but also the protected object. Using the higher average cost values, it is clear that a considerable amount of expense can be justified for protecting bridge rail ends, if the expected number of bridge-rail-end accidents is 8.6 per billion vehicles, as is indicated by Glennon and Tamburri (*1*). It should perhaps be mentioned that the cost of an injury accident probably is higher for accident types which also have a higher fatality rate, whereas all injury accidents were assumed to cost $2,000, whatever the fatality rate for that accident type, in these calculations.

Accident records may also be useful in predicting the number and type of accidents with new bridge rail designs. To fully evaluate new rail designs before they are installed and accident experience becomes available, however, it is necessary to be able not only to predict the numbers and types of accidents but also to predict what the severity of each of these types of accidents will be with the new design. Correlating the amount of vehicle damage and other in-

TABLE 12

NUMBER OF ACCIDENTS BY TYPE AND AVERAGE ACCIDENT COSTS FOR BRIDGE RAILS AND GUARDRAILS IN CALIFORNIA

	NUMBER OF ACCIDENTS				ESTIMATED AVERAGE COST PER ACCIDENT ($)	
FIXED OBJECT TYPE	FATAL	INJURY	PDO	TOTAL	I[a]	II[a]
Bridge rail ends	19	79	25	123	2,300	7,500
Guardrail at:						
(1) Bridge-rail ends	16	191	199	406	1,300	2,600
(2) Abutments and piers	8	36	28	72	1,800	5,600
(3) Light poles	1	23	13	37	1,500	2,500
(4) Steel sign posts adjacent to shoulder	1	36	31	68	1,300	1,800
(5) Steel sign posts in gore area	15	220	116	351	1,600	3,100

[a] Estimate I assumes cost of a fatal accident to be $6,000; Estimate II assumes cost of a fatal accident to be $40,000; both Estimate I and Estimate II assume cost of an injury accident to be $2,000 and cost of a Property Damage Only (PDO) accident to be $400. Averages are rounded to nearest $100. Source for number of accidents is (*1, p. 202*).

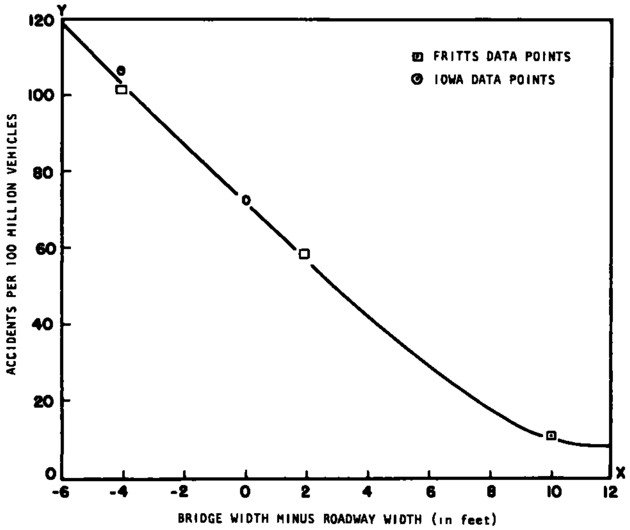


Figure 18. Accident experience at various bridge-roadway relative widths (33, p. 199).

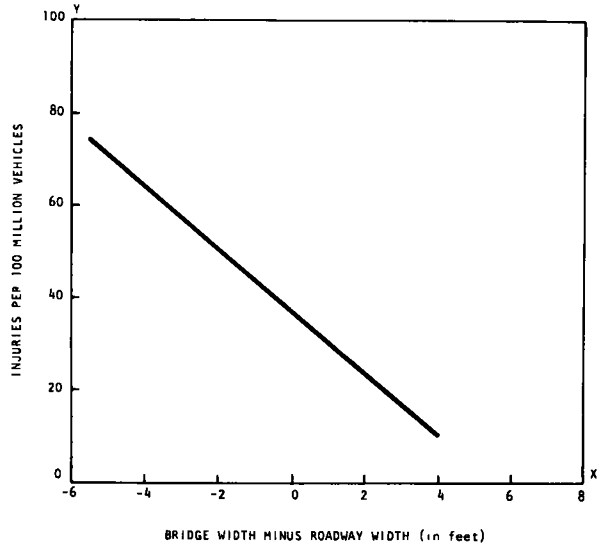


Figure 19. Injury experience at various bridge-roadway relative widths (33, p. 199).

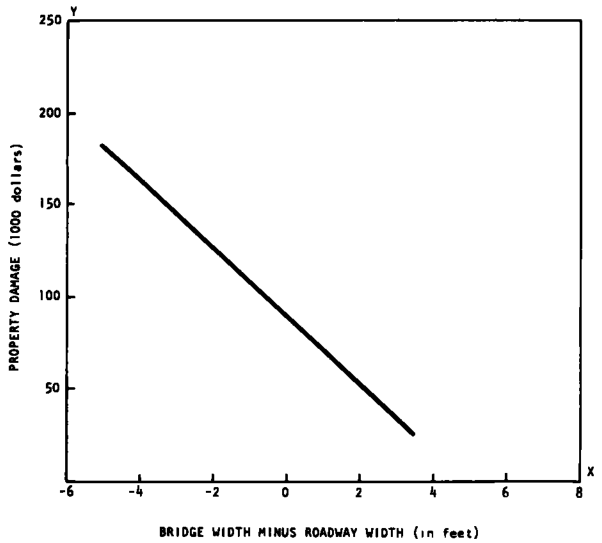


Figure 20. Property damage experience at various bridge-roadway relative widths (33, p. 200).

formation from crash tests involving new designs with actual accidents may be the best way to estimate the cost of accidents involving new designs. This possibility is discussed in another chapter of this report.

Because there is a large number of variables defining accident type, the number of crash tests needed to cover all possible variations would be considerable. Instead of considering the entire range of each variable, it is possible to specify "design" values for each variable and to compare alternate designs at only the stipulated design values for the variables.

The losses of time and increase in operating costs of motorists which are related to the amount of time spent repairing or maintaining bridge rails can be predicted if traffic volumes and bridge and highway geometrics are given. In general, the loss of time and increase in operating costs will be higher, the larger the amount of traffic using the bridge and the longer the time spent repairing the bridge rail, but this will also depend on the way traffic is handled in the repair or maintenance area, the number of traffic lanes, and the width of the bridge. Researchers at the Texas Transportation Institute (*38*) have developed a method of predicting the motorists' time and operating costs associated with repairing pavements and this same method could be used to predict motorists' costs associated with bridge rail repair and maintenance. In general, the following steps are necessary for making such predictions for an interval of one year:

(1) Estimate the number of times per year that a bridge rail will be repaired as the result of an accident and the number of times per year that other maintenance will be performed on the bridge rail.
(2) Estimate the number of hours required to perform each bridge rail repair and maintenance job.
(3) Given steps (1) and (2) and an estimate of the number of vehicles using the bridge per hour during the time when repairs and maintenance will be performed (which is usually about 5 or 6 percent of average daily traffic), estimate the number of vehicles that will pass over the bridge while the bridge rail is being repaired or having maintenance performed on it.
(4) Given the amount of traffic passing over the bridge during maintenance and repair work requiring a known amount of time, and given the roadway and bridge geometrics, estimate the number of vehicles stopped and the hours of stopped time. Given approach speeds and speeds at which vehicles pass the

work site, estimate the speed profile for vehicles which are not stopped when they pass the work site.

(5) The time and operating costs of motorists can then be estimated from the speed profiles of the vehicles that pass the work site.

The time and vehicle operating costs associated with bridge rail repair and maintenance work will be relatively high any time a large number of vehicles are affected by the work. It will be especially high if: (1) The bridge rail work necessitates the closing down of one of the two traffic lanes on a two-way, two-lane bridge with a relatively high traffic volume; and (2) the bridge rail work reduces the road capacity below the demand for a length of time. This would usually occur when one traffic lane is closed and the vehicle input is greater than approximately 1,400 vehicles per hour per lane left open, such as is common on urban freeways.

In addition to the traffic delay and increase in operating costs, increased time spent repairing and maintaining bridge rails probably increases the number of accidents, including accidents with highway department personnel and equipment.

To reduce the costs associated with repairing and maintaining bridge rails, they should be designed so that they can be quickly repaired and require little maintenance. The amount of money which can be justifiably spent for such bridge rail design is higher the higher the traffic volumes and traffic speeds on roadway; and, of course, the less such design adversely affects other effectiveness criteria.

In designing bridge rail to reduce accident repair and maintenance time, some considerations might be:

(1) Keep to a minimum the number, and the number of different types, of washers, bolts, and nuts that must be replaced after an accident.

(2) Use items that can be replaced, if possible; at least, do not use items that must be repaired on the bridge if such repairing takes considerable time.

(3) Use materials that require little routine maintenance.

In regard to item 3, a bituminous undercoat with a top coat of colored ceramic granules has been used in Indiana and New York at a cost of $0.27⅓ per linear ft and with an expected life of up to 15 yr. It has been said that this cost is comparable to a two-coat painting application and much below the cost of galvanizing (*32, 39*). Mr. John A. Robertson (*40*) of the New York Thruway Authority said that they had ". . . investigated a variety of materials including aluminum, concrete, and fiberglass guiderail and guiderail coatings ranging from porcelain to bitumen, including many expensive high quality paints. These studies generally led to the same conclusion; either the product was too expensive or it failed to satisfy minimum requirements." They found that in painting their railing, high quality paints were not as good as white zone paint with a small amount of rust inhibiting additive called Penetrol. It was estimated that, although they spent only $0.14 per linear ft per year on painting, proper cleaning, preparation, and painting would cost $0.40 per ft per year. They finally decided on a zinc galvanizing program whereby all railings were taken down, galvanized, and then replaced. Initially it was estimated that galvanizing would cost $0.63 per linear ft but actual production resulted in costs more than double original estimates.

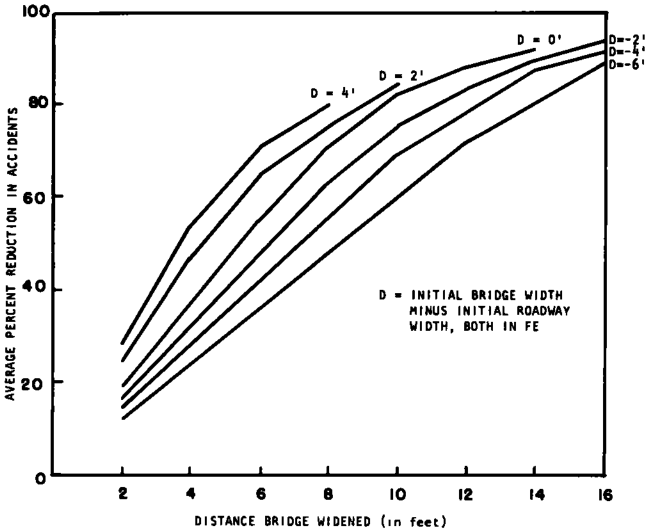


Figure 21. Forecast chart of accident reduction through widening bridges (33, p. 151).

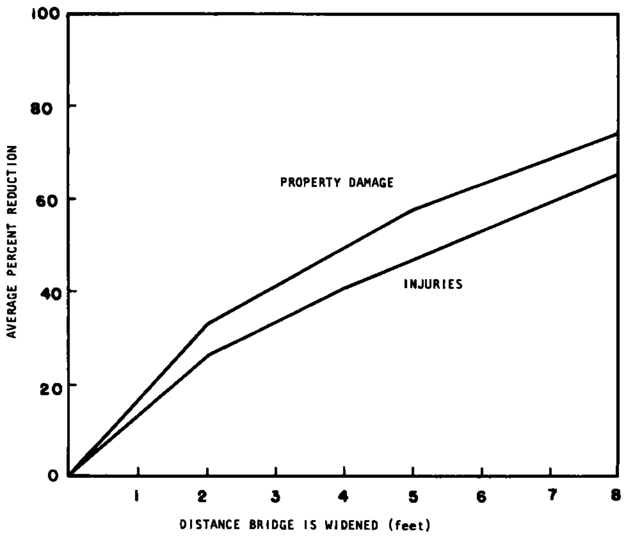


Figure 22. Forecast of fatality-injury and property damage reduction through widening bridges (33, p. 152).

COST OF BRIDGE RAIL

The non-motorist costs of bridge rails include initial costs, maintenance costs, and repair and replacement cost (related to accidents). These non-motorist costs may be expressed as:

$$C_T = C_I + C_M + C_R - S \qquad (13)$$

in which

C_I is total non-motorist costs for the bridge rail,
C_T is the initial installed cost of the bridge rail,
C_M is the routine maintenance cost of the bridge rail, figured over the entire life of the bridge rail,
C_R is the cost of repairing or replacing the bridge rail when damaged in collisions, and
S is the salvage value of the rail at the end of its life.

Each of the above costs may be calculated in "present worth" terms by discounting future costs to the present. Another cost which might be included in the formula is the accident cost resulting from highway department personnel and equipment being involved in an accident while repairing or maintaining the bridge rail. If such costs are not included then they should be considered under accident costs when considering the effectiveness criteria.

In evaluating the cost of different bridge rail designs, it is necessary to consider all costs over the life of the rail. The life of the rail would be the length of time that the rail is expected to stay in the place where it is first located, and this would ordinarily be the anticipated life of the structure. It might be less if it is expected that technological change will bring about replacement of the rail before the end of life of the structure.

The salvage value of the bridge rail depends on the type and the length of time which elapses before it is salvaged. The salvage value will equal the value of the salvaged materials in their new use or when they are sold less the cost of dismantling it and making it usable. Because salvage usually occurs a long time in the future, the present worth of the salvage value will usually be so small (and the differences between designs even smaller) that it can be ignored.

The expected cost of repairing and replacing bridge rail damaged in collisions can be calculated if the number of accidents and the average repair of replacement cost per accident can be calculated.

To calculate maintenance cost it is necessary to be able to predict the interval at which maintenance will be performed and the cost each time it is performed.

Several references in the literature give initial costs of bridge rail and guardrail (*1, 6, 41, 42*), maintenance costs for different types of maintenance (*32, 39, 40*), and average repair costs (*6*). More information on the cost for a particular location and bridge is needed before meaningful analyses can be performed.

In comparing new designs, the engineer should consider the costs of changing designs and also the costs of having several designs. Considerable savings may result if bridge rails or parts of bridge rails can be standardized. Standardization yields economies of large-scale production, reduces inventory holdings, simplifies replacement of damaged parts, and avoids the costs of tooling-up for many designs. Some comments by Morris (*43*) concerning standardization of bolts, washers, and terminal accessories are interesting:

> Today, however, as a result of the headlong rush to apply the recent research findings, there is near chaos. This is particularly true of accessory items that go with this standard product. There is no argument that highway safety is well worth a substantial investment. There can be no argument, either, that we are not getting the greatest return for the invested dollar. In short, we should be getting more safety per tax dollar.
>
> As an extreme example, consider the very simple post bolt and washer. A rectangular washer that is nominally 4 in. by 2 in. fits under the head of the post bolt. A survey of standard plans of 15 states shows that there are 14 variations in dimensions of the washer alone. Fortunately in this particular instance, AASHO has initiated action designed to standardize one specific size and thickness. The tooling cost of $2,000 per size per manufacturer, however, already amounts to a sizable investment.
>
> As a further illustration, requirements for the bolts vary from lengths of 2 in. to 26 in. to increments of 1 in. Unlike its companion washer, standardization on length is not possible because of the requirements for posts which can be wood, steel, or concrete in combinations of no-offset, single blocked-out, and double blocked-out.
>
> Most serious and by far the most costly to the taxpaying highway user is the effect of non-standardization of terminal accessories at bridge abutments and at ends of runs where the rail dips and twists into the ground. . . .
>
> The result has been as many as 40 solutions to the same problem. Each has its own bolt and hole size, slots, brackets, etc. The costly result is that the last section of rail is three to four times the normal price of rail plus the cost of brackets, plates and bolts. In some instances tooling can run as high as $20,000. The price of such tooling must be written off on the first contract for fear that future modifications may make it obsolete.

Standardization should be considered, but it should also be remembered that there may be several standardized designs, each being best for a particular situation.

COST-EFFECTIVENESS ANALYSIS

A cost-effectiveness analysis can be used to evaluate alternate bridge guardrail designs. Cost-effectiveness criteria are the tests used to compare alternatives after the cost and effectiveness of each alternative have been determined. There are two widely used cost-effectiveness criteria; the *equal cost* criterion and the *equal effectiveness* criterion. The equal cost criterion is used to compare alternatives with equal cost—the alternative which is most effective is chosen. The equal effectiveness criterion is used to compare alternatives with equal effectiveness—the alternative which is least costly is chosen.

There are extensions which can be made to the equal cost and equal effectiveness criteria. These extensions are needed for two reasons: (1) when use of the equal cost criterion is attempted and there are several measures of effectiveness, one alternative may not be the best for all measures of effectiveness, and (2) there are often several alternatives with several levels of cost and effectiveness and a criterion for choosing the combined level of cost and effectiveness is needed. To solve the first of these two problems, weights can be assigned to the various measures of effectiveness; this results in one "weighted" measure of effectiveness. If dollars are used as weights, the cost-effectiveness analysis in effect becomes a benefit-cost analysis. To solve the second problem, the decision maker must devise some rule stipulating how much extra cost he is willing to incur to obtain extra benefits. The use of judgment in such an evaluation is no more arbitrary than is a strict use of the equal cost and equal effectiveness criteria. In the "simple" use of the equal cost or equal effectiveness criterion, the subjective evaluation enters the analysis at the beginning of the analysis when the desired level of cost or effectiveness is chosen. In the case where several levels of cost and effectiveness are considered, the final choice of the desired level is subjective (or based on some rule) and this choice occurs at the end of the analysis.

Several dots are plotted on Figure 23. These are lettered, representing alternatives which have different levels of cost and effectiveness. Using the equal-cost criterion, alternative A is inferior to alternative B and alternatives D and C are inferior to alternative E. Using the equal-effectiveness criterion alternative, F is inferior to alternative D because it costs more but gives the same level of effectiveness. Alternative H is inferior to alternative G because it costs more and is also less effective. Therefore, the interesting alternatives are B, E, G, I, and J. The decision maker must subjectively choose one of these alternatives.

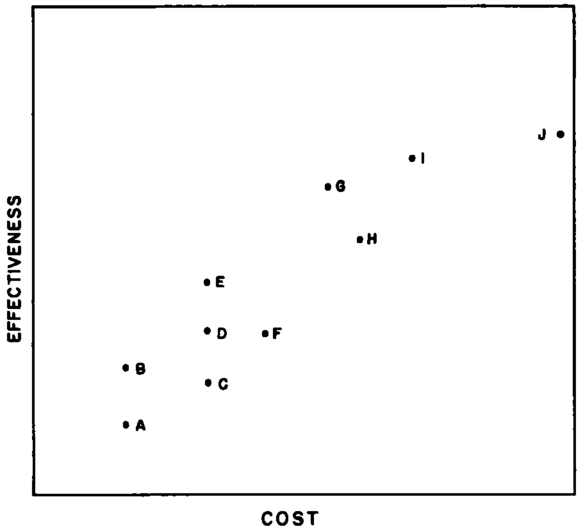


Figure 23. Hypothetical effectiveness and cost for several alternatives.

