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CONTENTS

FOREWORD

This RECORD presents two groups of papers of interest to highway designers. The first three papers deal with decision-making related to geometric features of the roadway, and the final three address the design of pipes and culverts.

The first paper presents an evaluation methodology for selection of interchange configuration. Mulinazzi and Satterly describe a graphical technique in which a cost-effectiveness profile is constructed through relevant interchange evaluation criteria plotted as a set of vertical scales. The selection of an interchange configuration is then determined from comparison of the initial cost and the effectiveness profile of each alternate design.

Neuzil conducted a study about alternative geometric treatments for a variety of common roadway design features to gauge the aesthetic preferences and perceived safety of a sample of the general public. Paired perspective sketches of alternative treatments were used to sample the reaction of two groups of subjects. The survey revealed marked preferences for attractiveness and perceived safety for certain treatments recommended by aesthetic design critics, especially for horizontal and vertical alignment.

Weaver and Marquis present results of a study to provide objective criteria for safe slope combinations and ditch configurations for roadsides. The research approach involved both full-scale vehicle tests and simulated traversals. Recommended design curves are presented for combinations of slopes and ditch shapes.

For the past 30 years, the Iowa deflection equation has been the accepted equation for the design of flexible culverts. One parameter defined as the modulus of soil reaction has a significant effect on the evaluation of pipe deflections. The paper by Parmelee and Corotis investigates the drawbacks of several methods used to evaluate this parameter.

Howard reports the results of field test deflections on two sections of reinforced plastic mortar pipe. One test section was installed in a dry trench by using several types of bedding, and the other section was installed in a wet and unstable subgrade. Results of field measurements indicate that when pipes were installed according to specifications, the pipe deflections were well under design criteria.

The final paper describes the results of a test program to verify the design method and standard designs for precast concrete box culverts reinforced with welded wire fabric. Three box sizes were selected to represent a range of spans, and three designs for each size were selected to represent a range of heights of fill. From the results of the test program, Boring, Heger, and Bealey conclude that the design method provides satisfactory designs for precast box culverts within the range of fill heights and dimensions used for standard designs.

EVALUATION METHODOLOGY FOR SELECTION OF AN INTERCHANGE CONFIGURATION

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The selection of pertinent evaluation criteria is fundamental to the evaluation methodology for deciding on an interchange configuration. The criteria chosen should measure differences between alternative interchange designs. If no such criteria exist, then there is no difference between the alternative designs, and the interchange configuration with the lowest initial cost should be selected. The initial cost was used as the cost indicator for each alternative interchange design. The initial cost was selected because it is easily obtained and does not include some of the uncertainties associated with calculation of road-user costs. The next step is the development of an effectiveness profile for each alternative interchange design. An effectiveness profile is a graphical technique that shows each alternative's effectiveness rating for every evaluation criterion. It is based on the cost-effectiveness approach of economic analysis and is the accumulation of several cost-effectiveness plots into a single graph. The final step is to analyze the initial cost and the effectiveness profile for each alternative interchange configuration. This analysis will provide the decision-maker with the necessary information to select an adequate interchange configuration for the given conditions.

•INTERCHANGES are the weak links in any freeway system because of the vehicular turbulence associated with the inherent merging, diverging, and weaving maneuvers. If the interchanges operate efficiently, then traffic on the freeway will probably flow smoothly.

It does not seem probable that many more miles of new freeway will be built, especially in urban areas. However, those that are built will have to pass a stringent ecological test. The same is true for the rehabilitation of existing freeways, which have become corridors lined with intense land development. Many of the existing interchanges need upgrading and yet, with the adjacent land development, there is no easy way to alter these interchange configurations. An interchange's impact on the community and its traffic operational requirements are opposing forces with which the interchange design engineer must work. He must somehow relate these two forces and arrive at an acceptable interchange configuration. This is the most difficult part in the design of an interchange.

INTERCHANGE SELECTION PROCESS

The purpose of this paper is to present an evaluation methodology that will assist the practicing design engineer in selecting an interchange configuration for a particular location. The total decision-making process recommended to select an interchange type is shown in Figure 1, which demonstrates that the interchange design engineer should be involved not only in the route location study for a new facility but also in the planning study for the rehabilitation of an existing facility. The interchange design engineer can provide valuable inputs to both of these preliminary highway design phases by evaluating the feasibility of the interchange locations and developing preliminary interchange types for these locations. The involvement of the interchange design engi-

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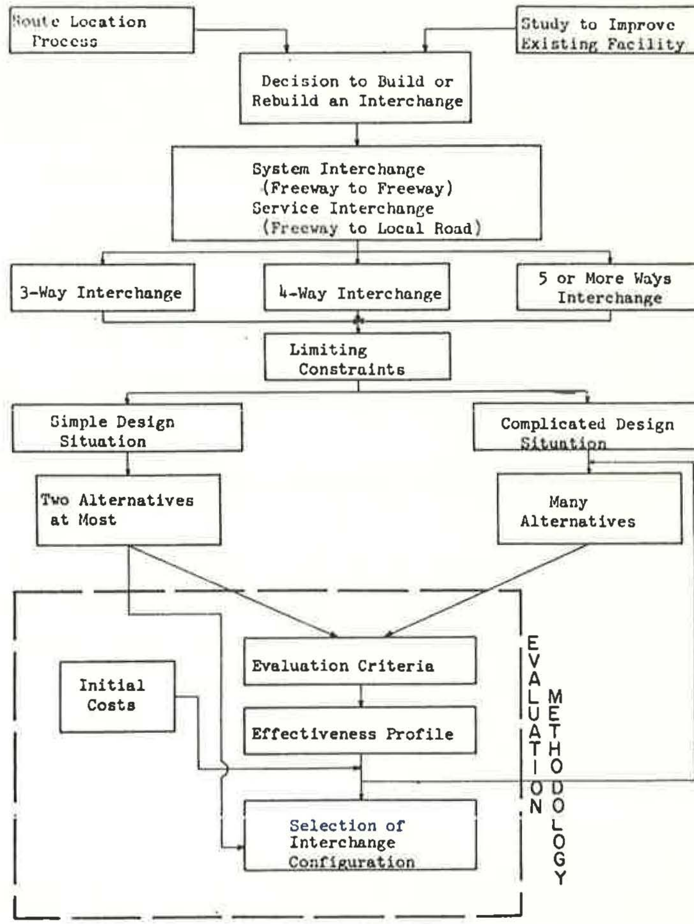


Figure 1. Interchange selection process.

neer at these stages will help minimize the situations where an adequate interchange cannot be built because of predetermined constraints.

Once the determination is made that an interchange is needed, the first step is to determine if a system interchange or a service interchange is required. A system interchange must have all free-flowing ramp terminals for the quick transfer of traffic from one freeway to another.

A service interchange, a freeway to local road connector, usually has stop-controlled or signal-controlled ramp terminals on the crossroad; but in certain areas, free-flowing ramp terminals may be desirable. This division into either a system interchange or a service interchange reduces the set of possible interchange configurations that can be used in any given location.

The number of possible interchange configurations is still further reduced by classifying the desired interchange by the number of approach legs or streets: three-way, four-way, and five or more ways. The interchange types that are applicable, based on the number of approach legs and the classification of the crossroad, follow:

1. Three-way interchanges (three approach roads)—A system interchange involves directional T or Y, trumpet A, and trumpet B. A service interchange involves directional T or Y, trumpet A, trumpet B, half diamond, and hybrids.

2. Four-way interchange (four approach roads)—A system interchange involves directional without loop ramps, directional with loop ramps, and cloverleaf with C-D roads. A service interchange involves directional with loop ramps, cloverleaf with

C-D roads, parclo A-4, parclo A, parclo B-4, parclo B, parclo A-B, diamond with its many variations, and hybrids.

3. Five-way or more interchange (five or more approach roads)—A system interchange involves directional without loop ramps, directional with loop ramps, and hybrids (local ramps within a system interchange). A service interchange involves directional with loop ramps, rotary, and hybrids.

Hybrids are interchange configurations that are modifications of the basic types of interchanges; the modifications are made to meet existing constraints. Rotary interchanges should not be used in this country because of the operational problems associated with their built-in weaving maneuvers.

After narrowing the number of possible interchange types by the functional classification of the interchanging facilities and the number of approach roads, the designer should then determine if the design location has any limiting constraints on the interchange configuration. The existing land use in one quadrant may force the designer to completely avoid that quadrant when he lays out the alternative interchange designs. For example, parks, schools, and other public land are bypassed, if possible. Frontage roads also limit the type of interchange. With a two-way frontage road system, partial interchanges are developed though the use of buttonhook ramps. There are, however, many disadvantages associated with buttonhook ramps. They are usually the second best solution, difficult to sign, induce wrong-way movements when ramps are isolated, and require low design speeds. Buttonhook ramps should be avoided if possible.

Likewise, slip ramps are appropriate for connecting the freeway to a one-way frontage road network, whereas interchanges with loop ramps are not readily adaptable. A natural or man-made obstruction greatly influences the type of interchange. A river or railroad paralleling the crossroad can force all of the ramps to be located in two quadrants on the same side of the crossroad.

The next step is to determine if the particular design problem under study is simple or complicated. A simple design situation would require only one or possibly two alternative interchange designs. Even with a simple or clear-cut design location, it is recommended that two alternatives be developed and compared. An example of a simple design situation is a service interchange between an Interstate route and a low-volume secondary state highway where access is needed because of the long distance between adjacent interchanges. In this case, a diamond interchange would probably be designed. Most interchange designers would find it difficult to justify the time and expense involved in developing another alternative interchange configuration and would consider it a waste of effort to use any detailed evaluation. The interchange design engineer is encouraged, however, to look over the list of evaluation criteria presented later to make sure the design situation is truly simple.

Several alternative interchange designs are developed when a complicated design situation is encountered. The number of alternatives usually varies from two to about ten, depending on the complexity of the design problem. The major obstacles involved in interchange design are in urban areas where development has already occurred and the impact on the environment (the surrounding land) is felt the most. It is also in the urban areas where some of the early freeways are becoming obsolete and are in need of rehabilitation. These highly congested routes have become corridors of high land development because of the accessibility afforded by these freeways. Serious trade-offs have to be made between the community impact factors and the traffic operational factors so that substandard acceleration and deceleration lanes, the closely spaced interchanges, and the congested ramp movements, can be corrected. The following evaluation methodology is proposed to compare these two dichotomous sets of factors.

EVALUATION METHODOLOGY

The evaluation methodology is made up of the following procedures for the interchange design engineer:

1. The list of evaluation criteria should be scrutinized to determine which are pertinent to the design situation under study and which factors should be added.

2. The initial cost for each alternative interchange design should be estimated. The initial cost should include construction costs, right-of-way costs, and relocation costs.
3. An effectiveness profile for each alternative interchange design should be developed.
4. The initial cost of each alternative design should be compared to its effectiveness profile, and the most cost-effective interchange configuration should be selected. If the interchange design engineer doing the work cannot make the final decision on the interchange type, then he should present the initial cost information and the effectiveness profile data to the decision-maker.

Scrutinize List of Evaluation Criteria

There are many criteria that should be considered to some degree in selecting an interchange type, and it is easy to overlook some. The following are some of the evaluation criteria that should be considered in the design of every interchange.

