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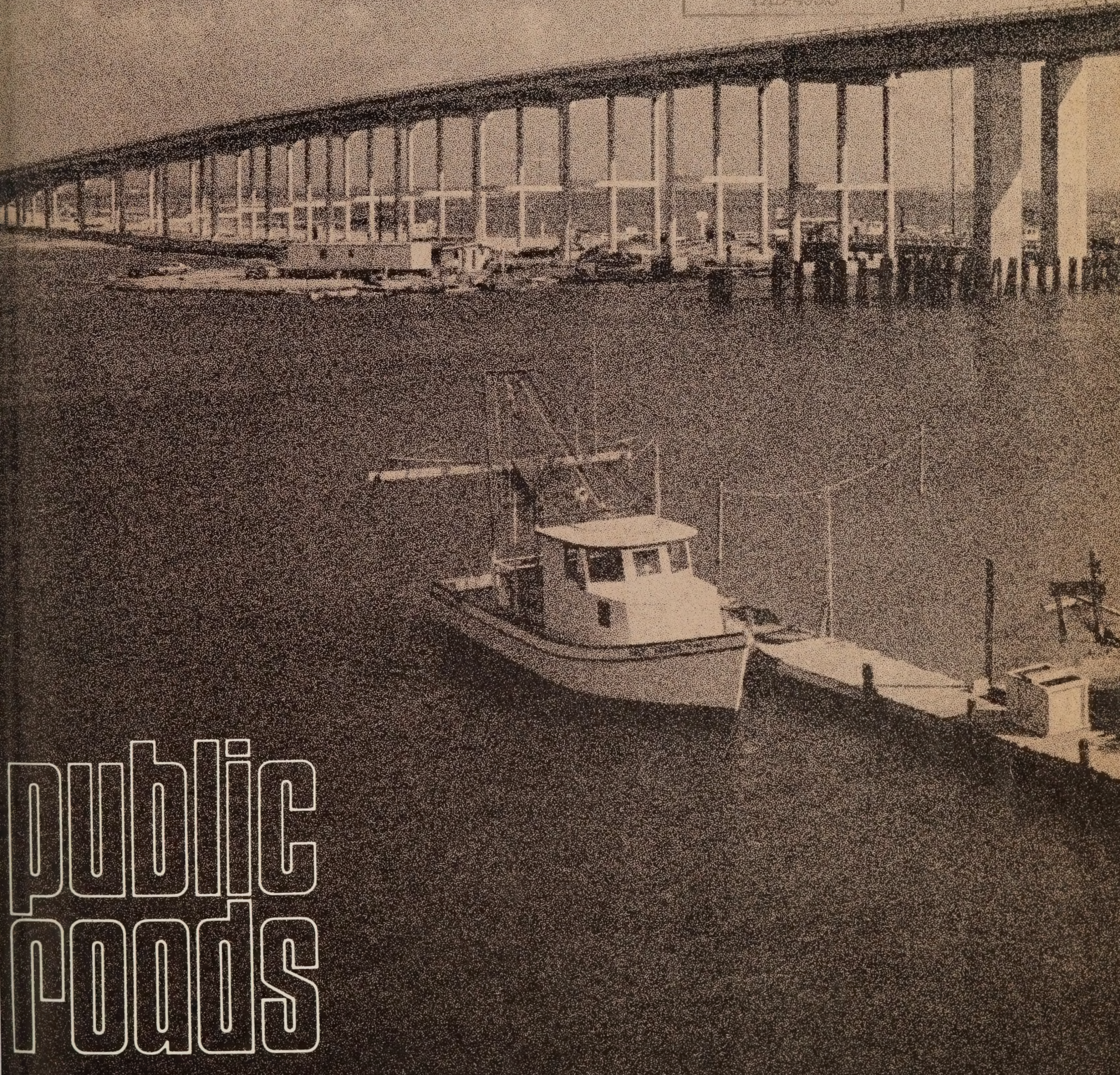

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COVER:
A three-span precast segmentally constructed box girder bridge completed in Corpus Christi, Tex., in 1973.

IN THIS ISSUE

Articles

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Segmental bridges - Design

~~Bridges, Segmental~~ - Design   Breen, John E.

Bridge design

Corpus Christi Bridge under construction.

Design of Segmental Bridges

by John E. Breen

(pt. 1)

This article was presented at the 1976 Federally Coordinated Program of Research and Development Conference at Pennsylvania State University, during the review of Project 5F, "Structural Integrity and Life Expectancy of Bridges." The article, which reviews the current state of the art for the design of segmental prestressed concrete box girder bridges, will be published in two parts. This part covers general design considerations, design sequence, conceptual and detailed design requirements, and analysis procedures. The second part will cover substructure design, specifications, construction techniques, and materials considerations; followed by a brief description of the current status of segmental bridge construction in the United States.

Introduction

Bridge engineers are challenged by requirements for safer, yet more economical bridge structures. Response to requirements imposed by traffic considerations, natural obstacles, more efficient use of land in urban areas, safety, and aesthetics, has been to use longer span structures. Currently, one of the most commonly used structural systems for highway bridges consists of prestressed concrete I-girders combined with a cast-in-place deck slab. This system has a practical span limitation of approximately 120 ft (35 m). Rising costs and maintenance requirements for steel bridges have increased the need for cost effective precast, prestressed concrete spans in the 120 to 400 ft (35 to 120 m) range.

In the United States, spans in the 160 ft (50 m) range have been achieved by using post-tensioned, cast-in-place girders. (*1*)[1] The box, or cellular, cross section shown in figure 1 is ideal for bridge superstructures, since its high torsional stiffness provides excellent transverse load distributing properties. Construction experience on the west coast has demonstrated that this type bridge is economical and structurally efficient.

In Europe, Japan, and Australia, during the late 1960's and early 1970's, the advantages of the cellular cross section were combined with the substantial advantages obtained by maximum use of prefabricated components. (*2*) By precasting the complete box girder cross section in short segments of a convenient size for transportation and erection, the entire bridge superstructure may be precast. These precast units are subsequently assembled on the site by post-tensioning them longitudinally. A number of extremely long span precast

[1] Italic numbers in parentheses identify the references on page 111.

and cast-in-place box girder bridges have been segmentally constructed in Europe (*2, 3*), and interest in this construction concept is rapidly growing in the United States. A three-span precast segmentally constructed box girder bridge with a 200-ft (61 m) maximum span was completed in Corpus Christi, Tex., in 1973, and several other similar U.S. projects have been completed more recently.

When construction of large numbers of prestressed concrete bridges is planned, precasting has a number of advantages over cast-in-place construction:

- Mass production of standardized girder units is possible. This is done at present with precast I-girders for shorter spans.
- High quality control can be attained through plant production and inspection.
- Greater economy of production is possible by precasting the girder units at a plant site rather than casting in place.
- Erection speed is greater. This is very important when construction interferes with existing traffic and is most critical in an urban environment.

In segmental box girder construction utilizing the cantilever erection procedure, precasting has several other advantages over cast-in-place construction:

- Strength gain of the concrete is essentially removed from the construction critical path. This allows faster erection times and higher concrete strength at the time of stressing.
- Shrinkage strains can substantially develop prior to erection and stressing if adequate lead times and stockpiling are used.
- Creep rates can be substantially reduced since the segments are considerably more mature at the time of stressing.

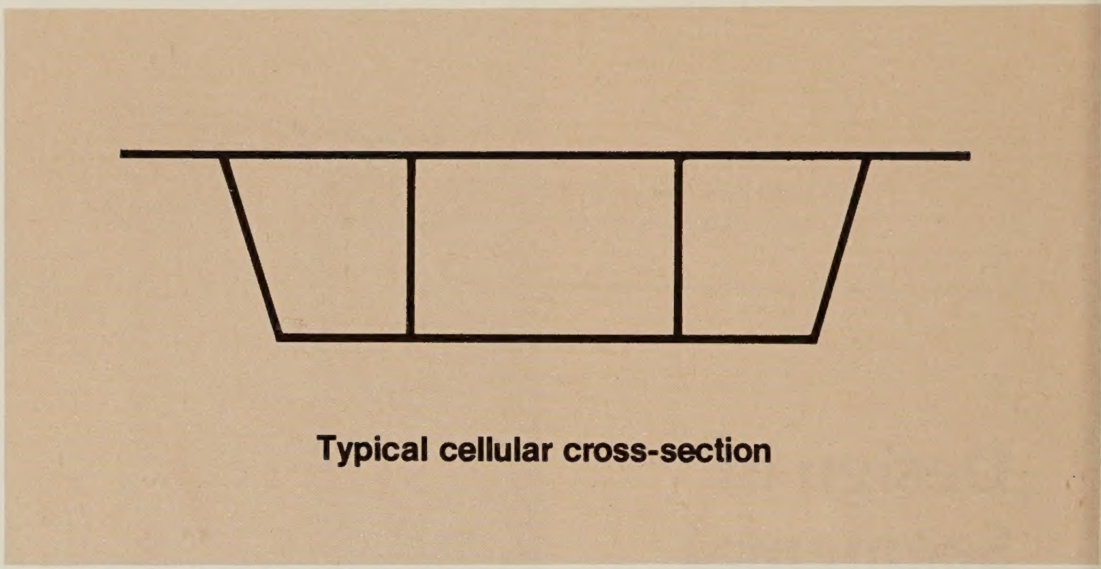


Figure 1.—Typical cellular cross section.

The major advantages frequently cited for utilization of cast-in-place segmental construction are as follows:

- Provision of positive nonprestressed reinforcement across the joints is easier.
- Continuous correction of girder grade and line is possible, to compensate for deformations.

Epoxy joints, grouted tendons, and shear keys have reduced dependence on the positive bonded joint reinforcement, while the versatility of the match casting procedure on numerous major projects involving complex horizontal and vertical alinement has shown that the precast procedures can deal with geometrical problems.

Currently, precast I-section girders are widely used in highway bridge construction for spans up to about 120 ft (35 m). They are cast in a manufacturing plant and transported to the bridge site for erection. Although the span length may be stretched by using "drop in" girders, the length of the girders may not be significantly increased because of transportation limitations. Also, a box girder is a better structural shape for long spans than an I-section girder.

The box girder is very compact, combining high flexural strength with high torsional strength and stiffness. It is superior to the I-section girder for long spans in that there is no lateral buckling problem so the compressive capacity of the bottom flange is fully utilized, and the torsional rigidity bring about a more even distribution of flexural stresses across the section, under a variety of live loads. A further advantage of the box girder in precast structures is that it is possible to precas the full cross section (apart from a longitudinal joining strip in some cases), whereas with I-sections the dec slab must largely be cast in place.

A considerable number of long span prestressed concrete box girder bridges have been constructed throughout the world, using both cast-in-place and precast construction techniques. The Bendorf Bridge in West Germany demonstrates the suitability of box girders even for extremely long spans; i was cast in place and has a span of 682 ft (208 m). In the United States cast-in-place box girder bridges are being widely used by the California Division of Highways, as well as in several othe States.

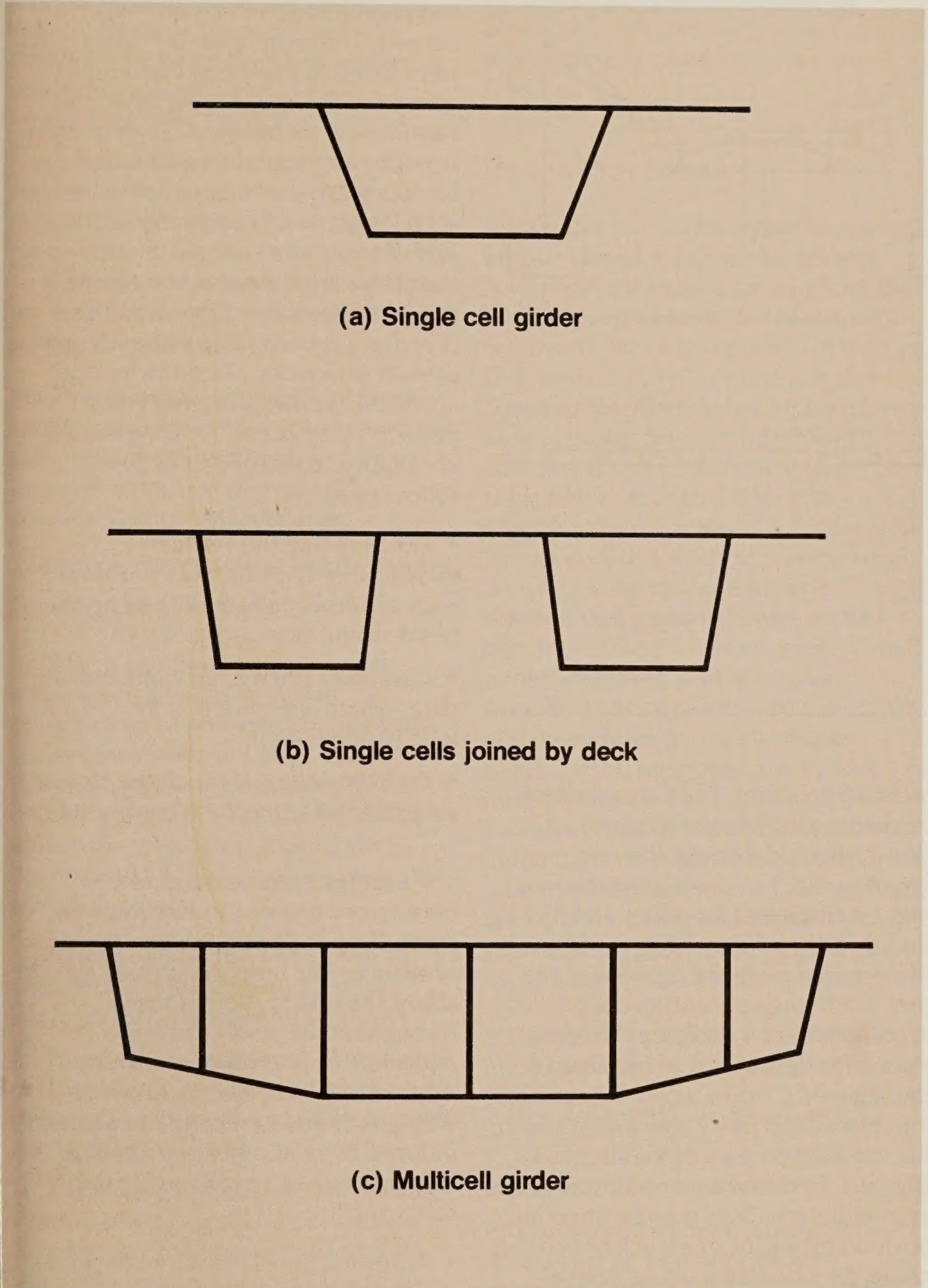


Figure 2.—Girder cross-sectional configuration.

When box girder bridges are precast, the casting is generally segmental, that is, the girders are cast in short, full-width units or "segments." Construction of box girders in short segments is necessary because box girders, unlike I-girders which have narrow widths, cannot be readily transported in long sections. In addition, the short units are suited to fairly simple methods of assembly and post-tensioning. During erection the segments are joined together, end to end, and post-tensioned to form the completed superstructure. The length and weight of the segments are chosen so as to be most suitable for transportation and erection.

Segmental Construction

Figure 2 illustrates three of many cross-sectional shapes which can be used. Various techniques have been used for jointing between the precast segments, with thin epoxy resin joints the most widely used. The most significant variation in construction technique is the method employed to assemble the precast segments. The most widely used methods may be categorized as construction on falsework and cantilever construction. Construction on falsework is the simplest method of erecting precast, segmental bridges. It also provides the simplest design approach. The joints used are usually cast-in-place concrete or mortar. This method is advantageous at locations where access by construction equipment is difficult, the project is of limited scope and traffic interruptions due to falsework are acceptable, and where single or twin spans are to be used so that balanced cantilevering is not feasible. The prestressing system for

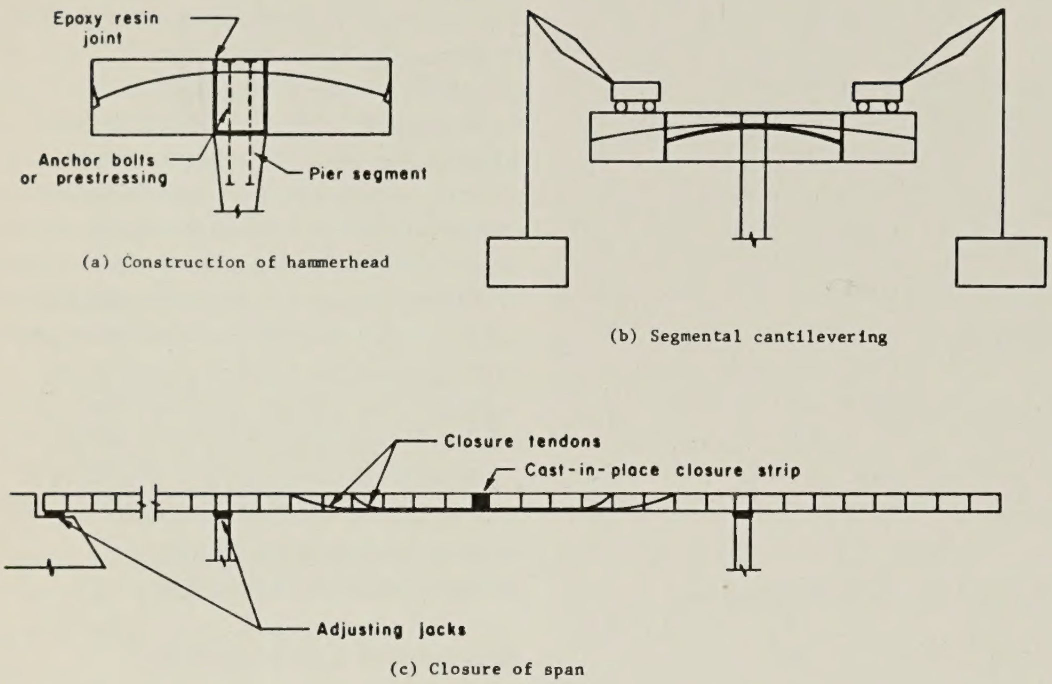


Figure 3.—Construction of complete span.

the superstructure will normally consist of long draped cables in the box girder webs. If the overall length of the bridge is moderate, it is possible to set all the segments in place and join them before inserting and tensioning full length cables. This method tends to economize on prestressing steel and hardware. Stressing operations are minimized, but falsework costs increase.

The outstanding advantage of the cantilever approach to segmental construction is that falsework is eliminated, greatly reducing traffic interruptions.

Assembly of the segments is accomplished by sequential balanced cantilevering outward from the piers toward the span centerlines. Initially the "hammerhead" is formed by erecting the pier segment and attaching it to the pier to provide unbalanced moment capacity. The two adjoining segments are then erected and post-tensioned through the pier segment (fig. 3a). Auxiliary supports may be employed for added stability during the cantilevering or to reduce the required moment capacity of the pier. Each stage of cantilevering is accomplished by applying the epoxy resin jointing material to the ends of the segments, lifting a pair of segments into place, and post-tensioning them to the standing portion of the structure (fig. 3b). Techniques for positioning the segments vary. They may be lifted into position by means of a truck or floating crane, by a traveling lifting device attached to (or riding on) the completed portion of the superstructure, or by using a traveling gantry. In the latter case the segments are transported over the completed portion of the superstructure to the gantry and then lowered into position.

The stage-by-stage erection and prestressing of precast segments is continued until the cantilever arms extend nearly to the span centerline. In this configuration the span is ready for closure. The term "closure" refers to the steps taken to make one continuous span from the two independent cantilever arms between a pair of piers. In earlier segmentally constructed bridges there was no attempt to ensure such longitudinal continuity. At the center of the span, where the two cantilever arms meet, a hinge or an expansion joint was provided. This practice has been largely abandoned in precast structures, since the lack of continuity allows unsightly creep deflections to occur. (*4, 5*) Achieving continuity usually involves the following items:

• Ensuring that the vertical displacements of the two cantilever ends are essentially equal and no sharp break in end slope exists.

• Casting in place a full width closure strip, which is generally from 1 to 3 ft long (0.3 to 0.9 m).

• Post-tensioning through the closure strip to ensure structural continuity.

The exact procedures required for closure of a given structure must be carefully specified in the construction sequence. The final step in closure is to adjust the distribution of stress throughout the girder to ensure maximum efficiency of prestress. Adjustment is usually necessary to offset undesirable secondary moments induced by continuity prestressing. The final adjustments may involve the following:

• Adjusting the elevation of the girder soffit, at the piers, to induce supplementary moments. This may be done by means of jacks inserted between the pier and the soffit of the girder with subsequent shimming to hold the girder in position.

• Insertion of a hinge in the gap between the two cantilever arms to reduce the stiffness of the deck while the continuity tendons are partially stressed. The hinge is subsequently concreted before the continuity tendons are fully stressed.

• A combination of hinges and jacks inserted in the gap to control the moment at the center of the span while the continuity tendons are stressed. The final adjustment is made by increasing the jack force and finally concreting the joint in the span. (*4*)

The first of these options is the most widely used. After final adjustments are complete, the operation is moved forward to the next pier and the erection sequence is repeated.

Research Program in Segmentally Constructed Bridges

In 1969, a comprehensive research effort dealing with segmental construction of precast concrete box girder bridges was initiated at The University of Texas at Austin. This multiphase project had the following six objectives (objective completion is documented in the reference cited after each objective):

1. To investigate the state of the art of . segmental bridge construction (6) (subsequently updated by reference 2).

2. To establish design procedures and design criteria in general conformance with provisions of existing design codes and standards. (*7*)

3. To develop optimization procedures whereby the box girder cross section dimensions could be optimized with respect to cost to assist preliminary design. (*8*)

4. To develop a mathematical model of a prestressed box girder, and an associated computer program for the analysis of segmentally constructed girders during all stages of erection. (9)

5. To verify design and analysis procedures using a highly developed structural model of a segmental box girder bridge. (*10*)

6. To verify model techniques by observance of construction and service load testing of a prototype structure. (*11*)

Design Procedures

Although over 100 long span segmental precast concrete box girder bridges have been constructed throughout the world, acceptance of this concept has been slow. Since completion of the first U.S. project in 1973, interest in this construction method has increased; several bridge projects have been completed and others are nearing completion. Probably the most important of many reasons for the slow adoption of this method is the general division of the engineering and construction responsibilities in the concrete industry. The segmental box girder bridge requires extensive consideration of construction methods and procedures during the design phase. In the same way, the erector must be responsible for substantial calculations for control of stresses and deflections throughout the erection phase. While such interaction has been very common in construction of long span steel bridges, it has not been as usual in long span concrete structures.

A segmental box girder bridge design must consider the following:

• Constructibility of the project.

• Provision for competitive systems and constructor improvements.

• Structural stability in all embryo stages and performance of the completed structure.