The choice of the best design becomes more complicated if several measures of effectiveness are used. Some alternatives may be clearly inferior but for others the choice may be more complicated. For example, presume that the measure of effectiveness is related to reduction in accidents severity. It might be the rail displacement when hit by a 4,000-lb automobile with an impact angle of 10° and a velocity of 60 mph. Suppose, however, that a comparison is being made between alternative G and H, but that alternative H is more "beautiful" than alternative G and also when hit by a vehicle requires less time to repair than does alternative G. Thus, H costs more than G and is inferior by one (perhaps the more important) measure of effectiveness, but superior by two other measures of effectiveness. In the absence of some way to explicitly assign weights to these measures of effectiveness, the decision maker must use his judgment in comparing such alternatives.

As mentioned previously, it is possible to assign dollar weights to accidents and the delay and operating costs of motorists associated with repairing and maintaining bridge rails. If only these measures of effectiveness are considered, the reduction in motorist costs afforded by a preferable design can be calculated. Also, incremental decreases in motorists' costs can be compared with incremental increases in the cost of better bridge rails. Such comparisons give an indication of the desirability of spending more for a better design.

In addition to over-all comparisons of alternate bridge rail designs, it is possible to consider specific design elements and to relate these to specific types of accidents. For example, the expected cost of collisions with the bridge end can be compared with the cost of providing an approach rail to the bridge rail end. The expected cost of collisions with approach rails progressing to collision with the bridge end can be compared with the cost of providing strength and alignment transition from the approach rail to the bridge rail. The expected cost of collisions with the approach rail or bridge rail which cause redirection into the bridge rail or oncoming traffic can be compared with the cost of providing a design which gives a small redirection angle. The reduction in cost of angular collisions with the bridge rail resulting from use of a design which displaces laterally can be compared with the extra cost of such a design. The higher the expected speed of impact, the larger the angle of impact, and the larger the expected number of impacts, the more would be the amount of lateral displacement which could be justified on a cost basis. The cost of providing increased strength can be compared with the reduction in accident cost resulting from fewer penetrations of the bridge rail (less any extra cost to non-penetrating vehicles which results from a stronger rail). The expected reduction in accident cost because of decreasing the number of vehicles vaulting the rail can be compared with the cost of (1) providing higher railing, and (2) eliminating discontinuities such as jutting curbs. Providing higher railing would, after a certain height, conflict with the criterion of letting people see over the rail and this should also be taken into consideration.

In making cost-effectiveness analyses or comparisons of motorists' costs with guardrail costs, the analysis will be different according to whether the bridge rail design is for a new bridge or an improvement on an existing bridge. For new bridges, there is the additional consideration of determining the bridge width; at least, there are possible trade-offs between additional bridge width and more sophisticated bridge rail designs. For improving existing railing, the alternative of widening the bridge might also be considered. If old rail is to be replaced, the cost of tearing down the old rail (less its salvage value) should be added to costs.

In the previous discussion, it has been indicated that the optimum bridge rail would depend on several variables which would be different from bridge to bridge. Even though this is true, the number of designs should be limited because of the extra cost of non-standardization. It may be possible, however, to have several over-all designs, each of which uses standardized parts.

In the comparison of designs, the engineer needs to be aware of the design factors which, when changed, will affect costs and effectiveness. Trade-offs between certain design factors may reduce cost and keep effectiveness at the same level. For example, it is possible to reduce post spacing (and/or change post dimensions, thicknesses, shapes, alloys, etc.) and reduce the size of the rail, and by doing so reduce cost, while maintaining the same lateral strength.

CHAPTER FOUR

INTERPRETATION, APPRAISAL, AND APPLICATION

Several tasks have been accomplished during the course of this study, and they are discussed in Chapters Two and Three and in the appendices. A list of bridge rail service requirements was presented in Chapter Two, and the following commentary extends and amplifies this list. The commentary reflects the opinions of the researchers, based on their understanding of the present state-of-the-art and practice of bridge railing design and development.

A general discussion of findings of this report follows the commentary. The commentary and discussion are intended to serve as an interpretation and appraisal of the information contained in this report. Some immediate applications of these research findings are indicated, and some revisions of the *AASHO Standard Specifications for Highway Bridges* are suggested.

COMMENTARY ON SERVICE REQUIREMENTS FOR A BRIDGE BARRIER RAIL SYSTEM

1. *A bridge rail system must laterally restrain a selected vehicle. The bridge barrier rail must contain the vehicle on the structure. The impacting vehicle must not penetrate or vault the barrier.*

COMMENT: The vehicle *selected* during the course of this study was 17.5 ft long, 6.5 ft wide, having its center of gravity 7.94 ft behind the forward bumper point and weighing 4,000 lb; this *selected* vehicle was considered representative of most of the passenger vehicles now in use on highways in the United States.

The *1965 AASHO Standard Specifications for Highway Bridges* provide adequate strength to contain standard size passenger vehicles traveling at 60 mph which collide at angles up to 25°. This is true provided adequate connections are made, and provided further that the barrier rail has a smooth, continuous character which will permit a colliding vehicle to be redirected without snagging or abrupt change from its original course. Collisions with rigid barriers designed under these conditions are capable of producing impact forces on a *selected* colliding vehicle of such magnitude that severe damage to the vehicle will result, and it can thus be inferred that some fatalities will occur as a consequence. However, adequate strength must be provided.

2. *A bridge rail system must minimize vehicle decelerations.*

COMMENT: A rational analytical approach has been presented which can be employed to estimate lateral and longitudinal decelerations in a vehicle-barrier rail collision. An examination of the "Limits of Tolerable Deceleration" section (Chapter Two) of this report provides an estimate of expectation of injuries at various magnitudes of deceleration. It is clear that even at low values of deceleration some injuries can be expected. It should be emphasized that expected injuries will occur at the extreme conditions for which a barrier system is designed, and collisions at lesser conditions will be tolerable.

An absolute value for the magnitude of minimum deceleration has not been established. It may be that an arbitrary limit will have to be established on the basis of probability or economics or a combination of these factors.

3. *A bridge rail system must smoothly redirect a colliding vehicle. Vehicle progression must be smooth following impact; it must not snag or pocket on bridge rail components.*

COMMENT: The first two service requirements describe necessary conditions for railing strength and vehicle deceleration. Methods of estimating impact forces and tolerable decelerations are contained in this report. Careful examination of crash test reports, written comments, and discussions with highway engineers indicate that a vehicle must be safely redirected following a collision with a bridge rail system. Satisfactory redirection implies that (1) the colliding vehicle will be turned parallel to the barrier, and (2) the exit angle will be equal to or less than the impact angle.

Some of the railing configurations shown in Figure 1.1.9 of the *1965 AASHO Standard Specifications for Highway Bridges* do not satisfy this requirement, because parts of the vehicle can come into contact with posts. This figure should be revised, and Article 1.1.9 (A) should be revised to read as follows:

(A) Traffic Railing

Although the primary purpose of traffic railing is to contain a selected vehicle using the structure, consideration should also be given to protection of the occupants of a vehicle in collision with the railing, to protection of other vehicles near the collision and to appearance and freedom of view from passing vehicles.

Materials for traffic railing shall be concrete, metal, timber or a combination. Metal materials with less than 10 percent tested elongation shall not be used.

A smooth, continuous face of rail on the traffic side must be provided with the posts set back from the face of rail. Structural continuity in the rail members, including anchorage of ends is essential. Bolted or welded splice material in the rails will be considered to provide this continuity. The railing system shall be able to resist the applied loads at all locations.

The height of traffic railing shall be no less than 2 ft 3 in., measured from the top of the roadway to the top of the upper rail member. (See Figure 1.1.9.) Railings other than those shown in Figure 1.1.9 are permissible provided the total applied loading is determined by a rational analytical approach, as outlined elsewhere in these specifications. Such railing designs should be verified by full-scale dynamic testing.

Careful attention should be given to the treatment of railing at the bridge ends. Exposed rail ends and sharp changes in the geometry of the railing must be avoided.

4. *A bridge rail system must remain intact following a collision. A vehicle impact must not dislodge any elements of the barrier system. Rails, posts, and concrete must not fall into the traveled way or over the side of the structure.*

COMMENT: This requirement can be realized in practice by over-designing connections in rigid installations; and when breakaway or impact attenuation systems are incorporated into designs, it will also be necessary to satisfy this requirement. It is recognized that small elements may become detached during a collision incident under severe conditions of impact without grave danger to other vehicles or pedestrians. Compliance with this article will produce a rail system which will remain intact during a collision incident (i.e., following the initial impact and until the vehicle loses contact with the rail), and also during the time required to repair the railing system following a major collision.

5. *A bridge rail system which serves vehicles and pedestrians must provide protection for vehicle occupants and pedestrians. Sidewalks must be placed outboard of the vehicle-barrier railing; and adequate pedestrian railing must be provided.*

COMMENT: This requirement separates vehicles and pedestrians. In some locations bridges must serve both vehicles and pedestrians. In the past, an unprotected walkway has been provided on many highway bridges. Such unprotected walkways are certainly no longer permissible on highway bridges.

The first paragraph of Article 1.1.9 of the *1965 AASHO Standard Specifications for Highway Bridges* should be revised to read as follows:

1.1.9—RAILINGS

Railings shall be provided at the edge of structures for the protection of traffic, and for the protection of pedestrians. Where pedestrian walkways are provided adjacent to roadways, a traffic railing must be provided between the two with a pedestrian railing outside.

6. *A bridge rail system must have a compatible approach rail or other device to prevent collisions with the end of the bridge rail system.*

COMMENT: Michie and Calcote (*44*) have prepared a report containing a compilation of recommended practices for locating, designing, and maintaining guardrails and median barriers. The California Division of Highways has conducted a series of fullscale crash tests on approach rails; Nordlin, Field, and Folsom (*29*) prepared a report on these tests which was presented to the Highway Research Board in January 1969. These two agencies and others are currently investigating the zone of transition between flexible guardrail and rigid bridge rail. It is clear that guardrails and bridge rails must be designed as an integrated system.

7. *A bridge rail system must define yet permit adequate visibility. The bridge rail must allow good visibility both above and below the horizontal line of sight of vehicle occupants. The driver's sight distance should not be obstructed by a bridge rail.*

COMMENT: Limits of the roadway are defined by a bridge rail. As a delineator, it has the same attributes as a guardrail or approach rail. The bridge rail must be aligned with approach railings; the bridge width must therefore conform to approaching roadway width. On certain major structures, this width requirement may have to be reduced. Aesthetic requirements are impossible of definition. Whenever possible, it is desirable to construct a bridge railing which conforms to the approach railing configuration. In such installations, the vehicle occupant may not be aware that he has crossed a bridge. This can have a salutary effect on vehicle operators.

This requirement is intended to emphasize the need for roadway designers and bridge rail designers to coordinate their efforts during the design phase. Horizontal and vertical alignment, entrance and exit ramps, and similar geometric factors must be given careful consideration in satisfying this requirement.

8. *A bridge rail must project inside the face of any required curb.*

COMMENT: General agreement exists among highway engineers and researchers concerning the need to locate curbs outboard of the vehicle-barrier rail. On many bridges it is necessary to provide curbs for drainage and other purposes. Therefore, Article 1.1.8 of the *1965 AASHO Standard Specifications for Highway Bridges* should be replaced by the following:

1.1.8—CURBS AND SAFETY CURBS

Curbs, when required, must be constructed outside of the roadway. Where curb and gutter sections are used on the roadway approach, at either or both ends of the bridge, the curb height on the bridge may match the curb height on the roadway approach, or if preferred, it may be made higher than the approach curb. Where no curbs are used on the roadway approaches, the height of the bridge curb above the roadway shall be not less than 8 in., and preferably not more than 10 in.

Curbs widened to provide for occasional pedestrian traffic shall be designated "Safety Curbs." Safety curbs shall be not less than 1 ft 6 in. wide. Curbs more than 2 ft wide shall be classed as sidewalks.

9. *A bridge rail system must be susceptible of quick repair. All elements of a barrier system must be so designed that when repairs are necessary to restore a damaged section, they can be done quickly and with a minimum of special equipment.*

COMMENT: This requirement is intended to remind the design engineer to include details which will facilitate repairs following a major collision with a bridge rail system. The cost of replacing damaged bridge rail components, safety of maintenance personnel, and possible lost time to vehicles using a bridge under repair must be considered. Many engineers are concerned about the economic aspects of future maintenance of bridge rail systems.

10. *The foregoing requirements must be met by giving emphasis first to safety, second to economics, and third to aesthetics.*

COMMENT: This service requirement is in accordance with the generally accepted concept of highway safety as a prime requirement in all designs. It may be necessary to apply an engineering economy analysis to satisfy this requirement. A discussion of this subject is presented in Chapter Three, and ways and means of combining safety, economics, and aesthetics are described.

GENERAL DISCUSSION

The following discussion is presented as a response to the comments and suggestions of engineers in the several highway departments. The researchers have attempted to make their work useful for the needs of today as well as tomorrow.

During the course of this research study, several highway departments were surveyed to acquire information concerning the type of bridge rail installations and operational characteristics of bridge railing systems in these states. Each of the several states has its own bridge railing designs, some of which conform to the requirements of the *1965 AASHO Standard Specification for Highway Bridges* (railing design). Of course, each of the states also has railing in place which was designed in accordance with earlier specifications. Accident information, received from the several highway departments and presented earlier in this report, include collisions with all types of bridge railing. Consequently, some of the accidents reported include penetration of bridge railings which were designed in accordance with earlier specifications.

Considerable detailed discussion has been presented to illustrate the point that rational design of bridge railing systems can be achieved by using a predictable impact force, computed by mathematical expressions such as Eqs. 7 and 8. Through this procedure it is possible to design bridge railing systems having adequate strength to restrain a colliding vehicle of selected dimensions, weight, impact speed, and impact angle. It might be well to reiterate some of the conclusions which have been reached in this report as a means of interpreting and appraising the results of this study.

At this time it is apparent that rigid bridge rails designed in accordance with the *1965 AASHO Standard Specifications for Highway Bridges* have adequate strength to restrain vehicles weighing up to 4,000 lb, traveling at 60 mph, and colliding at 25°; provided that no snagging occurs and that rail height is at least 27 in.

A collision under such conditions would produce an average lateral decelerative force of approximately 8 G's, corresponding to a damage rating of 6 or 7. Information presented in this study (see Figure 7) indicates that 75 percent of the vehicles would have injured occupants following such a collision. However, examination of photos of railing systems which satisfy strength requirements for restraining vehicles reveals that local failures occur in welds at the base of support posts and other local plastic deformation. These are in effect plastic hinges in the material and may reduce damage to the vehicle as a result. Some engineers have suggested that by sizing welds for a specified failure load, a strong system with local weak areas could produce a safer rail configuration. The vagaries of such controlled welding techniques make such a proposal a tenuous one at the present time. However, experience with breakaway devices and frangible inserts suggests the possibility of developing the concept of controlled areas of weakness under collision loads. Service Requirement 9 must be satisfied in developing such concepts because repairs on bridge rails can severely inhibit traffic operations unless such repairs can be effected quickly.

Figure 10 shows that the average lateral vehicle deceleration is plotted as a function of vehicle impact angle for various impact speeds. This figure was prepared by selecting a standard size vehicle having a length of 17.5 ft, and a width of 6.5 ft. It is recognized that there are many vehicles in operation on U.S. highways which are smaller and which weigh less than the selected vehicle shown in Figure 10. Some estimates of the impact force of smaller vehicles using the mathematical expression presented earlier have been made, and it has been found that, for most intents and purposes, these smaller vehicles will strike the rigid barrier and produce an impact force that is approximately equal to the weight of the vehicle times the predicted G force shown in Figure 10. That is to say that a 2,000-lb vehicle will strike with a force approximately one half that of a 4,000-lb vehicle. This is an approximation, and a more correct value for a selected vehicle can be determined by measuring the length and width of the vehicle, locating the center of gravity, and weighing the vehicle. Similarly, in Figures 11 and 12 the average lateral deceleration is plotted as a function of width of roadway traversed. The effect of the coefficient of friction between tires and the road is shown parametrically. These graphs were prepared for a selected vehicle traveling at 60 mph, and 70 mph, respectively. Curvilinear plots result because Eq. 1 has been used to compute roadway widths as a function of impact angle. Similar graphs could be prepared using the mathematical expression for smaller size vehicles or larger size vehicles based on the physical dimensions of the vehicles, and a variety of impact speeds.

It is recognized that if a vehicle is operated at some speed in excess of legal speed limits or design limits, a barrier rail designed in accordance with a rational design approach could be penetrated and, depending on the type of bridge (for example, overpass over a highway, etc.), an undesirable collision could result. The designer would need to take these other factors into account along with the dynamic impact force in making his design. Here again, the economic factors include not only the cost of a bridge rail system to restrain a legally operated vehicle, but also the location of the bridge in question. It seems reasonable to conjecture that if a bridge is located in a rural area, crossing a stream, it might be designed in such a way that one might expect an occasional penetration of a vehicle operated at illegally high speed. At any rate, the rational design approach does permit latitude in making such economic and safety determinations.

There is evidence from accident reports to indicate that, whatever design speed or legally established speed is in effect on whatever class of highway, some drivers will exceed these established speed limits. A difficult question arises whether the designer should spend the money required to restrain a vehicle operated in excess of the legally established limits. Once again, by using the rational design approach, the designer can determine the strength of a barrier rail required to restrain a vehicle at any selected speed in excess of the legal speed limit, and design his rail in accordance with this arbitrarily selected speed. He can make an estimate of the cost of such a rail and compare it with the cost of a rail designed for a legal speed limit, taking into account the number of lanes on the bridge, and the location of the bridge.

Thus, it is apparent that the rational design approach can be adequately used to aid the administrator and the designer in making safety and economic feasibility studies of projected bridge railings. Such an approach should be included in future AASHO design specifications.

It is demonstrable that certain bridges on streets or rural roads of lower speed classification might be safely designed for lower strength requirements than bridge railing to be used on high-speed highways. Other roadway characteristics such as horizontal and vertical alignment, approach conditions, lighting, and condition of pavement surface, appear to be of lesser significance than the parameters considered in the development of the rational analytical approach in this report. However, the selection of a coefficient of friction between the tire and the roadway is something which should be given careful consideration by the design engineer. At the present time some uncertainty exists concerning selection of a coefficient of friction; however, other researchers are studying this facet of highway safety (*45*).

It is apparent from all of the crash test information and information from accident reports that the minimum permissible height for a bridge rail is 27 in. above the roadway. It is the opinion of many highway engineers and researchers that a higher railing is necessary to restrain large trucks in a collision with a bridge railing system. However, at the present time, as has been discussed previously, the incidence of truck collisions with bridge rail systems appears to be small by comparison with collisions by passenger vehicles. It should be emphasized also that smooth railing must be provided (to eliminate snagging), blocking out railing is highly desirable, and the use of a rubbing rail (15 in. above the pavement) is also desirable.

