

Safer Roadside Structures

A State-of-the-Art Report

Prepared for



United States Steel Corporation

Texas Transportation Institute
Texas A&M University
College Station, Texas

Foreword

*“Carriages without horses shall go,
And accidents fill the world with woe.”*

—From “Prophecy”
by Martha (Mother) Shipton (1488-1561)

These prophetic words written more than 400 years ago have come to haunt highway engineers concerned with providing a safe highway environment for the travelling public. Highway engineers and researchers have been engaged for more than a decade in a concerted effort to conceive, design, and test new devices which incorporate safety features. Much of this work has been reported in technical literature, which is often not readily available to the engineer in the field, who must make decisions on the design or selection of safer roadside appurtenances. This report has been written with the needs of these practicing engineers in mind. A bibliography containing more than one hundred references has been included for those readers who have need for more detail than has been presented herein.

Acknowledgements

This state-of-the-art study of safer roadside structures has been prepared from information obtained from many sources, including published and unpublished literature, highway department engineers, engineers in industry, manufacturers, and others. A selected bibliography has been included from which much background information was obtained; other information was collected through personal discussions with highway engineers and researchers, review of responses to a questionnaire received from many highway departments, and information supplied by field representatives of the United States Steel Corporation.

It is impossible to express thanks to each of the many engineers and researchers who have contributed the information which is contained in this report. But it is clearly evident that without their endeavors a state-of-the art study could not have been written. So, to each of them we extend the heartfelt appreciation of all of those who travel the highways of our great nation.

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Summary

This report on safer roadside structures has been presented in seven parts for easy reference by engineers who may be concerned about various aspects of highway safety. This report has been written in quasi-technical language without footnotes in order to inform the reader without diverting his attention. The chronological bibliography contained in Part H of this report contains the major source documents from which this report was prepared. Certainly, details are of interest to each engineer in his decision-making processes; however, this study is intended to present the broad outlines of safer roadside concepts rather than the details. It is hoped that practicing engineers will find this report informative, and that it will encourage them to investigate available concepts, employ those which have merit, and continue to seek improvements.

Part A contains introductory remarks which emphasize the magnitude of the problem of highway safety. Part B presents information concerning warrants for guardrails, and Part C describes barriers which have safe characteristics, and which have been adopted for installation by several agencies. A discussion of break-away sign supports is presented in Part D. This safety device, conceived by D. L. Hawkins, an engineer in the Texas Highway Department, has been widely accepted by many highway engineers. More than 15,000 break-away structures have been installed on Texas highways since November 1965. Break-away devices for lighting support structures are described in Part E. A discussion of impact attenuation systems is contained in Part F, and a program to reduce hazards on existing highways in California is discussed in Part G of this report.

Safer Roadside Structures

Part A

Introduction

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Safer Roadside Structures

Introduction

The tremendous increase of automobiles and highway mileage in the United States since World War II has exposed the public to the hazards of highway travel. These hazards include people, vehicles, and road-way; the three comprise the hazardous highway environment. This hazardous environment is familiar to each of us who drives an automotive vehicle, whether he be a member of the travelling public, a highway designer, engineer, administrator, or anyone who might be involved with the highway program. Mr. Alfred E. Johnson, the Executive Director of the American Association of State Highway Officials wrote in *American Highways* in April, 1967:

“Great emphasis is being placed on highway traffic safety in the United States at this time, especially at the Federal level.

The modern traffic safety movement actually started in the late 1930's and the 1940's. However, there were so many highway needs competing for the inadequate available highway funds that the public was apathetic to spending much for safety programs.

When the fatality rate was first computed in the United States in 1925, it was 17.5 fatalities per hundred million vehicle miles. It declined steadily until it reached 5.2 in 1961. It then began to rise gradually to 5.7 in 1964, and at the present time, it is about 5.6.

There has to be a reason for this increase and it is felt that the accident reporting and analysis that has existed has been inadequate to determine the causes of accidents. In some instances, the cause is probably so complex as to defy diagnosis, especially if complicated human behavior may be a factor.

At the present time, the fatality rate on our newer Interstate System highways, which are freeways, is 2.5, while it is 8.6 for the conventional older two-lane facilities.

The State highway departments have not been insensitive to the traffic safety problem, and they have been constantly upgrading geometric and structural designs, and improving traffic control devices.

It is believed that such practices have had a significant role in decreasing the fatality rate over the years.

The new Interstate System of freeways in the United States is approximately 58% complete and is now saving over 4000 lives a year. When the 41,000-mile system is completed, it will save 8000 lives a year.

Since the Interstate System program was launched in 1956, there has been a 60% increase in motor vehicles.

At the present time, 92 million motor vehicles are registered. The population of the United States is approximately 200 million.

One hundred and five million of these people are registered drivers, and 50% of the population is under 25 years of age. In fact, traffic accidents are the greatest cause of death and injury for Americans under 35 years of age, and the accident history of those under 25 years of age is disproportionately high.”

This discussion by Mr. Johnson recapitulates information which is familiar to all highway engineers and others involved with the highway safety program. Mr. Johnson discusses the fact that in the United States most all streets, roads, and highways are under the jurisdiction of the State or a local political subdivision of the State. He also recounts the fact that the National Safety Council started the practice of predicting the number of people that would be killed on the Nation's highways during holiday periods or over long weekends. In most instances, he notes that the number of

people killed or injured rarely exceeded what would normally be expected for a similar time interval. Certainly public awareness of the problem of traffic safety has been increased through the efforts of the National Safety Council. Mr. Johnson continued:

"When one considers the numbers and exposure rate involved, it becomes evident that everyone is not going to be killed in a highway accident. A person has one chance in 4000 of being a traffic fatality during any one year. In presenting the matter in another way, a person would live 36 full lifetimes and drive the national average of 10,000 miles per year, for 50 years, before his 'number' would come up as a highway fatality.

However, one person out of every two in the United States can expect to be injured because of a motor vehicle accident during his lifetime.

At the present time, fatalities are in excess of 50,000 a year, 2 million people are injured, and the total economic loss due to traffic accidents is approximately \$10 billion annually.

Death and injury on the highways is a definite health hazard that cannot be tolerated, but is one that cannot be eliminated as long as people are in motion. It is the goal in the United States to achieve an irreducible minimum, whatever that might be.

It became evident some time ago that as the highway deaths per year reached 50,000, the public would require some action.

In 1964, the American Association of State Highway Officials named a Special Traffic Safety Committee, chaired by the President of the Association, and composed of Chairmen of appropriate key engineering Committees having responsibilities in the overall traffic safety field. This approach was taken because of the importance of the subject and because traffic safety transcends the responsibilities or interests of any one Committee of the Association.

The first action of the Special Committee was to draft guide lines for national standards for a comprehensive highway traffic safety program. This occurred in 1965.

During the same year, Federal legislation was enacted for improving traffic safety on the Federal-aid systems, which generally encompass the State highway systems.

The Federal Bureau of Public Roads also encouraged the several State highway departments to undertake 'spot improvement' programs to correct high accident locations or potentially high accident locations.

This refers to such work as widening bridges, making appropriate changes in geometrics or in applying some appropriate traffic engineering technique.

Many States had regularly conducted such a program, however, some were hesitant to go into the program on the philosophy that more safety could be accomplished by spending the available money on producing new modern facilities.

The other philosophy is that the public is best served by spending a significant portion of funds in correcting 'booby traps' on the existing road system.

In 1966, two landmark pieces of Federal legislation came into being in an emotionally-charged atmosphere, that resulted in a new safety agency being established under the Federal Highway Administration in the new Cabinet Department of Transportation.

The existing Federal Bureau of Public Roads is also placed in the Federal Highway Administration of the new Department.

The new Federal legislative actions were:

1. The National Traffic and Motor Vehicle Safety Act, and
2. The Highway Safety Act

The first has to do with requiring certain things in the manufacture of motor vehicles to improve safety.

The second has to do with developing a comprehensive traffic safety program at the State level. It furnishes Federal-aid for such a program with a certain percentage being earmarked for use by local governments in the State.

The money is not available for building or improving roads, or for furnishing traffic control devices. It provides that the State program is to be administered through the office of the Governor of the State, which is somewhat of a departure in the fifty year old Federal-aid highway program which has always been carried on through the duly constituted State highway departments.

National standards are to be developed covering the following subjects.

- a. Periodic motor vehicle inspection
- b. Motor vehicle registration
- c. Motorcycle safety
- d. Driver education
- e. Driver licensing
- f. Codes and laws.
- g. Traffic courts
- h. Alcohol in relation to traffic safety
- i. Identification and surveillance of accident locations
- j. Traffic records
- k. Emergency medical service
- l. Highway design, construction, and maintenance
- m. Traffic control devices

The controlling standards are developed in cooperation with the States and other appropriate agencies and officials. Each State program must then be in accordance with the standards and any State not complying on or after January 1, 1969, will be penalized 10% of its Federal-aid highway fund.

It is expected that highway design, construction and maintenance standards relating to the State highway systems, will continue to be developed by the State highway departments through the AASHO process, as will be the Official Sign Manual for the Interstate System.

The Manual for Traffic Control Devices for all other roads and streets will continue to be developed by the National Joint Committee on Uniform Traffic Control Devices, of which AASHO is one of the five parent organizations."

Mr. Johnson continued his discussion and described the suggestion, made early in 1966 by the Federal Highway Administrator, that the American Association of State Highway Officials make a study of existing roads and streets and develop recommendations on improving traffic safety similar to other studies that AASHO had done on geometric design practice and related matters in the past. The assignment was given to a Special AASHO Committee on Traffic Safety. Several people from the Bureau of Public Roads participated in the tour and study along with four members of the AASHO Committee on Traffic, who were named as Special Observers to accompany and assist the Special Committee.

The Report of the Special AASHO Traffic Safety Committee, dated February, 1967, has been published and a limited distribution has been made. The report is intended to be used as the basis for an accelerated traffic safety program in the United States. The inspection took the group to various parts of the country; and

all classes of public roads, urban and rural, were viewed in daylight and night conditions and also during adverse weather conditions. Much of the study centered on the fixed objects within the limits of the right-of-way that constitute lethal obstructions. Examples of these objects are trees, highway sign supports, guardrail ends. Lighting supports, overhead bridge piers and abutments, culvert headwalls, bridge end turnouts, protruding curbs, traffic islands and utility poles.

Even on the most modern freeway, the occurrence of a vehicle running off the road is too frequent. Approximately 35% of all motor vehicle accidents result from a motor vehicle out of control hitting some obstruction. Such accidents result in a large number of fatal accidents.

The objective of the study by the Special AASHO Committee on Traffic Safety, was to identify those aspects of design and operation on facilities in various sections of the country which could be improved to increase safety to the travelling public, and to improve the quality of traffic service.

The principal conclusions and recommendations of the committee, with respect to roadside design and appurtenances, are taken from the report and summarized in the following:

- “1. In the development of plans for highway improvements, all elements of design should be reviewed to insure that any feature likely to be associated with injury or accident to the highway user is eliminated or minimized in its effect. Special attention must be directed to the safety characteristics of the roadside so that they too are the result of deliberate design and not an unpredictable byproduct of grading, drainage or other construction activity.
2. An intensive crash program to remove roadside hazards on existing streets and highways and to engineer the roadsides of new facilities with safety as a major criterion should have a paramount place in the highway program of each State. Only in this way will the motorist who inadvertently leaves the traveled way have adequate protection against death or injury.
3. Design standards more liberal than the minimums prescribed will often increase safety. Constant field checks of the operating conditions with existing and new designs are recommended for evaluation of their effectiveness and cost efficiency.
4. Embankment and cut slopes 6:1 or flatter can often be negotiated by a vehicle with some chance for recovery and these should therefore be provided where possible.
5. A full shoulder width should be carried across all structures. Shoulders should be flush with the adjoining through lane. Contrast in color or texture or both, and the use of a conspicuous edge-line marking are recommended for the guidance of drivers and to discourage use of shoulders by through traffic.
6. To increase safety when vehicles leave the pavement, a clear recovery area, free of physical obstruction, should be provided along the roadway 30 feet or more from the edge of the traveled way in rural areas. Corrective programs should be undertaken at once to eliminate from the roadside or to relocate to protected positions such hazardous fixed objects as trees, drainage structures, massive sign supports, utility poles, and other ground-mounted obstructions that are now exposed to traffic. Where this is impracticable, an adequate guardrail or other type of protection should be provided.
7. The gore area at the divergence of two roadways, as at the exit from a freeway, must be kept clear of heavy structures, unyielding sign supports and similar installations that would not readily give way if struck by a vehicle out of control. The standard EXIT sign is a permissible installation in the gore but should always be mounted on a breakaway type support.

8. The use of appurtenances along the roadside must be reviewed continually to minimize the number of such objects that can be struck by vehicles. Each jurisdiction should periodically review its signing and retain only the essential signs. The continuing demands for additional nonessential highway signs must be firmly resisted.
9. Many ground-mounted highway signs can be placed farther from the pavement, laterally, and still retain their effectiveness. Under favorable viewing conditions, a minimum distance of 30 feet from the edge of pavement to the edge of sign is recommended. The detailed location of all individual signs and sign supports should be subjected to a field review of existing highway conditions prior to installation whenever possible to assure maximum effectiveness and safety.
10. On multilane facilities with heavy traffic volumes, additional use of overhead sign locations is recommended to provide information equally visible to all traffic and for specific lane assignment.
11. Much greater use of overhead crossing structures for support of overhead signs is recommended.
12. The adoption and use of a suitable breakaway or yielding design for lighting and sign supports by all jurisdictions is recommended. Concrete bases for these supports should be flush with the ground level.
13. A consistent nationwide policy for the application of guardrail should be established at the earliest possible date. Designers must keep in mind that the objective of guardrail installation is to lessen the hazard to highway users, and not to protect any part of the roadway. Guardrail should only be used where the result of striking an object or leaving the roadway would be more severe than striking the rail.

All guardrails on the approaches to structures must be securely attached to the structure. All approach ends of guardrail must be flared away from the road, anchored to the ground, or otherwise blended into the approach environment. A dike or curb should not be used in front of guardrail. When guardrail is used as a median barrier at high-exposure locations, the spacing of mounting posts should not exceed 6'3" to provide adequate strength and resistance against penetration. The bolt attaching the rail to the post should include a suitable washer to prevent the bolt pulling through the rail.

14. On new construction a median width of about 60-80 feet is highly desirable. Median barriers of a suitable design should be considered where the median is 30 feet or less in width.

Openings in a median lead to operating hazards and should be avoided. Proper signing should be installed to prohibit the general use of crossovers constructed for essential maintenance, patrolling or emergency purposes. Movable barriers for the necessary crossovers should also be considered.

Narrow grassed medians are undesirable. To eliminate maintenance operation hazards, narrow medians of this type should always be paved.

15. The adoption and use of two-span bridges for overpass crossing divided highways is recommended to eliminate the bridge piers normally placed adjacent to the outside shoulders.
16. Where twin bridges are used on divided highways, adequate median barrier protection for motorists should be provided. For separations up to 20 or 30 feet, the median should normally be made continuous by bridging the undercrossing."

This report of the Special Committee gives an excellent description of existing deficiencies in the roadside environment.

A significant statement has been made by Professors Huelke and Gikas who have been studying fatal automobile accidents in and about Washtenaw County, Michigan, since November, 1961. These two investigators are physicians, and the purpose of their study is to determine the causes of death of the occupants—the body areas injured, as well as the structures which were impacted to produce lethal injuries. The method they employ is as follows: The police of the area call them to all on-scene fatalities, anytime of the day or night. Photographs of the vehicles, roadway, and victims are taken using 35-mm color film. As of January 1, 1965, they had investigated 111 accidents in which 146 automobile occupants were killed. No pedestrians, cyclists, car-train, or truck-truck accidents were included in the data.

In studying each case certain conditions became important. The occupant would have lived if he was not driving too fast, if he had not fallen asleep, if he had not been drinking, if he had worn a seat belt, if the interior of the vehicle had been designed for safety, if the roadway had been better designed, if no roadway obstacles had been present, etc. The authors contend that there will always be the possibility of an automobile accident when there is a man-machine combination. Thus, in addition to attempting to decrease accidents by driver education, vehicle inspection, etc., the only alternatives are improvement in vehicular design or crash attenuation (especially the interior), and clearance of roadside obstacles. They observe that if an individual is going to lose control of his vehicle for any reason, the roadway must be designed to prevent cross-median accidents, and obstacles must be removed from the roadside so that serious or fatal injuries will not occur.

However, such observations only serve to emphasize the problems, the details of which, all of us are too painfully aware. The present report is addressed to the description of a number of devices which are already in existence on an experimental or operational basis, which may serve to produce a less hazardous roadside environment. In other words, certain devices have been proven through laboratory experimentation or full-scale crash studies by engineers and researchers throughout the United States. The results of such studies and investigations are not always readily available to the highway designer, traffic engineer, or other personnel who are faced with making the decisions on ways and means to provide a safer highway on which to travel. The reader will find in the following pages, information which should be of value to him in his decision-making processes.

The Highway Safety Act which has to do with developing a comprehensive traffic safety program at the State level, is certainly a broad-based piece of legislation, which in the long run, should provide National standards to serve as a guide to public officials responsible for the provision of public highways, roads and streets. Such a broad-based policy, by its very nature, will take time to develop, promulgate, and implement. It is conceivable that the operational phase of the results of such a program may be as much as twenty years in the future. In the meantime, several states have been involved in investigations which may serve as interim guidelines. Several examples of such research accomplished with the cooperation of the Bureau of Public Roads and the Highway Research Board will be discussed here.

Ways and means to improve safety of the roadway

A recommendation to provide a lateral clearance of 30 feet or more from the edge of the travelled way in rural areas is obviously prudent, however, where this is not possible a guardrail or other safety structures must be provided. It is apparent that indiscriminate installation of guardrail can worsen rather than improve the safety of the roadside environment.

Recently Deleys and McHenry of the Cornell Aeronautical Laboratory, (NCHRP Report 36, 1967) have published a review of current practice on highway guardrails. This research was aimed at providing design engineers with a choice of effective guardrail systems and with warrants for their use. The Foreword to this report by the Staff of the Highway Research Board is presented here:

"Engineers concerned with guardrail design and accident prevention will be those having most interest in this report. The research stemmed from a need for providing design engineers with a choice of effective guardrail systems and with warrants for their use. Toward this end, approximately six man-months of effort were devoted to an evaluation of existing data on the current state-of-the-art of guardrail design and warranting criteria with a view toward defining additional needed research. The results of the study are useful in providing both information essential to the conduct of additional research and a concise statement of national and international practices and current research.

Design engineers have been at a disadvantage for lack of a suitable basis for choice of effective guardrail systems (including median installations) and warrants for their use. Although a number of tests have been conducted on various systems, there had been a need for a comparison and appraisal of the resulting data in terms of structural stability of the systems, damage to vehicles, injury to occupants, maintenance and repairs, interference with roadway maintenance operations, visibility, etc. Similarly, a review of the basis for warrants has been needed.

The Cornell Aeronautical Laboratory has researched this problem by means of a combination of literature search and direct inquiries to numerous individuals and agencies in the United States and foreign countries. A review, summary, and evaluation of the present state-of-the-art has resulted and an extensive annotated bibliography of the reports and articles reviewed in the study has been developed. Throughout the review, primary attention was given to the consideration of three aspects pertaining to guardrails; i.e., (1) technical or factual basis for warrants, (2) prevailing conditions of off-road vehicle motions and guardrail impacts, and (3) criteria for guardrail structural design. Conclusions have been drawn concerning present gaps in the technology and recommendations have been made for the research considered necessary to fill these gaps. With the increasing emphasis being placed on highway safety, this compilation of pertinent information should be of considerable interest to both designers and other researchers.

This document constitutes a final report on the first phase of the research, which was intended to critically analyze past and current research and to define additional needed research. The second phase of the research will be under contract in June 1967."

For the benefit of highway engineers who do not have the latest research reports at their fingertips, excerpts will be made from the current literature. The following is quoted from the report by Deleys and McHenry.

"There is a need to provide highway design engineers with a choice of effective guardrail systems (including median installations) and warrants for their use. Although a number of agencies have conducted tests of various systems, the resulting data need to be compared and appraised in terms of structural stability, damage to vehicles, injury to occupants, maintenance and repair, interference with roadway maintenance operation, visibility, etc. A similar review of the basis for warrants is needed.

The foregoing paragraph is the problem statement for the Phase I studies of NCHRP Project 15-1 performed by the Cornell Aeronautical Laboratory. The results, consisting of a review, summary and evaluation of the present state-of-the-art of highway guardrail design and warranting criteria, are presented in this report. The scope of this investigation was limited primarily to considerations of three aspects of guardrails; (1) technical or factual basis for warrants, (2) prevailing conditions of off-road vehicle motions and guardrail impacts, and (3) criteria for guardrail structural design. The primary objectives of the study were (1) to search for, summarize and critically evaluate existing data on guardrail design, performance, and warrants, and (2) to define needs for additional research effort."

The researchers at Cornell obtained information by a literature search and letters of inquiry which were sent to more than 150 individuals and agencies, both in the United States and in foreign countries: a large amount of material was obtained through approximately 100 responses to the request for information. This NCHRP report presents an excellent review of data concerned with guardrail design and performance, warrants, vehicle impact conditions, and research currently in progress. It is recommended for reading by all researchers in highway safety.

Other highway researchers and engineers in the several states have been engaged in the development and crash testing of designs for safer guardrails, bridge rails, ground mounted sign supports, illumination supports, and impact attenuation or energy absorbing devices. All of these efforts have been directed toward a reduction in the hazards which fixed obstacles on the roadway present to the travelling public. Discussions of current developments of these several safer roadside structures will be presented in the following sections of this report.

Safer Roadside Structures

Part B

Guardrail Warrants

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Warrants for highway guardrail

At the annual meeting of the Highway Research Board in January 1961, a subcommittee of the Committee on Guardrails and Guide Posts was appointed to study the conditions for which guardrail is needed and the geometric requirements for its installation. Highway Research Board Special Report 81 (1964) contains the results of the study by this subcommittee, and excerpts from this Special Report follow.

Highway guardrail

Determination of Need and Geometric
Requirements with Particular Reference
To Beam-Type Guardrail*

Introduction

There are no comprehensive design guides in general use to establish where guardrail is needed and how it should be installed dimensionally or geometrically. This study is intended to begin the process by which complete design criteria for installation of guardrail will be developed. Structural aspects of guardrail design are distinctly different from the geometric and installation warrant aspects. Accordingly, they are part of a separate—although related—study now in progress by the Committee.

The terms “guardrail” and “guide rail” are used among the State highway departments to represent a similar type of protective device. For the purpose of this report the two terms are assumed to be synonymous and only the term “guardrail” is used.

The subcommittee first intended that its study include all varieties of guardrail. It was assumed that the design guides could be so developed as to fit all types of guardrail. Investigation showed this approach to be futile because of significantly different requirements for different types of guardrails. It was decided that any results thus evolved would have been too general and of little practical value.

The subcommittee then agreed to study each type of guardrail individually. By dealing with one form of guardrail at a time, design guides in sufficient detail could be developed for effective application in a reasonable length of time. Similar study and development of design criteria for other types of guardrail, it was decided, would be undertaken later.

The guardrail selected for initial study was the universal W-section formed metal sheet—hereafter referred to as the “beam-type guardrail.” The reason for considering this particular form first was its predominant use by the States. In a 1962 study by the AASHO Committee on Planning and Design Policies (1) it was shown that of 41 States reporting the forms of guardrails used, 40 use beam-type guardrail in varying degrees, 15 use some cable guardrail, and several use other forms. Of the 40 States which use beam-type guardrail, 24 do so exclusively or nearly so. Another study (1961) conducted by the U.S. Bureau of Public Roads (2) indicates that the 47 States which have provisions in their standard specifications for steel beams or steel plate, or both, 31 specify beam-type guardrail only.

The selection of the beam-type guardrail for development of design criteria in no way constitutes a preference in the use of this form of guardrail over others. On the contrary, it is hoped that this initial endeavor will stimulate further study of other types.

*Presented at the 43rd Annual Meeting of the Highway Research Board, January 1964, by Jack E. Leisch, Chairman of the special subcommittee for preparation of the report.

The objective of this report is three-fold: (1) To formulate the means for determining the need for guardrail; (2) To develop criteria for establishing geometric features of guardrail installations, including traffic operational characteristics, safety factors, and aesthetic considerations; and (3) To point up, in the process, further areas for research which, in turn, would bring about more appropriate and more complete standards of design.

General warrants for use of guardrail

General warrants for use of guardrail have been reasonably well defined. Specific warrants and installation details need formulation, however, to achieve appropriate balance in design.

Guardrails may be needed under the following basic conditions:

- A. Roadways on embankment, particularly on high fills and/or with steep side slopes.
- B. Divided highways with narrow medians, carrying large volumes of traffic.
- C. Highways with roadside obstacles and hazards such as structures and appurtenances.

Under the first category—roadways on embankment—the relative need for guardrail depends on a number of controlling conditions, the more important of which, are:

Basic controls

- 1. Height of embankment.
- 2. Steepness of fill slope.

Related site controls

- 3. Width of shoulder or roadway.
- 4. Horizontal curvature.
- 5. Gradient or profile conditions.
- 6. Roadside conditions—exclusive of obstacles in the form of appurtenances which are treated separately—such as toe walls, bodies of water, boulders on side slope, steeply sloping ground away from toe of slope, adjoining roadway or other development near toe of slope.
- 7. Climatic conditions, such as susceptibility to snow, ice and fog.

Accident experience

- 8. Accident experience, particularly frequent and localized off-the-roadway-type accidents.

Highway classification and design designation

- 9. Type or classification of highway.
- 10. Traffic characteristics (design designation), including speed, volume, and composition of traffic.

Specific values or warrants for installation of guardrail on embankments are presented under “Determination of Guardrail Need—Embankment Conditions.”

The need for guardrail under the second category—between roadways of divided highways—depends basically on the same variables, 1 to 10, noted for the first category. Where the paired one-way roadways are on independent alignments and profiles, these variables are directly applicable. When the one-way roadways form a conventional divided highway, particularly where the median is narrow, the more important variables determining guardrail need are highway type or classification, traffic characteristics (primarily volume), and width of median. Horizontal curvature, gradient, and climatic conditions are important but to a lesser degree. Accident experience is particularly significant on existing facilities. The need for guardrail under the third category—obstacles in the form of structures and highway appurtenances on the roadside—depends largely on the

lethal potentials of the obstacle, highway type or classification, traffic characteristics, and accident experience.

The three general categories involving guardrail need are covered in specific terms under succeeding headings.

In considering warrants for installation of guardrail, the following fundamental principle of design should be applied:

Every highway should be designed, through judicious arrangement and balance of geometric features, to preclude or minimize the need for guardrail.

A corollary of this principle follows:

Responsibility for application of guardrail and other protective devices should rest primarily with the design engineer who determines and coordinates all highway design features.

Maximum roadside safety with minimum use of guardrail entails thorough study and careful design. Thus, guardrail need cannot be left to field determination. Final check and adjustment of guardrail installations through field inspection and operational experience, however, should be part of the overall procedure. This step should be coordinated through the design office to assure compliance with design criteria and standards.

Determination of guardrail need for embankment conditions

Development of warrants

The guardrail need for embankment conditions cannot be determined by direct research. Although some findings are of considerable value in this regard, much reliance must be placed on actual experience and practice of the various State highway departments in the development of design criteria.

Practically all States agree that where 4:1 slopes are used guardrail may be omitted unless other hazards are present (1). Recent limited studies (3) indicate that side slopes should be in the order of 6:1 to allow out-of-control vehicles to "ride out" a slope at high speeds. This finding actually supports, rather than contradicts, the general practice of eliminating guardrail with 4:1 slopes. The flatter slopes would be desirable, but the use of 4:1 slopes is considered reasonable and compatible with the degree of protection provided by guardrail installations and other design features of the highway. Further research may show that guardrail should be used with slopes somewhat flatter than 4:1. Meanwhile, it should be considered acceptable design to omit guardrail with 4:1 slopes appropriately rounded at top and bottom. However, 5:1 and 6:1 slopes to enhance roadside safety should be used where feasible.

Where slopes are steeper than 4:1, the need for guardrail varies with the height of fill, h , which is the difference in elevation between the outer edge of shoulder and the point at which the side slope intercepts the natural ground. The need for guardrail under such conditions cannot be determined directly, but actual practice can serve as a guide. The AASHO survey (1) provides information which can be used in developing design criteria. According to the survey, with 37 States reporting on this feature, embankment height above which guardrail is used with slopes steeper than 4:1 is as follows:

Height of Fill, h , in Ft, with Slopes Steeper than 4:1	Percentage of States Using a Control Within Range of h Indicated for Installation of Guardrail
0-6	13
7-10	35
11-15	30
16-20	22

This shows that nearly one-half of the States (48%) install guardrail when the height of fill exceeds 10 feet. About three-quarters of the States (78%) call for guardrail on embankment when the height exceeds 15 feet. The average fill height with slopes steeper than 4:1 above which guardrail is installed, for all States reporting, is 12 feet.

This information provides a general guide from which the basic values given in Table B-1 have been established. These should be tempered by the other factors affecting need for protective treatment—namely, items 3 through 10 in the previous section.

Such an approach to the problem has been developed and reported recently by Grunerud (4). This method assigns a weighted value to the variables of height of fill, fill slope, horizontal curvature, icing conditions, width of roadway, nature of fill slope, depth of water at toe fill, and profile gradient. The sum of arbitrary values for each condition forms a total value. This is compared with a warranting value established for each highway type and range of average daily traffic. Guardrail is considered required if the total value for the particular case exceeds the warranting value.

The subcommittee examined this method thoroughly, concluded that it was an excellent procedure, and decided to adopt it in principle. With the use of the basic values given in Table B-1 for combinations of fill slope and heights of fill, however, it was found more appropriate to use factors as a product rather than as the plus or minus values employed by Grunerud.

Before adjustment factors could be established for the many variable conditions, it was necessary to expand the relations in Table B-1 and assign weighted values. To each combination of slope and height of fill indicated in Table B-1, a basic value of 50 was assigned. Then, larger values were assigned to higher fills and smaller values were assigned to lower fills for each slope to produce a full range of values, as given in Table B-2.

Thus, the basic controls for determining guardrail need for embankment conditions (items 1 and 2 of previous section) are accounted for directly in Table B-2 in terms of guardrail need indices.

In specific cases, the values in Table B-2 should be altered to account for the effects of related site conditions (items 3 to 7, inclusive). Appropriate adjustment factors are given in Table B-3. These were established through trial and error to produce consistent results. The maximum composite adjustment factor normally would not exceed about 1.5 for primary highways. For secondary highways the worst possible condition would result in a composite adjustment factor of about 2.5.

Item 8, dealing with accident experience, normally would not be considered directly in determining the guardrail need index on new construction. Those features which may affect safety are assumed to be accounted for generally by consideration of items 1 and 2 in Table B-2 and by adjustment factors developed for items 3 and 7 in Table B-3. On existing highways, on the other hand, accident experience at specific locations may become the major consideration in determining guardrail need. Accident experience alone, in some cases, may be reason enough for installing guardrail. In other instances, accident experience may be used as a factor in addition to those in Table B-3 in evaluating guardrail requirements on existing facilities. The adjustment factor, however, generally would be a matter of judgment for each individual case.

Items 9 and 10, dealing with type and classification of highway and with traffic characteristics (volume and speed), could be accounted for readily by assigning specific values to the warranting values for various combinations of these items. This would allow each State to choose its own warranting (index) values for the several highway traffic conditions. Thus, through the combined use of Tables B-

Table B-1. Minimum height of fill on primary highways requiring guardrail

Fill Slope	Height of Fill, h (ft)
1½:1	8
2:1	10
2½:1	12
3:1	15

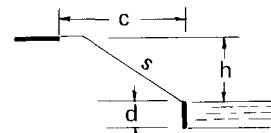
Table B-2. Basic guardrail need index for embankment conditions*

Height of Fill, h (ft)	Need Index				
	1½:1	2:1	2½:1	3:1	4:1
4	40	35	30	25	10
6	45	40	35	30	15
8	50	45	40	35	20
10	55	50	45	40	25
12	60	55	50	45	30
15	65	60	55	50	35
20	70	65	60	55	40
30	75	70	65	60	45
40 +	80	75	70	65	50

*For precipitous condition: General—relatively great drop = 80
Critical—with high-level view = 100

Table B-3. Adjustment factors to be applied to basic values of guardrail need index

Item	Factor
Shoulder width, over all (ft). 12 - 15	1.00
10	1.05
8	1.10
6	1.15
Horizontal curvature, as related to design speed: Tangent or flat curve	1.00
Intermediate curve	1.05
Inside curve: Min. or near min., or isolated interm. curve	1.10
Isolated min. or near min. curve, or curves over 10°	1.15
Outside curve: Min. or near min., or isolated interm. curve	1.20
Isolated min. or near min. curve, or curves over 10°	1.25
Downgrade or profile conditions: 2% or less	1.00
3%	1.05
4%, or moderate crest V. C. in comb. with horiz. curve	1.10
5%	1.15
6%, or extreme crest V. C. in comb. with horiz. curve	1.20
7% or more	1.25
Roadside conditions: Ground sloping away from toe of fill at the rate of: ¹ 10% or less	1.00
15%	1.10
20%	1.15
25% or more	1.20
Boulders on slope, or road or building at toe of slope ¹	1.20
Wall at toe of slope: ² Add 5 × d to height of fill, h, and enter Table B-2 with the larger equivalent h for the s indicated.	
Water at toe of slope: ² Add 8 × d to height of fill, and enter Table B-2 with the larger equivalent h for the s indicated.	
Climatic conditions: ³ Freezing and thawing: Little to none	1.00
Moderate	1.05
Severe	1.15
Fog, prevalent	1.10



¹Use only one adjustment factor for these items.

²Requirements in conjunction with toe wall or water at toe of slope should also be checked in Table B-4; use guardrail if either Table B-4 or solution by application of Table B-3 indicates the need.

³Use only one adjustment factor for this item, either freezing and thawing, or fog.

2 and B-3, the method outlined here could be used in all States, although the warrants might vary somewhat between States.

For purposes of this discussion, only two highway traffic conditions are assumed: (a) primary highways, including expressways, having relatively high volumes and high speeds; and (b) secondary highways, carrying generally low volumes. Warranting values assigned to these are:

Primary highways	50
Secondary highways	70

Warranting values for intermediate facilities might lie between values of 50 and 70; or the range might be increased or decreased, if deemed appropriate, to values of between 40 and 80. It might be logical to have as many as five designations for various highway classifications and traffic volume groups, and use warranting values of 40, 50, 60, 70 and 80.

The procedure for determining whether or not guardrail is required for a specific section of highway on embankment becomes a simple matter of finding the appropriate need index in Table B-2, adjusting the index by factors in Table B-3, and comparing the adjusted need index with the appropriate warranting value. If the adjusted need index is numerically greater than the warranting value, guardrail is required; if the need index is the lesser of the two values, guardrail is not required.

Table B-4 presents an additional guide rationalized for determination of guardrail need in conjunction with vertical drop-offs near the roadway. This supplements the material developed in Table B-3 for conditions of toe wall or water at toe of slope.

Thus, by the use of these tables, a practical arithmetical method for determination of guardrail need is presented and illustrated in the following examples. Although this method is quite complete, the numerous steps necessary for its application are apt to be time consuming. Therefore, a graphic analysis method, or nomograph, has been developed (Figure B-1) to simplify and shorten the procedure.

Table B-4. Guardrail requirements in conjunction with vertical or near-vertical drop-offs in vicinity of roadway.

Guardrail required when:					
c equals	10	20	30	40	(ft)
d equals or exceeds	1	2	3	4	(ft)
c is horizontal distance, in feet, from edge of traveled way to a drop-off, such as curbing, wall, or edge of water.					
d is height of curbing or wall, or depth of water, in feet, at point of drop-off. (Drop-off may be at toe of fill slope; see sketch, Table B-3 and Figure B-1).					
The sequence through the nomograph may be as follows: enter at left with given height of fill; proceed right to appropriate curve designating the fill slope; at this point turn 90 degrees and project downward to the given shoulder width; then proceed to the right to the next family of lines designating horizontal curvature; repeat this process through the remaining variables; read result—the adjusted guardrail need index—on the right vertical scale. The nomograph may also be used in the reverse order by entering at the lower right with the desired need index and reading the result—maximum height of fill without guardrail—on the upper left scale.					
Example 1					
Determine whether or not guardrail is required on a fill section of 2-lane primary highway in a rural area under the following conditions: height of fill, 12 ft on 4:1 slope; shoulder width, 10 ft overall; outside horizontal curve, $D = 12^\circ$;					

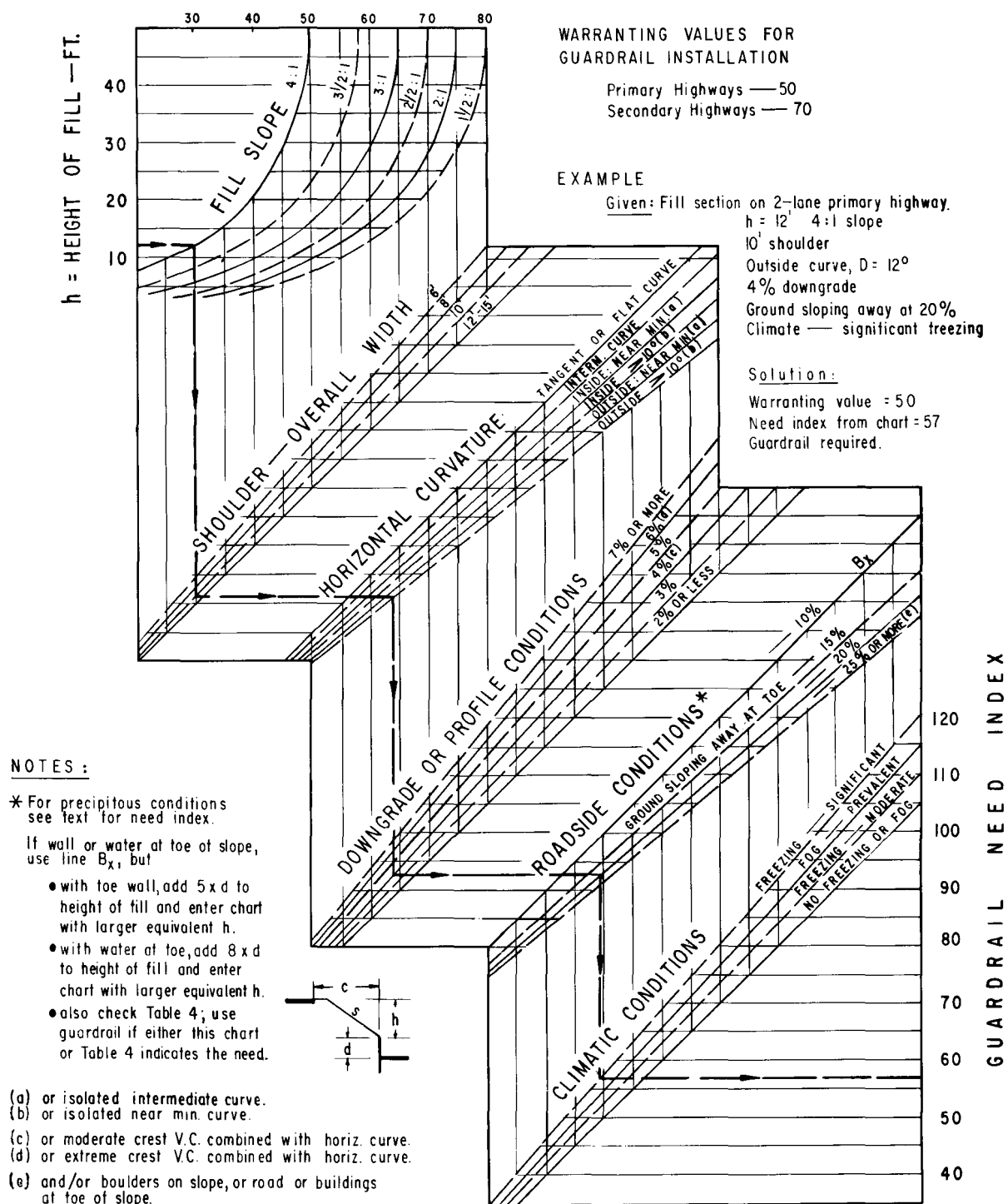


Figure B-1. Guardrail need index chart

downgrade, 4%; ground sloping away from toe of fill at approximately 20%; severe freezing during winter months.

Solution (Arithmetical):

Guardrail need index, basic value (Table 2) for $h = 12$ ft and 4:1 slope = 30.

Adjustments (Table B-3):

Shoulder width	1.05	Roadside condition	1.15
Horizontal curvature . . .	1.25	Climatic condition	1.15
Profile condition	1.10		

Combined factor = $1.05 \times 1.25 \times 1.10 \times 1.15 \times 1.15 = 1.91$

Need index = $30 \times 1.91 = 57$.

Warranting value for primary highway is 50.

Guardrail is required, because need index is greater than 50.

Solution (Graphical):

Enter the nomograph (Figure B-1) at upper left with $h = 12$ ft, follow the arrows indicated in chart (which take into account the various given conditions), and find guardrail need index of 57 at lower right.

Warranting value for primary highway is 50.

Guardrail is required, because need index is greater than 50.

Example 2

Determine whether or not guardrail is required on a fill section of expressway having the following characteristics: design speed, 60 mph; shoulder width, 15 ft overall; inside curve, $D = 2.5^\circ$; downgrade, 3%; height of fill, 6 ft on 4:1 slope, with toe wall 3 ft high; climatic conditions, favorable.

Solution (Arithmetical):

Because a toe wall is present, Table B-4 should be checked first. The wall is located laterally from edge of traveled way a distance $c = 15 + (6 \times 4) = 39$ ft. The drop-off at this point is $d = 3$ ft. For this combination of c and d , guardrail is not called for in Table B-4. It is necessary, therefore, to proceed to Tables B-2 and B-3.

Before entering Table B-2 to obtain the basic need index, the equivalent height of fill, as affected by the toe wall, must be determined in accordance with the requirements in Table B-3. Accordingly, equivalent $h = 6 + (5 \times 3) = 21$ ft.

Guardrail need index, basic value in Table B-2 for $h = 21$ ft and 4:1 slope = 41.

Adjustments (Table B-3):

Shoulder width	1.00	Roadside condition	1.00
Horizontal curvature . . .	1.05	Climatic condition	1.00
Profile condition	1.05		

Combined factor = $1.05 \times 1.05 = 1.10$

Need index = $41 \times 1.10 = 45$.

Warranting value for expressway is 50.

No guardrail is required, because need index is less than 50.

Solution (Graphical):

Because a toe wall is present, Table B-4 should be checked first in accordance with the note at lower left of Figure B-1. As in the arithmetical solution, guardrail is not required by this criterion, so it is necessary to proceed with the chart solution in Figure B-1.

Before entering the chart, the equivalent height of fill, as affected by the toe wall, must be determined in accordance with the note at the lower left of the figure. Accordingly, equivalent $h = 6 + (5 \times 3) = 21$ ft.

Using $h = 21$ ft and proceeding through the chart with the given conditions, a guardrail need index of 45 is found.

As for the arithmetical solution, no guardrail is indicated because the need index is less than the warranting value of 50.

Example 3

Same conditions as in Example 2, except that a frontage road is provided at the base of fill slope adjoining the retaining wall. Determine if guardrail is required.

Solution:

The procedure through the chart is the same as in Example 2 except that for the “roadside condition” the extreme lower line is used. This produces a need index of 54.

Guardrail is required, because the road at the bottom of embankment caused the need index to increase beyond the warranting value of 50.

Example 4

A major highway with 12-ft shoulders and 4:1 fill slopes follows the shoreline of a lake. The depth of water at the base of roadway embankment is 4 ft. The height of fill is 3 ft above the water. All other conditions—curvature, profile, climate—are favorable. Determine if guardrail is required.

Solution:

Because a body of water is present near the roadway, Table B-4 should be checked as indicated at the lower left of Figure B-1. The edge of water is located laterally from the traveled way a distance $c = 12 + (3 \times 4) = 24$ ft. Entering Table B-4 with this dimension and $d = 4$ ft, it is apparent that guardrail is required. It is not necessary, therefore, to proceed through the chart in Figure B-1. (As a matter of interest, the chart solution—using an equivalent $h = 3 + (8 \times 4) = 35$ —shows a need index of 47.)

Guardrail is required.

Example 5

A secondary highway designed with fill slopes of 2:1 has a long sustained embankment on one side. The height varies from several feet to a maximum of 25 ft. Shoulder width is 10 ft overall; alignment is favorable; natural ground slopes away from the fill at 15%; climatic conditions include moderate freezing and thawing. Find the height of fill at which guardrail is required.

Solution:

The warranting value for installation of guardrail on secondary highways, as set up in this report, is 70. To find the height of fill, enter the chart at the lower right with a guardrail need index equal to the warranting value of 70. Proceed through chart in reverse order, using the conditions given. Intercepting the curve in the chart for 2:1 slope yields a height of fill, h , equal to 14 ft.

Guardrail is required when $h = 14$ ft or more.

The use of this or any other method will not necessarily give complete or final results. The procedure establishes the need for guardrail at certain locations, predicated largely on the rate of fill slope selected in the design of the highway. This is only the initial step. Judgement must be applied and final adjustment made manually. In doing so, the entire highway should be analyzed in sections of reasonable length to produce sufficient balance of guardrail installation and overall roadside protection. Guardrail should be eliminated where practicable by improving other design features. The alternative most often available is flattening fill slopes as discussed later herein.

Critical height of fill for guardrail installation

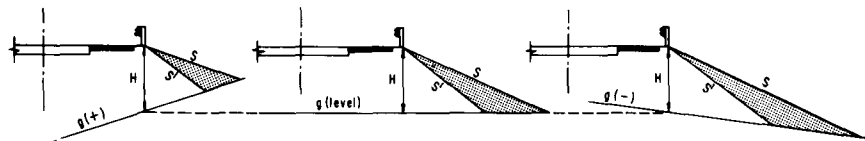
Highways should be designed, through good arrangement and balance of geometric features, to preclude or minimize the need for guardrail. The flattening of fill slopes is one of the primary means of realizing this objective.