Responsibility for design and construction must be divided with great care to ensure that the constructor is not forced into undertaking an unrealistic or unsafe construction procedure by orders of the designer and, conversely, that the designer is not responsible for errors made by the contractor.

One important reason for the slow development of this type of construction in the United States is the need for efficient construction procedures which are suited for the engineering, construction, labor, and material practices of the United States. Substantial emphasis should be given to research and development (R&D) studies to develop specifications and recommended practice guides which are applicable to U.S. conditions.

State of the Art

Report 121–1 summarized the state of the art in precast segmental box girder technology as of 1970. (6) Since then there have been rapid developments in this technology. Foreign experience by one of the world's foremost builders of this type structure was summarized in 1974 by Jean Muller. (*2*) One of the most interesting aspects of the developmental period of the last decade has been the evolution of the jointing and erection process. Epoxy joints are still the foremost type of jointing, but less reliance is being placed on the strength of the epoxy and more jointing surface is being provided for mechanical interlock keys between units. Muller shows pictures of recent French bridges with castellated or serrated web keys for a long portion of the web length. The multiple key designs decrease reliance on long term epoxy integrity and should be carefully studied and experimentally verified. Similarly, numerous projects are using procedures that move the negative moment (cantilevering) tendon anchorages out of the web end for locating anchorages. By moving the anchorage from the end surface of the unit, several units can be placed using temporary fasteners before stressing has to be accomplished. In this way, threading of cables and stressing of tendons has been removed from the critical path operations in erection. Detailed analysis procedures and proof testing of such anchorage methods are needed.

Another important aspect of foreign practice is the use of wider sections resting on single piers, rather than double box sections supported by parallel twin piers. Although the cost of the superstructure is somewhat higher for the wider single section, substantial savings have been achieved in pier costs.

The large number of current U.S. projects, as summarized by Koretzky and Kuo, indicates that this type of construction is emerging rapidly. (*12*) Their 1973 survey indicates that 56 bridges are under consideration and/or design in 16 States.

Design Sequence

The design sequence for a segmental precast concrete box girder bridge is a highly interactive one, primarily because the designer must consider the procedures and limitations of the construction techniques which might be used. Figure 4 shows the various stages of the design sequence and the usual paths between sequences. The main elements are as follows:

• Conceptual design—basic decisions regarding type of construction, span lengths and ratios, and cross section types.

• Preliminary design—choice of basic dimensions for cross section elements, tendon and reinforcement patterns, slab and web thicknesses, and optimization studies of the span and cross section layout. Analysis procedures are usually approximate.

• Detailed design—specific proportioning of a tentative cross section considering both construction sequence loads and normal design loads on the completed structure, and planning of the erection and closure sequences. Relatively detailed analysis to consider all major loads and conditions which will affect behavior of a structure.

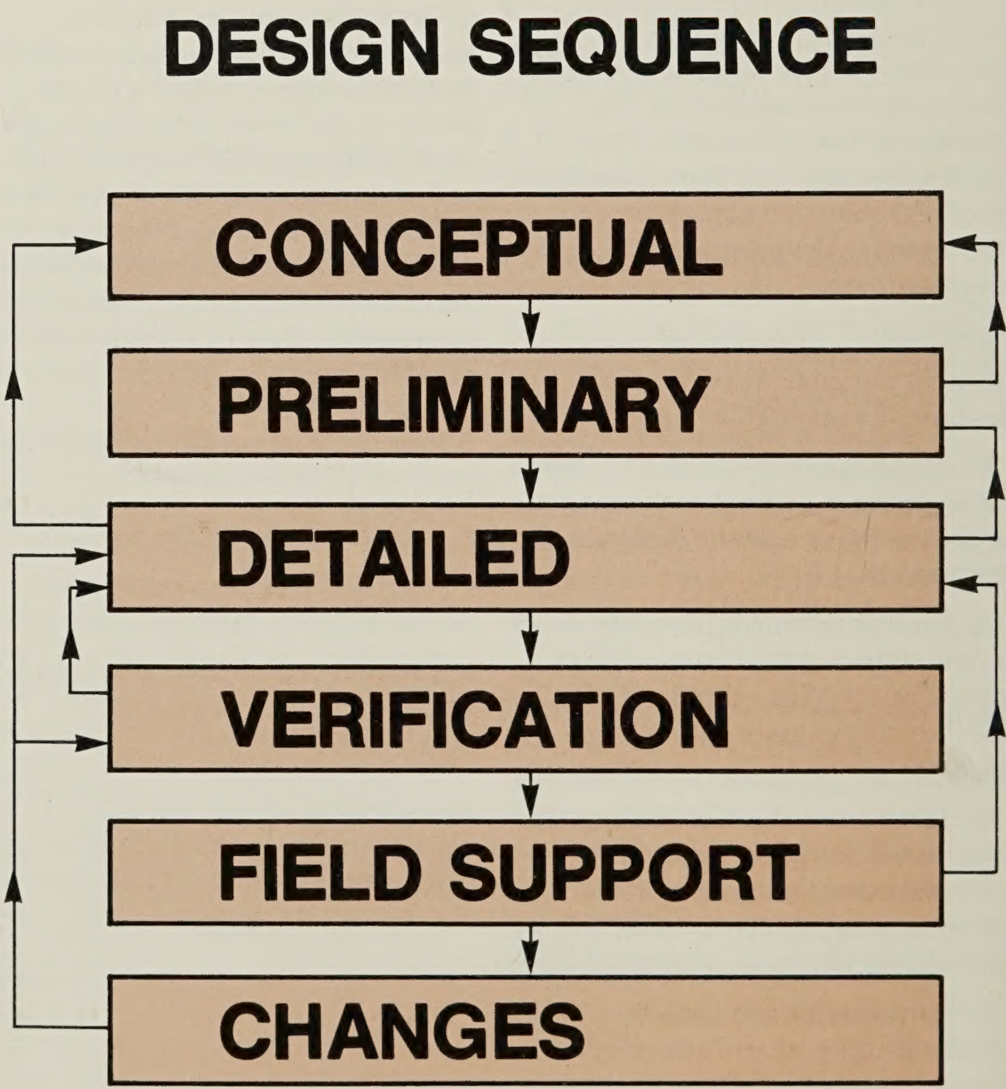
• Verification analysis—studies undertaken after most elements of the design are substantially fixed to check construction stresses and deformations and behavior under all critical design load conditions.

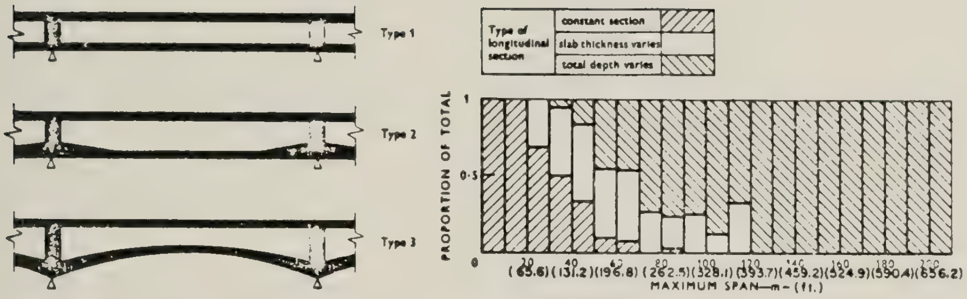
• Field support analyses—checks of working drawings, contractor's erection stresses, detailed stressing sequences, and development of deflection and closure information for guidance of field forces.

• Change order evaluation—providing rapid information to field forces and contractor on technical advisability of proposed changes in design requires quick response in technical decisions.

Specific recommendations for needed research and development studies for each of these stages will be developed in the following sections. The large number of interactions indicated in figure 4 implies that such a breakdown is extremely artificial, because the same person frequently handles several of the items in the sequence. The schematic is useful in organizing a discussion of the important elements in the design sequence.

Figure 4.—Interactive design sequence.





(a) Types of longitudinal section

(b) Distribution of longitudinal section types

Figure 5.—Types of longitudinal profiles. (*13*)

Span	Bridge Type
0–150 ft	I-type pretensioned girder
125–300 ft	Precast segmental constant depth
275–450 ft	Precast segmental variable depth
400–600 ft	Cast-in-place segmental
600–1,200 ft	Cable-stayed with precast segmental girders
1,200 ft up	Suspension

1 ft = 0.305 m

Conceptual design

The most important decisions to be made in the design of segmental bridges are generally made at the start when major questions have to be answered with relatively little hard information. Decisions usually involve span lengths, span ratios, box girder versus alternate structural system, cast-in-place versus precast, erection on falsework versus cantilever erection, single box versus multiple box versus multicell cross section, and constant depth versus variable depth.

These important decisions are best made after a careful review of the state of the art, consultation with experts who have been involved in the design and construction of successful projects, and intensive study. There is considerable information available to assist in these decisionmakings. Excellent summaries by Muller (*4*) and Swann (*13*) as well as a summary by Lacey, Breen, and Burns (*3*) describe many successful projects and can help one develop a feel for the "possible." In particular, the compilation by Swann of detailed dimensions of 173 concrete box girder bridges (segmental, nonsegmental, precast, and cast-in-place) is very useful. (*13*) Figure 5 illustrates the distribution of constant section, constant depth with variable slab thickness, and variable depth bridges reported by Swann. As in all studies, this distribution must be examined carefully, since it contains a variety of experiences. The majority of the short span structures were not built segmentally. In addition, most of the structures are located in Europe and are undoubtedly colored by the design criteria and economic experience of that region. A useful addition to U.S. practice would be analytical studies based on realistic cost data to indicate such trends for American practice.

Discussion with several other designers of segmental bridges indicates that the following rough "rule of thumb" represents the current state of the art:

Obviously, such a rule of thumb is only a crude indicator of the appropriate type structure. Decisions between use of precast and cast-in-place segmental units involve not only span length but ease of access to the site for heavy handling equipment, seasons during which construction will be underway, and project size.

Segmentally precast box girder bridges may be classified into two main types according to the method of erection, namely those constructed on falsework and those erected in cantilever. The third method, assembly on shore, will generally be too cumbersome to have widespread use.

Each construction method required tailored prestressing cable patterns and design procedures. In bridges constructed on falsework, long draped cables, traversing one or more spans, can be used. If the cables run the full length of the bridge, only one structural system—the completed continuous superstructure—need be considered in design.

For bridges erected in cantilever, a set of cables in the top of the girder is required for each length of the cantilever arm. Each stage of erection constitutes a separate design condition, with different bending moments in the cantilever. The completed superstructure contains additional cables in the bottom of the girder and constitutes an indeterminate continuous system. It is designed to withstand the dead and live loads under service conditions.

Erection on falsework with close-spaced supports is the simplest method of construction when conditions permit, as in the case of viaducts over land not passing over existing roads. Lifting and placing techniques will depend on the exact site conditions. For bridges having three or more spans over water or over existing roads, where intermediate support is not possible, the cantilever method will probably be the most suitable. However, there will be a critical span length below which it will be more economical to use a falsework truss. For two-span bridges over an existing highway, erection on a falsework truss or girder or with temporary braces or ties is probably the simplest procedure.

The superstructures of the box girder bridges generally conform to three main types: (1) single cell box girder, (2) pair of single cell box girders connected by the deck slab, and (3) multicell box girder. These types are sketched in figure 2. The simplicity, economy, and good appearance of these sections is evident.

Single cell box girders are generally used in relatively narrow bridges. As the width increases, the bending moments in the deck slab increase and hence the thickness must increase. Beyond some critical width it becomes more economical to use a multicell box or multiple single cell boxes.

In the case of multiple single cell box girders, the basic single cell units are cast separately and are connected after erection with a concrete joint. Usually the deck is post-tensioned transversely, but it is possible to use nonprestressed reinforcement only and to make the

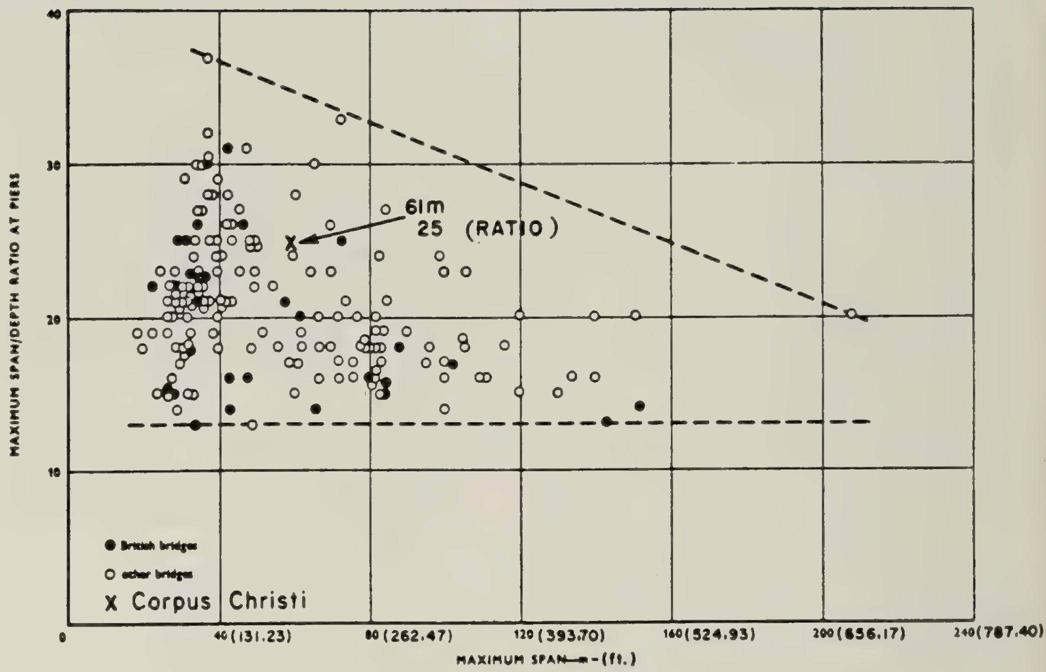


Figure 6.—Span/depth ratios. (13)

joint width sufficient for splicing. In general, it is possible to have smaller basic units with multiple single cell boxes than with a multicell box girder. The smaller units are easier to transport and erect. The bridge can be easily widened by the addition of another box. On the other hand, with a multicell box the cast-in-place longitudinal joint is not required. Also, a multicell box of relatively small base width may be advantageous when narrow piers are desired.

Preliminary design

In the preliminary design stage, the important structural parameters are determined. Such factors as span-to-depth ratios, minimum web thickness, upper and cantilever flange thicknesses, and preliminary tendon requirements can be fairly readily determined by conventional elastic analyses, determination of cantilever moments prior to closure, and utilization of normal or "beam" theory for stress analysis of sections.

The recent report of the Prestressed Concrete Institute Committee on Segmental Construction suggests span-to-depth ratios from 18:1 to 25:1 are currently considered practical and economical for constant depth segmental bridges. (*14*) They suggest that variable depth bridges may have span-to-depth ratios of 40 to 50, based on the depth at the center of the span. Figure 6 indicates a wider variation in the span-to-depth ratio. (*13*) The optimizing studies reported in Report 121–3 (*8*) indicate that very efficient structures can be obtained in the 25 to 30 span-to-depth ratio range. Past experience, as reflected in figure 6, may be colored by the much heavier live loads used in European design. Figure 7 is from a study by Rajagopalan which indicates that for 140-ft (43 m) spans, live load design moments in some European countries will vary from 150 to 300 percent of those used in the United States. (*15*) Use of lower span-to-depth ratio values are indicated when shears are heavy, little load balancing is utilized, or when preliminary design indicates extreme congestion of tendons. The experience with the Corpus Christi segmental bridge, which had a span-to-depth ratio of 25, indicates that even higher values could be used without substantial deflection difficulty. Analytical programs based on optimization techniques are currently available for use in parameter studies which could develop design aids reflecting American cost trends. Such studies would be a worthwhile R&D investment.

In many structures the web thickness will be based more on "placeability" considerations and providing adequate room for anchorages than on shear considerations. Based on successful French experience, the Corpus Christi structure was designed with a minimum web thickness of 12 in (305 mm). In retrospect, the congestion of the webs hindered placement and made detailing of anchorages difficult. Figure 8a shows a web thickness parameter for a wide range of bridges. (*13*) It can be seen from the calculations of figure 8b that the value of the parameter for the Corpus Christi bridge is one of the lowest, with a value of 2.88×10^{-3}. Retrospect would indicate that the webs should have been increased to about 14 in (355 mm) minimum, which would give a parameter value of 3.36×10^{-3} and essentially plot on Swann's curve. This indicates that such a graph could be quite useful in preliminary design.

While many of the cross-sectional elements can be designed utilizing normal slab design under specifications of the American Association of State Highway and Transportation Officials, the lower flange near the piers in narrow bridges is very critically affected by the cantilever moments and particularly

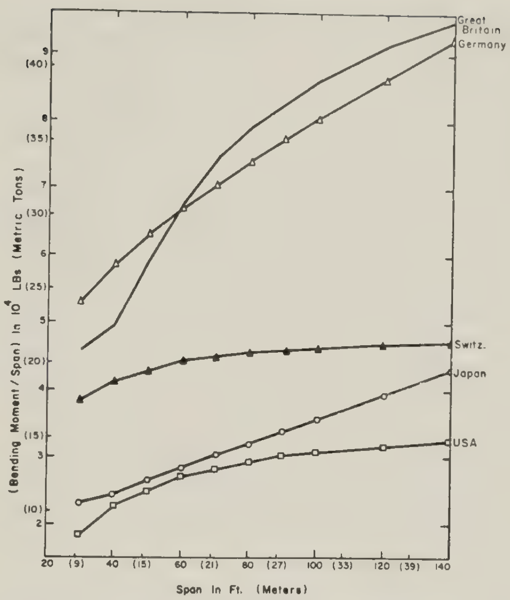


Figure 7.—Bending moment/span versus span in various countries. (15)

span-to-depth ratios. This slab often has to be thickened and may indicate the desirability of a greater cross section depth to increase the lever arm and cut down the thickness of the lower flange. This will be more prevalent on double box cross sections than on single box cross sections.

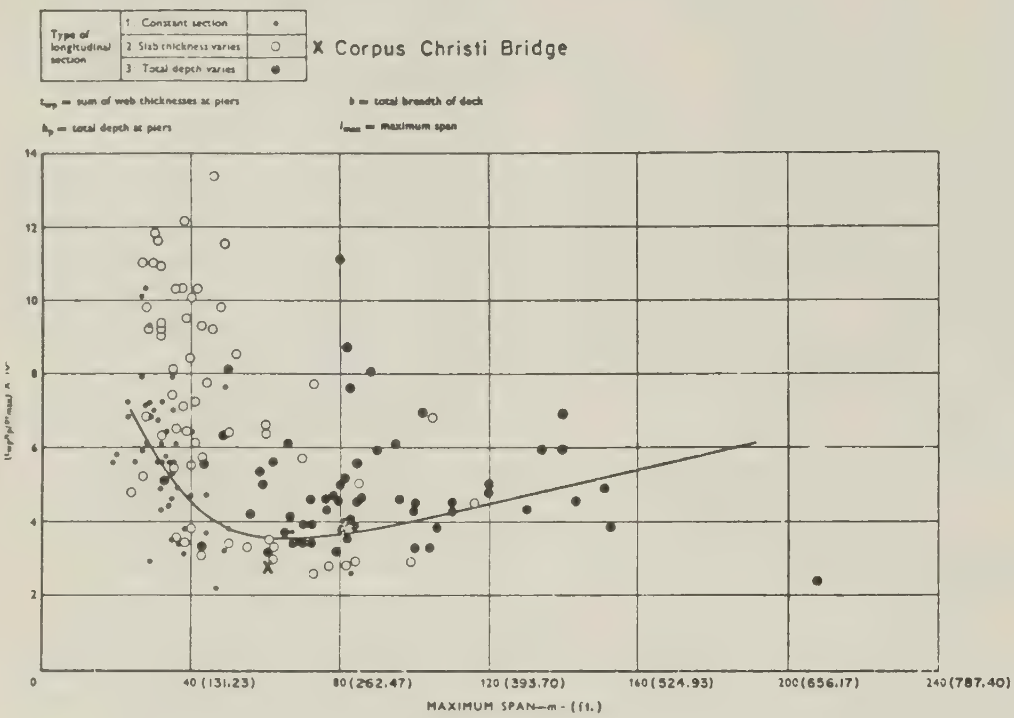
Detailed analysis

After a basic construction scheme, span arrangement, cross sectional type, and important section properties have been at least preliminarily decided upon, a detailed analysis can be made to determine tendon sizes and patterns, flange and web thicknesses, transverse and shear reinforcement, and stressing details. For the initial detailed analysis, ordinary equilibrium equations, elastic analysis, and normal "beam" design procedures are utilized. In many box girders, there will be substantial deviations from such stresses due to shear lag, section warping, and torsion due to unsymmetrical loading. After completion of a detailed design involving both construction and normal live load effects, it is advisable to check the structure with a "folded plate" type analysis. The author recommends the use of computer analysis programs such as MUPDI for constant depth sections or FINPLA2 for variable depth bridges. These programs were developed by A. Scordelis at The University of California under sponsorship of the California Department of Transportation and are widely available. A major contribution to bridge design would be a comprehensive parameter study using these programs to indicate span and cross section relationships where "beam" design procedures are adequate.

In Report 121–3, comprehensive examples of a segmental box girder erected on falsework and a structure erected by balanced cantilever were used to illustrate typical design procedures. (*8*) These examples illustrate the interaction between the preliminary and the detailed design phase and typical changes made in the detailed design phase to satisfy normal design requirements.

Verification analysis

Particularly when cantilever erection is to be used, it is advisable to run a check analysis which will verify the suitability of the proposed construction sequence and check for stresses and deflections to be expected during all stages of erection. In order to facilitate such an analysis, a program, SIMPLA2, was developed for constant depth box girders. Detailed information is given in Report 121–4. (9) Such a program can be used to determine longitudinal and transverse stresses, deformations, tendon friction losses, tendon incremental stressing losses, and to track the structure through all unbalanced states and closure operations. The program uses a "folded plate" analysis and so also gives indications of excessive shear lag or other effects. Because of the complexity of inputting the problem into this program and the



$$\frac{t_{wp}\,h_p}{b\,L_{max}}\,(10^{-3})$$

t_{wp} = sum of web thickness at piers

h_p = overall depth at piers

b = overall width at deck

L_{max} = maximum span

t_{wp} = 48 in. = 122cm = 1.22m

h_p = 8 ft. = 96 in. = 2.44m

b = 56 ft. = 17.2m $\quad \therefore \quad \dfrac{(1.22)(2.44)}{(17.2)(60)} = 0.00288 \;\therefore\; 2.88 \times 10^{-3}$

L_{max} = 200 ft. = 66 yd = 60m

Figure 8.—Web thickness parameter at piers (a). Corpus Christi Bridge parameter calculations (b). (13)

high cost of the analysis, it is ordinarily only undertaken at the completion of the design as a final check.