Bridge barrier rails should be aligned with approach rails. That is, shoulder width bridges must be provided on highways. Combination vehicle and pedestrian bridges must have a barrier rail between vehicles and pedestrians, and pedestrian walks must be provided outboard of this barrier rail. Where curbs are required for whatever reason, they must be built outside the barrier rail.

All of the foregoing statements are based on current vehicle designs and are responsive to service requirements to provide for automobile characteristics, roadway characteristics, and rail characteristics for current conditions. If vehicle configurations change, barrier configurations and design should be revised as appropriate.

One of the most important factors in design of a safety bridge rail is in the connections. The connections must be strong enough to restrain a vehicle traveling at the design speed. In such a procedure there will be many different sizes of bolts and other connection components. This could lead to costly installations in many locations. Most railing fabricators would agree with the point of view that uniformity or standardization of railing systems will result in lower costs for fabrication and erection. Here, there is an opportunity to consider the economic feasibility of any bridge rail design. The review of photographs of several bridge railing systems which were tested under full-scale crash conditions indicates that the failure of these systems often occurs at a connection. For example, the concrete spalls around anchor bolts, or anchor bolts are sheared, or the base plate is permanently deformed by the stresses which it must carry; or there are fractured welds at the juncture of base plate and support posts, or the bolted connections at the ends of lengths of railing are fractured. These are examples of inadequate connection design for rigid barriers.

To summarize, it appears that rigid barriers can be built which can restrain passenger vehicles and provide railing heights which vehicle occupants can see over in many installations. Further, such railing design can prove adequate from the viewpoint of reducing the number of fatal accidents involving bridge railing systems. How to provide bridge railing systems which will reduce injury-producing accidents has not been determined at this time. It is clear that impact attenuation devices for bridge rails must be provided to improve safety characteristics, and furthermore that such devices must be structurally compatible with approach railing.

Full-scale crash testing of prototype railing systems meeting the service requirements contained in this report will be necessary for the foreseeable future.

CHAPTER FIVE

CONCLUSIONS AND NEEDED RESEARCH

The procedures developed during this study should be used to develop design criteria for bridge rail systems. Outline specifications should be prepared on the basis of current knowledge from research and from operational information. It is recognized that writing design specifications is not within the province of research. However, it is believed that design specifications should be written on the basis of research and operational information. It is concluded that the rational design approach outlined in this report is a usable technique, and that the relations established for comparing crash test data with accident information (e.g., Figure 7) provide an insight into establishing estimates of the severity of accidents. The evaluation of full-scale crash tests should be implemented by agencies conducting such tests. Such activities should be conducted by a research organization, and the results should be furnished to highway engineers to aid them in their design, operation, and maintenance responsibilities.

The tentative service requirements presented herein should be reexamined and revised as appropriate. Such a task implies that highway engineers could prepare written criticisms of these service requirements, and these comments should be examined carefully in revising bridge rail service requirements.

Research by other agencies on guardrail installations should be reviewed, appraised, and incorporated into the design criteria for bridge rail systems.

Specific tasks are listed in the following paragraphs. These tasks should be accomplished concurrently with the development of design criteria.

OUTLINE OF SPECIFIC TASKS

1. *Verification of Mathematical Model*

It is recommended that further comparisons of the mathematical expressions be conducted to establish the validity of their generality for vehicles having a wider range of dimensions and weight. Such comparisons can be conducted only on the basis of data from full-scale crash tests. Several organizations are engaged in such testing; a cooperative effort to acquire data and to compare them with the expressions should be initiated.

2. *Parameter Studies*

Parameter studies could be accomplished by programming the model for electronic computer simulation of a vehicle-barrier collision.

Vehicle Characteristics

It is recommended that a parameter study be initiated to examine the significance of vehicle dimensions and weight on predicted values of impact force. Preliminary calculations indicate that such a study is needed to establish a selected vehicle for design calculations based on a rational analytical approach. Vehicle dimensions and weight should be described so that future railing designs can take into account changes in vehicle characteristics.

Roadway Characteristics

(1) The effect of coefficient of friction between tires and road should be examined. It has been shown that impact force is related to this parameter (Figures 11 and 12).

(2) The effect of horizontal roadway curvature should be studied. The present study has been limited to collisions which occur on straight roadways. The effect of short radii of curvature may be significant, and the significance should be determined.

(3) Effect of roadway width should be determined. A balance between wider bridges (more area in which a driver can maneuver) and estimated impact force (wider roadways permit greater impact angles, thus greater impact force) should be considered.

Railing Characteristics

(1) It has been observed in this study that the magnitude of lateral barrier displacement need not be great (see Figure 13) in order to reduce the lateral impact force to a tolerable value. Further examination of this observation could be beneficial in developing impact attenuation concepts.

(2) The effect of barrier displacement should be studied parametrically with a view to determining the need for establishing a subroutine for expressing barrier displacement as a function of material and physical properties (Young's modulus, section modulus, etc.). It may be that development of precise expressions for railing behavior is not necessary for purposes of design. Rigorous analytical expressions should be sought after, but they must be responsive to needs and current knowledge.

3. *Rational Design Approach*

This approach should be expanded through parametric studies to provide tables, curves, or nomographs for the design engineer, which, when used with sound economic judgment, should produce safer future bridge rail systems.

4. *Limits of Tolerable Deceleration*

It is recommended that the relationship between crash-test data and accident information (see Figures 7 and 8) be critically evaluated and revised as appropriate. This task could be accomplished by a cooperative effort between NCHRP and the National Safety Council.

Other tasks could be listed, but the foregoing are considered to be of primary concern at this time. The information contained in this report should be used in the next research study which should be aimed at developing design criteria for bridge rail systems.

REFERENCES

1. GLENNON, J. C., and TAMBURRI, T. N., "Objective Criteria for Guardrail Installation." *Hwy. Res. Record 174* (1967), pp. 184-206.
2. DELEYS, N. J., and MCHENRY, R. R., "Highway Guardrails—A Review of Current Practice." *NCHRP Report 36* (1967), 33 pp.
3. COMMITTEE ON GUARDRAILS AND GUIDE POSTS, *HRB Circular 482* (Sept. 1962).
4. NORDLIN, E. F., FIELD, R. N., and HACKETT, R. P., "Dynamic Full-Scale Impact Tests of Bridge Barrier Rails." *Hwy. Res. Record 83* (1965).
5. U.S. CONGRESS, Hearings Before the Special Subcommittee on the Federal-Aid Highway Program, Committee on Public Works, House of Representatives (May, June, July, 1967), 1243 pp.
6. BEATON, J. L., FIELD, R. N., and MOSKOWITZ, K., "Median Barriers: One Year's Experience and Further Controlled Full-Scale Tests." *Proc. HRB*, Vol. 41 (1962), pp. 433-468.
7. SACHS, W. L., "Effect of Guardrail in a Narrow Median Upon Pennsylvania Drivers." *Hwy. Res. Record 83* (1965), pp. 114-131.
8. OLSON, R. M., ROWAN, N. J., and EDWARDS, T. C., "Break-Away Components Produce Safer Roadside Signs." *Hwy. Res. Record 174* (1967), pp. 1-29.
9. JEHU, V. J., "Research on Crash Barriers." Organization for Economic Co-operation and Development, France, Number 22603 (Dec. 1967), 65 pp.
10. GRAHAM, M. D., BURNETT, W. C., GIBSON, J. L., and FREER, R. H., "New Highway Barriers: The Practical Application of Theoretical Design." *Hwy. Res. Record 174* (1967), pp. 88-189.
11. NORDLIN, E. F., and FIELD, R. N., "Dynamic Tests of Steel Box-Beam and Concrete Median Barriers." *Hwy. Res. Record 222* (1968), p. 63.
12. MICHALSKI, C. S., "Model Vehicle Damage Scale: A Performance Test." *Traffic Safety*, Vol. 12, No. 2 (June 1968), pp. 34-39.
13. GARRETT, J. W., and THARP, K. J., "Development of Improved Methods for Reduction of Traffic Accidents." *NCHRP Report 79* (1969) 163 pp.
14. NORDLIN, E. F., FIELD, R. N., and STOKER, J. R., "Dynamic Full-Scale Impact Tests of Steel Bridge Barrier Rails, Series XI." *State of California, Res. Report No. M&R 36356* (June 1967), 37 pp.
15. BEATON, J. L., NORDLIN, E. F., and FIELD, R. N., "Dynamic Tests of Corrugated Metal Beam Guardrail." *Hwy. Res. Record 174* (1967), pp. 42-87.
16. HENAULT, G. G., and PERRON, H., "Research and Development of a Guide Rail System for a High-Speed Elevated Expressway." *Hwy. Res. Record 152* (1967), pp. 36-65.
17. "Vehicle Damage Scale for Traffic Accident Investigators." Traffic Accident Data Project, National Safety Council, *TAD Project Technical Bull. No. 1* (1968), 18 pp.
18. BEATON, J. L., and FIELD, R. N., "Dynamic Full-Scale Tests of Bridge Rails." State of California Research Report (Dec. 1960), 23 pp.
19. TEXAS TRANSPORTATION INSTITUTE, Dynamic Full-Scale Crash Tests on Roadside Sign Supports, Test Series 446 (Films).
20. TEXAS TRANSPORTATION INSTITUTE, Dynamic Full-Scale Crash Tests on Impact Attenuation Devices, Test Series 505 (Films).
21. TEXAS TRANSPORTATION INSTITUTE, Dynamic Full-Scale Crash Tests on Lighting Supports, Test Series 534 (Films).
22. TEXAS TRANSPORTATION INSTITUTE, Dynamic Full-Scale Crash Tests on Lighting Supports, Test Series 1075 (Films).
23. *ASTM Manual on Quality Control of Materials*. Prepared by ASTM Committee E-11 on Quality Control of Materials, Special Technical Publication 15-C (Jan. 1951).
24. AASHO, *Standard Specifications for Highway Bridges*. Ninth Ed. (1965) pp. 5-7 and 19-20.
25. U.S. GOVERNMENT, *Proposed Specification for Bridge Railings*. Bureau of Public Roads (Apr. 1962) 23 pp.
26. AISC, "Manual of Steel Construction." American Institute of Steel Construction, Inc., Part 2.1, 6th Edition (1965).

27. STATE OF NEW YORK, "New Highway Barriers: The Practical Application of Theoretical Design." Department of Public Works, *Bureau of Physical Research Report 67-1* (May 1967).
28. HEWES, L. I., OGLESBY, C. H., "Highway Engineering." John Wiley (Sept. 1958) pp. 182-83.
29. NORDLIN, E. F., FIELD, R. N., and FOLSOM, J. J., "Dynamic Tests of Short Sections of Corrugated Metal Beam Guardrail." *State of California Highway Res. Report, M&R No. 636392-5.*
30. NORDLIN, E. F., STOKER, J. R., FIELD, R. N., and DOTY, R. N., "A Dynamic Full-Scale Impact Test on a Precast Reinforced Concrete Median Barrier." Series XII, *State of California Highway Research Report, H.W.O. 14030-951127* (Oct. 1966), p. 13.
31. EIBAND, A. M., "Human Tolerance to Rapidly Applied Accelerations: A Summary of Literature." *NASA Memo 5-19-59 E* (June 1959).
32. MALLOT, H. R., "Evaluating a New Guard Rail Coating System." *Public Works,* Vol. 96 (Apr. 1965), pp. 124-126.
33. LAUGHLAND, J. C., DIETZ, S. K., et al., "Evaluation of Criteria for Safety on the Highways." Roy Jorgenson and Associates and Westat Research Analysts, Inc. (1966), pp. 49-50, 151-152, 194-200.
34. GUNNERSON, H. E., "Narrow Bridge Study." Iowa State Highway Commission (Feb. 1961).
35. WILLIAMS, S. J., and FRITTS, C. E., "Let's Build Safety into Our Highways." *Public Safety* (May 1965) pp. 22-23.
36. STONEX, K. A., "Relation of Cross-Section Design and Highway Safety." Paper presented at the 35th Annual Highway Conference, University of Colorado, Denver (Feb. 23, 1962) (Supplemented Jan. 1963).
37. HUTCHINSON, J. W., and KENNEDY, T. W., "Medians of Divided Highways—Frequency and Nature of Vehicle Encroachments." *Bull. 487,* University of Illinois Engineering Experiment Station (1966).
38. SCRIVNER, F. H., et al., "A Systems Approach to the Flexible Pavement Design Problem." *Report 32-11.* Texas Transportation Institute (Oct. 1968), pp. 46-70.
39. "Ceramic Coatings for Highway Guardrails." *Safety Maintenance,* Vol. 130 (Dec. 1965), pp. 51-52.
40. ROBERTSON, J. A., "A Major Guardrail Galvanizing Program." *Public Works,* Vol. 95 (Oct. 1964), pp. 115-17.
41. GRAHAM, M. D., and VAN ZWEDEN, J., "Discussion of Dynamic Tests of Steel Box Beam and Concrete Median Barriers." *Hwy. Res. Record 222* (1968), pp. 86-98.
42. NORDLIN, E. F., and FIELD, R. N., "Dynamic Tests of Steel Box Beam and Concrete Median Barriers." *Hwy. Res. Record 222* (1968), pp. 53-85.
43. MORRIS, G. E., JR., "The Penalties of Non-Standardization." *Hwy. Res. News No. 32,* pp. 34-37.
44. MICHIE, J. D., and CALCOTE, L. R., "Location, Selection, and Maintenance of Highway Guardrails and Median Barriers." *NCHRP Report 54* (1968) 63 pp.
45. KUMMER, H. W., and MEYER, W. E., "Tentative Skid Resistance Requirements for Main Rural Highways." *NCHRP Report 37* (1967), 80 pp.

APPENDIX A

VERIFICATION OF MATHEMATICAL MODEL

The mathematical model, presented in Chapter Two, is a theoretical expression of a vehicle-barrier railing collision. The confidence one has in a mathematical model will depend on how accurately the model predicts the dynamic behavior of an actual collision. A comparative study of the mathematical model and full-scale dynamic test results, presented by various research organizations, was performed in considerable depth. In this type of comparison, it is to be noted that the accuracy of the model will depend on (1) the assumptions made in developing the model, and (2) the accuracy of the test data acquired in the full-scale barrier railing tests. This comparative study demonstrated that the values predicted by the mathematical model, such as time, velocity, and decelerations, compare with actual test results to an accuracy of approximately ±20 percent.

An accuracy of ±20 percent is satisfactory when one considers the wide range of test conditions represented in the comparative study, and the purpose for which the mathematical model has been devised. For example, it has been estimated that when the average lateral decelerations of a vehicle, smoothly redirected by a barrier railing, are 3 G's or less, it is of insignificant importance whether the "exact" value lies between 2.3 G's and 3.7 G's, since both the upper and lower deceleration values would be, in most instances, tolerable to the vehicle occupants. Similarly, when the predicted average lateral decelerations of a vehicle are 10 G's, the accuracy of ±20 percent would still be satisfactory, since the "exact" value that lies between 8 G's and 12 G's would not be, in most instances, tolerable to the vehicle occupants.

Finally, the mathematical model is intended to provide the design highway engineer a rational analytical approach for computing lateral impact forces, taking into consideration vehicle and roadway characteristics, for which the model would suffice at the present time.

COMPARATIVE STUDY OF MATHEMATICAL MODEL AND NEW YORK FULL-SCALE TEST RESULTS

The Bureau of Physical Research of the New York State Department of Public Works conducted a 6-yr program which led to the development of a design termed the box-beam barrier (*10, p. 88*). During the course of the study, 48 full-scale dynamic tests between standard size passenger vehicles and a variety of barrier configurations were carried out. Impact speeds up to 60 mph and impact angles up to 35° were selected as representing conditions on a high-speed highway.

The Bureau of Physical Research furnished copies of velocity-time data for 26 of the 48 tests to the research engineers of TTI. With this information, it has been possible to make comparisons of values computed by the mathematical model equations presented in Chapter Two and actual data acquired from full-scale crash tests. Examples of velocity-time data received, which were typical of the 26 tests studied by TTI, are shown in the upper portions of Figures A-1 through A-7. The data points plotted in these figures were obtained by the New York investigators from high-speed movies of each crash test. The techniques employed for acquiring and reducing data from the films are discussed in Appendix B of their *Research Report 67-1* (*27*).

Assumption (1) made in the development of the mathematical model states: "The longitudinal and lateral vehicle decelerations are constant during the time interval required for the vehicle to become parallel to the undeformed barrier." This assumption implies that velocity is a linear function of time. The time interval "limit" computed by Eq. 4 is shown as a dashed vertical line in Figures A-1 through A-7. Examination of the plotted data points revealed that Assumption (1) was valid during the time interval required for the vehicle to be redirected parallel to the undeformed barrier. For this reason, a straight line was visually fitted to the test data for longitudinal vehicle velocity-time data and another straight line was visually fitted to the test data for lateral vehicle velocity-time data. The slope of the longitudinal and lateral velocity-time curves represents a constant lateral and longitudinal deceleration-time relationship. Deceleration values computed from this test data are plotted in the lower portion of Figures A-1 through A-7 as heavy black lines, and the values of deceleration as predicted by the mathematical model, from information presented by New York in *Highway Research Record 174*, are shown as dashed lines. A comparison of the percent accuracy of the New York values of lateral vehicle deceleration as determined by TTI from the velocity-time data and values predicted by the mathematical model for 20 tests, in which snagging or pocketing did not occur, is given in Table A-1. Also, a graphical comparison is shown in Figure A-8. It is evident that 16 of the 20 compared values of lateral vehicle deceleration were within an accuracy of ±20 percent.

Assumption (3) made in the development of the mathematical model states: "The lateral component of velocity is zero after the vehicle is redirected parallel to the barrier railing." It is important to note that an examination of the test data points in Figures A-1 through A-7 indicated that, at or about the time "limit" of the mathematical model, the New York values for lateral vehicle velocity pass through zero. A comparison of values predicted by the mathematical model with values of exit velocity and corresponding time interval, as reported by New York, is given in Table A-1. Also, a graphical comparison of the exit velocities is shown in Figure A-9, and a graphical comparison of the corresponding time intervals is shown in Figure A-10. To compute the lateral velocity of the vehicle by Eq. 6, it was first necessary to obtain an average value for the coefficient of friction between the vehicle body and barrier railing by Eq. 9. Referring to Table A-1, an average coefficient of friction of 0.30 was computed by TTI from the velocity-time data furnished by New York. It is evident from the comparison, given in Table A-1 and shown in Figures A-9 and A-10, that 15 of the 18 values of exit velocity, and 13 of the 20 values of corresponding time interval were within an accuracy of ±20 percent. Thus, test data furnished by New York support Assumptions (1) and (3) made in the development of the mathematical model. These assumptions were first mentioned in Chapter Two, "Mathematical Model of a Vehicle-Barrier Railing Collision."

Further examination of the test data indicates that the lateral velocity of the vehicle is negative subsequent to the time interval "limit" of the mathematical model. This observation is important because it indicates that, since the lateral component of velocity is directed away from the barrier, the lateral impact force has been reduced to zero. This observation is corroborated by examination of photographs of the damage to crash test vehicles; in these the damage was concentrated at the front of the vehicles and only slight or no damage had occurred to the sides of the vehicles.