1. Operational and design factors include (a) level-of-service continuity between the main line and the ramps; (b) level-of-service continuity on the crossroad through the interchange area; (c) safety, i.e., uniformity of flow and accident potential; (d) uniformity, i.e., on- and off-ramp design, route continuity, and signing; (e) flexibility, i.e., basic number of lanes, lane balance, stage construction, and maintenance of traffic during construction; (f) number and length of weaving sections; and (g) other factors depending on the design situation and the designer's experience.
2. Community impact factors include (a) number of acres taken outside of the mainline right-of-way; (b) number of families relocated; (c) number of commercial establishments relocated; (d) number of tax dollars removed from the tax rolls; (e) number of local streets closed; (f) taking of a particular parcel of land, e.g., church, school, historical landmark, and public land; (g) lack of access to adjacent property; and (h) other factors depending on the design situation, designer's experience, and community feelings.

These basic criteria include measures of the traffic operational capabilities and design characteristics of an interchange. If certain minimum traffic operational constraints are not met, there is no reason to further consider that interchange configuration. For example, each of the alternative designs must be able to carry the forecast traffic volumes.

The individual designer may have a particular measure or measures that he has used in the past as operational and design criteria for the selection of an interchange configuration. The following are some of these additional criteria: (a) travel time; (b) travel distance; (c) radius of curvature; (d) ramp grades; (e) topography; (f) soil conditions; (g) drainage; (h) spacing of interchanges; (i) design speed; (j) composition of traffic; (k) operating costs—running costs (fuel, tires, oil, maintenance); and (l) level of service.

The community impact factors should be individualized for each interchange design; therefore, no set of criteria is recommended as a minimum measure of the impact on the community from the various alternative interchange configurations. The objective is to minimize the detrimental community impact while maximizing the traffic operational capabilities of the interchange. Trade-offs between these two dichotomous interchange consequences are always present.

There are several more prevalent community impact factors. Additional factors include noise and air pollution, local street connectors, landscaping opportunities, land development opportunities, local planning values, barrier effects, and aesthetics.

These operational and design factors and community impact factors are intended to be open-ended because it is impossible to include in this paper all of the factors that could influence the selection of an interchange configuration. The designer should anticipate the evaluation criteria considered important by the public and include these in the evaluation process. The important thing is to include the factors or evaluation criteria that affect the possible interchange type. Without a set of evaluation criteria

as a foundation to measure the differences between the alternative interchange configurations, the proposed evaluation methodology is weak at best.

Develop Initial Cost for Each Alternative Design

The initial cost of each alternative interchange design is used in the evaluation methodology because it is easily obtainable and does not include some of the uncertainties associated with calculating road-user costs. Included in the initial cost are construction costs, right-of-way costs, and relocation costs, e.g., utilities and families and businesses.

Road-user costs are not included in the determination of the cost of each alternative design because of the problems associated with calculating dollar values. In arriving at a value for time, the accumulation of small increments of time and the uncertainty associated with the monetary value of a fatality are some of the questionable areas. It is also felt that the road-user costs would not be significantly different for alternative interchange configurations.

If the designer feels that some measure of road-user costs should be included in the evaluation process, he could always include it as an evaluation criterion. For example, the present worth of operating cost could be included in the analysis as a measure of the effectiveness of the alternative designs: The lower the operating cost is, then the more attractive will be that alternative design. The designer should make an honest attempt, however, to accurately determine the operating cost. He should not take the average of the existing annual traffic and the projected annual traffic as the yearly traffic over the life of the project and apply the fuel, oil, maintenance, etc., factors. Operating costs vary not only over the duration of the project and over the increase in traffic but also by the hour of the day. Maintenance costs are not included because again it is felt that it would be better to include them as an evaluation criterion.

Develop an Effectiveness Profile

A technique is needed to compare the impact of the alternative interchange designs based on qualifiable as well as quantifiable criteria. There are several approaches that this evaluation procedure could take. It can simply be a rote process, similar to the interchange design table found in one of state highway design manuals. This technique of interchange configuration selection leaves nothing to the design engineer's imagination or ingenuity. The designer simply goes to a predeveloped table or chart and selects an acceptable interchange configuration.

One form of evaluation methodology applies economic measures such as the benefit-cost ratio, rate of return, or net present worth. These techniques are primarily based on (a) first costs such as cost of construction and right-of-way costs, and (b) motor vehicle operating costs, such as those associated with accidents, delays, and travel time costs. The alternative with the best ratio or economic index is the selected interchange configuration.

Another technique uses a point weighting scheme (1) similar to the sufficiency rating method of evaluating highway pavements to determine the best interchange configuration. The alternative with the highest numerical score is accepted as the most appropriate solution. Figure 2 (1, p. 21) shows this numerical approach for the selection of the proper interchange type (alternative 2). One of the noteworthy aspects of Leisch's methodology (1) is that the costs only constitute 25 percent of the evaluation weight.

Oglesby, Bishop, and Willeke (2) clearly state the basic problem with most of these previously mentioned evaluation techniques.

> A general criticism of these approaches is that they have failed to recognize the two basic principles of decision making: (a) decisions must be based on the differences among alternatives; and (b) money consequences must be separated from the consequences that are not reducible to money terms, and then the "irreducibles" must be weighted against the money consequences as a part of the decision making process.

Figure 2. Comparison of alternative interchange solutions.

Output Variables (Comparison Items)	Scale Value	1		2		3	
(1)	(2)	(3)	(4) 2x3	(5)	(6) 2x5	(7)	(8) 2x7
Operation [30]							
Speeds of operation	(5)	10	50	8	40	6	30
Travel distance	(5)	10	50	9	45	7	35
Safety - compr. & antic.	(5)	10	50	10	50	7	35
Safety aspects - other	(5)	8	40	10	50	7	35
Capacity	(10)	10	100	8	80	6	60
Costs [25]							
Capital	(15)	6	90	9	135	10	150
Operating	(10)	10	100	10	100	10	100
Implementation [15]							
Construction staging	(10)	6	60	10	100	8	80
Maintenance of traf.	(5)	8	40	10	50	10	50
Environmental [30]							
Traffic disturbances	(5)	6	30	10	50	10	50
Aesthetic qualities	(5)	6	30	9	45	8	40
Barrier Effect	(5)	5	25	10	50	7	35
Impact on develop.	(15)	5	75	9	135	10	150
Total (Index Value)	(100)		740		930*		850
(*Best Alternative)							

Grant and Oglesby (3) make the following statement about highways and freeways, but it also seems very pertinent to the design of an interchange.

> In many cases some consequences of decisions among highway alternatives (interchanges) cannot be expressed in terms of money. Furthermore, the "irreducibles" to whomever they may accrue are relevant to the decision. In these situations the "dollar" answers from the economy study do not dictate the final choice; but on the other hand they provide a money figure against which the irreducibles can be weighed and thereby narrow the area of uncertainty with which the decision maker is faced.

Wattleworth and Ingram (4) tried to overcome these problems by applying the cost-effectiveness methodology to the analysis of alternative interchange design configurations. They recognized the "need for a procedure that can be quickly used by a designer to compose alternative interchange design (or redesign) configurations and that considers the cost of each configuration as well as the effectiveness of the interchange." The effectiveness measure used in this research was the total interchange capacity, expressed in terms of equivalent average daily traffic entering the interchange. The cost measure was in terms of the initial costs of the project. Before this cost-effectiveness approach was developed Wattleworth and Ingram formulated a linear programming model to determine interchange capacity (5). This linear programming model, itself, would be a good tool to determine the proper interchange configuration, if capacity was the only measure of effectiveness that was used.

During field interviews with interchange designers it became apparent that there was no generally accepted evaluation methodology for the comparison of alternative interchange configurations. In most rural areas there is no problem; diamond interchanges are used most of the time without any comparison to other configurations or without any evaluation of traffic operations or of the effect on land use, etc. However, when a decision has to be made because of a complicated design situation, there is no accepted methodology that could be used in the selection of an interchange type.

An appropriate evaluation methodology for the comparison of alternative interchange

configurations must include nonmarket variables as well as market variables. The best way to incorporate these nonmarket variables into an evaluation methodology is through the use of the cost-effectiveness technique.

The application of the cost-effectiveness approach in this paper results in an effectiveness profile that is a set of vertical scales; each vertical scale represents a different criterion. For each alternative design, its effectiveness rating for every evaluation criterion is plotted on the proper vertical scale. Straight lines are then drawn that connect the appropriate effectiveness ratings to form an effectiveness profile for each alternative configuration. The final effectiveness profile is a compilation of two or more cost-effectiveness curves into one graph. The effectiveness profile is an expansion of the community factors profile developed by Oglesby, Bishop, and Willeke (2) as a method for decisions among freeway location alternatives based on user and community consequences. Figure 3 is an example of an effectiveness profile used to evaluate three alternative interchange configurations.

The effectiveness ratings are measured objectively if possible (in terms of level of service, acres required, number of families relocated, etc.) or subjectively (poor, fair, good, excellent) based on the designer's experience and community attitudes. The bottom line of the effectiveness profile represents the lowest or worst possible effectiveness rating, and the top line the highest or best possible effectiveness rating for each criterion. Each vertical scale is subdivided into equal segments between these two extreme measures of effectiveness. If no predetermined maximum or minimum value can be set for a vertical scale, then the best effectiveness rating for the given alternative designs should be scaled on the top line and the worst effectiveness rating on the bottom line.

Some of the evaluation criteria may have a minimum acceptable effectiveness limitation that is more restrictive than the lowest possible effectiveness rating and that is represented by a horizontal line across the vertical scales representing those criteria.