In a similar way, a final check of any structure where substantial shear lag or warping effects are suspected is advisable to verify all design load conditions. The MUPDI program is an excellent one, and indicated very high correlation with the measurements in the companion test program involving a model study of the Corpus Christi Bridge.

Further parameter studies should be undertaken to determine where "beam" analysis procedures are adequate for erection stages and where the more involved erection analysis procedures of the SIMPLA2 type program are necessary. The program should be extended to handle variable depth.

Dimensions of successful projects are often one of the best indicators of the practical marketplace. However, the use of more formal optimization techniques can indicate important trends to be investigated in design. In Report 121–3, an attempt was made to illustrate how relatively simple optimization techniques can be used in preliminary designs to give the designer information as to the cost "trade offs" of his or her basic parameter decisions. (*8*) Unfortunately, these optimization examples include only a relatively narrow number of span lengths and roadway widths. These studies should be extended to give more information as to the effect of variations in these important parameters. There seems to be a systematic relationship between length and width in the choice of cross section (fig. 9). The relatively narrower bridges go to single box units, while the wider structures go to twin box units. Further studies of variables would clarify the practical boundaries for these decisions.

Field support

As specified by the contract, the designer, the owner, or the contractor will have the primary responsibility for controlling the erection of the bridge. A substantial amount of technical information will be needed as a basis for making quality control checks. Upon completion of the design and award of the contract, detailed information on erection stresses, deflection profiles at various stages, tendon stressing patterns and limits, tendon elongation values, and closure computations will have to be developed and transmitted to the responsible parties. Many of the procedures are repetitive and a computer analysis is often advantageous. Because of the complexity of input for the SIMPLA2 program, it may be advisable to utilize simpler "beam theory" programs to develop the less critical values where possible. Such simpler programs should be based on a guideline or recommended practice.

It is especially important that working drawings be cross-referenced and compared so that careful coordination exists in placing reinforcing post-tensioning tendons and post-tensioning anchorages. It is advisable to develop high modularity in details to make maximum use of precast technology.

A systematic program of reporting problems encountered in the field and solutions should be developed to facilitate updating state-of-the-art reports.

Change order evaluation

After initiation of the work, numerous items will come up requiring technical decisions. Some of these will be major: for example, the contractor submitting a major revision in the tendon layout, stressing sequence, or erection plan. One of the great advantages of programs like SIMPLA2 is that they can be reprogramed relatively quickly to handle such proposed changes and give a complete reanalysis of all stages of construction. In this way the designer will be able to see the overall effect of the plan change in a clearer fashion. Continued emphasis should be given to development of such programs.

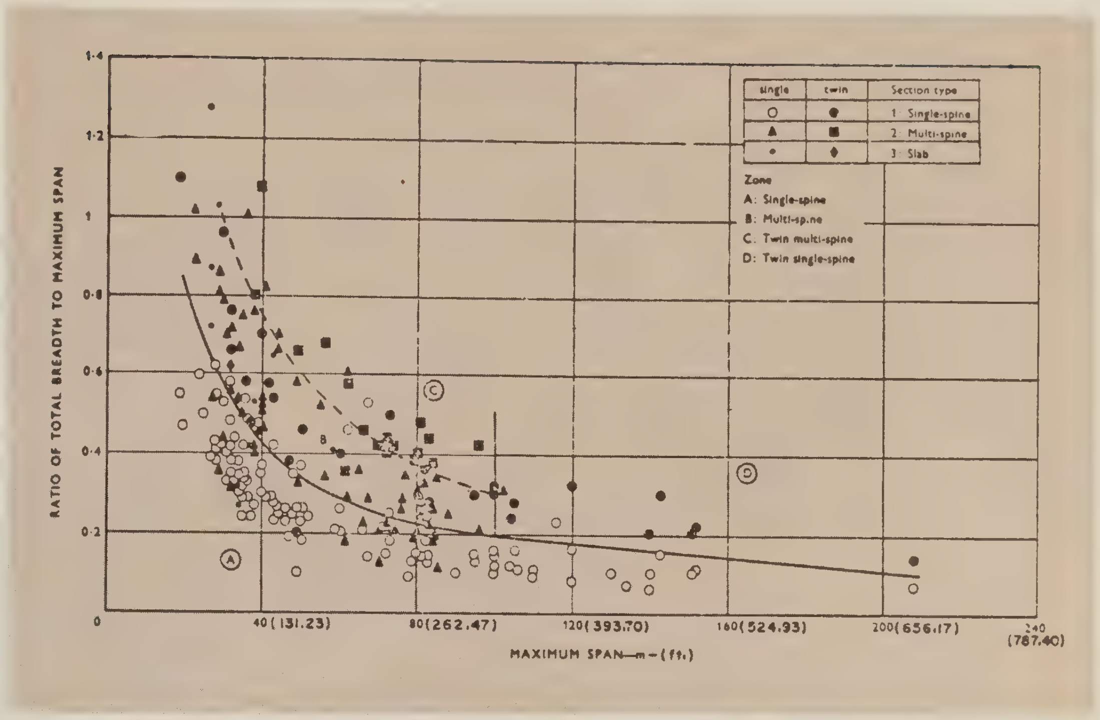


Figure 9.—Usage of basic section types.

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Motor vehicle - Drivers - Visibility
U.S. Federal Highway Administration

Reduced visibility caused by industrial effluents.

The Escalating War on Reduced Visibility

by Jerry A. Wachtel

The Federal Highway Administration's Office of Research has been concerned with the problem of reduced visibility on our Nation's highways for several years. Efforts to date have been concerned with identifying and understanding the problem, and we are now beginning to apply this growing knowledge to the development of measures to reduce accidents and other traffic operations problems caused by restricted visibility. This article, the first of two, will report on the causes of reduced visibility; the nature and extent of the problem; the impact of reduced visibility on motor vehicle operators; and some of the techniques which have been developed to deal with the problem. The second article, which will appear in a future issue of *Public Roads,* will report on the findings of a study currently underway in this area and discuss plans for the future.

Introduction

The term "reduced visibility" has traditionally been associated with fog, and fog continues to be a major contributor to highway problems. Meteorologically, fog is a condition which reduces visual range to 3,300 ft (1 km) or less. (*1*)[1] Dense fog is defined as that which reduces visual range to 1,300 ft (0.4 km) or less. Figure 1 shows the distribution of dense fog in the United States. It can be seen that such fog occurs on an average of more than 40 days per year along the west coast, much of the Northeast, the Appalachian Mountains, and portions of the gulf coast. Dense fog is rarely a problem in the Southwest and Plains States.

[1] Italic numbers in parentheses identify the references on page 118.

A recent survey of the States (as well as many foreign nations) indicated that 38 States had what they considered to be serious reduced visibility problems on the highway. (*2*) Serious was defined as those conditions requiring some form of remedial aids. A somewhat surprising finding was that, whereas fog was reported as a major cause of reduced visibility, many other natural and manmade factors also caused serious restrictions. Several of the Rocky Mountain and Northern Plains States, for example, had visibility problems caused by blowing snowstorms called "white-outs." Some Southwestern States, particularly Arizona, were afflicted with summer duststorms (known as "haboobs") which are swept across the desert by gusty winds. Several other causes of reduced visibility—including burning sugar cane, smoke from roadside fires, and industrial effluents—were also reported.

It would seem that the specific locations of many of these hazards could be readily identified, and the problems thus overcome. This is true in some cases, as when smoke or smog crosses the highway from an industrial source. But in the case of natural hazards, which predominate over manmade causes, authorities may have great difficulty localizing the problem to within a 20 or 30 mile (30 or 50 km) highway section. Even within these confines, the area of reduced visibility can be so small and can be shifted so rapidly by wind that accurate location prediction is impossible. Figure 2 demonstrates the patchiness of fog. Whereas some areas of the bridge roadway are clear, other areas only a few hundred feet away have severely restricted visibility. Not only does the localization and rapid change of the hazard condition create a problem of prediction for authorities, but such conditions may be more of a hazard for the motorist than a more widespread restricted visibility condition.

Consequences of Reduced Visibility

The most publicized consequence of reduced visibility on the highway is the multiple-vehicle chain reaction accident that occasionally results (fig. 3). Such accidents often result in substantial property damage and personal injuries and fatalities. In absolute terms, the number of accidents caused by reduced visibility is probably small. The actual figures are unknown due to differences in accident reporting techniques and definitions of reduced visibility, but it has been estimated that fog-related accidents may claim as many as 1,000 lives and cause 60,000 injuries per year on our Nation's highways. (*3*)

Figure 1.—Areas with some dense fog per year—1/4 mile (0.4 km) or less visibility.

The pattern of fog-related accidents is rather unusual. First, although the presence of fog tends to *reduce* both the accident and the fatality rate on conventional roads, the opposite occurs on freeways. An early California accident study showed that the fatality rate for fog accidents on freeways was nearly twice that for non-fog conditions. (*4*) Second, the probability of a multiple-vehicle accident is greater in fog than in clear weather. Nearly two-thirds of all accidents involving nine or more vehicles occur in fog. (*5*)

Driver expectancy is the most likely explanation for these reversals in freeway safety records under conditions of reduced visibility. Because freeways are typically wide, smooth, well delineated, and without intersections or sudden changes in geometry, the motorist may tend to become overconfident and drive at speeds greater than his or her actual sight distance dictates. The problem of overconfidence is a real one because of its interaction not only with the reduction of sight distance but also with the motorist's degraded ability to see and interpret those visual cues which he or she normally uses in driving without conscious attention (such as other vehicles, pavement markings, and roadside delineators).

This degraded visual ability in such conditions as fog, smoke, dust, and blowing snow is due primarily to the presence of one or more of three factors. The first, lack of contrast, is perhaps the most common and significant. Driving is predominantly a visual task, and much of the driver's visual information comes from the contrast between target objects and their visual background. For example, pavement markings are white or yellow against a dark surface; pedestrians and cyclists wearing bright, reflective clothing at night can be detected with relative ease; and the outlines and lights of vehicles (during the day and night respectively) help guide drivers. Without adequate contrast, objects are more difficult to distinguish. This is the phenomenon which occurs most frequently in fog, and to a lesser extent in other adverse weather conditions.

The second factor which degrades the driver's visual ability is glare. This is due to the breakup or reflection of light sources into the viewer's eye, and it is present mostly in fog and in rainstorms.

The third factor causing visual degradation is impaired distance judgment. One of the visual aids that humans unconsciously use in judging distance is the clarity or sharpness of an object. Due to the presence of particulate matter in the Earth's atmosphere, objects that are far away from the observer appear not only smaller, but less distinct than those that are close. The presence of fog, dust, snow, rain, or industrial pollutants in the atmosphere tends to confound this cue to distance. Thus, while following a car in fog, the driver may perceive the vehicle ahead to be farther away than it really is, because the vehicle's outline is less distinct than it would be in clear weather. Under this illusion, the following driver may increase or at least maintain his speed, until suddenly the lead vehicle seems to "jump out at him" from the fog. The proximity of the two vehicles suddenly allows the cue of size to overcome the optical illusion. Although this phenomenon has not been systematically explored with regard to driving in reduced visibility, it is likely that it is of considerable importance as a direct cause of accidents under such conditions.

Methods to Overcome Reduced Visibility

Throughout the world many attempts have been made to overcome reduced visibility hazards. Some highway authorities have tried to eliminate the cause of the problem whereas others have sought to minimize the effects.

Methods of eliminating the cause of the problem include fog dispersal and dissipation. The vast majority of fog that occurs in the United States is of the so-called warm type—fog composed of water droplets above 32° F (0° C). Various methods of dissipation (including heating to evaporate the droplets and chemicals to alter their configuration) and dispersal (such as "brooms" to literally "sweep" the fog away from the road) have been tried with varying degrees of success. But such methods have little utility in the highway environment. Not only are these methods extremely expensive, but their usefulness is confined to clearing small areas for brief periods of time. They have been used somewhat successfully by the Air Force and the Federal Aviation Administration to clear airport runways, but they are not presently viable for use on the highway.

Examples of approaches toward minimizing the reduced visibility hazard are both more numerous and more promising. Fixed highway lighting, including high-mast installations, aids in revealing the general contour of the road—especially when the visibility restriction is not severe. Specialized lighting systems, such as airport runway-type lights set into the pavement, provide significantly enhanced delineation. However, some controversy surrounds their use. Some researchers believe that these systems will provide such strong positive guidance that they will increase the driver's confidence to an unrealistic level for the prevailing conditions. A system employing pavement inset lights has

Figure 2.—The Severn Bridge in Great Britain under patchy fog conditions.

been installed in Virginia; although there have been several installation and operational problems, subjective motorist response has been highly favorable. (6) A study of the effects of this system on traffic flow is presently underway.

Traffic platoons, or convoys, have found occasional use in various parts of the world. "Operation Fogbound," for example, has been in use for several years along fog-prone sections of Interstate 5, State Route 91, and other nearby facilities in California. Police, stationed in patrol cars at both fringes of the fog zone, stop vehicles prior to their entering the area of restricted visibility. When several vehicles are in the queue, the police car leads them, single-file, at slow speed, until they are safely beyond the fog. On February 28, 1975, several multiple-vehicle collisions occurred in fog near Corona, Calif., while convoys were operating. In its investigation of these accidents, the National Transportation Safety Board concluded the following:

Figure 3.—Multiple-vehicle chain reaction accident in Great Britain due to foggy conditions.

> First, the CHP (California Highway Patrol) did not activate enough units when the fog became hazardous. Second, most of the collisions involved vehicles that were not in convoys. The westbound convoy vehicles . . . were not damaged as severely, and their occupants were not injured as severely, as were the vehicles and occupants not in the convoys. Therefore, since the convoy procedure was not fully implemented at the first collisions, the effectiveness of the convoys cannot be fully evaluated. (*7*)

Radio advisories in the United States currently take the form of traffic and weather alerts during regular commercial programing. The listener may be warned of poor driving conditions in a certain area and asked to start out a few minutes earlier and to drive with extra care. Unfortunately, information presented in this way is not sufficiently specific either in time or location and is of little use to the motorist about to enter a fog bank at 55 mph (89 km/h) from a previously clear road. More promising are traffic advisory broadcasts that may be heard by selecting a particular radio frequency in the extremities of the AM broadcast band. Although such systems have seen growing use, many are still experimental. Further into the future, perhaps, are systems whereby ongoing programs can be interrupted by an emergency broadcast from police, rescue vehicle personnel, or traffic or weather authorities. Such systems have been used in Europe and, to a limited extent, in highway tunnels in this country. Citizens Band radio communications can provide timely and localized information about weather conditions ahead; however, not every vehicle is so equipped, nor do drivers monitor the band constantly.

Although other approaches have been tried with differing degrees of success, they are beyond the scope of this article. The major ones include lowered speed limits, road closure, and driver education.

All of the remedial aids discussed above have limitations which weaken their effectiveness as overall solutions to the reduced visibility problem. Lighting installations, for example, may be effective at night, but not during daylight; convoys require the presence of police in the right place at the right time; speed restrictions are difficult to enforce; and there may be legal problems associated with road closure or emergency radio broadcasts.

Reduced Visibility Guidance Systems

In an attempt to overcome many of these limitations, various types of reduced visibility guidance (RVG) systems have been developed in recent years. Although diverse in their sophistication, application, and cost, such systems have at least two common characteristics: (1) a detector to detect the presence of a problem, and (2) a subsystem to advise the motorist of appropriate action. At their hypothetical best, such systems promise a reliable, maintainable, cost-effective means to minimize accidents and delay time resulting from reduced visibility on the highway. At their worst, they do none of this. Presently, their promise remains largely unfulfilled.

Problem detection

As stated above, the first common characteristic of RVG systems is problem detection. In most cases, this implies some means of determining that a condition of degraded visibility exists, although some systems measure a parameter of traffic flow as the first indicator of a problem. The simplest (and least reliable) approach to the problem detection phase depends on police and weather reports. Reduced visibility detectors, which are produced in many forms, are more sophisticated. There are forward- and back-scatter units (fig. 4), and others using infrared and ultraviolet light and lasers. (References 8 and 9 include discussions of the relative merits of several such devices.) In essence, all such units measure the loss of light transmission in the atmosphere adjacent to the road.

Another approach is traffic flow detection. The philosophy behind this approach is that, if the goal is to determine what the traffic in the area is actually doing, traffic flow detection is a more direct measure of a developing problem—and hence a more reliable one. Unfortunately, its usefulness is probably limited to roads with high traffic volumes.

Motorist advisory systems

The second major component of an RVG system is the motorist advisory function. Most systems now in use or being developed use variable message signs to convey to the motorist some indication of the nature of the problem and a recommended response, generally a "safe" speed (fig. 5).

Figure 4.—Reduced visibility detector.

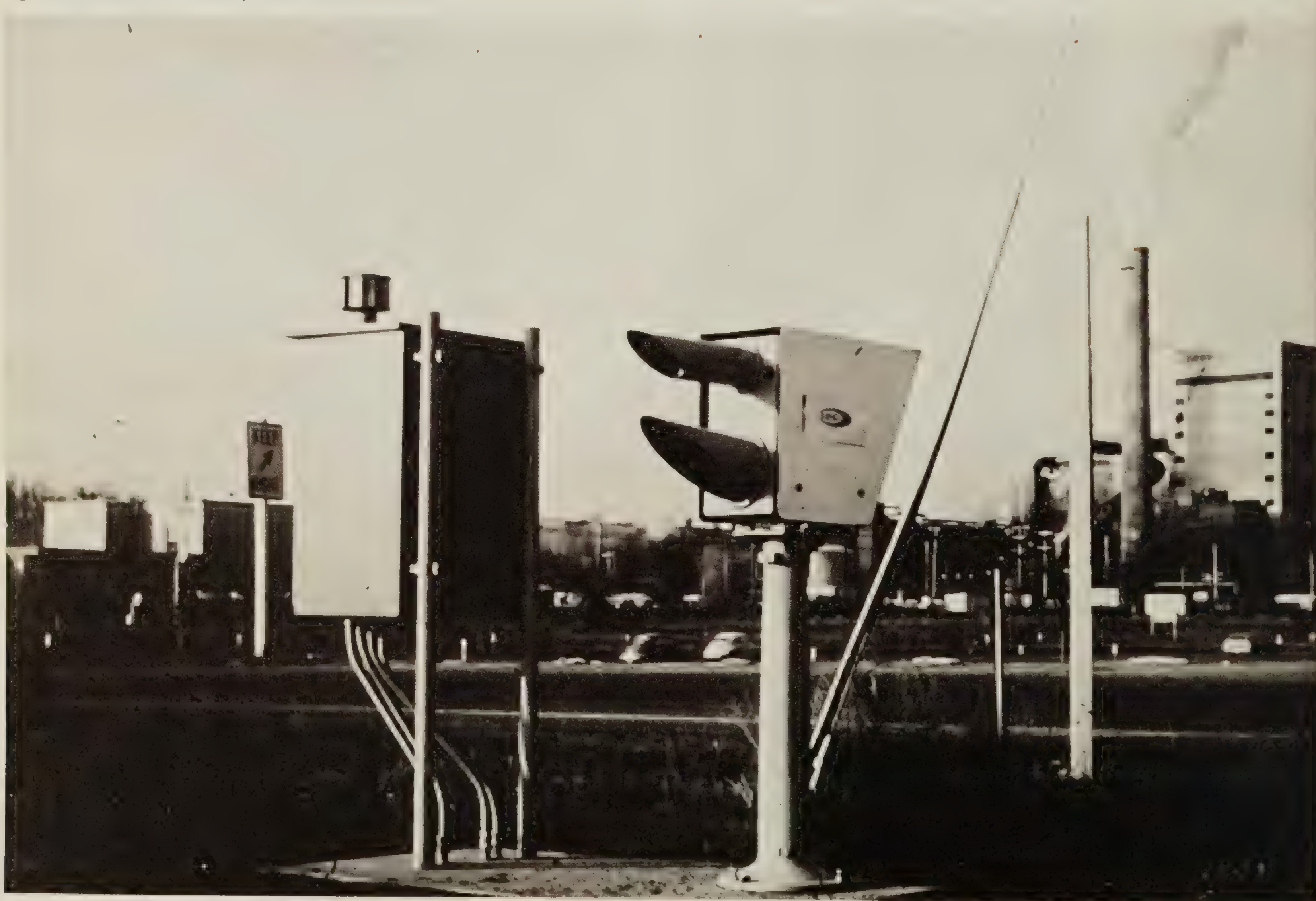

Figure 5.—Variable message signs in use in Arizona (a) and Oregon (b).

Problems With RVG Systems

As stated earlier, RVG systems have promised great results. Unfortunately, the results to date have been far from encouraging. Why have modern RVG systems often failed to solve the problem for which they were expressly designed? There are several possible answers. First, the visibility detectors themselves may present difficulties. These detectors are generally located alongside the highway, and are carefully adjusted and calibrated to measure visibility degradation in a specifically defined area. Fog, however, may be so highly localized that visibility is limited on the road, while a detector only 10 metres away detects clear conditions. The reverse situation can also occur.

Another problem of detectors is the difficulty of relating "visibility" measured objectively by a fixed roadside detector to the driver's subjective experience of seeing while moving through the hazard area. Additionally, roadside detectors are subject to vandalism; many must be frequently calibrated; and they may be affected by extraneous factors such as dust buildup on the optics, or a motorist stopping on the side of the road in the path of the detector's signal.

The second problem of many RVG systems is the signing. Figure 6 shows a changeable message sign that is of relatively little value. The sign (inset), in addition to being too small and surrounded by a myriad of other signs and the bridge structure, does not give the motorist enough information. How far ahead is the fog? How dense is it? What action should be taken? Signs such as this may be one reason why motorist conformance with displayed guidance information has not been high. The California Department of Transportation, which reported on the system utilizing this sign, recognized the sign's limitations, and suggested improvements for future systems. (9) Research underway will begin to tell us the right combination of sign message, speed, and display format to obtain the desired results.

A third major weakness in most RVG systems has been the "Cry Wolf" effect. This refers to the degree of confidence a motorist has that the system is giving him or her correct, up-to-date information for the present conditions. The picture in figure 7 was made during a test; if it had been a real situation, a motorist reading the sign under those visibility conditions would find it difficult to trust that display—as well as similar displays that he or she might encounter in the future. The "Cry Wolf" effect can occur for many reasons. Perhaps the signs are not activated or deactivated in a timely manner; perhaps the detectors which activate the signs are adversely affected by irrelevant stimuli; or perhaps the distance between signs, or between detectors and signs, is so great that visibility conditions change. Whatever the cause, the "Cry Wolf" effect must be eliminated if an RVG system is to succeed.

Figure 6.—Ineffective changeable message sign.

Figure 7.—Situation which could cause the "Cry Wolf" effect.

Several increasingly sophisticated RVG systems are being planned and installed. Yet we still do not know the relative merits of different system designs and configurations for different highway requirements. Accordingly, FHWA is increasing its research efforts in this area. Two administrative research contracts now underway deal directly with RVG systems development. Preliminary findings of the first of these studies will be reported in a future article in *Public Roads*. The war against reduced visibility hazards on the highway has not yet been won, but the battle is escalating.