One other observation should be made—the examples shown in Figures A-1 through A-7 include tests on rigid bridge rails for which the deflection is zero, and for semi-rigid to flexible guide rails which had measured deflections up to 10.7 ft. Thus, it is concluded that the assumption of constant vehicle decelerations and linearity of velocity time relationships is independent of the magnitude of lateral displacement of the various rail types (rigid, semi-rigid, and flexible).

COMPARATIVE STUDY OF MATHEMATICAL MODEL AND CALIFORNIA FULL-SCALE TEST RESULTS

During the past two decades, the Materials and Research Laboratory of the California Division of Highways has designed and conducted many full-scale dynamic tests on new and modified barrier railing concepts. As a result, sig-

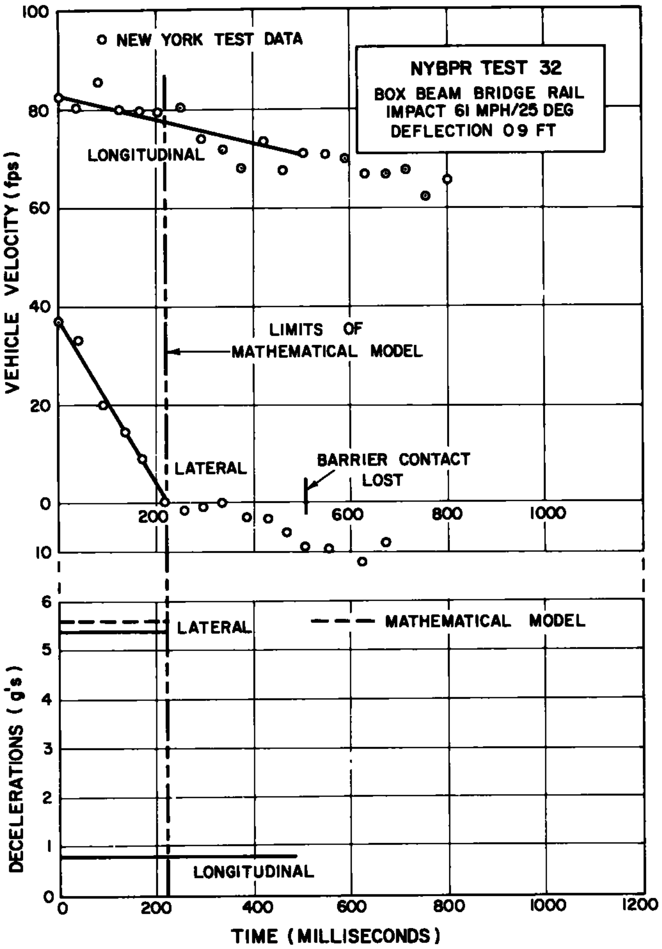


Figure A-1. Comparison of mathematical model with velocity-time test data furnished by New York, Test 32.

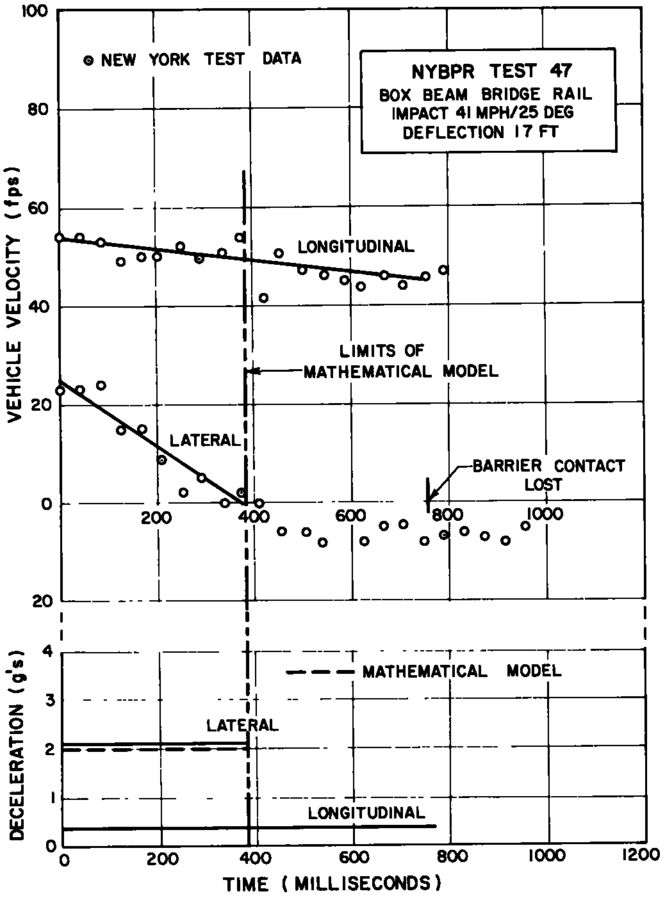


Figure A-2. Comparison of mathematical model with velocity-time test data furnished by New York, Test 47.

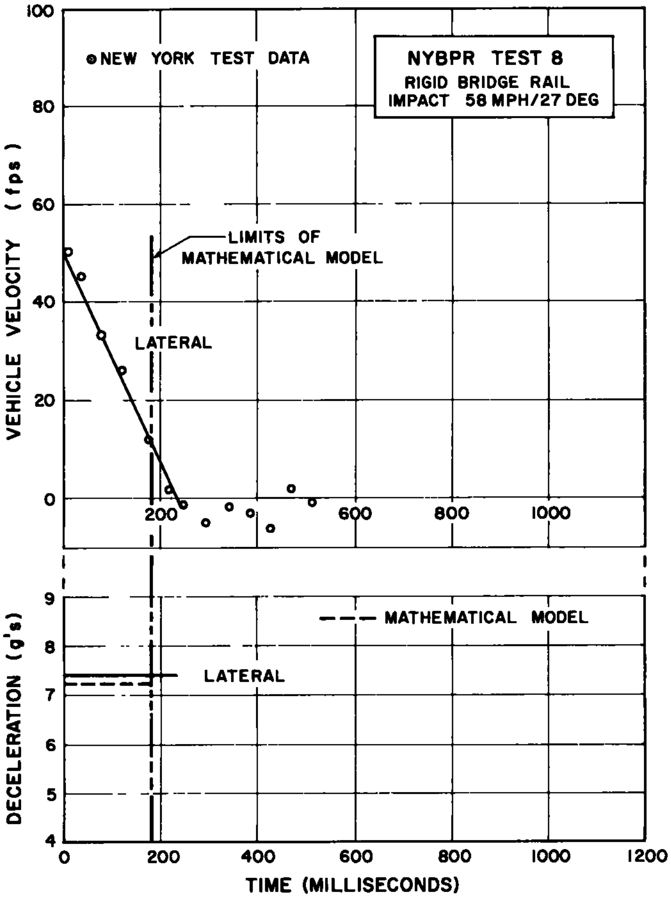


Figure A-3. Comparison of mathematical model with velocity-time test data furnished by New York, Test 8.

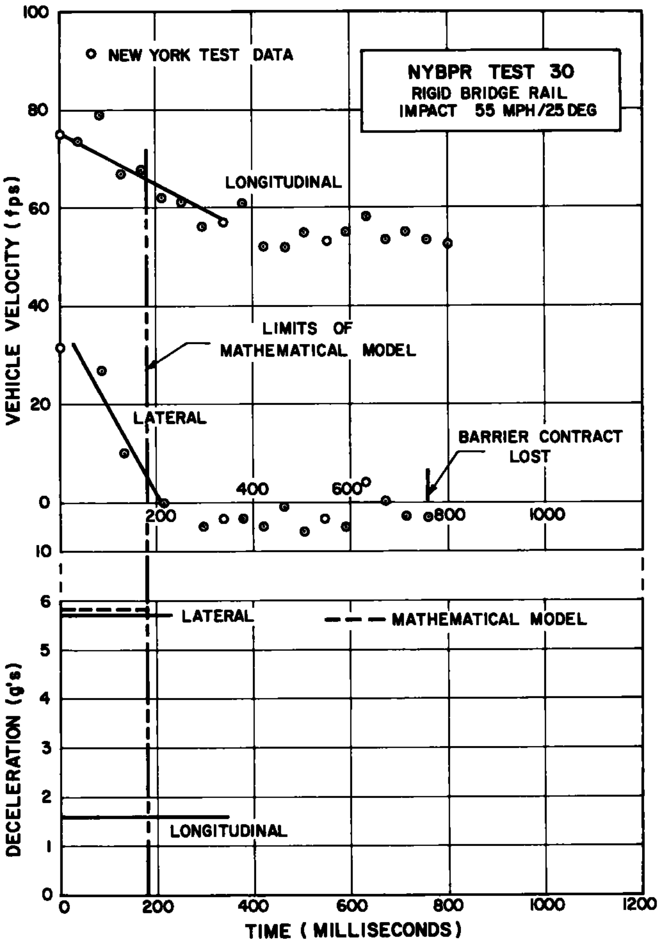


Figure A-4. Comparison of mathematical model with velocity-time test data furnished by New York, Test 30.

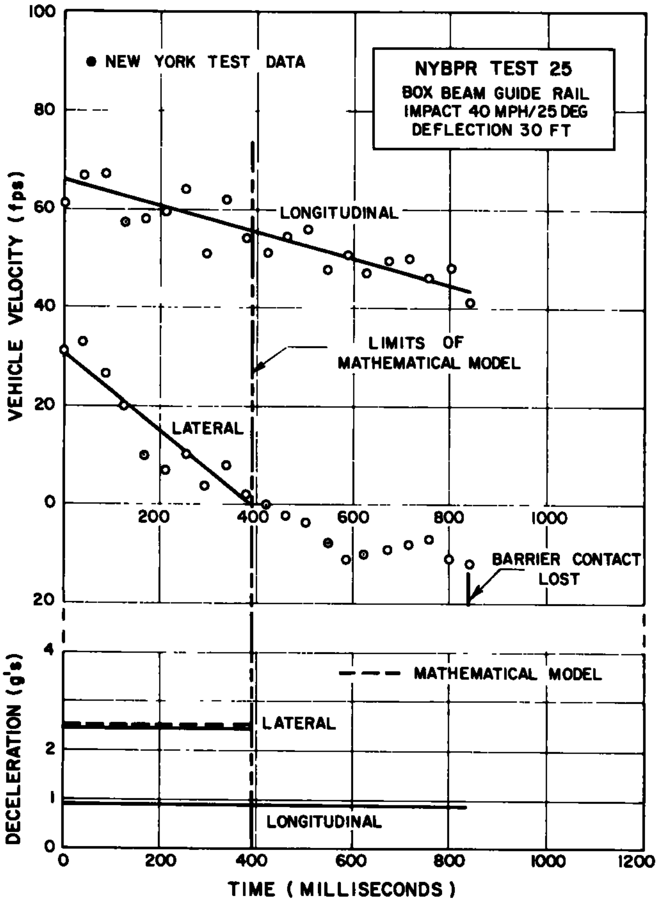


Figure A-5. Comparison of mathematical model with velocity-time test data furnished by New York, Test 25.

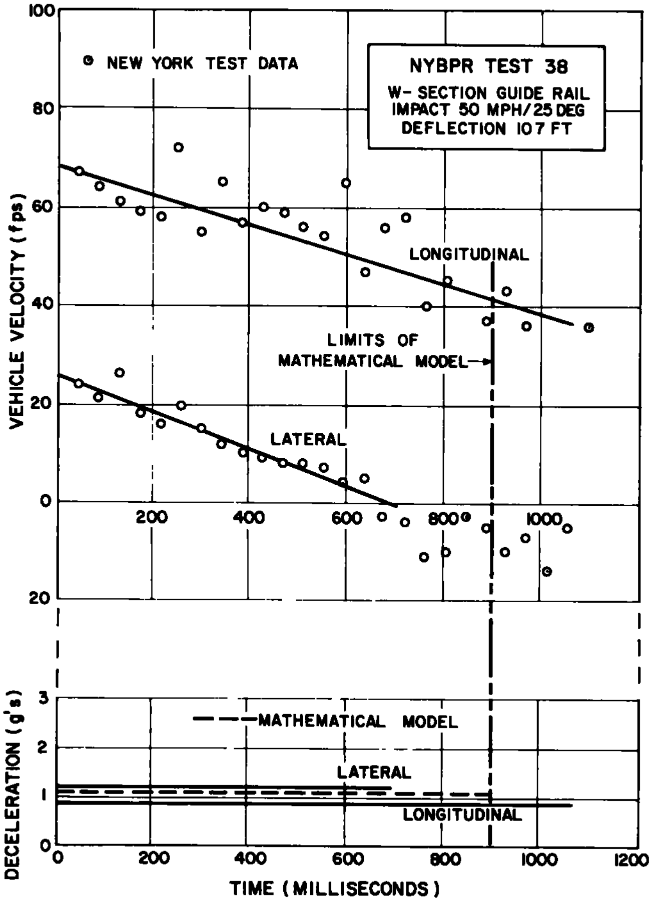


Figure A-6. Comparison of mathematical model with velocity-time test data furnished by New York, Test 38.

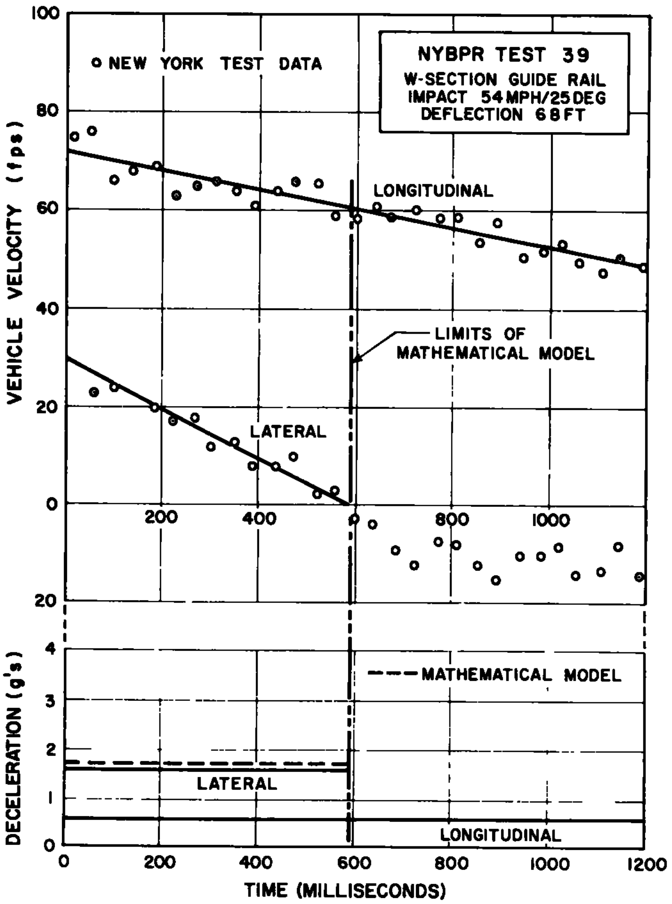
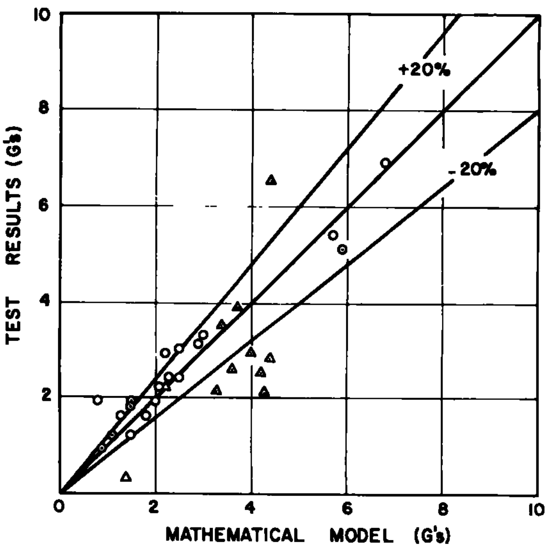


Figure A-7. Comparison of mathematical model with velocity-time test data furnished by New York, Test 39.



° New York (*HRR 174*) values of average lateral deceleration of vehicle during time interval required for vehicle to become parallel to undeformed barrier railing were determined by TTI from furnished copies of Velocity-Time Test Data (*10*).

Δ Montreal (*HRR 152*) reported values of average lateral deceleration of vehicle (*16*).

Mathematical Model values of average lateral deceleration, determined by use of Equation 5, were based on information presented by New York in *HRR 174* and by Montreal in *HRR 152*

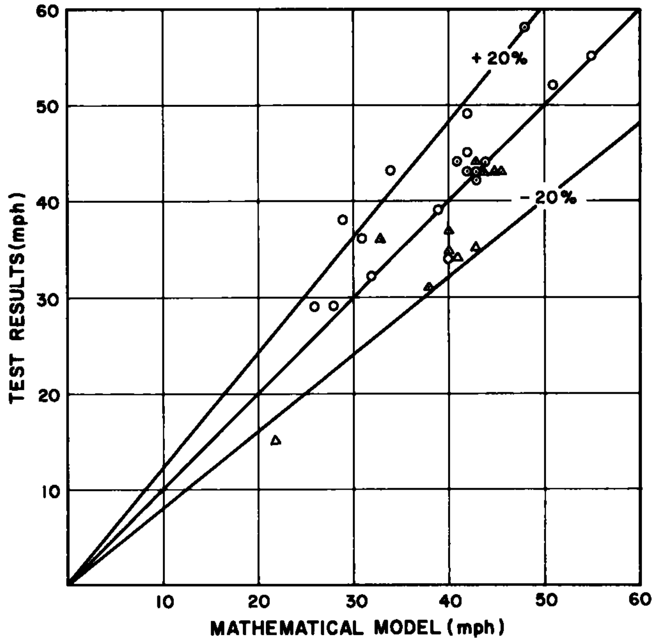
Figure A-8. Graphical comparison of average lateral vehicle decelerations.