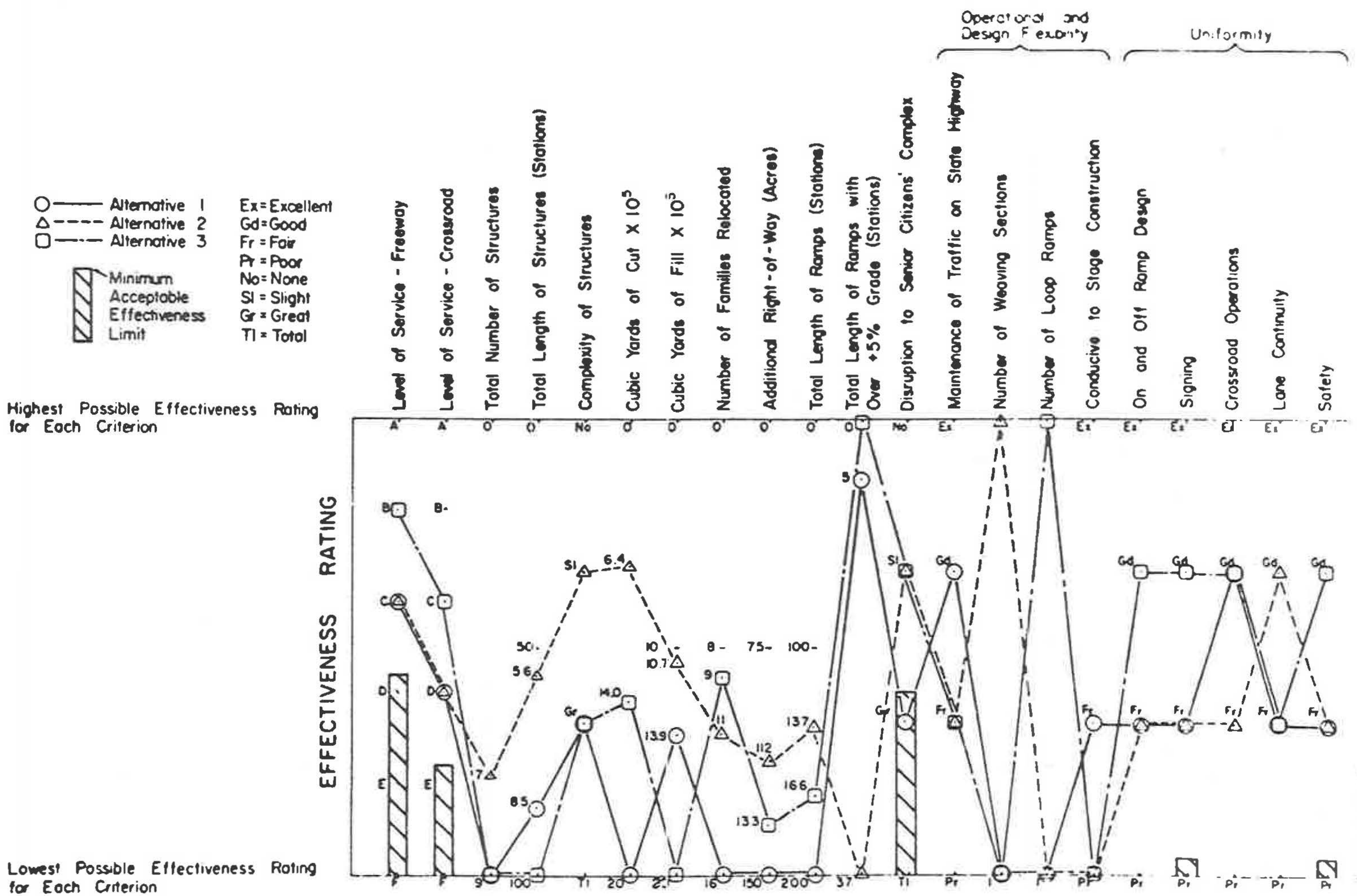
If a minimum acceptable effectiveness limit is assigned to an evaluation criterion, it should be done a priori and not after the effectiveness profile has been developed. The segment of the vertical scale below this minimum acceptable effectivenes limit is an area that indicates rejection of any alternative whose effectiveness rating falls in it. This rejection of the alternative design should be final unless conditions are changed that either alter the minimum acceptable effectiveness limit or improve the interchange design so that the alternative's effectiveness rating increases above this limiting constraint. For example, in Figure 3 the criteria, level of service on the freeway and on the crossroad and the disruption to the senior citizens' complex, have minimum acceptable effectiveness limits.

The changing of either the minimum acceptable effectiveness limit or the effectiveness rating because of some design alteration lends itself quite readily to a rough form of sensitivity analysis. By making either of these changes, alterations occur relative to the differences between the alternatives and possibly result in the selection of a different alternative design.

Evaluation criteria that indicate similar characteristics for the three alternative interchange designs are not included in the effectiveness profile; however, they are important for deciding whether an interchange should be constructed. If all three alternative configurations have a similar positive characteristic, then any of the three types could be built based solely on this factor. But if all three alternative configurations possess the same absolute negative characteristics, then the decision process becomes more complicated. For example, if all three alternatives require taking a certain parcel of land that is unattainable, then there is no feasible alternative among the three given, and either additional alternative designs must be developed or the total project abandoned.

It is also possible to place confidence limits on the effectiveness ratings for certain subjectively measured criteria. For example, the effectiveness rating for alternative 1 for the safety criterion might range from good on the high side to fair on the low side. As long as the confidence limits do not intersect a minimum acceptable effectiveness limit, they will show the possible ranges of acceptable effectiveness ratings. If they do go below the minimum level, then a judgment has to be made on the probability of attaining an unacceptable design.

Figure 3. Effectiveness profile.



Select an Interchange Configuration

In a simple design situation for which only one interchange configuration is developed, there is no need for an evaluation methodology because the interchange configuration is already selected. However, when a choice must be made between two or more alternative interchange types, the decision-maker, be he the interchange design engineer or his superior, should analyze the effectiveness profile of each alternative design. After eliminating those alternative designs that do not meet all of the minimum attractive effectiveness limits or that are dominated by another alternative design, the decision-maker is left with interchange configurations that meet minimum requirements. In the effectiveness profile (Fig. 3), one of the alternative designs could be quickly eliminated from further consideration. Alternative 1 causes too much disruption to the senior citizens' complex, and this is unacceptable to the community. The basic decision, then, is between alternatives 2 and 3. After comparing the initial cost of each of these remaining interchange types, the decision-maker should be able to make a decision on the type of interchange to design.

DISCUSSION

This graphical display of alternative consequences, the effectiveness profile, should be useful in many ways for the design engineer. It will provide him with an easily understood representation of the overall effects of each alternative design. The effectiveness profile (besides being an aid to the design engineer and his technical associates) should be a helpful visual aid at a public meeting because it clearly illustrates which criteria were used and their effectiveness ratings. The public may not agree with some of the effectiveness ratings, but at least they will be able to see how the designer arrived at his decision. The public will also be able to see the influence of any absolute criterion by seeing which alternatives were dropped from further consideration because they did not meet certain minimum acceptable effectiveness limits.

The effectiveness profile could be very useful as an indicator of the monetary value of qualifiable variables. After many years of interchange design evaluations, it may be possible to review the effectiveness profiles of past evaluations and quantify the monetary value of the qualifiable variables or at least recognize which qualifiable criterion carried weight in previous decisions. For example, if a certain evaluation criterion seems to be prevalent when the cheapest design alternative in terms of dollars is not chosen, then it should be possible to assign some dollar value to this criterion.

The effectiveness profile should encourage design variations after the initial alternatives have been developed. If an alternative meets all of the evaluation criteria except one or two, the decision-maker should feel compelled to see what would happen to the decision outcome if he were to make modifications to the rejected alternative design so that it would at least meet all of the minimum acceptable effectivness limits. This procedure will provide the decision-maker with a method of evaluating the results of placing certain constraints on the design.

The effectiveness profile (depending on the selection of evaluation criteria) should be sensitive enough to register any significant differences in alternative interchange configurations. The operational differences between a tapered off-ramp and a parallel off-ramp will not be noticed unless the designer makes this design element one of the evaluation criteria. Significant design variations, e.g., a loop ramp versus a diamond type ramp, will definitely register in the effectiveness profile.

The strength of the proposed evaluation methodology is contingent on the selection of the evaluation criteria and the development of the effectiveness profile. The evaluation methodology is simple to apply and should not require much time. These attributes are necessary for practicing interchange design engineers to use in the selection of an interchange configuration.

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AESTHETIC PREFERENCE AND PERCEIVED SAFETY IN HIGHWAY DESIGN TREATMENTS: A PILOT SURVEY

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This study was conducted to test the hypotheses that the public does indeed have a marked preference for highway design treatments incorporating design principles promulgated by students of aesthetic highway design and that these treatments are also perceived as offering greater motoring safety than those provided by aesthetically more austere design practice. A survey of two groups of subjects, which used paired sketches of alternative design treatments for specific design features, revealed marked preferences in terms of attractiveness and perceived safety for certain treatments recommended by aesthetic design critics. These preferences were particularly great in connection with elements of horizontal and vertical alignment. Survey respondents generally showed a greater preference for the critics' designs in terms of perceived safety than on the basis of attractiveness of appearance.

•PRINCIPLES and guidelines underlying aesthetic highway design are by no means recent developments in highway design technology produced in response to current widespread public concern over the impact of engineered works on physical and social environments. A small but historically significant number of segments of aesthetically designed parkways and turnpikes were built in the eastern United States and Europe long before the Interstate Highway System was begun (1, 2), and 20 years have passed since AASHTO summarized the most important guidelines for producing a rural highway design that would be pleasing to the eye as well as functional and economical (3). Indeed, AASHTO and others have indicated that aesthetic design guidelines can often be satisfied without conflicting with total economy requirements in highway construction and operation, and careful attention to design aesthetics can help to ensure safe and smooth flow of traffic over the completed facility (4, 5, 6).

A cursory review of announcements of awards for aesthetically designed highways reveals consistent use by the judges and critics of terms such as flowing alignments and blending with the terrain—terms indicative of the critics' strong preference for certain approaches to the design of horizontal and vertical alignment, cross section, and structures. These design approaches have been emphasized in AASHTO's rural design policies and are of increasing importance in state highway department design manuals. The existence of such award programs is perhaps, however, an acknowledgment that aesthetic design principles have yet to become incorporated as major components of design philosophy in the day-to-day activities of many highway designers.

The views of students and critics of aesthetic highway design have been well-documented. This paper summarizes the results of a pilot effort aimed at gauging the aesthetic preferences of the public about alternative treatments for common roadway and roadside design situations. At the same time individual personal appraisals of the comparative safety of these alternative treatments were obtained and were compared with aesthetic preferences. The survey was designed to provide a test of the hypotheses that the public does indeed have a strong preference for the aesthetic design

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*This paper is based on work performed while the author was with VTN Washington, Bellevue, Washington.

treatments promulgated by design critics and that these design treatments are also perceived as offering greater motoring safety.

SOME BASIC CONSIDERATIONS

Aesthetic preferences in highway design treatments are a matter of individual judgment, and, as in all matters involving aesthetic judgments, there exist no true or correct answers or statements. There are of course the critics' consensuses, but these have not been absolute. Nevertheless, if there are marked aesthetic preferences among the public about specific highway design features, it is incumbent on the highway designer to incorporate the preferred design treatments where possible, consistent of course with the dictates of safety, economy, and traffic flow requirements. The complete highway design maximizes the aesthetic benefits to the motorist within the matrix of engineering, economic, and environmental constraints. Certainly, that a highway should be attractive as well as functional is no more an arbitrary rule than that main rural highways should be designed for 70- to 80-mph (113 to 129 km/h) design speeds. (An enriched motoring experience due to provision of a high level of visual design amenity should be taken as a given.)

The pleasurable experience of driving on an attractively designed highway may contribute to reduced driving strain and fatigue—factors that bear significantly on highway safety. More directly, one must consider the perceived safety or perceived hazard and the driver's response to them. For example, the minimum curve for a given design speed provides the driver with an adequate level of operational safety physically. As he approaches the curve, however, he may perceive the curve as somewhat unsafe in appearance (even at his reasonable approach speed), particularly if other geometric design features prevail over the segment of highway—vertical alignment conditions, presence or absence of a transition curve, etc. If the driver's perception and judgment of safety of the curve ahead cause him to slow down unnecessarily, traffic friction, accident potential, and motoring strain and discomfort may be unnecessarily increased for the motorist and possibly for adjacent drivers on the road as well. Each mile of travel provides many such potential experiences. In this regard, compare driving on a highway with graceful flowing alignment with driving on a highway with design-minimum horizontal and vertical curves and uncoordinated long-tangent-short-curve alignment and profile.

In their classic treatise on aesthetics of highway design, Tunnard and Pushkarev (1) cite accident fatality rates on 13 parkways and turnpikes, which appear to indicate a rather strong relationship between aesthetic quality and safety. Highways with monotonous character were shown to have fatality rates generally twice that of more attractive highways, although other data reveal no clear differences in total accident rate (7).