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A Human Factors Assessment of Decreased Traffic Signal Brightness

by Harold Lunenfeld

Suggestions have been made to decrease night traffic signal lamp intensity as a fuel conservation measure. Proponents indicate that dimming signals will result in energy savings from reduced wattage and safety benefits from decreased glare. (*1, 2*)[1] There has not, however, been a detailed assessment of the operational implications of lower traffic signal brightness. Before adopting signal dimming as an energy saving measure, it is necessary to determine what effects the measure may have on the safety and efficiency of the highway system and to specify where it would be applicable.

This article considers decreased nighttime traffic signal lamp brightness in terms of these factors. Its emphasis is on the reception and use of information from round traffic signals and the effects of reduced brightness on visibility. Human factors considerations involved in error-free perception of traffic signal indications are discussed.

Background

A literature review on round traffic signal intensity requirements found little U.S. research. With the exception of one subjective evaluation (*3*), all empirical studies have been performed in Australia or Europe. Because of this, existing intensity information was developed from similar but not necessarily comparable drivers, signal lamps, environments, and other factors. Thus, many conclusions in this article were derived from adapting these data to U.S. conditions, and it is recognized that there may be areas of incompatibility. U.S. studies should be performed to identify and rectify differences and to fill in gaps in the data. These studies should also establish visibility requirements for arrow and pedestrian indications.

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[1] Italic numbers in parentheses identify references on page 125.

The standard governing U.S. traffic signal requirements is the Manual on Uniform Traffic Control Devices (MUTCD). (*4*) Section 4B–10 of the MUTCD specifies that an illuminated, unobstructed lens shall be clearly visible for a minimum distance of one-fourth mile (400 m) under normal atmospheric conditions. Section 4B–12(1) specifies minimum continuous signal installation visibility for various approach speeds as shown in table 1. These requirements are less than the 1/4-mile (400 m) signal lens visibility criteria and represent the minimum visibility of the installation at a given speed, taking into account approach geometry.

Table 1.—MUTCD signal visibility requirements (*4*)

85th percentile speed		Minimum visibility distance	
Mph	*km/h*	*Feet*	*Metres*
20	30	100	25
25	40	175	55
30	50	250	80
35	55	325	95
40	65	400	125
45	70	475	140
50	80	550	165
55	90	625	195
60	95	700	210

The MUTCD specifies the Institute of Traffic Engineers' (ITE) Standard for Adjustable Face Vehicular Control Signal Heads as the standard for intensity and light distribution. (*5*) This standard covers the three vehicular signal indications: an 8-in (200 mm) diameter, nominal 60-watt standard lens; and two 12-in (300 mm) diameter, 150-watt alternate narrow-angle lenses—a standard wide-angle and an allowed narrow-angle configuration. The transmission of the color filters is specified as 9.5 percent for red, 44 percent for yellow, and 20 percent for green. This yields the minimum intensities shown in table 2 for the maximum points on the lamp's distribution pattern.

Table 2.—ITE Minimum luminous intensity at maximum brightness points (2½° on either side of vertical, 2½° below horizontal)

Color	Lens		
	8-inch diameter standard	12-inch diameter standard—wide angle	12-inch diameter alternate—narrow beam
	Candelas	*Candelas*	*Candelas*
Red	157	399	808
Yellow	726	1,848	3,740
Green	330	840	1,700

1 in=25.4 mm

Perceptual Factors

Reaction time and the probability of detection are both dependent on signal intensity. These parameters are used to establish traffic signal brightness requirements. When assessing the effects of dimming, this article adheres to Fisher and Cole's perceptual criteria. (*6*) Rather than following a traditional perceptual threshold based on a 50 percent probability of detection, their signal perception criteria are based on a probability of detection approaching 100 percent accompanied by minimum reaction time. These criteria are in accordance with the traffic control functions of signals and correspond to the operational environments in which the signals must function. Failure of drivers to perceive an indication or to react quickly to its message could have an adverse effect on the safety and efficiency of a signalized location.

Driver visual capabilities

Although information used to assess traffic signal perception is derived from young drivers with normal vision and reaction time, it must be recognized that increased signal intensity is needed for aged and color-weak drivers.

Age: Fisher shows that reaction time increased by one-half between ages 25 and 65. He suggests that a 50-percent increase in signal intensity is needed to insure detection by older drivers. (*7*)

Anomalous Color Perception: Approximately 8 percent of the adult male population and 0.5 percent of the adult female population are color weak. Two percent of all male drivers experience reduced sensitivity to red and must have optimum signal intensity to insure red detection.

Color

Error-free traffic signal perception requires that drivers perceive and readily discriminate both the fact that a signal is on and its color. Traffic signals employ a simple three-color alphabet of red, yellow, and green. The chromaticity of colors has been established with color boundaries set to accommodate the majority of color-weak drivers. (*5*) Cole and Brown have shown the red signal to be most critical, insofar as detection is concerned. (*8*) The red indication sets the intensity of all traffic signals since yellow and green brightness levels are multiples of red.

Table 3.—Minimum daylight intensity for a round signal (6)

Color	Low-speed urban road	High-speed rural road
	Candelas	*Candelas*
Red	200	800
Yellow	600	2,400
Green	270	1,070

Optimum luminance

Fisher and Cole specify the intensities shown in table 3 as minimum daytime levels for a round signal on the line of sight. (6) Although U.S. lens transmission standards result in a yellow four times as intense as red and a green twice as bright, Fisher and Cole suggest that yellow should be three times as luminous and green one and one-third times as bright. (6) Research is needed to resolve these differences.

The intensities in table 3 are for photopic (light-adapted) vision. The values are based on a 100 m stopping sight distance, a 10^4 cd/m^2 background luminance,[2] and a signal in the line of sight. Fisher indicates that signal intensities would have to be increased fourfold if the signal is offset 30° from the line of sight. (9)

The minimum red signal intensities are specified for low-speed urban and high-speed rural roads. The low-speed levels were derived from a 65-watt, 240-volt, 200 mm lamp with a silvered-glass reflector. The high-speed levels were obtained from a 150-watt, 240-volt, 300 mm lamp. Although both lamps and lenses are similar to U.S. signal lamps, there are differences; studies should determine if the differences are significant.

[2]Daylight, clear sky.

Because of background luminances and a visibility distance considerably less than MUTCD requirements, it is necessary to adjust data to U.S. nighttime conditions. Since no other empirical evidence exists, the Fisher and Cole intensity criteria were used as a yardstick for assessing applicable U.S. night signal intensity requirements. (6)

Contrast

Signal detection largely depends on the difference between the brightness of a signal and the background luminance. The daytime, clear-day luminance of 10^4 cd/m^2 used to compute optimum signal intensities in table 3 is several orders of magnitude higher than most background luminance that will be found at night.

There is, however, a considerable range of night background luminances that may occur. Data are lacking to provide a specification of average background luminances as a function of road environment. Furthermore, there is so much variability because of such things as luminaires, outdoor advertising, traffic, and misaimed headlights, that a background luminance at any intersection, approach, or signal face may be completely different for the next intersection or for a different approach or face in the same intersection.

Table 4 presents a synthesis of data from several sources. (*10, 11, 12*) It shows a large range of intensity from moonlight (10^{-2} cd/m^2) to sun reflections on snow (5×10^5 cd/m^2).

Table 4.—Luminance levels for various backgrounds (*10, 11, 12*)

Background	Range of background luminances (cd/m^2)
Moonlight	10^{-2} to 10^{-1}
Street illumination (residential)	10^{-1} to 5
Street illumination (urban)	5 to 5×10^2
Natural daylight cloudy days	5×10^2 to 5×10^3
Sunny streets	5×10^3 to 10^4
Snow	10^4 to 5×10^5

The applicable range in assessing night intensity requirements is essentially the values for residential and urban area street illumination. It may be possible, however, to attain a background luminance equivalent to a cloudy day in some brightly lit urban downtown locations. In fact, Rutley, Christie, and Fisher state that in locations such as major urban central business districts (CBD's) where there is a high level of background lighting, or where there is considerable visual noise, daytime signal brightness is needed to maintain an acceptable contrast ratio. (*13*)

Glare

The Illumination Engineering Society Lighting Handbook defines glare as a sensation caused by a luminance source within the visual field of a greater luminance than that which the eyes are adapted to. (*10*) Consequences of glare can range from a loss of comfort (discomfort glare) to a decrease in visual performance (disability glare). Discomfort glare is a psychological phenomenon which does not interfere with visual performance, but does cause affected observers discomfort and alters their willingness to look at the source. Disability glare, on the other hand, adversely affects visual performance.

Studies have shown that lamp intensities required for traffic signal detection during daylight hours cause excessive glare at night. (*2, 14*) Schmidt and Connolly cite evidence that discomfort glare is caused when the difference between the luminance of the background and the glare source exceeds 2 log units. (*11*) There are few driving situations when this large a difference occurs because drivers' eyes are never fully dark adapted (scotopic). Even under low background luminance levels, drivers' eyes will be twilight adapted (mesopic)—an intermediate level between the lower limit of day vision (photopic) and the upper limit of night vision (scotopic). Glare that may occur due to steady signal intensity will be discomfort glare, rather than disability glare. (6) Discomfort glare is most likely to occur on low volume rural roads with few extraneous light sources. Glare will be more pronounced with flashing indications as well as with the higher intensity yellow—a potential problem recognized by the MUTCD in Section 2B-10 where dimming of 12-in (300 mm), 150-watt flashing yellow signals is suggested. (*4*)

Table 5 presents discomfort glare thresholds at 33 m as a function of various road environments. Since the discomfort glare threshold is a function of glare angle and the aiming pattern of the signal, data are needed to establish glare thresholds at distances other than 33 m. However, given the beam pattern of traffic signals, and the fact that the eyelid protects the eye when a glare source is overhead, drivers in the vicinity of a signal should experience less discomfort than they would approaching an intersection.

Table 5.—Discomfort glare threshold at 33 m (*13*)

Color	Glare threshold			
	Residential road	Shopping street	Central business district	High-speed rural road
	Candelas	*Candelas*	*Candelas*	*Candelas*
Red	350	700	715	460
Yellow	980	1,580	2,350	710
Green	450	760	940	450

Discussion

In assessing the effects of dimming traffic signals, it is necessary to consider the visibility of the signal at locations beyond the 100 m specified by the Fisher and Cole research. (6) Cole and Brown developed a nomogram (fig. 1) for optimum red signal intensity at or near the line of sight. (*15*) This nomogram enables an evaluation to be made of signal intensities over a range of background luminances. Since this article assesses dimming of signals at night, background luminances from 3.4 cd/m^2 to between 340 and 3,400 cd/m^2 are most applicable. Figure 1 also clearly shows that signals should not be dimmed during daytime background luminances.

The MUTCD 1/4-mile (400 m) visibility requirement and the ITE Minimum Luminus Intensity at Maximum Brightness Points have been plotted on the nomogram as shown in figure 1. The U.S. standard 8-in (200 mm) signal does not meet Fisher and Cole's 200 cd recommendations, nor does the 12-in (300 mm) standard wide angle lens meet their 800 cd criteria. (6) The 157 cd intensity of the 8-in (200 mm) signal, however, is not sufficiently below the 200 cd recommendation to alter night MUTCD conformance, although it might be significant during some daylight ambient conditions. The 399 cd, 12-in (300 mm), red lens brings high night background luminances more in line with MUTCD requirements than does the 12-in (300 mm), narrow beam, 808 cd alternative lens. Here again, the 399 cd intensity is greater than is needed at night in most rural locations where this lens might be used, but the intensity is less than needed during high day background luminance.

Thus, the low-speed urban road recommendation of 200 cd is essentially met by the 8-in (200 mm) signal under high night background luminances that would be found in urban CBD's, and optimum signal intensity requirements exceeded under lower background luminances that may occur in residential and rural areas. On the other hand, the 800 cd high-speed rural road recommendation is met by the 12-in (300 mm) narrow beam signal, although the intensity is higher than that needed for night background luminance that might be expected in most rural

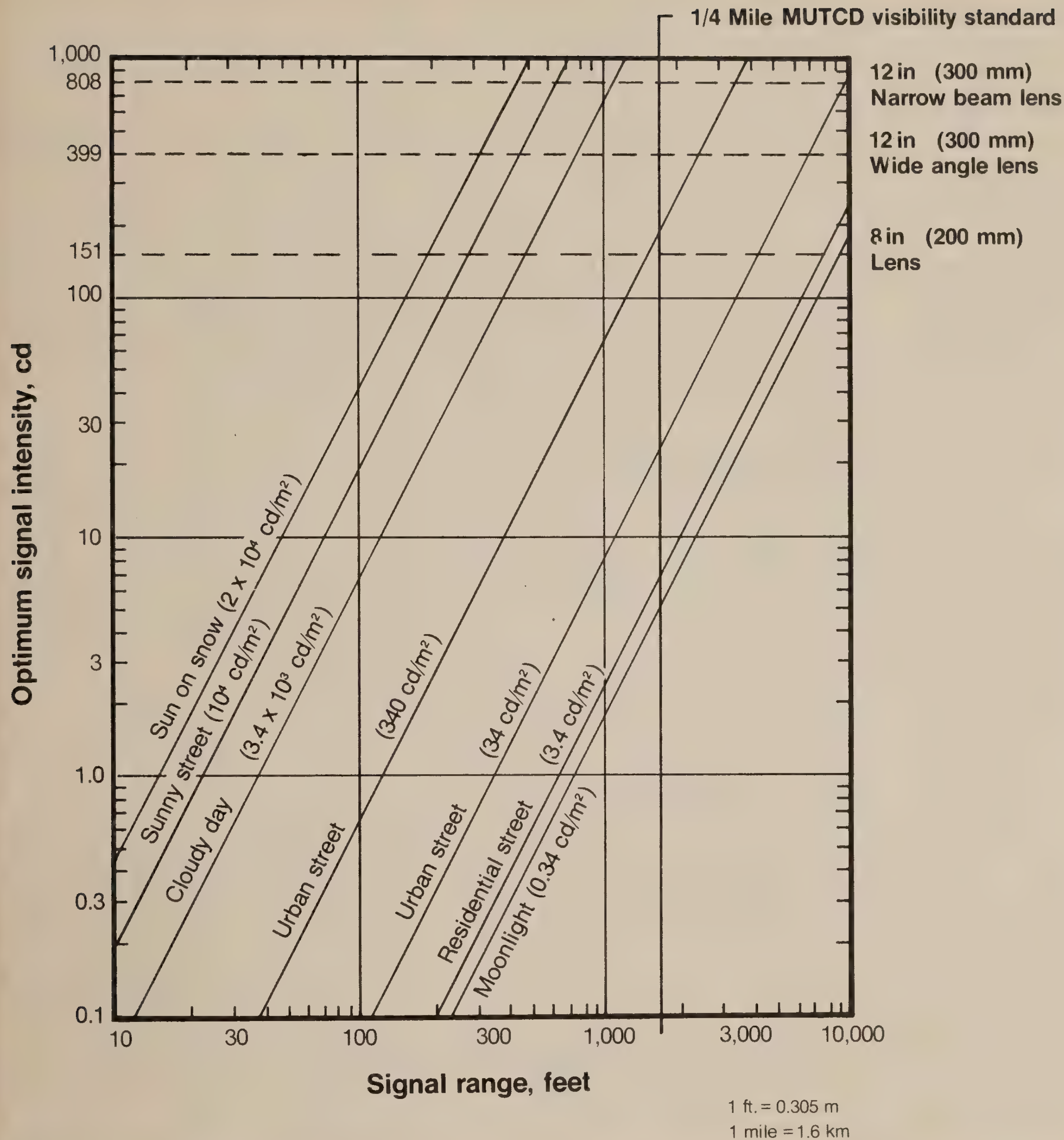

Figure 1.—Nomogram showing optimum luminance for a red traffic signal as a function of maximum signaling range for various background luminances. (15)

situations. However, there may be high background luminance or high visual noise situations in urban locations where the MUTCD criteria will require an 800 cd night intensity, as provided by the 12-in (300 mm) narrow beam signal.

Considering the MUTCD values for minimum signal installation visibility (table 1), it was found that all lamp intensities exceed the visibility distances specified at night. However, the 1/4-mile (400 m) MUTCD requirements are not met. Meeting the 1/4-mile (400 m) MUTCD criteria provides a greater safety margin and is in line with current decision sight distance recommendations, particularly at high-speed approaches. (*16*)

A consideration of the glare threshold values in table 5 shows that the 8-in (200 mm) lens is below threshold for all road environments and therefore does not represent a glare problem. The 12-in (300 mm) wide angle signal is below threshold for shopping streets and CBD's, but exceeds glare threshold on residential roads and high-speed rural approaches. The 12-in (300 mm) narrow beam lens exceeds glare thresholds for all roads.

It is difficult to assess the current population of signals in the United States. An early study cited by Box showed that about 7 percent of all red indications in 1965 were 12 in (300 mm). (*14*) He predicted that this would increase to 18 percent of all intersections by 1970.[3] The higher intensity 12-in (300 mm) signal has considerable application on high-speed rural and suburban approaches, and it is probable that many of the 12-in (300 mm) signals are found on these roads. However, rural intersections represent only 9 percent of all signalized intersections. Box found about 10 percent of rural signalized intersections used 12-in (300 mm) signals. (*14*) Even if this were increased to 30 to 50 percent, the majority of 12-in (300 mm) signals would still be in urban areas.

Data indicate that there are few instances where the 8-in (200 mm) signal should be dimmed. Since the intensity of the 8-in (200 mm) signal is less than the glare threshold at 33 m, there is no loss of safety in maintaining brightness levels. Although there are locations where the 8-in (200 mm) signal intensities exceed visibility requirements (such as rural roads and residential streets), higher brightness provides a margin of safety for aged drivers and red color-weak individuals.

There are cases where there is justification for dimming 12-in (300 mm) signals at night. At most rural and residential background luminances, signal intensity—particularly that for the yellow signal—is beyond the discomfort glare threshold. In addition, the day intensity is well beyond that needed for night perception, and lowering the intensity of a 12-in (300 mm) signal to the 8-in (200 mm) level would still provide a margin of safety. However, there are also locations where the day intensity may be needed. If the signal is well off the line of sight, or the background luminance is very high, then the day levels may be needed. Therefore, dimming should only be provided when conditions warrant. The need for dimming should be established on a location-by-location basis and predicated on the background luminance, geometric design, and visual noise of the situation.

[3]Currently, about 18 to 20 percent of all signals are 12 in (300 mm).

Conclusions and Recommendations

Successful driving performance is dependent on the reception and error-free handling of information. Drivers encounter situations which put heavy demands on their information handling capability when they approach locations where signals are warranted. It is critical to the safety and efficiency of these locations that drivers receive information and respond to traffic signals correctly and rapidly. This requires that the visibility of the signal meets or exceeds the needs and capabilities of drivers.

Accident statistics show that driving at night is inherently more hazardous than driving during daylight hours. (*17*) This partly results from impaired driving performance due to lack of visibility. Daylight cues are not present, and drivers are more dependent on traffic control devices to aid their driving performance.

Since signal perception is dependent on intensity and contrast ratio, night visibility of signals is often superior to that under high daytime brightness levels. This superior night visibility is desirable for a number of reasons: The possibility of extraneous glare sources (for example, misaimed headlights, outdoor advertising, and luminaires); the lack of other environmental cues; the potentially adverse effects of rain, snow, and fog; and the greater probability of fatigue and other factors impairing driver performance. From a safety standpoint, it is only when signal intensity exceeds glare thresholds that a trade off is necessary between visibility and intensity.

Data show that the 8-in (200 mm) signal intensities do not exceed the glare threshold and that the day intensity of the 8-in (200 mm) signal is required at night for many urban background luminances. Therefore, it is concluded that 8-in (200 mm) signals should not be dimmed.

There are situations where the 12-in (300 mm) signal, particularly the narrow beam configuration, exceeds both intensity needs and glare thresholds. In these instances, it is concluded that dimming is justified. However, these cases represent a small portion of the traffic signals used in the United States, and their impact on overall energy conservation will be minimal.

The following recommendations are made:

- Eight-inch (200 mm) signals should not be dimmed.
- Twelve-inch (300 mm) signals may be dimmed at night when conditions warrant. Dimming should be implemented on an intersection-by-intersection or signal face-by-signal face basis and based on empirically derived visibility criteria and engineering judgment.
- Empirical research should be undertaken to establish night and day visibility criteria for U.S. traffic signal indications, including arrows and pedestrian signals.

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Marking of roads – Traffic lanes
Marking of roads – Traffic lines

New Materials and Systems for Improved Traffic Lane Delineation

by Edward T. Harrigan

Introduction

A 1924 article in the *Engineering News-Record* described the delineation of a brick road in Ohio by the use of a centerline of white bricks. (*1*)[1] This may be the earliest recorded example of traffic lane delineation. The cost was $185 per mile ($296 per km).

The next year H. S. Mattimore formulated a scheme for the laboratory testing of highway marking paints to ascertain their expected serviceability. (*2*)

After 1925, the literature on traffic lane delineation research increased steadily by the year, reflecting the increasing importance of highway lane marking throughout the United States. A 1952 annotated bibliography of this literature from 1924 to 1951 required 29 pages in the Highway Research Board Bulletin. (*3*) In the next 25 years, the volume of this literature increased greatly, complementing the increase in new materials and techniques for traffic lane delineation developed in this period.

In 1952, the use of reflective glass beading for making highway lane markings visible at night and the use of hot-applied thermoplastics as more durable replacements for conventional traffic paints were still new and relatively untested. Considering their importance today, it is curious that both these materials received their impetus from World War II. Thermoplastic formulations were developed in the United Kingdom to replace materials diverted to higher priority war use. Glass beads were first widely used to compensate for the drastically reduced headlight illumination required at night under blackout conditions in the United States.

In 1977, glass beads and thermoplastic are no longer novel, but are essential materials in the mix of products used in current highway delineation practice. Although no longer spurred by wartime needs, research in this area is active and widespread. This article will review the Federal Highway Administration's (FHWA) program in highway delineation materials research and attempt to place it in the perspective of the important needs for future delineation systems.

Current FHWA Delineation Materials Research Programs

Since 1971, the research and development (R&D) function of the FHWA has been organized as the Federally Coordinated Program of Highway Research and Development (FCP). FCP Project 1I, "Traffic Lane Delineation Systems for Adequate Visibility and Durability," includes all FHWA R&D activity in the area of delineation materials and systems.

Specifically, FCP Project 1I involves three discrete tasks: (1) Systems for Improved Wet-Night Visibility With Adequate Snowplow Resistance; (2) Systems for Improving Durability, Rapid Drying, and Reduced Costs; and (3) Equipment and Methods for Installation and Maintenance. Each task is a major ongoing effort and involves FHWA-sponsored contract research, in-house staff research, research by State highway organizations through the Highway Planning and Research (HP&R) program, and research sponsored by the National Cooperative Highway Research Program (NCHRP). This article will survey the research underway or recently completed in each task to give an overview of the entire program.