There seems to be general agreement among highway officials that guardrail may be omitted where the fill slope is 4:1 or flatter, unless physical obstructions or other hazards are present. There is no agreement, however, as to height of fill below which 4:1 or flatter slopes should be used in lieu of guardrail. This decision has been based largely on economics. The cost of embankment for flattening a slope from 2:1 to 4:1 compared with the cost of guardrail is the basis normally used for establishing the critical height of fill. Usually the comparison is predicated on direct costs of embankment and guardrail per lineal foot of highway. Based on such analyses, most of the States use a critical height of fill somewhere between 10 and 15 ft. Below these heights it is considered more economical to use flat slopes than to install guardrail.

Critical heights of fill for guardrail installation are given in Figure B-2, where the standard formula for critical height of fill is shown at the upper center. Relevant items are rates of slope before and after flattening, the slope of the natural ground, and the unit costs of guardrail and embankment. The second formula is for a basic slope of 2:1 and a flattened slope of 4:1. The tabulation was prepared on the basis of the latter formula, together with assumed costs of embankment in the range of \$0.40 to \$0.75 per cubic yard, and costs of guardrail in the range of \$3.00 to \$7.00 per lineal foot.

SYMBOLS:

- H — Height of fill at which cost of slope flattening is balanced by cost of guardrail — feet.
 F — Fill cost of highway embankment — dollars per cubic yard.
 G — Guardrail cost — dollars per lineal foot.
 g — Ground slope, plus when sloping upward and minus when sloping downward from toe of highway embankment — feet per foot.
 s — Fill slope after flattening — feet per foot.
 s' — Fill slope before flattening — feet per foot.



STANDARD FORMULA

$$H = 3 \sqrt{\frac{6G}{F \left(\frac{1}{s+g} - \frac{1}{s'+g} \right)}}$$

When $s = 0.25$ (4:1)
 and $s' = 0.50$ (2:1)

$$H = 3 \sqrt{\frac{36(1+6g+8g^2)}{F}}$$

HEIGHT OF FILL (H in Feet) AT WHICH COST OF GUARDRAIL INSTALLATION EQUALS COST OF ALTERNATIVE FLATTENING OF FILL SLOPE BASED ON FLATTENING OF SLOPE FROM 2:1 TO 4:1, ASSUMING NO ADDITIONAL RIGHT-OF-WAY REQUIRED

GROUND SLOPE (g)		EMBANKMENT FILL COST (F) PER CUBIC YARD, DOLLARS																							
		\$ 0.40						\$ 0.50						\$ 0.60						\$ 0.75					
		GUARDRAIL COST (G) PER LINEAL FOOT, DOLLARS						GUARDRAIL COST (G) PER LINEAL FOOT, DOLLARS						GUARDRAIL COST (G) PER LINEAL FOOT, DOLLARS						GUARDRAIL COST (G) PER LINEAL FOOT, DOLLARS					
		3.00	3.25	3.50	4.00	6.00	7.00	3.00	3.25	3.50	4.00	6.00	7.00	3.00	3.25	3.50	4.00	6.00	7.00	3.00	3.25	3.50	4.00	6.00	7.00
UP (+)	STEEP 0.125 (8:1)	19.5	20.0	21.0	22.5	27.5	30.0	17.5	18.0	19.0	20.0	24.5	26.5	16.0	16.5	17.0	18.5	22.5	24.5	14.0	15.0	15.5	16.5	20.0	21.5
	MEDIUM 0.050 (20:1)	16.0	17.0	18.0	19.0	23.0	25.0	14.5	15.0	16.0	17.0	20.5	22.0	13.0	14.0	14.5	15.5	19.0	20.5	12.0	12.5	13.0	13.0	17.0	18.0
LEVEL		14.0	15.0	15.5	16.5	20.0	21.5	12.5	13.0	13.5	14.5	18.0	19.5	11.5	12.0	12.5	13.5	16.5	18.0	10.0	11.0	11.0	12.0	14.5	16.0
DOWN (-)	MEDIUM -0.050 (20:1)	12.0	12.5	13.0	14.0	17.0	18.5	11.0	11.5	12.0	12.5	15.5	16.5	10.0	10.5	10.5	11.5	14.0	15.0	9.0	9.0	9.5	10.0	12.5	13.5
	STEEP -0.125 (8:1)	8.5	9.0	9.5	10.0	12.5	13.5	8.0	8.0	8.5	9.0	11.0	12.0	7.0	7.5	8.0	8.5	10.0	11.0	6.5	7.0	7.0	7.5	9.0	10.0

Figure B-2. Critical height of fill for guardrail installation

The unit prices for embankment are considered to be representative for rural conditions. Under average conditions and for the purpose of this report, the cost of installing beam-type guardrail is generally in the range of \$2.85 to \$3.25 per lineal foot with post spacing of 12 ft 6 in. Tests have indicated, however, that post spacing for high-speed conditions should be 6 ft 3 in. (5, 6, 7). This increases the costs by 15 to 20 percent.

Critical heights of fill are presently established on direct comparison of the cost of guardrail and the additional cost of embankment due to slope flattening, assuming no additional right-of-way necessary. It was on this basis that critical heights of fill in the range of 10 to 15 feet had been indicated.

Such comparisons would be more accurate if predicated on the approximate life of the various elements involved. Taking the life of the fill conservatively as 20 years, it is appropriate to assume that the guardrail would be replaced at least once during this period. Therefore, it is logical to establish the critical height of fill by equating the cost of embankment due to slope flattening with at least double the initial cost of guardrail.

The use of the higher estimate for cost of guardrail in this analysis is further justified when maintenance expenses are included. Such expenses are due to additional operations in grass cutting and (in some areas) snow removal, introduced by the guardrail. Other related expenses in conjunction with guardrail installations have to do with the maintenance of 2:1 fill slopes. Compared with 4:1 embankments, the steeper slopes are more costly to maintain due to greater erosion as well as difficulties in using tractor-powered equipment.

Right-of-way costs normally would not enter into this analysis, because the width of land reserve is set to a reasonable standard on modern highways.

Using a unit price of \$7.00 per lineal foot of guardrail (based on double the cost of initial installation), a representative cost of \$0.50 per cubic yard of embankment, a relatively level ground line, and assuming no additional right-of-way, the critical height of fill would be on the order of 20 ft (see Figure B-2).

The critical height of fill varies considerably with the slope of natural ground. For example, a relatively steep upward ground slope results in a critical height of nearly 27 ft, whereas a relatively steep downward slope produces a critical height of 12 ft. By comparison, 20 ft is the indicated critical height for level ground.

The critical height of fill, H , as shown in Figure B-2, is measured vertically from the outer edge of shoulder to a point on the ground line directly beneath. Another measurement of height of fill affecting guardrail warrants has been discussed previously and is enumerated in Tables B-1 and B-2. This is the outer height of fill, h , represented by the difference in elevation between the outer edge of shoulder and the toe of fill slope.

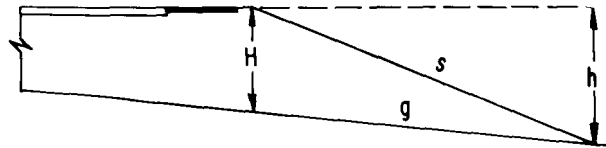
Corresponding values of these two heights of fill may be expressed by

$$h = \frac{sH}{(s + g)} \quad (1)$$

in which the units for s and g are the same as those indicated in Figure B-2.

The relationship between the critical height of fill, H , and the outer height of fill, h , is demonstrated in Figure B-3. The previously noted values of $H = 27$ ft for upward ground slope and $H = 12$ ft for downward ground slope would correspond to $h = 21$ ft and $h = 16$ ft, respectively, for the 2:1 slope; and to $h = 18$ ft and $h = 24$ ft, respectively, for the 4:1 slope.

- h = Difference in elevation between the outer edge of shoulder and the point at which the side slope intercepts natural ground.
- H = Critical height of fill, measured vertically below outer edge of shoulder — feet. (See Fig. 2)
- s = Fill slope — feet per foot.
- g = Ground slope, plus when sloping upward and minus when sloping downward from toe of highway embankment — feet per foot.



STANDARD FORMULA:

$$h = \frac{sH}{(s+g)}$$

RELATIONSHIP BETWEEN H AND h

GROUND SLOPE g	CRITICAL HEIGHT OF FILL H , FEET *	CORRESPONDING OUTER HEIGHT OF FILL h , FEET, WHEN	
		$s = 2:1$	$s = 4:1$
UPWARD (8:1)	27	21	18
LEVEL	20	20	20
DOWNWARD (8:1)	12	16	24

* Based on guardrail cost of \$ 7.00 per lineal foot, and embankment cost of \$ 0.50 per cu.yd.; see Fig. 2.

Figure B-3. Critical height of fill related to outer height of fill

The foregoing excerpt from Highway Research Board Special Report 81 describes the procedures used by many states to establish the need for guardrail on an embankment. The reader is directed to the research report for information concerning the geometric requirements for guardrails along the shoulder.

The special report also contains a discussion of the geometric requirements for protective treatment at highway appurtenances such as illumination supports, sign supports, and particularly sign supports located in the gore of exit roadways. Developments of break-away concepts for such appurtenances which have occurred since the Special Report was published, will be discussed later in this report. Highway Research Board Special Report 81 provides a design guide for determining where guardrail is required, but some of the recommendations have been superseded by more recent developments. The principles of guardrail installation covered in the HRB Special Report applies for the most part to various forms of guardrail, although the type specifically selected in this study was the universal W-section form. The subcommittee noted that the criteria were not intended to serve as a recommendation for the use of this type of guardrail in preference to some other form, but their objective was to furnish a design guide for the installation of one particular type of guardrail which was then widely used throughout the United States. The development of design criteria for other types of guardrail and some performance information have become available since 1964, these developments will be discussed later.

A more recent report titled "Objective Criteria for Guardrail Installations" was published by the California Division of Highways, Traffic Department, in July, 1966. The study was based on single vehicle accident statistics compiled during 1963 and 1964, and evaluated when the installation of guardrail is safer for the occupants of a colliding vehicle than an unprotected embankment or fixed object.

The primary purpose for placing guardrail on embankments is to increase the relative safety of ran-off-road type accidents at embankment locations. This includes increasing the safety to vehicle occupants and to people and property off the roadway. The California study was aimed at an objective determination of the combination of roadway geometry and embankment conditions which require guardrail placement to maximize the safety of ran-off-road accidents at embankment locations.

The variables considered for analysis as having an effect on the severity of down-the-embankment accidents were as follows:

1. Height of embankment.
2. Slope of embankment.
3. Size of embankment surface material.
4. Firmness of embankment material.
5. Slope of original ground at toe of embankment.
6. Water at the toe of the embankment.
7. Fixed objects on slope.
8. Speed of vehicle.

After examining these variables, a selection was made of the following four variables for use in a multiple regression analysis:

1. Height of embankment (including natural hillside height).
2. Slope of embankment.
3. Size of embankment material.
4. Slope of the "original ground" at toe of embankment.

Not using the other four variables could possibly reduce the degree of correlation, but the following were reasons for not using these variables:

1. The firmness of the embankment material is difficult to evaluate because it is variable over time.
2. Fixed objects contribute considerably to severity but this is a factor which should be considered separately from embankment conditions.
3. Water at the toe of the slope should also be considered separately.
4. Speed definitely contributes to severity but is not a predictable quantity for any single vehicle involved in an accident. Generally, however, if large accident samples are used, it is expected that the distribution and range in speeds for accidents within each embankment category will be similar. If this is true, speed would not affect the relative severity between embankment categories.

Reports of all 1963 and 1964 single vehicle embankment guardrail accidents were obtained. Each accident report was read to verify that embankment guardrail was involved. The investigators report a total of 331 embankment guardrail accidents: 14 fatal, 147 injuries to occupants, and 170 property damage only. Computer analyses of the accident data and the roadway variables indicated that height and slope of embankment were the significant variables. A comparison of Embankments vs. Guardrails based on this research is presented in Figure B-4. The investigators state:

"Figure B-4 is not completely objective, because the guardrail need is determined only on reduced severity basis. Because guardrail can be a costly item, it would be economically feasible to install it only at potentially high frequency ran-off-road accident locations (i.e., on the outside of horizontal curves, on higher volume roadways, etc.).

If an embankment condition plots in the lower area of the chart, guardrail should not be installed on that embankment unless other severe conditions may warrant it (i.e., numerous fixed objects on the slope or at the toe, permanent water at the toe of slope, etc.).

It should be kept in mind that at locations where the guardrail need is determined, guardrail placement is not the only method to minimize the Severity Index. For lower embankment heights (say less than 20 feet) with steep slopes (steeper than 2:1), it may be more economical to flatten the slope."

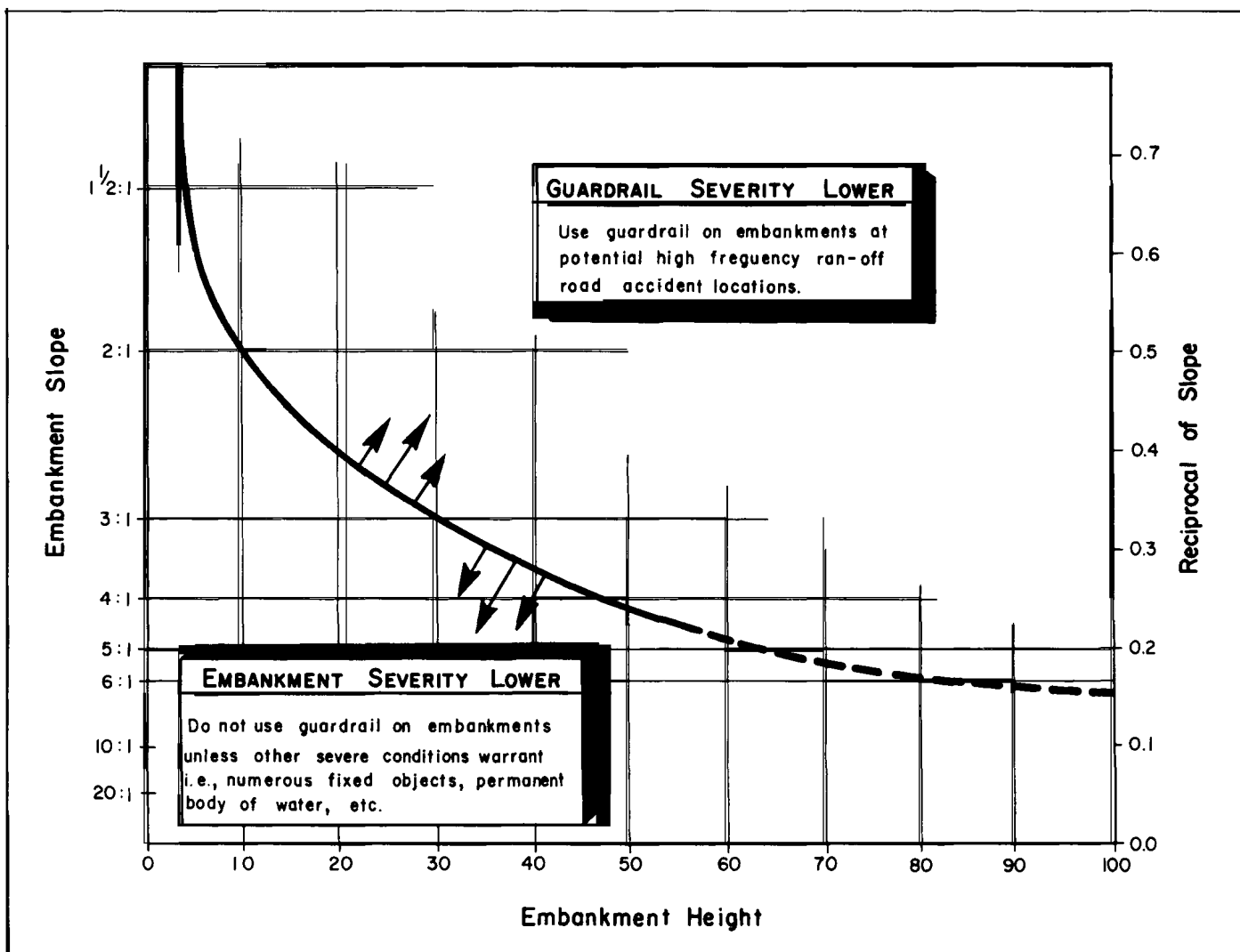


Figure B-4 Severity comparison of embankments vs. guardrail

The California study also considered the need for guardrail adjacent to freeway fixed objects. The Severity Index is defined in the report and relates the number of fatal accidents, the number of injury producing accidents, the number of property damage only accidents, and the total number of accidents. Interested readers should consult the report for details. Table B-5 illustrates that California freeways have significantly lower rates than all other California highways for total accidents, fatal plus injury accidents, and number of fatalities.

Table B-5. 1963-64 Accident Rates for California Highways

	Total Accidents per Million Vehicle Miles	Fatal + Injury Acc. Per M.V.M.	Fatalities per 100 M.V.M.
Freeways	1.46	0.64	2.71
Other	3.68	1.42	7.55

However, by examining relative Severity Indices, Table B-6, which represent the average per involvement severity, it becomes apparent that freeways have a higher per involvement severity than all other highways.

**Table B-6. Relative Severity Indices of California Freeways
vs
All Other California Highways for 1963-64**

	Total Accidents		Total Accidents		Total Accidents		Total Accidents	
	No.	SI	No.	SI	No.	SI	No.	SI
Freeways	847	25	23,192	6	31,700	1	55,739	3.45
Other	2,696	25	59,820	6	98,999	1	161,515	3.25

It might appear that the per involvement severity of freeways should be lower than all other highways because of the minimization of three severe accident types: head-on, right-angle, and pedestrian accidents. Over-all safety of freeways results from the elimination of conflicting traffic. However, this elimination of conflict necessitates grade separations and introduces a new contributor to the severity picture; namely, fixed objects. Grade separations require structures, complex signing, and interchange illumination which account for a large number of the fixed objects on freeways. Any such fixed object contributes to the increase in the freeway accident Severity Index.

The California study emphasizes that it is possible to increase the overall safety of freeways by reducing (1) the number of fixed objects, (2) exposure to fixed objects, and (3) the consequences of striking fixed objects. The report suggests the following methods for accomplishing these three objectives:

- A. Methods to reduce the number of fixed objects.
 1. Place overhead signs on overcrossing structures where appropriate.
 2. Enclose overcrossing abutment in cut slope or fill cone.
 3. Avoid construction of separate bridges with interior bridge rails whenever possible.
 4. Place electroliers on overcrossing structures where possible.
 5. Place signs back to back in median.
 6. Investigate use of advance information signs for possible reduction in number.
 7. Combine signs and lightpoles.
 8. Avoid indiscriminate use of guardrail.
- B. Methods to reduce exposure to fixed objects.
 1. Place large overhead directional signs adjacent to the right shoulder in lieu of the more vulnerable gore position.
 2. Place signs and lightpoles on top of or immediately beyond bridge rails where convenient.
 3. Place signs and lightpoles behind bridge rail and abutment guardrail flares where convenient.
 4. Place signs and lightpoles adjacent to right shoulder instead of in the median (reduced exposure to total traffic).
- C. Methods to increase safety of fixed object accidents.
 1. Place guardrail in front of those objects which have a higher Collision Index than the guardrail.
 2. Employ wood posts for smaller directional signs.
 3. Design less rigid and less penetrable bridge rails.
 4. Design a more contiguous bridge-rail-guardrail system.
 5. Place fixed objects at greatest possible distance from the edge of the traveled way."

The California study considers what affect adjacent protective guardrail has in reducing the Collision Index of various fixed objects. The Collision Index is defined as the

product of the Severity Index and the ratio of the number of total accidents to the number of vehicles exposed during the accident study period. The details of investigation are contained in the report, and the recommendations of the California engineers follow.

- “1. Embankment guardrail need should be determined on the basis of Figure B-4, and modified by considerations of cost, alignment, grade, traffic volume, climate, and accident experience.
2. Guardrail should be placed adjacent to:
 - a. Bridge-rail approach ends.
 - b. Bridge piers and abutments.
 - c. Steel signposts.The guardrail increases the relative safety (decreases the product of accident frequency and severity) at these fixed objects.
3. Guardrail should *not* be placed adjacent to lightpoles. The guardrail accidents generally are more severe than lightpole accidents.
4. Steel signposts in the off-ramp gore area should be avoided. Similar signposts placed adjacent to the right shoulder are safer.
5. Dimensional lumber signposts should be used in lieu of steel signposts whenever possible.
6. A review of present material and dimensional requirements of signposts should be made with objective of providing posts of the minimum strength consistent with structural requirements to reduce the severity of accidents involving signposts.
7. A subsequent investigation should be undertaken with the purpose of evaluating the effects of highway geometry and traffic on the frequency of run-off-road accidents. With this information, a more objective basis for embankment guardrail placement can be developed.”

The California study of guardrail installation on embankments and adjacent to fixed objects provides a rational guide, based on accident records for a two year period, to aid the highway engineer in deciding whether or not to use guardrails.

Judgment is required in making the decision to install guardrail, and experience is a necessary requirement. At the present time no systems engineering approach is available to guide the highway engineer in his decision making processes. Possibly such a method will ultimately become available, but even then such techniques will require “stochastic inputs,” which is one way to describe information based on experience and judgment.

Deleys and McHenry discuss the problem which confronts the designer:

“The crux of the problem faced by the highway engineer when posed the question, ‘When is the installation of guardrail warranted?’ lies in the answer that basically defines its purpose. The answer, at least in part, may be correctly stated as: ‘Whenever the consequences of vehicles leaving the roadway are hazardous and would be more severe or damaging than those that would prevail if guardrail were to be installed.’ The key words are ‘whenever’ and ‘hazardous’ and the foregoing statement implies that as guardrail performance is improved, the need for guardrails increases; i.e., hazards that formerly did not warrant the installation of a guardrail become relatively more hazardous as better guardrails are developed.

At the present time there is a need for a more factual or scientific basis for warrants. Such a basis for warrants must include consideration of the relative hazards of specific roadside features and the various configurations of barriers under the prevailing conditions of vehicle operation (i.e., speed, density, probable frequency of accidents, etc.) and in view of the mixture of vehicle weights and sizes. It would seem that this problem could be approached from the view points of (1) accident statistics and the results of staged accidents (i.e., statistical and experimental measures of hazards), and (2) analysis of the dynamics of vehicles that (a) encounter roadside objects, and (b) impact guardrails.”

This quotation is taken from the National Cooperative Research Program Report 36, which also contains a discussion of a limited number of investigations of actual accidents. Some excerpts follow.

Many different types or classes of highway are in use today, ranging from the low- to medium-speed rural and urban roads to multiple-lane divided or undivided high-speed highways and expressways found in the Interstate System and in metropolitan areas. Clearly, the guardrail performance requirements as related to the different traffic and geometric characteristics of the various types of roads are variable and establishment of guardrail design criteria requires a definition of the prevailing conditions of vehicle off-road movement for the various types of road.

A number of investigations of actual accidents have been conducted for the purpose of gathering statistical data on accident causation, frequency of occurrence, injury and fatality rates, median encroachments, etc. These studies are reported in NCHRP Report 36.

For a vehicle initially travelling parallel to a guardrail there is a maximum angle at which the vehicle can impact it (i.e., the angle between the direction of motion of the center of gravity of the vehicle, as opposed to its direction of heading, and the longitudinal centerline of the undeflected barrier) that depends on the vehicle speed, the friction coefficient between the tires and the road surface, and the lateral distance from the barrier.

This relationship, developed by the New York State Department of Public Works and the Cornell Aeronautical Laboratory, is

$$\psi = \text{Cos}^{-1} \left[\frac{1}{V^2} g y (\mu + \phi) \right] \quad (1)$$

in which

- ψ = impact angle, in degrees;
- y = initial lateral distance from the barrier, in feet;
- V = vehicle speed in feet per second;
- g = acceleration of gravity, in feet per second²;
- μ = friction coefficient between tires and road; and
- ϕ = road camber or superelevation, in radius.

Equation 1 is based on the assumption that the vehicle is initially travelling parallel to the barrier on a straight road and subsequently turns into the barrier on a constant minimum radius path (at the speed being considered) that is determined by equilibrium of lateral forces on the vehicle (centrifugal and tire friction forces) for incipient skidding.

Data extracted from a limited survey of the Automotive Crash Injury Research (ACIR) files of the Cornell Aeronautical Laboratory indicates that a surprisingly large number of impacts (50 percent of the total) occurred on the end of the guardrail.

The next most prevalent failure modes were vehicles penetrating or vaulting over the guardrail or being reflected back onto the highway at high angles. However, a comparison of the number of times the barriers performed successfully versus the number of failures is, as in all data found in the literature in this regard, not a valid indication of the present state of the art of guardrail performance because the number of times vehicles strike guardrails and are successfully returned to the highway or otherwise go unreported is unknown. It should be noted that the ACIR data include only injury-producing accidents.

Warrants for median barriers

Warrants for the installation of guardrails on embankments and adjacent to fixed objects have been presented. Installation of guardrails or other types of barrier in the median must also be considered. In rural areas, opposing traffic may be separated by wide median strips of 50 feet or more. However, in urban areas, right of way widths are understandably restricted and the highway engineer must use narrow medians. Prior to the advent of high speed, high density traffic such as occurs on freeways, opposing streams of traffic were separated by narrow medians consisting of a raised

curbing, or in some cases by a slight depression to permit drainage of the adjacent lanes of traffic. Such narrow medians are easily crossed and have resulted in spectacular head-on collisions, with an alarming number of fatalities. The primary warrant for median barriers is apparent, namely: installation of a barrier to prevent head-on collisions. Operational experience in California indicates that although median barriers have been effective in reducing the frequency of cross-median accidents, the rate of accidents, involving the median has increased at locations where barriers have been installed. The New Jersey State Highway Department has published a brochure titled "Center Barriers Save Lives" in which they state:

"A positive median divider makes the serious cross-median type accident a very rare or freakish occurrence. However, by its very nature, the barrier restricts the distance a driver can move to the left. Some have believed that a center barrier might thus cause an abnormal increase in rear-end accidents, but in areas where they have been installed local law enforcement officers say the barrier has not."

The New Jersey highway engineers suggest that the use of a five foot wide paved shoulder on each side of a barrier is narrow enough to prevent misuse as a passing lane, but does provide room for emergencies.

It is apparent that construction of a median barrier results in a potential hazard adjacent to the travelled way. Highway engineers and researchers have addressed themselves to this problem and have attempted to develop median barriers which will reduce the affect of a collision with such barriers.

In the following section, various types of guardrails for use on embankments, adjacent to fixed objects, and in median locations, will be discussed.

Safer Roadside Structures
Part C
Guardrails, Median Barriers
and Bridge Rails

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Guardrails, median barriers and bridge rails

When the highway engineer has established the warrants for the installation of a guardrail, median barrier, or bridge rail, it is then necessary to choose the type of barrier which will provide most satisfactory results from the viewpoint of safety. Many types and designs of guardrails and barriers are in use today, commonly divided into three broad classifications: (1) rigid barriers, (2) semi-rigid barriers, (3) flexible barriers, depending upon the relative stiffness of the barrier's longitudinal elements and the amount of lateral deflection in a collision by a vehicle. These three classifications imply slight to no deflection in the case of rigid barriers, small to moderate deflections in the case of semi-rigid barriers, and relatively large deflections in the case of flexible barriers.

Some of the physical characteristics and dimensions of more commonly used barriers in each of these classifications are presented in the following discussion, taken in part from NCHRP Report 36 by Deleys and McHenry.

Rigid barriers

Rigid barriers are generally used only where the space available for deflection is limited, as on very narrow medians and bridge structures. Because they must essentially be made unyielding, these barriers are often constructed of reinforced concrete.

Perhaps the best known rigid barrier design in the United States is the so-called New Jersey concrete median barrier shown in Figure C-1 (a). This barrier contains approximately 2.8 cubic feet of concrete per linear foot and is constructed with white concrete to accentuate visibility. Approximately 200 miles of this type of barrier were in place on New Jersey highways in March, 1966. An adoption of the basic concept and dimensions fabricated from 5/16-inch steel plate has been employed on the Hackensack River Bridge Lift Span.

Another rigid median barrier, called "Isle-Guard" (Figure C-1 (b)), has been in use for a number of years in at least one installation in New York City. The effectiveness of this patented design has been demonstrated by the inventor on several occasions by deliberate collisions, and also by the reduction of accidents since the barrier was installed. One significant difference between this barrier and the New Jersey median barrier is the thin steel sheath on the exterior surface, which, by virtue of the smaller coefficient of friction, is believed to facilitate a smooth redirecting action of the vehicle as the wheels momentarily ride up the sloped side. The shapes of both barriers are designed to minimize contact and damage to vehicles in shallow-angle impacts.

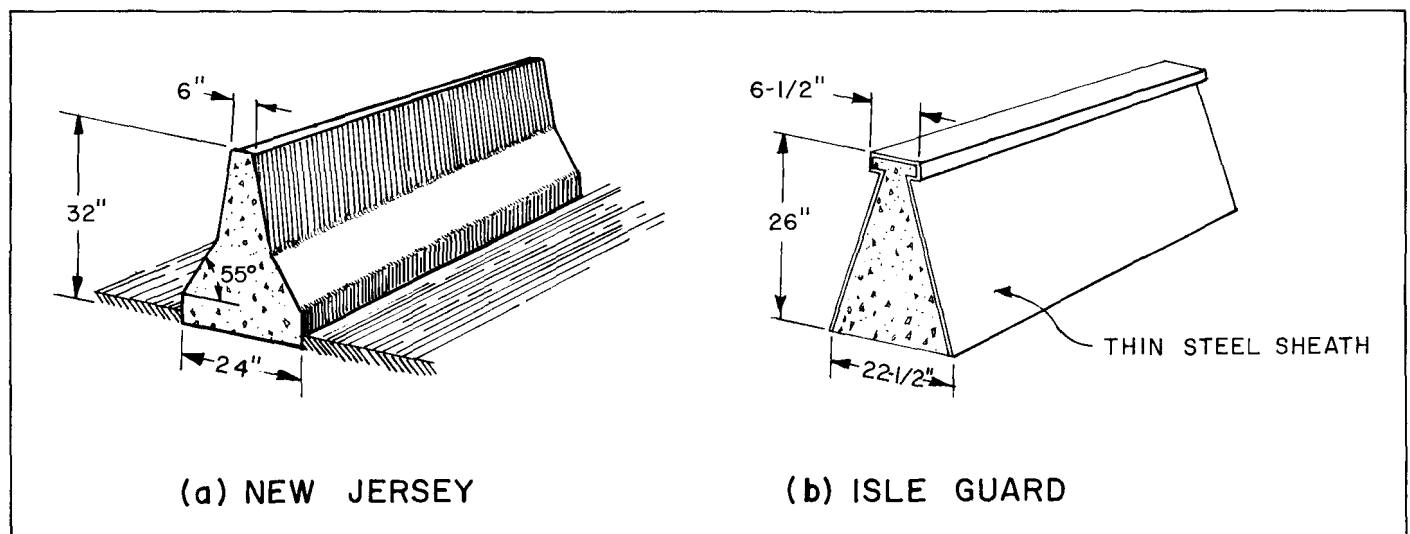


Figure C-1. Rigid median barriers

Deleys and McHenry conclude in NCHRP Report 36:

“On the basis of presently available information, the advantages of rigid barriers, particularly of the solid wall type, would seem to be: (1) they can be designed to withstand the most severe impact without penetration or pocketing; (2) there are no posts upon which a vehicle can become snagged; (3) they can be designed so as to cause little or no vehicle damage for impacts of low severity; (4) reflection angles of impacting vehicles are low; and (5) they are not easily damaged, hence are easy to maintain. Among the disadvantages are: (1) being unyielding, they absorb little kinetic energy of the vehicle and tend to aggravate the acceleration environment of the vehicle occupants; (2) they perhaps are not as aesthetically attractive as some of the other types of barriers; (3) in some climates, they may intensify the snow removal problem; and (4) although no substantiating information has been found, they would appear to have a higher installation cost.”

Semi-rigid barriers

Corrugated beam. The most prevalent type of semi-rigid barrier presently used is the longitudinally corrugated metal rail mounted on posts. Typical roadside guardrail and median barrier configurations are shown in Figure C-2 (a). The rails are frequently attached directly to the posts, but the barrier performance is improved when they are blocked-out from the posts because the possibility of snagging the vehicle is reduced. In most states, the mounting height of the top of the rail is 27 inches above the ground and the standard post spacing is 12.5 feet. For median barrier installations, a mounting height of 30 inches and a reduced post spacing of 6 feet 3 inches, with the addition of an auxiliary lower rubbing rail to prevent snagging, has been found to be an effective design.

Although some of the lateral force to restrain and redirect impacting vehicles is produced by beam bending, the major portion is obtained through the tension forces developed because of local flattening of the rail at or near the point of impact. These forces stretch the rail as it is deflected laterally and are distributed among several posts. Specifications for rail strength are based on tensile strength and allowable deflections when a simply supported beam is subjected to a concentrated load at mid-span.

The most common types of support post are 6" x 4" x 8.5#, 6" x 6" x 15# I-shaped steel sections, 6" x 8", 8" x 8", or 8" diameter timber or concrete posts. Steel posts may be bolted to concrete foundations or embedded in soil. Posts of all types are usually embedded three and one-half to four feet in soil. Since a variety of post types and base connections are specified, it is apparent that the dynamic load-deflection characteristics of embedded posts vary widely. Little information is available concerning the behavior of posts under dynamic conditions. Recently the New York Department of Public Works, Bureau of Physical Research, conducted tests on the dynamic behavior of selected steel and wood posts. Force-deflection information were obtained by measuring the load on the bumper of a truck as it was driven into a line of posts embedded in sand and glacial till. The results are reported in NYDPW, Physical Research Report 67-1.

Cable. Cable guardrail, one configuration of which is shown in Figure C-2 (b), is classified as a semi-rigid barrier because the heavy posts limit the deflections to moderate amounts. Again, many variations of this type of barrier are used. The cables are usually 3/4-inch diameter wire rope with a minimum tensile strength of 25,000 pounds. The number of cables varies between two and four frequently mounted on offset spring brackets that hold the cables at a separation of 4 to 6 inches. In some installations, however, the cables are attached directly to the posts. The posts are generally of the types previously described, but post spacing varies considerably (between 10 and 16 feet) in states using this type of barrier. Accident reports and results of test indicate that despite the economical initial cost of such barriers the cable and post barrier causes severe pocketing of a colliding vehicle with a resulting abrupt stop.

Box-beam. Relatively recently New York State developed and adopted as standard a semi-rigid box-beam barrier of the type shown in Figure C-2 (c). This design will be discussed more fully later. A spade plate, the optimum dimensions of which were determined in the post test program previously mentioned, is welded to the bottom of

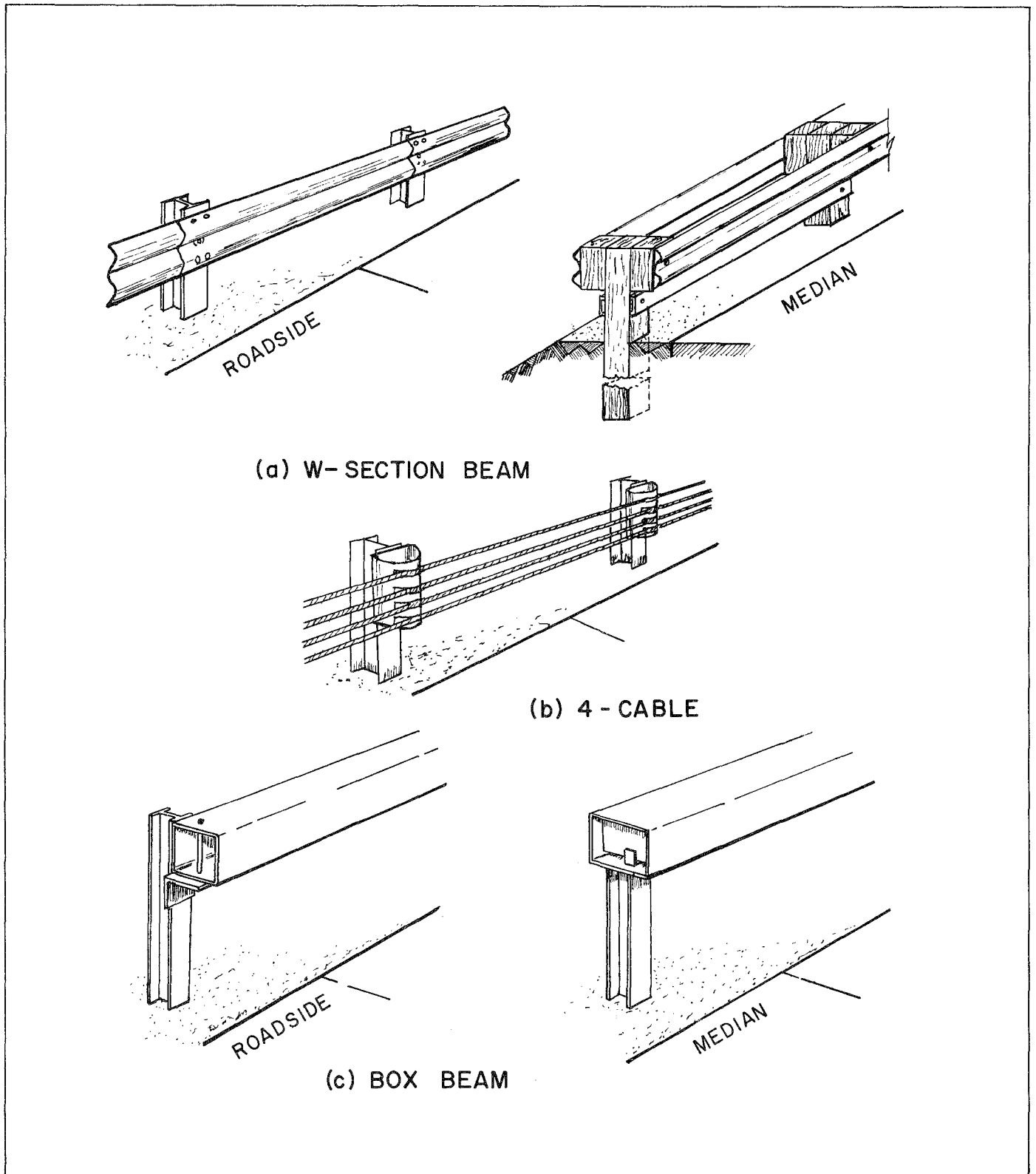


Figure C-2. Semi-rigid barriers

each post in order to obtain proper soil reactions over a range of variable soil conditions. The posts are sunk into the ground to a minimum depth of 36 inches and the top of the box-beam rail is nominally 27 inches above the ground.

The operating principle of this barrier design is different from those previously described in that the forces of impact are resisted by the strong beam rail and are distributed over a large number of relatively weak posts. Unlike other barriers, which have large variations in load-deflection characteristics, depending upon whether the load is applied between posts or at a post location, the more uniform deflection characteristics provided by the box-beam barrier reduce the possibility of the vehicle becoming pocketed between or snagging on posts. In addition, design of the posts to yield above the ground line, results in barrier performance that is much less likely to be affected by variations in soil conditions.

Flexible barriers

Flexible barriers, by allowing large deflections in comparison to the other types previously described, are advantageous because they redirect or stop colliding vehicles more gradually and thereby subject the occupants to lower, more tolerable deceleration levels. One such barrier design, investigated quite thoroughly by the California Division of Highways, is the cable-chain link fence median barrier shown in Figure C-3. The barrier consists of two 3/4-inch diameter wire rope cables fastened by U-bolts

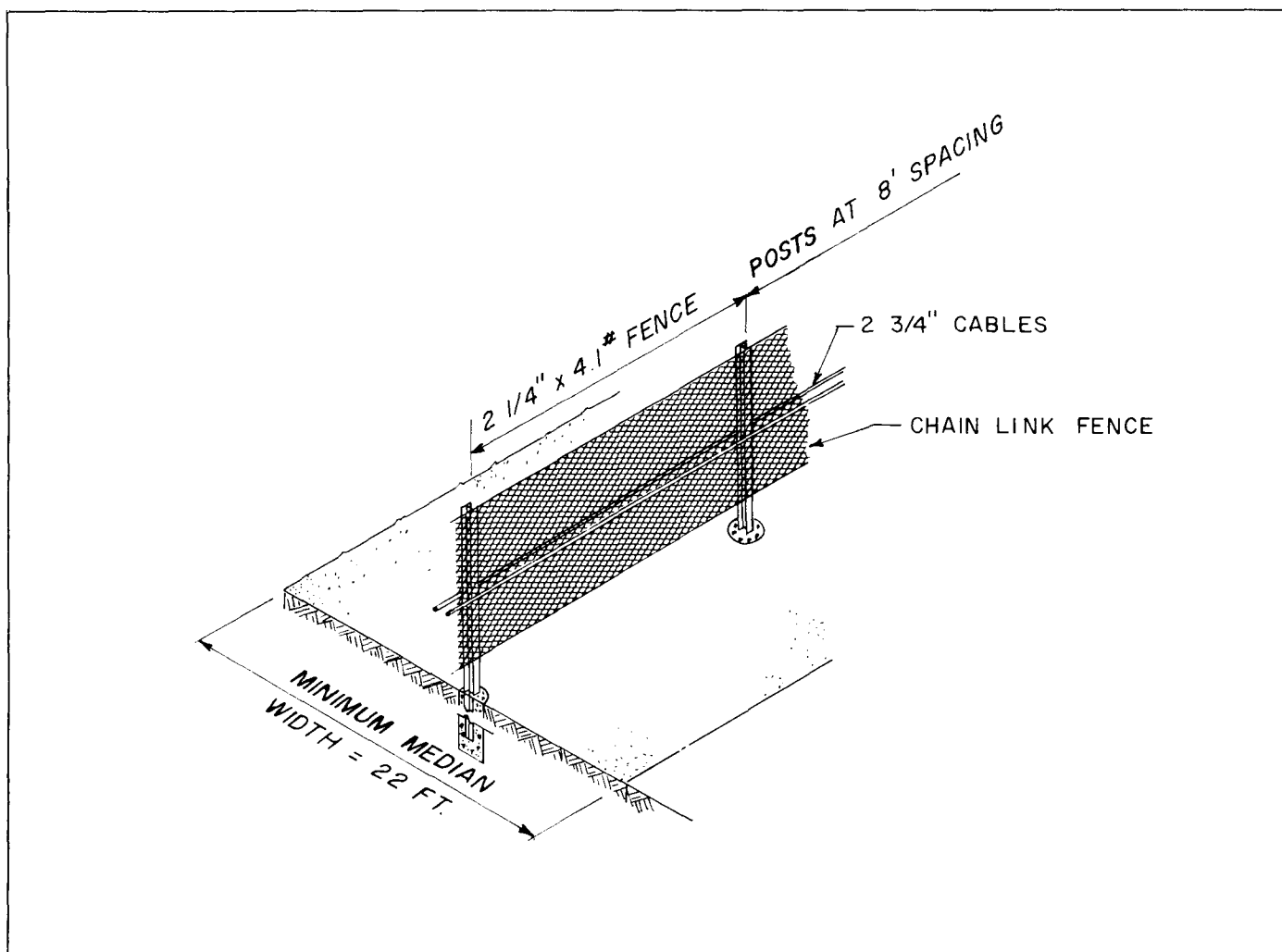


Figure C-3. Flexible median barrier

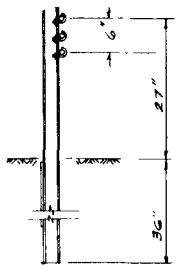
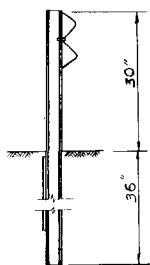
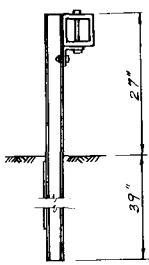
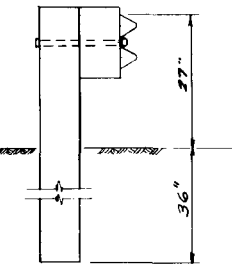
to fence posts, at a height of 30 inches above the ground. In addition, a 48-inch chain-link fence is attached to the posts by steel wire tires. The posts, spaced on 8 foot centers, are embedded in 10-inch diameter concrete post footings extending about 30 inches into the ground. When a vehicle strikes this barrier, the wire cables are stripped off the posts, which bend over as the barrier deflects, and the wire mesh is gathered up in a bundle ahead of the vehicle as it comes to a stop. This type of barrier has been recommended for use on California medians having a minimum width of 22 feet to provide safe allowance for cable deflection during impact and to permit maintenance to be performed completely off the traffic lanes. Similar cable barriers without the chain-link fence have been designed by the British Road Research Laboratory and New York State.

The New York design consists of three 3/4-inch cables spaced 3 inches apart, with the top cable at a height of 27 inches above the ground, and attached by small hook bolts to the same type of post used for the box-beam barrier. Post spacing for this barrier is normally 16 feet. Another type of flexible barrier design, recently adopted by New York State, employs a standard W-section steel beam instead of the three cables. This latter design is believed to result in less vehicle and barrier damage for the less severe, low-speed, brushing-type impacts. Post spacing may vary from 6 to 26 feet, depending on the space available for deflection.