STUDY PROCEDURE

A two-part questionnaire consisting of paired sketches of alternative design treatments for alignment, cross section, and minor structures was administered on a controlled basis. In part 1, respondents were requested to select the alternative (from two sketches) for each design treatment that they judged to be the most attractive. In part 2, they were asked to appraise the same design treatments and select the alternatives that they judged to be safer in appearance. Respondents were to answer on a first, quick-impression basis.

Alternative Design Treatments

A number of perspective sketches of alternative design treatments (1) were supplemented by similar sketches prepared by the author. A pair of sketches illustrating two basically different approaches to handling a particular design element or design feature were placed together on each page of the questionnaire. The questionnaire covered most of those design elements that are considered to have substantial significance for aesthetic design character. For each design element under consideration, one sketch

showed a treatment that is considered by aesthetic design critics to be the more attractive, and the other showed a less attractive treatment, often typical of design minimums and a design philosophy emphasizing minimum construction cost or minimum design effort (Figs. 1 through 12). The design treatment alternative preferred by critics for its superior aesthetic character is indicated on each figure. The distinguishing characteristics of the treatments are discussed later.

Some of the design elements used in the survey questionnaire were selected on the basis of their frequent occurrence of dominance in the driver's motoring experience, and others reflect visual situations that are of greater significance for the passenger, abutter, pedestrian, bike rider, or motorist parked at a scenic turnout or rest area. In addition, nearly all of the design elements or features used were those for which the alternative treatments were believed to elicit a strong preference for perceived safety.

The Survey

The questionnaire was administered to two groups of subjects: (a) secretaries and typists and (b) undergraduate civil engineering students. The former were personnel in engineering departments of a mid-Atlantic university and regular and temporary secretarial staff in eastern and western consulting engineers' offices. The students were surveyed in a lower division surveying course and an upper division transportation engineering course. The student survey was conducted early in the term to minimize any possible bias by instruction in courses involving geometric design of highways. Total sample size was 109 persons.

In part 1 of the questionnaire (aesthetic preference), highway scenes appeared in the same order as in Figures 1 through 12. In part 2 (safety appraisal), scenes were presented in the following order (based on Fig. number): 9, 5, 1, 8, 10, 2, 12, 7, 4, 11, 3, 6. The various scenes were ordered so that the several classes of design elements under consideration would be alternated—horizontal alignment, vertical alignment, roadside treatments, etc. The purpose of the differing orders for the aesthetic and safety portions of the questionnaire was to minimize any mental carry-over from the respondent's aesthetic selection to his safety selection and to hinder the respondent should he want to use his aesthetic choice when making his safety appraisal. In the larger engineering course, some of the questionnaires had the aesthetic part first (the standard order) and others had the safety part first. No significant differences were found in the results for these two groups, and they were combined in later analyses.

RESULTS

The percent response to each design feature is shown in Figures 1 through 12. The response for secretaries, civil engineering students, and both groups combined is as follows:

1. A—Percentage of respondents who preferred the critics' aesthetic choice when they were questioned about aesthetics;
2. S—Percentage of respondents who preferred the critics' safety choice when they were questioned about safety;
3. AS—Percentage of respondents who preferred the critics' choice for both aesthetics and safety; and
4. S/A—Percentage of respondents who preferred the critics' choice for safety, given preference of the critics' choice for aesthetics.

Table 1 gives a summary of the results for the entire survey sample. In general, agreement with the critics was greatest on alignment features as opposed to cross section elements, with the exception of the top rating given to cut remnant removal (Fig. 1). Substantial preferences were shown for long vertical curves and spiral transition curves. Responses in the two cases involving spirals (Figs. 4 and 12) were nearly identical.

The hypothesis that design treatments recommended by aesthetic critics are preferred was rejected in 4 of the 12 design cases (0.01 significance level), with all of

Table 1. Summary of survey results.

Figure Number	Design Feature or Element	*Design Treatment Recommended by Aesthetic Critics*	Percent of "ALL" Respondents Favoring Aesthetic Design Preferred by Critics			
			When appraised for: Aesthetics (A)	When appraised for: Safety (S)	For both Aesthetics & Safety (AS)	For Safety, given a favorable response for Aesthetics (S/A)
1	Cut remnant in vista area	*Remove remnant*	94	66	62	66
2	Roadside drainage channel and cut slope	*Broad, rounded channel and rounded slopes*	78	79	64	81
3	Abutment wing-wall I (wall profile)	*Smooth wall profile (avoid "steps")*	52	64	42	79
4	Horizontal curve transtion I (vicinity of sag vertical curve)	*Use spiral transition*	76	92	71	93
5	Fill slope and guardrail	*Eliminate guardrail, use flat, rounded slopes*	51	35	24	46
6	Flow of alignment	*Continous curvilinear alignment*	75	94	74	98
7	Vertical curves over hump	*Use long vertical curves*	76	87	74	95
8	Sag curve on long horizontal curve	*Use long vertical curves*	70	82	65	91
9	Abutment wing-wall II (straight or flared wall)	*Flared wall*	53	63	39	72
10	Embankment slope and abutment	*Flat, rounded slopes and open end span*	59	69	49	83
11	Sag vertical curve	*Use long vertical curve*	78	93	76	98
12	Horizontal curve transition II (on level tangent)	*Use spiral transition*	73	89	71	96

○ Reject hypothesis that "ALL" respondents prefer design preferred by aesthetic critics (at 0.01 level)

the rejections involving cross-section and minor structure elements. Three of these four treatments were also rejected in the test of the hypothesis that the critics' design treatments are perceived as the safer designs.

Favorable responses about safety for the design treatments recommended by critics were greater than for aesthetics in 10 of the 12 cases. In the appraisal for safety, six of the critics' preferred design treatments were favored by over 80 percent of the respondents; three of these treatments received responses above 90 percent. In the appraisal for aesthetics, however, only one design treatment received a preference above 80 percent (cut remnant removal, 94 percent).

For those respondents who preferred the critics' choice for aesthetics, the percentage favoring that design for safety (S/A) was generally quite high: above 90 percent in six of the 12 cases and 80 percent scores for two additional design cases. For Figures 6 and 11, which show alignment features, response percentages were 98 percent. The hypothesis that those who prefer the critics' choice for aesthetics also prefer it for safety was rejected in only one design case: fill slope and guardrail (Fig. 5).

On a purely random choice basis the percentage of respondents preferring the critics' design treatment for both aesthetics and safety (AS) would be 25 percent (0.5×0.5).

However, only one design case (fill slope and guardrail, Fig. 5) received less than 25 percent, and responses greater than 50 percent obtained in eight of the design cases.

Overall, the engineering students registered a higher preference than the secretaries for the critics' design treatments when they were rated for either aesthetics or safety, with greater differences in the safety ratings. Nevertheless significant differences between the two survey groups (0.01 significance level) were found only for safety in abutment wall treatments (Figs. 3 and 9) and for aesthetics in Figure 7 (hump vertical curve). In the abutment wall cases, the tests of the basic hypotheses of the study were statistically rejected on the basis of the total sample response.

CHARACTERISTICS OF DESIGN TREATMENTS AND DETAILED RESULTS

Removal of Cut Remnants

Removal of cut remnants (Fig. 1) in heavy terrain can often provide the motorist with a dramatic view of distant scenery, which is similar to scenic enhancement through selective clearing of trees along the roadside. Although the viewpoint for Figure 1 is actually from the roadside (a scenic turnout, rest area, or abutting development), it adequately demonstrates the scenic enhancement that can be made available to the motorist through removal of a cut remnant. This treatment was preferred by 94 percent of the respondents from the standpoint of aesthetics, although only 66 percent appraised it as the safer of the two treatments.

The basis for perceived safety aspects of these two treatments is no doubt rather complex. Both the cut remnant and the distant view effectively tell the motorist that a horizontal curve is being approached. The remnant together with the roadway curvature could present a somewhat restricted or confined appearance to some motorists, which could possibly cause unnecessary deceleration. On the other hand the dramatic view could cause inattention of some drivers as well as cause deceleration so that scenery could be viewed longer or in anticipation of a possible scenic turnout along or beyond the curve. A few motorists will no doubt stop on the shoulder in the absence of a turnout to take photographs or to leisurely view the scene. The latter presents considerable safety hazard in the case of the freeway, where such behavior is less anticipated by other motorists, than in the case of the two-lane "shun-pike", parkway, or public domain highway.

Rounded Drainage Channels and Slopes

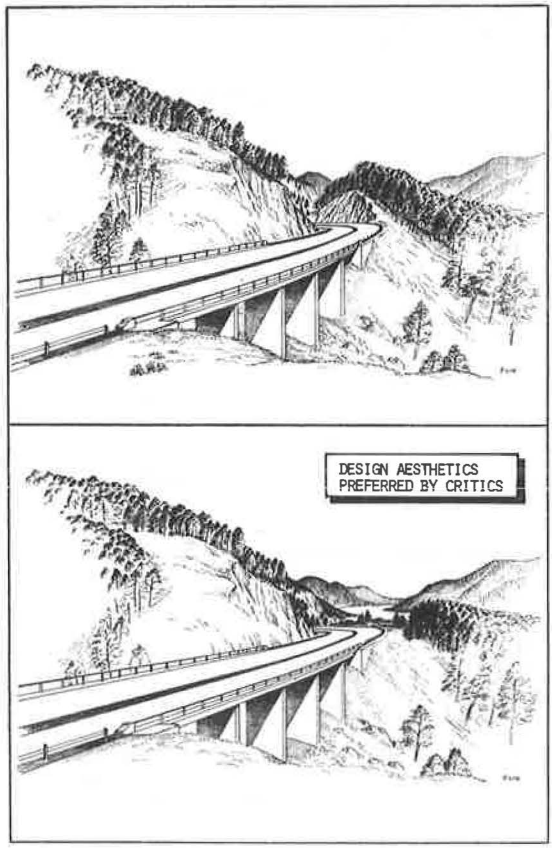
Wide, rounded drainage channels and cut slope rounding (Fig. 2) soften the grading scar, reduce slope and channel maintenance, and provide a safer roadside for the out-of-control vehicle. The greater usable shoulder width obtained with the rounded foreslope also adds to traffic safety. The vee-ditch treatment, especially when combined with narrow shoulders, causes some drivers to encroach on the adjacent lane. The critics' preference was supported by 78 percent of the respondents in terms of aesthetics, and 79 percent appraised it as the safer design. Nearly all students who agreed with the critics' choice for beauty also rated that treatment safer, although a much lower percentage of the secretaries who preferred the aesthetic design perceived it as safer.

Retaining Wall Profiles

Smooth wing wall and retaining wall profiles (Fig. 3) provide a clean, attractive appearance and usually require less construction effort than stepped designs and other "meaningless attempts at ornamentation" (1). However, for the alternative treatments shown, the percentage of respondents agreeing with the critics was not significantly greater than 50 percent (chance agreement) to accept the hypothesis that the respondents prefer the critics' choice for aesthetics.