[1]Italic numbers in parentheses identify references on page 131.

U.S. Federal Highway Administration

The Problem of Wet-Night Visibility

A critical problem in the use of traffic paint with reflective glass beads is that at night in the rain the beading becomes flooded and loses its reflectivity. The stripe then becomes practically invisible, and the motorist loses the stripe's guidance under circumstances when it is most necessary.

The Congress recognized the severity of this problem in the Highway Safety Act of 1973, Section 206, which authorized the Secretary of Transportation "to develop new traffic control materials, devices and related delineators to assist the traveling public during adverse weather and nighttime driving conditions."

Research in California in the 1950's indicated that in localities which receive little or no snow, raised markers bonded to the pavement provide excellent rainy weather delineation. (*4*, *5*) Since then, the use of these markers has become common in the South and Southwest as well as in California. Retroreflective markers are used to provide wet-night delineation while ceramic markers either replace or supplement painted lines under daylight conditions.

Unfortunately, these raised markers are incompatible with snowplow use; the plows easily scrape the markers from the pavement. Current research is aimed at providing a snowplow-compatible wet-night delineation system, but this problem has proven to be a very difficult one to solve. The research sponsored under Task 1 of FCP Project 1I is an effort to develop a practical solution to this problem.

One approach has been to retain the raised retroreflective marker, but to protect it from snowplows. The most widely tested marker of this type is the commercially developed Stimsonite 99 snowplowable raised reflective marker. This marker consists of a reflective unit housed in a hardened steel casting which guides plow blades up and over the plastic reflective unit without damage (fig. 1).

Figure 1.—Stimsonite 99 snowplowable raised retroreflective pavement marker.

An HP&R study evaluation of this marker was conducted by the New Jersey Department of Transportation (DOT). (*6*) In areas where snowfall ranges from 15 to 25 in (380 to 635 mm) per year, this marker provides excellent wet-night delineation and a good maintenance record when steel plow blades are used. Under severe conditions, particularly where tungsten carbide inserts are used on the plow blades, the results have been mixed. Under these conditions, both markers and plow inserts are damaged more often, though it may be questioned whether the plow insert damage actually affects plowing efficiency or is merely cosmetic. In any case, data are still being gathered from installations in high snowfall areas.

A promising alternative approach has developed from research conducted on pavement grooving to increase wet-night delineation. In the original studies conducted in California, Kentucky, and Utah, the pavement was grooved longitudinally and the grooves painted with traffic paint. (*7*, *8*)[2] Results as a whole were inconclusive, and the New York DOT undertook further research in this area under FHWA sponsorship. (*9*, *10*) New York DOT found that painted grooves under wet-night conditions provide only marginally better delineation than ordinary painted stripes. However, they found that the use of a recessed reflective marker combined with the grooved pavement (fig. 2) provided excellent wet-night delineation and sustained very little damage from traffic and snowplows after two winters at a test site in northern New York State. Further evaluation of this test site as well as evaluation of new sites in other States is planned, and development of mechanized equipment to install this type of marker quickly and economically is being pursued.

In the area of specialized delineation treatments, a delineation system for open-graded asphalt friction courses (OGAFC) has been developed. This system consists of prefabricated blocks composed of aggregate and pigment bonded with a high-strength adhesive. The blocks have the same thickness as the OGAFC pavement and have been both placed as lane markings simultaneously with the laying of the open-graded pavement and inlaid in slots cut into existing

[2] "Trial Installation of Traffic Markers in Snow Conditions," unpublished report on California study HPR-1(3), D-5-8, June 1966.

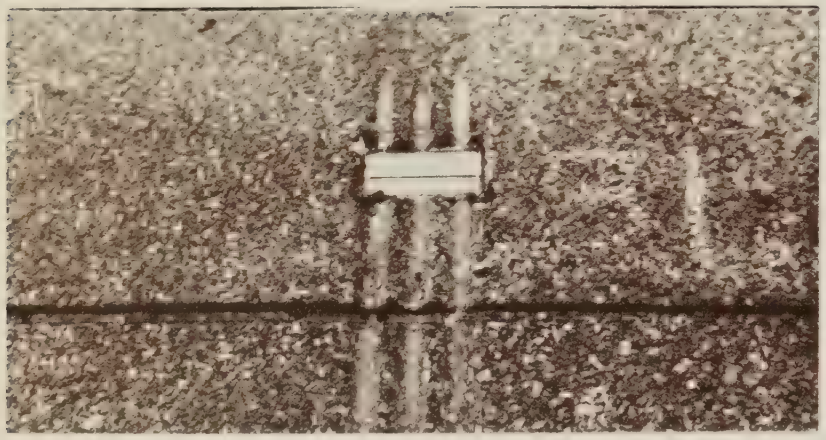

Figure 2.—New York Department of Transportation recessed reflective marker system.

pavements. The blocks are fabricated so that their porosity matches the porosity of the pavement. Thus, rainwater is drained from the surface rapidly, preventing flooding of the reflective beads and loss of marker reflectivity in rainy weather. This system is currently being field tested in northwestern Virginia, and after one severe winter shows excellent durability and good wet-night delineation.

While the approaches described above offer promise of yielding a workable wet-night delineation system, FHWA has also investigated several systems for which further development is not planned. Notably, extensive field testing was conducted on a low-profile marker. (*11, 12*) This marker consists of a basic plastic unit 7 mm thick, 100 mm wide, and 50 mm long (fig. 3). The units are transparent with a smooth top surface and a corner-cube pattern molded into the bottom of the top surface to provide retroreflection. Five-foot (1.5 m) lengths of these units are bonded with epoxy adhesive into slots cut in the pavements so that the top surface of the marker is flush with the pavement surface. The field tests demonstrated that at best the markers provided poor wet-night delineation, and that they are very susceptible to damage from traffic loads and snowplowing. Because the markers are poor delineators and costly to install, further development is not being considered.

A variety of self-luminous markers have been examined. Self-luminous markers offer the considerable benefit of not requiring headlight retroreflection to be visible. Thus, they could provide guidance even in situations where climatic conditions or roadway geometry limit the utility of headlight illumination. Electrically powered markers employing both incandescent and solid-state lamps have been tested by a number of agencies. These are extremely bright, and under favorable situations of roadway geometry can be visible for up to 2,000 ft (610 m) at night. However, because of their high installation and maintenance costs, use of electrically powered markers in the near future will probably be limited to special hazardous situations. Practically, the markers must be wired into a central power system since available types of batteries have too short a service life for practical use—even if current draw is minimized using solid-state electronic components and lamps. Commercial, battery-powered units are being test-marketed for use in construction zones where battery replacement is a manageable problem.

Radioluminescent and chemiluminescent markers have also been investigated. A radioluminescent marker consisting of a plastic Fresnel surface flush with the pavement surface is coated on the bottom with an inorganic phosphor material. Beneath the lens there is a series of tubes filled with radioactive tritium or krypton gas. Radioactive decay of the gases excites the phosphor material, causing it to luminesce. These markers demonstrated acceptable wet-night delineation capability; but no further work is planned due to the high cost of the gases, their probable limited supply in future years, and potential safety hazards associated with use of the markers in highway operations. Paints incorporating a chemiluminescent system which would luminesce when wet were studied. (*13*) The necessary chemical system was synthesized; but no further work is planned in this area due to high developmental costs, the short service life of the system, and the susceptibility of any potential paint system to traffic and snowplow wear.

Materials With Improved Durability and Lower Cost

Conventional traffic paints have provided the backbone of the Nation's delineation system since its inception. Continual improvements have been made in paint composition and application techniques to provide increased service life and rapid drying. While present day traffic paints can be placed economically and with minimal disruption of traffic, they still are limited by their susceptibility to wear from traffic, snowplows, and deicing salts and abrasives.

Thermoplastic materials have received increasing use as highway lane marking materials because of their long service life, compared to conventional paints. Increase in service life with these materials compensates for their substantially higher installation cost and the need for specialized application equipment and the use of primers, particularly on portland cement concrete pavements. However, the use of thermoplastic has not been widespread in areas with substantial snowfall and where problems of poor adhesion to portland cement concrete have been noted.

FHWA is sponsoring research in this area, as Task 2 of FCP Project 1I, in an effort to develop new delineation materials to solve some of the problems indicated above with paints and thermoplastics. The Texas Transportation Institute has explored the optimization of thermoplastic durability by changing the resin composition. In addition, laboratory studies have been conducted to characterize the failure mechanisms of thermoplastic on portland cement concrete surfaces and in high snowfall areas. Field testing of a number of new formulations has been conducted in New York, Colorado, Nebraska, Illinois, and Oklahoma. These trials also include the testing of new primers and application techniques which should provide better adhesion and durability under all conditions.

Another research contract, being conducted by Southwest Research Institute for FHWA, was initiated as an open study to develop a more durable, generic marking material. The study was not limited to specific types of material, but rather investigated many diverse materials. From this laboratory survey, a thermoplastic material composed of a mixture of epoxy resins was developed. Initial field tests of this thermoplastic demonstrated that it combines the good qualities of conventional thermoplastics with those of epoxy paints.

Specifically, when applied as a spray at 400° F (204° C), the material sets instantaneously and requires no coning, adheres equally well to bituminous and portland cement concrete without primer, and has survived three winters at a Colorado test site without significant damage from heavy traffic or snowplows and abrasives. In Texas the results have been equally good. Like epoxy paints, this material has high strength; but unlike the paints, it is a one-component material requiring neither curing periods nor complex application equipment. However, in Texas it has been found that the original material, like any thermoplastic, tends to pick up dirt during the dry summer months. Use of hardeners in place of a portion of the epoxy resins is being explored as a method of eliminating this problem. The initial field results of the hardened material are encouraging.

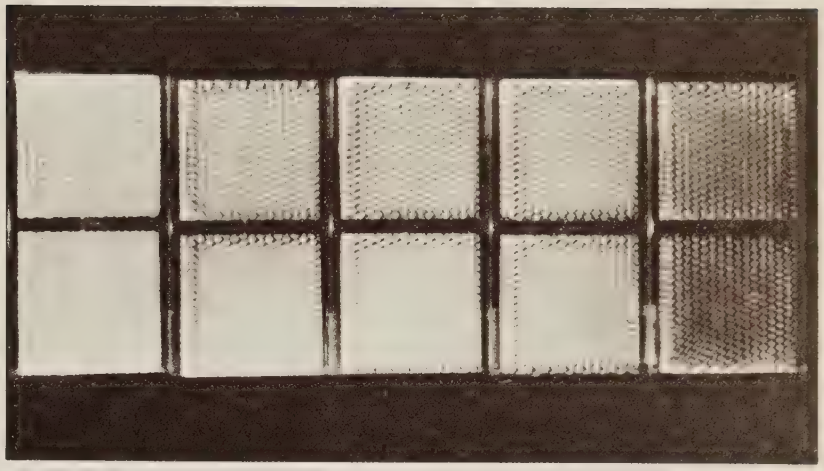

Figure 3.—Low-profile marker (top view).

This study will continue for the next 2 1/2 years. Large-scale field tests of an optimized material tailored to varying environmental conditions will be conducted in three States with diverse climatic conditions. These tests will also investigate the best type of reflective beads for durable marking materials. This adjunct study will assure that durable materials retain adequate night reflectivity throughout their service life.

Several commercially developed, proprietary durable materials are being tested under FHWA sponsorship. A two-component epoxy paint is under test on a number of high volume highways in Minnesota. In Ohio two different polyester paints are undergoing a large-scale, 3-year field test. Additional tests of these materials in other States are planned under the implementation program.

A number of collateral materials studies have also been conducted in this research area. A rapid setting and curing epoxy adhesive has been developed. This material is designed primarily for bonding raised traffic markers to pavement surfaces. Field testing was concluded in late spring of 1977, and the results indicate that this adhesive combines the strength and adhesion of currently accepted epoxy adhesives with greatly increased setting and curing rates. This combination would allow more rapid maintenance operations and less disruption of traffic during the operations.

Research has been successfully concluded on the formulation of yellow traffic paints using organic pigments in place of lead chromate. There has been much concern in recent years over the toxicological and environmental effects of lead and hexavalent chromium on the general population and, in particular, on those persons involved in the manufacture and use of products containing these elements. Indeed, there has been a gradual tightening of exposure limits, and this trend can be expected to continue. Due to the importance of lead chromate pigment in traffic paints, FHWA judged it advantageous to have definitive information available on acceptable alternative yellow pigments in the event that restrictions are placed on the manufacture or use of lead chromate in the future. Several such organic pigments which have the requisite properties have been identified through extensive laboratory, exposure, and field testing.

Finally, the problem of rapidly determining the acceptability of new or different paints for operational use has become acute in recent years. As the cost of traffic paint increases and as materials shortages force a greater reliance on rapid substitution of components in established formulations, the need for a reliable accelerated performance test for paints has increased. Actual field trials of various competing materials are, of course, the ideal method of determining relative performance or durability; but this method is time-consuming and expensive, even when accelerated transverse testing is used. Also, there is some question as to whether transverse testing provides a true test of a paint's performance since it stresses abrasive wear. This type of wear is not of prime importance in the failure of longitudinally placed pavement markings. FHWA-sponsored research is attempting to devise an accelerated laboratory durability test which will yield the identical relative rankings of paints as those obtained from field testing of longitudinal markings. This test (which will probably consist of a group of tests, rather than one) will allow for variations in climate, traffic volumes, pavement types, and other factors, and will be designed for practicality and economy of operation.

Development of Application Equipment and Delineation for Construction Zones

As originally constituted, Task 3 of Project 1I dealt with the development of practical and economical application equipment for the new delineation systems and materials developed in Tasks 1 and 2. Recently, however, the task has expanded to include applied research on problems arising from delineation in construction zones. The emphasis here has been on quick-reaction solutions for pressing safety problems connected with highway construction zones, and this effort is generally being handled as an implementation activity.

Currently, as mentioned above, development of installation equipment for the recessed reflective marker system developed by New York DOT is being initiated. An installation procedure for the open-graded asphalt friction course delineation system will be considered beginning in FY 1978, and equipment development for applying durable marking materials is planned for FY 1979.

A recently concluded study by the California Department of Transportation focused on the optimization of that State's delineation system to develop the most effective and most economical treatments. (*14*) Changes in traffic paint thickness, bead densities, reflective and ceramic marker spacing, stripe width, and skip length were recommended and implemented by the State. A considerable savings in materials and maintenance costs was achieved through this program. Although the study was conducted in California, the results have national applicability.

An effort to develop an effective pavement marking removal technique was recently concluded. The prime qualification of the technique was complete removal of the marking without scarring the underlying pavement or leaving traces of the marking. The aim was to eliminate as much as possible the unsafe conditions arising in construction zones due to scars or traces appearing as delineation, especially at night. This has been a common problem with accepted removal techniques, such as sandblasting and overpainting.

Three methods of pavement marking removal were developed as a result of this effort. The first method was an inexpensive hand-propelled burner assembly that rapidly burns off the stripe using a propane-oxygen flame supplied with excess oxygen to achieve a very high flame temperature. (*15*) The use of very high temperatures allows rapid combustion and minimizes pavement damage. In field tests this equipment gave good marking removal with minimal pavement damage on most types of surfaces. While some minor traces were visible to motorists during the day, at night no pavement defects were discernible under either wet or dry conditions. Several of these units are being built by individual States using FHWA-supplied plans.

The second method was a similar technique in which the stripe is subjected to an externally mixed propane-oxygen flame with the residue removed by a scarifier. (*15*) This unit produces a lower temperature flame than burning with excess oxygen.

The third method took an entirely different approach. (*16*) A mechanical removal unit was used in which the paint-pavement bond is weakened by the action of hardened steel cutter wheels and broken by the application of high-pressure water jets. This method has the advantage of being adaptable to higher speed, large-scale equipment for large removal jobs. Development of such equipment is planned.

A study sponsored by the National Cooperative Highway Research Program is aimed at the development of temporary traffic paints for use in marking construction areas and detours. In service, this paint would provide performance equal to conventional traffic paints, but it would be formulated for easy removal. At this time, additives for conventional paints that would greatly accelerate the destruction of the stripe by combustion, again without leaving pavement scars or traces, are being investigated.

Finally, a variety of implementation activities are underway to develop adequate delineation for safe traffic flow under all conditions in construction zones and other high hazard locations. Present emphasis is on the generous use of raised reflective and ceramic markers on both pavements and barricades. Results to date have been very encouraging.

Future Activities

No new research areas will be addressed within Project 1I in the future. The work ongoing in Tasks 1 and 2 will provide acceptable solutions to the problems addressed therein, at least within the boundaries of present technology. In Task 3 more work remains to be initiated, but this deals essentially with equipment development and implementation activities dealing with the output of the first two tasks as well as the specialized problem of construction and high-hazard zone delineation techniques.

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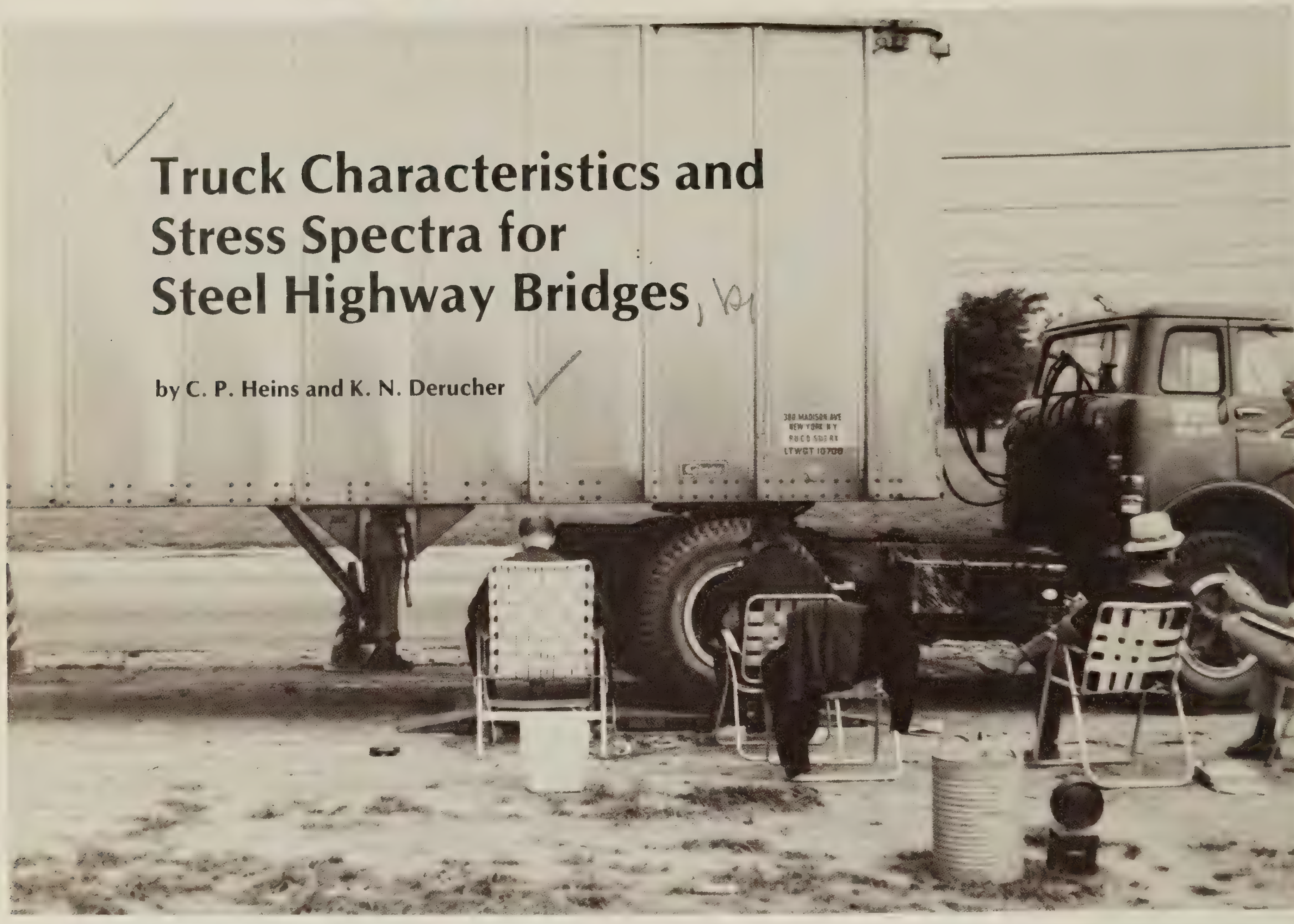
Bridge-Load Bridges- Strains and stresses
Motor trucks- Load
Steel bridges



Truck Characteristics and Stress Spectra for Steel Highway Bridges

by C. P. Heins and K. N. Derucher

Portable weighing station for determining vehicle loads.

Introduction

Highway bridges are designed using assumptions relative to the loadings and frequency of loadings expected during the life of the structure. Bridges are generally designed to carry a static load, caused by a design truck, augmented to simulate dynamic effects. These design methods are then employed to estimate the fatigue life of the structural elements in the bridge. Because bridges are subjected to a great variety of live loads, these design methods are approximations and provide only a crude evaluation as to the effect random traffic has on bridge elements. A more rational estimate of fatigue life of bridge elements can be made by determining the following:

- The loading history of highway bridges.
- The induced stresses-spectra.
- Loading frequency.
- Truck types.

Various types of steel highway bridges have been extensively tested throughout the world for the past 10 years to ascertain the induced stresses on various elements caused by random traffic. In addition to obtaining stress histograms, vehicle characteristics were recorded. These characteristics include truck type, dimensions, and weights. This article summarizes the results of these tests and clarifies the experimental and data reduction procedures that were used to collect this information.

The need for obtaining stress and load spectra information was first advanced by the Bureau of Public Roads (BPR) in 1965 and subsequently by the Highway Research Board in January 1967 in a discussion of highway research problems. (*1, 2*)[1] The loading history

[1] Italic numbers in parentheses identify the references on page 139.

Hake p 139 American Association of State Highway & Transportation Officials

problem was further amplified by the Federal Highway Administration (FHWA) in March 1968. (*3*) The problem proposals provided the necessary impetus for the initiation of bridge studies in Illinois, Ohio, Connecticut, Maryland, Michigan, Pennsylvania, Indiana, Alabama, Tennessee, Missouri, Virginia, and Canada. (*4–16*) A summary of the results of completed tests and the experimental procedures used for data collection is presented.

Survey of Past Bridge Tests

Bridge type and description

The bridge types examined during study of the loading history problem (*4–17*) included the following:

- Single-span bridges (18).[2]
- Three-span continuous (15).
- Four-span continuous (1).
- Five-span continuous (1).
- End-anchored bridges (2).
- Suspended-span bridges (2).
- Semisuspended-span bridge (1).