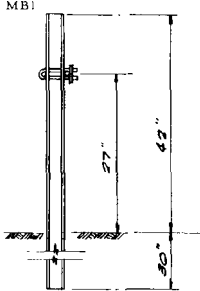
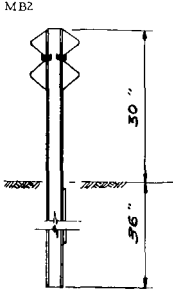
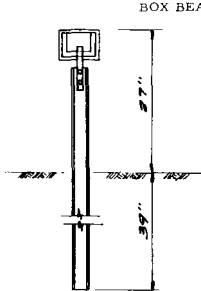
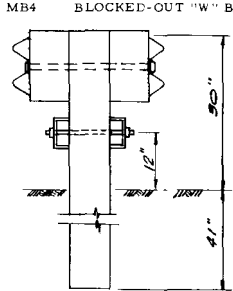
Flexible barriers resist and redirect impacting vehicles by tension forces developed in the cables as they are deflected laterally. Therefore, these barriers must be terminated securely by end anchorages in the ground. In long installations, intermediate anchorages also may be necessary. The barriers are designed to permit large deflections under impact so vehicles are not turned abruptly with high decelerations, as is the case with more rigid barriers. For this reason, relatively weak posts, from which the cables are readily stripped and which are easily knocked over to prevent snagging, are employed. Because of the large deflections and the long distances that impacting vehicles remain in contact with this type of barrier, relatively more damage to the barrier results, which increases the cost of maintenance.

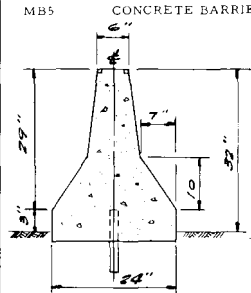
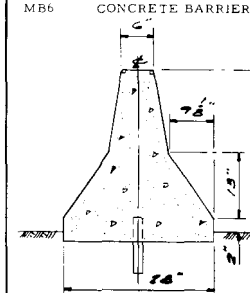
Highway engineers and researchers have suggested designs for safer barriers and some of these designs have been subjected to full-scale crash tests. A discussion of recent developments in barrier design follows.

SUMMARY OF GUARDRAIL CHARACTERISTICS

STANDARD TYPE	G1 CABLE	G2 "W" BEAM	G3 BOX BEAM	G4 BLOCKED-OUT "W" BEAM
				
DEFLECTION	12 ft	8 ft	4 ft 2 ft	2 ft
POST SPACING	16'-0"	12'-6" Nominal	6'-0" 4'-0"	6'-3"
POST	3" 1 5, 7	3" 1 5, 7	3" 1 5, 7	8" x 8" Douglas Fir
BEAM	Three 3/4" Dia Steel Cables	Steel "W" Section	6" x 6" x 0.160" Steel Tube	Steel "W" Section
OFFSET BRACKETS	—	—	5" x 3-1/2" x 1/4" Steel Angle 4-1/2" Lg	8" x 8" x 1'-2" Douglas Fir Block
MOUNTINGS	5/16" Dia Steel Hook Bolts	5/16" Dia Steel Bolt	3/8" Dia Steel Bolt (Beam to Angle)	5/8" Carriage Bolts
FOOTINGS	1/8" Steel Plate Welded to Post	1/8" Steel Plate Welded to Post	1/8" Steel Plate Welded to Post	None
DEVELOPED BY	New York	New York	New York	California
REFERENCE	Appendix A	Appendix A	Appendix A	Appendix A
REMARKS				For cases where a guardrail is warranted in a median, raise beam height to 30 inches and install rubbing rail as shown in MB4.

SUMMARY OF MEDIAN BARRIER CHARACTERISTICS

STANDARD TYPE	MB1 	MB2 	MB3 	MB4 
DEFLECTION	11 ft	7 ft	2 ft	2 ft
POST SPACING	8'-0"	12'-6" Nominal	6'-0"	6'-3"
POST	2-1/4" H 4.1	3" I 5.7	3" I 5.7	8" x 8" Douglas Fir
BEAM	Two 3/4" Dia Steel Cables	Two Steel "W" Sections	8" x 6" x 1/4" Steel Tube	Two Steel "W" Sections Two 6" x 8.2 Steel Sections
OFFSET BRACKETS	—	—	—	Two 8" x 8" x 1'-2" Douglas Fir Blocks
MOUNTINGS	1/2" Dia Steel "U" Bolts	5/16" Dia Bolts	Steel Paddies (see details)	5/8" Carriage Bolts
FOOTINGS	Details vary with application	1/8" Steel Plate Welded to Post	1/8" Steel Plate Welded to Post	None
DEVELOPED BY	California	New York	New York	California
REFERENCE	Appendix A	Appendix A	Appendix A	Appendix A
REMARKS	Use on flat medians or on saw-tooth sections with slope flatter than 3:1 or with step less than 6 inches in height.	For saw-tooth medians use two guardrail installations	Use on flat medians or on saw-tooth sections with slope flatter than 3:1 or with step less than 6 inches in height.	Stagger beam heights when the saw-tooth step is over 6 inches high and/or the median slope is 3:1 or steeper.

STANDARD TYPE	MB5 	MB6 
DEFLECTION	0	0
DOWELS (into existing pavement)	1" Dia x 8" Long Steel Rod	1" Dia x 8" Long Steel Rod
DOWEL SPACING	4'-0"	4'-0"
CONCRETE	Class B*	Class B*
DEVELOPED BY	New Jersey	General Motors
REMARKS	Use on narrow medians Use of barrier profile is recommended at retaining walls, rock cuts, etc. (see Figure B-7)	Use on narrow medians Use of barrier profile is recommended at retaining walls, rock cuts, etc. (see Figure B-7)

*AASHTO Class B

Recent developments in barrier design

The mechanics of a collision with a barrier are so complicated that it has been necessary to determine actual barrier performance from full-scale crash tests. Several agencies have been involved in such dynamic testing of selected barrier designs, and through observation of the behavior of a design, modifications have been made and a modified barrier has been crash tested. Earlier performance has been judged primarily on how well the barriers satisfy three criteria: (1) The barrier must prevent a colliding vehicle from entering another travelled way or into an area of hazard. (2) The vehicle must be redirected parallel to the barrier in such a way that it does not become a hazard to other vehicles. (3) Vehicle barrier interaction in a collision must be such as to produce minimum injury to occupants of the colliding vehicle. Engineers of the New York State Department of Public Works, Bureau of Physical Research report in Highway Research Record No. 174, 1967, on a six year research program which resulted in the complete revision of the standard barrier designs for roadsides, medians, and bridges specified by the New York State Department of Public Works. The study included a comprehensive theoretical analysis of the forces generated between vehicle and barrier during a collision. In addition, 48 full-scale crash tests between standard sized passenger cars and various barrier designs were conducted. Speeds up to 60 miles per hour and impact angles up to 35 degrees were selected as representing the most severe conditions expected on a highway.

This research led to the development of a new design termed the box-beam barrier. A commercially available hollow structural rail section of considerable beam strength is supported by relatively weak posts; such a barrier deflects and absorbs collision forces while decelerating and redirecting the vehicle. By using box-beams of different strengths and by varying the spacing of posts, barrier deflection can be controlled, thus making this type of barrier suitable for a guide rail, a median barrier, or a bridge railing. It appears that the selection of post spacing can aid the engineer in providing a smooth transition from the relatively flexible guardrail configuration to the relatively rigid bridge railing. The New York engineers established the following design criteria: (1) Colliding vehicle must not pass through the barrier. (2) Gradual deceleration of the colliding vehicle must be provided to permit occupants to survive a collision. (3) Colliding vehicle must be redirected as nearly parallel as possible to normal vehicular movements to minimize the possibility of collisions with other vehicles. (4) Damage to vehicle colliding with a barrier should be minimized. (5) Cost of barrier construction and maintenance must be reasonable, safety must be taken into consideration in the determination of the economical selection of a barrier.

Current practice in New York State includes standard designs for (1) cable, (2) W-section guardrail, (3) box-beam median barrier, shoulder guard rail, and bridge rail. Field investigations of collisions with railing installations are being conducted in New York State, but results of these investigations have not been published at this time.

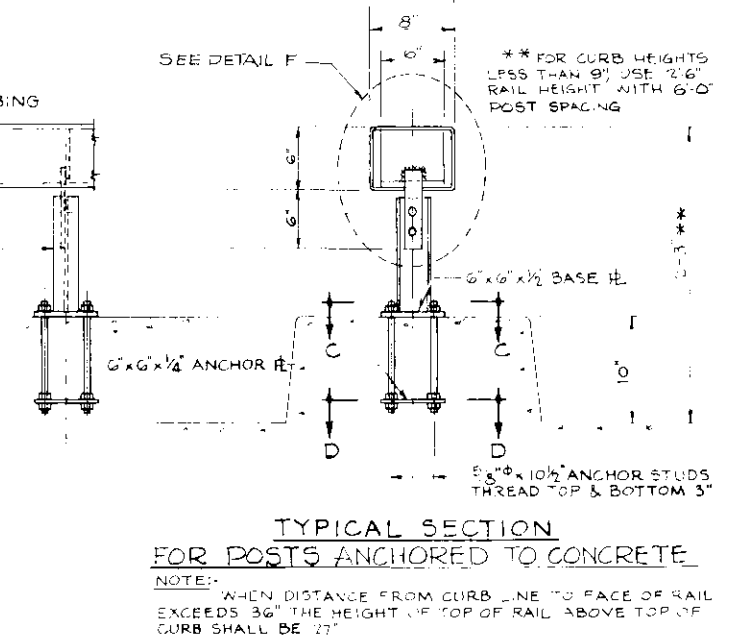
A discussion of the Box-Beam Median Barrier follows. Method of installing the Strong Beam/Weak Post Median Barrier is illustrated in Figure C-4. Working drawings of the Box-Beam Median Barrier and the Box-Beam Guide Rail are presented in Figures C-5 and C-6.

Figure C-4. Installing the first
box beam median barrier
on the
Cross-Bronx Expressway,
New York City.



Figure C-5

Box Beam Median Barrier

[illegible]

Hollow structural tubing shall conform to the requirements of A.S.T.M. designation A500 or A501, except bare COR-TEN structural tubing shall conform to the requirement of A.S.T.M. designation A618.

All other material except splice bolts and bare COR-TEN material shall conform to the requirements of A.S.T.M. designation A36. Bare COR-TEN material shall conform to the requirements of A.S.T.M. designation A588.

Splice bolts and nuts shall conform to the requirements of A.S.T.M. designation A307. Bare corrosion resistant splice bolts and nuts shall be of an approved corrosion resistant material and conform to or exceed the mechanical properties of A.S.T.M. designation A307.

All material shall be galvanized except for systems which are specified as bare COR-TEN.

All fabrication for systems to be galvanized shall be complete and ready for assembly before galvanizing. No punching, drilling, cutting or welding shall be permitted after galvanizing.

The Box Beam Median Barrier concept was developed during a study conducted by the New York State Department of Public Works, Bureau of Physical Research.



United States Steel Corporation
Highway Construction Marketing

Box Beam Median Barrier

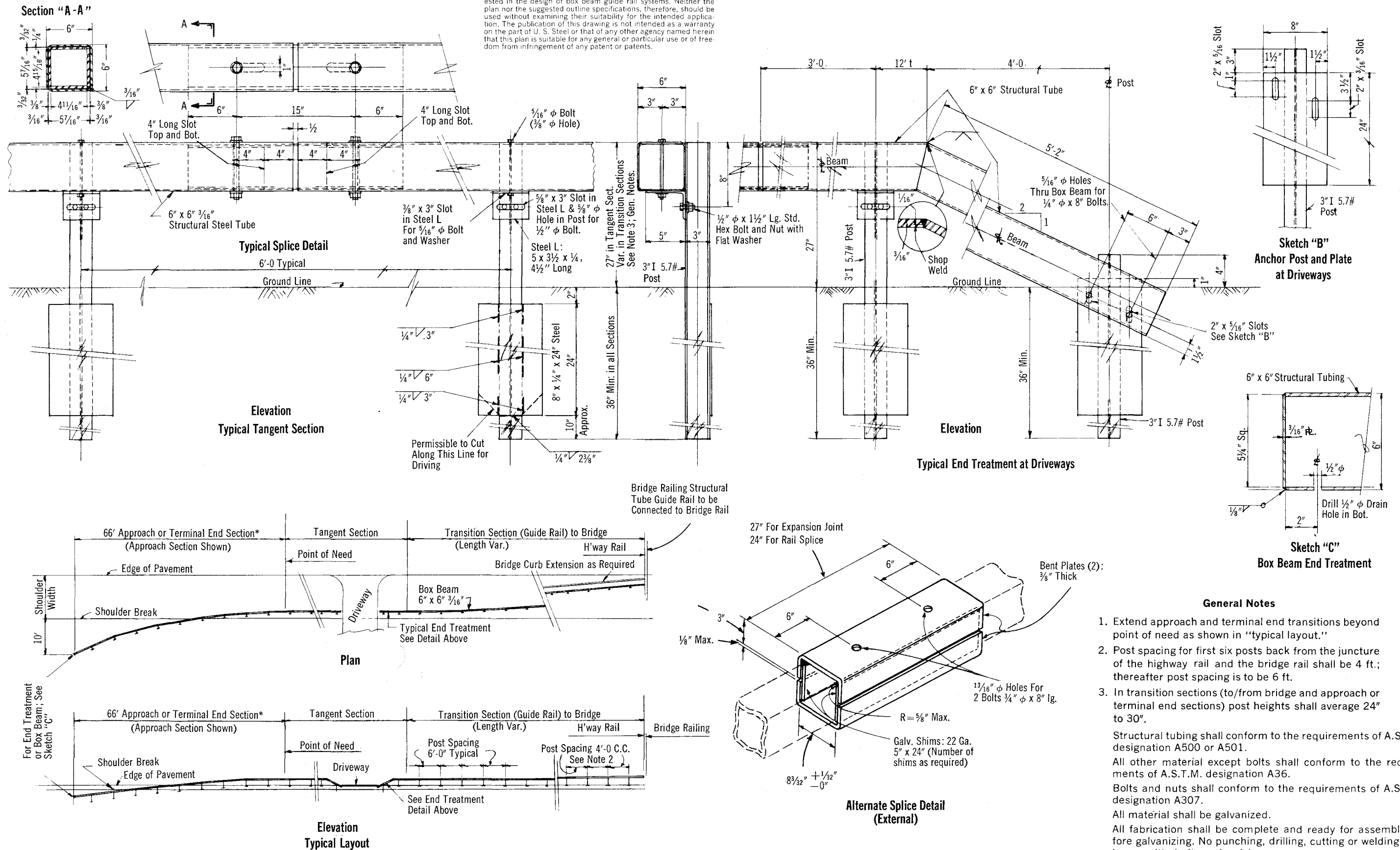
Figure C-5

ADUSS 90-1487-04
Printed in U.S.A.
Revised May, 1969

Figure C-6

Box Beam Guide Rail

This drawing has been prepared in accordance with recognized engineering principles for the general information of those interested in the design of box beam guide rail systems. Neither the plan nor the suggested outline specifications, therefore, should be used without examining their suitability for the intended application. The publication of this drawing is not intended as a warranty on the part of U. S. Steel or that of any other agency named herein that this plan is suitable for any general or particular use or of freedom from infringement of any patent or patents.



*On Divided Highway With Mall or Median Barrier — Use Only An Approach End
 On Highways With One-Way Traffic — Use Only An Approach End
 On Highways Without Mall or Median Barrier — Use On Both Approach and Terminal Ends

Box beam median barrier

During the Summer of 1963, New York State's Department of Public Works, in cooperation with the Bureau of Public Roads, sponsored a series of dynamic impact tests on different types of highway barrier systems. Electronically controlled vehicles were crashed into the barriers at various angles at speeds up to 60 miles per hour. One of the significant results of this combined analysis and testing program which was conducted for New York State Department of Public Works' Bureau of Physical Research by Cornell Aeronautical Laboratory, Inc., was the development of a *new* highway barrier concept; the idea that a hollow rectangular metal tube could be mounted as a rail on relatively weak posts. This new approach to roadway barrier design is known as the "strong beam/weak post" concept.

In the past, one of the problems associated with conventional barrier systems, standard beam type guard rail attached directly to strong posts, has been severe vehicle deceleration resulting from contact with the posts under heavy impact. In recent years, performance of barrier systems of this type has been greatly improved by 1) increasing the rail bending strength through reduction of post spacing and 2) the placement of 6" or 8" spacer blocks between the rail and posts. The "blocked-out" beam barrier system has rightfully gained wide acceptance among highway engineers because of the resulting improvement in performance over former systems.

"Strong beam / weak post" concept for highway barrier design

The "strong beam/weak post" approach to eliminating the possibility of impacting vehicles "pocketing" on heavy posts, is handled in a different manner. This system incorporates posts of sufficient weakness that vehicle contact with them will not result in undesirable severe decelerations. Dynamic tests of posts have shown that sufficient lateral support for a horizontal rail element can be obtained from closely spaced but relatively weak posts.

Box beam median barrier

A box beam barrier system of the "strong beam/weak post" type has been developed. It is especially suitable for narrow bridge and roadway medians. A discussion of the important features of the design follows:

Since it is desirable that rail deflection not exceed three feet in a narrow median, a rail having bending strength was necessary. The relatively stiff rail would have to distribute the lateral impact force to about ten posts. To allow the beam to function without buckling, the posts needed to be more closely spaced than was common practice. In addition to providing about 4000 pounds of lateral support, the posts needed to be as weak as possible longitudinally so that a colliding vehicle could bend them out of the way without difficulty. Two additional items had to be considered in the rail design. For the expected deflections of two feet or more, stresses in the rail would exceed the yield point of the metal. When this happened the rail had to continue to bend plastically with a nearly constant resisting moment (yield hinge). The rail material, then, had to be able to withstand considerable elongation. In addition, an impacting vehicle would contact and knock down posts as it slid along the rail. Elimination of these posts would alter the lateral support for the beam and this had to be considered in the solution. Hollow structural tubing conforming to the requirements of ASTM Designation A 501-64 "Hot Formed Welded and Seamless Carbon Steel Structural Tubing" was a logical choice for the rail.

While not a factor in the structural analysis of the barrier, a very important consideration in the overall design was the connection between posts and rail. The post could not be fastened to the rail with heavy bolts for two reasons. First, such a connection would pull the rail down as the posts bent laterally; possibly far enough to allow a car to roll or go over the rail. Secondly, the posts

would be much too strong to be easily knocked down when the car slid along the rail. Consequently, for the box beam median barrier, a slit is cut in the bottom of the rail to accept a plate fastened to the top of the post. The plate is relatively strong laterally but easily bent when the post is struck longitudinally.

The expected big advantage of this “strong beam/weak post” box beam median barrier is the ability to provide a nearly uniform restraining force on a contacting vehicle over the full range of beam deflection. This desirable feature will absorb kinetic energy of the car within the limits of allowable deflection while imparting to it the minimum possible deceleration. The reduction in deceleration, by allowing the maximum permissible deflection, greatly reduces the probability of fatal injury to vehicle occupants.

Development of the blocked-out beam guardrail

The following excerpt from a report titled “Objective Criteria for Guardrail Installation” discusses the development of the California blocked-out beam guardrail. The present design consists of timber posts spaced at 6'-3" center to center; overall beam height is 27 inches.

The primary reason for installing guardrail on embankments and adjacent to fixed objects is to reduce the combined effect of accident severity and accident frequency of ran-off-road accidents. Guardrail will reduce accident severity only for those conditions where the over-all severity of striking the guardrail is less than the over-all severity of going down the embankment or striking the fixed object. Guardrail will reduce the accident frequency only if it provides increased delineation at high frequency ran-off-road accident locations. Generally, however, it would be expected that installing guardrail adjacent to fixed objects would increase the accident frequency because the guardrail would be a larger obstacle.

Three types of guardrail are currently in place on California Highways: (1) W-section corrugated steel beam mounted on timber posts, Figure C-7, (2) spring mounted curved metal plate mounted on timber posts, and (3) W-section corrugated steel beam similar to that shown in Figure C-7, but having 27-inch over-all beam height and 6'-3" center to center post spacing. The increase in over-all height and decrease in post spacing were made following full-scale dynamic testing of the design shown in Figure C-7. Crash tests demonstrated that, at 58 mph and a 25 degree impact angle, a passenger vehicle could vault the 24-inch high rail.

Several highway departments are currently using a blocked-out beam median barrier. The beam consists of W-sections fabricated from steel mounted on posts spaced at 12'-6" centers, each post consists of an 8" x 8" timber post, with a block of the same material on each side to which the W-beams are bolted, as shown in Figure C-2 (a); this configuration is employed in medians which are less than 10 feet in width. This type of barrier is a semi-rigid barrier, and the deflections under collision force are small.

The selection of a semi-rigid barrier must take into account future maintenance requirements. It has been found that following a collision with a flexible barrier or a narrow median, that maintenance forces are often in danger during the time that the median barrier is being repaired. Thus, another accident or collision may occur during the reinstallation of a damaged median barrier. Damage to a semi-rigid barrier is not as extensive as to a flexible barrier.

When the median width is great enough to permit the maintenance truck to pull off of the roadway during reinstallation, the selection of a flexible median barrier is considered appropriate. It seems that a flexible beam will produce less damage to a colliding vehicle than a semi-rigid or rigid barrier. However, adequate space must be available so that the flexible barrier is not deflected into the oncoming lanes of traffic, and room *must be provided* for maintenance forces to work off the main travelled way during reinstallation.

A rigid barrier which provides a satisfactory collision behavior is one which is constructed out of concrete or steel clad concrete such as those shown in Figure C-1. In such installations the cross sectional geometry of the median barrier is such that

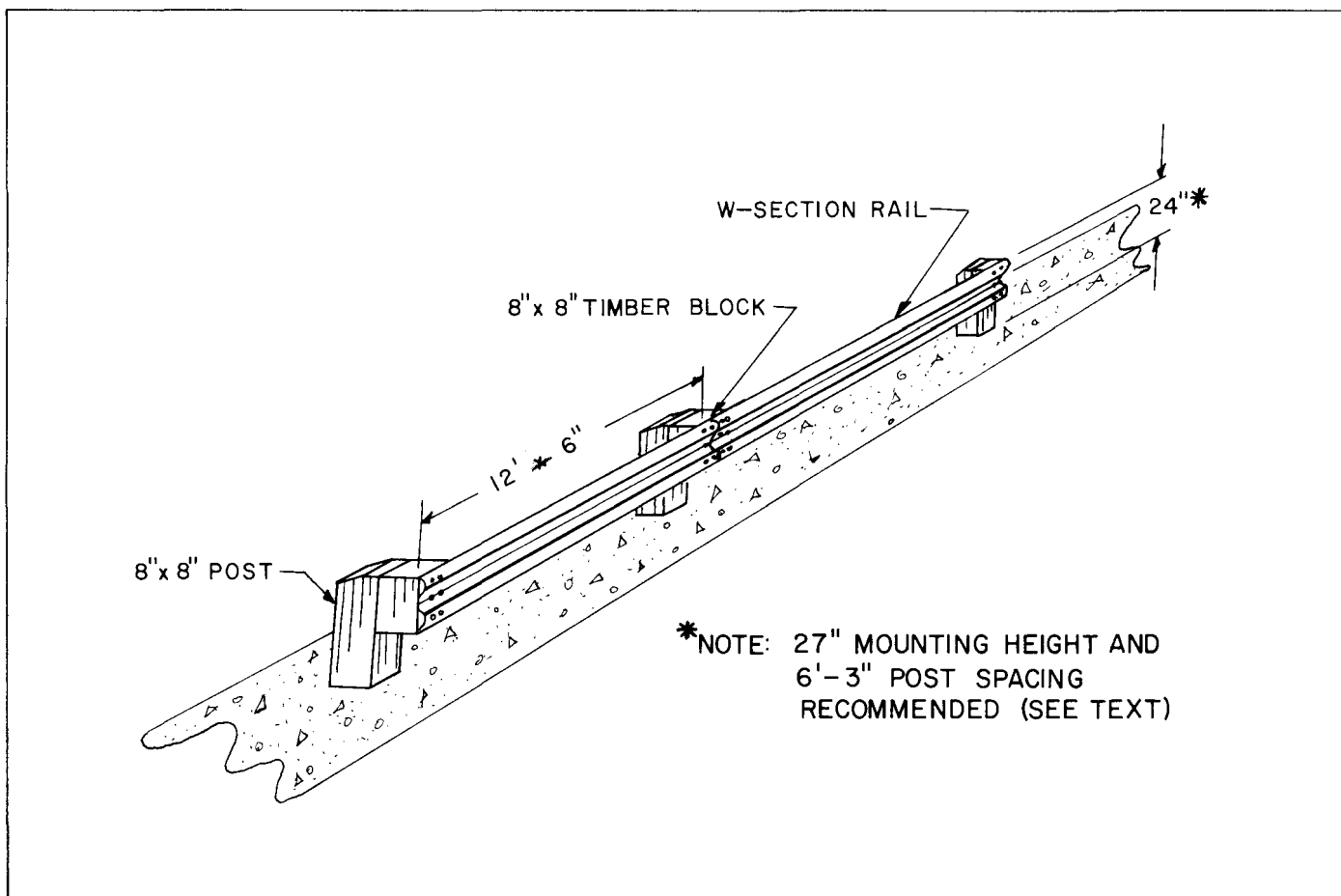


Figure C-7. Blocked-out beam guardrail

the colliding automobile cannot cross the median barrier, but it is redirected into one of the lanes in the direction in which it was travelling prior to impact. This is not an unmixed blessing, of course, because the angle of incidence and angle of reflection may be such that the colliding automobile could be redirected into the path of one of the vehicles travelling in the same direction in which the colliding vehicle had been travelling.

Thus, the design engineer must balance the favorable and unfavorable features of the three classes of median barriers which are available at this time. Some of the requirements which must be satisfied, either completely or partially, are as follows:

- (1) Insure that a colliding vehicle does not cross a median.
- (2) Whenever possible, eliminate curbs in the median, or use lay-down curb, that is, curb which is mountable without an abrupt jolt to the vehicle.
- (3) Provide for tolerable deflections. In narrow medians, it will be necessary to use a more rigid barrier than in wide medians.

Bridge rail design

A median barrier and a guardrail installed adjacent to the edge of the travelled way have similar safety requirements. It would seem that a bridge rail should also have similar requirements for safety; and of course, it does in some locations. However, in certain installations such as overpass structures, and bridges over waterways and railroads, the undercrossing traffic must also be considered. Thus certain limitations on flexibility of a bridge rail are imposed. Consideration must be given to insure that support posts and other hardware are not hurtled onto a travelled way beneath a

bridge, thus producing a hazard to other travellers.

A bridge rail *must restrain* a colliding vehicle, *prevent* it from vaulting, and at the same time *slow* the vehicle to a safe speed without severe redirection, pocketing, or snagging. The installation of a bridge rail must also be coordinated with the type of approach rail installed to insure that an out-of-control vehicle does not collide with the end of a rigid bridge rail after safely negotiating a collision with a flexible or semi-rigid median barrier or guardrail.

Highway bridge railing systems have evolved through need and experience using design information not fully substantiated by research. The railings on early bridges had only to restrain pedestrians and slow-moving vehicles not capable of producing large impact forces. Aesthetics of the railing systems were of small import. The construction and maintenance of bridge railing systems were not major items of expense as is often the case today.

In more recent times, however, the advent of high-speed highways necessary to accommodate the large volume of heavier and faster vehicles has brought bridge railing systems into major importance. For example, some highway bridge railings in recent years have proved to be decorative but not structurally adequate when subjected to the magnitude of impact forces produced by modern vehicles. Vehicle penetration of such railings has often occurred as a result of incomplete design criteria and inadequate service requirement definition.

During the last two decades, progress toward more reliable bridge rail systems has resulted from the efforts of engineers involved in designing new systems and conducting full-scale dynamic tests.

Because trucks are involved in a small percentage of the fatal accidents—and bridge rails to restrain trucks are stronger and more costly—it would seem reasonable to eliminate trucks from design considerations except in unusual circumstances.

A bridge rail design based on a vehicle impact speed of 70 mph would have included approximately 75% of the standard and smaller-sized passenger vehicles in a group of 640 single-vehicles fixed-object fatal accidents as compiled by the California Highway Traffic Department. It is recommended that full-scale dynamic tests of bridge rails be conducted at an increased speed of 65-70 mph rather than the current 60 mph.

Evidence is available which indicates that for more than 50% of the fatal accidents involving bridge railing systems the collision involves the end of the railing.

Accident information indicates that approximately 20% of the fatalities involving bridge rail accidents result from penetration of the railing. Penetration can be eliminated by proper design for strength.

Vaulting of a bridge railing can be eliminated by proper attention to railing height in addition to structural strength, and elimination of abrupt discontinuities such as curbs, safety walks, and sidewalks in front of the railing.

Highway safety programs appear to be moving in the direction of safer installations if reduction in fatal accidents is accepted as the criterion.

It is evident from accident records that many bridge railing systems in existence are not structurally adequate to restrain or smoothly redirect an out-of-control standard-size passenger vehicle.

It is evident from photographic observations of actual failures of bridge railing systems that the weak link in most designs is usually located at post connections.

At the present time, the specifications of AASHO (1965) or BPR (1962) are not sufficient to provide the design engineer assurance that localized failures will not occur at connections unless full-scale dynamic tests are conducted.

Proof tests will be required for the foreseeable future in order to evaluate selected systems.

In order to further limit vehicle decelerations it will be necessary to develop an impact attenuation device to provide lateral displacement of the bridge railing system. Indications are that a lateral displacement of 2 ft will result in a much lower deceleration level—other conditions remaining unchanged.

It is clear that guardrails and bridge rails must be designed as an integrated system to insure that transition zones between them will perform satisfactorily.

Bridge rail service requirements

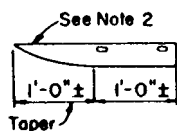
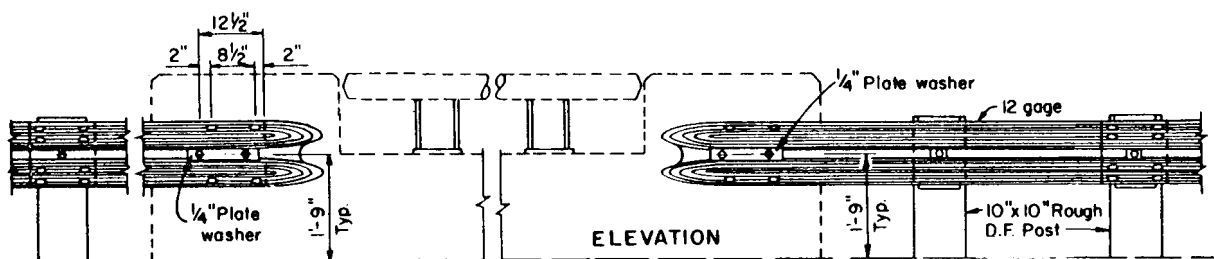
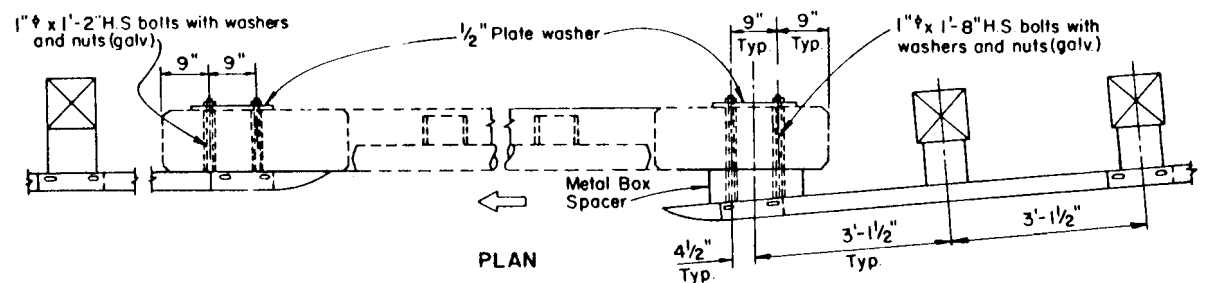
1. A bridge rail system must laterally restrain a selected vehicle.
2. A bridge rail system must limit vehicle decelerations to some tolerable level.
3. A bridge rail system must smoothly redirect a colliding vehicle.
4. A bridge rail system must remain intact following a collision.
5. A bridge rail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

Guard rail to bridge rail transition

Some of the most spectacular highway collisions occur when vehicles strike the end of a bridge, such collisions frequently produce fatalities and serious injuries. Historically, as highways have been constructed for higher speeds, bridges have become wider and wider. At present time, bridge railings on interstate highways are being installed at the edge of the shoulder. Transverse location of bridge rails corresponds to the location of approaching guard rails, and this has eliminated the undesirable narrowing of roadways at bridges. Thus, a more uniform alignment has been provided; even with this uniformity of alignment, it has become apparent that engineers must provide a continuity of strength or rigidity of the two barrier systems. Guard rails are normally attached to posts which are embedded in the ground and bridge rails are normally attached to posts which are fixed rigidly to the bridge slab. The guard rails are flexible or semi-rigid under impact, whereas, the bridge rails tend to be much more rigid when struck by a colliding vehicle. This rigidity of bridge railing is desirable from the viewpoint of restraining an out-of-control vehicle or from permitting it to vault over the bridge railing. Recently highway designers have begun to provide a structurally compatible transition from a flexible system to a rigid system.

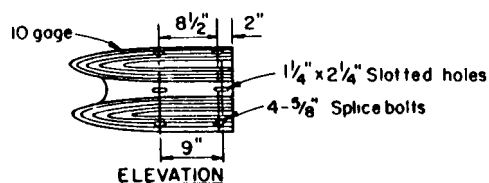
Closer spacing of guard rail posts at the ends of a bridge has proven to be a satisfactory method of providing this transition. Some full-scale crash testing of transition elements has been conducted by the California Division of Highways, and their current practice of attaching guard rail to a bridge is shown in Figure C-8. Adequate bolting of the approach guard rail to the bridge structure and anchoring at the beginning of the approach railing appears to be a necessary requirement. On approach rails, the California Division of Highways uses a blocked-out W-beam guard rail with post spacing of 6'-3", and this post spacing is reduced to 3'-1-1/2" adjacent to the end of the bridge. The intent here is to stiffen the guard rail by decreasing post spacing and when the end of the bridge is reached, the guard rail is rigidly bolted to the concrete parapet.

At the present time, there is considerable disagreement concerning the amount of flare to be used with guard rail at the approach to a bridge rail, however, the types of flare shown in Figure C-9 are being recommended in several states. Care should be exercised in determining the amount of flare to be used on an approach rail since the greater the distance which a vehicle can traverse prior to striking a guard rail, the greater the lateral component of impact force. The importance of end anchorage provided for approach guard rail cannot be underestimated, and the reader is encouraged to study the results of the California full-scale crash tests, and the details of end anchorage which was employed in these tests, and which has been adopted by the California Division of Highways.

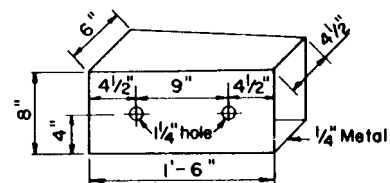


PLAN

END SECTION DETAILS

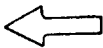


ELEVATION



METAL BOX SPACER (See Note 5)

GENERAL NOTES

1. These connection details apply to bridge rails, abutments, piers, retaining walls and other flat surface concrete objects.
2. End sections may be cut from standard terminal sections or fabricated.
3. Direction of traffic indicated by 
4. For post size and spacing See Type 1 Flare on Sheet 6.
5. When metal box spacer is installed, place 1-1/4" x 5" and 1-1/4" x 4" pipe spacers on 1" bolts passing through interior of box.

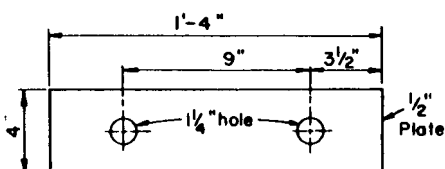


PLATE WASHER
FOR BACKSIDE OF PARAPET

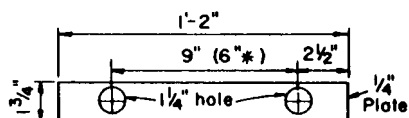


PLATE WASHER
FOR GUARD RAIL

* 6" Spacing to be used where
Detail "A" conditions are
encountered.

Figure C-8. Connection details blocked out "W" beam

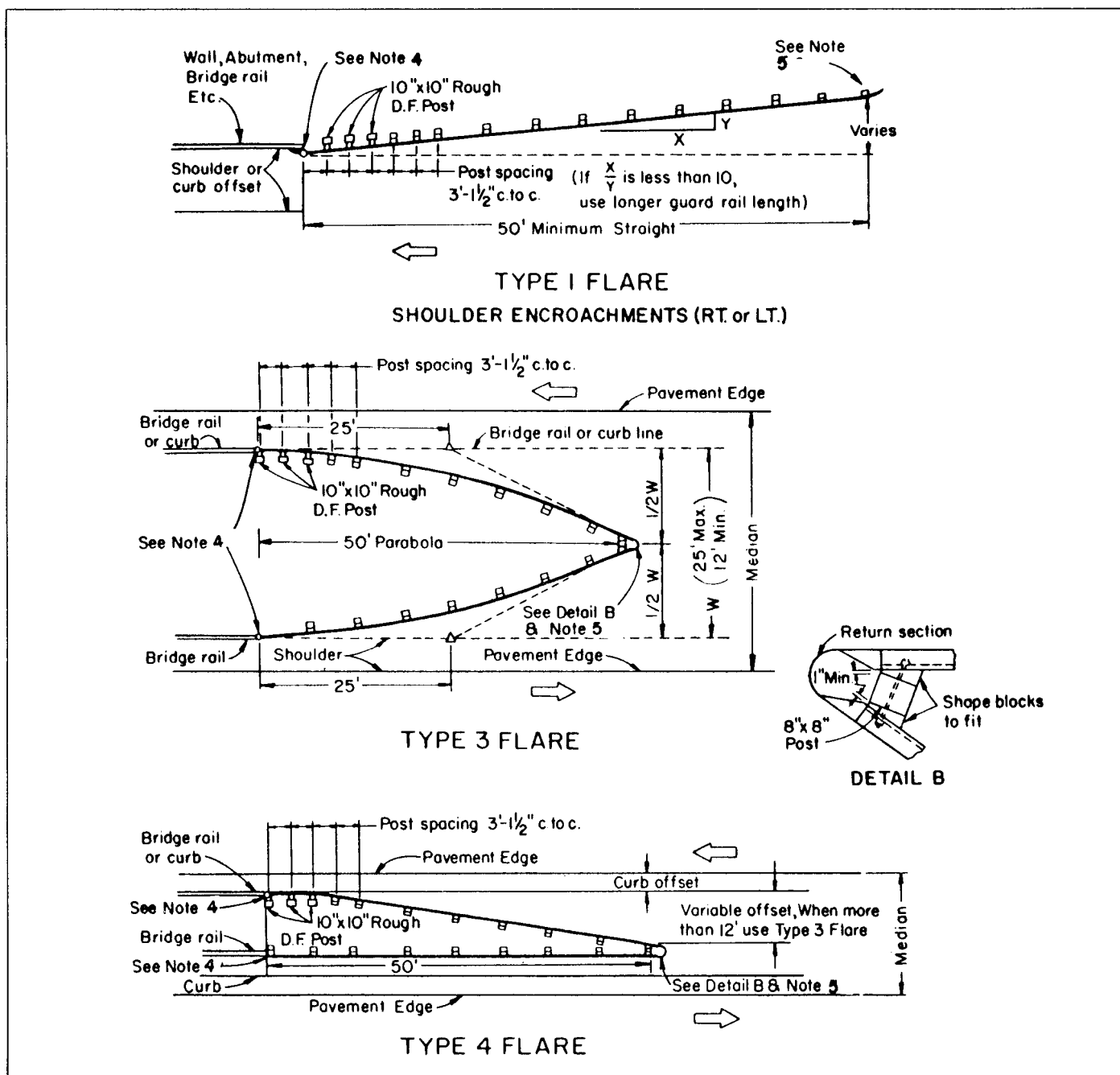
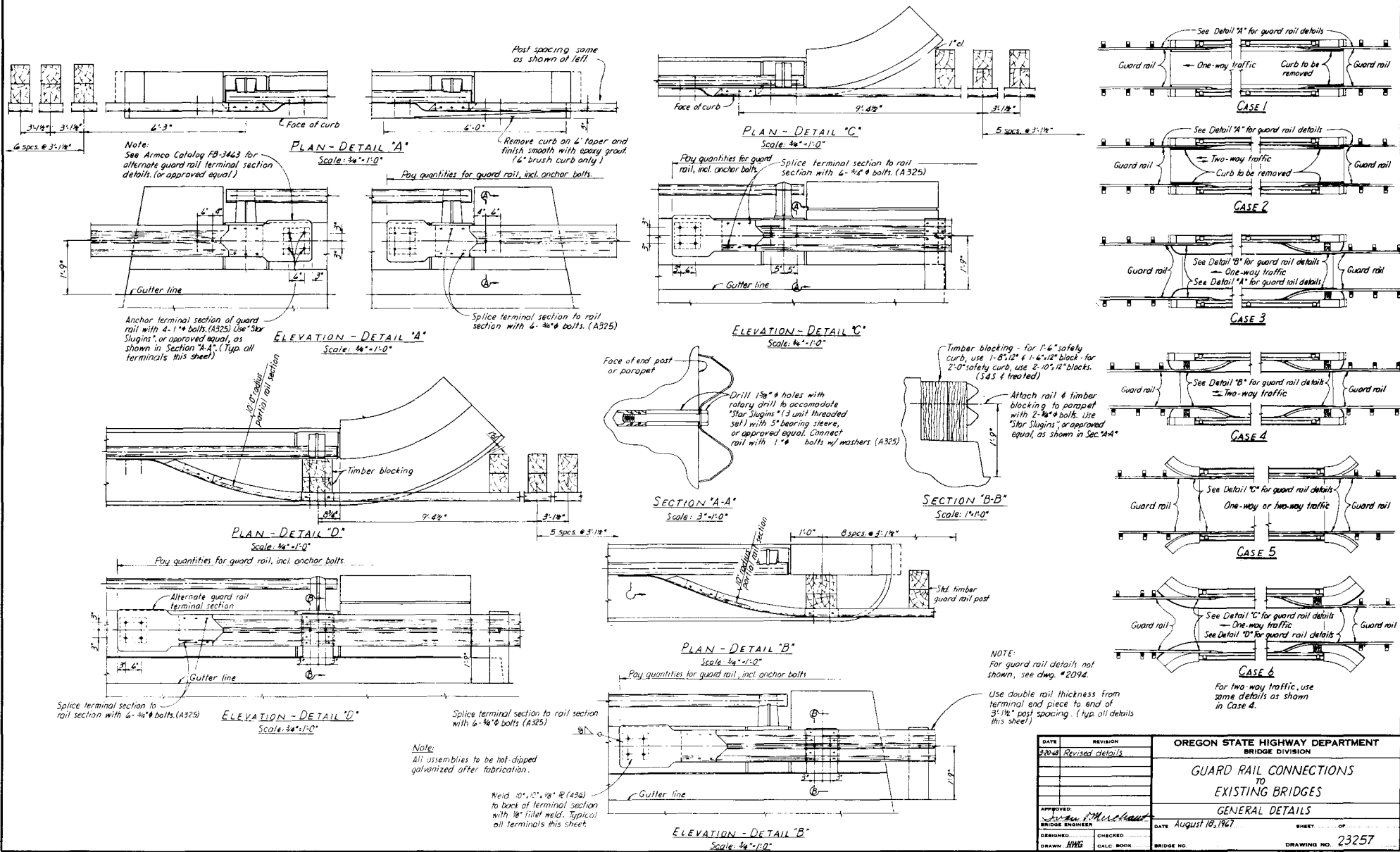


Figure C-9. Flare details blocked out "W" beam

An interesting connection between approach railing and bridge structure is seen in Oregon State Highway Department Drawing No. 23257. This connection employs timber blocking in some installations, which may prove to have impact attenuation advantages in a collision incident. Such a connection is apparently used where elimination of curbing is not permissible. Several cases for installation are shown on the drawing.

Other state highway departments have developed connection details similar to those shown in this report. Strength of the end connection must be comparable to the tensile strength of the approach rail to eliminate connection failures. The use of ASTM A325 high strength bolts is desirable. Until more field experience is obtained or until additional crash tests are performed it appears that the transition details just described are examples of the best practice known at this time.



DATE	REVISION	OREGON STATE HIGHWAY DEPARTMENT	
8/20/67	Revised details	BRIDGE DIVISION	
		GUARD RAIL CONNECTIONS	
		TO EXISTING BRIDGES	
		GENERAL DETAILS	
APPROVED:		DATE	August 18, 1967
BRIDGE ENGINEER		SHEET	OF
DESIGNED:		CHECKED:	
DRAWN: HMG		CALC. BOOK	
		BRIDGE NO.	
		DRAWING NO.	23257

Safer Roadside Structures
Part D
Break-Away Ground Mounted
Sign Supports

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-- CTR Library Digitization Team

Break-away ground mounted sign supports

The basic concept of the break-away base sign support is to provide an adequate moment connection at the base to resist wind loads; and conversely to provide a shearable connection at the base which will release when the support is subjected to an automotive collision.

The break-away base connection must be located four inches or less above finished grade to permit vehicles to clear the fixed lower section. It consists of slotted plates which are bolted together, and the static shear strength of the connection is the result of pressure between the plates. It is important to emphasize that the break-away connection must be designed to provide moment strength to resist the forces caused by the wind. The slotted plate configuration will insure that the base connection will break away under the impact (high shearing) forces of an automotive collision.

The transfer of wind moment across the base connection is by means of a couple on the base produced by a combination of bearing between the plates on the one side and the restraint from increasing bolt tension on the other. The necessary shear resistance to oppose the shearing forces of the wind loading is automatically developed through action of the couple.

The break-away sign loading conditions shown in Figure D-1 illustrate the concept just described, and define the three critical connection locations. The required characteristics of each of these connections are described for wind load conditions and collision conditions. A discussion of the design requirements for each of these locations will be presented later.

Three styles of sign support configurations have been investigated: (1) large sign supports in which the post spacing is greater than the width of a standard passenger vehicle, (2) small sign supports in which the post spacing is less than the width of a standard passenger vehicle, and (3) single post sign supports. The wind loading conditions for each of these three types of support are identical for designing the break-away base, but collision loading conditions make an inclined base for small sign supports and for single post sign supports, a desirable addition.