Although the vertical faces of the stepped design present additional hazard to a motorist who runs off the road, the overall difference in safety between the two designs is not dramatic. There was a significantly higher safety preference among engineering students compared to secretaries for the straight design, but the percentage for both

Figure 1. Cut remnant.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	96	91	94
S	64	69	66
AS	60	63	62
S/A	60	69	66

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	55	54	109
S	55	54	109
AS	55	54	109

LEGEND

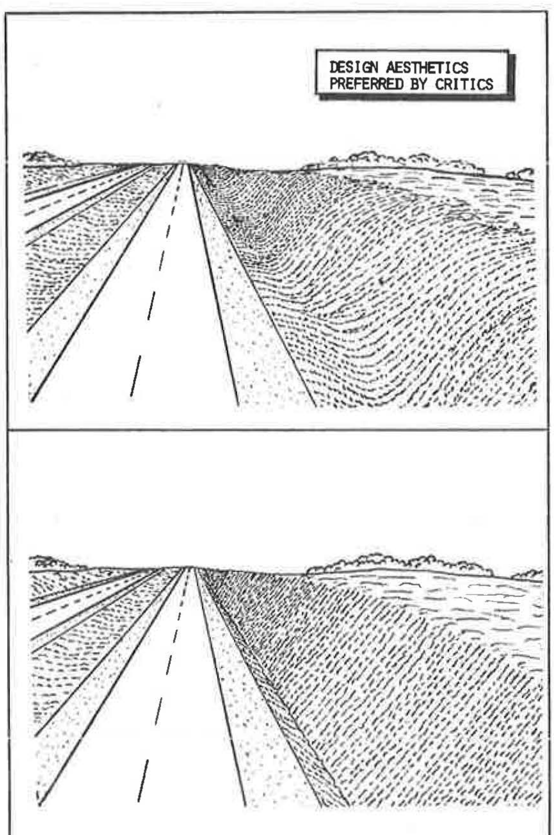
A Appraised for aesthetics

S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

Figure 2. Roadside drainage channel and cut slope.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	78	78	78
S	70	87	79
AS	53	74	64
S/A	67	95	81

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	54	54	108
S	54	54	108
AS	53	54	107

LEGEND

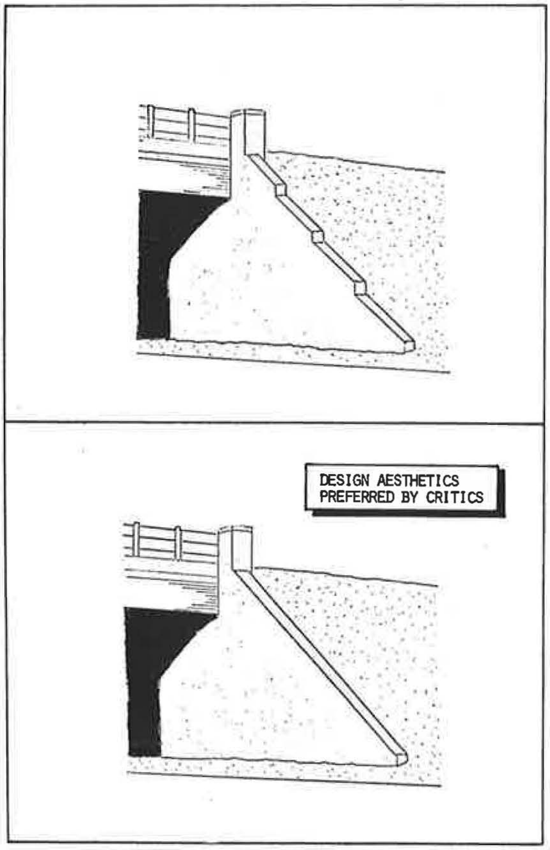
A Appraised for aesthetics

S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

Figure 3. Abutment wing wall 1.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	42	63	52
S	52	76	64
AS	32	52	42
S/A	74	82	79

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	55	54	109
S	54	54	108
AS	54	54	108

LEGEND

A Appraised for aesthetics

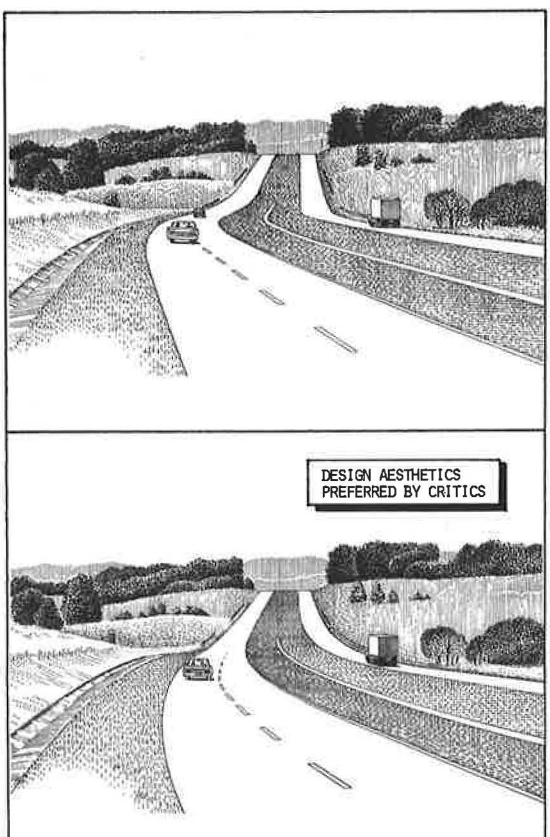
S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

○ Reject hypothesis that "ALL" respondents prefer design preferred by aesthetic critics (at 0.01 level)

Figure 4. Horizontal curve transition 1.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	73	79	76
S	87	96	92
AS	66	77	71
S/A	90	95	93

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	55	53	108
S	55	53	108
AS	55	52	107

LEGEND

A Appraised for aesthetics

S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

groups combined (64 percent) was not significantly greater than 50 percent. Nevertheless, the percentage of respondents who preferred the critics' aesthetic choice and believed it to be safer (S/A) was great enough that the hypothesis that the attractive design is also perceived as the safer design could not be rejected.

Curvature at Horizontal Curves

Transition curvature at horizontal curves (Fig. 4) is an essential element of smooth flowing highway alignment. The visually disturbing kinks in horizontal alignment resulting from lack of a spiral or another suitable form of transition curve are heightened in the presence of certain vertical alignment features, e.g., in the vicinity of a sag vertical curve (Fig. 4). The appearance is worse in divided highways than in two-lane highways because of the presence of two additional edge-of-pavement lines and, possibly, a median barrier. The appearance of the opposing roadway particularly suffers when spirals are absent in designs with intermediate width medians, especially where the roadways are not on independent alignments.

Although 76 percent of the respondents preferred the aesthetics of the design with spirals, an even higher percentage, 92 percent, preferred that treatment from the standpoint of perceived safety. The favorable response for safety among those who preferred the aesthetics of the spiral design was 93 percent.

Flat, Rounded Fill Slopes

Flat, rounded fill slopes (Fig. 5) are preferred by aesthetic design critics over fill slopes with guardrails, and in many situations the former excel in both safety and total economy (5). However, the 51 percent overall aesthetic preference and 35 percent safety preference for the flattened slope treatment do not support the basic hypothesis of this study. This aesthetic design treatment received the lowest aesthetic and safety preferences of the 12 design features tested. The sketches may well have been of inadequate quality to effectively convey the basic differences in the two alternative treatments, and it is possible that the adjacent flat area could have been interpreted as a body of water rather than as field, although there was no attempt to verify this.

Flowing Alignment

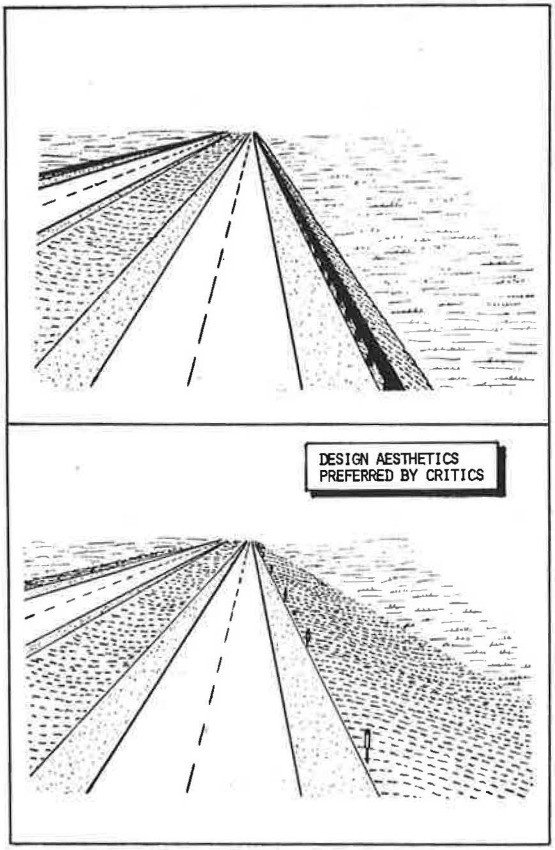
The flowing alignment (Fig. 6) created by careful coordination of horizontal and vertical alignment is frequently used to characterize classic aesthetic design in rural highways and is portrayed in the upper sketch. The long-tangent-short-curve treatment with poorly coordinated plan and profile, still all too common in American highway design, is shown in the lower sketch. Of the respondents, 75 percent preferred the aesthetics of the flowing alignment, and 94 percent favored the safety of it. The 98 percent favorable response for safety given a favorable response for aesthetics was the highest obtained in the survey (also true for generous vertical curve length, Fig. 11).

Long Vertical Curves

Long vertical curves (Fig. 7) are necessary to reduce the abrupt appearance of the profile changes often associated with minor structures in flat terrain. Although minimum length curves are often used to save on fill, use of approach curves several times the minimum lengths does not greatly increase fill requirements and markedly improves alignment appearance. This was confirmed by the 76 percent aesthetic preference for such treatment; 87 percent found the long curve design safer in appearance. Of the respondents, 95 percent favored the treatment for safety when there was a favorable response for aesthetics.