TABLE A-1

COMPARISON OF MATHEMATICAL MODEL AND FULL-SCALE TEST RESULTS OF NEW YORK

TEST RESULTS OF NEW YORK (*10*)[a]							MATHEMATICAL MODEL			COMPARISON		
TEST NO.	TYPE OF BARRIER RAILING	TIME (SEC)	ΔV (MPH)	G_{lat}	G_{long}	μ	TIME (SEC)	V_E (MPH)	G_{lat}	TIME (%)	VEL. (%)	G_{lat} (%)
8	Rigid bridge rail	.227	—	6.9	—	—	.178	44	6.8	−28	—	−2
20	Cable guide rail	.672	42	1.6	0.5	0.31	.824	43	1.3	+19	+2	−23
21	Box beam transition	.378	45	2.4	0.8	0.33	.455	42	2.3	+17	−7	−4
22	Box beam end treatment	.251	29	3.1	1.0	0.32	.365	26	2.9	+31	−12	−7
24	Box beam median barrier	.370	44	2.9	0.7	0.24	.491	44	2.2	+25	0	−32
25	Box beam guide rail	.403	39	2.4	0.8	0.33	.388	39	2.5	−39	0	+4
26	Box beam median barrier	.495	29	2.2	1.1	0.50	.526	28	2.1	+6	−4	−5
28	Cable guide rail	.587	44	1.9	0.6	0.32	.702	41	1.5	+16	−5	−27
30	Rigid bridge rail	.202	43	5.1	1.7	0.33	.178	43	5.9	−12	0	+14
32	Box beam bridge rail	.218	58	5.4	0.8	0.15	.208	48	5.7	−5	−21	+5
33	Cable guide rail	.453	49	1.8	0.4	0.22	.702	42	1.5	+36	−14	−20
34	Box beam guide rail	.386	32	3.1	1.2	0.39	.439	32	2.9	+12	0	−7
38	W-section guide rail	.655	34	1.2	0.8	0.67	.869	40	1.1	+25	+18	−9
39	W-section guide rail	.580	43	1.6	0.6	0.37	.587	42	1.8	+1	−2	+11
41	W-section guide rail	.151	55	1.2	0.4	0.36	.185	55	1.5	+18	0	+20
43	Box beam median barrier	.403	—	3.0	—	—	.388	39	2.5	−4	—	−20
45	Box beam bridge rail	.319	52	0.9	0.1	0.11	.346	51	0.9	+8	−2	0
46	Cable guide rail	.420	43	1.9	0.2	0.11	1.030	34	0.8	+59	−29	−138
47	Box beam bridge rail	.386	36	1.9	0.4	0.20	.381	31	2.0	−1	−16	+5
48	Box beam bridge rail	.370	38	3.3	0.6	0.18	.399	29	3.0	+7	−31	−10

[a] Values were determined by TTI from velocity-time test data furnished by New York Bureau of Physical Research; Ref. *10* contains other data.

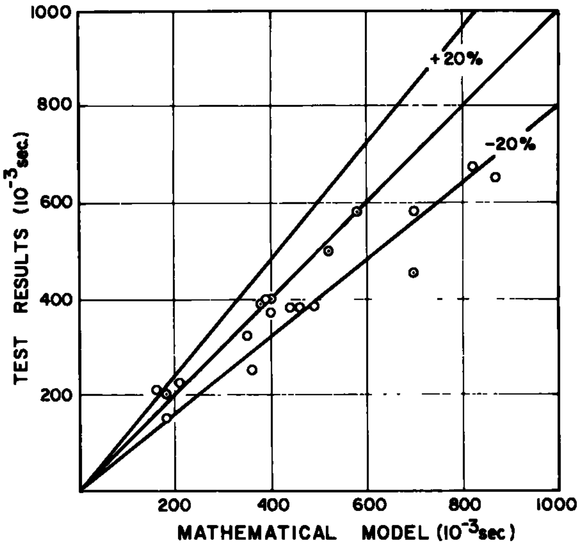


° New York (*HRR 174*) values of exit velocity at instant vehicle becomes parallel to undeformed barrier railing were determined by TTI from furnished copies of Velocity-Time Test Data (*10*).

Δ Montreal (*HRR 152*) reported values of exit velocity modified by the cosine of exit angle by TTI (*16*).

Mathematical Model values of exit velocity, determined by use of Equations 6 and 9, were based on information presented by New York in *HRR 174* and Montreal in *HRR 152*.

Figure A-9. Graphical comparison of vehicle exit velocity.



° New York (*HRR 174*) values of time required for vehicle to become parallel to undeformed barrier railing were determined by TTI from furnished copies of Velocity-Time Test Data (*10*).

Mathematical Model values of time, determined by use of Equation 4, were based on information presented by New York in *HRR 174*.

Figure A-10. Graphical comparison of vehicle exit time interval.

nificant results having practical application have evolved.

Information on vehicle decelerations was not available; however, a comparison as given in Table A-2 was obtained between values predicted by the mathematical model and the time interval "range" estimated by TTI personnel from high-speed photographs, which show the position of a smoothly redirected vehicle before, at, and after the instant a vehicle becomes parallel to the undeformed barrier railing. Model computations (Eq. 4) were based on values of impact velocity, impact angle, and lateral displacement of the barrier railing reported by Nordlin, et al. (*4, 11, 14, 15*). Predicted values of average lateral deceleration of the test vehicles are also presented in Table A-2 to furnish the reader an indication of: (1) the expected injury level of the occupants, as estimated from Figure 7, and (2) the magnitude of the impact force under various test conditions.

It is evident from Table A-2 that 16 of the 19 values of the computed time interval are within or extremely close to the range in values observed from the photographs of each full-scale test presented by the California investigators. Although it is not possible to obtain an estimate of the degree of accuracy of the mathematical model based on this comparative study, these findings strengthen the validity of the mathematical model and its assumptions.

COMPARATIVE STUDY OF MATHEMATICAL MODEL AND MONTREAL FULL-SCALE TEST RESULTS

In 1962, a research study involving full-scale testing was undertaken by Henault (*16*) to develop an effective guide rail system for a six-lane elevated expressway in Montreal.

A comparison of values predicted by the mathematical model and the values of reported average lateral vehicle deceleration varied over a wide range as given in Table A-3 and shown in Figure A-8. Because of the time interval over which the average decelerations had not been reported, it is felt that this type of comparison is of little significance.

However, a meaningful comparison of values predicted by the mathematical model and the values of reported exit velocity was obtained as given in Table A-3 and shown in Figure A-9. Using an average coefficient of friction of 0.57 computed by Eq. 9, and modifying the exit velocity by the cosine of the exit angle to obtain the component parallel to the barrier railing, it is evident from Table A-3 and Figure A-8 that 10 of the 11 values of exit velocity are within an accuracy of ±20 percent. Thus, the test results of Henault also support the validity of the mathematical model.

TABLE A-2

COMPARISON OF MATHEMATICAL MODEL AND FULL-SCALE TEST RESULTS OF CALIFORNIA

TEST RESULTS OF CALIFORNIA						MATHEMATICAL MODEL	
REF.	TEST NO.	SPEED (MPH)	ANGLE (DEG.)	BARRIER DISPLACEMENT (FT)	TIME ESTIMATED FROM SEQUENCE PHOTOGRAPHS (SEC)	TIME (SEC)	G_{lat}
				Calif. Type 1, 2			
4	B1	76	25	Rigid	$t \leq 0.143$	0.130	11.2
4	B2	76	25	Rigid	$0.094 < t < 0.194$	0.130	11.2
4	B3	73	25	Rigid	$0.065 < t \leq 0.152$	0.143	10.4
4	B4	77	25	Rigid	$0.069 < t < 0.211$	0.127	11.3
4	B5	78	25	Rigid	$0.063 < t < 0.169$	0.126	12.0
				Calif. Type 8			
14	111	68.5	25	0.25	$0.10 < t < 0.25$	0.156	8.4
14	112	58.5	25	0.33	$0.21 \leq t < 0.31$	0.191	6.1
14	113	61.5	23	0.25	$0.20 \leq t < 0.25$	0.178	6.2
14	114	66.2	25	Penetrated	— — —	—	—
				New Jersey			
11	161A	38	7	Rigid	$t < 0.20$	0.275	0.8 [a]
11	161B	65	7	Rigid	$t < 0.20$	0.162	2.2
11	162	63	25	Rigid	$0.10 < t < 0.20$	0.156	7.8
				W-Guardrail			
15	101	69	25	1.25	$0.182 < t < 0.241$	0.202	6.6
15	103	67	25	0.75	$0.163 < t \leq 0.243$	0.183	7.1
15	104	68	25	1.00	$0.159 < t < 0.228$	0.193	6.8
15	106	60	25	1.75	$0.181 < t < 0.243$	0.258	4.5
15	107	60	25	1.50	$0.180 < t < 0.360$	0.245	4.8
15	108	59	25	1.50	$0.171 < t < 0.304$	0.249	4.6
15	109	60	25	2.00	$0.241 < t < 0.272$	0.271	—
				NY Box Beam			
11	142	64	25	Est. 3.2	$0.160 < t \leq 0.255$	0.314	3.9
11	143	49	10	0.75	$0.200 < t \leq 0.340$	0.333	1.2

[a] Value only approximate, as mathematical model assumption (*2*) neglects vertical and rotational accelerations.

TABLE A-3

COMPARATIVE STUDY OF EXIT VELOCITIES BETWEEN MONTREAL AND MATHEMATICAL MODEL OF A BARRIER RAIL COLLISON

TEST RESULTS OF MONTREAL (*16*)									TTI COMPUTED VALUES			COMPARISON	
TEST NO.	TYPE OF BARRIER RAIL	V_I (MPH)	V_E' (MPH)	θ (DEG)	β (DEG)	G_{long} (G's)	G_{lat} (G's)	μ	G_{lat}[a] (G's)	$V_E' \cos\beta$ (MPH)	V_E[b] (MPH)	G_{lat} (%)	VEL. (%)
2	Concrete median	53	35	19	14	1.8	2.9	0.62	4.0	34	40	+27	+15
3	Steel guide rail	59	45	19	14	1.0	2.5	0.40	4.2	44	45	+40	+2
4	↓	42	36	19	8	0.4	2.2	0.18	2.2	36	32	0	−12
5	↓	57	35	20	9	0.9	2.1	0.43	4.3	35	43	+51	+19
6	↓	57	44	18	11	1.5	3.9	0.38	3.7	43	43	−5	0
7	Aluminum guide rail	53	38	18	13	2.1	3.5	0.60	3.4	37	40	−3	+7
8	↓	58	44	19	10	5.2	6.5	0.80	4.4	43	44	−48	+2
9	↓	50	32	20	15	1.3	2.1	0.62	3.3	31	38	+36	+18
11	Concrete guide rail	56	44	15	6	1.5	2.6	0.58	3.6	44	43	+28	−2
12	↓	29	15	22	9	0.3	0.3	1.00	1.4	15	22	+79	+36
13	↓	53	36	20	8	1.8	2.8	0.64	4.4	35	40	+36	+12

in which: V_I and V_E' = impact and exit velocities
θ and β = impact and exit angles
μ = coefficient of friction between body and barrier railing.

[a] Average lateral vehicle deceleration computed by Eq. 5.
[b] Exit velocity of vehicle computed by Eqs. 6 and 9.

APPENDIX B

ALLOWABLE VEHICLE DECELERATIONS

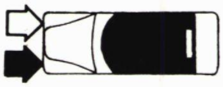
Based on the development of a mathematical model, the research work of Michalski (*12*), and that of various research agencies conducting full-scale dynamic barrier tests, it will be demonstrated in this appendix that the severity of damage to a vehicle provides an indication of the vehicle decelerations and the incidence of injury to unrestrained occupants.

From a recent test in Oregon involving 951 vehicles in traffic accidents of which there were 184 personal injuries and 7 fatalities, Michalski demonstrated, as shown in Figure 6, that the proportion of damaged vehicles in which injuries occurred was proportional to the square of the severity of damage to a vehicle as rated on a 7-point scale. Michalski indicates that the probability integral and parabolic curves may be used with equal facility to predict incidence of injuries in relation to damage rating.

To assist police officers and civilian investigators in appraising the severity of damage sustained by vehicles involved in traffic accidents, a Vehicle Damage Rating Scale (*17*) was developed by Michalski. Basically, the rating scale consists of photographs of passenger vehicles damaged in accidents that are so arranged that a separate page is provided for each of the common type impacts. On each page there are three sets of photographs showing vehicles in various stages of deformation, as shown in Figure B-1. To the right of the photographs there is a 7-point rating scale and, in the upper right corner of each page, there is a small diagram of a car and an arrow, or series of arrows, showing direction of the principal impact force. In addition to the diagram, there is a symbol (FC, for example) which indicates the part of the vehicle damaged and type of impact. In order to rate damage of a vehicle, the user must select the proper page of photographs, and then attempt to match the damage on the subject vehicle with one of the photographs appearing on the page.

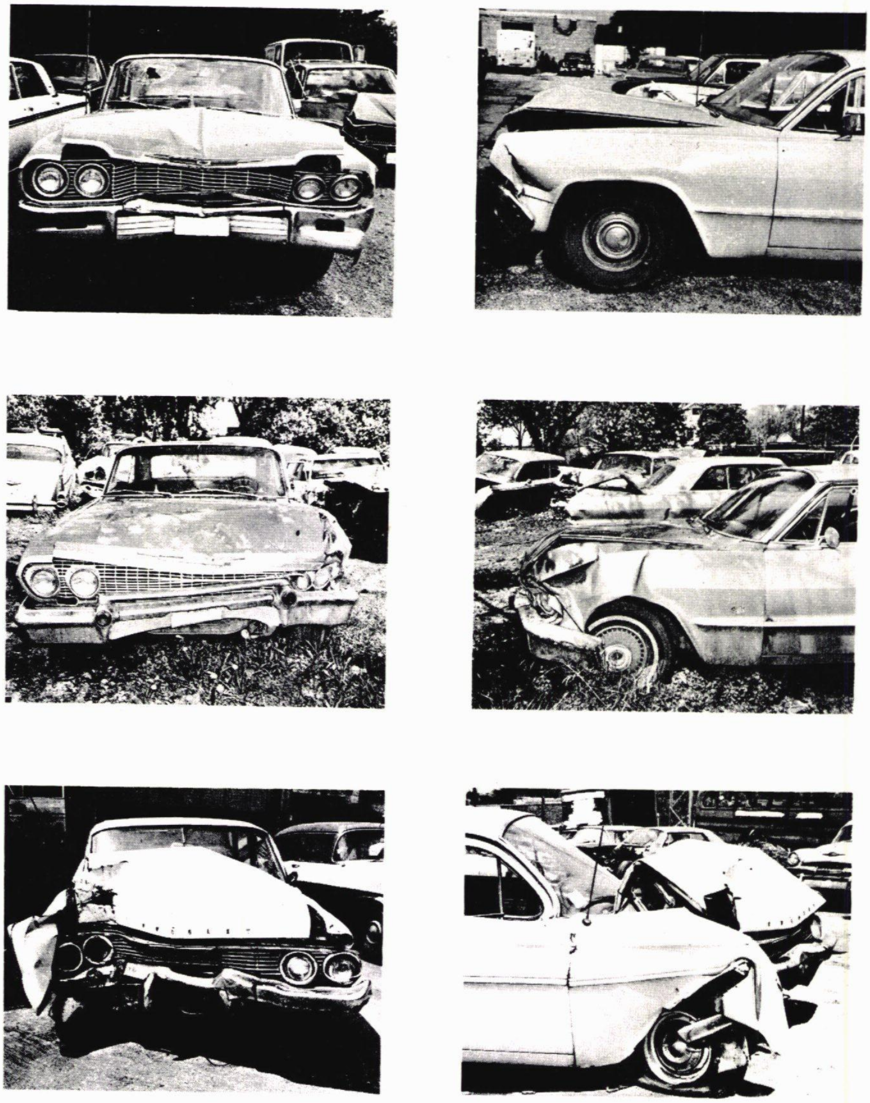
In order to apply and extend the work of Michalski to include average vehicle decelerations, vehicles damaged in full-scale dynamic tests by various research agencies were selected for evaluation. The average vehicle decelerations, as shown in Table B-1, were determined by use of the mathematical model on the angle highway barrier railing impact tests conducted by California (*4, 14, 11, 15*) and New York (*10*), and by a film analysis on the front-end impact tests conducted by Texas Transportation Institute

(*19, 20, 21, 22*). To obtain unbiased results, the photographs of the damaged vehicles, as shown in Figure B-2, were placed in a random manner on ten 8 × 11 cards. Unaware of what the average vehicle decelerations were, six research engineers rated the severity of damage to each vehicle using the vehicle damage rating scale developed by



FL/FR

This scale is applicable to damage resulting from partial contact of front end (left front corner or right front corner) of subject vehicle with another vehicle or object.



Damage Rating

FL-1 or FR-1

FL-2 or FR-2

FL-3 or FR-3

FL-4 or FR-4

FL-5 or FR-5

FL-6 or FR-6

FL-7 or FR-7

Figure B-1. Vehicle damage rating scale photographs copyrighted by National Safety Council (1968).

TABLE B-1

AVERAGE DECELERATIONS OF VEHICLES DAMAGED IN FULL-SCALE DYNAMIC BARRIER TESTS

TEST NO.	RESEARCH TEST NO.[a]	TYPE IMPACT ANGLE	TYPE IMPACT FRONT	G's	TEST NO.	RESEARCH TEST NO.[a]	TYPE IMPACT ANGLE	TYPE IMPACT FRONT	G's
34	T-505-1B		X	14.6	41	T-505-4B		X	6.7
26	N-32	X		5.6	22	N-4	X		3.2
60	T-446-10		X	1.3	5	C-B3	X		10.4
36	T-505-2A		X	6.6	33	N-39	X		1.8
42	T-505-4C		X	7.6	17	C-106	X		4.5
55	T-446-4		X	19.2	31	N-25	X		2.5
28	N-45	X		0.9	48	T-534C		X	2.1
44	T-1075-S2		X	0.7	6	C-B4	X		11.3
16	C-104	X		6.8	13	C-113	X		6.2
23	N-24	X		2.2	46	T-534A		X	1.8
12	C-112	X		6.0	21	C-143	X		1.2
14	C-101	X		6.6	30	N-30	X		5.9
3	C-B1	X		11.2	27	N-44	X		0.4
58	T-446-7		X	0.9	19	C-108	X		4.6
18	C-107	X		4.8	56	T-446-5		X	1.3
57	T-446-7		X	0.9	35	T-505-1C		X	14.2
49	T-534D		X	1.8	54	T-446-3		X	14.3
40	T-505-4A		X	7.8	59	T-446-9		X	1.8
10	C-162	X		7.8	29	N-29	X		4.8
52	T-446-1		X	13.9	32	N-34	X		2.8
47	T-534B		X	2.6	8	C-161A	X		0.8
7	C-B5	X		12.0	9	C-161B	X		2.2
37	T-505-2B		X	12.3	38	T-505-2C		X	9.9
39	T-505-2D		X	8.1	15	C-103	X		7.1
53	T-446-2		X	15.7	20	C-142	X		3.9
24	N-26	X		2.0	45	T-1075-S3		X	0.4
43	T-505-4D		X	5.3	4	C-B2	X		11.2
1	C-16	X		8.0	51	T-534F		X	2.0
25	N-11	X		5.0	2	C-17	X		7.5

[a] C = California (20 tests); N = New York (12 tests); T = TTI (26 tests).

Michalski. A description of the four common types of impacts that were selected, out of the ten available, is as follows:

- FC Front-end damage due to concentrated impact resulting from collision of subject vehicle with a tree, utility pole, or other narrow fixed object.
- FD Front-end damage due to distributed impact resulting from full contact of front-end of subject vehicle with other vehicle or broad object.
- FL/FR Front left or right corner damage due to partial contact of front-end of subject vehicle with other vehicle or object.
- LFQ/RFQ Left or right front quarter damage (ahead of passenger compartment) due to angular impact by another vehicle or object. Applicable to angle collisions and accidents in which a vehicle strikes an object after skidding or spinning.