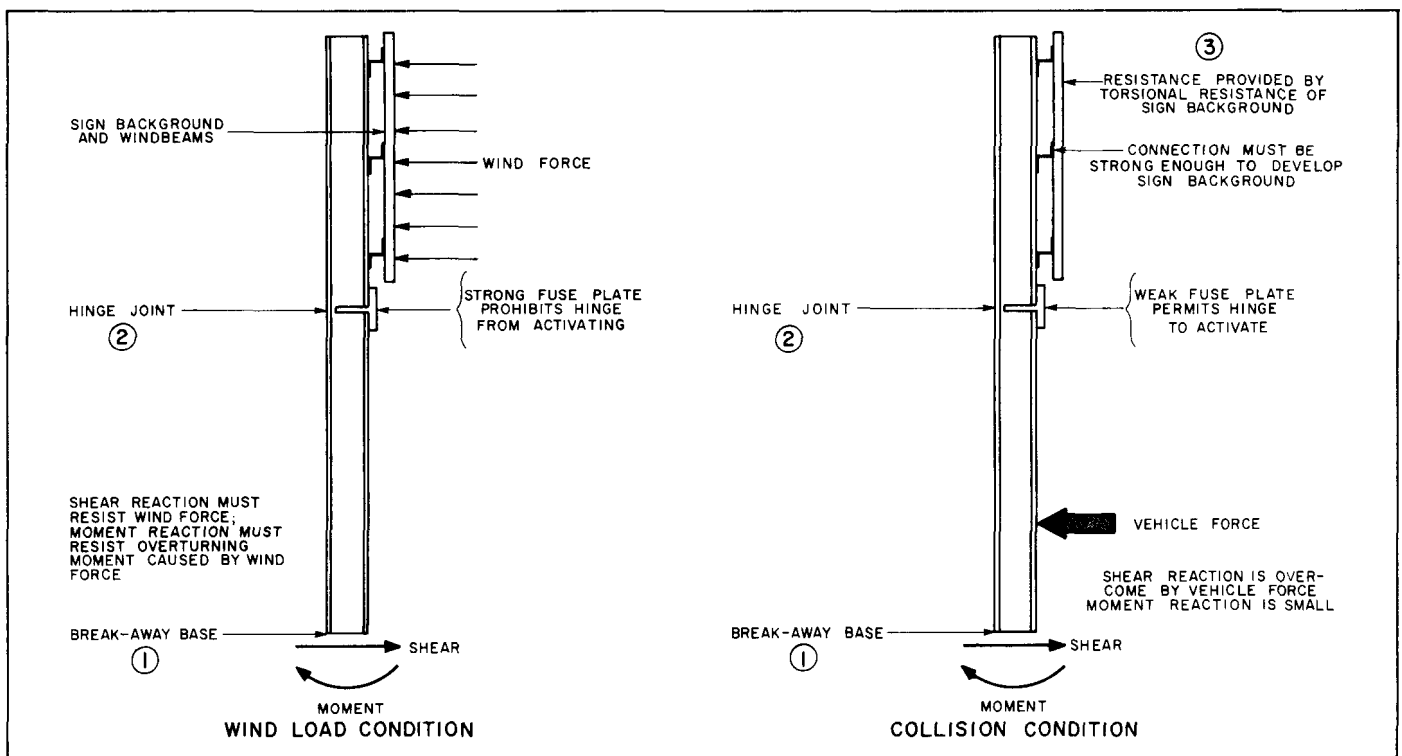
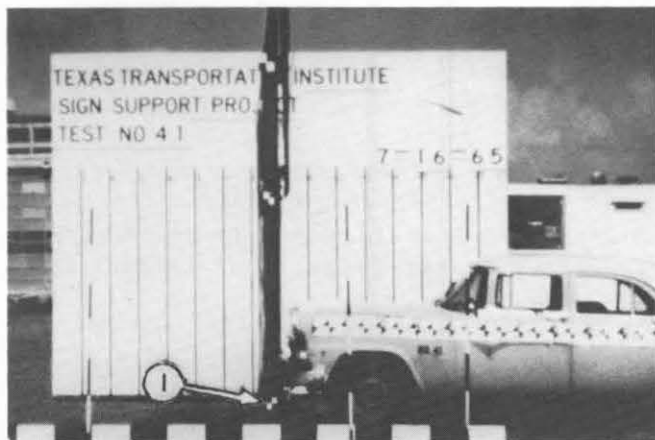


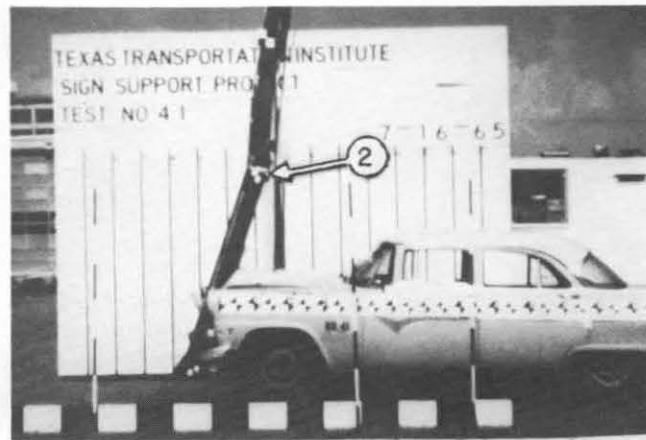
Figure D-1. Break-away sign loading conditions

The collision behavior of a large sign support is illustrated in Figure D-2. It should be noted that the vehicle struck only one support and the remaining support held the sign in position above the crash vehicle during the collision incident.

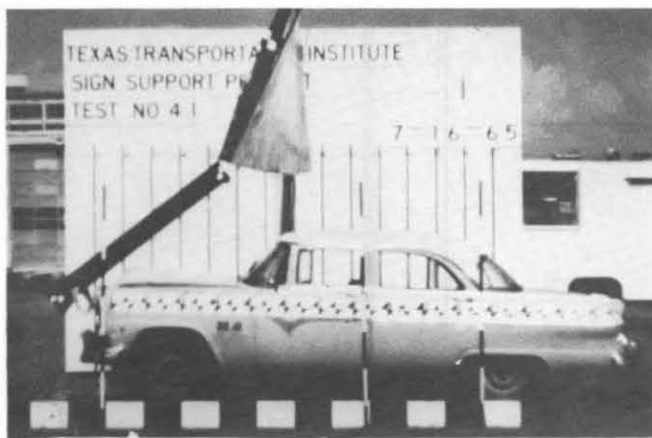
These sequence photographs are taken from high-speed film records of a full-scale crash test conducted in 1965 by the Texas Transportation Institute for the Texas Highway Department in cooperation with the Bureau of Public Roads. Installation of break-away sign supports began in November 1965 on the Texas Interstate System. Operational or in-service experience since that time has indicated that the behavior illustrated occurs under actual collision conditions. During the developmen-



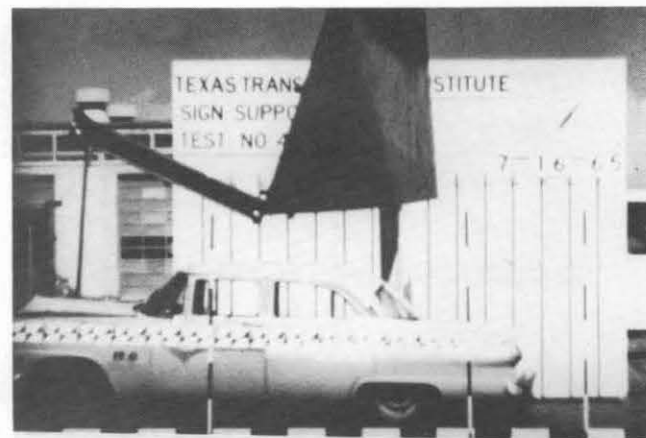
a) BREAK-AWAY BASE ① DISENGAGES



b) HINGE JOINT ② ACTIVATES



c) POST LOSES CONTACT WITH VEHICLE



d) SIGN SUPPORT POST CLEARS VEHICLE

Figure D-2. Break-away sign support behavior

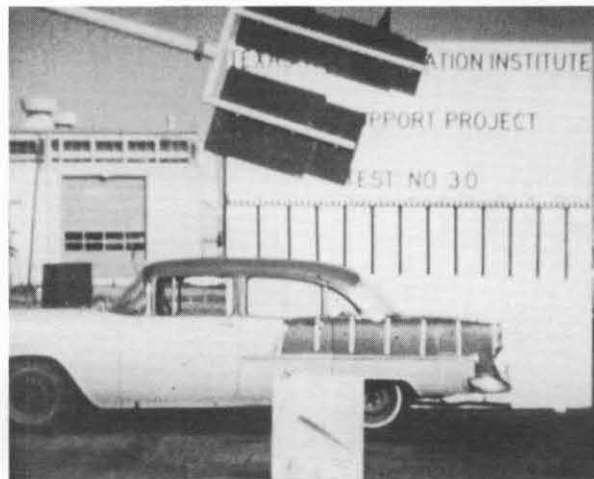
tal research studies, the crash vehicle was towed into a head-on collision with the break-away sign support; at roadside installations, accident reports indicate that supports have been struck at impact angles of as much as 45°. Colliding vehicles have been in various attitudes at the instant of collision: skidding, turning, and veering. Support posts have been struck by both right and left sides of colliding vehicles. Impacts have occurred at locations along the length of the vehicle from front bumper to rear bumper. In many collisions the vehicle damage is unknown, because the vehicle continued following the accident.

Many other states have begun installing break-away sign supports, among them Kansas, which has reported several collisions in which the vehicle occupants were spared.

Small sign and single post sign supports exhibit a different behavior in a collision incident. During the developmental research it was found that small signs, having a horizontal base, fell on the top of the colliding vehicle, an example of this behavior is shown in Figure D-3 (a); in another test, the base plate connection was inclined approximately 20° from the horizontal and the sign was catapulted clear of the crash vehicle, Figure D-3(b). These two tests illustrate the behavior of single pipe mounts. Small signs having two supports must also have inclined base plate connections to permit the sign to catapult upward as shown in Figure D-3(c). The crash vehicle struck both support posts in this crash test.



**(a) 4" STEEL PIPE SUPPORT
HORIZONTAL BASE
45 MPH**



**(b) 4" STEEL PIPE SUPPORT
INCLINED BASE
35 MPH**



**(c) 2-3I5.7 STEEL SUPPORTS
INCLINED BASE
25 MPH**

Figure D-3. Impact behavior of small sign supports

The Texas Transportation Institute conducted a series of static load tests on the break-away base design in order to determine the effect of varying the bolt torque on the base bolts. ASTM A325 galvanized bolts were initially tightened so as to bring plates into snug contact and then a predetermined torque was applied to the bolts. The coefficient of sliding friction was found to be approximately 0.21 at initial slip of the connection.

As previously stated, an initial tension in the bolts of the base connection is *not* essential for adequate resistance to the wind shearing forces. However, tightening of these bolts is necessary to insure snug contact between the upper and lower plates. If there is free play in the connection, a rocking or slapping motion is set up under oscillations from variations in wind loading. This free motion could eventually result in lateral movement, or "walking," which would cause separation of the base plates.

It is recognized that the initial tensile force in the bolts at the base connection must be kept within specified limits in order to permit the break-away base to function under collision conditions. Consideration of Figure D-4 will clarify the behavior of the base connection under applied wind forces. The base connection is made by initially tightening the bolts only enough to insure that the base plates are in intimate contact. Recommended values for torque on the bolts to create a snug condition at the base connection are contained in Table D-1. The forces shown (see Figure D-4) in the base bolts are the result of the applied wind load only. It can be seen that 100 lbs. shear force is required to keep the free body in equilibrium when the resultant wind force per post is 100 lbs. Now when this condition exists there is an available resisting shear force at the base due to the base couple of 2100 lbs., assuming a

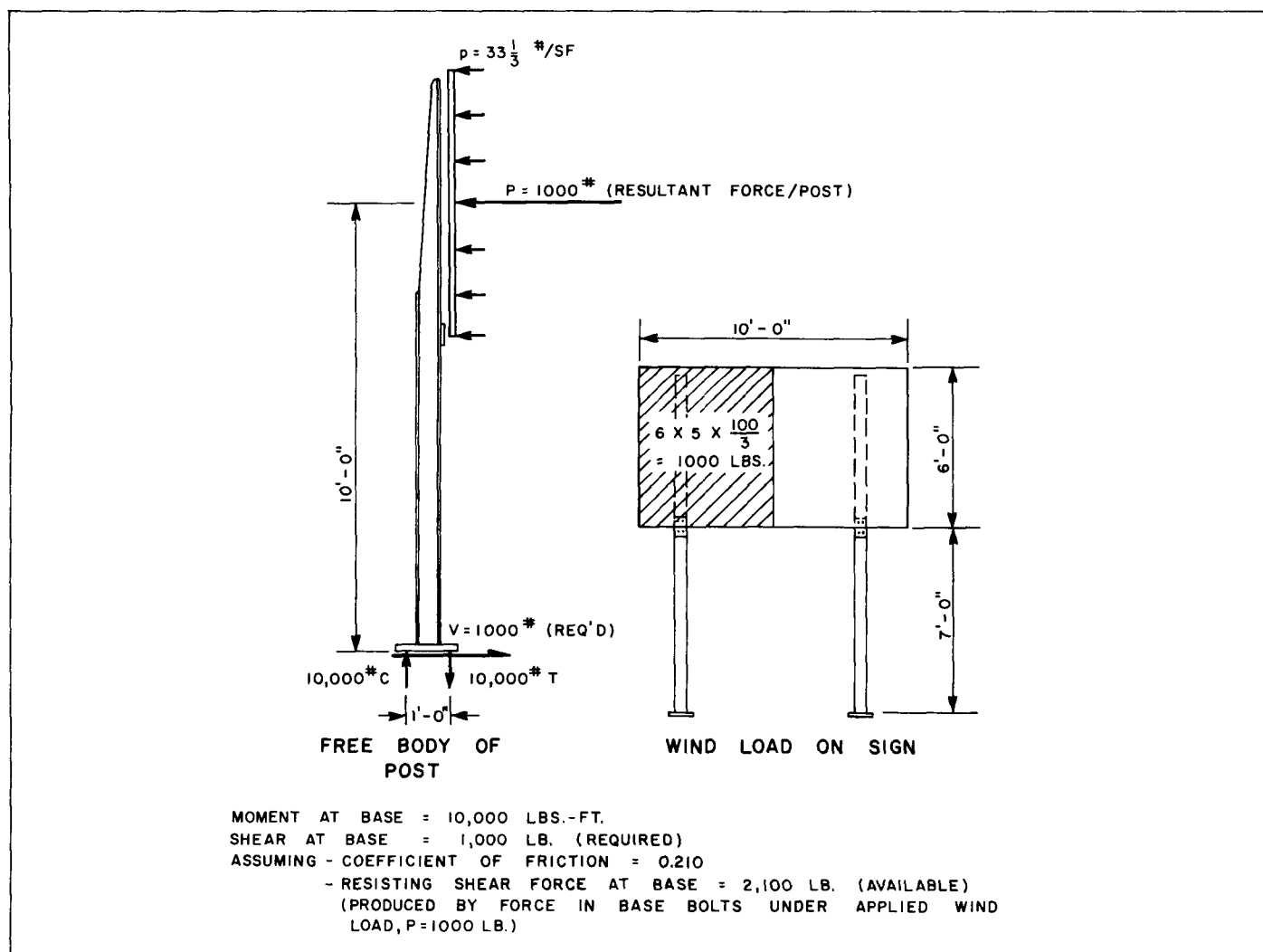


Figure D-4. Required and available shear force at base

coefficient of sliding friction of 0.210. Therefore, the available strength of the base connection to resist shear caused by the wind is greater than the actual shear force caused by the wind load. The additional tensile force provided by specifying the 200 lb.-in. to 1470 lb.-in. torque increases the shear resistance of this connection, and this added resistance will keep the plates in contact and will eliminate “walking” of the support. Or, to put it another way, if the bolts are tightened as recommended, the developed strength of the connection will be adequate to resist wind forces, and the base connection will be weak enough to break away under the force of a collision incident. This is an important concept which should be borne in mind by the engineer when he designs the base connection to resist applied wind force, and at the same time to break away under a collision force. The greatest danger, and one which must be avoided, is overtightening the base bolts. Fabricators and maintenance personnel should be instructed not to overtighten the bolts in the field, this overtightening can be avoided by using a hand wrench in tightening the bolts to insure that the base plates are snug.

Table D-1. Recommendations for design of “breakaway” supports

1. Post sizes A standard structural section weighing less than 45 lb./ft. selected to resist the maximum wind load moment. 2. Base plate and base connection The base plate should be designed for the maximum wind loads and not weigh more than the maximum values shown in the table below. The base bolts should be designed to resist the maximum wind load assuming no pre-tension. The initial bolt forces in the table below are recommended:													
Post Size (lb./ft.)	Bolt Diam. (in.)	Bolt Force (lb.)	Torque (A325, galv.) (lb.-in.) (lb.-ft.)		Base Plate Wt. (lb.)								
0- 8	½	920-1380	200-300	16.7- 25.0	8.0								
9-20	⅝	1740-2660	460-680	37.5- 56.5	12.0								
21-30	¾	2400-3600	750-1060	67.5- 88.3	21.0								
30 +	⅞	2400-3600	850-1280	70.8-106.8	21.0								
30 +	1	2400-3600	450-1470	77.1-118.2	21.0								
3. Fuse connection The moment capacity for the fuse connection is determined by the maximum wind load moment at the fuse. If slotted plates are used the initial bolt force can be determined by $N' = \frac{M}{mnf(s)r} \quad \text{where } f(s)_{\max} = 0.26$ $f(s)_{\text{avg}} = 0.21$ $f(s)_{\min} = 0.17$ and, m = number of bolts, n = number of faying surfaces per bolt, and r = depth of post section. ASTM turn-of-nut tightening methods are satisfactory if background-to-post connections are adequately designed. For torque wrench tightening, the bolt force may be calculated using $N' = K_T (\tau) \text{, where } \tau = \text{bolt torque in in.-lb.}$ Bolt Diameter (ASTM A325 galvanized) <table> <tr> <td>(in.)</td><td>K_T</td></tr> <tr> <td>½ - 13UNC</td><td>4.940</td></tr> <tr> <td>⅝ - 11UNC</td><td>3.870</td></tr> <tr> <td>¾ - 10UNC</td><td>3.185</td></tr> </table> 4. Background-to-post connection The maximum connection force anticipated is 10,000 lb. 5. Rotational stiffness of sign background A minimum stiffness of 100-ft.-lb./degree (5,730 ft.-lb./radian).						(in.)	K_T	½ - 13UNC	4.940	⅝ - 11UNC	3.870	¾ - 10UNC	3.185
(in.)	K_T												
½ - 13UNC	4.940												
⅝ - 11UNC	3.870												
¾ - 10UNC	3.185												

In service and under the action of wind, the components of the base connection tend to seat and, where galvanized, wear down the galvanizing runs. There is a tendency for the grip of the bolt to lessen and free play results. It is recommended that the bolts be retightened after one or two months of service. Generally, one retightening is sufficient.

The recommendations contained in Table D-1 are based on experience with installations in Texas, information from mathematical simulation, and full-scale crash tests. A discussion of the several break-away devices follows.

The horizontal base plate is illustrated in Figure D-5 and certain important details are emphasized. It is believed that the use of stiffened "ear" plates on these

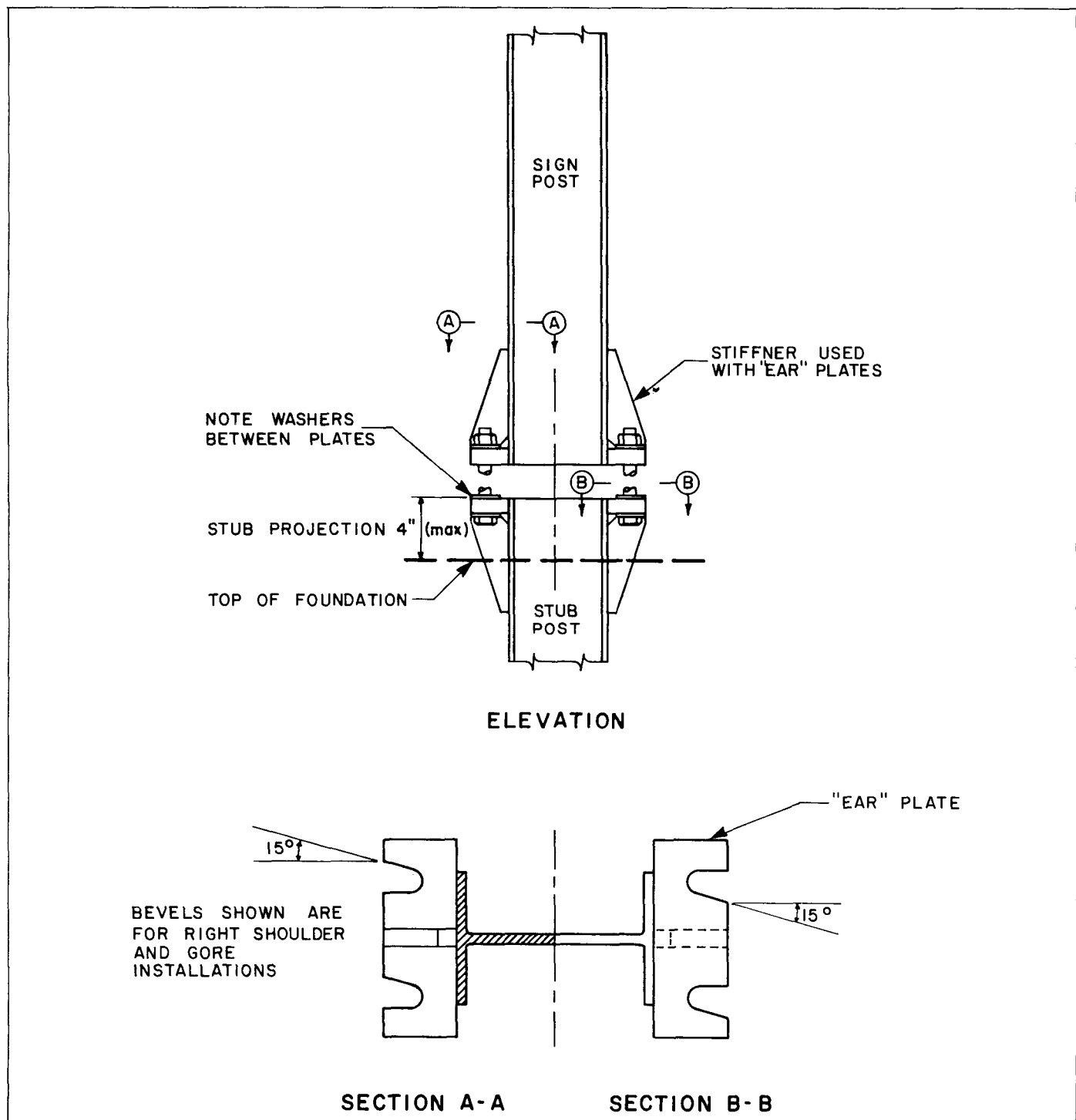


Figure D-5. Horizontal base plate for large sign supports

large supports provides the optimum design configuration to resist wind load and to keep the weight of the upper "ear" plates at a minimum to produce the desired behavior under collision conditions. The bevel on the slots has proven to be a satisfactory optimum value, and the maximum value of a four inch stub projection permits clearance of small automobiles such as compact cars and sports cars.

Smaller sign supports require an inclined base to permit the sign to be catapulted over the crash vehicle. Details of this design are shown in Figure D-6. It is recognized that the catapulting behavior can only be obtained when the support is struck from a favorable direction. Therefore, location of installations should be made with this in mind, particularly when installations may be required on two-way undivided feeder roads, etc.

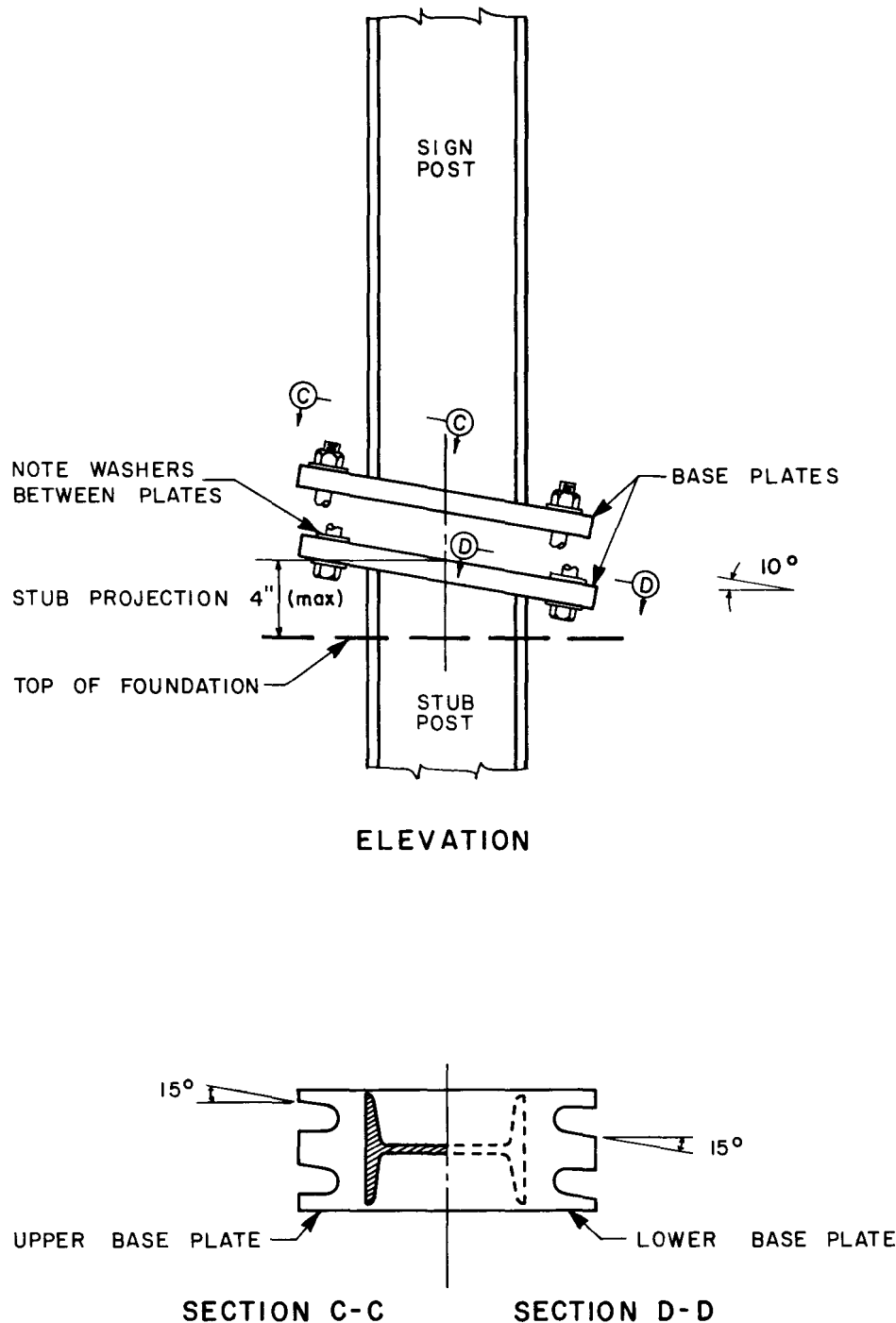


Figure D-6. Inclined base plate for small sign supports

Recommendations for design of break-away supports are contained in Table D-1, support posts can be fabricated from mild steel such as ASTM A7, A36, or A441. The Texas Transportation Institute study recommends that a standard structural section weighing less than 45 lb./ft. be selected to resist the maximum wind load moment. It has been found that the use of ASTM A441 steel is beneficial because of its advantageous allowable stress to weight ratio. When the higher strength steel is specified, the size of the required post leads to a more aesthetically proportioned sign support.

The behavior of the break-away base in a collision incident is somewhat affected by the weight of the base plate. Maximum values of base plate weight are contained in Table D-1. The use of a stiffener plate in conjunction with the base plate is recommended for larger sign supports in order to keep the base weight to a minimum. This detail is shown in Figure D-5. Some highway designers use a thicker base plate and eliminate the stiffener plate. When this is done, the total weight of the base plate should be kept within the allowables shown in Table D-1.

One of the most vexing problems which has confronted the engineer in designing break-away sign supports has been the detailing of a satisfactory fuse connection at the hinge joint. In the initial concept for the hinge joint, a cast iron fuse plate was used to connect the upper and lower portions of the post, however, owing to the nature of cast iron materials, stress concentrations at the bolted connection caused uncertainty concerning the behavior of the fuse plate when subjected to wind forces. A discussion of current practices follows.

Hinge joint

The hinge joint is located approximately at the level of the bottom of the sign (usually seven feet above finished grade). It is fabricated by cutting the forward flange and the web to the rear fillet; the forward flange is connected by bolting a mechanical fuse plate on the outside of the forward flange. The rear flange serves as a "plastic hinge" when the fuse plate releases under collision conditions.

The purpose of the hinge joint is to permit the lower portion of the disengaged support post to swing upward and away from contact with a colliding vehicle as illustrated in Figure D-7.

Cast iron fuse plate connection. Two types of fuse plates have been employed in break-away sign supports. At first, a fuse plate fabricated of cast iron was used and later a slotted steel plate was adopted. It should be emphasized that each type of plate was subjected to full-scale crash tests and each performed satisfactorily. It should further be emphasized that each type of fuse has performed satisfactorily in actual collisions with installations in Texas.

Each type of fuse must meet the requirements that it be strong under wind loads and weak under collision loads. Currently, the slotted steel plate is specified. In some accidents, the slotted steel plate has released only partially and the support has been ripped from the sign background. In these accidents, no injuries have occurred, but separation of the support from the background results in additional expense in re-erecting a damaged support.

Originally, the fuse plates were fabricated of Class 30 cast iron. These fuse plates were designed to fracture in tension; computations were made on the net section through bolt holes using ultimate stress for Class 30 cast iron. Thus, the area required was computed by

$$A_{\text{net}} = \frac{F}{\sigma_{\text{ult}}} \quad (1)$$

where

A_{net} = area on net section.

F = force in plate caused by design wind load.

σ_{ult} = ultimate stress in Class 30 cast iron.

Some failures of cast iron fuse plates occurred in early installations in Texas; the wind load in these failures was below the design wind load. At the time, an immediate solution was sought to avoid the cast iron failures, and the alternate slotted steel fuse

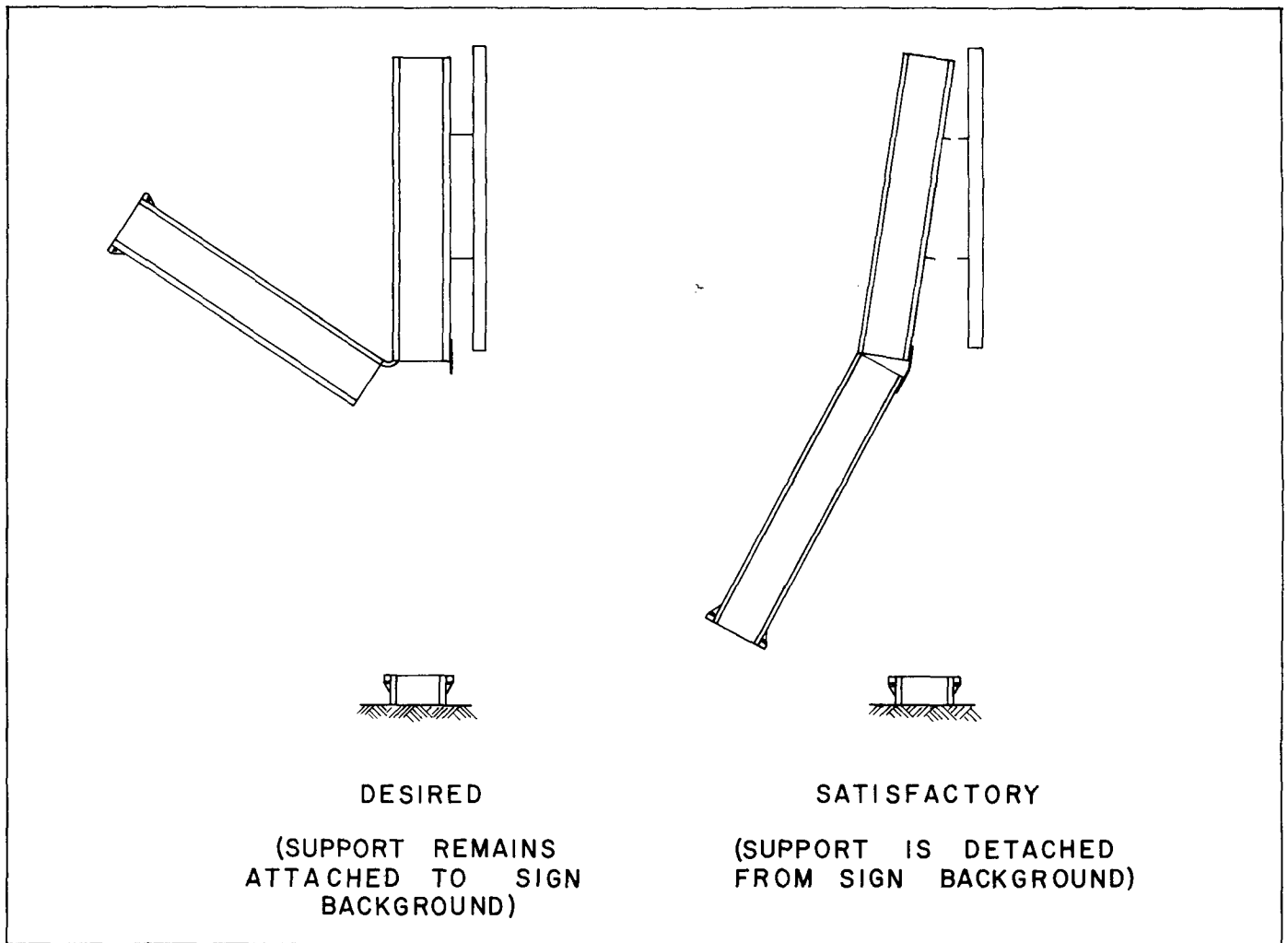


Figure D-7. Collision behavior

plate was proposed to replace the cast iron fuse plate. This slotted fuse plate will be discussed in a subsequent section; however, a discussion of cast iron fuse plate investigations at TTI follows.

A survey of literature and a limited series of static load laboratory tests were conducted. The effect of circular holes on stress distribution in plates has been presented by Timoshenko, Seely and others. Theory indicates that the stress concentrations at holes in a brittle material produce stress concentration factors of approximately 3.0, thus, the resulting ultimate stress on the net section of the fuse plate should be estimated by the relation

$$\sigma_{ult} = \frac{3F}{A} \quad (2)$$

or equation (1) should be replaced by

$$A = \frac{3F}{\sigma_{ult}} \quad (3)$$

where

- A = area of the gross section, in.²
- F = force in plate caused by design wind load, lb.
- σ_{ult} = ultimate stress in brittle material, psi.

For example: given a 5-1/4" x 5-1/4" x 3/8" plate with 1" diameter holes, fuse plate made of Class 30 cast iron having an average ultimate strength of 32 ksi. The theoretical fracture force is:

$$F = \frac{1}{3} (5\frac{1}{4} \times \frac{3}{8}) \times 32 = 21 \text{ kips}$$

The theory was developed for a semi-infinite elastic plate containing only one hole with a fixed value of stress. A cast iron plate at rupture does not meet these conditions, however, several specimens have been tested in tension and flexure, and the average fracture force was higher than the theoretical force. This can be explained because cast iron fabricators generally provide material which is stronger than the minimum average requirements.

It is recognized that practical considerations of load application through the bolts, variation in ultimate stress and other influences resulting from casting and handling techniques can affect the theoretical value of the stress concentration factor. An empirical value could be arrived at by sufficient laboratory testing and such a program is recommended before such brittle materials are specified for roadside installations.

Slotted steel fuse plate connection. The strength or capacity of the slotted plate fuse connection is dependent upon friction, and can be expressed as

$$F = mnN'f \quad (4)$$

where

- F = joint strength or capacity, lb.
- m = number of bolts.
- n = number of friction surfaces per bolt.
- N' = initial tensile force per bolt, lb.
- f = coefficient of friction.

The determination of a value for the coefficient of friction is dependent upon laboratory investigations. A limited series of laboratory tests was conducted for the Texas Highway Department in 1965 and the coefficient of friction using equation (4) was found to vary between 0.17 and 0.26. It should be noted that the above expression is independent of plate dimensions.

D. L. Hawkins of the Texas Highway Department proposed an empirical equation which includes the plate dimensions:

$$F = \frac{(N')^2 e}{(t)^2 R} \quad (5)$$

where

- F = joint strength or capacity, lb.
- N' = initial bolt tension, lb.
- e = edge distance, in.
- t = fuse plate thickness, in.
- R = empirical constant of proportionality based upon limited laboratory investigations = 175.

Equation (5) provides a direct method for fuse plate design. Examination of Hawkins' equation reveals that the designer has a freedom of choice in selecting values for initial bolt tension, edge distance and plate thickness. Each choice is restricted, however, if one employs available bolt sizes, acceptable edge distances, and available plate thicknesses. These limitations are:

- (1) Using ASTM A325 bolts limits the choice to diameter increments of 1/8" between 1/2" and 3/4"
- (2) Edge distance is limited by AISC standard gage requirements for the flange of the support post.

*Maximum diameter limited by mathematical model parameter studies conducted by the Texas Transportation Institute.

(3) Plate thickness is limited by AISC standard rolling practices.

Of course, other bolt sizes could be obtained, other edge distances specified, and milled plates could be employed. Specifications of such speciality items could result in very expensive fuse plate connections.

Design within the limitations listed above permits considerable freedom of choice. However, it is recommended that the following procedure be followed. Select bolt diameter, edge distance, and plate thickness, compute capacity using equation (5), then compute the coefficient of friction using equation (4), if the value of f is between 0.17 and 0.26, then the design is within the currently known limits for capacity of a slotted plate fuse.

It is necessary to specify the value of initial tensile force in the bolts, (N'). Current practice conforms to "Specification for Structural Joints Using ASTM A325 or A490 Bolts," approved by the Research Council on Riveted and Bolted Structural Joints. The minimum bolt tension values are (1966):

Bolt Size (Inches)	Minimum Tension A325 Bolts (kips)
$\frac{1}{2}$	12
$\frac{5}{8}$	19
$\frac{3}{4}$	28

These minimum tension values may be attained by tightening bolts with properly calibrated wrenches or by the turn-of-nut method.

It should be emphasized that each design must also meet the requirements imposed by the AISC Manual and other applicable specifications for structural design. Thus, the designer will find that the selection of rolled sections, bolt diameters, plate thicknesses and other variables is restricted.

Panel-to-post connection

The sign panel may be attached to the support post in several ways, and such a connection is partly dependent upon the material from which the sign panel is fabricated. The use of horizontal wind-beams and clamped connections facilitates sign erection procedures.

It has been found in previous crash tests that direct bolting provides more fixity to permit the desired behavior illustrated in Figure D-7; however, it has been observed from field experience, that the clamped connections provide a secondary fuse in the event that the hinge connection fails to function in a collision incident. A desirable compromise appears to be a combination of a bolted and clamped connection. That is, the lower windbeams may employ clamps to facilitate erection, and after the panel is in place, the upper windbeams can be bolted directly to the posts. In this way erection requirements can be satisfied, and a satisfactory collision behavior can be realized since the support post can rotate about the bolted connection at the top windbeam, a recommended detail is shown in the drawings which follow. The positively bolted upper connection can thus serve as a back-up system to the fused hinge and assist in retaining control over the released post.

A panel with sufficient structural integrity, adequate torsional stiffness, and satisfactory connection strength will provide the optimum conditions for "desired" behavior of the break-away sign support concept. Parameter studies indicate that the critical panel-to-post connection strength varies from two to ten kips (depending upon the size of the sign); thus it has been recommended in Table D-1 that connections be provided by direct bolting or by clamping to resist a force of 10,000, pounds. The recommended rotational stiffness of the sign background should be greater than 100 lb.-ft./degree of rotation.

Typical working drawings of the break-away concept employed in several states are contained in the following pages.

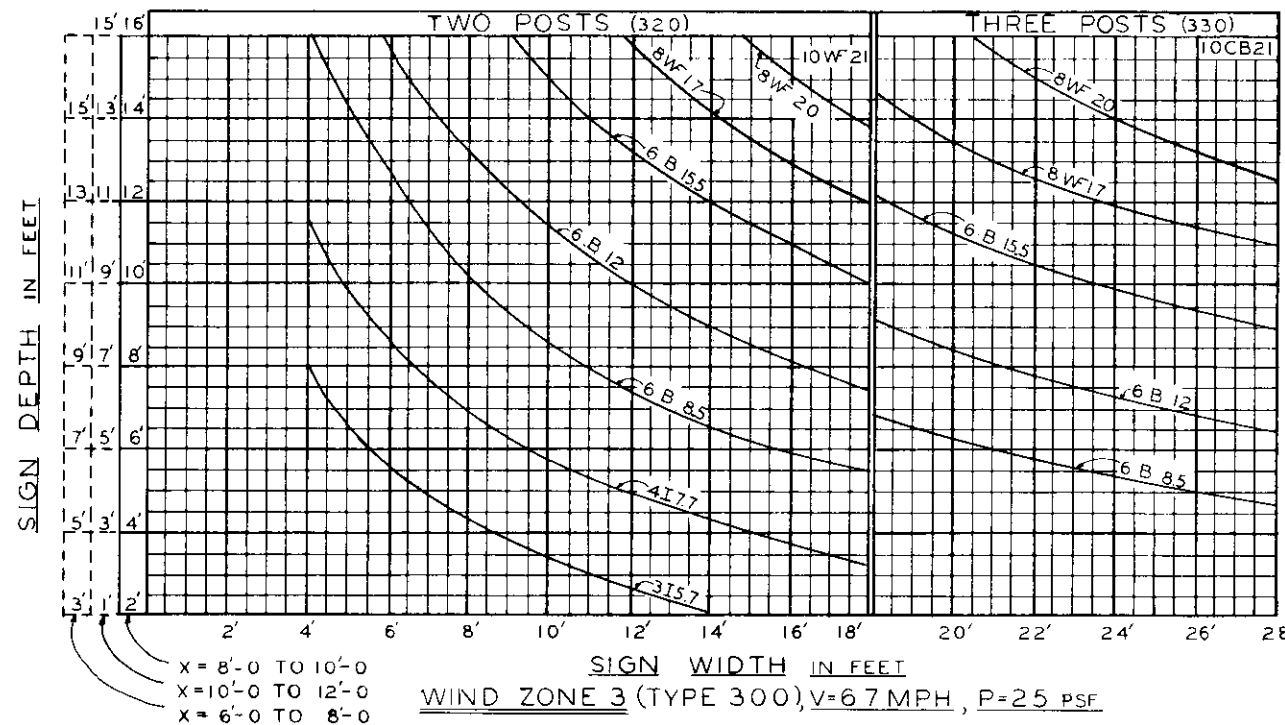
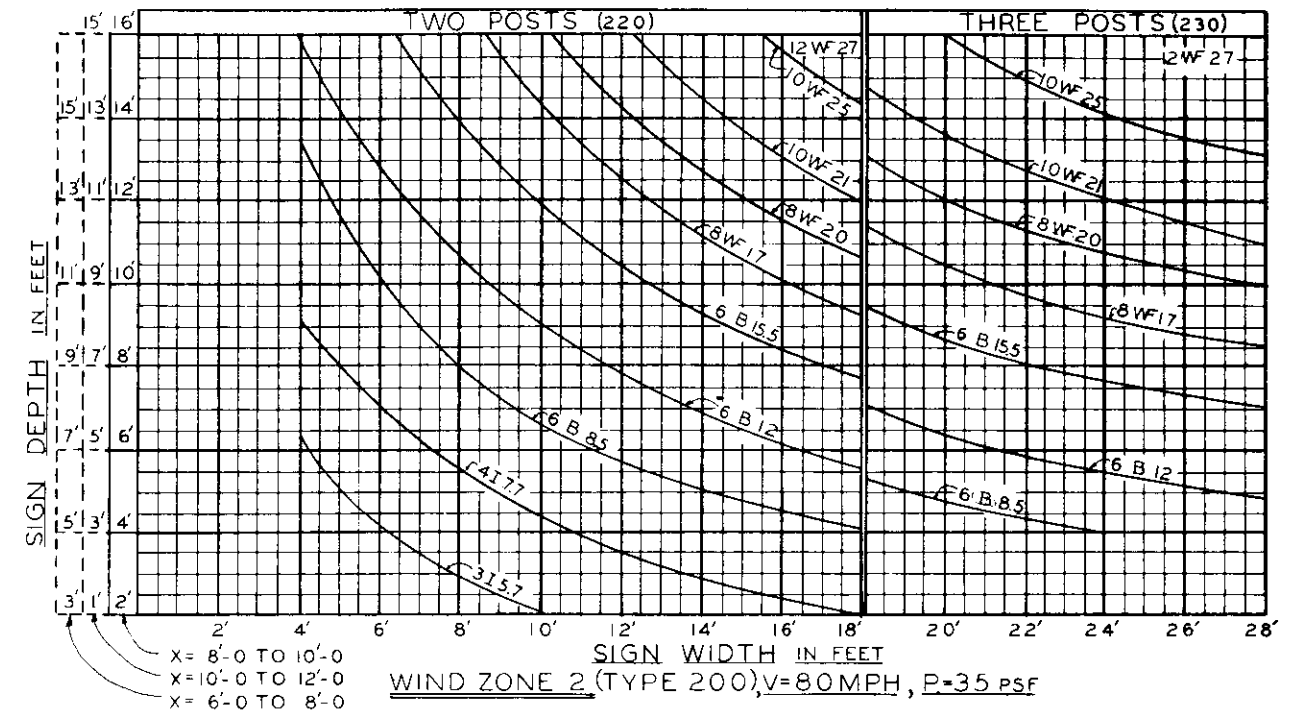
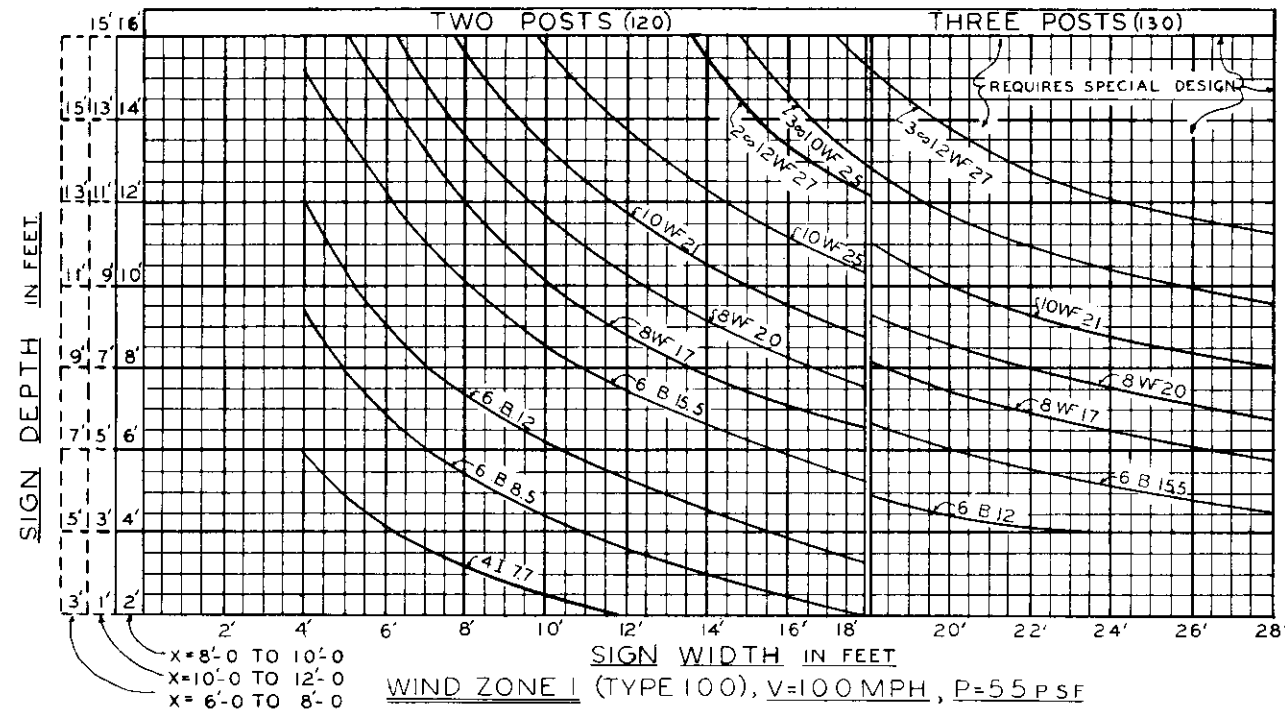
Figure 1

Ground Mounted Sign Supports Breakaway Type

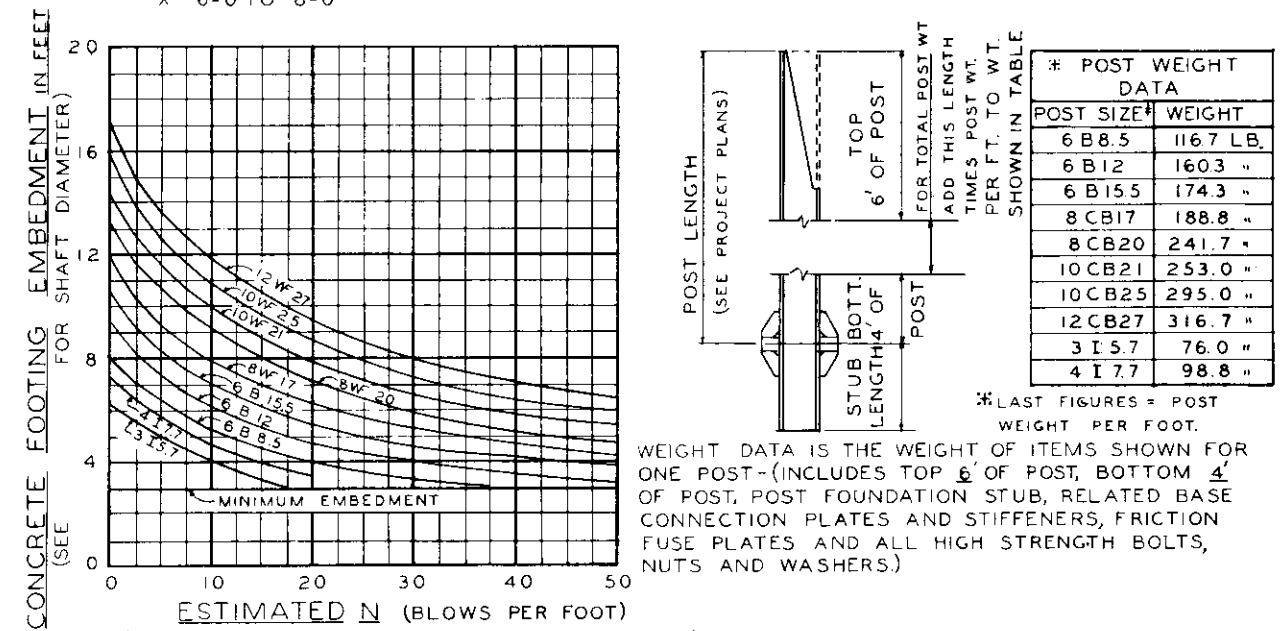
Post Details
Foundation Data
Fuse Plate Data

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NOTE: "X" EQUALS THE AVERAGE HEIGHT FROM THE GROUND LINE TO THE BOTTOM EDGE OF THE SIGN.