Long vertical curves (Fig. 8) are necessary with sag curvature that occurs along a long horizontal curve to avoid the sunken or collapsed roadway appearance (evident in the upper sketch). Here again is demonstration of the amplification of visual distortion

Figure 5. Fill slope and guardrail.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	56	46	51
S	29	41	35
AS	22	26	24
S/A	39	56	46

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	55	54	109
S	55	54	109
AS	55	54	109

LEGEND

A Appraised for aesthetics

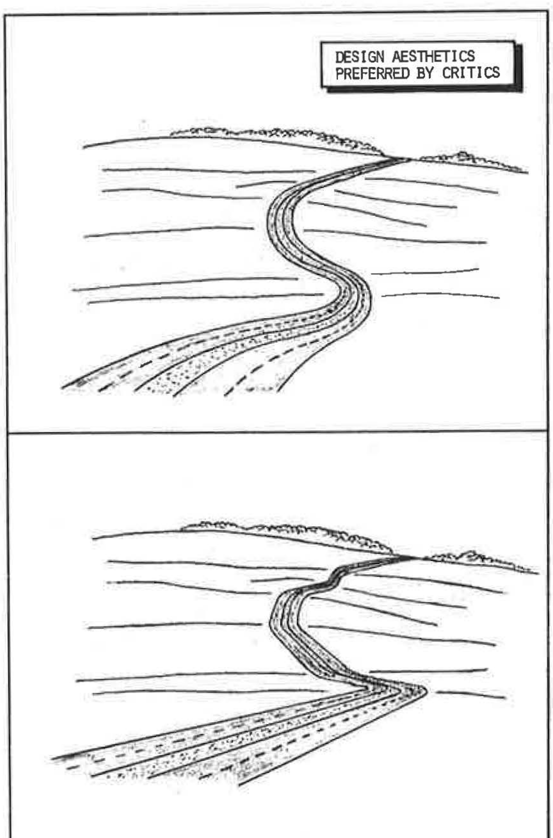
S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

○ Reject hypothesis that "ALL respondents prefer design preferred by aesthetic critics (at 0.01 level)

Figure 6. Flow of alignment.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	71	80	75
S	95	93	94
AS	70	78	74
S/A	98	98	98

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	55	54	109
S	54	54	108
AS	54	54	108

LEGEND

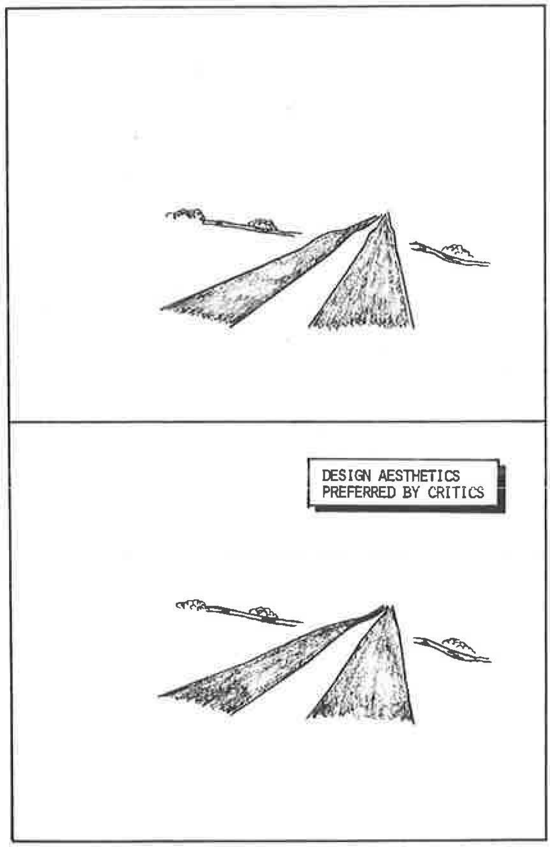
A Appraised for aesthetics

S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

Figure 7. Vertical curvature on hump.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	63	89	76
S	76	98	87
AS	61	87	74
S/A	91	98	95

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	54	53	107
S	54	53	107
AS	51	53	104

LEGEND

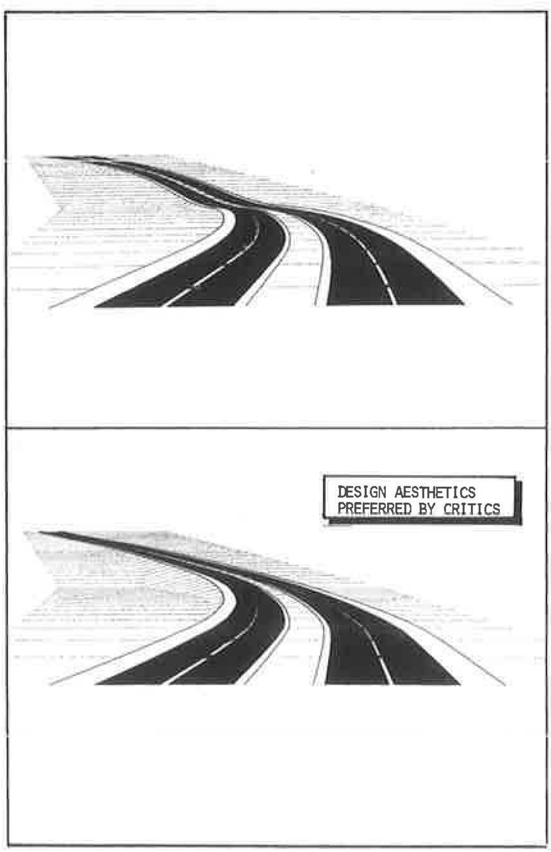
A Appraised for aesthetics

S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

Figure 8. Sag curve on long horizontal curve.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	66	74	70
S	80	85	82
AS	61	68	65
S/A	92	90	91

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	55	54	109
S	54	53	107
AS	54	53	107

LEGEND

A Appraised for aesthetics

S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

that occurs when minimum vertical curvature is superimposed on horizontal curvature (or vice versa). The minimum length sag curve, abrupt enough in appearance on tangent, becomes even more visually disturbing when imposed on horizontal curvature. The choice of the aesthetic design critics was considered more attractive by 70 percent of the respondents and safer by 82 percent of the respondents. Of those who preferred the long curve treatment for beauty, 91 percent also preferred that design for safety.

Abutment Wall Design

The alternative design treatments shown (Fig. 9) were in part inserted as a control test because the author is unaware of any strong preference among students of aesthetic highway design for either flared or straight abutment walls. The flared wingwall treatment shown in the upper sketch might possibly be considered as the preferred design because it creates a more open appearance to the underpass and hence possibly a higher level of perceived safety. This treatment was assumed to be preferred by aesthetics critics. In view of this, it is not surprising that only 53 percent of the respondents preferred the flared treatment for aesthetics, and only 63 percent preferred it for safety, with neither response rate significantly higher than 50 percent to support the central hypothesis of this study (although the safety preference by engineering students was found to be significantly higher than for secretaries). Nevertheless, 72 percent of those who preferred the flared treatment for beauty also preferred it for safety.

Varying Fill Slopes and Open Abutments

Varying fill slopes and open abutments (Fig. 10) are two preferred design features incorporated in the upper sketch of this figure. With a uniform catch line, the embankment slope becomes progressively flatter as the upper roadway approaches grade in the distance, which blends the fill with adjacent terrain and softens its man-made appearance. The open end-span treatment eliminates the massive wall effect evident in the lower sketch; gives the structure a lighter, more graceful appearance; and reduces the dark tunnel effect often created with closed abutments. However, only 59 percent of the respondents favored the attractiveness of the critics' recommended treatment (not significant), although the preference was greater from the safety standpoint—69 percent overall and 83 percent among those who preferred the aesthetics of the critics' choice.

Vertical Curve Length

Vertical curve length well above minimum is necessary to avoid the broken board effect that is typical of minimum-design sag curves. In the upper sketch, the kinked appearance of the 700-ft-long (213 m) vertical curve connecting a 2 percent downgrade to a 3 percent upgrade is heightened in the opposing roadway. In the lower sketch, a 3,000-ft (914 m) vertical curve is used, and this design was preferred by 78 percent of the respondents for attractiveness and 93 percent for perceived safety. Among those who preferred the critics' treatment for beauty, there was near unanimous preference for the appearance from the safety standpoint—98 percent.

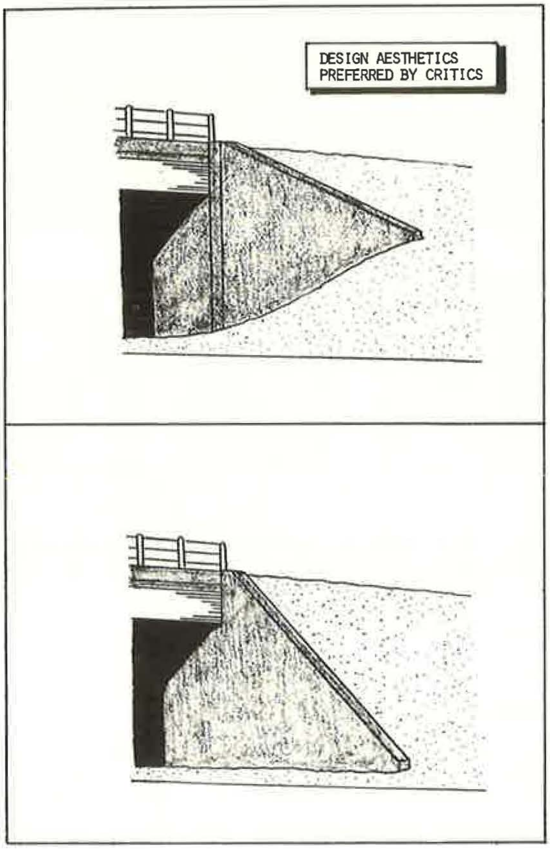
Spiral Transition Curve

Use of a spiral transition curve (Fig. 12) was preferred by 73 percent of the respondents in terms of attractiveness and by 89 percent in terms of safety. Of those, 96 percent preferred the aesthetics of the spiral design treatment and also favored it for safety. It is noteworthy that these results are nearly identical to those for the transition curve treatment (Fig. 4).

SUMMARY AND CONCLUSIONS

There are marked personal preferences for certain highway design treatments that incorporate the principles promulgated by students of aesthetic highway design,

Figure 9. Abutment wing wall 2.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	53	54	53
S	51	76	63
AS	33	44	39
S/A	62	83	72

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	55	54	109
S	55	54	109
AS	55	54	109

LEGEND

A Appraised for aesthetics

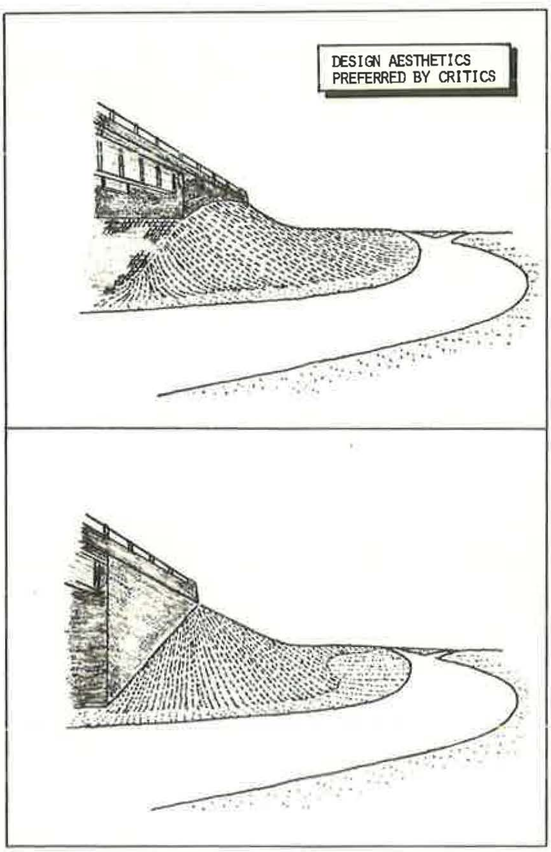
S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

○ Reject hypothesis that "ALL" respondents prefer design preferred by aesthetic critics (at 0.01 level)

Figure 10. Embankment slope and abutment.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	60	57	59
S	66	72	69
AS	46	52	49
S/A	76	90	83

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	55	54	109
S	55	54	109
AS	55	54	109