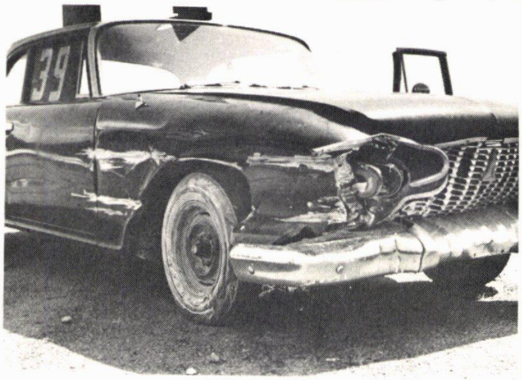
In the use of the damage rating scale, vehicles damaged in front-end impacts were rated incorrectly as angle impacts (LFQ/RFQ), and conversely for 4.9 and 1.5 percent of the observations, respectively. Michalski indicated that police officers selected the proper page of photographs in the damage scale for over 90 percent of the vehicles rated and that nearly all page selection errors could be corrected by an analyst in a relatively simple office procedure.

Also, some difficulty was encountered in distinguishing between LFQ/RFQ and LF/RF damage rating when evaluating the vehicles damaged in angle impact tests. As a result, the front side impact ratings (LF/RF) were selected for 9.1 percent of the observations. It is felt that had the damaged test vehicles been rated by other than research engineers, the LF/RF rating would have been selected more often. This particular situation is considered to be of minor importance because the basic intent is to demonstrate the application of the damage rating scale to achieve the objectives of this study. Therefore, in order to use all the information obtained, the LFQ/RFQ and LF/RF ratings were combined—perhaps in future research, the damage rating scale could be refined to include a damage rating scale of vehicles damaged in highway barrier railing angle impacts.

A simple statistical analysis of the damage ratings made by the six research engineers on vehicles used in full-scale dynamic tests is given in Table B-2. The factors used

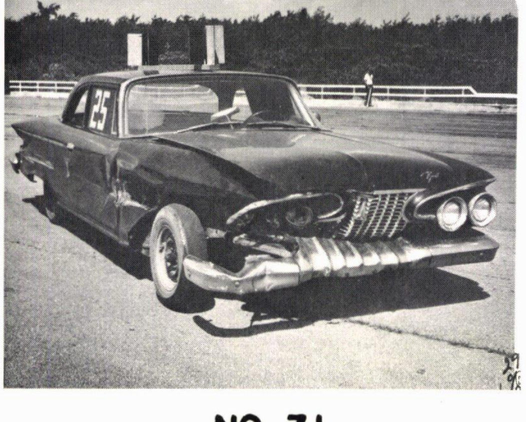
NO. 22

NO. 5



NO. 33

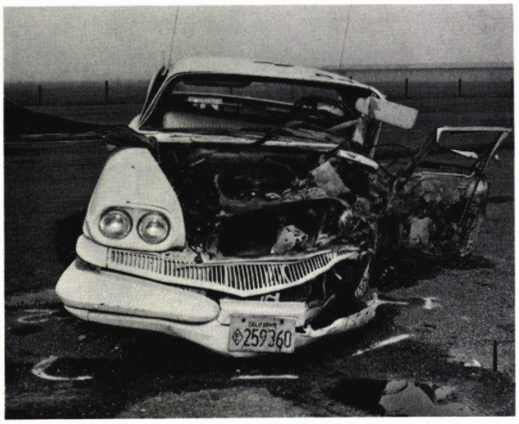
NO. 17



NO. 31

NO. 48

Figure B-2. Vehicles damaged in highway barrier dynamic tests.



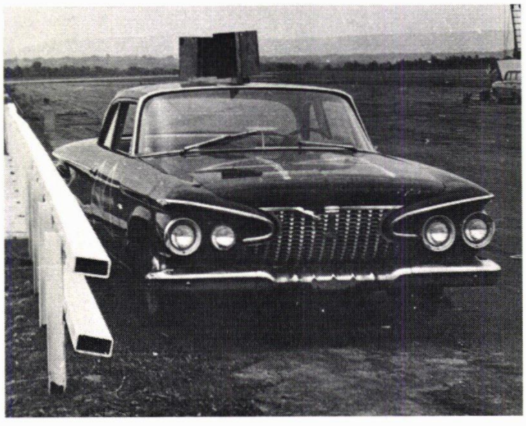
NO. 6

NO. 13

NO. 46

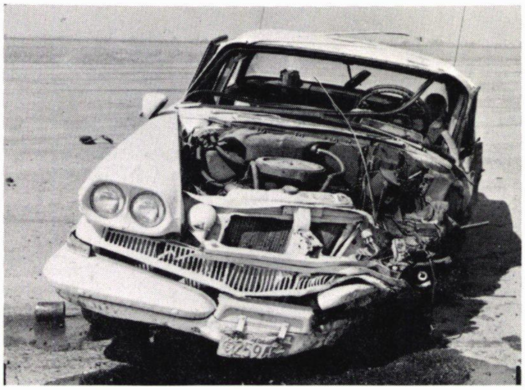
NO. 21

NO. 30

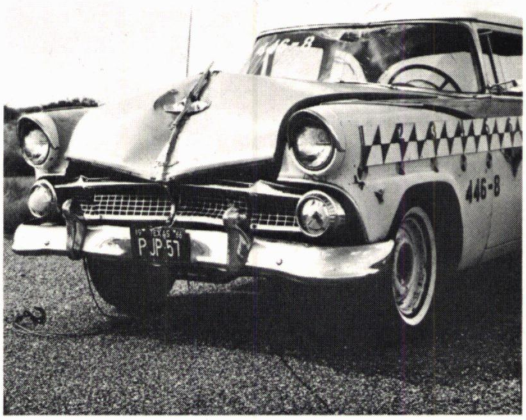


NO. 27

Figure B-2. Vehicles damaged in highway barrier dynamic tests (continued).



NO. 3



NO. 58

NO. 18

NO. 57

NO. 49

NO. 40

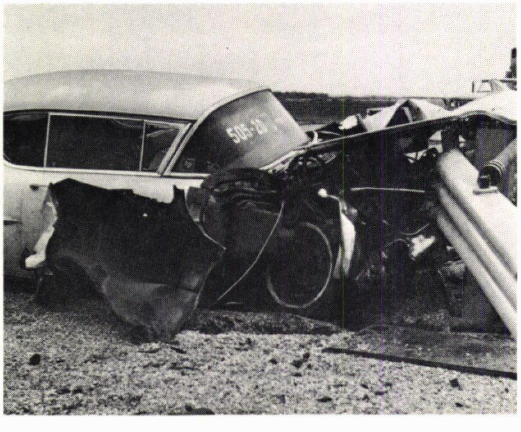
NO. 10

NO. 52

NO. 47

NO. 7

NO. 37



NO. 39

Figure B-2. Vehicles damaged in highway barrier dynamic tests (continued).

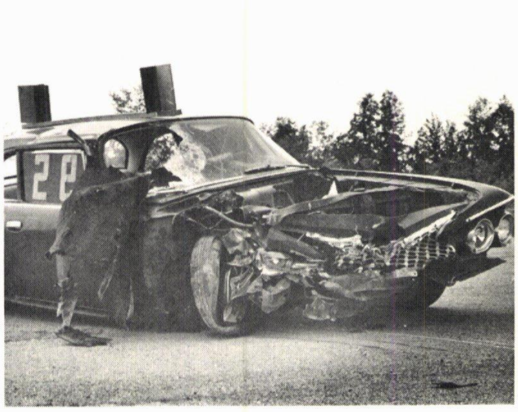
NO. 19

NO. 56

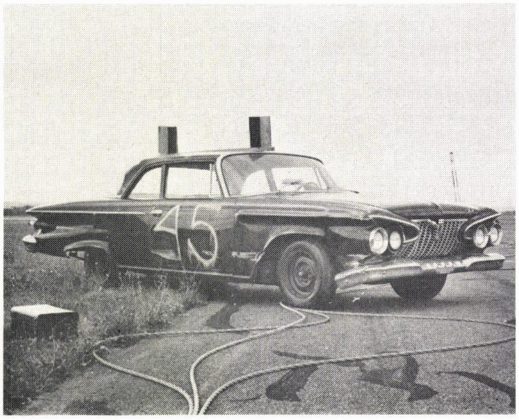
NO. 35

NO. 54

NO. 59

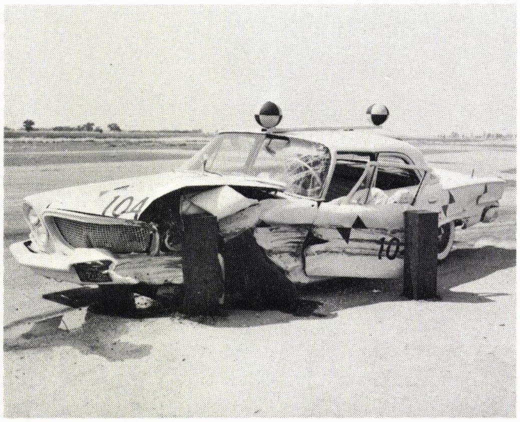


NO. 29



NO. 28

NO. 44



NO. 16

NO. 23

NO. 12

NO. 14

Figure B-2. Vehicles damaged in highway barrier dynamic tests (continued).

NO. 32

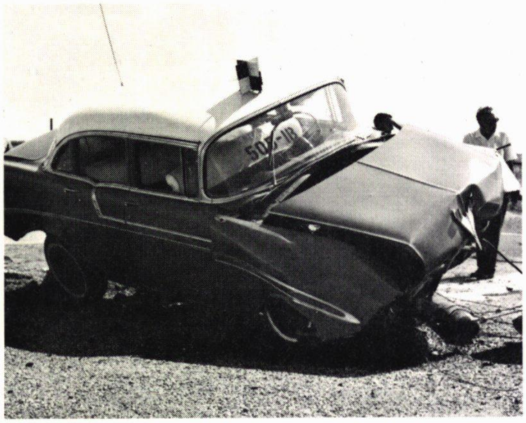
NO. 8

NO. 9

NO. 38

NO. 15

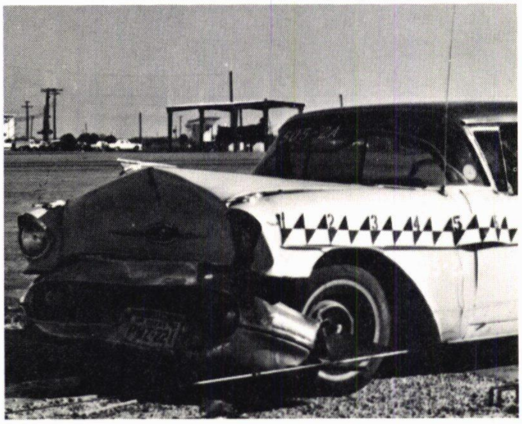
NO. 20



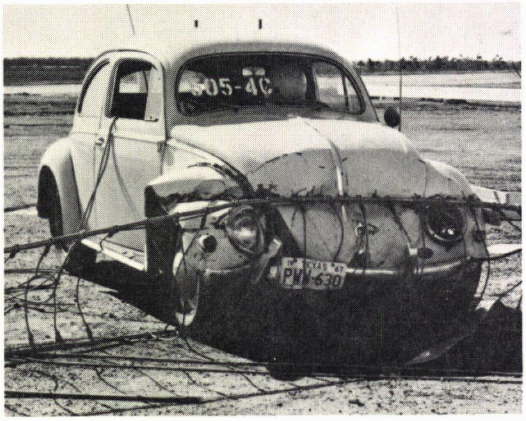
NO. 34

NO. 26

NO. 60



NO. 36



NO. 42

NO. 55

Figure B-2. Vehicles damaged in highway barrier dynamic tests (continued).

NO. 53

NO. 24

NO. 43

NO. 1

NO. 25

NO. 41

NO. 45

NO. 4

NO. 51

NO. 2

Figure B-2. Vehicles damaged in highway barrier dynamic tests (continued).

TABLE B-2

STATISTICAL ANALYSIS OF AVERAGE VEHICLE DECELERATIONS AND SEVERITY OF VEHICLE DAMAGE

TYPE VEHICLE IMPACT	DAMAGE RATING	NUMBER OBERVATIONS, η	AVERAGE [a] $\overline{X}$ (G's)	STANDARD DEVIATION, σ (G's)
Front	1	2	0.7 ($\pm$0.0)	0.0
	2	20	1.8 ($\pm$0.7)	1.7
	3	34	2.1 ($\pm$0.6)	2.0
	4	29	3.6 ($\pm$1.0)	3.0
	5	22	8.6 ($\pm$1.7)	4.6
	6	28	9.4 ($\pm$1.7)	5.1
	7	27	14.3 ($\pm$0.9)	2.6
		162 All		
Angle	1	14	0.8 ($\pm$0.3)	0.5
	2	13	1.3 ($\pm$0.3)	0.6
	3	10	2.0 ($\pm$0.4)	0.6
	4	24	3.6 ($\pm$0.7)	2.1
	5	50	5.5 ($\pm$0.4)	1.8
	6	45	6.1 ($\pm$0.5)	2.0
	7	36	9.7 ($\pm$0.8)	2.7
		192 All		

[a] The $\pm$ entry indicates 90% confidence limits of the objective value of $\overline{X}$ for the universe sampled.

for calculating the 90 percent confidence limits for the averages ($\overline{X} \pm \nabla\sigma$) of the longitudinal and lateral average vehicle decelerations were obtained from an *ASTM Manual* (*23*). To obtain a correlation to the test results obtained by Michalski, the results of this study are presented graphically as shown in Figures 7 and 8.

The parabolic curves were first selected to represent the averages ($\overline{X}$) of the test results obtained in this study. In each graph, the parabolic curve lies within the 90 percent confidence limits of the averages ($\overline{X} \pm a\sigma$) over the greater portion of its length. Next, it was assumed that at the point where the parabolic curve intercepts the ordinate through the vehicle damage rating scale of seven the proportion of damaged vehicles in which occupant injuries would occur was equal to 1.0. Hence, this assumption defined the scale necessary to plot the test data obtained by Michalski.

The points plotted on the graphs representing the information obtained by Michalski are a summation of the FC, FD, FL, and FR damage area ratings for front-end impacts, and the LFQ, RFQ, FL and FR damage area ratings for the angle impacts as presented in Table B-3. As previously discussed, due to some difficulty in distinguishing between LFQ/RFQ and FL/FR damage area rating when evaluating the full-scale test vehicles damaged in angle impacts on highway barrier railing, it was considered reasonable to combine these two types of ratings. It is apparent from the graphs that the information obtained by Michalski can also be represented by the same parabolic curves selected to represent the information obtained in this study.

Therefore, based on the information presented, it can be concluded that the average vehicle decelerations are directly proportional to: (1) the proportion of damaged vehicles involved in traffic accidents in which occupant injuries occurred, and (2) the square of the vehicle damage rating. In mathematical notation, this would be described by Eqs. 10 and 11, repeated here.

Type Vehicle Impact	*Mathematical Equation*	
Front	$G_{long.} = 0.280\,R^2 = 13.7\,P$	(10)
Angle	$G_{lat.} = 0.204\,R^2 = 10.0\,P$	(11)

in which

G = average vehicle deceleration;
R = vehicle damage rating; and
P = proportion of vehicles in which injuries occurred.

TABLE B-3

VEHICLE DAMAGE SCALE EVALUATION OF DRIVABLE AND NON-DRIVABLE VEHICLES (C. S. Michalski)

DAMAGE RATING	FRONT END IMPACTS: FC	FD	FL	FR	ALL	P [a]
0	0(0)	0(0)	2(0)	1(0)	3(0)	0.000
1	13(0)	21(1)	25(1)	47(1)	106(3)	0.028
2	8(3)	32(5)	34(1)	30(2)	104(11)	0.106
3	2(1)	39(8)	25(4)	24(3)	90(16)	0.178
4	4(1)	25(13)	23(11)	13(4)	65(29)	0.446
5	4(2)	10(6)	11(7)	12(8)	37(23)	0.622
6	2(2)	6(5)	11(10)	5(2)	24(19)	0.792
7	1(1)	5(5)	1(0)	6(6)	13(12)	0.923
All	34(10)	138(43)	132(34)	138(26)	442(113)	—

DAMAGE RATING	ANGLE IMPACTS: LFQ	RFQ	FL	FR	ALL	P [a]
0	0(0)	1(0)	2(0)	1(0)	4(0)	0.000
1	9(1)	10(0)	25(1)	47(1)	91(3)	0.033
2	16(0)	12(0)	34(1)	30(2)	92(3)	0.033
3	13(2)	14(2)	25(4)	24(3)	76(11)	0.145
4	6(1)	5(3)	23(11)	13(4)	47(19)	0.404
5	5(2)	3(0)	11(7)	12(8)	31(17)	0.548
6	1(1)	1(1)	11(10)	5(2)	18(14)	0.778
7	1(0)	0(0)	1(0)	6(6)	8(6)	0.750
All	51(7)	46(6)	132(34)	138(26)	367(73)	—

Numbers in ( ) indicate vehicles in which personal injuries occurred.
[a] Proportion of vehicles in which personal injuries occurred.

APPENDIX C

COMPARATIVE STUDY OF 1965 AASHO AND 1962 BPR BRIDGE RAILING SPECIFICATIONS

Written comments and discussions with highway engineers suggested that designing in accordance with the *1965 AASHO Standard Specifications for Highway Bridges* (*24*) produces a barrier railing system of adequate strength to restrain most of the passenger vehicles in operation on high-speed highways. Furthermore, it appeared that an elastic barrier railing design based on the 10-kip transverse force specified by AASHO was equivalent to a plastic design, using a dynamic stress coefficient,* based on the 30-kip transverse force suggested in 1962 by the Bureau of Public Roads (*25*). Subsequently, it was found that the mathematical model presented in this report predicts that a 4,000-lb vehicle traveling 63 mph and having an impact angle of 25° will produce an impact force of 30 kips upon colliding with a rigid barrier.

It will now be demonstrated that the loading criteria of AASHO are for all practical purposes equivalent to BPR in regard to the railing; whereas, the AASHO loading criteria are more severe than that of BPR for the posts. The *BPR Proposed Specification* is based on plastic design methods using a dynamic stress coefficient, whereas the AASHO specification applies a reduction factor of 3 to the BPR transverse force and then specifies that a design be based on allowable working stresses for the appropriate material. Thus, in an indirect manner, the AASHO specification in reality results in a design in which permanent deformations occur. In Appendix D, additional verification is provided by a critical evaluation of the California Type 8 bridge barrier railing which was subjected to full-scale dynamic tests.

BRIDGE RAILING

For purposes of illustration, the bridge barrier rail configuration shown in Figure C-1 will be used. In Figure 1 of the *1962 BPR Proposed Specification*, it is assumed that the top rail will resist the entire transverse force. However, because the basic intent of this comparative study is to demonstrate that the 1965 AASHO elastic and the 1962 BPR plastic design methods are equivalent, it will be assumed that the transverse force will be equally distributed to each rail. Nomenclature of the equations is presented at the end of this appendix.

1965 AASHO Standard Specifications

Rail members shall be designed for a moment, due to concentrated loads, at the center of the panel and at the posts of $P_A L/6n$ by elastic methods to allowable stresses for the appropriate material.

$$M_y = (f_a)S$$
$$= (df_y)S$$

Thus,

$$\frac{P_A}{2} = \left(\frac{6}{L} S f_y\right) d \qquad \text{(C-1)}$$

1962 BPR Proposed Specification

Rails shall be designed for a moment at the center of the panel and at the posts by the formula:

$$M_P = P_B L/6n$$

A fully plastic moment of $P_B L/6n$ would correspond to a railing continuous over two panels for the mechanism shown in Figure C-2.