NOTE: ESTIMATED N SHOULD BE BASED AT APPROXIMATELY THE UPPER ONE-THIRD POINT OF THE DRILLED CONCRETE FOOTING BELOW THE GROUND LINE.

DRILLED CONCRETE FOOTING DEPTH CHART

SIGN TYPE 130
WIND DESIGN ZONE
SERIES NO. (ZERO DENOTES PLYWOOD)
CODE L NO. OF POSTS

Ground Mounted Sign Supports Breakaway Type

Post Details
Foundation Data
Fuse Plate Data

Figure 1



United States Steel Corporation
Highway Construction Marketing

Figure 2

Ground Mounted Sign Supports Breakaway Type

Post Selection Chart
Footing Depth Chart

Revised November, 1967

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	TWO POSTS (120)								THREE POSTS (130)								
SIGN WIDTH(W)	4'THRU 12'	13'	14'	15'	16'	17'	18'	19'	20'	21'	22'	23'	24'	25'	26'	27'	28'
MAX WIND BEAM SPA	4'-0	3'-8	3'-4	3'-0	2'-8	2'-4	2'-0	4'-0	4'-0	3'-9	3'-6	3'-3	3'-0	2'-9	2'-6	2'-3	2'-0

ZONE 1 (TYPES 100)

	TWO POSTS (220)								THREE POSTS (230)							
SIGN WIDTH(W)	4'THRU 16'	17'	18'	19'	20'	21'	22'	23'	24'	25'	26'	27'	28'			
MAX. WIND BEAM SPA.	4'-0	3'-9	3'-6	4'-0	4'-0	4'-0	4'-0	4'-0	4'-0	3'-9	3'-6	3'-3	3'-0			

ZONE 2 (TYPES 200)

	TWO POSTS (320)	THREE POSTS (330)
SIGN WIDTH(W)	4'THRU 18'	19'THRU 28'
MAX. WIND BEAM SPA.	4'-0	4'-0

ZONE 3 (TYPES 300)

NOTE: SPLICE BARS SHALL BE SECURED WITH $\frac{5}{8}$ LONG $\times$ $\frac{1}{2}$ ROUND HEAD, CADMIUM PLATED, BRASS WOOD SCREWS SPACED AT 6" MAX. ALONG DOUBLE ROW SPACING WITH THE EXCEPTION AT EXTREME ENDS OF SPLICE BARS, ALONG OUTSIDE EDGE OF SIGN, WHERE PLYWOOD BOLTS SHALL BE USED AS SHOWN.

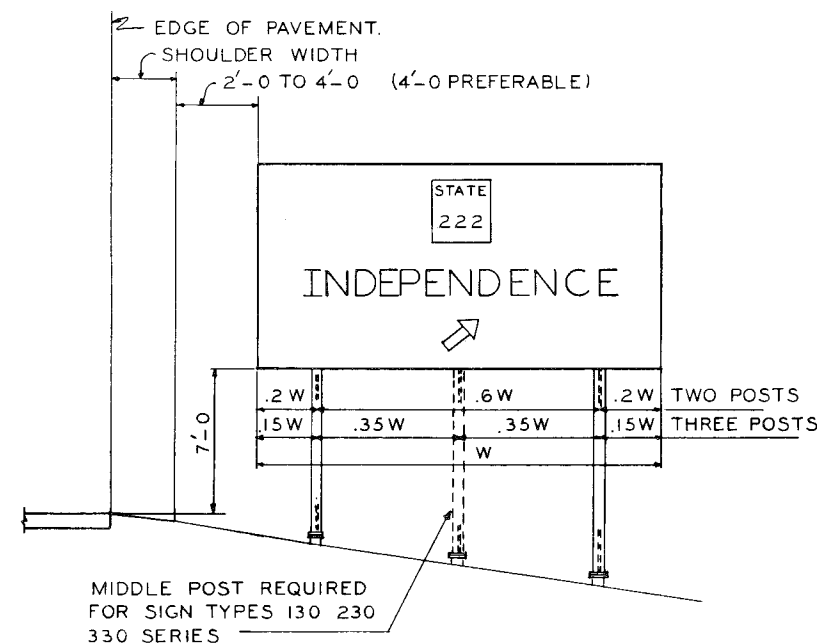
NOTE

A TOLERANCE OF PLUS OR MINUS $\frac{1}{4}$ INCH WILL BE PERMITTED IN THE PLAN DIMENSIONS FOR FABRICATION OF EACH SINGLE INCREMENT SIGN PANEL WHEN NECESSARY FOR SQUARING. A TOLERANCE OF PLUS OR MINUS $\frac{1}{4}$ INCH WILL BE PERMITTED FOR EACH INCREMENT OF A MULTI-INCREMENT SIGN PANEL WHERE NECESSARY TO SECURE SQUARE TIGHT-FITTING JOINTS.

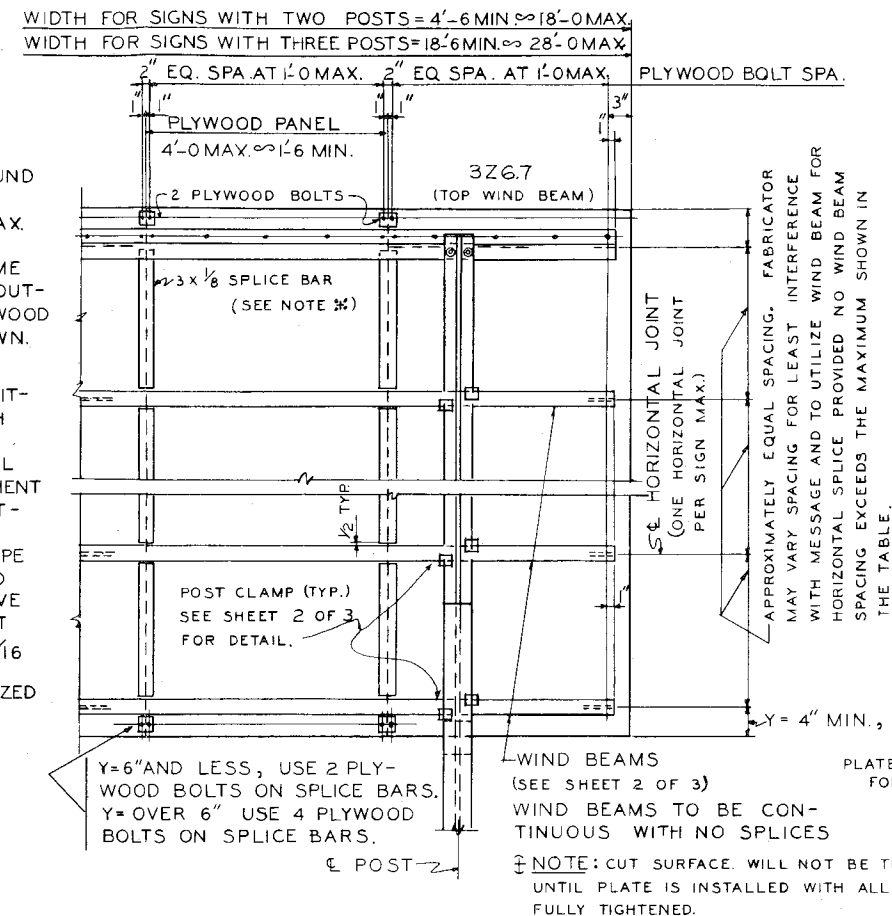
PLYWOOD BOLTS - $\frac{5}{16} \times 1\frac{1}{2}$ FLAT HEAD ELEVATOR TYPE STEEL BOLT WITH $\frac{3}{4}$ MINIMUM THREAD LENGTH. THE HEAD SHALL HAVE A MINIMUM DIAMETER OF $\frac{3}{4}$ AND SHALL HAVE A MINIMUM OF TWO FINS ON THE UNDERSIDE. EACH BOLT SHALL BE PROVIDED WITH ONE $\frac{5}{16}$ STEEL HEX. NUT, ONE $\frac{5}{16}$ STEEL FLAT WASHER AND ONE $\frac{5}{16}$ STEEL LOCK WASHER. BOLTS, NUTS AND WASHERS SHALL BE HOT DIPPED GALVANIZED AS PER ASTM A-123

WIND BEAM TABLES

NOTE-WIND BEAM SPACINGS FOR SIGN WIDTHS OTHER THAN EVEN FEET: SELECT FROM NEXT LONGER EVEN FEET.



TYPICAL SIGN INSTALLATION AND LOCATION



REAR ELEVATION

(SHOWING TYPICAL ASSEMBLY FOR GROUND MOUNTED SIGNS)

GENERAL NOTES:

DESIGN CONFORMS WITH AASHO SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS.

MATERIALS AND FABRICATION SHALL CONFORM TO THE REQUIREMENTS OF THE STATE HIGHWAY DEPARTMENT STANDARD AND SPECIAL SPECIFICATIONS.

ALL STRUCTURAL STEEL SHALL CONFORM TO ASTM-A441. FLANGE HOLES FOR FUSE BOLTS SHALL BE DRILLED.

ALL HIGH STRENGTH BOLTS, NUTS AND WASHERS SHALL CONFORM TO ASTM-A325.

TIGHTEN THE HIGH STRENGTH BOLTS IN THE BASE PLATE CONNECTION ONLY TO THE TORQUE SHOWN IN THE TABLE. DO NOT OVERTIGHTEN.

NOTCHED STEEL FUSE PLATES SHALL CONFORM TO THE REQUIREMENTS OF ASTM-A441.

ALL HOLES SHALL BE DRILLED. ALL PLATE CUTS SHALL PREFERABLY BE SAW CUTS, HOWEVER; FLAME CUTTING WILL BE PERMITTED PROVIDED ALL EDGES ARE GROUND. METAL PROJECTING BEYOND THE PLANE OF THE PLATE FACE WILL NOT BE TOLERATED.

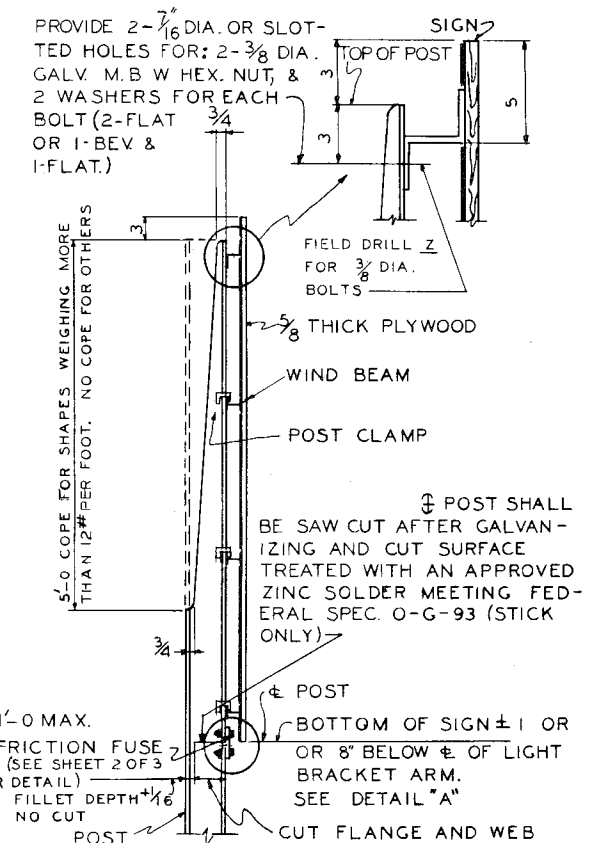
ALL BOLTS OTHER THAN HIGH STRENGTH BOLTS SHALL CONFORM TO ASTM-A307, CLASS A.

ALL STRUCTURAL STEEL, BOLTS, NUTS AND WASHERS SHALL BE GALVANIZED AS PER ASTM A-153

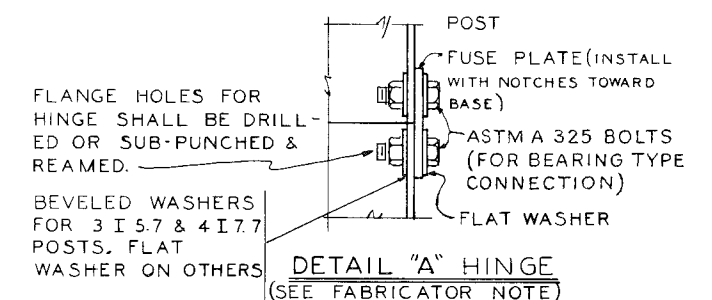
STEEL SHALL BE GALVANIZED AFTER FABRICATION EXCEPT AS NOTED.

FABRICATOR NOTE: ALL FRICTION FUSE BOLTS SHALL BE TIGHTENED IN THE SHOP FOLLOWING A METHOD APPROVED BY THE ENGINEER. TIGHTENING SHALL BE TO SUCH A DEGREE AS TO OBTAIN THE FOLLOWING RESIDUAL TENSION IN EACH BOLT:

BOLT SIZE	MIN. RESIDUAL TENSION
1/2 DIA.	12050 LBS.
5/8 DIA.	19200 LBS.
3/4 DIA.	28400 LBS.
7/8 DIA.	36050 LBS.



SIDE VIEW



Ground Mounted Sign Supports Breakaway Type

Post Selection Chart
Footing Depth Chart

Figure 2



United States Steel Corporation
Highway Construction Marketing

Figure 3

Ground Mounted Sign Supports Breakaway Type

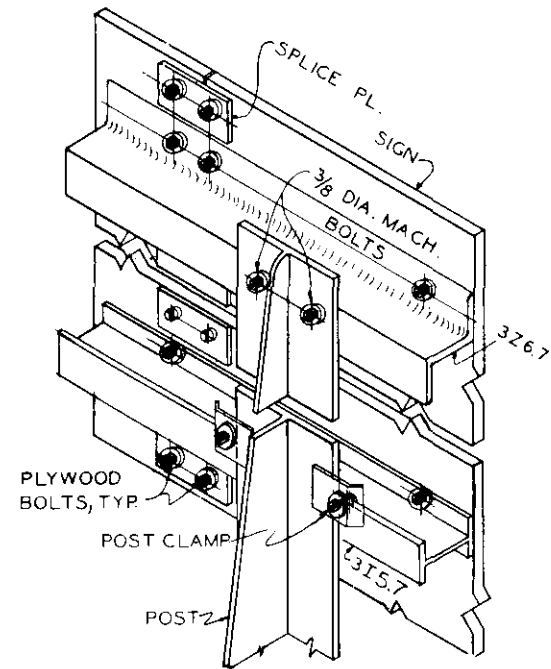
General Notes

Hinge Details

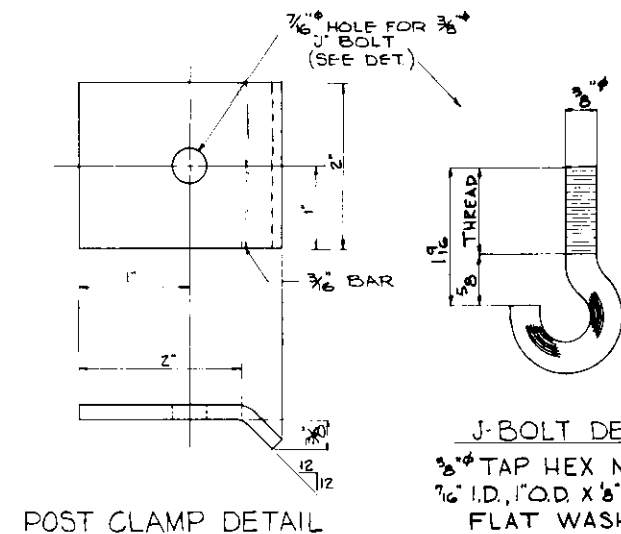
Wind Beam Tables

Revised November, 1967

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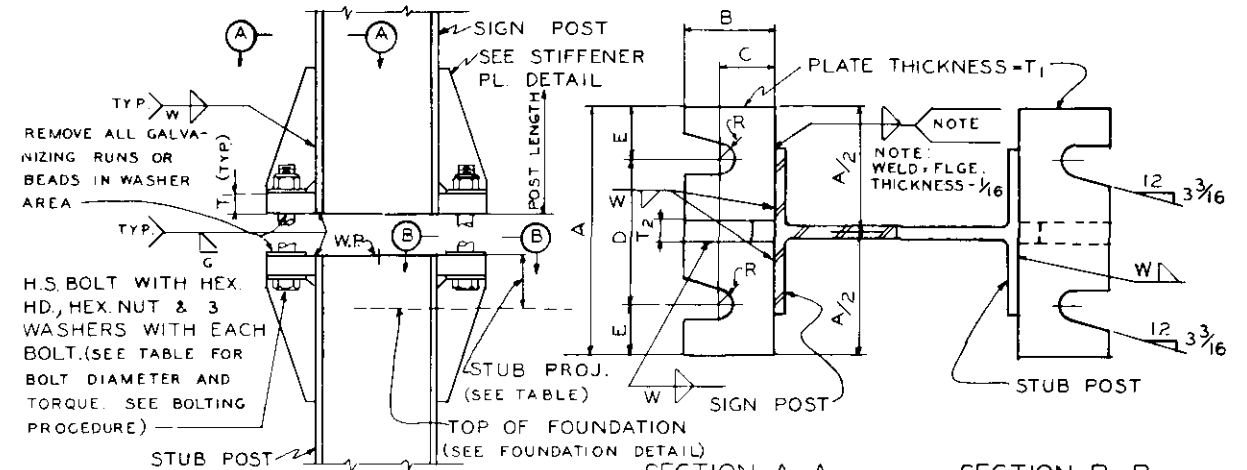


ISOMETRIC SHOWING
SIGN COMPONENTS



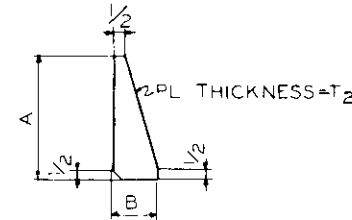
POST CLAMP DETAIL

J-BOLT DETAIL
3/8" TAP HEX NUT &
1/2" I.D. 1" O.D. X 8" THK.
FLAT WASHER

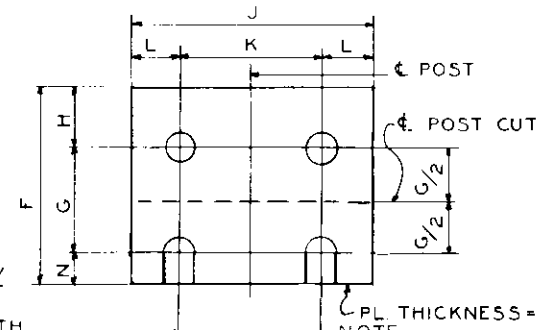


SIGN POST AND STUB POST
ELEVATION
(FOR WF AND W SHAPES)

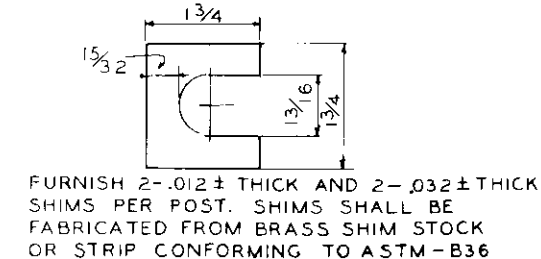
SECTION A-A
SECTION B-B
(SEE TABLE FOR DIMENSIONS)
SECTIONS SHOWN ARE FOR INSTALLATIONS ON RIGHT SHOULDER
AND IN GORE FOR INSTALLATIONS ON LEFT SHOULDER, PLATE
AND SLOT BEVELS ARE OPPOSITE HAND



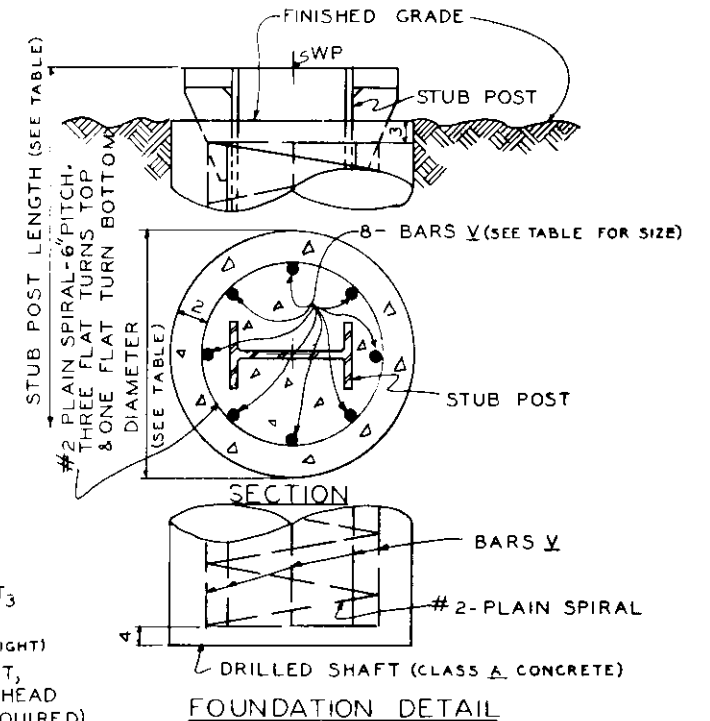
STIFFENER PLATE DETAIL
(SEE TABLE FOR DIMENSIONS)



FUSE PLATE DETAIL

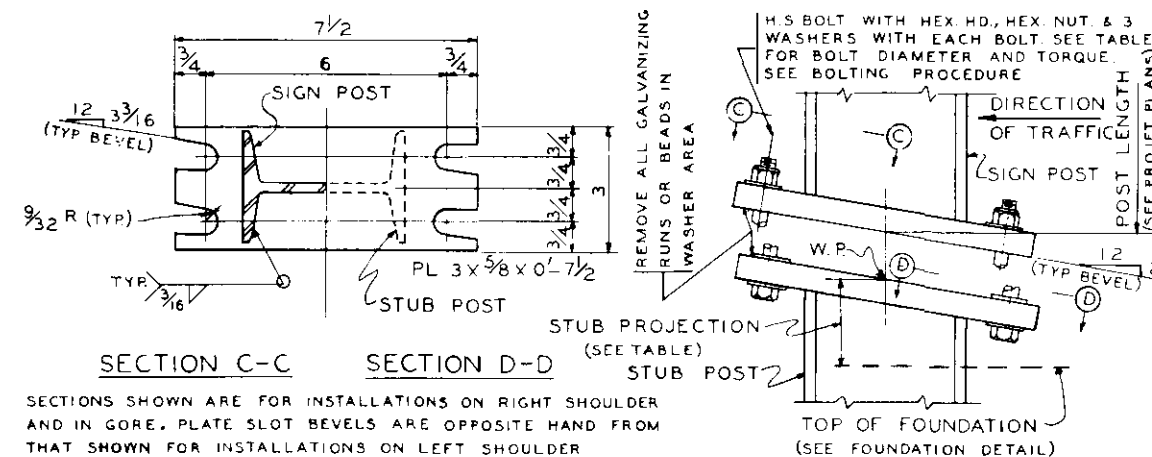


SHIM DETAIL



FOUNDATION DETAIL

BASE CONNECTION DATA TABLE											FUSE PLATE DATA TABLE											FOUNDATION DATA			
DIMENSION POST SIZE	BOLT SIZE & TORQUE	A	B	C	D	E	T ₁	T ₂	W	R	F	G	H	J	K	L	N	D ₁	T ₃	BOLT DIA.	WT. OF EACH FUSE PL.	STUB LENGTH	STUB PROJ.	DR. SHAFT DIA.	BAR SIZE
6B 8.5	5/8" x 2 3/4"										3 3/8	2	1 1/8	4	2 1/4	1/8	1/2	9/16	1/4	1/2	1.09 #	2'-0"	3	2'-0"	#5
6B 12	TORQUE	5	2	1 1/4	2 3/4	1 1/8	3/4	1/2	1/4	1 1/32	3 3/4	2	1 1/8	4	2 1/4	1/8	5/8	1 1/16	3/8	5/8	1.60 #	2'-0"	3	2'-0"	#5
6B 15.5	450" #										4 1/2	2 1/2	1 1/4	6	3 1/2	1 1/4	3/4	1 3/16	1/2	3/4	3.75 #	2'-6"	3	2'-0"	#6
8WF 17											4 1/2	2 1/2	1 1/4	5 1/4	2 3/4	1 1/4	3/4	1 3/16	1/2	3/4	3.27 #	2'-6"	3	2'-0"	#7
8WF 20	3/4" x 3 1/2"										4 7/8	2 1/2	1 1/2	5 1/4	2 3/4	1 1/4	7/8	1 5/16	1/2	3/4	3.93 #	3'-0"	2 1/2	2'-0"	#8
10WF 21	TORQUE	6	2 1/4	1 3/8	3 1/2	1 1/4	1	3/4	5/16	1 3/32	5 3/8	3	1 1/2	5 3/4	2 3/4	1 1/2	7/8	1 5/16	1/2	3/4	4.75 #	3'-0"	2 1/2	2'-0"	#9
10WF 25	750" #										5 3/8	3	1 1/2	5 3/4	2 3/4	1 1/2	7/8	1 5/16	1/2	3/4	4.79 #	3'-0"	2 1/2	2'-0"	#10
12WF 27											5 3/8	3	1 1/2	6 1/2	3 1/2	1 1/2	7/8	1 5/16	1/2	3/4	5.42 #	3'-0"	2 1/2	2'-0"	#11
3I 5.7	1/2" x 2"	SEE DETAIL									3 1/8	1 1/2	1 1/8	2 5/8	1 1/2	9/16	1/2	9/16	1/4	1/2	0.64 #	1'-6"	3 1/2	1'-6"	#5
4I 7.7	TORQUE 200"	SEE DETAIL									3 1/8	1 1/2	1 1/8	2 5/8	1 1/2	9/16	1/2	9/16	1/4	1/2	0.64 #	1'-6"	3 1/2	1'-6"	#5



SECTION C-C

SECTION D-D

SIGN POST AND STUB POST
ELEVATION

(FOR 4I 7.7 AND 3I 5.7 SHAPES)

PROCEDURE FOR ASSEMBLY OF BASE CONNECTION

1. ASSEMBLE POST TO STUB WITH BOLTS AND WITH ONE FLAT WASHER ON EACH BOLT BETWEEN PLATES.
2. SHIM AS REQUIRED TO PLUMB POST.
3. TIGHTEN ALL BOLTS THE MAXIMUM POSSIBLE WITH 12 TO 15 INCH WRENCH TO BED WASHERS AND SHIMS AND TO CLEAN BOLT THREADS THEN LOOSEN EACH BOLT IN TURN AND RETIGHTEN IN A SYSTEMATIC ORDER TO THE PRESCRIBED TORQUE. (SEE TABLE)
4. BURR THREADS AT JUNCTION WITH NUT USING A CENTER PUNCH TO PREVENT NUT LOOSENING.

Ground Mounted Sign Supports Breakaway Type

General Notes
Hinge Details
Wind Beam Tables

Figure 3



United States Steel Corporation
Highway Construction Marketing

Figure 4

Ground Mounted Sign Supports Breakaway Type

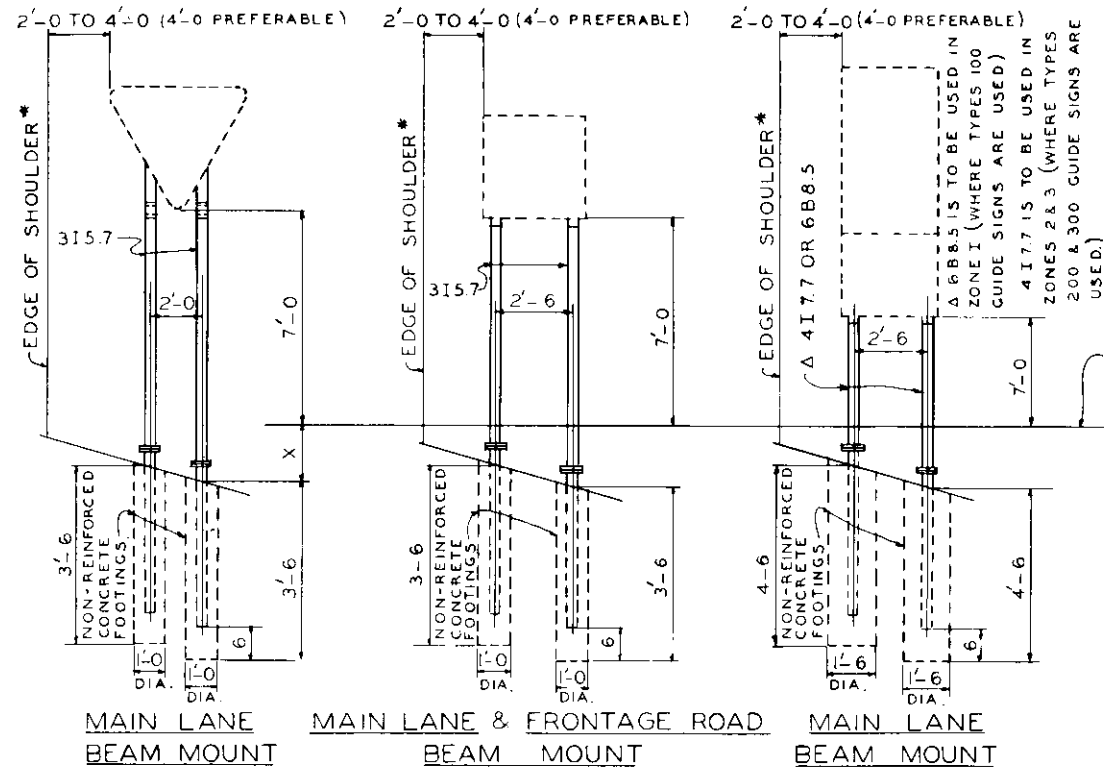
Sheet Signs—Twin Supports
Details & General Notes

Revised November, 1967

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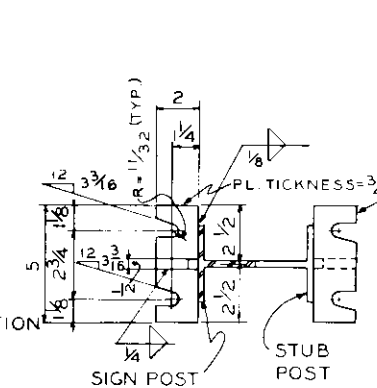
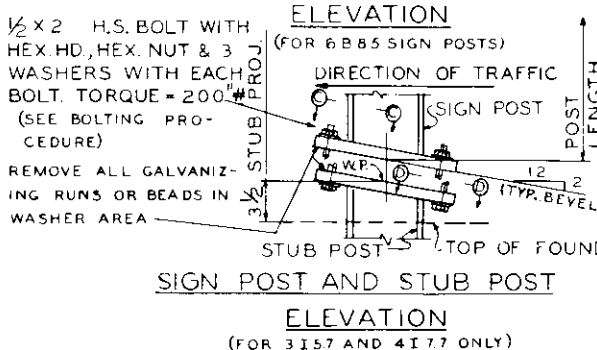
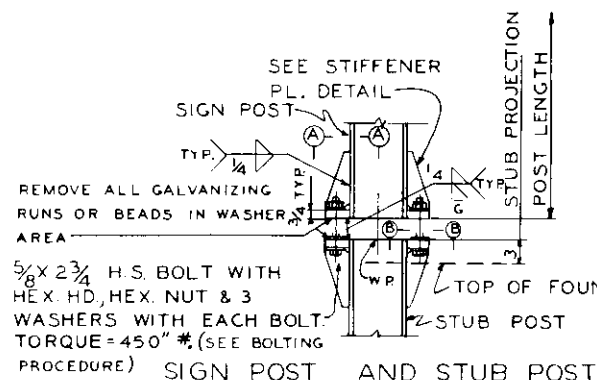
*OR FACE OF MOUNTABLE CURBS, THE CLEARANCE SHALL BE 2'-0" FOR THOSE SIGNS AT LOCATIONS WHERE THERE ARE UNMOUNTABLE CURBS.



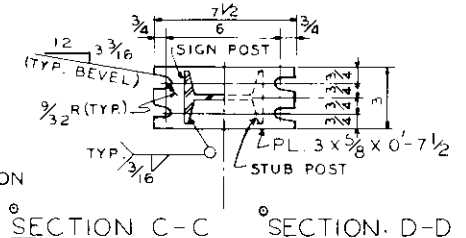
THIS TYPE MOUNT TO BE USED FOR TRIANGULAR-TYPE SIGNS HAVING AN AREA OF NOT MORE THAN 12 SQ. FT.

THIS TYPE MOUNT TO BE USED FOR SQUARE SIGNS HAVING AN AREA OF NOT MORE THAN 20 SQ. FT.

THIS TYPE MOUNT TO BE USED FOR COMBINATION OF SQUARE AND RECTANGULAR SIGNS HAVING A TOTAL AREA OF NOT MORE THAN 40 SQ. FT.



NOTE: SECTIONS SHOWN ARE FOR INSTALLATIONS ON RIGHT SHOULDER AND IN GORE. PLATE SLOT BEVELS ARE OPPOSITE HAND FROM THAT SHOWN FOR INSTALLATIONS ON LEFT SHOULDER.

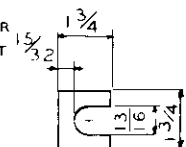


POST WEIGHT DATA	
POST SIZE	WEIGHT
6 B 8.5	135.7#
3 I 5.7	85.9#
4 I 7.7	119.9#

*LAST FIGURES = POST WT. PER FT

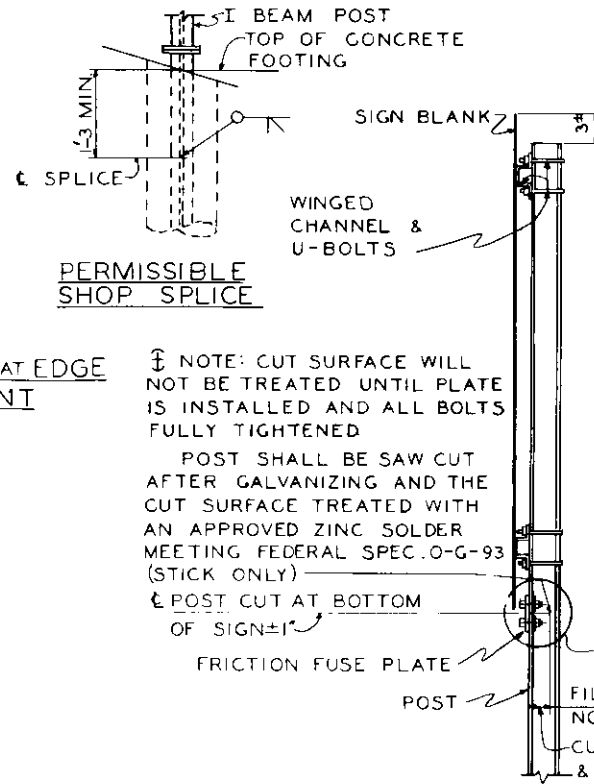
WEIGHT DATA IS THE WEIGHT OF ITEMS SHOWN FOR ONE POST - (INCLUDES 10' OF POST LENGTH, POST FOUNDATION STUB, RELATED BASE CONNECTION PLATES AND STIFFENERS, FRICTION FUSE PLATE AND ALL HIGH STRENGTH BOLTS, NUTS AND WASHERS)

POST WEIGHT DATA



FURNISH 2.0012 ± THICK AND 2.0032 ± THICK SHIMS PER POST. SHIMS SHALL BE FABRICATED FROM BRASS SHIM STOCK OR STRIP CONFORMING TO ASTM-B 36.

SHIM DETAIL



STIFFENER PL. DETAIL

GENERAL NOTES:

DESIGN CONFORMS WITH AASHTO SPECIFICATIONS FOR THE DESIGN AND CONSTRUCTION OF STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS.

MATERIALS AND FABRICATION SHALL CONFORM TO THE REQUIREMENTS OF APPLICABLE SPECIFYING AGENCIES

ALL HIGH STRENGTH BOLTS, NUTS, AND WASHERS SHALL CONFORM TO ASTM-A325.

NOTCHED STEEL FUSE PLATES SHALL CONFORM TO THE REQUIREMENTS OF ASTM-A441. ALL HOLES SHALL BE DRILLED. ALL PLATE CUTS SHALL PREFERABLY BE SAW CUTS, HOWEVER; FLAME CUTTING WILL BE PERMITTED PROVIDED ALL EDGES ARE GROUND. METAL PROJECTING BEYOND THE PLANE OF THE PLATE FACE WILL NOT BE TOLERATED.

TIGHTEN THE HIGH STRENGTH BOLTS IN THE BASE CONNECTION ONLY TO THE TORQUE SHOWN. DO NOT OVERTIGHTEN.

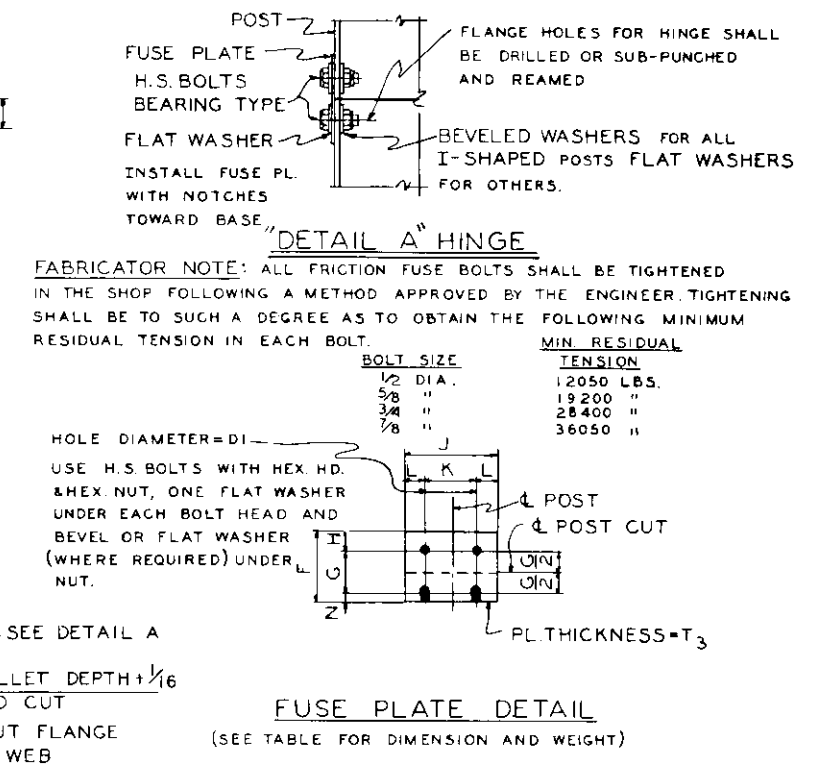
ALL BOLTS OTHER THAN H.S. BOLTS SHALL CONFORM TO ASTM-A307, CLASS A.

ALL STRUCTURAL STEEL, BOLTS, NUTS, & WASHERS SHALL BE GALVANIZED AS PER ASTM A153

STRUCTURAL STEEL TO BE GALV. AFTER FABRICATION EXCEPT AS NOTED.

WHERE SOLID ROCK IS ENCOUNTERED, FOOTING SHALL BE PLAN DEPTH OR EXTEND TWO FEET MIN. INTO THE ROCK.

STRUCTURAL STEEL SHALL CONFORM TO REQUIREMENTS ASTM-A441 GALVANIZING ALL POSTS TO BE PROJ. A MIN. 6" ABOVE CONCRETE.