Thirty-five of these bridges were constructed of rolled steel shapes with coverplate or built-up steel sections and a concrete roadway, generally composite. Five bridges were composed of prestressed concrete girders or reinforced concrete T-beams.

A majority of these bridges had span lengths from 38 to 80 ft (12 to 25 m). Seven of the bridges did exceed 80 ft (25 m):

- Two in Michigan—95 ft and 129 ft (29 and 39.3 m). (*15*)
- Two in Connecticut—113.5 ft each (34.6 m). (*5*)
- Two in Ohio—114 ft each (34.7 m). (*14*)
- One in Pennsylvania—144 ft (43.9 m). (*16*)

The longitudinal girder spacing for the 40 bridges tested ranged from 5 to 10 ft (1.5 to 3.0 m). The average girder spacing for all bridges was 7 to 8 ft (2.1 to 2.4 m). The average thickness of the slab was 7 in (178 mm).

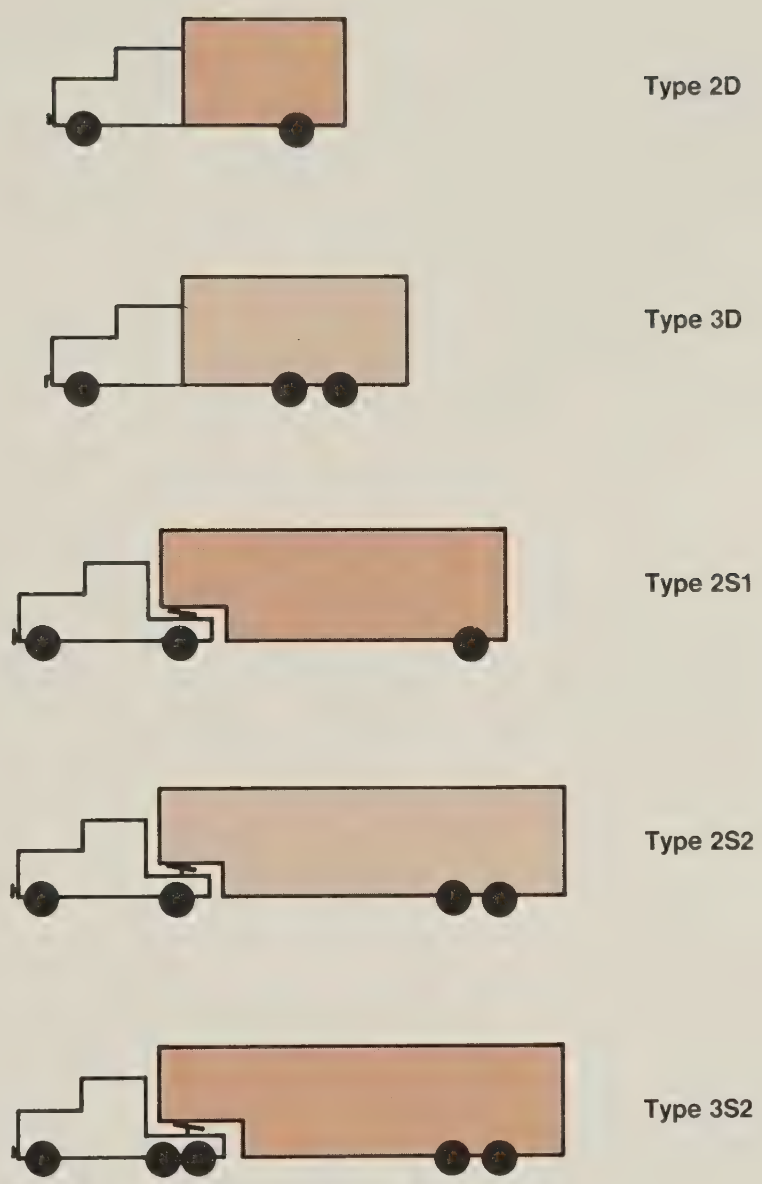


Figure 1.—Identification of truck types.

Vehicle characteristics

Stress response of a bridge is influenced by its characteristics—such as span length, girder geometry, and girder spacing—and the vehicles crossing the bridge. Observations of truck characteristics and their frequency of distribution during a given time interval were made during most of the load history tests reported to

[2]Numbers in parentheses identify the number of bridges studied.

Table 1.—Average distribution of trucks by type[1]

Truck type	Type of location		
	Metropolitan	Urban	Rural
	Percent	*Percent*	*Percent*
2D	35	13	21
3	23	3	6
2S-1	6	10	7
2S-2	11	30	25
3S-2	25	44	41

[1]Data from 16 locations were used.

date. (*4–16*) In general these results can be related to characteristics of five truck types; several of the studies (*14, 15*) included other truck configurations, but the most significant types are the five shown in figure 1. The average distribution of these five truck types, identified by type of area (metropolitan, urban, and rural), is shown in table 1.

In addition some studies collected axle weight distribution data (*5, 7–9*), vehicle gross weight data (*5–9, 11, 15*), and vehicle axle spacing data (*5–9, 11*) for the truck types studied (tables 2–4).

The data have been used to develop typical truck types[3] as shown in figure 2. Ranges and percent distribution in gross weights for each truck type are shown in figure 3, which can be used in the study of fatigue of bridges. (*18, 19*)

[3]"Bridge Loadings and Serviceability Requirements International State-of-the-Art," by C. P. Heins and F. Moses. Paper presented at Transportation Research Board Meeting, Washington, D.C., January 1976.

Table 2.—Percent vehicle axle weight distribution by type

Truck type	Axle weight distribution								
	A		B		C		Average		
	Maryland	Connecticut	Maryland	Connecticut	Maryland	Connecticut	A	B	C
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
2D	25	41	75	59	—	—	33	67	—
3	25	33	75	67	—	—	29	71	—
2S-1	20	27	40	40	40	33	24	40	36
2S-2	10	19	40	36	50	45	14	38	48
3S-2	20	18	40	42	40	40	19	41	40

Table 3.—Vehicle gross weights by truck type (in kips)

Truck type	Virginia		Michigan		Maryland (1)		Maryland (2)		Connecticut		Average		
	Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation	Range
2D	13.1	5.51	15.0	5.0	14.7	6.32	13.0	—	15.7	—	14.3	5.6	3≤GW≤25.5
3	22.4	9.91	—	—	32.4	12.89	48.0	—	38.4	—	35.3	11.4	12≤GW≤58.1
2S-1	29.7	15.93	36.6	9.7	31.4	9.67	29.8	—	54.7	—	36.1	11.8	12≤GW≤59.7
2S-2	38.5	9.86	37.0	13.0	43.2	15.19	38.0	—	45.8	—	40.5	12.7	15≤GW≤65.9
3S-2	54.9	13.96	48.7	13.7	56.6	19.79	53.0	—	48.2	—	52.3	15.8	21≤GW≤83.9

1 kip=4.4 kN

Table 4.—Vehicle axle spacing by type

Truck type	Axle spacing							
	A-B			B-C			Average	
	Virginia	Maryland	Connecticut	Virginia	Maryland	Connecticut	A-B	B-C
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
2D	14	16	15.7	—	—	—	15.2	—
3	14	18	19.1	—	—	—	17.0	—
2S-1	11	12	11.8	29	28	23.0	11.6	27.0
2S-2	11	12	12.1	27	28	28.7	11.7	27.9
3S-2	12	12	11.1	30	20	33.3	11.7	30.1

1 ft=0.305 m

Truck characteristic data from the United States, France, Belgium, India, Canada, and Sweden are also available (table 5).[4] A comparison of the data from each country shows reasonable correlation and might suggest the development of a standard design vehicle.

Stress measurement results

During the load history field tests, the induced strains on various elements of the bridges were monitored. Strain gages were attached to the bridges at the following locations:[5]

- Center span of girder on the bottom flange.
- Off and on the end of the coverplate.
- Diaphragms.
- Deck slab.
- Reinforcing bars in deck slab.

The resulting response of each gage was recorded, giving a strain (stress) versus time curve (fig. 4). The procedure for reducing this response curve is discussed in the next section. The important feature of the response curve relative to fatigue is the stress range (Sr) as shown in figure 4. During passage of various vehicles, these accumulated stress ranges can be recorded and tabulated resulting in a stress range history of the section under study. This information is conventionally represented in a frequency distribution of stress ranges as shown in a stress range histogram (fig. 5). Such diagrams have been recorded for truck traffic along the various test bridges and represent the response of particular bridge elements. (*4–16*)

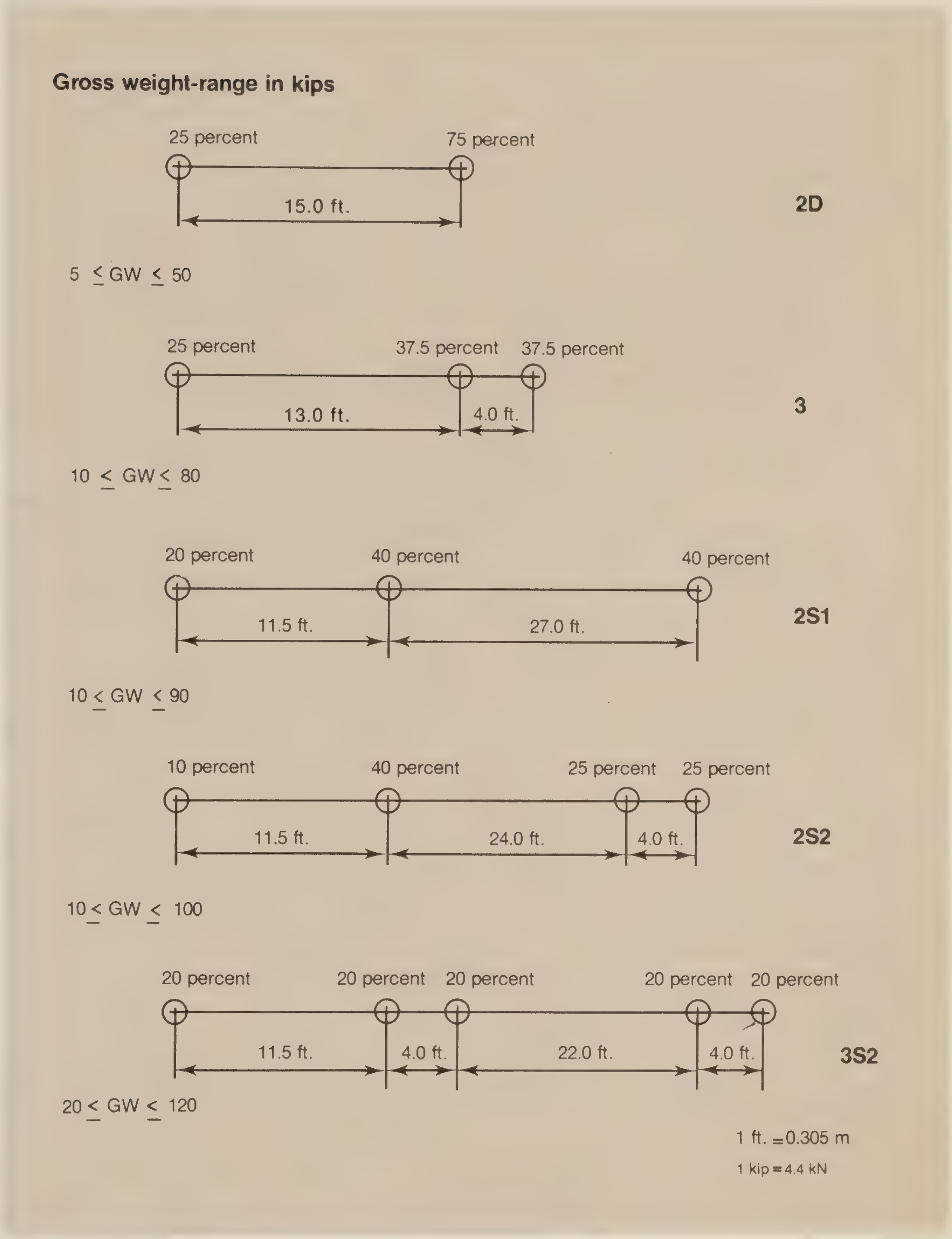


Figure 2.—Typical truck types.

Techniques in Measuring Stresses and Loads

Stresses

Stresses are usually monitored by strain gages; the gage readings are converted to stresses by use of Young's Modulus since strains are well within the elastic range. The recording and reduction of these strains (stresses) have been accomplished by various techniques and equipment.

In 1966, the BPR acquired a computer system which was programed to seek peaks and valleys of a strain trace of various gages. (*10*) The sampling periods ranged from 1/2 to 2 hours. The data were digitized and printed out at the end of each interval.

[4] "Bridge Loadings and Serviceability Requirements International State-of-the-Art," by C. P. Heins and F. Moses. Paper presented at Transportation Research Board meeting, Washington, D.C., January 1976.

[5] These measurement locations were not necessarily at points of maximum strain.

The data acquisition system consisted of signal conditioning modules, amplifiers, an analog-to-digital converter, a digital processing unit (computer), and an input-output device (teletype with paper punch). This system was used in many of the load history studies reported. (*5, 6, 11, 16*)

The studies conducted in Alabama used a 14-channel oscillograph and a 7-channel FM tape recorder in conjunction with conditioning modules. (*4*) The data were then reduced to the form of histograms.

The bridge studies conducted in Maryland used a light beam oscillograph and two 4-channel carrier amplifiers to record induced live-load strains. (*7-9*) The resulting oscillograph records were then reduced using a digital data reduction system.

The studies conducted in Michigan used a 12-channel system with a direct writing light beam oscillograph visicorder, similar to the Maryland tests. (*15*) The resulting oscillograph traces were processed through a strip chart reader.

The Canadian tests used a system which consisted of signal conditioning and strain amplification units. (*12*) The strains were recorded on a 7-channel analog 7-tape recorder. The data were then processed through a computer system.

Table 5.—Truck characteristics from various countries

Truck characteristics by type	Country					
	U.S.	India	Belgium	France	Canada	Sweden
2D:						
Gross weight—maximum	50 (14)[1]	24, 32	32	31, 38	40*[2]	—
(in kips)			38	32*		
Axle load distribution	25–75	30–70	36–64	—	50–50*	—
(percent of gross weight)			32–68	50–50*		
Percent truck type distribution	35	—	56	80	—	—
(population)			59	—		
3:						
Gross weight—maximum	80 (35)	50	36	52	60*	—
(in kips)			52	60*		
Axle load distribution	25–37.5–37.5	20–40–20	25–38–37	—	33–33–33*	—
(percent of gross weight)			24–38–38	20–40–40*		
Percent truck type distribution	23	—	7	4.4	—	—
(population)			6	—		
2S–1:						
Gross weight—maximum	90 (36)	—	36	52	80*	—
(in kips)			120*			
Axle load distribution	20–40–40	—	19–35–46	—	25–25–50*	—
(percent of gross weight)			33–33–33*			
Percent truck type distribution	6	—	7	4.5	—	—
(population)			—			
2S–2:						
Gross weight—maximum	100 (41)	60	56	70	100*	146*
(in kips)			76			
Axle load distribution	10–40–25–25	16–30–27–27	14–29–29–28	—	20–40–20–20*	12–34–20–34*
(percent of gross weight)			13–35–26–26			
Percent truck type distribution	11	—	30	10.6	—	—
(population)			27			
3S–2:						
Gross weight—maximum	120 (52)	90	80	70	92*	—
(in kips)				140*		
Axle load distribution	20–20–20–20–20	12–22–22–22–22	12–20–20–18–30	—	12–22–22–22–22*	—
(percent of gross weight)				9–18.5–18.5–27–27		
Percent truck type distribution	25	—	8	6	—	—
(population)				—		

1 kip=4.4 kN

[1]Numbers in parentheses are mean values.

[2](*) identifies data which are suggested design loading; all other data observed from typical traffic.

The Ohio tests used an in-house designed system, which made the recording of vehicle types and locations easier and induced strains. (*14*) Commercial signal conditioners and amplifiers were used. Button boxes and photoelectric cells were constructed to record traffic type and lane location. The input signals were monitored on tape recorders. The data were then processed through a computer system.

Loads

The determination of vehicle loads was generally performed by portable weighing scales or at a permanent weighing station. In some instances, if the bridge were calibrated, this provided a means for obtaining the vehicle weight. (*14*)

Vehicle distribution

The vehicle distribution by classification was generally noted visually and recorded. The strain recording for a particular vehicle is attached or input into the recording by means of a signal. Telephones (*7–9*) or button boxes (*14*) have been used for inserting these data.

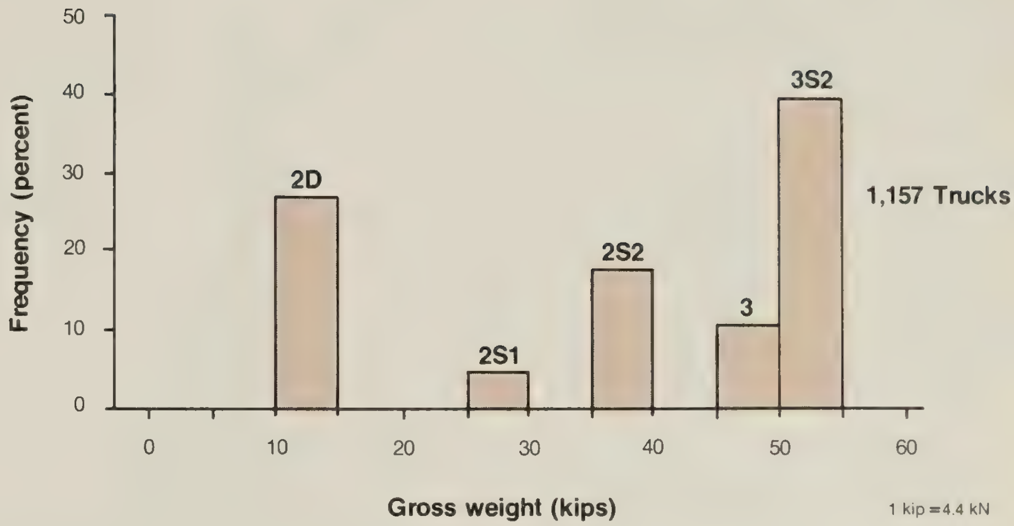


Figure 3.—Frequency of mean gross weight. (18)

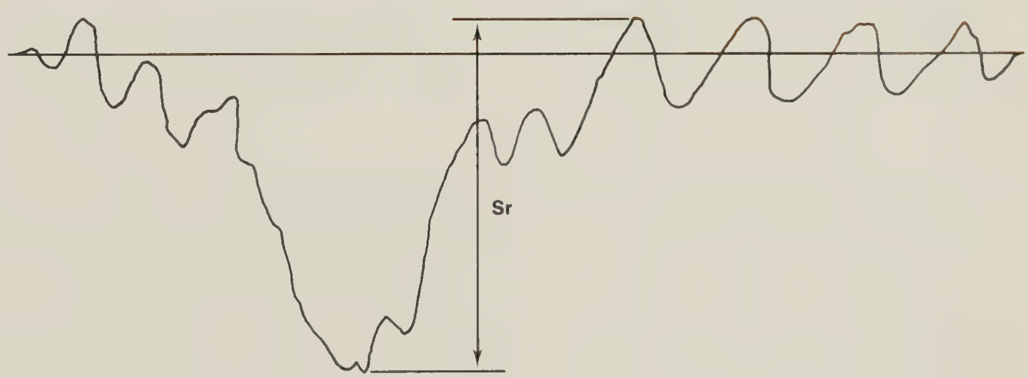


Figure 4.—Typical strain trace caused by a 3-axle dump truck.

General Trends

Stresses

Induced stress data, as mentioned previously, are generally presented by stress histograms. Examination of these stress range histograms indicates that the stress range intervals selected by the researchers in presenting their data varied from 0.2 to 1.2 ksi (1.4 to 8.3 MPa). The maximum stress range values recorded varied from 0.2 to 10.5 ksi (1.4 to 72 MPa). (*11, 16*) In some instances the data were presented relative to a zero stress range (fig. 5). In other instances stress range levels below 0.45 to 1.0 ksi (3.1 to 6.9 MPa) were neglected. A study of all stress range histograms has indicated that most of the data presented had not neglected data exceeding 25 percent of the highest stress range. (*17*)

A plot of this data neglecting all data below 0.25 Sr and normalizing Sr by the ratio Sr/Sr(max) is shown in figure 6. (*17*) The average of these data can be represented by the following probability density function:

$$f(s) = -12(X-1.0)^3 + 0.07$$

for the interval $0.25 \leq X \leq 1.0$ where $X = (Sr)/(Sr)max$.

The evaluation of the frequency during certain intervals of X can then be determined by integrating $f(s)$ or

$$\text{frequency (percent)} = \left\{-12 \int_{X_1}^{X_2} [(X-1.0)^3 + 0.071 dx\right\} 100,$$

or

$$\text{frequency (percent)} = \left\{-3.0[(X-1.0)^4]_{X_1}^{X_2} + 0.071(X)]_{X_1}^{X_2}\right\} 100,$$

where the frequency (percent) is to be determined over the interval of X_1 to X_2.

In addition to the prediction of this induced frequency, a study was conducted on the relationship between induced stresses on the bridge girder and the gross weight of the various types of vehicles (fig. 1). (*20*) This relationship was made possible by recording the induced stresses and the corresponding vehicle gross weight at the various bridge sites. (*7–9*) These relationships are only valid for single-span composite structures, and are as follows:

Centerline of girder

$$S_r = f_{Test} = [A + B(GW)]\frac{S}{12L}$$

Where,

$A = 0.1835$

$B = 0.0328$ for 2D and 3 type trucks.

$$S_r = f_{Test} = [C + D(GW)]\frac{S}{12L}$$

Where,

$C = 0.3338$

$D = 0.01824$ for 2S–1, 2S–2, and 3S–2 type trucks.

Off end of coverplate

$$S_r = f_{Test} = [E + F(GW)]\frac{S}{12L}$$

Where,

$E = 0.0720$

$F = 0.0250$ for 2D and 3 type trucks.

$$S_r = f_{Test} = [G + H(GW)]\frac{S}{12L}$$

Where,

$G = 0.1211$

$H = 0.0153$ for 2S–1, 2S–2, and 3S–2 type trucks.

In all of the above,

GW = Gross weight of vehicle.

S = Composite girder section modulus at referenced location, bottom of steel flange.

L = Girder span length in feet.

$S_r = f_{Test}$ = Induced girder stress on bottom flange.