LEGEND

A Appraised for aesthetics

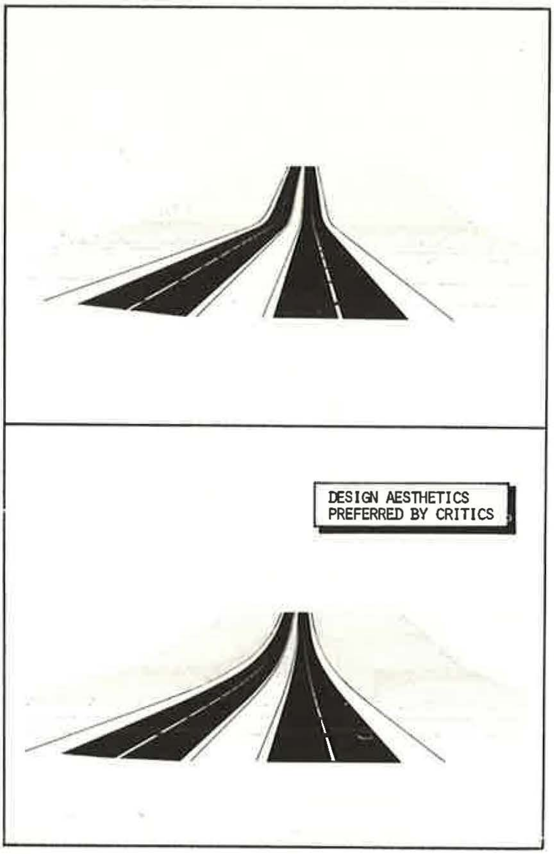
S Appraised for safety

AS Respondents favoring critics preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

○ Reject hypothesis that "ALL" respondents prefer design preferred by aesthetic critics (at 0.01 level)

Figure 11. Sag vertical curve.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	69	87	78
S	89	96	93
AS	67	85	76
S/A	97	98	98

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	54	53	107
S	54	53	107
AS	54	53	107

LEGEND

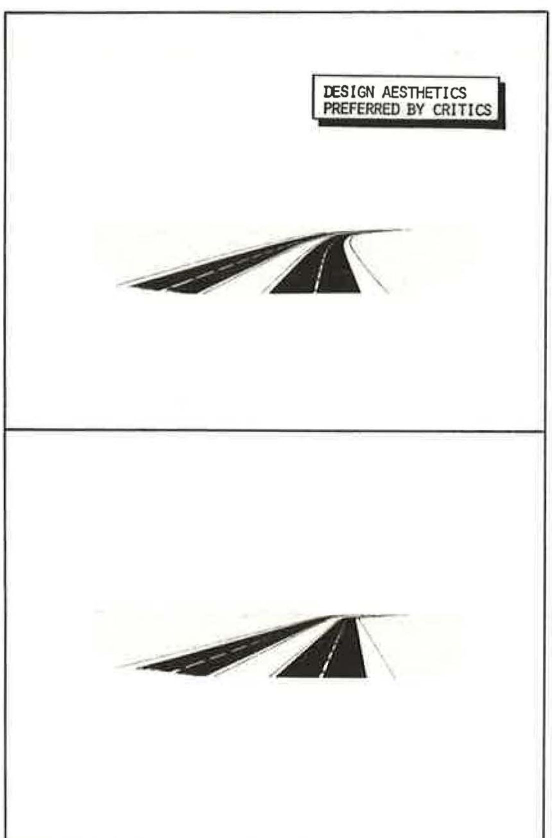
A Appraised for aesthetics

S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

Figure 12. Horizontal curve transition 2.



PERCENT FAVORING AESTHETIC DESIGN PREFERRED BY CRITICS

	SECR.	ENGRS.	ALL
A	76	71	73
S	85	92	89
AS	73	69	71
S/A	95	97	96

SAMPLE SIZE

	SECR.	ENGRS.	ALL
A	53	52	105
S	53	53	106
AS	52	52	104

LEGEND

A Appraised for aesthetics

S Appraised for safety

AS Respondents favoring critics' preference in both aesthetic and safety appraisals

S/A Favorable response for safety, given a favorable response for aesthetics

particularly with respect to horizontal and vertical alignment features. These treatments were viewed favorably, in terms of both aesthetic quality and perceived safety, by two sample populations that might well be taken as reasonably reflective of the range of preferences of the broader public. It should be noted, however, that only simple preferences were revealed in this study as opposed to measures of strength of preference that might have been obtained by asking respondents to rate the paired alternatives on a numerical scale.

Although this study treated design situations perhaps more commonly associated with rural highway locations, many of the findings of this study also have merit for urban highway design, although the more complex urban setting does impose greater constraints on the full exploitation of the aesthetic potential of urban route corridors.

ACKNOWLEDGMENT

The author is indebted to Christopher Tunnard, Boris Pushkarev, and Yale University Press for permission to use some figures that appeared in another publication (1) in this study.

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SAFETY ASPECTS OF ROADSIDE SLOPE COMBINATIONS

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ABRIDGMENT

•TO provide objective criteria for safe slope combinations and various ditch configurations is the specific objective of research efforts discussed in this paper.

The research approach involved simulated traversals of approximately 168 combinations of front and back slopes from 3:1 to 6:1. Some 24 full-scale vehicle tests were conducted to provide validation data for the highway-vehicle-object simulation model (HVOSM). All simulated and full-scale tests were conducted at 60 mph (96.5 km/h) and 25-deg (0.4 rad) encroachment angles; therefore, the recommended criteria are based on these operating conditions.

CRITERIA

Recommended design curves are shown for combinations of slopes forming vee, round, trapezoidal, and rounded trapezoidal ditch configurations with widths to 16 ft (4.9 m). A method of evaluating the resultant effect of vehicle accelerations in the longitudinal, lateral, and vertical axes was developed by assigning a severity index to the resultant acceleration and by relating this index to the degree of potential hazard as follows:

$$SI = \sqrt{\left(\frac{ALON}{G_{XL}}\right)^2 + \left(\frac{ALAT}{G_{YL}}\right)^2 + \left(\frac{AVER}{G_{ZL}}\right)^2} \tag{1}$$

where

SI = severity index;
ALON = acceleration experienced in longitudinal axis, g;
ALAT = acceleration experienced in lateral axis, g;
AVER = acceleration experienced in vertical axis, g;
G_{XL} = tolerable acceleration in longitudinal (X-axis) direction, g;
G_{YL} = tolerable acceleration in lateral (Y-axis) direction, g; and
G_{ZL} = tolerable acceleration in vertical (Z-axis) direction.

Substituting the unrestrained occupant values from Table 1 in Eq. 1 produces

$$SI = \sqrt{\left(\frac{ALON}{7}\right)^2 + \left(\frac{ALAT}{5}\right)^2 + \left(\frac{AVER}{6}\right)^2} \tag{2}$$

A severity index of 1.0 represents a resultant acceleration that may be safely tolerated by an unrestrained occupant. A severity index of 1.6 represents the upper limit of acceleration considered safe for seat belt restraint.

APPLICATION

Desirably, slope combinations would be selected so that unrestrained occupants could be expected to sustain no injury and the vehicle would not incur major damage during traversal. However, site conditions such as restricted right-of-way or other factors beyond the designer's control may dictate the use of slope combinations steeper than desirable. Therefore, design curves are shown for both conditions in Figures 1

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Table 1. Tolerable acceleration limits established for ditch traversal study (tentative).

Restraint Condition	Maximum Acceleration (g) Lateral (G_{YL})	Longitudinal (G_{XL})	Vertical (G_{ZL})
Unrestrained occupant	5	7	6
Lap belt restraint	9	12	10
Lap belt and shoulder harness	15	20	17

Figure 1. Tentative design recommendations for vee ditch; round ditch, width <8 ft (2.4 m); trapezoidal ditch, width <4 ft (1.2 m); and rounded trapezoidal ditch, width <4 ft (1.2 m).

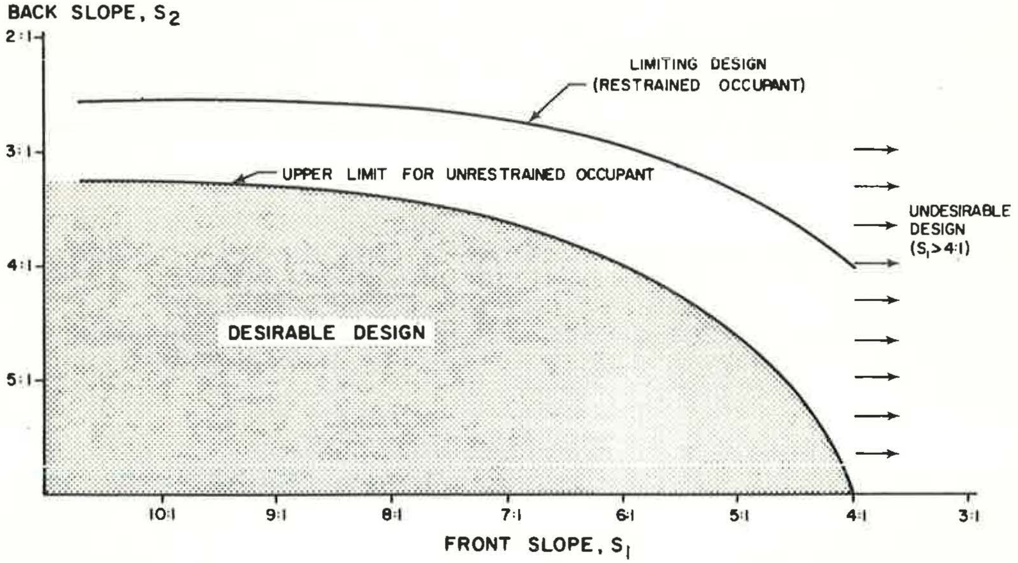


Figure 2. Tentative design recommendations for round ditch, width 8 to 12 ft (2.4 to 3.7 m) and trapezoidal ditch, width 4 to 8 ft (1.2 to 2.4 m).