Because External Work = Internal Work, the moment is

$$P_B \frac{L}{2n} \theta = \theta\,(M_P + 2M_P)$$
$$M_P = P_B \frac{L}{6n}$$

Design requirements of metal rails shall be determined by the formula:

$$M_P = 0.85\, c f_y\, kS$$

The bending stress ($0.85\, c f_y$) is taken at 85 percent of the dynamic tensile strength to allow for axial force in the railing member (see page 9 of the BPR Commentary).

Thus,

$$\frac{P_B}{2} = \left(\frac{6}{L} S f_y\right) 0.85\, ck \qquad \text{(C-2)}$$

POSTS

1965 AASHO Specifications

Posts shall be designed for a 10-kip transverse force plus a simultaneous longitudinal loading of ½ of the transverse loading by the elastic method to the allowable stresses for the appropriate material. When the tensile strength of the rail members is maintained through a series of post spaces, the longitudinal loading may be divided among as many as four posts in this continuous length.

Referring to Figure C-1, the moments at the base are:

$$M_{xx} = \frac{P_A}{2}(a + b)$$

$$M_{yy} = \frac{P_A}{2(2m)}(a + b)$$

* A dynamic stress coefficient was specified by BPR to take into consideration the increase in the strength of a material when subjected to dynamic forces. The dynamic stress coefficient is not to be misconstrued as a load factor which, for example, is defined in Part 2 of the *1965 AISC Manual of Steel Construction* (*26*).

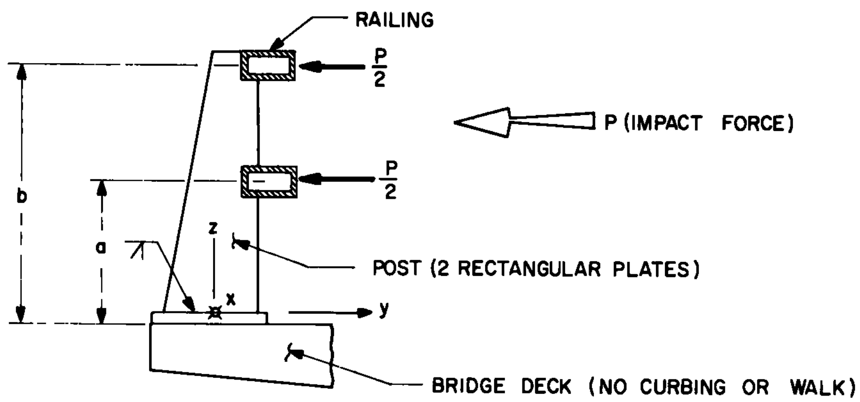


Figure C-1. Cross section of bridge barrier railing.

Letting $f_a = df_y$, the yield stress level is:

$$f_y = \frac{P_A\,(a+b)}{4\,md}\left[\frac{2m}{S_{xx}} + \frac{1}{S_{yy}}\right] \qquad \text{(C-3)}$$

1692 BPR Proposed Specification

Posts shall be designed for the cantilever effects of a transverse force of 0.8 P_B and a simultaneous longitudinal force of 0.4 P_B divided equally to each post in the continuous length.

Design requirements for metal posts shall be determined by the formula:

$$f_y = \frac{1}{ck}\left[\frac{M_{xx}}{S_{xx}} + \frac{M_{yy}}{S_{yy}}\right]$$

Referring to Figure C-1, the moments at the base are:

$$M_{xx} = \frac{4}{10} P_B\,(a+b)$$

$$M_{yy} = \frac{4}{10\,(2m)} P_B\,(a+b)$$

Thus,

$$f_y = \frac{P_B\,(a+b)}{5\,ckm}\left[\frac{2m}{S_{xx}} + \frac{1}{S_{yy}}\right] \qquad \text{(C-4)}$$

RELATIONSHIP BETWEEN AASHO (65) AND BPR (62)

Combining Eqs. C-1 and C-2, and Eqs. C-3 and C-4, the relationship between transverse forces is:

$$P_A = \left(1.177\,\frac{d}{ck}\right) P_B \; . \; . \; . \; . \; \textit{Railing}$$

$$P_A = \left(0.800\,\frac{d}{ck}\right) P_B \; . \; . \; . \; . \; \textit{Posts}$$

A pipe and rectangular box rail sections, which are widely used, have a shape factor (k) equal to 1.35 and 1.22, respectively, whereas a rectangular plate post section has a shape factor (k) equal to 1.50. Assuming that the mechanical properties of the material used conform to the require-

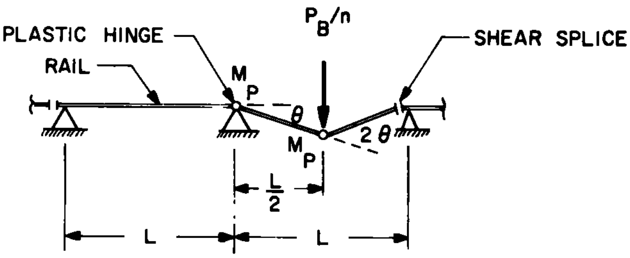


Figure C-2. Plastic mechanism of bridge railing.

ments of A36 Steel ($d = f_a/f_y = 0.555$), a dynamic stress coefficient of 1.4 is obtained (see Table 1 of BPR—page 3). Based on this information, the relationship between the transverse force specified by AASHO (65) and BPR (62) is:

$$P_A = \begin{bmatrix} 0.345\,P_B \; . \; . \; . \; \textit{Pipe Rail} \\ 0.382\,P_B \; . \; . \; . \; \textit{Box Rail} \\ 0.212\,P_B \; . \; . \; . \; \textit{Post} \\ 0.333\,P_B \; . \; . \; . \; \textit{Specified for Rail and Post} \end{bmatrix}$$

Nomenclature

P_A = transverse force, elastic design methods, AASHO (1965);
P_B = transverse force, plastic design methods, BPR (1962);
M_y = elastic bending moment at yield stress;
M_p = plastic bending moment;
S = elastic section modulus;
k = shape factor $= M_p/M_e$;
f_a = allowable working stress for material;
f_y = yield strength of material;
d = constant $= f_a/f_y$;
c = dynamic stress coefficient of static yield strength;
L = bridge rail span length between post;
m = number of posts over which railing is continuous; and
n = number of rail members.

APPENDIX D

CRITICAL EVALUATION OF CALIFORNIA TYPE 8 RIGID BRIDGE BARRIER RAILING

The California Type 8 bridge barrier railing configuration (*14*), as shown in Figure D-1, was selected by the research agency engineers for a critical evaluation because: (1) it had been subjected to full-scale dynamic tests in accordance with *Highway Research Board Circular 482* (*3*); (2) the California research engineers had provided information in sufficient depth concerning the dynamic behavior of the barrier railing and the test vehicle, which could be used as input into the mathematical model; and (3) it provided an opportunity to compare the findings of such a critical evaluation based on the output of the mathematical model with the design specification criteria of AASHO (1965) upon which the barrier railing was designed. Because of time and monetary limitations, only two of the four tests (Tests 112 and 113) conducted by California were evaluated; however, it is to be noted that no attempt was made to analyze the post-bridge deck anchorage details. The two tests selected were considered to be the best suited for evaluation and illustration purposes.

AUTHORS' ABSTRACT OF REPORT

The results of a series of full-scale dynamic impact tests of a new steel bridge barrier design are reported. The new bridge barrier design was developed by the Bridge Department of the California Division of Highways to provide improved lateral visibility and self-cleansing of the bridge deck. Four full-scale dynamic impact tests of a basic design and two modifications were conducted by the Materials and Research Department of the California Division of Highways to determine the effectiveness in retaining and redirecting an impacting vehicle. It is concluded that the basic barrier design and one of the two modified designs will adequately retain a medium-weight sedan impacting at an angle of 25° and at a speed of 60 mph. All three of the designs tested would provide definite visibility and maintenance improvements.

CRITICAL EVALUATION

Safety Considerations

In an area of an entrance ramp onto a bridge structure, the California Type 8 bridge barrier railing or a similar "open type" of railing configuration would enhance highway safety as the result of providing maximum visibility to drivers of vehicles on both the bridge structure and the entrance ramp onto the bridge structure. On the other hand, it was pointed out in the discussion sessions held with the highway engineers of the California Division of Highways that this type of railing configuration could also have an adverse psychological height effect on a driver of a vehicle. California engineers have discovered from oil traces on the roadway surface of high long span bridges, usually over large bodies of water, that drivers have a tendency to crowd the adjacent inside traffic lane wherever "open type" barrier railing configurations were used. It is surmised that such a situation could possibly create a hazardous condition.

Snagging of Vehicle by Intermediate Upper Post-Flange

It was noted by the authors and illustrated by photographs in the report that the section of the post-flange between the upper and lower rails was bent as shown in Figure D-2. In response to comments included in the June 30, 1968 quarterly progress report, the authors indicated that no stepped deceleration was noted as contact with this upper portion of the post flange occurred. Based on the results of a plastic analysis and output of the mathematical model as follows,

$$M_{\text{ult}} = c(\sigma_y)\int y\,dA = 3.25\,P_{\text{ult.}}$$

$$P_{\text{ult.}} = \frac{c(36)(0.156)(0.313)(10)(2)}{3.25}$$

$$= 10.8\ (c)$$

in which

c = dynamic stress coefficient (see Appendix C).

It is felt that it would be desirable to eliminate this snagging if at all possible. Assuming a yield strength of 36 ksi for the post flange, which is fabricated from A36 Steel, a plastic analysis indicates that a dynamic force of 10.8 kips is required to plastically bend the post-flange plate if the dynamic stress coefficient is neglected ($c = 1$), or a force of 15.1 kips is required if a dynamic stress coefficient of 1.4 is used as recommended in the *1962 BPR Proposed Specification*. Based on the reported impact test conditions and assuming a coefficient of friction of 0.4 between the vehicle body and the roadway surface, it is predicted by use of Eqs. 5 and 9 of the mathematical model that, if no snagging occurs, the vehicle will be subjected to an average longitudinal deceleration of approximately 2.5 *G*'s. As a result of snagging on the post-flange between the rail members, the vehicle will, therefore, be subjected to an additional deceleration of approximately 2.4 to 3.4 *G*'s ($P_{\text{ult.}}$/Vehicle Weight), the latter value taking into consideration the dynamic stress coefficient.

Design of Rail Members

The *1965 AASHO Standard Specifications for Highway Bridges* require that a rail member be designed by elastic methods to the allowable bending stresses for the appropriate material at the center of a panel and at the support posts by the formula:

$$f_a = \frac{PL}{6NS}$$

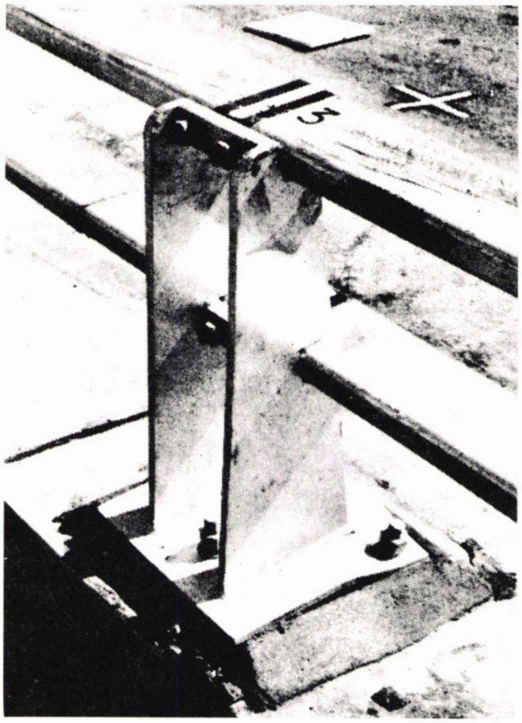


Figure D-1. California Type 8 barrier railing configuration.

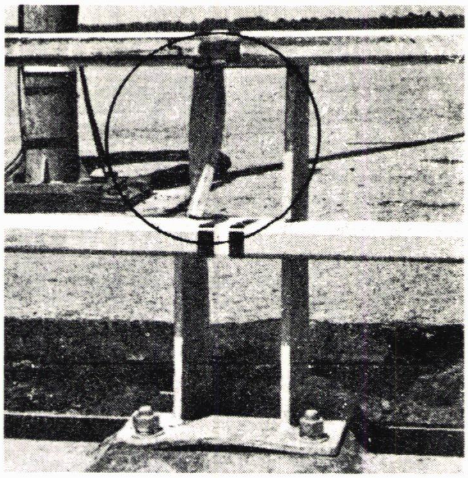


Figure D-2. Vehicle snagging on post flange (circled) between two rail members.

TABLE D-1

MECHANICAL PROPERTIES OF RAIL MEMBERS (A36 STEEL)

CALIF. TYPE 8 TEST NO.	YIELD STRENGTH (KSI)	TENSILE STRENGTH (KSI)
111, 112	63.0	75.7
113	57.3	63.1
114	58.1	65.6

Values reported are averages of a minimum of three samples.

in which

$P =$ 10 kips;
$L =$ panel length;
$S =$ section modulus of rail member;
$N =$ number of rail members; and
$f_a =$ allowable bending stress.

As shown in Figure D-1, the California Type 8 bridge railing has two rail members. The rail members selected for California Tests 112 and 113 were $2 \times 6 \times 1/4$-in. A36 structural tube shapes, which weigh 12.02 plf. and have a section modulus about the strong axis of 4.434 in^3. The computed bending stresses of a rail member with a panel length of 8 ft (Test 112) were 18.0 ksi, and the computed bending stresses for a panel length of 10 ft (Test 113) were 22.6 ksi. The allowable bending stresses for A36 steel are 20 ksi.

From static tensile tests on samples of the rail members, California determined that the yield strength of the rail members varied from 57.3 to 63.0 ksi, and the tensile strength varied from 63.1 to 75.7 ksi, as given in Table D-1. It is apparent that the yield strength values greatly exceed the guaranteed minimum requirement of 36 ksi. It is felt that the rather high yield strengths were due to the cold working of the material required to form the rectangular tube section from a circular section. If a design engineer were unaware that a design based on the criteria of AASHO results in a design in which plastic deformation can be expected to occur as demonstrated in Appendix C, it would appear from the static tensile test results given in Table D-1 that a safety factor of at least 2.5 had existed.

It was evident from the photographs included in the California report that plastic deformations occurred in the rail members which were continuous over two intermediate posts, whereas rail members of one panel length appeared to have simply rotated about the splice connections.

The magnitude of the lateral impact force of each test required to develop the average yield strength of the rail members (Table D-1) can be computed by use of Eq. C-2. This equation was derived by plastic theory for the mechanism shown in Figure C-2.

$$\frac{P_I}{2} = \left(\frac{6}{L} S f_y\right) 0.85\, ck \qquad \text{(C-2)}$$

in which

$P_I =$ Lateral impact force
(P_I was substituted for P_B)
$S =$ 4.434 in.^{3}
$k =$ 1.22 for box section
$f_y =$ 59.5 (Table D-1)
$L =$ Panel length
$c =$ Dynamic stress coefficient

$$\frac{P_I}{2} = 1{,}642\left(\frac{c}{L}\right)$$

The forces obtained by use of Eq. C-2 are now compared to the impact forces predicted by Eq. 5 of the mathematical model as shown in Table D-2.

It is apparent in Table D-2 that the impact forces predicted by the mathematical model, which has an accuracy of ±20 percent, are in reasonable agreement with the forces computed by Eq. C-2 when the dynamic stress coefficient is neglected ($c = 1.0$). It is believed that a better comparison could have been achieved if: (1) the yield stresses of the tensile test specimens of the rail members had been determined at the time the elongations were occurring in the plastic range, which will usually result in a lower value of a yield stress, and (2) the restraint conditions of the splice connections against rotation had been better defined; for example, a rail member forming only one plastic hinge at midspan and which is free to rotate at the posts because of shear-type splice connections would require only two-thirds of the lateral impact force computed by Eq. C-2.

The magnitude of the impact force of each test can also be determined by an elastic stress analysis presented earlier in this section. Referring to Table D-3 and using the average yield strength of the rail members given in Table D-1, a lateral impact force of 16.5 kips is obtained for a panel length of 8 ft (Test 112), and a force of 13.2 kips is obtained for a panel length of 10 ft. These computed values are also in reasonable agreement to the forces predicted by the mathematical model as given in Table D-2.

It is believed that plastic yielding of the rail members is a very desirable characteristic which will tend to reduce the lateral decelerations imparted to the vehicle during a bridge barrier collision as shown in Figure 13. If plastic deformations of the rail members are desirable, these findings suggest that a design based on the AASHO 10-kip force would perform satisfactorily when subjected to the *HRB Circular 482* full-scale dynamic test conditions of 60 mph and 25°. However, the rail components of the barrier railing would require considerable repairs subsequent to a high-speed collision.

Design of Splice Connections

In the discussion on rail member splice connections, the authors indicated that beaming action and resistance to axial rotation were successfully maintained. The interpretation of resistance to axial rotation is not well-defined.

It appears from the design of the splice connections and from visible observations of the photographs included in the report that the splices offered little restraint to rotation of particularly the one panel length rail members. These findings indicate that the splice connections functioned primarily as shear type connections.

It is felt that a shear connection should be restricted as to the magnitude of rotation and expansion permitted, otherwise, the railing will contribute little to the strength of the barrier railing system if an excessive post deflection or post-anchorage failure were to occur, as in Test 114, for some unforeseen reason. For example, if an anticipated expansion of a bridge deck joint is ¼ in., then the railing splice connections in the same area should be permitted to expand only the same magnitude. In order for a splice connection to be most effective it should be designed to transmit a large portion of a bending moment. Thus, by restriction of the rotation and expansion of a splice connection, an added feature of safety is incorporated into the barrier railing system.

Design of Post Base-Plate Connection

The dimensions of the post and the base plate, which were both fabricated from A36 Steel, are shown in Figure D-1. The section modulus of the 5/16-in. fillet welded connection between the two ⅝-in. post plates and the ½-in. base plate would be approximately the same as that of the post. Therefore, to simplify the calculations the section modulus of the post will be used. The section modulus of the post about an axis parallel (S_{xx}) to the barrier railing is 16.9, and the section modulus about an axis perpendicular (S_{yy}) to the barrier railing is 45.0 in.³. Designing in accordance with AASHO (1965) and assuming that the rail members are continuous over two posts, the working stresses in the fillet welds can be computed by use of Eq. C-3, as follows:

TABLE D-2

COMPARISON OF LATERAL IMPACT FORCES COMPUTED BY MATHEMATICAL MODEL AND PLASTIC THEORY

IMPACT TEST CONDITIONS				IMPACT FORCE (KIPS) (EQ. C-2)		IMPACT FORCE (KIPS) (EQ. 5)
TEST NO.	IMPACT SPEED (MPH)	IMPACT ANGLE (DEG)	RAIL LENGTH (FT)	$c=1.0$	$c=1.4$	
112[a]	58.5	25	8	17.1	23.9	13.8
113[a]	61.5	23	10	13.7	19.2	14.1

[a] The average lateral displacement of the barrier railing in Test 112 was 4 in. and in Test 113 it was 3 in.