DIMENSION		FUSE PLATE DATA TABLE										BOLT DIA.	WT. OF EACH FUSE PL.
		F	G	H	J	K	L	N					
POST SIZE													
6 B 8.5	3 3/8	2	1 1/8	4	2 1/4	1 1/8	1/2					9/16	1.09 #
3 I 5.7	3 1/8	1 1/2	1 1/8	2 5/8	1 1/2	9/16	1/2					9/16	0.64 #
4 I 7.7	3 1/8	1 1/2	1 1/8	2 5/8	1 1/2	9/16	1/2					9/16	0.64 #

PROCEDURE FOR ASSEMBLY OF BASE CONNECTION

1. ASSEMBLE POST TO STUB WITH BOLTS AND WITH ONE FLAT WASHER ON EACH BOLT BETWEEN PLATES.

2. SHIM AS REQUIRED TO PLUMB POST.

3. TIGHTEN ALL BOLTS THE MAXIMUM POSSIBLE WITH 12" TO 15" WRENCH TO BED WASHERS AND SHIMS AND TO CLEAN BOLT THREADS THE LOOSEN EACH BOLT IN TURN AND RETIGHTEN BOLTS IN A SYSTEMATIC ORDER TO THE PRESCRIBED TORQUE. (SEE TABLE)

4. BURR THREADS AT JUNCTION WITH NUT USING A CENTER PUNCH TO PREVENT NUT LOOSENING.

Ground Mounted Sign Supports Breakaway Type

Sheet Signs—Twin Supports
Details & General Notes

Figure 4



United States Steel Corporation
Highway Construction Marketing

Figure 5

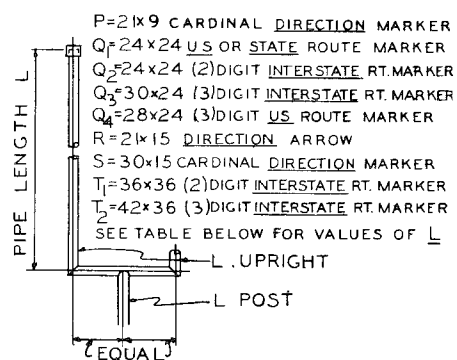
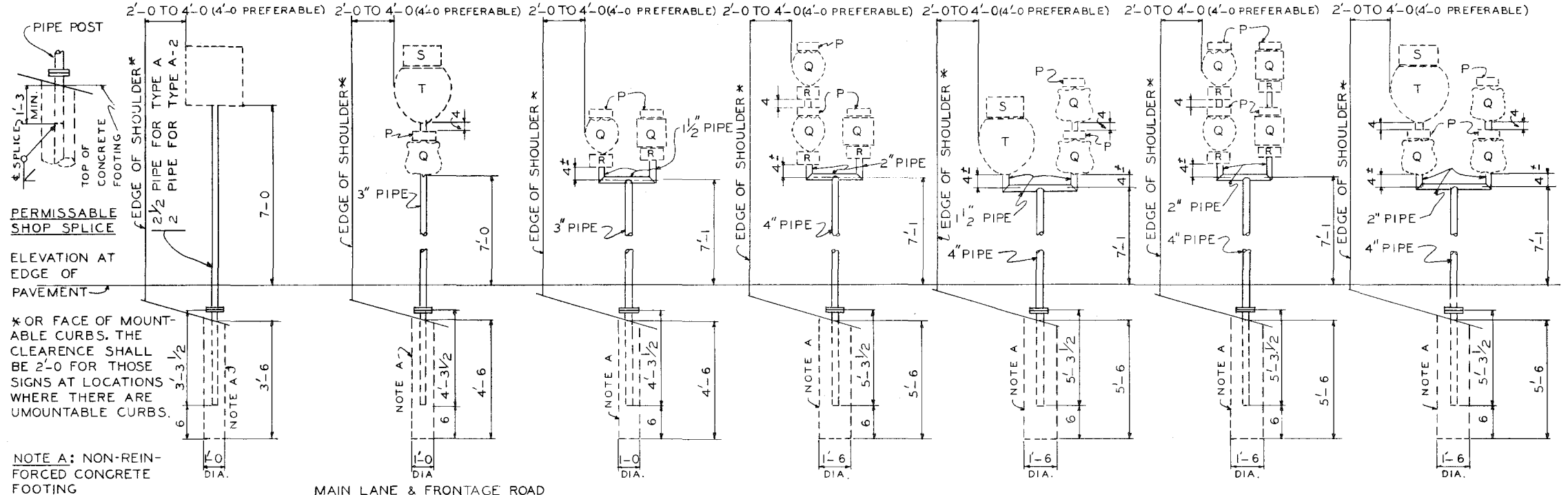
Ground Mounted Sign Supports Breakaway Type

Sheet Signs—Single Support

Post Details

Foundation Data

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MARKER COMBINATIONS "L"

2P+2Q+2R+7	8'-7
P+2Q+2R+7	7'-10
2Q+2R+7	7'-1
Q+R+7	3'-6
S+T+3	4'-6
P+Q+R+3	4'-3
P+Q+S+T+7	7'-7

DIMENSION S
S = 1'-4 1/2 FOR Q1 OR Q2
S = 1'-7 1/2 FOR Q3
S = 1'-6 1/2 FOR Q4
S = 1'-10 1/2 FOR T1
S = 2'-1 1/2 FOR T2
DIMENSION S EQUALS THE DISTANCE FROM THE C OF UPRIGHT TO THE C OF CLEARANCE BETWEEN SIGN GROUP.

TYPE OF MOUNT	PIPE A (DIA.)	PIPE B (DIA.)	FOOTING DIA.	FOOTING DEPTH
D-1-1	3	2	1'-0	4'-6
D-1-2	2 1/2	1 1/2	1'-0	4'-6
D-1-3	2	1 1/2	1'-0	4'-6
D-2-1	4	3	1'-6	5'-6
D-2-2	3 1/2	2 1/2	1'-6	4'-6
D-2-3	3	2	1'-6	4'-6
D-3-1	4	3	1'-6	5'-6
D-3-2	3 1/2	2 1/2	1'-6	4'-6
D-3-3	3	2	1'-6	4'-6
D-4-1	5	3	1'-6	5'-6
D-4-2	4	3	1'-6	5'-6
D-4-3	3 1/2	2 1/2	1'-6	4'-6
D-5-1	3 1/2	2 1/2	1'-6	4'-6
D-5-2	3	2	1'-6	4'-6
D-5-3	2 1/2	1 1/2	1'-6	4'-6
D-6-1	4	3	1'-6	5'-6
D-6-2	3 1/2	2 1/2	1'-6	4'-6
D-6-3	3	2	1'-6	4'-6



NOTE:
DI-A SIGNS WITH A VERTICAL DIMENSION IN COMPLIANCE AASHTO SPECIFICATION FOR DESIGN OF TRAFFIC SIGNS MAY BE USED IN LIEU OF DI-A SIGNS HAVING A VERTICAL DIMENSION OF 12" AS SHOWN ON THE PLANS, WHERE A SIGN MANUFACTURER HAS SCREENS ALREADY MADE FOR THE 11 1/2" VERTICAL DIMENSION BUT NOT FOR THE 12" VERTICAL DIMENSION. MEASUREMENT, HOWEVER, SHALL BE MADE ON THE BASIS OF THE DIMENSIONS FOR THE SIGNS AS ACTUALLY ERECTED.

Ground Mounted Sign Supports Breakaway Type
Sheet Signs—Single Support
Post Details
Foundation Data
Figure 5

Figure 6

Ground Mounted Sign Supports Breakaway Type

Sheet Signs—Single Support

General Notes

Base Connection Data

Notice

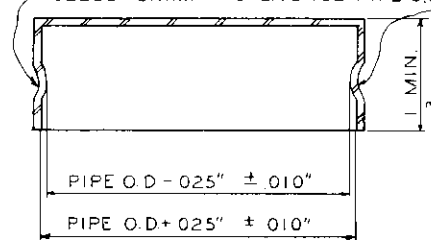
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NOTES:

FRICTION CAPS—CAPS MAY BE MANUFACTURED FROM EITHER HOT ROLLED OR COLD ROLLED STEEL SHEETS. FOR PIPE SIZES 3 1/2" AND SMALLER THE MINIMUM SHEET METAL THICKNESS SHALL BE 24 GAUGE.

THE RIM EDGES SHALL BE REASONABLY STRAIGHT AND SMOOTH. CAPS SHALL BE SIZED AND FORMED IN SUCH A MANNER AS TO PRODUCE A DRIVE-ON FRICTION FIT AND HAVE NO TENDENCY TO ROCK WHEN SEATED ON THE PIPE. THE DEPTH SHALL BE SUFFICIENT TO GIVE POSITIVE PROTECTION AGAINST THE ENTRANCE OF RAIN-WATER. THEY SHALL BE FREE OF SHARP CREASES OR INDENTATIONS AND SHOW NO EVIDENCE OF METAL FAILURE.

CAPS SHALL HAVE AN ELECTRODEPOSITED COATING OF ZINC IN ACCORDANCE WITH THE REQUIREMENTS OF ASTM SPEC. A164, TYPE GS.



FRICTION CAP
DETAIL

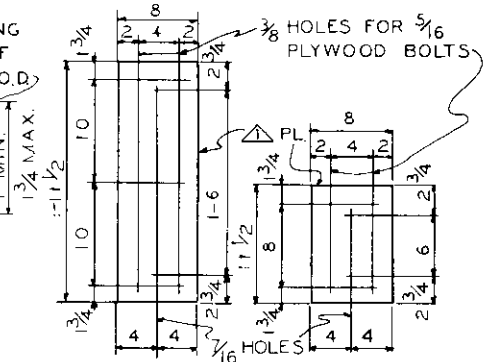
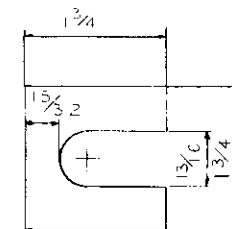


PLATE (ASTM-A36) STEEL 1/8" THICK
SPLICE PLATES

PROCEDURE FOR ASSEMBLY OF BASE CONNECTION

1. ASSEMBLE POST TO STUB WITH BOLTS AND ONE FLAT WASHER ON EACH BOLT BETWEEN PLATES
2. SHIM AS REQUIRE TO PLUMB POST.
3. TIGHTEN ALL BOLTS THE MAXIMUM POSSIBLE WITH 12" TO 15" WRENCH TO BED WASHERS AND SHIMS AND TO CLAN BOLT THREADS THEN LOOSEN.
4. RETIGHTEN BOLT IN A SYSTEMATIC ORDER TO THE PRESCRIBED TORQUE (SEE TABLE)
5. LOOSEN EACH BOLT AND RETIGHTEN TO THE PRESCRIBED TORQUE IN THE SAME ORDER AS INITIAL RETIGHTENING.
6. BURR THREADS AT JUNCTION WITH NUT USING A CENTER PUNCH TO PREVENT NUT LOOSENING.



FURNISH 2 0.012"± THICK AND 2 0.032"± THICK SHIMS PER POST SHIMS SHALL BE FABRICATED FROM BRASS SHIM STOCK OR STRIP CONFORMING TO ASTM-B36

SHIM DETAIL

GENERAL NOTES:

DESIGN CONFORMS WITH A.A.S.H.O SPECIFICATIONS FOR THE DESIGN AND CONSTRUCTION OF STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS. MATERIAL AND FABRICATION SHALL CONFORM TO THE REQUIREMENTS OF THE ABOVE SPECIFICATION.

ALL STRUCTURAL STEEL, BOLTS AND WASHERS SHALL BE GALVANIZED AS PER ASTM A 153

GALVANIZING OF ALL POSTS SHALL BE THE PROJECTION ABOVE CONCRETE PLUS 6 MIN.

ALL HIGH STRENGTH BOLTS AND WASHERS SHALL CONFORM TO ASTM-A325. ALL HIGH STRENGTH NUTS SHALL BE OF SUCH CAPACITY AS TO DEVELOP THE BOLT STRENGTH.

TIGHTEN THE HIGH STRENGTH BOLTS IN THE BASE CONNECTION ONLY TO THE TORQUE SHOWN, **DO NOT OVERTIGHTEN.**

ALL BOLTS OTHER THAN HIGH STRENGTH BOLTS SHALL CONFORM TO ASTM-A307, CLASS A

SEE FIGURE 5 FOR SIGN FASTENING DETAILS.

WHERE SOLID ROCK IS ENCOUNTERED, FOOTING SHALL BE PLAN DEPTH OR EXTEND A MINIMUM OF TWO FEET INTO THE ROCK.

Ground Mounted Sign Supports Breakaway Type

Sheet Signs—Single Support

General Notes

Base Connection Data

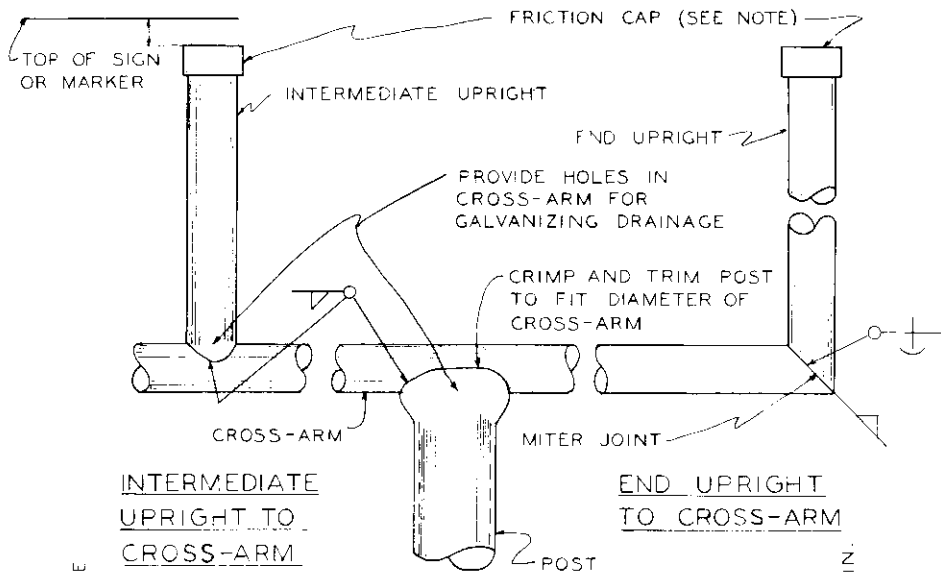
Figure 6

NOTE:

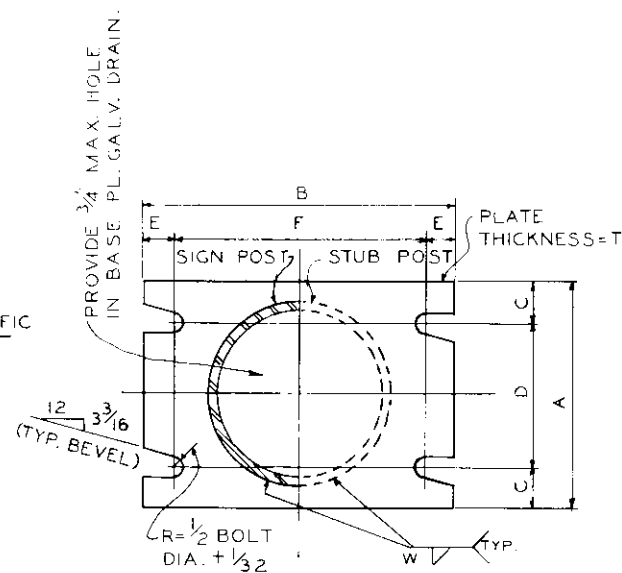
THE CONTRACTOR, AT HIS OPTION, MAY FURNISH STANDARD WEIGHT PIPE CONFORMING TO ASTM SPECIFICATION A-53, OR STANDARD WEIGHT PIPE CONFORMING TO API5L, GRADE B PIPE MADE FROM MATERIALS CONFORMING TO THE API5L SPECIFICATIONS EXCEPT THAT THE OFFICIAL API MONOGRAM IDENTIFICATION IS NOT REQUIRED.

COMBINATIONS OF PIPE MATERIAL MEETING DIFFERENT SPECIFICATIONS MAY BE USED IN THE SAME PROJECT.

THE PIPE MAY BE WELDED OR SEAMLESS. HYDROSTATIC TESTS ARE NOT REQUIRED.



WELDED PIPE MOUNT
DETAILS



SECTION A-A SECTION B-B
(SEE TABLE FOR DIMENSIONS)

SECTIONS SHOWN ARE FOR INSTALLATIONS ON RIGHT SHOULDER AND IN GORE. PLATE SLOT BEVELS ARE OPPOSITE HAND FROM THAT SHOWN FOR INSTALLATIONS ON LEFT SHOULDER.

BASE CONNECTION DATA TABLE *									
DIMENSION NOM. PIPE SIZE	BOLT SIZE & TORQUE	A	B	C	D	E	F	T	W
2	1/2 X 2 TORQUE 200	4 1/2	6 1/4	1	2 1/2	3/4	4 3/4	1/2	1 1/4
2 1/2	1/2 X 2 1/2 TORQUE 200	4 1/2	6 1/4	1	2 1/2	3/4	4 3/4	1/2	1 1/4
3	1/2 X 2 1/2 TORQUE 200	5 1/2	7 3/4	1	3 1/2	3/4	5 1/2	3/4	1 1/4
3 1/2	1/2 X 2 1/2 TORQUE 200	5 1/2	7 3/4	1	3 1/2	3/4	5 1/2	3/4	1 1/4
4	5/8 X 2 3/4 TORQUE 450	6 1/2	9 3/4	1 1/4	4	7/8	8	1	1 1/4
5	5/8 X 2 3/4 TORQUE 450	6 1/2	9 3/4	1 1/4	4	7/8	8	1	1 1/4

*PLATES FOR BASE CONNECTION SHALL CONFORM WITH THE REQUIREMENTS OF ASTM-A36.

H.S. BOLT WITH HEX. HD., HEX. NUT & 3 WASHERS WITH EACH BOLT. SEE TABLE FOR BOLT DIAMETER AND TORQUE. SEE BOLTING PROCEDURE.

REMOVE ALL GALVANIZING RUNS OR BEADS IN WASHER AREA

SIGN POST AND STUB POST
ELEVATION

Break-away sign supports enhance highway safety

**CHARLES J. KEESE, M.D. SHELBY, F. ASCE,
ROBERT M. OLSON, M. ASCE, NEILON J. ROWAN,
and THOMAS C. EDWARDS, A.M. ASCE,**
Executive Officer, Research Engineer, and
Assistant Research Engineers, Texas Transportation Institute,
Texas A&M University, College Station, Tex.

In 37 weeks, 27 experimental safe roadside signs have been hit along the Interstate Highway System in Texas without injury or loss of life to drivers or passengers. In 1963, 4 people were killed and 36 injured from hitting Interstate roadside signs in Texas; only a few Interstate signs had been installed at that time. In 1964, however, 14 people were killed and 85 injured from hitting roadside signs along the Interstate in Texas. In 1965 the statistics were 13 killed and 81 injured.

The potential hazard of roadside signs was recognized, and a cooperative research project was initiated by the Texas Transportation Institute (TTI) and Texas Highway Department (THD) in cooperation with the U.S. Bureau of Public Roads. The first phase of this project was designed to find an acceptable design that might permit modifications of existing sign supports.

From the cooperative efforts of the TTI researchers and personnel of the highway department, two satisfactory designs evolved for large roadside signs. One was based on a reduction of mass and rigidity through the use of a tubular A-frame design, and the other was based on a reduction of fixity by use of a slip joint and a hinge joint for the unbraced post supports commonly used. The latter design was selected for refinement since it could easily be applied as a remedial measure to the standard unbraced post supports used in Texas.

Even though gore signs are generally smaller than roadside signs, accident records clearly indicated that the conventional gore signs, such as the EXIT sign, constitute a definite hazard. Tests on smaller signs with safety features of a slip base and hinge joint showed that the collision behavior of the smaller signs was acceptable at high speeds; however when both legs were struck at slow speeds, the sign struck the top of the automobile. This was corrected by inclining the slip base to induce an uplifting force on the supports, which would alter the trajectory of the sign following a collision.

The Bureau of Public Roads approved the installation of some of these supports on an experimental basis. They received immediate—but unintentional—field testing from motorists, and functioned as expected. The posts have been hit at various angles and even broadside in collisions. Instances involving collisions with sign supports have been much more frequent than was anticipated and have averaged about three per month. The research has already paid for itself many times in lives saved and injuries prevented.

The Texas design was an acceptable “quick answer” that allowed only limited variation to be made with any reliable confidence. For this reason a project of larger scope was undertaken in September 1965. It was planned to search for the best design through basic study of the fundamental principles involved, with the

objective of providing design criteria that would permit designers to apply safe impact features to a wide variety of design configurations and structural materials.

This project was initially sponsored by twelve state highway departments (Ala., Calif., Kansas, La., Minn., Miss., Nebr., N.D., Okla., S.D., Tenn., and Texas) in cooperation with the Bureau of Public Roads. It included three areas of related research.

- Development of safe impact design criteria for roadside signs
- Feasibility studies on impact attenuation systems to stop vehicles within tolerable limits of deceleration
- Development of wind load characteristics and investigation of designs of sign backgrounds with reduced wind load characteristics

This project is not completed but has shown that research involving the joint participation by engineers, or others who are involved in day-to-day engineering application, and researchers of various disciplines can produce significant engineering advancements. The philosophy of the TTI-THD cooperative research program, which involves the development of theory, verified through experimentation and practical application, has been highly successful.

Development

Because of substantial mileage of the Interstate system in Texas under construction or scheduled there was urgency to develop basic design concepts that could be incorporated into an experimental design. These concepts were subjected to full-scale crash tests. After each test or series of tests was conducted, design modifications were made to improve the impact behavior of the supports. The new design concepts were incorporated in the design standards for the state as soon as they had been proven by test and evaluation by the Texas Transportation Institute.

Facilities and procedures were developed to create controlled vehicle collisions with sign supports and to obtain time dependent data pertaining to the collision incident. To launch the crash vehicles into sign supports, a reverse-tow procedure was used. Data were obtained by photographing the incident using a camera operating at 1,000 frames per second. In later tests electronic instrumentation, including strain gauges and accelerometers, was employed in data acquisition.

Concurrent with development of test facilities, consideration was being given to the fundamentals of the problem. Accident reports and photographs of damage due to collisions with large sign supports led to a hypothesis that three primary characteristics of the sign support contribute substantially to the severity of the collision: (1) the mass, (2) the structural rigidity or stiffness, and (3) the condition of fixity at the base of the sign support.

Braced-leg design

In the initial phase of this project, research efforts were devoted to development, testing and evaluation of a braced-leg structure (Figure D-8a) fabricated from high-strength steel tubing, which reduced the mass and rigidity of the support. Crash tests involving the structure showed that the colliding vehicle ripped out the members of the support structure with little resistance or damage to the vehicle.

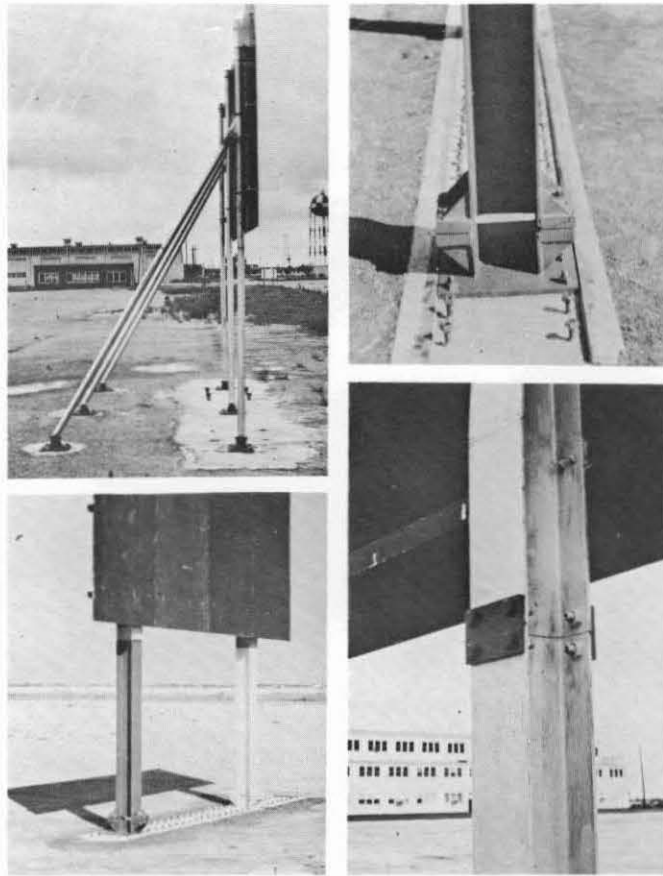


Figure D-8. Developmental characteristics of sign supports: (a) braced-leg structure (b) unbraced post support (c) closeup of slip base and (d) fracture joint.

Unbraced support

Later research efforts were devoted to the development of an unbraced post support (Figure D-8b), retaining the simplicity in design and ease of maintenance afforded by the unbraced post type of support. To improve the safety of this design, it was necessary first to introduce a break-away base (Figure D-8c) which would slip free when subjected to horizontal forces due to impact, but would resist the overturning moment due to wind loads. Further, it was necessary to develop a release mechanism in the post to prevent a secondary collision between the base of the post and the windshield of the colliding vehicle. The first attempt in this respect was a fracture joint (Figure D-8d) which permitted complete removal of the lower portion of the post in a collision. This modification in the design changed the phenomenon of the secondary collision but did not reduce the potential hazard to the occupants of a colliding vehicle. As a solution to this problem, the fracture joint was replaced by a hinge joint formed by cutting the front flange and web of the post, leaving the back flange intact to serve as a hinge. The front flange was reconnected with a fuse plate. Crash tests on this design showed that the cast-iron fuse plate failed after the base slipped and permitted the lower section of the post to swing up, clearing the colliding vehicle (Figure D-9).

Experimental design of small signs

Experiments were conducted also with smaller signs. The first phase of this research dealt with roadside signs in which the posts were so widely spaced that it was impossible for a vehicle to collide head-on with more than one support. However there are numerous signs in use on access-controlled facilities which are small enough that a vehicle can collide with both supports. The EXIT sign used in the gore of exit ramps is an example.

To reduce the hazard of collisions with smaller signs, the same safety features (the slip base and hinge joint) were proposed, and a program of testing and evaluation was conducted. It was found that the collision dynamics of the smaller signs were satisfactory at high speeds, but when both legs were struck at low speeds the sign struck the top of the automobile. This objectionable characteristic was corrected by inclining the break-away base 10 deg to induce an uplifting force on the supports, which would alter the trajectory of the sign following a collision (see Figure D-10).



Figure D-9. Effectiveness of the break-away post is shown by crash test, above, and by roadside sign after test by an obliging—but probably disconcerted—motorist.

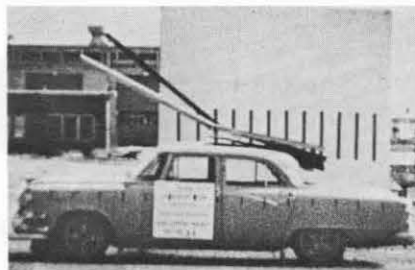


Figure D-10. At slow speed, break-away sign (at top) struck the top of the car. By inclining the break-away base 10 deg, an uplifting force threw the sign over the car.

Wood and aluminum supports

The highway department of one state has installed experimental wood posts as supports for EXIT signs on an Interstate route. These pentatreated pine posts are provided with notches that create stress risers and permit the post to break upon impact. An alternative design was proposed by TTI personnel which employed a slotted hole through the post near the base to provide a shear plane. Results of full-scale crash tests are shown in Figure D-11.

Full-scale crash tests have been conducted on aluminum posts and A-frames, and the behavior of aluminum posts is promising. Details of welds and connections in general are being given careful consideration.

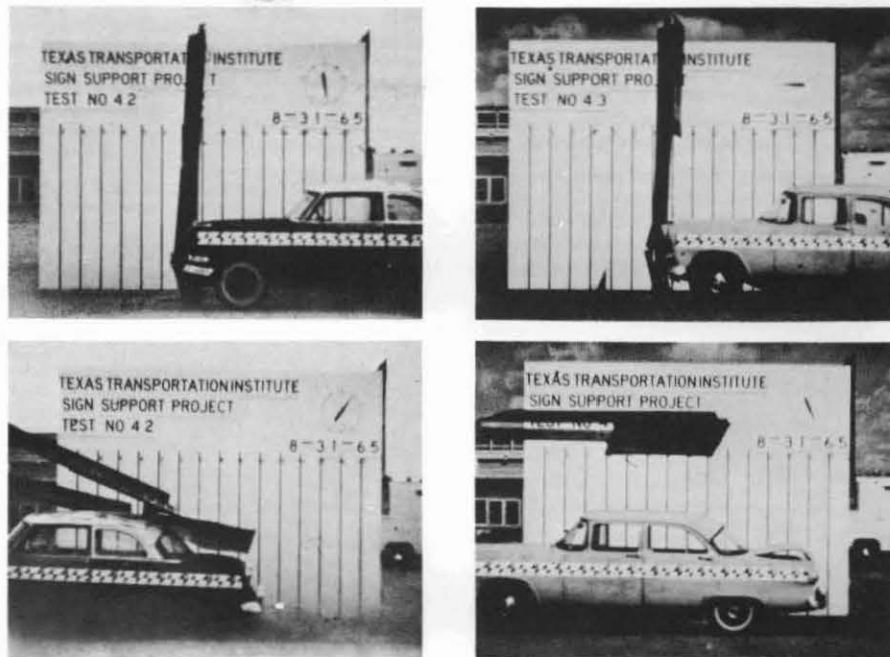


Figure D-11. Test 42 was on pentatreated pine post supports that were notched to create “stress risers.” A slotted design to provide a “shear plane,” right photos, seems to be safer.

Mathematical simulation

A program of full-scale crash testing is a slow, laborious and expensive research and development procedure. Such testing provides qualitative and quantitative information concerning the phenomenological behavior of the structural system. To augment this testing program and to provide a mathematical means of analysis, a mathematical simulation has been programmed for the IBM 7094 computer at the Texas A&M University Campus. This mathematical model expresses quantitatively the dynamic behavior of a sign support subjected to impact by a vehicle. To obtain a working model in as short a time as possible, a thorough study was made to determine what variables were most pertinent to the behavior of the actual sign. Detailed observations of high-speed films revealed that the portion of the post above the hinge, and the attached sign, were rigid against rotation and translation for the initial period of response.

This observation led to the assumptions made in determining the mathematical model used in the study. This idealized model is shown in Figure D-12. It consists of a rigid post connected by a plastic hinge to a rigid support at the top, and by a slip plane at the base. To simulate the action of the real sign post, the slip base is assumed to offer a constant resistance to slipping until maximum slip occurs.

The plastic hinge is assumed to behave in an elastic, perfectly plastic, manner until the cast-iron fuse plate ruptures. Provisions have been made in the mathematical model to express the quantitative behavior of the mechanical fuse. The method used in the solution of this problem is a modified constant velocity technique which utilizes a forward step integration in time of the finite difference equations of motion.

The model has been adequately correlated with data from full-scale field tests. It has proved very valuable in the development of the break-away post now installed on Texas Interstate highways. Its use has enabled the designers to choose the parameters of the break-away features so that their operation would be insured under a wide range of vehicle collisions.

Research and observation

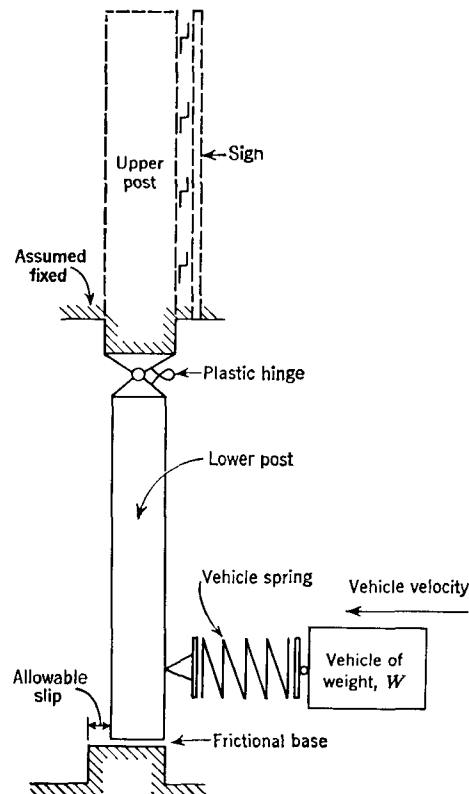
Throughout all of the research to provide an immediate solution to the problem of safeguarding lives and reducing damage in vehicular collisions with highway sign posts, the interdisciplinary team approach was employed. Specialists of many fields cooperated in achieving the objectives. After preliminary testing, a cantilever design involving the WF-beam was selected for development and application. As findings were amassed, slip-base and mechanically fused saw-cut modifications were made to existing I-beam sign posts of the Interstate system in Texas. Almost immediately, as these were involved in accidents, the base slipped free and the beam bent upward; virtually no injuries were experienced by motorists and their vehicles suffered little damage. The cantilever post, giving ease of maintenance and simplicity in design, was a natural design for safety development.

A computer technique for simulating the conditions of a vehicle striking a sign support paralleled design work and crash testing. With proper reference data the simulation technique can be used to predict the structural behavior of the sign support and the vehicle at successive time intervals during a simulated collision. It is anticipated that the technique will be useful in evaluating various design configurations of sign support structures in lieu of evaluation by full-scale crash testing.

Further research involving laboratory and full-scale testing of break-away components is contemplated. Data from such hardware testing are needed for input for the computerized mathematical simulation technique for analysis of all types of signs. Consideration is also being given in research to the total roadside environment, such as location of roadways with emphasis on objects that cannot be moved and with regard to the geometrical configuration of roadways.

References and some additional data may be obtained from the Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843.

Figure D-12. Idealized model of sign support post consists of rigid lower post with slip plane at its base and a plastic hinge connection to a rigid support at its top. The vehicle is represented as a single mass and spring.



Safer Roadside Structures

Part E

Break-Away Lighting Supports

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Break-away lighting supports

Luminaire supports must often be located in close proximity to the roadway. Involuntary departure of vehicles from the roadway often results in serious collisions with luminaire supports. However, this problem may be alleviated considerably through the application of recent research results and good engineering judgment. Research on the illumination system design indicates that luminaires may be placed over the edge of the shoulder with equal or greater effectiveness than placing them over the roadway, thus permitting a greater lateral positioning of the luminaire support. This research has also shown that higher mounting heights and higher intensity light sources permit substantially greater longitudinal spacing of luminaires and supports, thus reducing the number of supports required in the illumination system.

The length of the mast arm supporting the luminaire may be increased to permit a greater lateral positioning of the luminaire support from the roadway. However, good engineering judgment should be exercised in the “trade off” of greater clear distance for increased strength in the support to accommodate the longer mast arm. It is logical that the probability of collision will be reduced by increasing the lateral distance to the support but it is equally logical that collisions will not be *eliminated*. The net results will probably be fewer collisions of a more serious nature! Therefore, it is imperative that break-away features be incorporated in the design of all luminaire supports that may be struck by a vehicle.

Considerable research effort has been devoted to the development of break-away devices for luminaire supports. Although most of these devices were developed for use in new lighting systems, many are immediately applicable in modification of existing fixed-base supports.

Several agencies have pursued research and development of break-away devices for luminaire supports. These include: The Road Research Laboratory in England, the General Motors Proving Grounds, the Millerbernd Manufacturing Company, the Minnesota and California Highway Departments and the Texas Transportation Institute in cooperation with the Texas Highway Department and the Bureau of Public Roads.

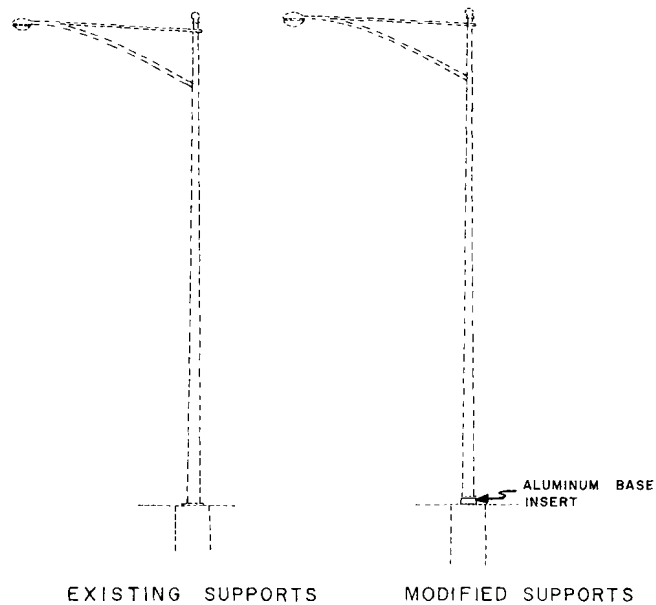
The basic concept of the break-away luminaire support is similar to that of the break-away sign support, i.e., to provide an adequate moment connection at the base to resist wind induced loads (in luminaire supports high moment loading conditions are accompanied by low shear forces); and conversely to exhibit weak shear resistance to loads applied by a colliding vehicle (primarily high shear, low moment loading conditions). Break-away bases for luminaire supports can be classified as indicated in the following paragraphs.

Frangible bases

Frangible bases, for steel supports are fabricated from brittle materials which are designed to fracture when subjected to impact loads. Materials used for frangible bases must have low energy absorption characteristics to insure rapid base release upon impact. In present practice, bases of this type are made of cast aluminum, although there are indications that cast iron could be used as an alternate material.

In practice or application, the frangible base takes several different forms. A short cast insert, approximately six inches in height has been developed for remedial design where flange mounted supports now exist. A typical example is shown in Figure E-1. The cast inserts are installed between the flange of the support or a steel transformer base and the foundation. In such a modification, the existing support is removed, the cast insert is installed on the existing foundation using the existing anchor bolts, and the support is bolted to the insert.

Another and preferable form of the frangible base is normally referred to as a “transformer base,” a cast aluminum box approximately 20 inches in height which may have either vertical or tapered sides. These may be used in new construction or in replacement of steel transformer bases. Also, they may be placed under existing flange-mounted supports. These are bases with vertical sides to provide the same bolting configuration top and bottom, as illustrated in Figure E-2.



NOTE:
THE BASE AND ACCESS COVER PLATE SHALL BE ALUMINUM
CAST IN A PERMANENT MOLD COMPLYING WITH ASTM
DESIGNATION: B-108, ALLOY SG70A HEAT TREATED
TO T-4 OR T-6 TEMPER.

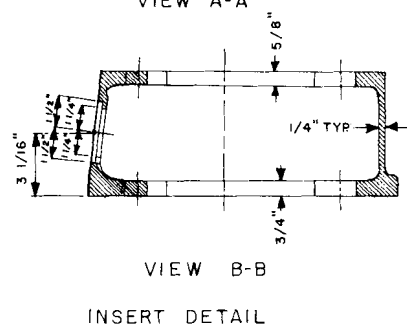
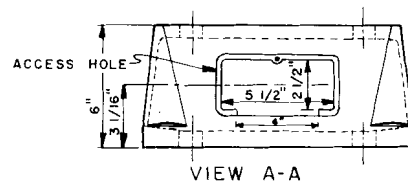
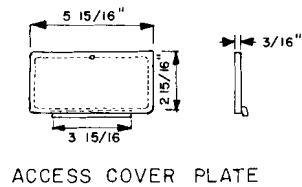
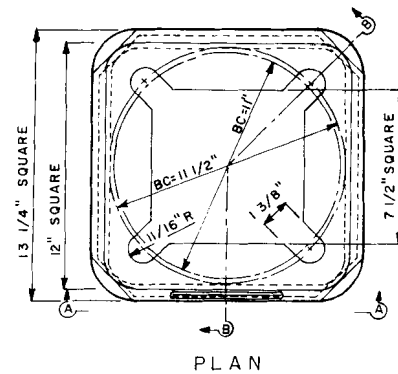
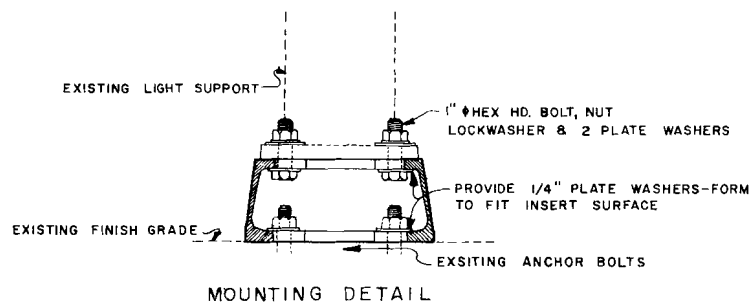


Figure E-1. Modification of lighting supports

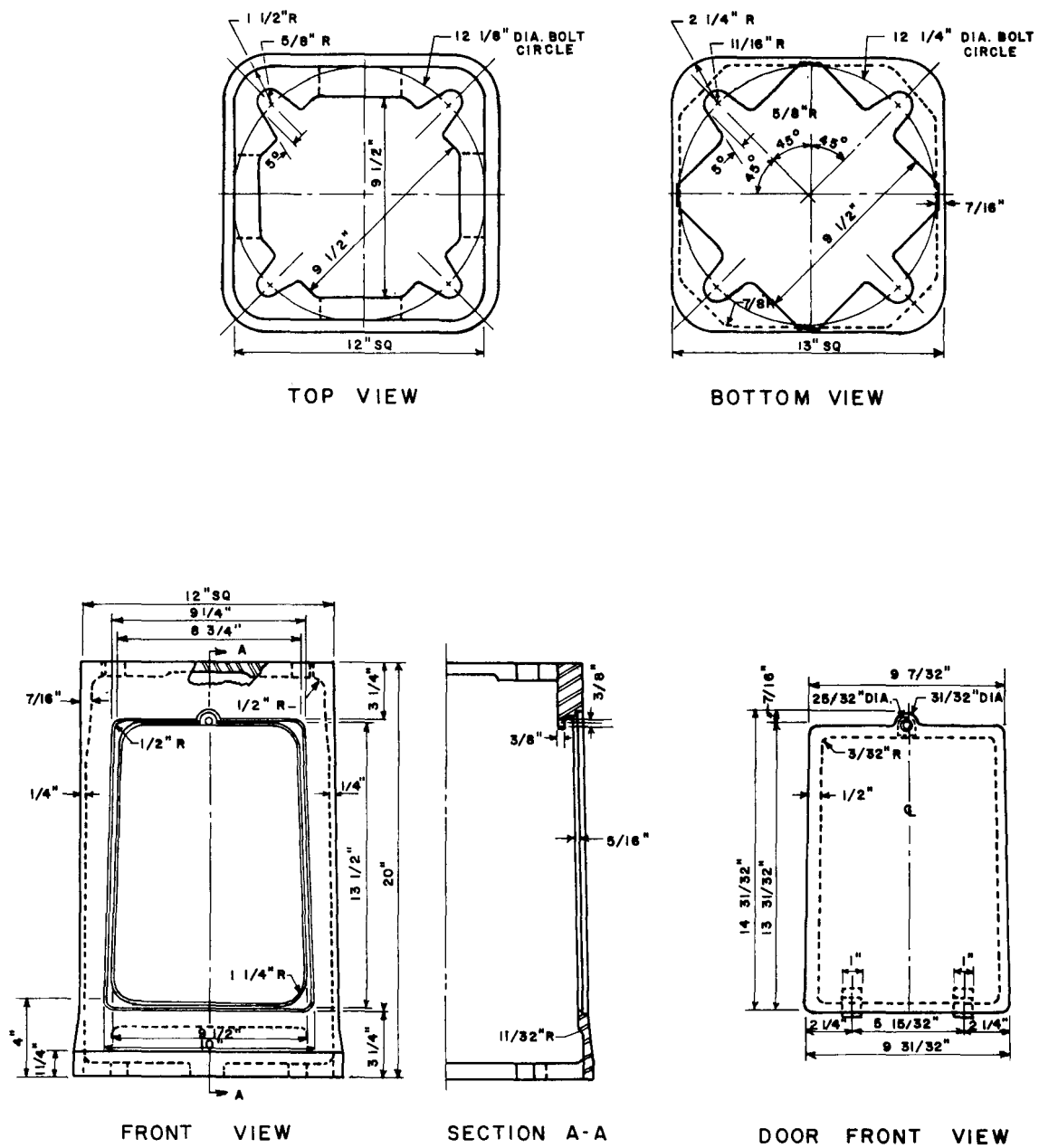


Figure E-2. Cast aluminum transformer base

The cast aluminum transformer base is the most extensively used of the frangible bases, mainly because it behaves more favorably under collision conditions. The break-away action is more positive when the vehicle strikes the frangible material rather than the support above it. When the vehicle strikes the support, much of the shock required to break the frangible device is absorbed in crushing of the support, resulting in a more severe collision.

Progressive failure shear bases

These bases are fabricated of sheet steel in the shape of transformer bases. The break-away feature is obtained by attaching a sheet metal transition section (section between the shaft base and anchor base) to a base plate by means of rivets or button welds. The shear resistance of the base is controlled by the number of connections. Under a collision load, the connections shear progressively allowing the shaft and transition section to separate from the anchored base. Full-scale crash tests conducted by the Millerbernd Manufacturing Company have shown the validity of this concept. Static laboratory tests, conducted by the State of Minnesota, showed that the shear capacity of this base was comparable to a cast aluminum transformer base of similar geometry. Present cost data indicate that the sheet steel progressive-failure shear base is less expensive than comparable cast aluminum bases.

Multi-directional slip bases

The multi-directional slip base developed by the Texas Transportation Institute in cooperation with the Texas Highway Department operates on the slip-base concept described earlier, i.e., high resistance to overturning and low shear resistance. The multi-directional slip base has application in new construction as well as remedial design. Figure E-3 shows a method of providing the break-away concept with an adaptor made specifically for installation under flange-mounted supports. Another alternative, and perhaps a more economical approach is illustrated later in Figure E-7. To add to the break-away feature, the shaft is cut about three inches above the flange and the slip plates are welded in place.