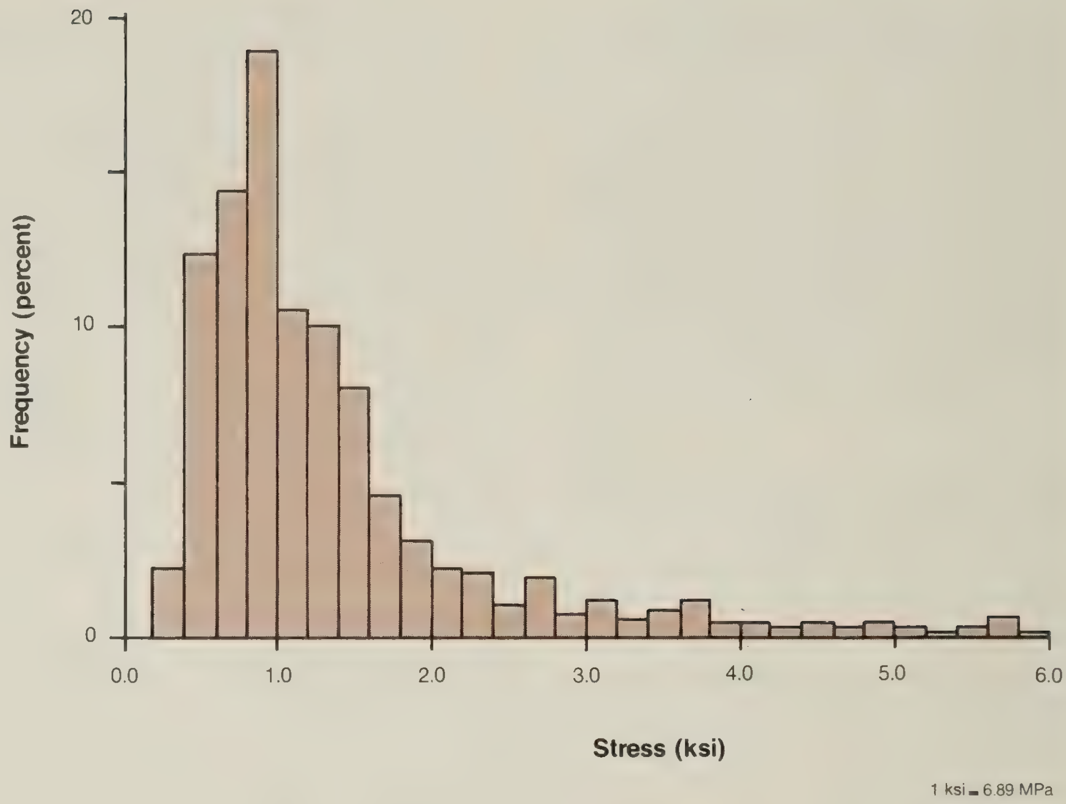


Figure 5.—Frequency of stress range (673 trucks). (7)

These equations permit evaluation of the induced field stress ranges due to the five truck types as a function of *GW, S,* and *L*. Thus, by knowing the vehicle distribution, the frequency and magnitude of stresses can be evaluated for a given bridge under design.

During the collection of these data, the following observations were made:

- The frequency of more than one truck at a time on a bridge (simple or continuous) is 4 percent or less.
- When there is more than one truck on a continuous bridge, the induced stress ranges are less than those caused by a single truck. (*19*)
- The distribution factor for typical trucks is approximately $\bar{S}/12$ (where $\bar{S}$ is the girder spacing), as noted from the test data and from previous studies. (*14, 21*)

Vehicles

As described previously, a survey of the truck data collected during the various tests has resulted in a series of typical trucks as shown in figure 2. The average values of the gross weights for the various truck types have been computed and are as follows:

Truck type	Average gross weight (kips)[6]
2D	14.0
3	35.0
2S–1	36.0
2S–2	41.0
3S–2	52.0

[6] 1 kip = 4.4 kN.

Results and Conclusions

The results of numerous field load history tests have been discussed and reported. These data give the induced live load stresses on various members of highway bridges resulting from passage of random traffic and the characteristics of the traffic.

The results show that the recorded stresses are much lower than the design values, and the characteristics of the vehicles which induce such stress also differ from the American Association of State Highway and Transportation Officials' truck loadings.

These differences suggest a possible revision in the fatigue design codes. Alternate procedures, which include the information collected from field tests, have been suggested. (*17, 22, 23*)[7]

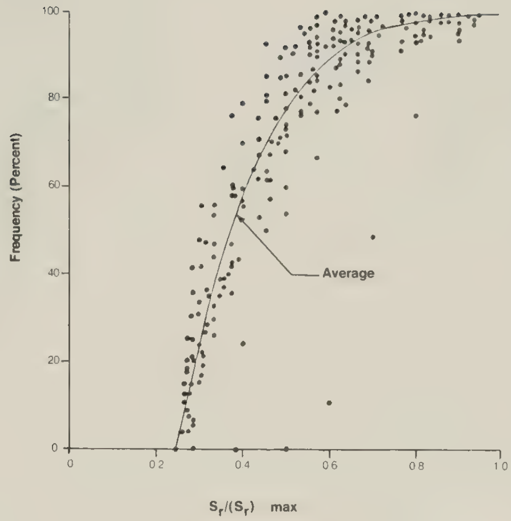


Figure 6.—Nondimensional cumulative frequency plot of available data with cutoff point at 0.25 (S_r)max.

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(*19*) C. F. Galambos and C. P. Heins, "Loading History of a Highway Bridge—Comparison of Stress Range Histograms," *Public Roads*, vol. 36, No. 9, August 1971.

(*20*) R. L. Khosa and C. P. Heins, "Study of Truck Weights and the Corresponding Induced Bridge Girder Stresses," Report No. 40, Civil Engineering Department, *University of Maryland*, February 1971.

(*21*) R. C. Forbes and C. P. Heins, "Analysis Charts for Use in Issuing Vehicle Permits," Report No. 49, Civil Engineering Department, *University of Maryland*, June 1973.

(*22*) F. Moses and R. Garson, "New Procedure for Fatigue Design of Highway Bridge Girders," Transportation Research Record 507, *Transportation Research Board*, Washington, D.C., January 1973.

(*23*) "Guide to 1974 AASHTO Fatigue Specifications," *AISC*, New York, N.Y., May 1974.

[7] "Bridge Loadings and Serviceability Requirements International State-of-the-Art," by C. P. Heins and F. Moses. Paper presented at Transportation Research Board meeting, Washington, D.C., January 1976.

Our Authors

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Harold Lunenfeld has been an engineering psychologist with the Human Factors Branch, Office of Traffic Operations, Federal Highway Administration, since 1972. Prior to joining FHWA, he served as a human factors engineer at General Instruments and AIL. His current responsibilities include analyzing highway information systems, defining driver information needs, and developing procedures and standards.

Edward T. Harrigan is a research chemist in the Materials Division, Office of Research, Federal Highway Administration. He is the project manager for the FHWA's Federally Coordinated Program of Research and Development Project 1I, "Traffic Lane Delineation Systems for Adequate Visibility and Durability."

Conrad P. Heins is a professor of civil engineering at the University of Maryland. Dr. Heins has been engaged in field testing of bridges during the past 10 years. He is chairman of the Transportation Research Board Committee on Bridge Dynamics and Field Testing. He has written several articles on bridge analysis, design, and testing.

K. N. Derucher is an assistant professor of civil engineering at the University of Maryland. Dr. Derucher has a strong background in the theory and application of concrete and concrete materials as it relates to bridge deck failure and load response.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Office of Research, Federal Highway Administration, which includes the Structures and Applied Mechanics Division, Materials Division, Traffic Systems Division, and Environmental Design and Control Division. The reports are available from the address noted at the end of each description.

Survey of Alternatives to the Use of Chlorides for Highway Deicing, Report No. FHWA-RD-77-52

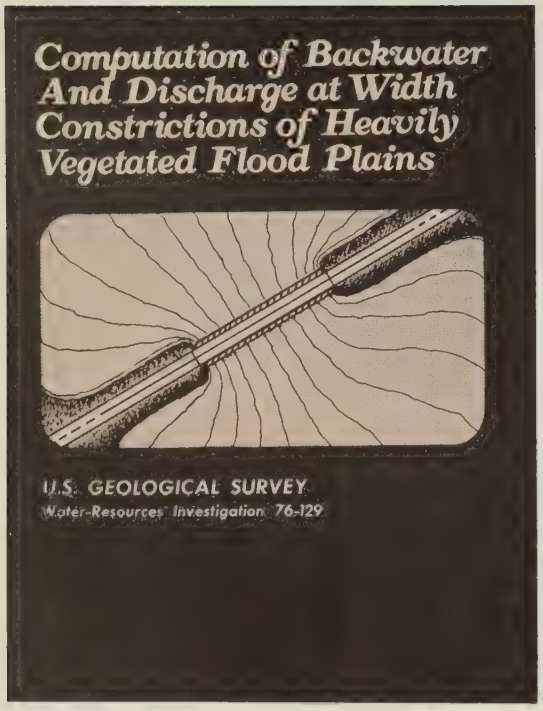
by FHWA Materials Division

The increased reliance on the motor vehicle has caused motorists to insist that streets, roads, and major highways remain open for safe travel during adverse weather—under conditions of snow, ice, and freezing temperatures. Compliance with these needs has resulted in a large increase in the use of sodium and calcium chlorides to melt ice or prevent its formation on pavements.

Extensive applications of chloride deicing salts have made winter driving safer but have caused various problems. Many State highway departments avoid excessive use of salt because of the destructive effects. Problems persist, however, even with judicious salt application. Some agencies have investigated the use of alternative deicing methods.

This report provides a state-of-the-art survey of various chemical and physical alternatives which have been or are being used by State highway agencies, Canadian provinces, European countries, Federal agencies, and private organizations. In addition, pertinent information from cited references as well as evaluations, suggestions, and recommendations are given.

The report is available from the Materials Division, HRS-20, Federal Highway Administration, Washington, D.C. 20590.



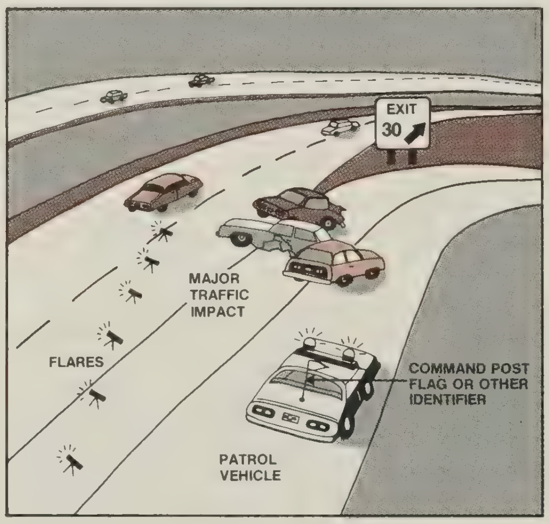
Computation of Backwater and Discharge at Width Constrictions of Heavily Vegetated Flood Plains

by the Mississippi State Highway Department, State of Alabama Highway Department, Louisiana Department of Transportation and Development, and FHWA

The Flood Insurance Program has made bridge backwater—the difference between the expected water surface profile after a bridge is installed and the natural water surface profile before a bridge is installed—a critical aspect of the analysis of new and reconstructed bridges.

A method for computing backwater and discharge at width constrictions of heavily vegetated flood plains has been developed from data collected at 20 single opening bridges for 31 floods. This report describes the improved method which has an average error of only 1 percent. The described method includes a rational adjustment of the average flow-path length, which is a significant parameter for highly constricted flood plains. The report will be useful to hydraulic and bridge engineers who are interested in bridge backwater computations.

The report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 266418).



Alternative Surveillance Concepts and Methods for Freeway Incident Management: Volume 5—Training Guide for Onsite Incident Management (Report No. FHWA-RD-77-62)

by FHWA Traffic Systems Division

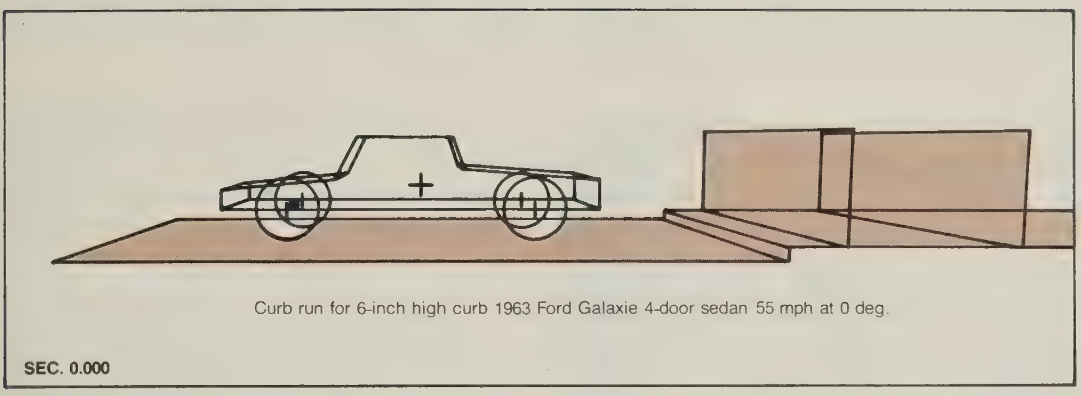
Safety and operations can be improved at traffic incident sites if good onsite traffic management is used. This training guide has been prepared to assist public agency personnel with traffic control and incident cleanup at the site of a freeway accident, load, spill, or vehicular breakdown. This guide may be used as a text for training new personnel and for review courses to improve existing operating procedures.

Critical issues for both minor and major traffic impacts are identified, and recommended traffic control procedures are shown. The recommended procedures have been developed from interviews and information given by traffic engineers, police, and operating organizations, and from good traffic management procedures observed during onsite incidents.

The report provides guidelines to improve existing onsite traffic operations procedures and performance, suggests new techniques or procedures that may be applicable under various conditions, and identifies the critical issues that surround catastrophic incidents and the best action to take.

The other five volumes of the six-volume final report are expected to be available in spring of 1978. These reports will identify low cost incident management systems and approaches for responding to freeway disturbances.

The report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 259007).



Curb run for 6-inch high curb 1963 Ford Galaxie 4-door sedan 55 mph at 0 deg.

Highway-Vehicle-Object Simulation Model—1976: Volume 1 (Report No. FHWA-RD-76-162), Volume 2 (Report No. FHWA-RD-76-163), Volume 3 (Report No. FHWA-RD-76-164), and Volume 4 (Report No. FHWA-RD-76-165)

by FHWA Structures and Applied Mechanics Division

Two upgraded versions of the Highway-Vehicle-Object Simulation Model (HVOSM) computer program for a single-vehicle situation, the HVOSM-RD2 Version (Roadside Design) and the HVOSM-VD2 Version (Vehicle Dynamics), are documented in these reports.

These versions were developed to provide an improved analytical means of evaluating the effects of the highway/roadside environment on vehicle safety. Typical modeling capability includes the general three dimensional motion resulting from vehicle control inputs, traversal of irregular terrain, or collisions with simple roadside barriers.

The HVOSM program has been successfully applied to the analysis of collisions with concrete safety shapes, ditch traversals, curb jump investigations, and general vehicle handling analyses. A graphical output is available to provide single-frame perspectives of the interaction of the vehicle and the object being investigated or to provide animated movies with appropriate hardware.

Volume 1, **User's Manual**, is a guide for HVOSM users. It contains a cross-referenced listing of both analytical and program symbols, a general discussion of program capabilities and limitations, a description of the mathematical model, and a discussion of general program solution procedures. Program input and output are described and sample applications are presented.

Volume 2, **Programer's Manual**, is written for those interested in detailed computer programs. It contains descriptions of the computer code (including a discussion of subroutine functions), annotated flow charts, and program listings. Also included are a list of program changes, a description of program stops and messages, and the computer system requirements that must be met in order to run the programs.

Volume 3, **Engineering Manual—Analysis**, is for the engineer who wishes to become familiar with the mathematical model that is the basis for the HVOSM computer simulation. This volume contains derivations of governing equations, assumptions, and the development of controlling logic.

Volume 4, **Engineering Manual—Validation**, also is addressed to the engineer and contains validation of the HVOSM mathematical model and computer simulation. Rigorous validation tests included both comparisons with extensively documented full scale experiments and general comparisons.

These reports are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock Nos. PB 267401–PB 267404).

Development of a Collapsing Ring Bridge Railing System, Report No. FHWA-RD-76-39

by FHWA Structures and Applied Mechanics Division

The development of bridge railings which deflect under impact to reduce collision severity is a relatively new concept. The bridge railings must be capable of retaining large impacting vehicles such as buses and tractor-trailers without increasing the severity of collisions involving passenger vehicles.

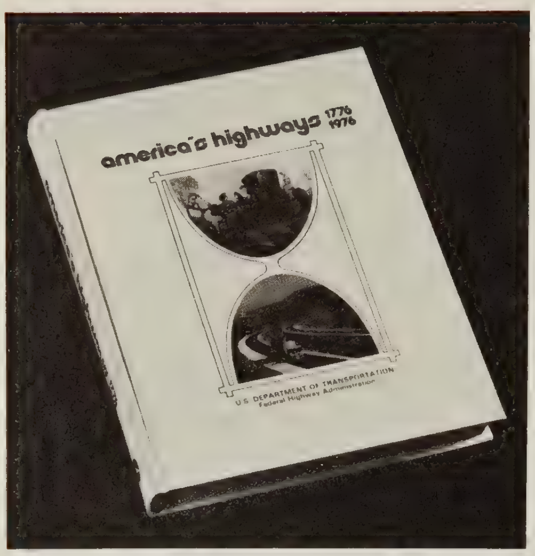
The collapsing ring bridge railing system described in this report is capable of reaching these goals. Steel rings serve as primary energy absorbing devices in the system. Vehicle impact energy is dissipated by the partial or total collapse of the rings.

This energy absorbing bridge railing system was found to be effective for restraining large impacting vehicles such as scenicruiser buses and tractor-trailers as well as for limiting accelerations to acceptable levels in small impacting passenger vehicles. The results of 14 full-scale crash tests with vehicles ranging in weight from 2,090 lbs (950 kg) to 70,000 lbs (32,000 kg) are described.

The collapsing ring bridge railing system represents an advanced state-of-the-art design which can be constructed with conventional materials and barrier elements currently used in highway construction. The system is designed for quick repair, using readily available handtools.

The report is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161 (Stock No. PB 265577).

New Publication



America's Highways 1776–1976, a new hard cover history and reference book, is the story of highway development in the United States from colonial days, through the expansion westward, and ultimately to today's Federal-aid highway program. It documents early decisions and experiences that were previously available only in scattered writings or were known only to those officials directly involved during the development of the program.

Part I of the book covers the economic and social needs that led to enactment of the various pieces of legislation up to the time of the landmark Federal-Aid Highway Act of 1956, and its companion, the Highway Revenue Act of 1956. Part II deals with program administration, planning and research, design, construction and maintenance of highways and bridges, and with the Federal Highway Administration's overlapping responsibilities with other Federal, State, and local governments and agencies. Biographical information on a few key individuals has been included at the end of Part I and in selected chapters of Part II where each man's contribution is directly related.

This book has been written for a widely diversified audience—for those interested in the general history of our Nation's highways and for those whose interest is more narrowly confined to matters relating to the technical aspects of highway transportation.

This publication may be purchased for $17 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–001–00123–3).

Instructions to Authors

All articles proposed for publication in *Public Roads* magazine are reviewed for suitability by the technical editors. Authors will be notified of acceptance or rejection as soon as possible.

Recent issues of the magazine should be reviewed for type of articles, style, illustrations, tables, references, and footnotes. *Public Roads* follows the U.S. Government Printing Office Style Manual.

Submission of Manuscripts

Authors in the Washington, D.C., area should submit three copies of the manuscript to the Managing Editor.

Managing Editor (HDV–10)
Public Roads Magazine
U.S. Department of Transportation
Federal Highway Administration
Washington, D.C. 20590

Authors outside the Federal Government, or in State, city, or local government agencies, should submit manuscript copies through appropriate Federal Highway Administration regional offices (see page 125).

Manuscript Treatment

Manuscripts should be typewritten, double spaced, with at least 1-inch margins on 8½ by 11-inch paper. Excluding art, 1 magazine page requires about 3 pages of manuscript. Indent paragraphs and end each page with a completed paragraph. Center main headings and type in initial caps. Capitalize the first letter of the first word in a sentence of heading, and the first letter in each other word except for articles and prepositions of less than 4 letters. Subheadings should be flush left and the first letter only capitalized. The article title and the name of each author should be typed on a separate page. If the article has been presented at a meeting, that should be indicated in a footnote at bottom of title page. Each page of the text should be numbered in the upper right margin. Since the Federal Government does not endorse products of manufacturers, avoid trademarks and brand names in articles unless their use is directly required by the objectives of the article.

Biography

A brief biographical sketch should be supplied. This should include the author's present position and responsibilities and previous positions relevant to the subject matter of the article. Biographies are limited to approximately 100 words.

Abstract

An abstract should be supplied with technical articles. For a *development report* the abstract should tell: (1) What has been accomplished, (2) its outstanding features, and (3) its applications, if known. The abstract of a *research paper* should focus on: (1) What has been accomplished, (2) its most important facts and implications, and (3) logical steps open to study.

Tables

Nonessential technical tables should not be included in the article. Each table should be typed on a separate page. It should be identified by an Arabic number and a caption. Note the position of the table in the text in the margin, but avoid putting it on the first (title) page. Details of data already presented in tables or charts should not be repeated in the text.

Illustrations

Illustrations *referenced in the text* are called figures, and numbers and captions should be assigned to each. Organize the text so that illustrations can be scattered throughout the article. Avoid referencing several illustrations in one page, to prevent problems in the layout. Black and white, glossy photographs of good quality are preferred; however, color photographs and art are acceptable. Send original artwork to the editor when the author is notified that his or her article has been accepted for publication. Send legible copies with the manuscript. All captions (numbered and unnumbered) should be typed underlined on a separate page.

Metrication

Under present law and FHWA regulation, *Public Roads* is required to show the metric equivalent of every quantity expressed in customary (English) units in the text. In tables and illustrations, indicate the equivalent units in a legend (for example, 1 in = 25.4 mm); it is not necessary to show metric equivalent for each point plotted or quantity tabulated.

References

Number references consecutively in the body of the text, enclosed in parentheses and underlined. Copyrighted material referenced and quoted will require copyright releases. Unpublished material referenced in the text will be described in a footnote. Type citations on a separate page under the heading REFERENCES. Number citations in the same manner as in the text and list in the same sequence.

Galley Proofs

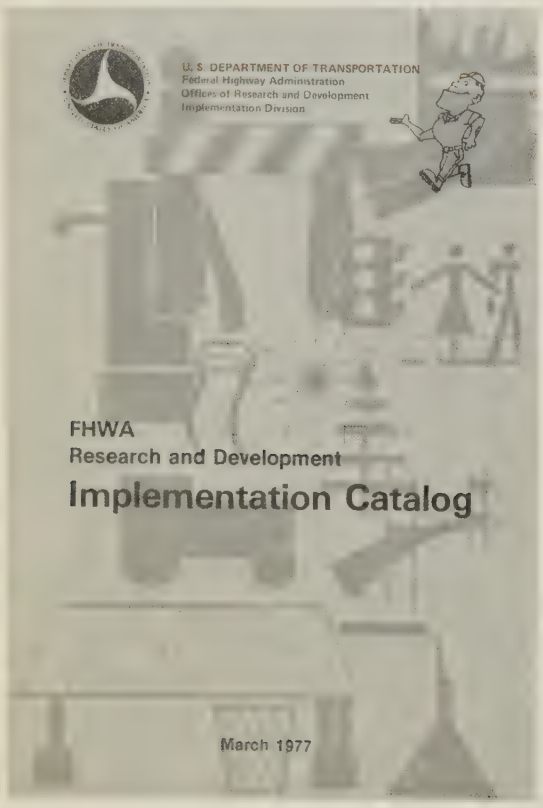
Galley proofs will be sent to authors for their inspection.