TABLE D-3

LATERAL IMPACT FORCES COMPUTED BY ELASTIC THEORY (RAIL MEMBERS A 36)

	STRESS LEVEL (KSI)	LOAD/RAIL (KIPS)	
		L = 8 FT	L = 10 FT
Allowable	10.0	2.6	2.2
	20.0	5.5	4.4
Determined	22.5	6.3	5.0
yield strength	30.0	8.3	6.7
	40.0	11.1	8.9
	50.0	13.9	11.2
	59.5	16.5	13.2

(1) No Snagging

$$f_a = f_y\ (d)$$
$$= \frac{P_A(a+b)}{4m}\left[\frac{2m}{S_{xx}} + \frac{1}{S_{yy}}\right] \qquad \text{(C-3)}$$
$$= \frac{10(12.5+24.5)}{4(2)}\left[\frac{2(2)}{16.9} + \frac{1}{45.0}\right]$$
$$= 12.0 \text{ ksi}$$

(2) Snagging (see Figure D-2)

$$f_a = 12.0 + \frac{(10.8)(18.5)}{45.0}$$
$$= 16.4 \text{ ksi}$$

If no snagging occurs, the 12.0 ksi stress level would be less than the 14.2 ksi allowable stress level permitted for a A233 Class E 70 series electrode fillet weld; whereas, if snagging is taken into consideration, the computed stress level would exceed the value permitted by approximately 15 percent. These findings would indicate that the 5/16-in. fillet welds were for all practical purposes adequate. However, recalling the relationship established between AASHO and BPR in Appendix C, the actual stress level in the fillet welds could be expected to be in the plastic range. Further verification has now been provided by this series of tests conducted by California, in which localized failures of the fillet welds had occurred as shown in Figure D-3. These localized failures were readily visible in several of the photographs included in the California report.

Because of the complex nature of the stress distribution in an area of a connection, a theoretical analysis would at best be a rough approximation. Therefore, if these localized fillet weld failures are undesirable because of safety and/or maintenance considerations, it is suggested that an adjustment in the design load of AASHO be made when designing a connection to restrain a vehicle under the impact conditions of 60 mph and 25°. Analyzing the situation now from a different viewpoint, it is to be emphasized that localized failures, which would have essentially the same effect as a plastic hinge (mechanism) forming, would appear to be highly desirable to reduce the lateral decelerations of the colliding vehicle as shown in Figure 13. It would also appear that if one attempts to achieve a plastic hinge in a barrier railing design, one must have a method, such as the mathematical equations presented in this report, to determine the magnitude of the actual impact force, which takes into consideration the vehicle and roadway characteristics.

At the present time, the specifications of AASHO (1965) or BPR (1962) are considered by the research agency engineers not sufficient to provide the design engineer the assur-

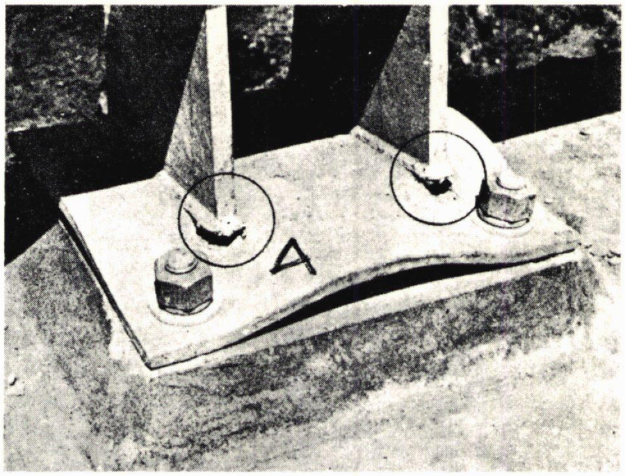


Figure D-3. Localized fillet weld connection failures. (Note that the base plate is also deformed and the anchorage bolts were forced out of vertical alignment, which could possibly constitute a maintenance replacement problem.)

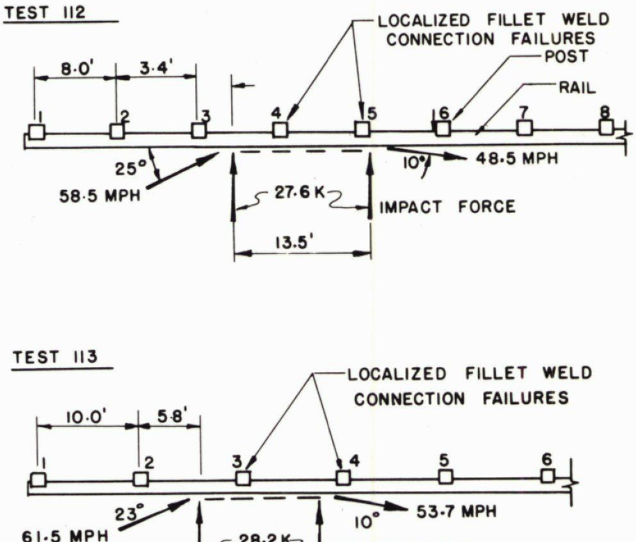


Figure D-4. California Type 8 barrier rail configuration—localized fillet weld connection failures as function of location and magnitude of impact force.

ance that localized connection failures will or will not occur in areas of a connection unless, however, full-scale dynamic tests are conducted.

Post Spacing and Damage

The authors indicate that no appreciable increase in post damage was noted when the post spacing was changed from 8 ft in Test 112 to 10 ft in Test 113; in both cases, there were failures of the fillet welded connection between the post flange and base plates, and the extent of the base plate plastic bending deformations was almost identical.

It is felt that as long as the structural strength of the rails is adequate and within either the elastic or plastic range, one would not expect an increase in post damage for an increase in the spacing. This is apparent by use of the mathematical model. Neglecting beaming action and the inertia forces of the continuous rail members, and referring to Figure D-4, the extent of damage was approximately the same to posts 4 and 5 of Test 112, because at some instant in time the 27.6 lateral impact force was fully resisted by each post.

Likewise, the damage to the post 3 of Test 113 was almost identical to that of the previous test, because it was subjected to approximately the same magnitude of lateral force of 28.1 kips. It appears from observations of the photographs included in the California report that the extent of damage to post 4 was slightly less because, referring to Figure D-4, it was never subjected to the full impact of the load.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Highway Research Board
National Academy of Sciences
2101 Constitution Avenue
Washington, D.C. 20418

Rep. No.	*Title*
—*	A Critical Review of Literature Treating Methods of Identifying Aggregates Subject to Destructive Volume Change When Frozen in Concrete and a Proposed Program of Research—Intermediate Report (Proj. 4-3(2)), 81 p., $1.80
1	Evaluation of Methods of Replacement of Deteriorated Concrete in Structures (Proj. 6-8), 56 p., $2.80
2	An Introduction to Guidelines for Satellite Studies of Pavement Performance (Proj. 1-1), 19 p., $1.80
2A	Guidelines for Satellite Studies of Pavement Performance, 85 p.+9 figs., 26 tables, 4 app., $3.00
3	Improved Criteria for Traffic Signals at Individual Intersections—Interim Report (Proj. 3-5), 36 p., $1.60
4	Non-Chemical Methods of Snow and Ice Control on Highway Structures (Proj. 6-2), 74 p., $3.20
5	Effects of Different Methods of Stockpiling Aggregates—Interim Report (Proj. 10-3), 48 p., $2.00
6	Means of Locating and Communicating with Disabled Vehicles—Interim Report (Proj. 3-4), 56 p. $3.20
7	Comparison of Different Methods of Measuring Pavement Condition—Interim Report (Proj. 1-2), 29 p., $1.80
8	Synthetic Aggregates for Highway Construction (Proj. 4-4), 13 p., $1.00
9	Traffic Surveillance and Means of Communicating with Drivers—Interim Report (Proj. 3-2), 28 p., $1.60
10	Theoretical Analysis of Structural Behavior of Road Test Flexible Pavements (Proj. 1-4), 31 p., $2.80
11	Effect of Control Devices on Traffic Operations—Interim Report (Proj. 3-6), 107 p., $5.80
12	Identification of Aggregates Causing Poor Concrete Performance When Frozen—Interim Report (Proj. 4-3(1)), 47 p., $3.00
13	Running Cost of Motor Vehicles as Affected by Highway Design—Interim Report (Proj. 2-5), 43 p., $2.80
14	Density and Moisture Content Measurements by Nuclear Methods—Interim Report (Proj. 10-5), 32 p., $3.00
15	Identification of Concrete Aggregates Exhibiting Frost Susceptibility—Interim Report (Proj. 4-3(2)), 66 p., $4.00
16	Protective Coatings to Prevent Deterioration of Concrete by Deicing Chemicals (Proj. 6-3), 21 p., $1.60
17	Development of Guidelines for Practical and Realistic Construction Specifications (Proj. 10-1), 109 p., $6.00
18	Community Consequences of Highway Improvement (Proj. 2-2), 37 p., $2.80
19	Economical and Effective Deicing Agents for Use on Highway Structures (Proj. 6-1), 19 p., $1.20
20	Economic Study of Roadway Lighting (Proj. 5-4), 77 p., $3.20
21	Detecting Variations in Load-Carrying Capacity of Flexible Pavements (Proj. 1-5), 30 p., $1.40
22	Factors Influencing Flexible Pavement Performance (Proj. 1-3(2)), 69 p., $2.60
23	Methods for Reducing Corrosion of Reinforcing Steel (Proj. 6-4), 22 p., $1.40
24	Urban Travel Patterns for Airports, Shopping Centers, and Industrial Plants (Proj. 7-1), 116 p., $5.20
25	Potential Uses of Sonic and Ultrasonic Devices in Highway Construction (Proj. 10-7), 48 p., $2.00
26	Development of Uniform Procedures for Establishing Construction Equipment Rental Rates (Proj. 13-1), 33 p., $1.60
27	Physical Factors Influencing Resistance of Concrete to Deicing Agents (Proj. 6-5), 41 p., $2.00
28	Surveillance Methods and Ways and Means of Communicating with Drivers (Proj. 3-2), 66 p., $2.60
29	Digital-Computer-Controlled Traffic Signal System for a Small City (Proj. 3-2), 82 p., $4.00
30	Extension of AASHO Road Test Performance Concepts (Proj. 1-4(2)), 33 p., $1.60
31	A Review of Transportation Aspects of Land-Use Control (Proj. 8-5), 41 p., $2.00
32	Improved Criteria for Traffic Signals at Individual Intersections (Proj. 3-5), 134 p., $5.00
33	Values of Time Savings of Commercial Vehicles (Proj. 2-4), 74 p., $3.60
34	Evaluation of Construction Control Procedures—Interim Report (Proj. 10-2), 117 p., $5.00
35	Prediction of Flexible Pavement Deflections from Laboratory Repeated-Load Tests (Proj. 1-3(3)), 117 p., $5.00
36	Highway Guardrails—A Review of Current Practice (Proj. 15-1), 33 p., $1.60
37	Tentative Skid-Resistance Requirements for Main Rural Highways (Proj. 1-7), 80 p., $3.60
38	Evaluation of Pavement Joint and Crack Sealing Materials and Practices (Proj. 9-3), 40 p., $2.00
39	Factors Involved in the Design of Asphaltic Pavement Surfaces (Proj. 1-8), 112 p., $5.00
40	Means of Locating Disabled or Stopped Vehicles (Proj. 3-4(1)), 40 p., $2.00
41	Effect of Control Devices on Traffic Operations (Proj. 3-6), 83 p., $3.60

* Highway Research Board Special Report 80.

Rep. No.	Title
42	Interstate Highway Maintenance Requirements and Unit Maintenance Expenditure Index (Proj. 14-1), 144 p., $5.60
43	Density and Moisture Content Measurements by Nuclear Methods (Proj. 10-5), 38 p., $2.00
44	Traffic Attraction of Rural Outdoor Recreational Areas (Proj. 7-2), 28 p., $1.40
45	Development of Improved Pavement Marking Materials—Laboratory Phase (Proj. 5-5), 24 p., $1.40
46	Effects of Different Methods of Stockpiling and Handling Aggregates (Proj. 10-3), 102 p., $4.60
47	Accident Rates as Related to Design Elements of Rural Highways (Proj. 2-3), 173 p., $6.40
48	Factors and Trends in Trip Length (Proj. 7-4), 70 p., $3.20
49	National Survey of Transportation Attitudes and Behavior—Phase I Summary Report (Proj. 20-4), 71 p., $3.20
50	Factors Influencing Safety at Highway-Rail Grade Crossing (Proj. 3-8), 113 p., $5.20
51	Sensing and Communication Between Vehicles (Proj. 3-3), 105 p., $5.00
52	Measurement of Pavement Thickness by Rapid and Nondestructive Methods (Proj. 10-6), 82 p., $3.80
53	Multiple Use of Lands Within Highway Rights-of-Way (Proj. 7-6), 68 p., $3.20
54	Location, Selection, and Maintenance of Highway Guardrails and Median Barriers (Proj. 15-1(2)), 63 p., $2.60
55	Research Needs in Highway Transportation (Proj. 20-2), 66 p., $2.80
56	Scenic Easements—Legal, Administrative, and Valuation Problems and Procedures (Proj. 11-3), 174 p., $6.40
57	Factors Influencing Modal Trip Assignment (Proj. 8-2), 78 p., $3.20
58	Comparative Analysis of Traffic Assignment Techniques with Actual Highway Use (Proj. 7-5), 85 p., $3.60
59	Standard Measurements for Satellite Road Test Program (Proj. 1-6), 78 p., $3.20
60	Effects of Illumination on Operating Characteristics of Freeways (Proj. 5-2) 148 p., $6.00
61	Evaluation of Studded Tires—Performance Data and Pavement Wear Measurement (Proj. 1-9), 66 p., $3.00
62	Urban Travel Patterns for Hospitals, Universities, Office Buidings and Capitols (Proj. 7-1), 144 p., $5.60
63	Economics of Design Standards for Low-Volume Rural Roads (Proj. 2-6), 93 p., $4.00
64	Motorists' Needs and Services on Interstate Highways (Proj. 7-7), 88 p., $3.60
65	One-Cycle Slow-Freeze Test for Evaluating Aggregate Performance in Frozen Concrete (Proj. 4-3(1)), 21 p., $1.40
66	Identification of Frost-Susceptible Particles in Concrete Aggregates (Proj. 4-3(2)), 62 p., $2.80
67	Relation of Asphalt Rheological Properties to Pavement Durability (Proj. 9-1), 45 p., $2.20
68	Application of Vehicle Operating Characteristics to Geometric Design and Traffic Operations (Proj. 3-10), 38 p., $2.00
69	Evaluation of Construction Control Procedures—Aggregate Gradation Variations and Effects (Proj. 10-2A), 58 p., $2.80
70	Social and Economic Factors Affecting Intercity Travel (Proj. 8-1), 68 p., $3.00
71	Analytical Study of Weighing Methods for Highway Vehicles in Motion (Proj. 7-3), 63 p., $2.80
72	Theory and Practice in Inverse Condemnation for Five Representative States (Proj. 11-2), 44 p., $2.20
73	Improved Criteria for Traffic Signal Systems on Urban Arterials (Proj. 3-5/1), 55 p., $2.80
74	Protective Coatings for Highway Structural Steel (Proj. 4-6), 64 p., $2.80
75	Effect of Highway Landscape Development on Nearby Property (Proj. 2-9), 82 p., $3.60
76	Detecting Seasonal Changes in Load-Carrying Capabilities of Flexible Pavements (Proj. 1-5(2)), 37 p., $2.00
77	Development of Design Criteria for Safer Luminaire Supports (Proj. 15-6), 82 p., $3.80
78	Highway Noise—Measurement, Simulation, and Mixed Reactions (Proj. 3-7), 78 p., $3.20
79	Development of Improved Methods for Reduction of Traffic Accidents (Proj. 17-1), 163 p., $6.40
80	Oversize-Overweight Permit Operation on State Highways (Proj. 2-10), 120 p., $5.20
81	Moving Behavior and Residential Choice—A National Survey (Proj. 8-6), 129 p., $5.60
82	National Survey of Transportation Attitudes and Behavior—Phase II Analysis Report (Proj. 20-4), 89 p., $4.00
83	Distribution of Wheel Loads on Highway Bridges (Proj. 12-2), 56 p., $2.80
84	Analysis and Projection of Research on Traffic Surveillance, Communication and Control (Proj. 3-9), 48 p., $2.40
85	Development of Formed-in-Place Wet Reflective Markers (Proj. 5-5), 28 p., $1.80
86	Tentative Service Requirements for Bridge Rail Systems (Proj. 12-8), 62 p., $3.20

Synthesis of Highway Practice

1	Traffic Control for Freeway Maintenance (Proj. 20-5, Topic 1), 47 p., $2.20
2	Bridge Approach Design and Construction Practices (Proj. 20-5, Topic 2), 30 p., $2.00
3	Traffic-Safe and Hydraulically Efficient Drainage Practice (Proj. 20-5, Topic 4), 38 p., $2.20

THE NATIONAL ACADEMY OF SCIENCES is a private, honorary organization of more than 700 scientists and engineers elected on the basis of outstanding contributions to knowledge. Established by a Congressional Act of Incorporation signed by President Abraham Lincoln on March 3, 1863, and supported by private and public funds, the Academy works to further science and its use for the general welfare by bringing together the most qualified individuals to deal with scientific and technological problems of broad significance.

Under the terms of its Congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the Federal Government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the Government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the Government.

THE NATIONAL ACADEMY OF ENGINEERING was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its Act of Incorporation, adopted Articles of Organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the Federal Government, upon request, on any subject of science or technology.

THE NATIONAL RESEARCH COUNCIL was organized as an agency of the National Academy of Sciences in 1916, at the request of President Wilson, to enable the broad community of U. S. scientists and engineers to associate their efforts with the limited membership of the Academy in service to science and the nation. Its members, who receive their appointments from the President of the National Academy of Sciences, are drawn from academic, industrial and government organizations throughout the country. The National Research Council serves both Academies in the discharge of their responsibilities.

Supported by private and public contributions, grants, and contracts, and voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

THE DIVISION OF ENGINEERING is one of the eight major Divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

THE HIGHWAY RESEARCH BOARD, organized November 11, 1920, as an agency of the Division of Engineering, is a cooperative organization of the highway technologists of America operating under the auspices of the National Research Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of transportation. The purpose of the Board is to advance knowledge concerning the nature and performance of transportation systems, through the stimulation of research and dissemination of information derived therefrom.

HIGHWAY RESEARCH BOARD
NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL
2101 Constitution Avenue Washington, D. C. 20418

ADDRESS CORRECTION REQUESTED

NON-PROFIT ORG.
U.S. POSTAGE
PAID
WASHINGTON, D.C.
PERMIT NO. 42970

003901
LIBRARIAN
NATL ACADEMY SCIENCES

P.O. STOP 44