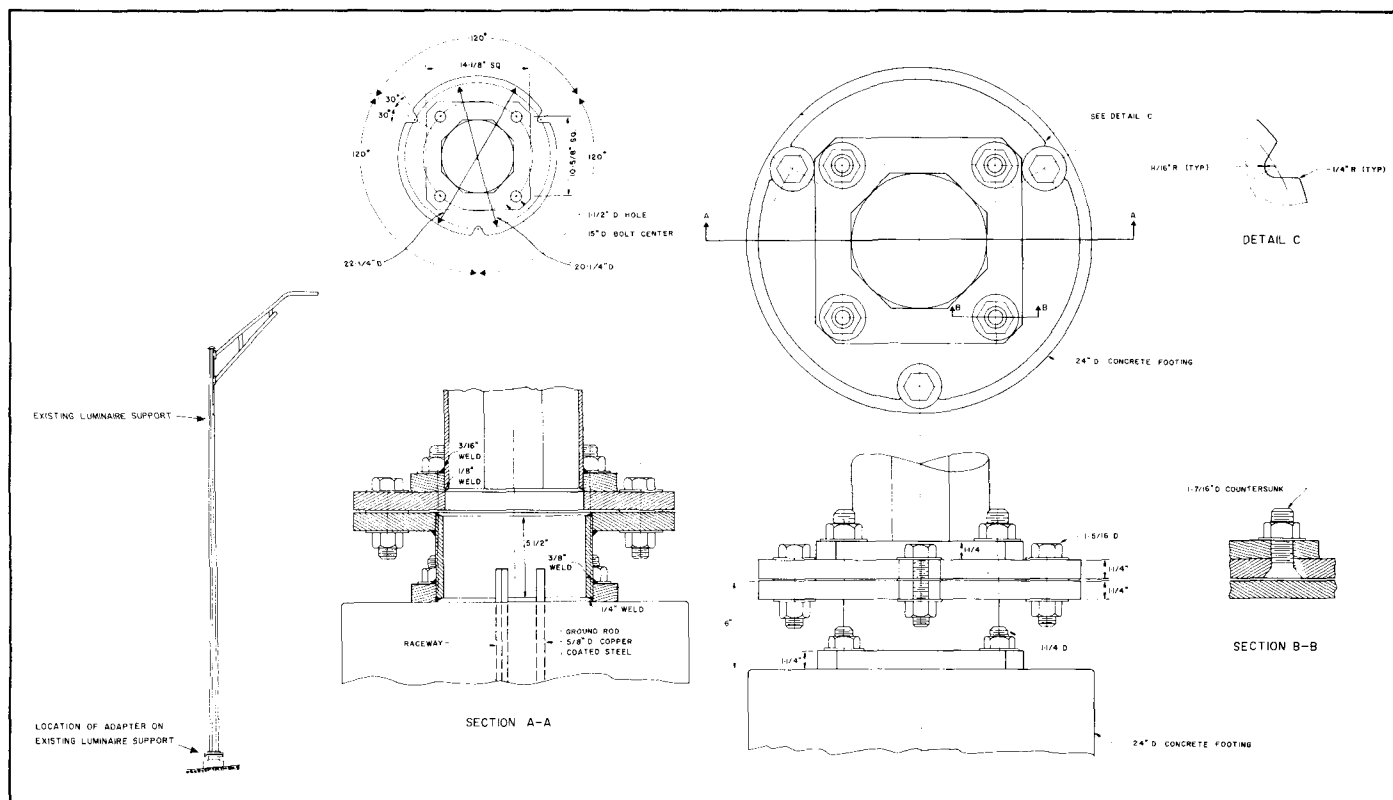


Figure E-3. Multi-directional slip-base adapter

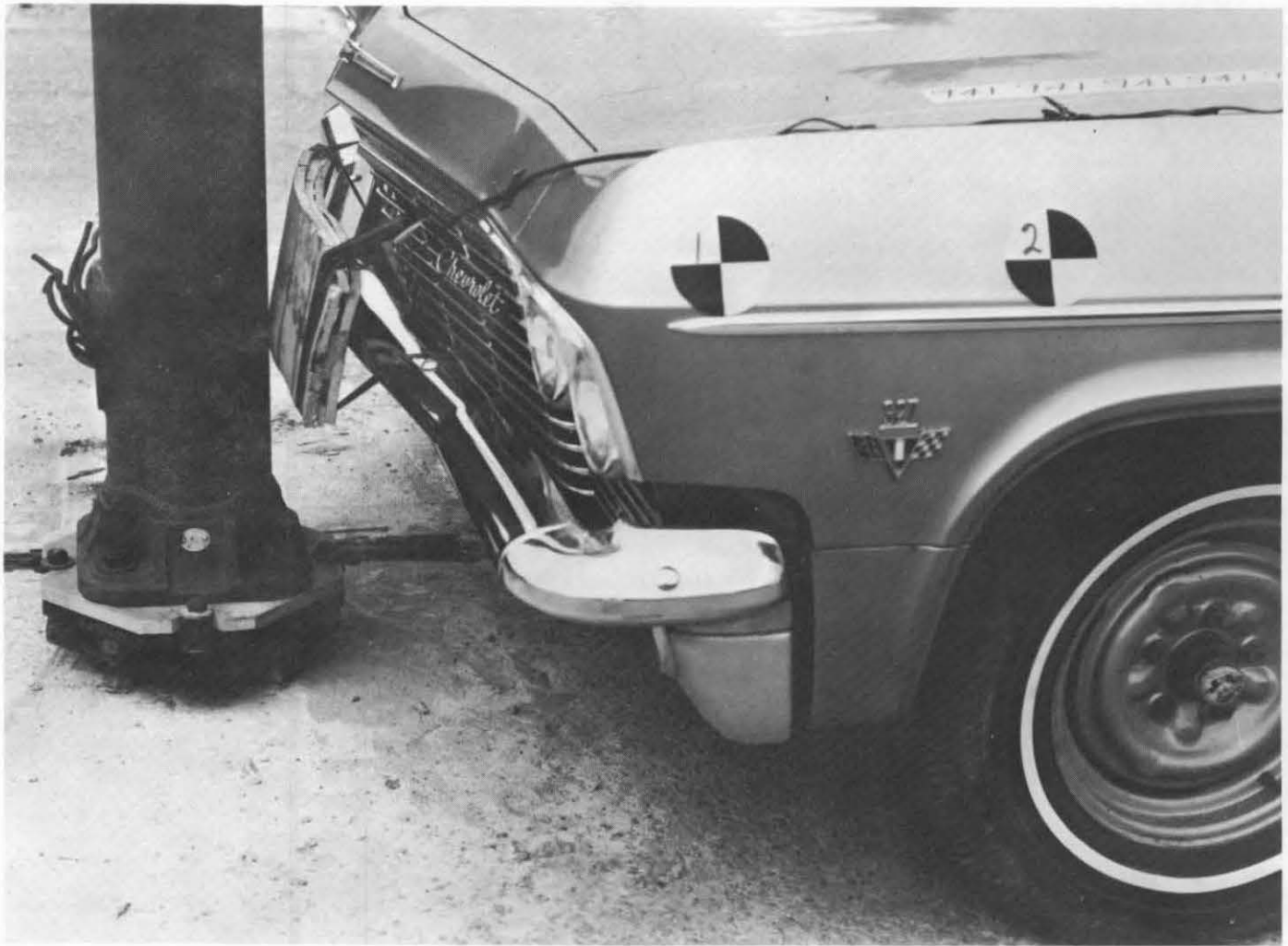


Figure E-4. General Motors slip base

A similar design, shown in Figure E-4, was developed by the General Motors Proving Ground and is currently being specified as a standard by the Michigan Highway Department. The base is essentially the same in detail as that shown in Figure E-3 except that four slots are used instead of three. Full-scale crash tests conducted by the General Motors Proving Grounds have proved the validity of this design.

The multi-directional slip base concept is desirable because normally the support is not destroyed by the impact of collision; thus it can be reinstalled which should reduce the total cost of an installation when the economic factors of first cost, maintenance, and reinstallation are considered.

Stainless steel support

This concept was developed by the Millerbernd Manufacturing Company of Winsted, Minnesota. It is currently being specified by the Minnesota Highway Department. The shaft for this support is fabricated from stainless steel and employs a stainless steel transformer base with stainless steel rivets. The support shaft is of the davit type. The high strength-to-weight ratio and flexural resilience of this stainless steel support contribute to its safety characteristics. Because it is light in weight, little damage is done to a vehicle by either the primary collision or any secondary collision. The flexural resilience of the stainless steel prevents permanent damage to the shaft.

For the lower end of the collision spectrum, (low speed impact) this concept is excellent. The pole is designed to give way when hit by a 2000 pound automobile travelling at 20 miles per hour. The rivets or button welds at the base shear, which permits rapid release of the base. Pole damage is normally confined to the replaceable base shown in Figure E-5.

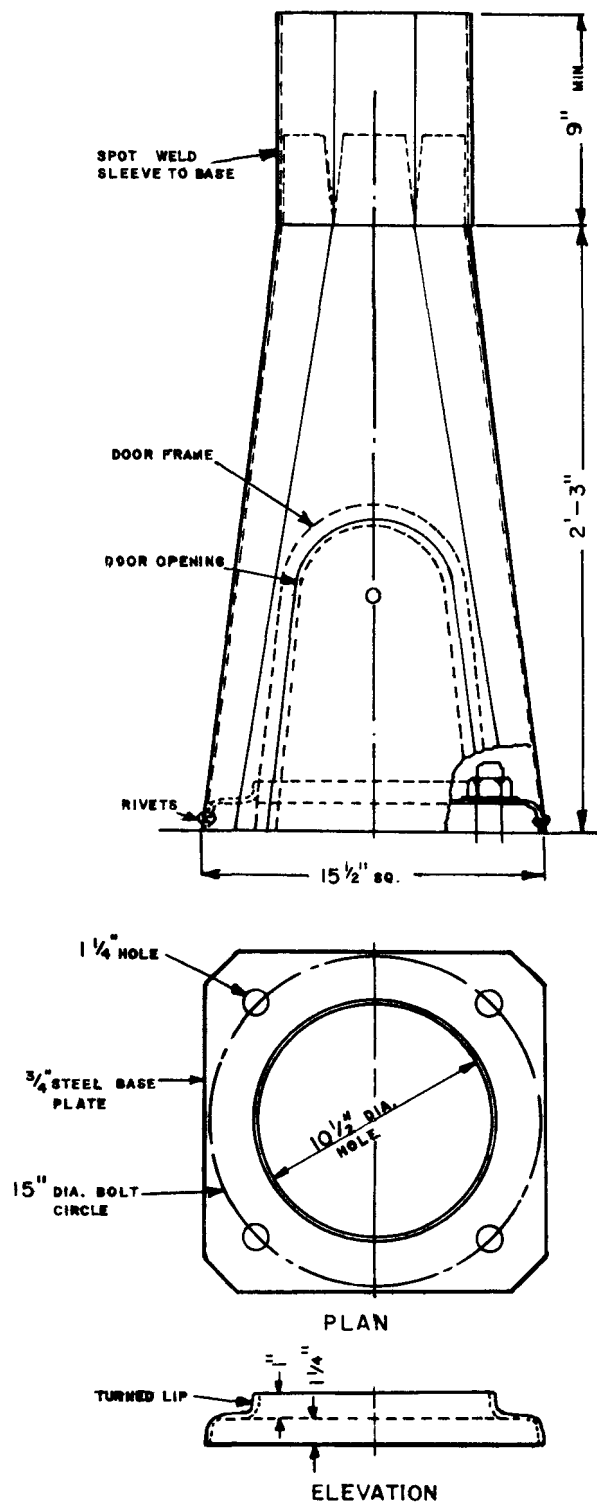


Figure E-5. Millerbernd stainless steel base

Current research

A reduction in the number and severity of collisions with luminaire supports can be effected by (1) location of these obstacles at a greater distance from the edge of the travelled way, (2) the use of a break-away base, and (3) a combination of both of these. At the present time, the employment of a break-away device to modify existing installations appears to be the least expensive approach to providing a less hazardous supporting structure.

Current research indicates that the trajectory and final position of luminaire supports can be predicted mathematically and confirmed by full-scale crash tests. Such techniques will prove useful to the highway designer in selecting break-away devices and in choosing locations for installation of supports.

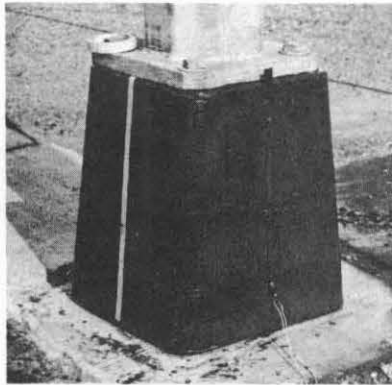
At the present time, it is apparent that the use of the several devices discussed previously has resulted in a reduction of injuries and fatalities in actual accidental collisions on the roadway. Therefore, the highway designer has a choice of several devices which have been proved in reported highway accidents. Collisions with any of the break-away devices produce less harmful effects on vehicles and occupants than do fixed-base supports. Collisions at low speed present the most concern at this time. It is apparent that the time required for the fracture or disengagement of a break-away device is a critical factor in the successful behavior of such devices. This, as well as other factors, is being considered in current research. Until more information becomes available, the designer is constrained to use the types of devices currently available. These include frangible aluminum bases (transformer bases, or inserts), progressive-failure shear bases, and bolted-base slip bases.

The following has been taken from a paper titled, "Impact Behavior of Luminaire Supports," was presented at the 47th Annual Meeting of the Highway Research Board, January, 1968. The paper describes the research recently conducted at the Texas Transportation Institute.

The Texas Transportation Institute has been engaged since 1964 in research on highway illumination with the Texas Highway Department in cooperation with the Bureau of Public Roads. Initially, this research was concerned only with the illumination aspects, but the severity of collisions with lighting poles on Texas highways prompted the inclusion of a phase dealing with the impact behavior of lighting poles. A study has been conducted to determine the impact characteristics of various pole and base mounting designs now in use on Texas highways. In addition, part of the research effort has been devoted to the development and evaluation of a slip base design similar to that used in break-away sign supports. The designs tested are illustrated in Figures E-6 and E-7, and test data are summarized in Table A.

In this study, full-scale crash tests were conducted on luminaire supports using high-speed motion picture photography as the means of obtaining data on the crash tests. The high-speed motion picture films were analyzed to determine vehicle speeds before and after impact. A relative comparison of the severity of impact in each of the collisions is apparent by examination of the change in velocity of the vehicle in the collision with the lighting supports and the measured vehicle deformation or crumpling. A combination of small velocity change and slight vehicle deformation indicates satisfactory behavior of the pole and base combination.

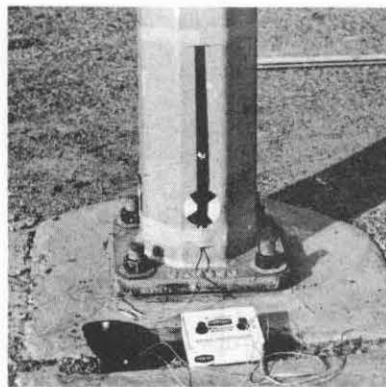
Several tests were devoted to evaluating the impact behavior of cast aluminum transformer bases and the effect of vehicle weight and speed on their behavior. These tests showed that the transformer base is an acceptable break-away design as previously discussed in this report. In this research, it was also shown that vehicles striking luminaire supports at slow speed are likely to be involved in a secondary collision with the support. However, this secondary collision is minor when considered in comparison with a collision with a fixed-base support.



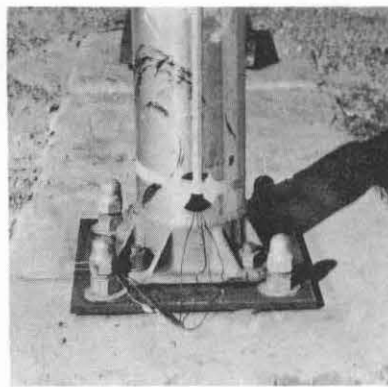
STEEL POLE - STEEL
TRANSFORMER BASE



STEEL POLE - STEEL TRANSFORMER
BASE WITH ALUMINUM INSERT



FLANGE MOUNTED STEEL POLE



FLANGE MOUNTED ALUMINUM POLE



STEEL POLE - ALUMINUM
TRANSFORMER BASE



ALUMINUM POLE - ALUMINUM
TRANSFORMER BASE

Figure E-6. Various pole and base combinations tested



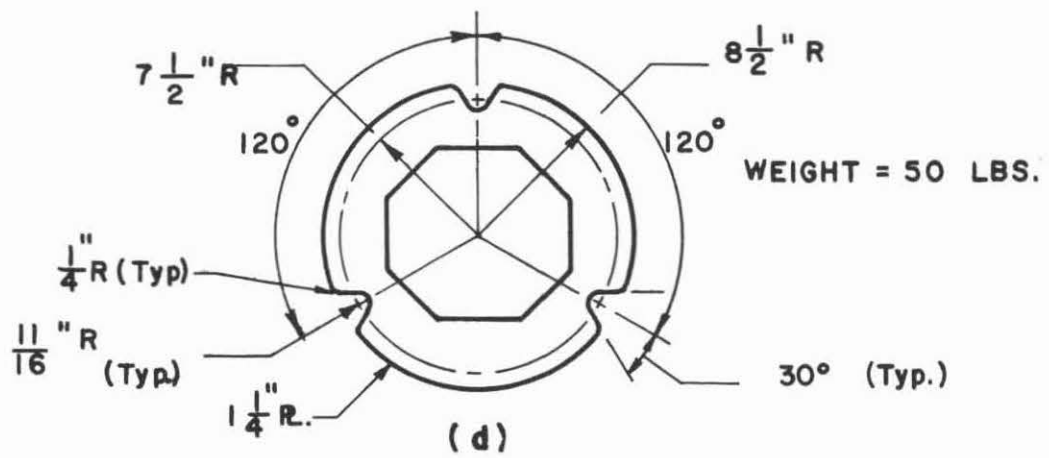
(a)



(b)



(c)



(d)

Figure E-7. Details of slip base

Table E-A. Change in speed and deformation of vehicle in collision with lighting pole and base combinations

Combination	Vehicle Make	Vehicle Weight Lb.	Velocity		Change in Velocity MPH	Deformation of Vehicle In.
			Speed Before Impact MPH	Speed After Impact MPH		
Steel Pole—	1959 Ford	3460	22.2	17.8	4.4	12.7
Aluminum	1959 Ford	3700	44.8	41.5	3.3	15.5
Transformer Base	1960 Simca	2140	45.7	38.0	7.7	12.3
Aluminum Pole—	1959 Ford	3680	21.3	17.0	4.3	10.9
Aluminum Trans-former Base	1957 Ford	3600	43.2	38.0	5.2	10.2
Steel Pole—Steel	1955 Ford	3460	32.2	27.3	4.9	14.4
Transformer Base	1955 Ford	3580	53.2	47.0	6.2	15.8
Aluminum Insert						
Flange-Mounted Steel Pole	1958 Ford	3600	40.5	29.2	11.3	27.4
Flange-Mounted Aluminum Pole	1957 Ford	3500	44.0	37.2	6.8	23.1
Steel Pole—Steel Transformer Base	1958 Ford	3700	39.4	0.0	39.4	30.0
Steel Pole—	1959 Ford	3400	38.3	35.7	2.6	7.8
Triangular						
Break-Away—	1959 Ford	3500	35.7*	34.0	1.7	12.9
Bolted Base						

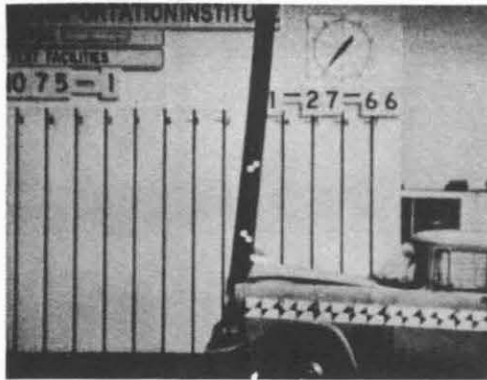
*Collision Angle of Incidence of 30°.

This study also included tests of a cast aluminum insert to be used under existing steel transformer bases. Sequence photographs of the behavior of a steel pole mounted on a steel transformer base and modified by installing a cast aluminum insert are shown in Figure E-8. Maximum vehicle deformation of 15.8 inches is illustrated in Figure E-8(a). This deformation was measured from the forward bumper point and is primarily crumpling of the bumper, grille and hood of the vehicle. Figures E-8(b) through E-8(e) show how the pole was vaulted upward and over the vehicle when struck at 53.2 miles per hour, the vehicle was slowed to 47.0 miles per hour by the collision. The fractured cast aluminum insert is illustrated in Figure E-8(f).

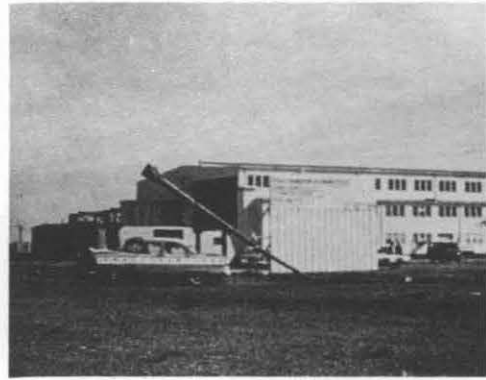
In a slower speed test on the same type of mounting, the performance was essentially the same; however, the pole struck the top of the vehicle, Figure E-9. The crash vehicle was travelling 32.2 miles per hour just before impact and was slowed to 27.3 miles per hour by the collision. Deformation of the front of the vehicle was 14.4 inches and was limited to bumper, grille and hood as shown in Figure E-9(g). The roof over the back seat was dented and the rear window was shattered, Figure E-9(d).

At present, use of the cast aluminum insert is an acceptable method of modifying existing installations when steel transformer bases are in place. This recommendation is based on accident reports as well as the research work conducted by the Texas Transportation Institute.

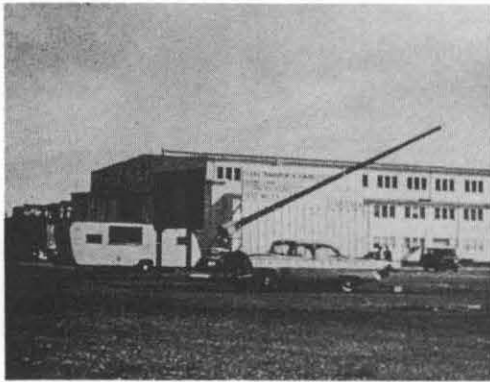
In new installations, it may prove more economical and possibly safer to install a bolted break-away base. Tests conducted by the General Motors Proving Ground, the Road Research Laboratory in England, and TTI, have proven the feasibility of this concept. Laboratory tests on small bases indicated that a slot angle of 30°, Figure E-7(d), provides optimum behavior.



(a)



(d)



(b)



(e)

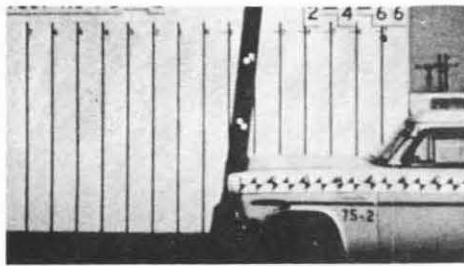


(c)



(f)

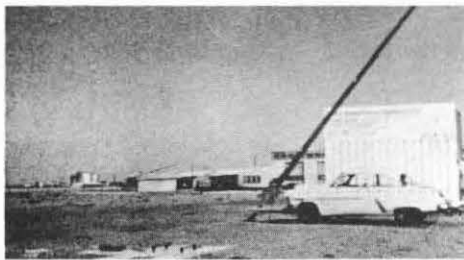
Figure E-8. Sequence photographs of test of steel pole-steel transformer base with cast aluminum insert combination



(a)



(e)



(b)



(f)



(c)



(g)



(d)



(h)

Figure E-9. Sequence photographs of test of steel pole-steel transformer base with cast aluminum insert combination

Full-scale crash tests have been conducted, and the results of one test are shown in Figure E-10. The 3500 pound vehicle was travelling at 35.7 miles per hour at impact, and was slowed to 34.0 miles per hour following the collision. The base bolts were torqued to 1000 pound-inches and three galvanized washers were used; one on top, one between and one beneath the bolted plates. The support had a 45-foot mounting height and a 10-foot long arm oriented as shown in Figure E-10(e).

The TTI investigators summarized their research results as follows:



a) BEFORE



b) AFTER

SUPPORT

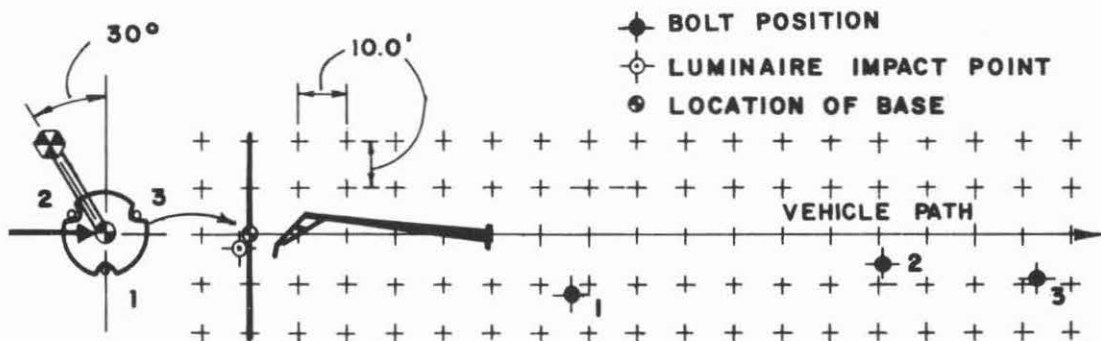


c) BEFORE



d) AFTER

VEHICLE



e) SUPPORT POSITION AFTER TEST

Figure E-10. Bolted break-away base

State of the art study

1. The *steel transformer base* for luminaire supports is definitely an unsatisfactory design and should not be used in any case.
2. The *cast aluminum transformer base* for luminaire supports appears to be a satisfactory failure mechanism to reduce the impact severity of vehicular collisions with luminaire supports. However, this statement should be conditioned to apply only to head-on collisions at this time. No studies were conducted to determine the behavior of these designs under the conditions of skidding or side impact of the automobile. This is a very important consideration and further study is contemplated.
3. A *cast aluminum insert* placed between the foundation and a steel transformer base for luminaire supports appears to be satisfactory *for remedial design measures*. However, this does not appear to be a feasible consideration for new design.
4. The *forty-foot flange-mounted luminaire supports*, both steel and aluminum, left much to be desired in their impact behavior in vehicular collisions. In these tests, the aluminum support exhibited a lower degree of impact severity, but both appear critical when vehicle damage and post-collision behavior are considered.
5. In all tests where support failure occurred, it was found that the supports generally aligned themselves with the direction of the crash vehicle. Also, it was observed that the top of the support struck the ground near the foundation in the tests on cast aluminum transformer bases, while the other “travelled” or were carried a considerable distance beyond the point of impact.

Development of multi-directional slip base

1. The slip base is a feasible design for a multi-directional break-away base for luminaire supports.
2. For optimum effectiveness, the triangular base should have a slot angle of 30° .
3. The optimum collision angle of incidence is 30° measured from a line through the geometric center of the triangular plane form (formed by the location of the three base bolts) to the center of a bolt. The support should be oriented such that the most probable vehicle collision path will coincide with this line.
4. In order to keep the base from “walking” under oscillating wind loads the base bolts should be pre-tightened to not less than 2000 lb/bolt, nor more than 3000 lb/bolt.
colliding vehicle.
5. The test data indicate that the support will align itself with the path of the colliding vehicle.

Safer Roadside Structures

Part F

Impact Attenuation Systems

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Impact attenuation systems

The “DRAGNET” automobile arresting system developed by Van Zelm Associates has stopped vehicles moving at speeds up to 135 mph without injury to occupants and with only superficial damage to the vehicle, and is now successfully operating at drag strips throughout the country.

The following excerpts are taken from a proposal prepared for the Bureau of Public Roads titled: “Arresting Systems for Snagging a Vehicle Leaving the Roadway Near Fixed Highway Obstacles.”

Technical discussion

A. Definition of problem

The problem set forth by the Bureau of Public Roads and as interpreted by Van Zelm involved the design of a low cost energy absorbing highway barrier system, capable of arresting, within 15 feet, vehicles ranging in weight from 1500 to 4500 lbs., and traveling at speeds up to 60 miles per hour. Ideally, to maintain minimum “G” loadings, this system should arrest all vehicles regardless of weight at the maximum distance of 15 feet.

In this proposal, Van Zelm discussed the considerations and plan required to properly evaluate this problem.

B. Design considerations

The highway arresting system should be designed based on criteria developed from studies of the following topics.:

1. Energy and load calculations for various modes of impact.
2. Structural stress analysis of arrester components.
3. Effect on driving conditions.
4. Maintenance factors.
5. Environmental resistance.
6. Susceptibility to vandalism and theft.
7. Aesthetic qualities for highway beautification.
8. Reusability and replaceability.
9. Effect on present automobile components.
10. Human psychological factors to an arresting device.
11. Effect of arrestment on vehicle occupants.

C. Methods of energy absorption used for arresting systems

There are many types of energy absorber systems in use today. These systems will be reviewed with respect to the design considerations listed previously for possible use in this highway vehicle arresting system.

Below is a partial list of these systems with a brief description of their function and use.

1. Types of energy absorbers having potential use for arresting systems.
 - a. *Weight lifting*—This is perhaps the simplest of all energy absorbers. It was the first energy absorber used for an aircraft carrier arresting gear. Components consist of a cable or chain attached to a weight which is raised when load is applied by the vehicle, converting the aircraft’s kinetic energy to potential energy of a mass.
 - b. *Springs*—Metallic springs of all types have been used for stopping moving vehicles from limited velocities. An example would be the final emergency brake of an elevator. Air springs offer great potential but are more costly. Both nylon and rubber springs have also found use in this field.
 - c. *Linear hydraulic*—Consisting essentially of a dashpot, this type of energy absorber has been highly successful for aircraft arresting gears. Examples range from the highly sophisticated Navy Mark VII Aircraft Carrier Arresting Gear to the almost elementary emergency “Water Squeezer” land based aircraft arresting system.

d. *Rotary hydraulic*—This “egg beater” type of energy absorber has found acceptance for land based aircraft arresting systems; examples are the Navy’s M-21 and E-28 Arresting Gears.

e. *Linear friction*—Consisting of brake blocks which are pressed against rails or bands, this device has been used for aircraft arrestments, (the USAF “Rabbit Catcher”) and for stopping jet cars on the U.S.N. Arresting Gear test track at Lakehurst, New Jersey.

f. *Rotary friction*—Similar to an automotive wheel brake, it is rotated by a tape or cable wound around a drum. This principle is used on the U.S.N. M-20 land based aircraft arresting gear.

g. *Ponds and sand traps*—In their simplest form such energy absorbers consist of a water pond or soft sand runway which decelerate the vehicle by displacement of the fluid material. Such systems have proven a limited capability for stopping both aircraft and automotive vehicles. A more elaborately controlled version of the water pond brake can be found at the Holloman Air Force Base High Speed Sled Track where water is scooped from a pond between the rails. Frangible dams insure an ever deepening pond as the sled velocity decreases.

h. *Material destruction or alteration*

(1) *Metallic elongation*—certain metals, such as the annealed austenitic stainless steels, possess high elongation (approximately 60%) under load; this makes them ideal for short stroke, one shot, energy absorbers.

(2) *Plastic elongation*—some plastics such as Unilon (raw nylon) possess even better energy absorption characteristics under a tension load than the austenitic stainless steels; they are easily damaged by ultraviolet light, however.

(3) *Crushing of honeycomb or plastic foam*—honeycomb, either paper, plastic or metal, possesses good short stroke energy absorption characteristics. Its use is well-known on such devices as space craft landing struts. Plastic foam also possesses good energy absorption characteristics when crushed.

(4) The “metal bender”—this arresting system consists of a metal tape, rod or wire which is bent back and forth by being pulled over rollers or pegs. It has been found successful when used on aircraft arresting gears, automobile arresting barriers, and other shock attenuation systems.

(5) *Existing highway barricades*—such devices consist of cable and wooden post fences which can absorb a limited amount of energy by breaking successive posts, or metal fences which, in bending failure, can absorb some energy.

2. *Classification of energy absorbers by performance.*

a. *Geometrically progressive load devices*—These energy absorbers produce a steadily increasing load as the stroke increases. Examples of such devices are metallic springs, air springs, metallic elongation, and plastic elongation.

b. *Constant load devices*—As the name implies, a constant load device is one which produces the same load over the entire stroke independent of speed or mass. This is the most efficient work curve possible. Previously mentioned energy absorption systems of this type are *Weight Lifting*, *Linear Friction*, *Rotary Friction*, *Crushing of Honeycomb or Plastic Foam*, and the *Metal Bender*.

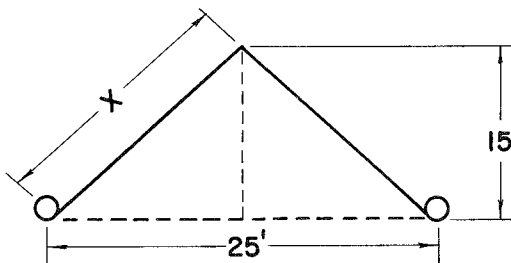
c. *Velocity sensitive devices*—These energy absorbers produce a load which increases with velocity. In most cases the load varies directly with velocity squared. Examples are *Linear Hydraulic*, *Rotary Hydraulic*, and *Ponds and Sand Traps*.

d. *Controlled and constant runout systems*—By control devices it is possible to alter most of the previously mentioned energy absorbers in order to enhance their performance. Thus a linear or rotary hydraulic device which is normally velocity sensitive can be made to produce a near constant load regardless of velocity. The ultimate of controlled system is constant runout; here the system automatically senses the weight and velocity of the engaging object and automatically applies the minimum deceleration necessary to stop the vehicle within the desired stroke. An example of a constant runout system is the U.S.N. Mark VII Aircraft Carrier Arresting Gear.

D. Required loads and deceleration distances

1. Loads imposed on vehicle—The forces required to meet the requirements set forth by the Bureau of Public Roads are quite large. To arrest a 4500 lb. vehicle at 60 mph with uniform deceleration over 15 ft., a 36,000 lb. force would be required. There are few, if any, places on the average car which can withstand this force. A careful study will be necessary to determine whether major structural modifications are necessary for a car to withstand such loads. An even greater problem exists with the case of the lightweight car. If constant force energy absorbing devices are used, the light vehicle must also withstand this 36,000 lb. force. This would result in a 24 g arrestment for a 1500 lb. car, quite an excessive stop. In order to meet the requirements of a barrier capable of arresting a range of vehicle weights of 4500 to 1500 lbs. within 15 ft. without exceeding 12 g, it will be necessary to resort to some type of constant runout energy absorber such as now exists on aircraft carrier arresting gears. Such a system automatically senses the weight of the engaging vehicle and provides the proper arresting force to bring the vehicle to a stop with more or less uniform deceleration over the entire stroke. Unfortunately the present state of the art of such devices requires a degree of sophistication that would make the cost prohibitive for a highway safety device. A thoroughly planned engineering study by personnel experienced in this field may reveal some way to make a constant runout system that would be compatible with a highway safety system's cost and maintenance requirements.

2. Loads imposed on arresting systems—The actual arresting gear loads required by the specifications set down by the Bureau of Public Roads are not excessive, and are well within the state of the art for several types of inexpensive energy absorbers. Geometric conditions imposed by a 15 ft. runout and a 25 ft. span between anchor terminals limit the actual arresting system yield to 7.02 ft. per side as shown below:



$$\text{Arresting System Runout (R)} = X - 12.5 \text{ Ft.}$$

$$X = \sqrt{15^2 + 12.5^2} = 19.52 \text{ Ft.}$$

$$R = 19.52 - 12.5 = 7.02 \text{ Ft.}$$

The kinetic energy of a 4500 lb. weight at 60 mph (88 fps) is:

$$KE = \frac{WV^2}{64.4} = \frac{4500 \times 88^2}{64.4} = 541,000 \text{ Ft. Lbs.}$$

Thus the required arresting cable tension (T) equals:

$$T = \frac{541,000}{2 \times 7.02} = 38,533 \text{ Lbs.}$$

Standard 1 inch diameter 7 X 7 construction heavy galvanized highway guard cable possesses a breaking strength of 45,000 lbs. and could be used for a pendant, however, the low margin imposed would seem to justify a larger cable to the same specifications. As a temporary arrangement to check an experimental system, 7/8 inch diameter 6 X 19 cable with a breaking strength of 79,600 lbs. could be used. An energy absorber for such a system would be designed to produce a 38,500 lb. load.

The Van Zelm report proposed to produce for the Bureau of Public Roads a system which would be manufactured at the Van Zelm facilities for shipment to the Texas Transportation Institute Safety Proving Grounds for testing. At this writing, the system has been subjected to preliminary crash tests at the TTI Safety Proving Grounds and it is anticipated that evaluation of the device will be completed soon.

The device which the Van Zelm Associates introduced in 1959 for aircraft arresting gear is called the "Metal Bender." The energy absorption principle used in this arresting gear was that of plastic deformation of a metal strip which was bent back and forth over a series of rollers. The braking force produced by this method proved to be insensitive to velocity and environmental conditions; it is a constant-load device that can function after years of installation. Arresting systems have been produced for such diversified items as Aeromedical Deceleration Towers and Rocket Sled Terminal Brakes, Load Limiters have also been developed for seat belts, aircraft pilot seat restraint, helicopter troop seats, aircraft cargo restraint systems, astronaut seat restraint systems, and many others. The Metal Bender is a reliable, low cost system which makes it ideal for use as an energy absorber to stop automobiles. The Metal Benders were set to yield at 20,000 lbs., a figure lower than the cable and anchor strength. This would, in some cases, prevent a car from crashing completely through the fence.

The "DRAGNET" arresting system consists of a modified chain link fence both ends of which are attached to energy absorbers. The energy absorbers selected are "Metal Benders" which consist of a case containing a coil of metal tape and a series of pegs over which the tape must bend back and forth as it is extracted from the case. It is this bending that produces the arresting force, which is constant regardless of the velocity at which the tape is extracted, or what the environmental condition may be. In order to prevent injury to the driver and to minimize damage to the car, the force selected for each reel was held to 2500 lbs., producing a total automobile restraining force of 5000 lbs. This results in a deceleration of about 1.25 g for an average car of 4000 lbs. weight. During the development program a total of 102 automobile arrestments were made by Van Zelm Associates. The final configuration was tested consistently at speeds of 80 to 85 miles per hour, with approach angles up to 30° and off net center distances of up to 30% span; weather conditions included rain and snow, as well as fair weather. In all cases, the final configuration functioned satisfactorily producing a smooth controlled stop, with little deviation from the original car path.

Nylon and modified chain link fence materials were used for the net. Nylon proved satisfactory but was ruled out in the final selection because of its poor weathering characteristics and the ease with which it could be stolen or destroyed by vandals. Chain link fence did not have these faults and surprisingly produced less car damage during an arrestment. The fence, in its final configuration, functioned satisfactorily every time showing no tendency to be shed either over or under the car. Figure F-1 shows the manner which the "DRAGNET" drapes over the car hood; the automobile shown had previously engaged the net 21 times, note the slight damage that exists. Figure F-2 illustrates how "DRAGNET" might be used to correct the dangerous condition which exists on many highway overpasses at the present time.

The "DRAGNET" system has been approved by the National Hot Rod Association for use as an emergency stopping device on drag strips. Over a dozen "DRAGNET" systems are presently in use at various drag strips, and have proved quite effective in saving both men and equipment; emergency arrestments have been made at speeds as high as 135 miles per hour. An installation at Capitol Raceways near Washington, D. C. is shown in Figure F-3; the installation was in operation for over two years with no adverse effects caused by weathering, in spite of the fact that no maintenance had been given during this time period.

The installation of a "DRAGNET" system required ample clear space beyond the net to permit adequate runout of the arresting system with moderate slowing of the colliding vehicle. Systems currently being considered indicate that a runout of approximately 30 to 40 feet is desirable to safely slow a colliding vehicle.

The Texas Highway Department is considering the installation of a modified "DRAGNET" system at the approaches to one of its ferry landings. The location of the proposed system will permit a runout in excess of 50 feet.

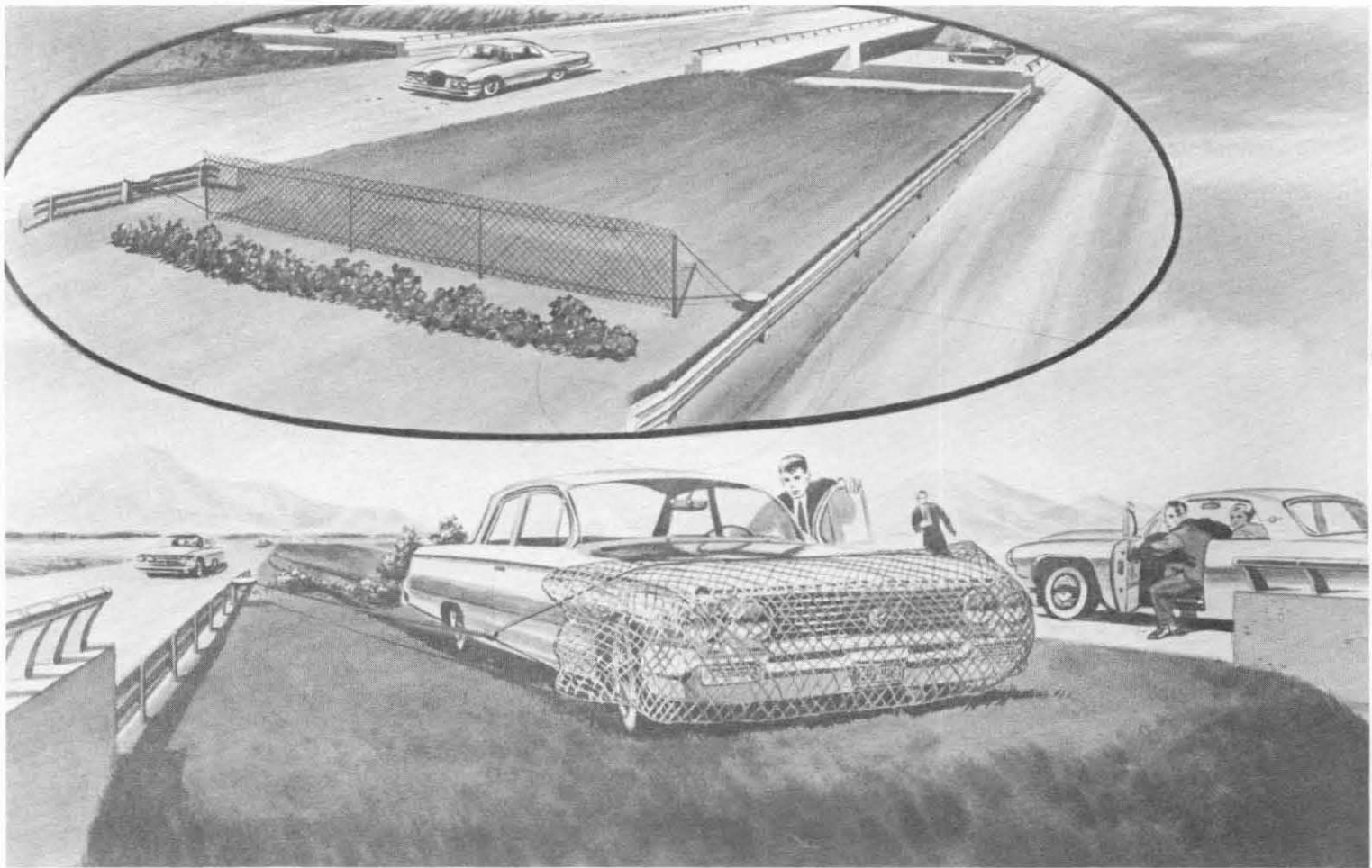
The principles of different types of energy absorbing systems have been presented in the preceding discussion, and a system now in use on drag strips has been described. Another concept has been suggested: a reusable energy absorbing system designed and fabricated by Aerospace Research Associates, Inc., which is constructed of high-strength, lightweight steel tubes, and which employs a number of impact attenuators to absorb the energy of a collision. The impact attenuators or "TOR-SHOK" devices contain a large number of stainless steel torus elements that are squeezed between two cylindrical steel tubes. Axial force transmitted to the cylindrical devices is absorbed by the torus elements which roll between the cylinders.

Other systems and devices are being considered including a system composed of 55 gallon steel drums and a system fabricated from polyurethane foam. Testing and evaluation of these devices are underway at the present time.



PHOTOGRAPH- COURTESY OF VAN ZELM ASSOCIATES, INC.

Figure F-1. Engagement of automobile by "Dragnet"



PHOTOGRAPH-COURTESY OF VAN ZELM ASSOCIATES, INC.

Figure F-2. Proposed "Dragnet" installation at overpass



PHOTOGRAPH-COURTESY OF VAN ZELM ASSOCIATES, INC.

Figure F-3. "Dragnet" installation at Capitol Raceways, Maryland

Barrel protective barrier

Early in 1968, The Texas Transportation Institute developed and tested a successful vehicle crash attenuation test using 55 gallon tighthead universal drums of 18 gage steel as a barrier protecting rigid obstacles which exist along our highways. The barrier was composed of 29 drums with part of the metal cut out and removed from the tops and bottoms in order to achieve the desired crush strength and energy absorbing capability of each drum. Work on this barrier was accomplished by the Texas Transportation Institute under Contract CPR-11-5851 under the Office of Research and Development, Bureau of Public Roads.

A 3200 lb. four-door sedan travelling 60 mph struck this barrier. The stopping distance was 13.33 feet, and the average deceleration of the vehicle was 7.9 g's. Damage to the vehicle was considered minor; only one of the four headlights was broken in the head-on crash. The front bumper and grille of the automobile were crumpled approximately four inches.

At the present time, this barrier system appears to be a most effective, economical, and practical device for attenuation of a vehicle on a collision course with a fixed obstacle, and several experimental installations using 55 gallon tighthead metal drums have been installed by the Texas Highway Department.

On March 28, 1968, a very successful vehicle crash attenuator test was conducted.

Figures F-4 through F-9 show the crash vehicle and barrel protective barrier before and after the 60 mph vehicle impact. Table F-1 presents a summary of the high-speed film crash test data. The vehicle weighed 3,200 lbs. and was a 1964 Dodge four-door sedan. Its initial velocity before impact was 60 mph. The vehicle penetrated the barrier 13.33 ft. before coming to a complete stop. The average deceleration force on the vehicle was 7.9 g's. The peak deceleration on the vehicle was 12.7 g's. As can be seen from the photographs, only minor damage was inflicted on the vehicle. One of the four headlights was broken, and the front bumper and grillwork were mashed in approximately 4 in. The vehicle was in running condition immediately after the impact.