For further information, contact the editor: (703) 557-4301.

Implementation/User Items
"how-to-do-it"

The following are brief descriptions of selected items which have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Offices of Research and Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies. These items will be available from the Implementation Division unless otherwise indicated.

U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division, HDV-20
Washington, D.C. 20590



1977 Implementation Catalog

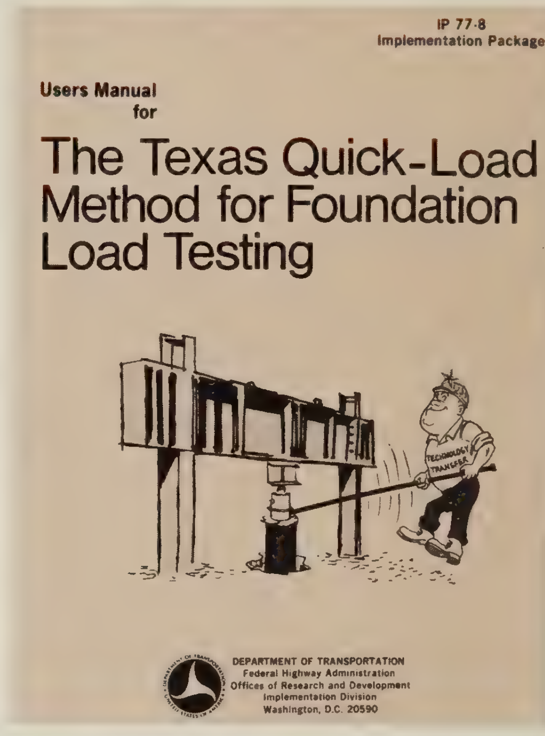
by FHWA Implementation Division

This catalog lists selected publications, visual aids, computer programs, and training materials that are available as a part of the FHWA implementation program.

Items are listed alphabetically under program areas. Sixteen specific areas are designated, and the remaining material is collected under the heading, General. Availability information is included in each listing, and prices are quoted if applicable. Indexes at the back of the catalog are arranged both alphabetically and by program area. A list of Implementation Packages by number is also included.

This catalog replaces an earlier one dated September 1975. The 1977 edition contains more information on the completed works which describe new technology in the highway field.

The catalog is available from the Implementation Division.



The Texas Quick-Load Method for Foundation Load Testing—User's Manual, Implementation Package 77-8

by Texas State Department of Highways and Public Transportation for the FHWA Implementation Division

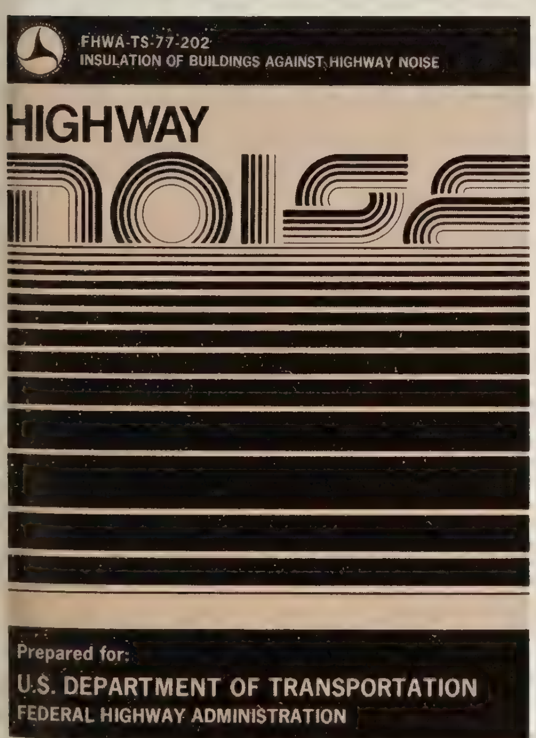
The Texas State Department of Highways and Transportation has developed a method to quickly determine the load bearing capacity of piles and drilled shaft foundations. This method, The Texas Quick-Load Method, can be performed in 1 to 2 hours compared to the more than 100 hours required for the Standard

American Association of State Highway and Transportation Officials (AASHTO) Test Method.

Use of this method on large projects can result in a substantial decrease in the cost of the tests and in the time required to complete a test program. On small projects, where the time-consuming AASHTO test cannot be economically justified, use of the quick-load method makes full-scale load testing feasible.

This report and slide-tape presentation can provide highway engineers with the information necessary to prepare specifications and cost estimates for quick-load testing.

The report is available from the Implementation Division and the slide-tape presentation is available from FHWA regional offices (see p. 125).



Insulation of Buildings Against Highway Noise

by FHWA Implementation Division

When noise strikes a structure such as a wall or window, most of it is reflected, with the remainder being transmitted through by vibrating the structure. Since only a small portion appears on the other side of the structure, the noise has been reduced. When noise enters a building, the level of the noise is reduced. The amount of noise reduction depends on the type of construction used for the walls of the building, the sizes and types of windows and doors, the presence of noise leaks such as ventilation openings, and the amount of acoustically-absorptive material inside the building.

The purpose of this report is to provide members of the highway engineering field with the necessary tools to assess the noise insulation requirements of buildings. This analysis will enable the highway engineer to determine the effectiveness of existing buildings in insulating interior spaces against highway noise. It will also aid in evaluating building modifications which are supposed to increase insulation effectiveness.

The report is available from the Implementation Division.

Bridge Deck Drain Cleanout Device, Implementation Package 77–7

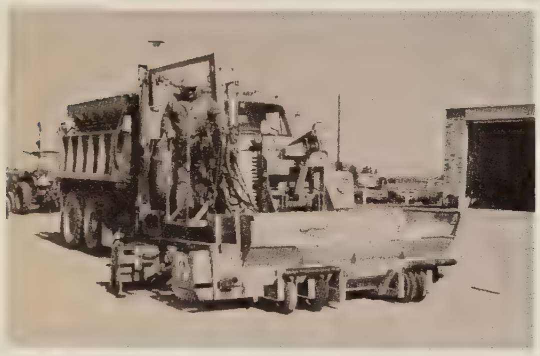
by FHWA Implementation Division

Unclogging bridge deck drain pipe systems can be a difficult and dangerous activity for a highway agency. If the drain cannot be cleared with standard cleaning methods, ponded water may become a hazard for the motorist.

A new cleaning device has recently been developed which uses reverse flushing with compressed air to dislodge the obstruction and then uses water to flush loosened debris from the pipe. A rubberlike plug is inserted in the lower end of the drainpipe to make it air tight for pressures up to 70 psi (490 kPa). The device is relatively simple to assemble and use and should be of interest to highway maintenance engineers.

This report explains the new device, its method of operation, and its advantages, and lists the needed materials and assembly plans.

The report is available from the Implementation Division.



Value Engineering Study of Highway Shoulder Maintenance, Report No. FHWA-TS-77-210

by Arizona Department of Transportation, Idaho Transportation Department, Iowa Department of Transportation, West Virginia Department of Highways, and FHWA

This report summarizes the results of an unpaved shoulder maintenance study undertaken by the Arizona, Idaho, Iowa, and West Virginia State highway agencies. Implementation of the recommendations in the four study States would result in a total estimated improved service value of more than $1 million annually.

Two main shoulder maintenance activities were included in the study: (1) Reshaping the shoulder using material which is in place on the shoulder or shoulder slope, and (2) reshaping the shoulder using material from a separate source.

Specific recommendations varied among the States in response to local conditions. The report presents a selection of techniques for reducing the unit cost of shoulder maintenance activities with the hope of covering a variety of local situations. Individual State circumstances are described in the comments accompanying the recommendations.

The use of larger trucks for hauling, use of side-discharge shoulder spreaders, modifications to the standard motor grader, and spot paving of shoulders in high maintenance locations are among the recommendations made in the report. Some of the suggested changes can be put into practice using existing general purpose equipment. Purchase of additional units may be desirable in some cases. Other recommendations require modification to existing equipment or the purchase of specialized machinery.

The report is available from the Implementation Division.

Value Engineering Study of Snow and Ice—Operations, Report No. FHWA-TS-77-208

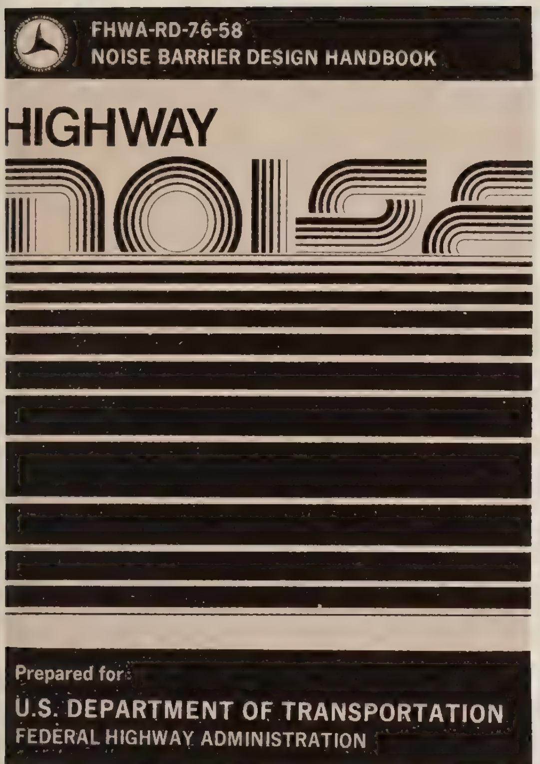
by California Department of Transportation, Colorado Division of Highways, Pennsylvania Department of Transportation, Utah Department of Transportation, and FHWA

This report presents the major results of an indepth study of snow and ice control operations in California, Colorado, Pennsylvania, and Utah during the winter of 1975-76. Aspects of snow and ice control activities—material, equipment, and labor—were analyzed.

Initial estimates indicate potential savings of $5 million among the four States upon implementation of changes recommended as a result of this project. This estimated savings was derived from the accumulation of potential savings from implementing all of the recommendations listed in the report. The control of application rates and the adoption of ground control spreaders provide the greatest potential for immediate cost reduction. It is estimated that at least a 20 percent savings in material (sand, salt, or mixture) can be gained in jurisdictions which do not currently use these procedures and equipment.

Results of the study show that the geographic and climatic differences between the States involved had little effect on snow and ice control operations. The report provides details on the application of salt and abrasives in its discussion of materials. The section on equipment provides data on the use of ground control spreaders, snowplow blades, and single-purpose equipment, such as rotaries and hopper blades.

The report is available from the Implementation Division.



Noise Barrier Design Handbook, Report No. FHWA-RD-76-58

by FHWA Environmental Design and Control Division

This handbook, developed to provide information and guidance in the area of the design and construction of noise barriers, is a useful tool for predicting noise exposure, defining criteria, assessing noise impact, and describing means of noise control.

A simple manual procedure which can be used by the highway designer to design noise abatement barriers is described. The handbook provides a design, evaluation, and selection procedure in which specific barriers are detailed and then evaluated in terms of costs, acoustical characteristics (including reflection and absorption), and nonacoustical characteristics (such as durability, ease of maintenance, safety, esthetics, and community acceptance).

All portions of the design procedure that deal with the acoustic characteristics of the barriers are approved for use in the design of noise barriers on Federal-aid projects. Those portions of the design procedure that deal with costs and nonacoustical characteristics are informational.

The handbook should be included in the "Abatement and Mitigation Section" of the Highway Noise Notebook.

The handbook is available from the Environmental Design and Control Division, HRS–42, Federal Highway Administration, Washington, D.C. 20590.

Water Quality Manual, Implementation Package 77–1

by FHWA Implementation Division

Changes in water quality associated with construction of transportation systems occur mainly as physical pollution in the form of sediment. Chemical and bacteriological pollution can also be present. Hydrologic and hydraulic changes occur when bodies of water are near a project. Effects on the downstream user depend upon the use made of the water. Effects vary from the loss of esthetic appeal to damage to elements of the aquatic ecosystem.

This five-volume manual is designed to assist transportation agencies in conducting water quality studies for use in location, planning, and design.

Volume I, **Planning, Conducting, Analyzing, and Reporting Water Quality Studies for Transportation Projects,** provides an overview of transportation related water quality studies and deals with concepts and the process of planning a water quality study.

Volume II, **Hydrologic and Physical Aspects of the Environment**, discusses physical pollution, primarily in terms of erosion and sedimentation. Hydrologic and hydraulic equilibrium are discussed and techniques for the mitigation of physical impact are proposed.

Volume III, **Erosion Measurements for Road Slopes**, deals with methods for the study and prediction of erosion from existing and future slopes.

Volume IV, **Glossary of Terms for Water Quality Studies,** provides definitions of the names and terms encountered in water quality studies.

Volume V, **Chemical, Bacteriological, and Ecosystem Analysis of Water From Highway Sources for Environmental Impact Studies,** gives a detailed discussion of chemical and bacteriological pollution, an analysis of the effects of impacts on aquatic ecosystems, data analysis, and applicable statistical techniques.

All of the volumes are subdivided into separately numbered sections. A list of references for further reading is included at the end of each section.

The manual is available from the Implementation Division.

New Research in Progress

The following items identify new research studies that have been reported by FHWA's Offices of Research and Development. Space limitation precludes publishing a complete list. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R)—Performing State Highway Department; National Cooperative Highway Research Progran (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1C: Analysis and Remedies of Freeway Traffic Disturbances

Title: Testing and Evaluation of Ramp Control Strategies. (FCP No. 41C3594)
Objective: Determine the effectiveness and computer requirements of selected freeway control strategies for both recurrent and nonrecurrent congestion situations.
Performing Organization: Illinois Division of Highways, Springfield, Ill. 62706
Expected Completion Date: July 1979
Estimated Cost: $115,000 (HP&R)

FCP Project 1T: Advanced Vehicle Protection Systems

Title: An Assessment of Performance of Impact Attenuators Mounted on Curbs. (FCP No. 31T5052)
Objective: Determine the effect of curbs on the performance of impact attenuators through full-scale impact testing and computer simulations. Develop recommendations of guidelines for the use of impact attenuators installed on raised curb gore areas.
Performing Organization: Dynamic Science, Inc., Phoenix, Ariz. 85027
Expected Completion Date: December 1979
Estimated Cost: $194,000 (FHWA Administrative Contract)

FCP Project 1Y: Traffic Management in Construction and Maintenance Zones

Title: Speed Control in Construction and Maintenance Zones. (FCP No. 41Y1664)
Objective: Develop effective traffic protection schemes and test warning devices that can be used in paint striping and other slow moving maintenance operations. Examine the following signing schemes: size and legend of warning signs, height and placement of warning signs, size and shape of arrow boards, and number and intensity of lights on arrow boards.
Performing Organization: New York Department of Transportation, Albany, N.Y. 12207
Expected Completion Date: June 1979
Estimated Cost: $56,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4F: Develop More Significant and Rapid Test Procedures for Quality Assurance

Title: Feasibility of Development of a Portable Nuclear Density Gage for Determining the Density of Plastic Concrete at a Particular Stratum. (FCP No. 44F3303)

Objective: Develop a single- or dual-probe nuclear density gage for determining the degree of consolidation of plastic concrete pavement at a particular stratum.
Performing Organization: Louisiana State University, Baton Rouge, La. 70803
Funding Agency: Louisiana Department of Transportation and Development
Expected Completion Date: March 1979
Estimated Cost: $68,000 (HP&R)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5A: Improved Protection Against Natural Hazards of Earthquake and Wind

Title: Wind Vibration Study of Bridge Over Snake River Near Twin Falls, Idaho. (FCP No. 35A1082)
Objective: Modify and install the FHWA wind instrumentation system to collect, analyze, and present onsite bridge member motion and natural wind data for correlation and prediction of the aeroelastic behavior of bridge members in the presence of known air masses.
Performing Organization: Systems Technology Associates, Inc., Falls Church, Va. 22046
Expected Completion Date: December 1978
Estimated Cost: $70,000 (FHWA Administrative Contract)

FCP Project 5C: New Methodology for Flexible Pavement Design

Title: Fundamental Properties of Construction Materials. (FCP No. 45C2102)
Objective: Conduct laboratory tests on prepared specimens and field specimens to determine those properties which can be used with layer theory and mechanistic analyses.
Performing Organization: Louisiana Technical University, Baton Rouge, La. 70804
Funding Agency: Louisiana Department of Transportation and Development
Expected Completion Date: June 1980
Estimated Cost: $128,000 (HP&R)

Title: A Study of Flexible Pavement Base Courses and Overlay Designs: Second Cycle of Research at the Pennsylvania Transportation Facility. (FCP No. 45C3254)
Objective: Prove a Pennsylvania Department of Transportation overlay design procedure. The performance approach is based on deflection data and the mechanistic approach uses VESYS damage predictions. Both approaches require laboratory testing. Continue analysis of base course structural coefficients. Evaluate the influence of increased axle loadings on pavement performance.
Performing Organization: Pennsylvania Transportation Institute, University Park, Pa. 16802
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: June 1978
Estimated Cost: $248,000 (HP&R)

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Prediction of Structural Condition From Dynamic Deflections in Arizona. (FCP No. 45D1254)
Objective: Examine dynamic deflections in relation to construction

of pavement layers in new pavements, existing structural condition and performance, and varying overlay thicknesses.
Performing Organization: Arizona State Highway Department, Phoenix, Ariz. 85007
Expected Completion Date: June 1980
Estimated Cost: $134,000 (HP&R)

Title: Design of Overlays Based on Pavement Condition, Roughness, and Deflections. (FCP No. 45D2514)
Objective: Develop a tentative method of designing overlay thicknesses for flexible pavements for temporary use by the Virginia Department of Highways and Transportation, and help develop a refined design method for permanent use.
Performing Organization: Virginia Highway Research Council, Charlottesville, Va. 22903
Expected Completion Date: June 1980
Estimated Cost: $58,000 (HP&R)

FCP Project 5F: Structural Integrity and Life Expectancy of Bridges

Title: Experimental Study of the Segmental Box Girder Bridge at Turkey Run. (FCP No. 45F3762)
Objective: Complete development and installation of instrumentation; monitor strain, bending, and temperature changes during and after construction; and evaluate the criteria for designing the subject segmental prestressed concrete box girder bridge.
Performing Organization: Purdue University, West Lafayette, Ind. 47907
Funding Agency: Indiana State Highway Commission
Expected Completion Date: June 1981
Estimated Cost: $60,000 (HP&R)

Title: Curved Box Girder Bridges. (FCP No. 45F3782)
Objective: Develop an analysis/design production computer program for curved box girder bridges, and prepare design equations and tables related to such bridges.
Performing Organization: University of Maryland, College Park, Md. 20740
Funding Agency: Maryland State Highway Administration
Expected Completion Date: June 1981
Estimated Cost: $160,000 (HP&R)

FCP Project 5H: Protection of the Highway System from Hazards Attributed to Flooding

Title: Investigation and Analysis of Floods from Small Northwestern, Strip-Mined, and Forested Drainage Basins in Ohio. (FCP No. 45H3642)
Objective: Develop flow models for prediction of peak discharges of various recurrence intervals for ungaged small drainage basins in these areas in Ohio. Collect 10 years of stream flow data from 30 selected small basins located in reclaimed strip-mined and forested areas and in the Maumee River basin above Antwerp, Ohio.
Performing Organization: U.S. Geological Survey, Columbus, Ohio 43212
Funding Agency: Ohio Department of Transportation
Expected Completion Date: June 1987
Estimated Cost: $629,000 (HP&R)

FCP Project 5J: Rigid Pavement Systems Design

Title: Determination of Importance of Various Parameters on Performance of Rigid Pavement Joints. (FCP No. 45J1364)
Objective: Come up with the best cost joint configuration, pavement slab configuration, and type of subbase that will affect the durability and life of portland cement concrete rigid pavements.
Performing Organization: University of Cincinnati, Cincinnati, Ohio 45221
Funding Agency: Ohio Department of Transportation
Expected Completion Date: August 1980
Estimated Cost: $64,000 (HP&R)

FCP Category 7—Improved Technology for Highway Maintenance

FCP Project 7C: Management

Title: Highway Maintenance Simulation Model. (FCP No. 47C2042)
Objective: Develop computer model to assist highway departments in the management of highway maintenance operations.
Performing Organization: Louisiana State University, Baton Rouge, La. 70804
Funding Agency: Louisiana Department of Transportation and Development
Expected Completion Date: July 1979
Estimated Cost: $124,000 (HP&R)

Non-FCP Category 0—Other New Studies

Title: Use of Asphalt Emulsions in Connecticut. (FCP No. 40M2824)
Objective: Determine how emulsified-asphalt systems compare with conventional mixes and treatments using asphalt cements, cutback, and tars.
Performing Organization: Connecticut Department of Transportation, Hartford, Conn. 06115
Expected Completion Date: January 1980
Estimated Cost: $59,000 (HP&R)

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THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF JUNE 30,1977

COMPLETED OR IMPROVED AND OPEN TO TRAFFIC

Completed to full or acceptable standards, or improved to standards.

Adequate for present traffic; built with Interstate or other public funds.

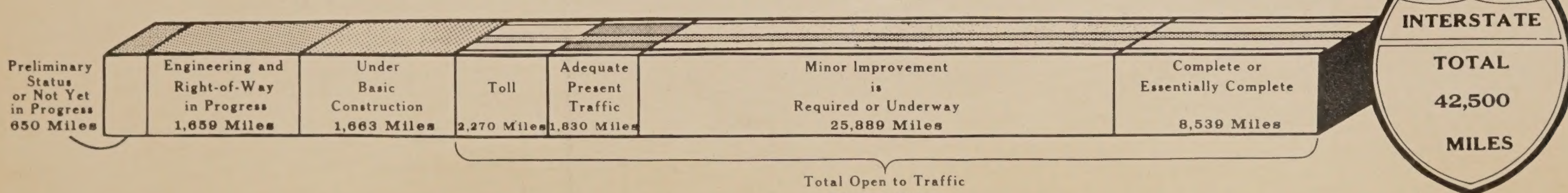
MAJOR TOLL ROADS

Incorporated in the Interstate System

UNDER CONSTRUCTION

PRELIMINARY STATUS OR NOT YET IN PROGRESS

Plan preparation and right-of-way acquisition completed or underway on many portions of these sections



1 mile = 1.6 km

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