Figure F-4. Crash vehicle before test



Figure F-5. Barrel protective barrier installed in front of 30 in. diam. simulated bridge pier

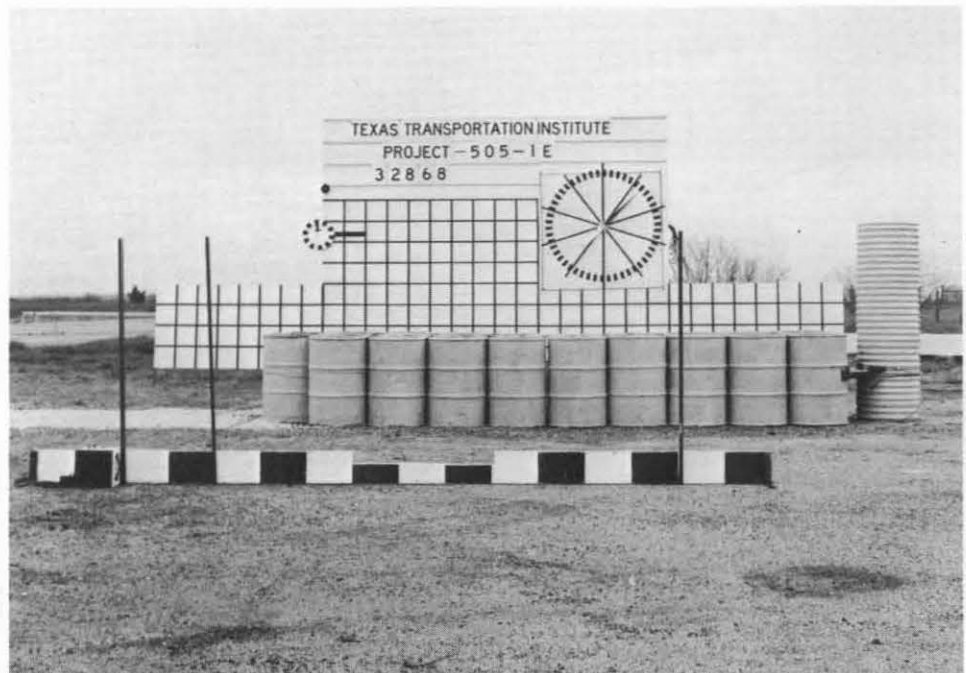


Figure F-6. Side view of barrel protective barrier in front of 30 in. diam. simulated bridge pier

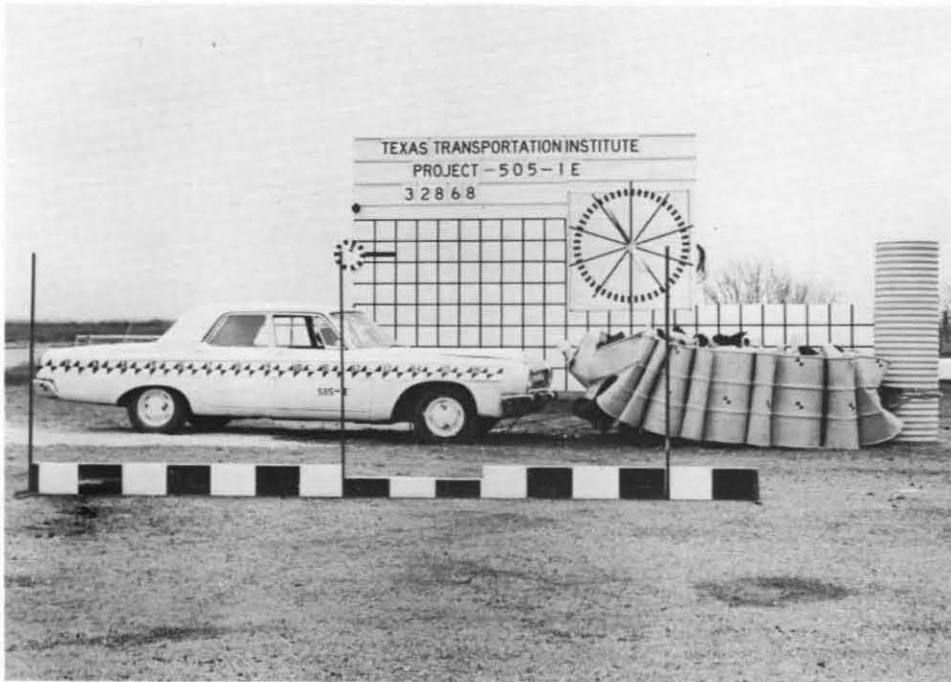


Figure F-7. View of barrel protective barrier and test vehicle after impact. Initial vehicle velocity 60 mph, stopping distance 13.33 ft., average vehicle deceleration 7.9 g's



Figure F-8. View of minor vehicle damage. Only one of four headlights was broken. Bumper and grill deformed approximately 4 in.

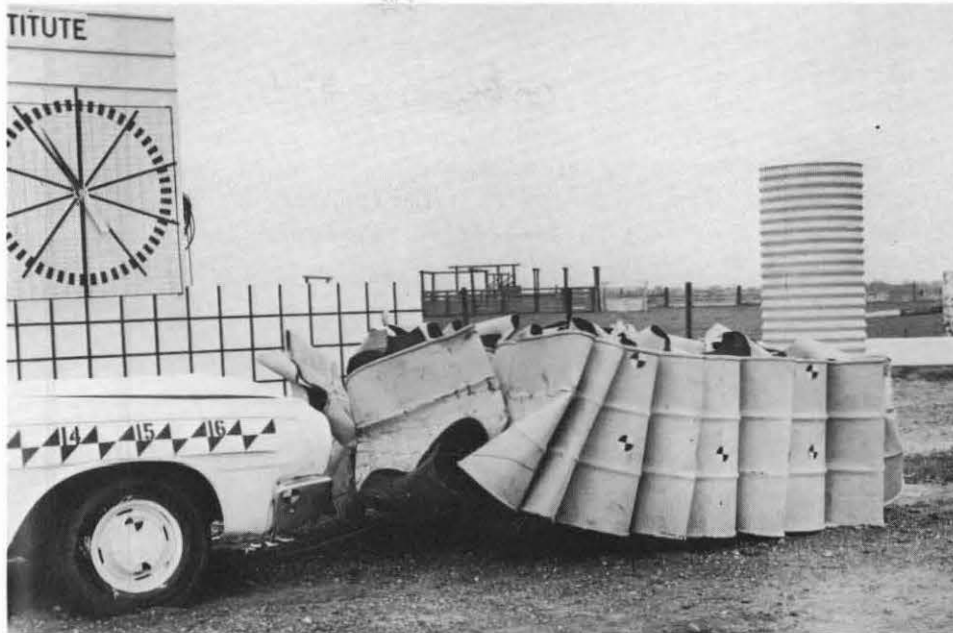


Figure F-9. View of vehicle and barrier after impact

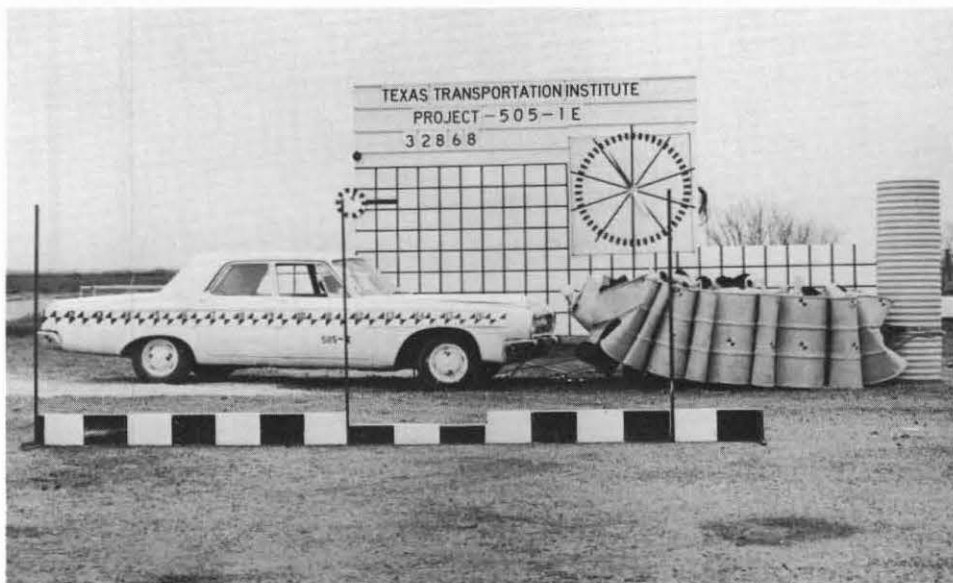


Table F-1. Summary of high-speed film crash test data

Test 505-1E 55 Gallon Drums with Tops and Bottoms Cut, 3 Drums Wide by 10 Drums Deep, Protecting 2 1/2 ft. diameter Simulated Bridge Pier

Vehicle weight = 3200 lb. (1964 Dodge, 4-door)

Vehicle velocity = 60.2 mph or 88.3 fps

Change in velocity = 60.2 mph or 88.3 fps

Average deceleration = 7.9 g's

Peak deceleration = 12.7 g's

Duration of impact = 0.346 sec.

Stopping distance = 13.33 ft.

Remarks: Very minor damage to vehicle. One of four headlights broken.

Table F-3 presents a comparison of this barrel barrier crash test with a "rigid" barrier crash test. If this vehicle had struck a "rigid" barrier, the maximum deceleration would have been approximately 54.2 g's and the average deceleration would have been approximately 34.6 g's. Using these rigid barrier decelerations, the deceleration obtained from the barrel barrier can be compared by taking a ratio which is defined as an Attenuation Index. From Table F-3, it can be seen that the Attenuation Index for this barrel barrier test was 0.23. This index indicates that the impact was only 23% as severe as a rigid barrier crash.

Table F-2. High-speed film data

Test RF505-1E—55 gal. drums with tops and bottoms cut, 3 drums wide by 10 drums deep

Time Milliseconds	Displacement ft	Velocity ft/sec	Deceleration	
			ft/sec ²	g's
0	0		0	0
6.3	0.57		0	0
16.8	1.50	88.3	0	0
Impact *27.3	*2.41		0	0
48.3	4.14	82.4	-271.4	- 8.4
69.3	5.75	76.7	-409.5	-12.7
90.3	7.18	68.1	-247.6	- 7.7
111.3	8.50	62.9	-295.2	- 9.2
132.3	9.69	56.7	-319.0	- 9.9
153.3	10.74	50.0	-295.2	- 9.2
174.3	11.66	43.8	-181.0	- 5.6
195.3	12.50	40.0	-247.6	- 7.7
216.3	13.23	34.8	-228.6	- 7.1
237.3	13.86	30.0	-271.4	- 8.4
258.3	14.37	24.3	-114.3	- 3.5
279.3	14.83	21.9	-295.2	- 9.2
300.3	15.16	15.7	- 90.5	- 2.8
321.3	15.45	13.8	-204.8	- 6.4
342.3	15.65	9.5	-247.6	- 7.7
363.3	15.74	4.3	-114.3	- 3.5
384.3	15.70	-1.9	-204.8	- 6.4
405.3	15.65	-2.4	-133.3	- 4.1
426.3	15.54	-5.2	0	0
447.3	15.43	-5.2		

Table F-3. Comparison of Texas barrel protective barrier crash test with rigid barrier impact

	Barrel Barrier Impact Test 505-1E	Rigid Barrier Impact
Vehicle Weight (w)	3200 lb	3200 lb
Vehicle Velocity (V)	60.2 mph	60.2 mph
Max. Deceleration	12.7 g 's	54.2 g's*
Avg. Deceleration	7.9 g's	34.6 g's*
Attenuation Index		
$AI_{(max)} = \frac{G(\text{max. barrels})}{G(\text{max. rigid})}$	0.23	
$AI_{(avg)} = \frac{G(\text{avg. barrels})}{G(\text{avg. rigid})}$	0.23	

*Note: Estimated Rigid Barrier Impact Deceleration

Determined by $G(\text{max.}) = 0.9 V$

Determined by $G(\text{avg.}) = 0.574 V$

Where V is in mph; from Emori, Richard I., "Analytical Approach to Automobile Collisions," SAE Paper 680016, Auto. Engr. Congress, Detroit, January 8, 1968.

Table F-2 presents a complete tabulation of the high-speed film data showing time after impact in milliseconds, vehicle displacement, vehicle velocity and deceleration. The total duration of the impact was 0.346 seconds. Figure F-10 shows a plan and side view of the barrier as installed. The barrier was constructed as shown by Figure F-10 with the exception that only one 8WF17 backup beam was used instead of the two shown on the drawing. The barrier consisted of twenty-nine 55-gallon drums of 16-gage steel. There were 9 rows of three drums, with two drums on the nose, making a total of 29 drums altogether. The total length of the barrier was approximately 19 ft. The vehicle penetrated the barrier 13.33 ft. indicating approximately 70% of the energy capacity of the barrier was used up. The vehicle had 387,000 ft.-lbs. of kinetic energy.

The dotted lines on the drawing indicate 1/2 in. cables which were tied to the simulated bridge pier and threaded between the rows of barrels, supported on the rolling hoops, and tied off to a reinforced concrete anchor shaft located flush with the ground in front of the nose of the barrier. The 1/2 in. cables were designed to give the barrier lateral stability in case of an angle hit by a vehicle. These cables also hold the barrels on the ground during vehicle impact. The barrels must not be attached to the cable in any manner. They must remain free to slide down the cable during vehicle impact. Additional tests are needed on this system in order to verify the lateral stability of this barrier when struck by a vehicle at angles other than head on.

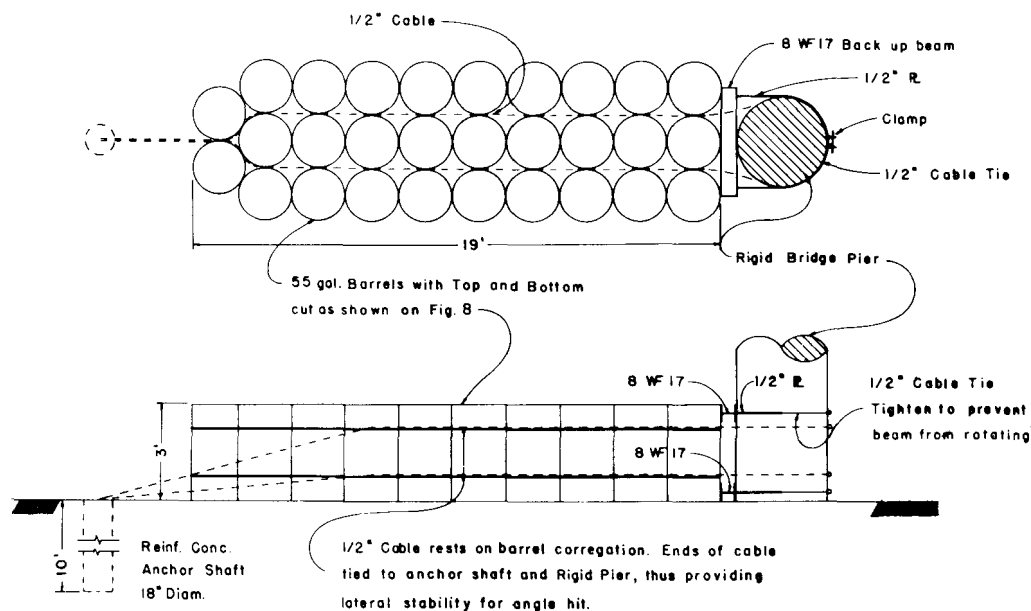


Figure F-10. Barrel protective barrier

Figure F-11 shows the detail of how the top and bottom of the drums were welded together at all points of contact between adjacent barrels. A piece of No. 5 reinforcing bar 2 in. long was placed between the barrel rims and fillet welded to each barrel. Figure F-11 shows how a circular hole is punched in the tops and bottoms of the barrels to reduce the crush strength of a barrel to the desired level.

Figure F-12 shows the static force-deformation curves for the 55-gallon 16 gage steel drums used to construct this barrier. The top curve on this figure indicates a peak crush force of approximately 20,000 lbs. for a 55-gallon drum with the top and bottom left intact. When the elliptical shape holes are cut in the top and bottom as shown in Figure F-11, the crush strength is shown by the lower curve on Figure F-12. A peak crush force of approximately 8,000 lbs. was developed under this static test. Figure F-12 indicates the importance of removing some of the metal from the top and bottom of the drum in order to reduce the crush strength of the barrel. The uncut barrels will generate approximately three times as much stopping force as the barrels cut as shown.

Figure F-13 shows an idealized barrier force-deformation curve under vehicle impact. From an analysis of the high-speed film data it was apparent that the crush strength of the total barrel system welded together was somewhat larger than that obtained from the sum of the individual barrels. This increase in the total barrier force can be attributed to cable friction, ground friction, and lateral support provided to the barrels by adjacent barrels. Additional laboratory tests on barrel crush strengths are now being conducted in order to better establish the barrel strength characteristics.

Fifty-five-gallon drums can be fabricated with various gage metals from gage No. 12 up to gage No. 24. In the very near future, laboratory tests will be conducted on barrels of 18- through 24-gage metal. Such barrels will be much lighter in weight and could feasibly have the desired crush strength without the requirement of punching the circular hole in the tops and bottoms.

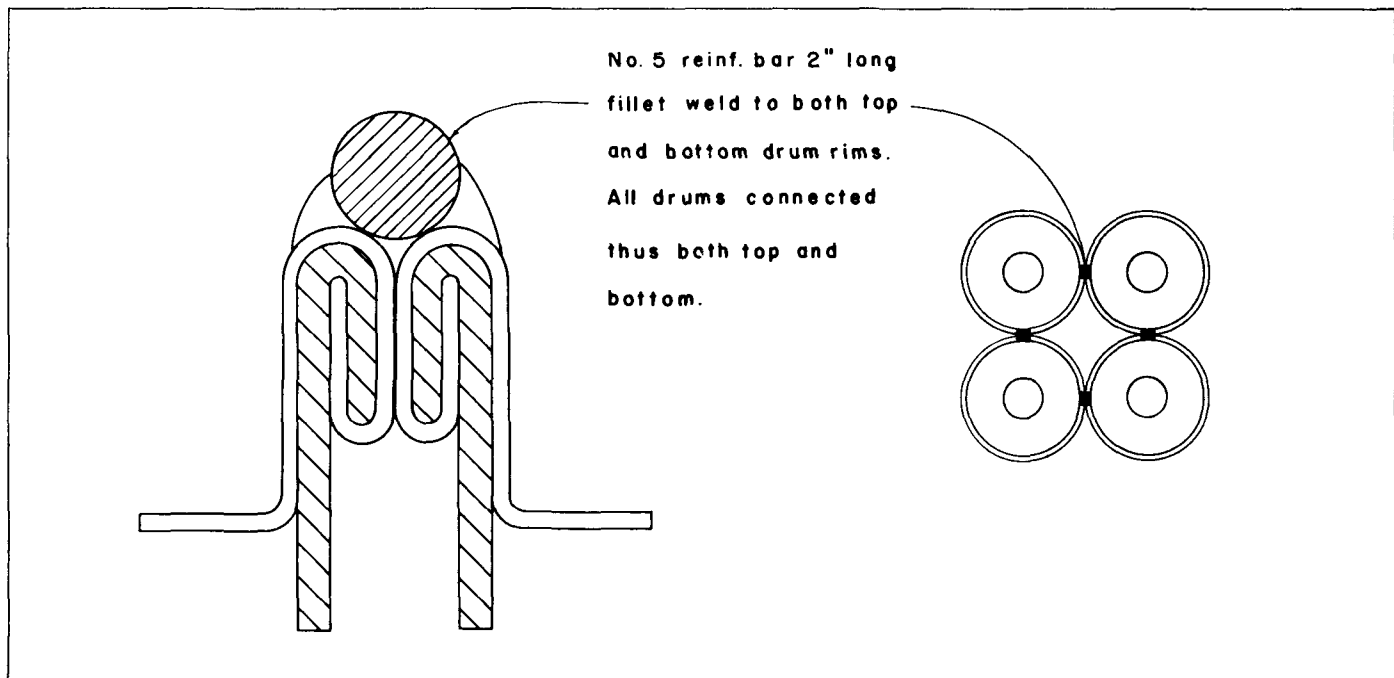


Figure F-11. Detail of top or bottom of 55 gal. tight-head univ. drum, 16 gage steel

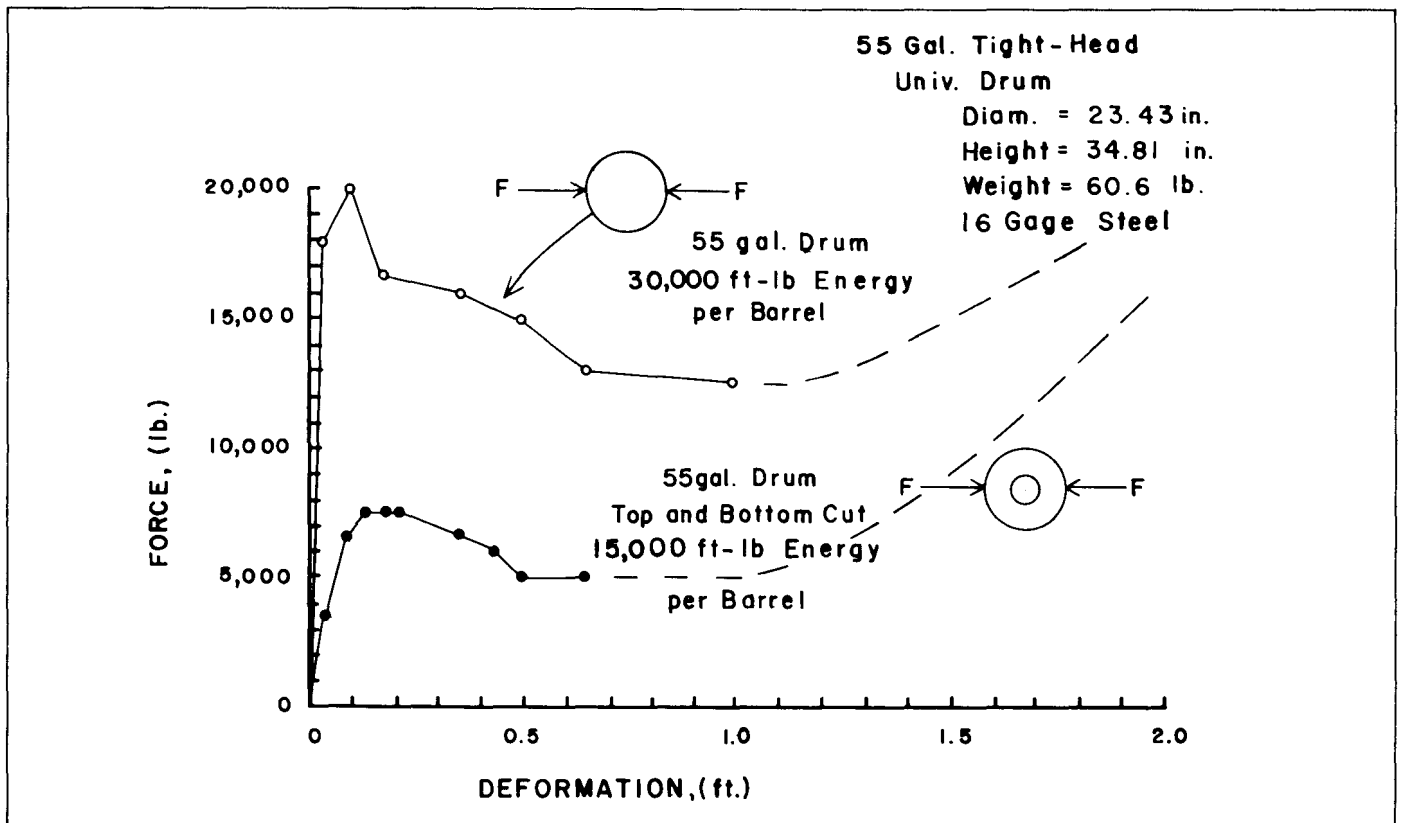


Figure F-12. Force-deformation curve for 55 gal. drums

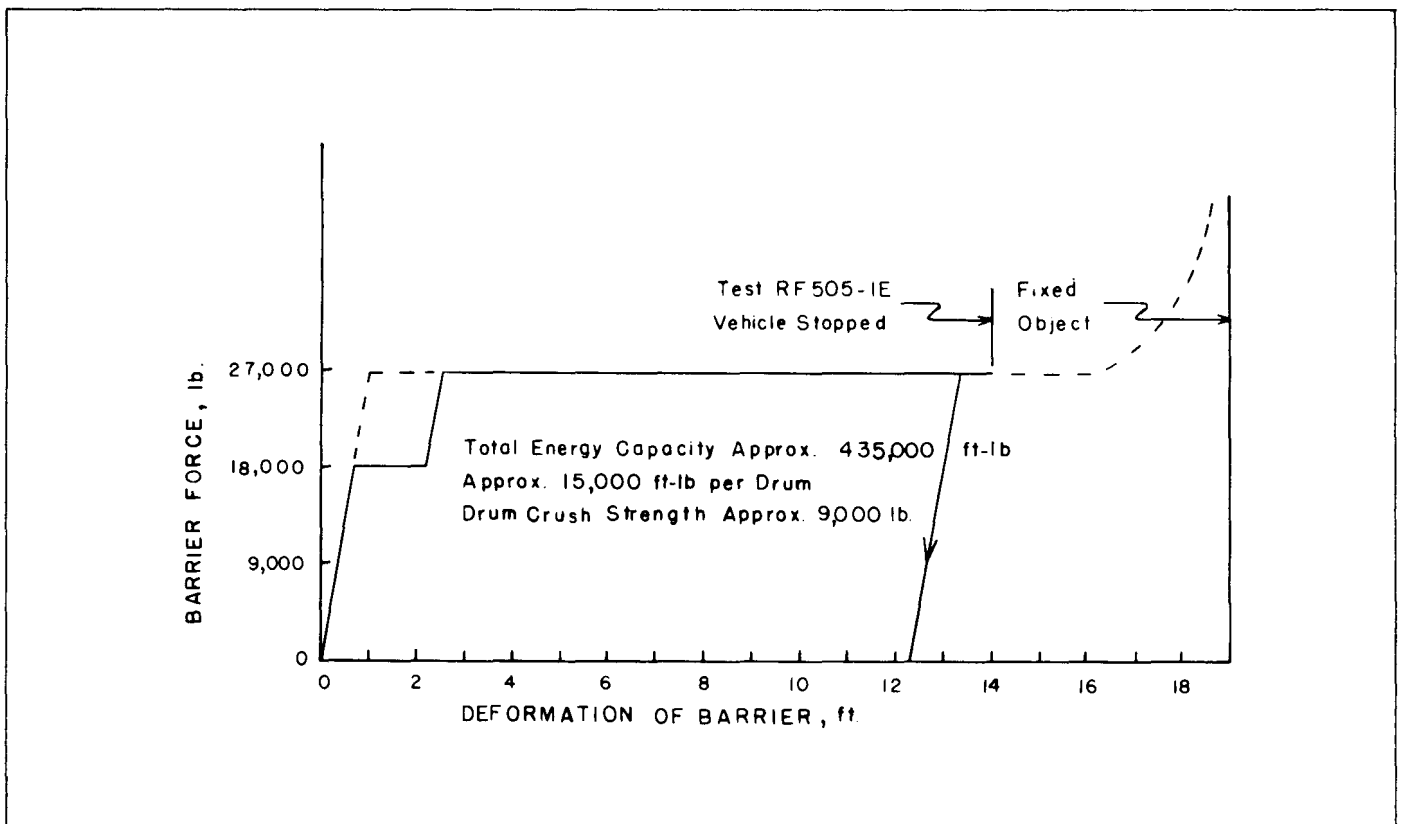


Figure F-13. Idealized barrier force-deformation curve under vehicle impact (includes drum crush strength, cable friction, ground friction, etc.)

Figure F-14 shows some typical hazards where barrel protective barriers could be effectively employed. Several locations in the state of Texas of this type are now being considered for possible employment of this protective system. Figures F-15 and F-16 show some other possible configurations which could be used in the employment of 55-gallon drums as an energy absorption barrier.



Figure F-14. Typical hazardous locations where barrel protective barriers could be employed

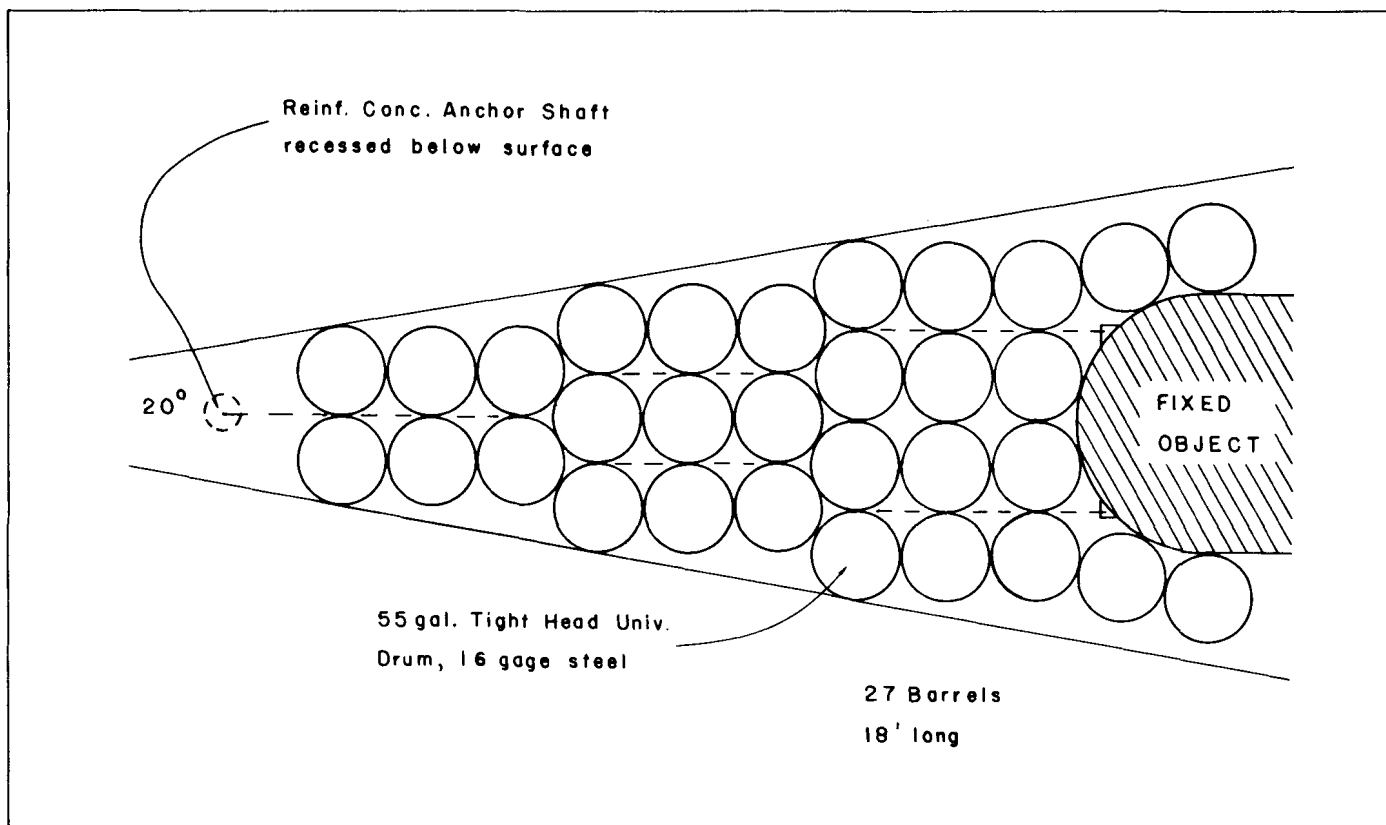


Figure F-15. Possible configuration which could be used

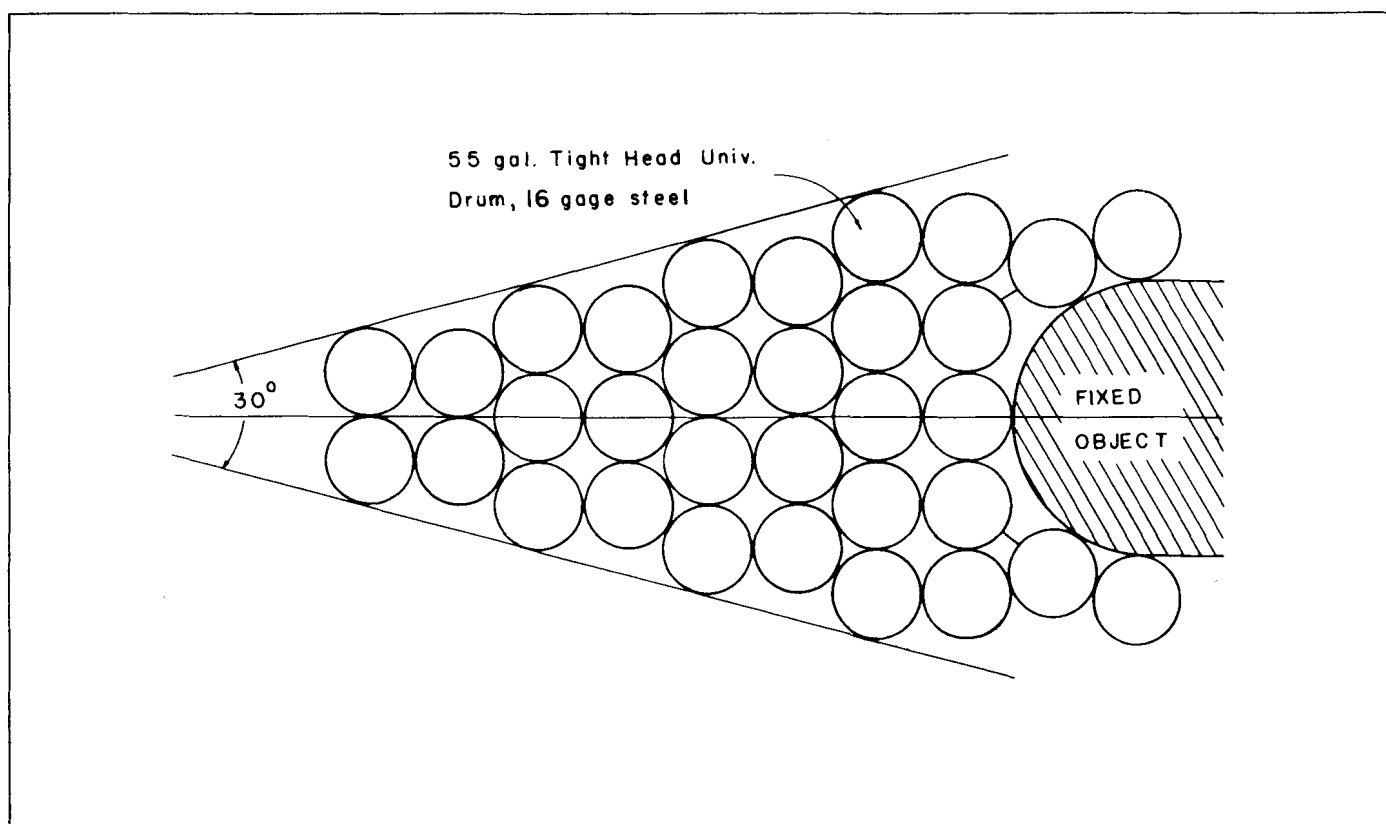


Figure F-16. Possible configuration which could be used

Summary and conclusions

This barrel protective barrier appears to be a very effective, economical and practical vehicle crash attenuation device. Based on the single test conducted to date the impact behavior of the system appears very good. The system appears very economical, since the cost of barrels delivered from a barrel factory will range from \$6 to \$7 each. Second hand barrels can be purchased for as little as \$2 each. The system fabricated and tested here was made of second hand barrels costing a total of \$58 (\$2 each). The cables, steel plates, 8 in. wide flange backup beam, etc. were very minimum in cost.

The system can be fabricated and installed by semi-skilled laborers. Maintenance and reliability of the system also appears good. The system should behave satisfactorily under extremely high or low temperature conditions in either a wet or dry condition. After such a barrier is struck by a vehicle, it will probably prove very feasible to replace the whole barrel system with a spare or replacement system which could be stored in a highway department maintenance yard. Since the barrels are all welded together and are tied down by the anchor cable it is believed the system will behave satisfactorily under angle impact. The stability of this system under angle impact, however, needs verification by further impact tests which are anticipated in the near future.

Safer Roadside Structures

Part G

**An example of a program to reduce hazards
which exist on highways**

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-- CTR Library Digitization Team

A program to reduce hazards on existing highways

Safety features to reduce hazards within highway rights-of-way are being included in plans for new construction. Improvements to existing highways may not keep pace with new construction. Highway engineers are concerned about the existing highway system as well as additions to that system. Operational, financial, and other problems do not permit immediate solutions for all hazardous conditions. However, some action can, and must be taken immediately to make our existing highways more safe.

The California Division of Highways has instituted a program called "Clean Up the Roadside Environment" (CURE), and has published guidelines for this program which it has distributed to the District Engineers of that state. The purpose of this program is to provide a roadside environment which is free of hazardous obstacles, and it is a detailed description of California's method of attacking the problem of the hazardous roadside environment. This program, includes most of the known protective devices and techniques in use in the several states today. A synopsis of the CURE program follows. The California report included photographs which illustrated the features which were to be corrected in the CURE program.

Guidelines for Cure projects

The CURE "Clean Up The Roadside Environment" Program pertains to existing highways. Its purpose is to provide a clean recovery area with a "forgiving quality" for vehicles which leave the traveled way for any reason.

It would be desirable to clean up all of the objectionable features the first time around. Availability of funds, of course, will not permit this; and certain features, such as widening narrow bridges, relocating overhead gore signs, etc., will be included in the initial projects only when justified by the accident experience at each individual location.

CURE projects shall be a minimum of five miles in length; however, as much length as possible is desirable. Priority should be given to Interstate routes, freeways and expressways. It is intended that each District should undertake at least one CURE project immediately.

As many correctable features as possible should be covered in the pilot projects. The top priority features listed below should be covered in all projects. Headquarters Traffic personnel will be available at any time for consultation with or to provide assistance to the Districts in developing the pilot CURE projects. Joint reviews by both Headquarters and District personnel should be made prior to developing contract plans for the pilot projects. Since the future of the over-all program will hinge on the successful reduction of accident rates on the pilot projects, it is essential that great care be used in selecting locations for and choosing features to be corrected on these pilot projects.

IBM tabulations covering the three year period from 1964 through 1966 and showing all fixed object type accidents are being forwarded to the Districts under separate cover. These tabulations are for the District's use in choosing locations for the pilot projects.

The tabulation sheets include listings which show fixed object type accidents. Since items shown on these listings include objects such as guide posts and fences as well as more hazardous items, it is recommended that more detailed information be obtained for tentative project locations.

The features to be included in this program are as follows:

1. Guardrail

- (a) This is a top priority item and should be included in all CURE projects.
- (b) Fixed objects in the roadway plane which are within 30 feet of the edge of the traveled way and *cannot* be eliminated or replaced with breakaway bases should be protected by guardrail.

(c) Existing guardrail which is not essential for protection of the motorist creates a hazard in itself and should be removed.

(d) Envelopes around fixed objects in the median should be closed to avoid exposing posts to opposing traffic.

(e) Short runs of guardrail should be connected to eliminate gaps where vehicles can strike the ends of sections of rail.

(f) The minimum length of guardrail in any one run should be 75 feet except where specified differently on the Standard Plans as at locations such as envelopes around median overhead sign standards.

(g) New instructions relating to the use of washers for guardrail bolts and details for rigid connections to structures are being forwarded by separate circular letter and should be followed on all CURE projects.

(h) Guardrail posts should be added so as to meet post spacings of 6' 3" in compliance with new standards whenever rigid fixed objects such as bridge piers are being protected. In cases where guardrail has been placed to prevent vehicles from going off less hazardous embankments, it will be satisfactory to hold to the 12' 6" spacing on existing installations.

(i) All guardrail installations should include the standard flares as illustrated on the Standard Plan Sheets.

(j) When ground mounted signs are converted to breakaway or wood posts, all guardrail protection shall be removed.

(k) Whenever guardrailing can be eliminated by minor amounts of earthwork, whether imported fill material is needed or not, the changes should be made.

(l) All metal plate guardrail and timber guardrail should be replaced on CURE projects.

2. Roadside signs

(a) This is a top priority item and should be included in all CURE projects.

(b) All superfluous signs which can be eliminated under present standards should be removed.

(c) Replace all steel sign posts with metal breakaway posts, timber poles or wood posts, except those which are presently installed in protected locations. A protected location is considered to be adjacent to a bridge abutment, behind a bridge rail, behind a guardrail required for some other obstruction, etc. An existing sign protected by guardrail which was placed solely for the protection of the sign, is *not* considered to be in a protected location.

(d) Move all large ground mounted signs out to provide a minimum horizontal clearance of 22 feet from the edge of the traveled way. Where right of way is ample and slopes are flat, minimum clearances of 30 feet are recommended. This includes existing signs presently mounted on wood or timber posts as well as those mounted on steel posts.

(e) Every effort should be made to place signs in protected locations even though they may be mounted on breakaway standards. Vehicles striking timber poles and breakaway steel standards will experience a severe shock even though such installations are less hazardous than rigid designs.

(f) As many sign standards as possible should be eliminated by mounting signs back to back or by utilizing structure mounted installations.

3. Light standards

(a) This is a top priority item and should be included in all CURE projects.

(b) All light standards on freeways, except those in protected locations, should be revised to include breakaway bases.

(c) Light standards which encroach on shoulders should be relocated prior to converting to breakaway designs.

(d) Draw lights should be eliminated when gore areas are to be wiped clean or where existing gores are clean except for light standards and minor facilities such as guide posts and signs mounted on dimensioned wood posts. When draw lights are eliminated the lights at the gore should be replaced by Type XXV standards located 18 feet from the traveled way and lamp wattage should be increased to 700.

(e) When fixed objects such as gore signs are not to be removed under CURE, breakaway bases shall be installed under the draw lights.

(f) Any light standard which has been hit repeatedly or is located in a particularly hazardous location, should be eliminated or moved laterally and provided with a breakaway base.

(g) All concrete lighting standards shall be replaced with Type XXV standards located 18 feet from the traveled way and with a 700 watt lamp.

(h) Where existing concrete foundations extend 6 inches or more above surrounding grade, they should be replaced by a new foundation located as shown on Drawing SES-11-1.

(i) No new lighting will be provided at minor interchanges (those with total ramp volumes less than 1,000 ADT) in rural areas. Raised pavement markers should be installed on the gore stripes at these locations.

4. Gore signs

(a) This is a lower priority item. Each existing overhead gore sign should be considered on its own merits. Moving such signs should be deferred to later phases of the CURE Program unless they present severe hazards demonstrated either by a past accident history, or by some unusual condition, such as head-on to the through freeway lanes, or at hidden off-ramps.

(b) When warranted, overhead gore signs shall be replaced either with a sign bridge spanning the entire freeway or with a left hand cantilever positioned 100 to 150 feet in advance of the gore nose. Ordinarily, second cantilever signs will be required for the left hand panels of the former butterfly signs; however, in many cases these panels may be either eliminated or mounted on over-crossing structures.

(c) Overhead sign posts, when moved, shall be located a minimum of 18 feet from the edge of the traveled way. They shall be protected by guardrail when located within 30 feet of the traveled way unless they are placed on slopes.

5. Trees

(a) This is a top priority item and should be included in all CURE projects.

(b) All trees in the roadway plane within 30 feet of the edge of the traveled way should be removed.

(c) Consideration should be given to replanting trees in less hazardous locations. This may offset some of the criticism which may arise due to the tree removal.

6. Curbs and dikes

(a) This is a lower priority item and each individual case should be justified on its own merits.

(b) Dikes in particular can be removed at relatively low costs, and at locations where they are not essential for drainage control, they should be removed.

(c) Curbs should be removed at locations where past accident history indicates a hazard exists.

7. Drop inlets, headwalls and other drainage facilities

(a) These are lower priority items but should be included in initial CURE projects unless costs are extremely high as compared to accident potential.

(b) Drop inlets extending above ground line should be replaced with flush grade inlets in locations where there is a good possibility of their being struck by vehicles.

(c) Culverts should be extended; headwalls removed; bridge rail removed and median areas between parallel bridges decked over. Holes and depressions within 30 feet of the edge of the pavement should be back filled wherever the accident history indicates that improvement is needed.

8. Narrow bridges and bridge rails

(a) These are low priority items. Widening or replacing narrow bridges in ini-

tial CURE projects should be resorted to only when the accident history is severe enough to justify major expenditures.

(b) Old style bridge rails which are not of sufficient strength to restrain vehicles should be replaced at locations which have serious accident potentials.

9. Slopes and miscellaneous

(a) There are a number of miscellaneous improvements which can be made in CURE projects. These improvements will be included whenever the hazards involved are severe enough to justify the expenditure of funds required to correct the conditions.

(b) Slopes should be flattened, soft shoulders stabilized, rough pavements patched, dips removed, and holes beyond the shoulder areas filled.

(c) Serious consideration should be given to improving recovery areas by modest amounts of grading on all CURE projects.

The CURE program recommends specific solutions for specific hazards; other state highway departments may have developed alternate methods of solving these problems. For example, the use of wooden posts for sign supports, with or without slots or holes near the ground may be used in other states. However, some states will recommend using breakaway base sign supports in lieu of timber supports.

One shortcoming which has been noted on many miles of highways is the indiscriminate use of guardrail, for example, guardrail which protects a sign support or lighting standard which has a rigid connection at its foundation. It should be emphasized that guardrail should be installed as a safeguard for the public and not as protection for a structure. Also, one should bear in mind that guardrail installations should be of sufficient length to provide adequate beam strength, and should be properly anchored at the ends.

A coordinated program, such as CURE, in which traffic, design, construction, and maintenance engineers participate in planning as well as construction is recommended.

Safer Roadside Structures

Part H

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