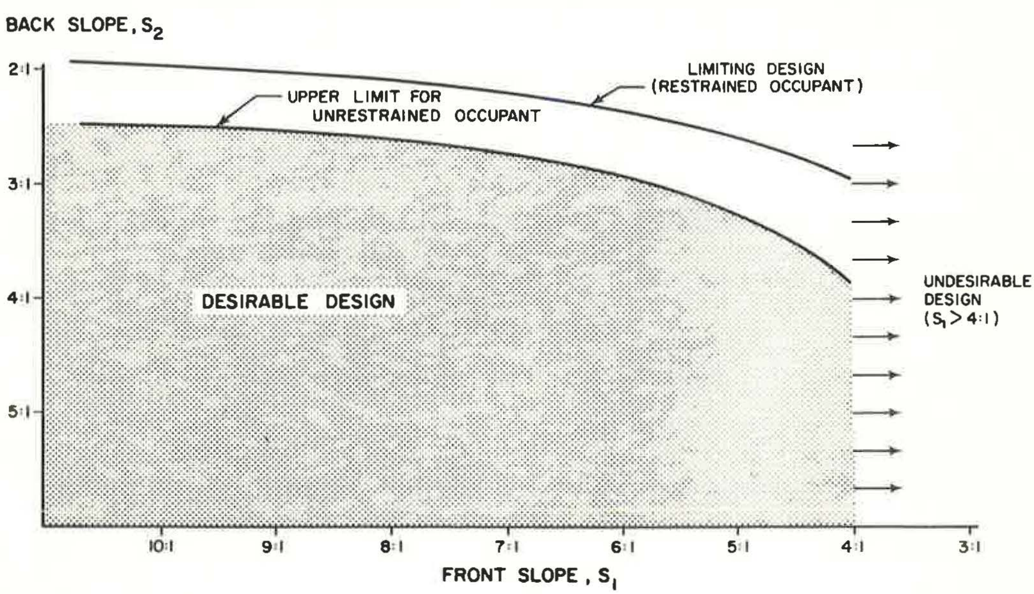
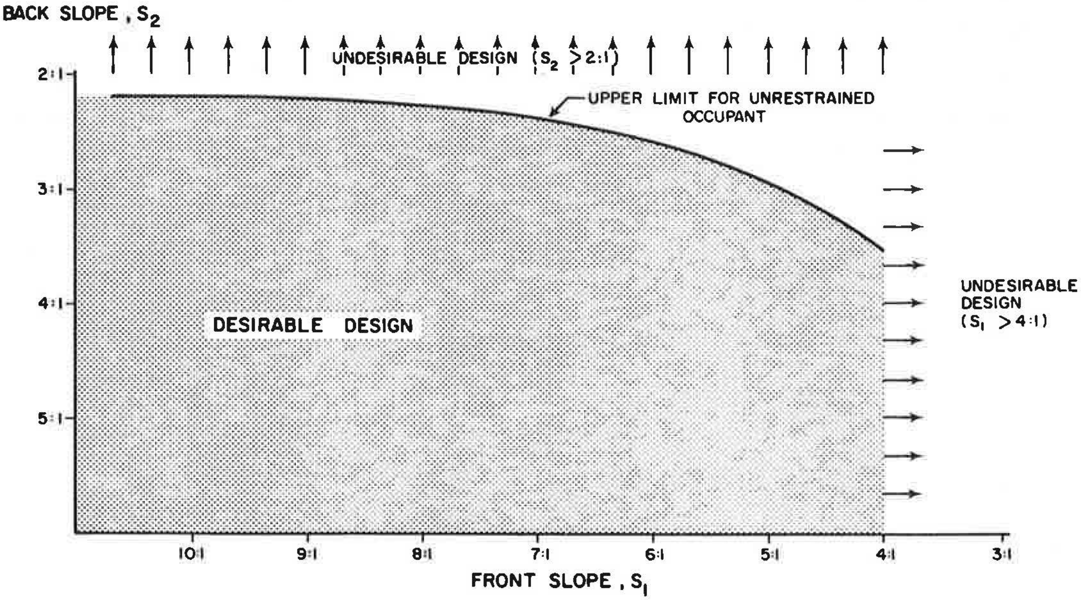


Figure 3. Tentative design recommendations for round ditch, width > 12 ft (3.7 m); trapezoidal ditch, width > 8 ft (2.4 m); and rounded trapezoidal ditch, width > 4 ft (1.2 m).



and 2 for various ditch configurations. Only the desirable design curve is shown in Figure 3 because the limiting design curve would produce slopes of 2:1 or steeper. Earth slopes steeper than 2:1 are difficult to construct and maintain and are therefore not considered practical. The desirable design curve is based on a severity index of 1.0 and a bumper penetration of 4 to 4.5 in. (10.2 to 11.5 cm), whereas the limiting design curve is based on a severity index of 1.6 and bumper penetration of 6.0 in. (15.2 cm).

These curves provide the design engineer with objective criteria for selection of traversable slope combinations and ditch shapes under 60 mph, 25-deg (96.5 km/h, 0.4 rad) encroachment conditions such as might be encountered on high-speed facilities. The design curves are applicable for a ditch location up to 60 ft (18.3 m) from the edge of the roadway.

DISCUSSION OF RESULTS

The vee ditch generally produced g forces that were less severe than those caused by traversing the round or trapezoidal ditches that had widths of 8 ft (2.4 m) or less or traversing the rounded trapezoidal ditch in the 4- to 8-ft (1.2 to 2.4 m) range.

Round ditches generally produced g forces that were more severe than the other three configurations for comparable slope combinations, particularly for steep slope combinations and narrow ditch widths. Little difference in severity can be expected between the shaped ditches with widths in the 16-ft (4.9 m) range.

The trapezoidal ditch configuration offers a cross section that is safer to cross than the others at high speeds, particularly for the steeper slope combinations. The g forces, in general, were lower than those of the vee or rounded ditch. Little safety benefit was realized by rounding the basic trapezoidal cross section to produce the rounded trapezoidal ditch.

ACKNOWLEDGMENT

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The contents of this paper reflect the views of the authors who are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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ANALYTICAL AND EXPERIMENTAL EVALUATION OF MODULUS OF SOIL REACTION

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The Iowa deflection formula has been the accepted design equation for flexible culverts for over 30 years. Unfortunately, design based on this formula is very sensitive to the modulus of soil reaction (E′). Because direct full-scale test measurements of this parameter have been scarce, researchers have attempted to supplement the data by several methods: finite element models and other elastic analyses, correlations with laboratory tests, use of test cells, rearrangement of the Iowa deflection formula, and monitoring of quantities simpler to measure (often with requisite assumptions). This paper investigates the drawbacks of some of these approaches.

•SINCE its introduction over 30 years ago, the Iowa deflection formula has served as a basic criterion for the design of buried flexible pipe. The formula is based on the assumption that the supporting strength of corrugated metal pipe installations arises through the lateral reactive soil pressures induced at the sides of the pipe. This behavior is characterized in the formula by the parameter E′, which is the modulus of soil reaction.

This parameter has a very significant effect on the evaluation of pipe deflections, and, therefore, realistic values should be assigned to it. It is important to note that E′ is a derived parameter and is not related to any fundamental soil properties. Its appearance in the Iowa deflection formula can be viewed as exclusive, because it is not found in any other type of reference or literature on soil mechanics. The basic definition of E′ as presented by Spangler (4, 7) is

$$E' = \frac{h}{\Delta} D \qquad (1)$$

where h = the maximum pressure at the springline, D = the pipe diameter, and Δ = the horizontal deflection of the pipe.

Because E′ is an implied property of the soil, it must be evaluated on the basis of experimental evidence. In 1971, Spangler stated that he was aware of only 18 full-scale field installations of flexible pipe in which the proper types of data had been recorded to permit a correct evaluation of E′. A detailed study of these data has been made elsewhere (3). In this study, no strong correlation could be found between E′ and the usual parameters of soil-culvert systems. The computed median of E′ was 715 psi (4930 kPa). Therefore, until sufficient field data are made available and analyzed statistically, extreme caution should be exercised in assigning values to E′

TEST CELL EVALUATION

The results of an alternative method for evaluating E′ from field data are shown in Figure 1 (5, p. 58). These values of E′ range from a maximum of approximately 11,000 psi (75 900 kPa) to a minimum of -500 psi (-3450 kPa). These values were

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Figure 1. E′ as a function of soil density.

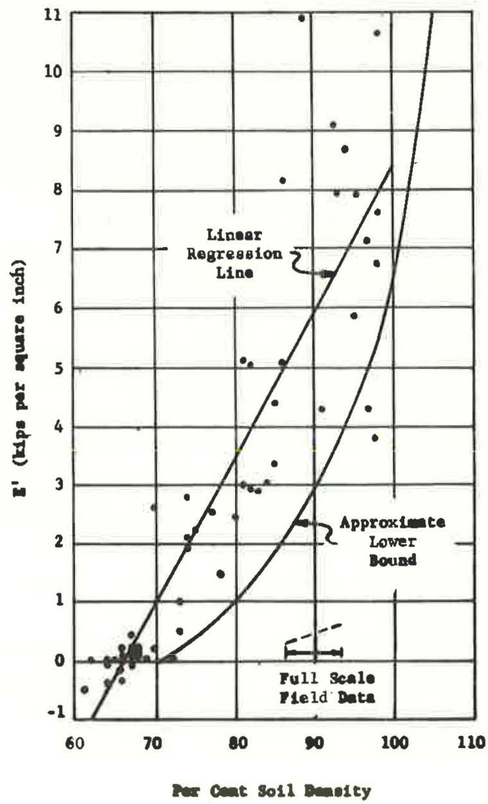
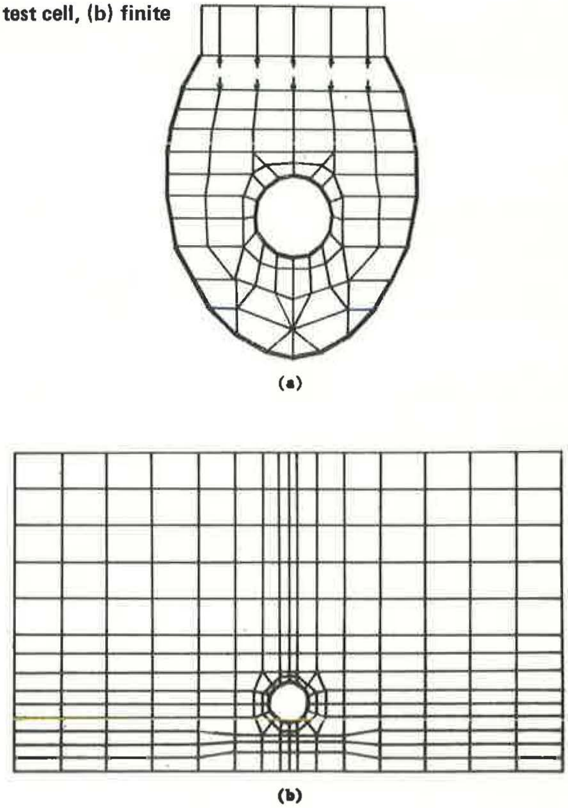


Figure 2. (a) Finite element model of test cell, (b) finite element model of embankment.



obtained from a study of full-scale pipe sections buried in soil in a large test cell. The test cell was designed to simulate the effect of an embankment loading on a pipe section (8); however, the expression used to evaluate the quantities shown in Figure 1 is not described in that report. The test cell consisted of a concrete-supported steel plate frame, which held the test pipe and surrounding soil. I-beams across the top carried a total of 50 vertical hydraulic cylinders or rams. Figure 1 also shows the linear regression lines fit through the test cell data ($E' = -16.2 + 0.246$ SD) and the full-scale field installations ($E' = -3.8 + 0.047$ PPD) reported in a previous study (3). To reflect the difference between soil density as a percent of AASHTO maximum (SD) and percent standard Proctor density (PPD) requires that the latter data be shifted to the left by something less than about 10 percent depending on the type of soil. The field data are still significantly less than the test cell data.

So that the validity of the proposed similitude between the test cell and the embankment could be checked, a finite element mathematical model was formulated for each soil-culvert system (Fig. 2). It was possible, with the aid of the digital computer, to make a comparison of the pipe behavior in the two models for a wide range of pipe diameters [36 to 60 in. (1 to 1.5 m)], soil conditions, corrugation size and configuration, and fill heights H [50 to 150 ft (15 to 45 m)]. The effective fill heights for the test rig were obtained by varying the hydraulic pressure in the rams to match the pressure that would exist because of the soil overburden. The output of the computer program included for a linear elastic analysis the pipe's vertical deflection (Δ_v), the horizontal deflection (Δ_h), and the normal stresses at the soil-pipe interface for the crown (σ_v) and springline (σ_h) of the pipe. With these data, it was possible to evaluate three different values of the modulus of soil reaction for each system:

1. The true E′ as defined by Eq. 1

$$E' = \frac{\sigma_h}{\Delta_h} D \qquad (2)$$

2. A modulus value based on similar data at the crown of the pipe

$$E_v' = \frac{\sigma_v}{\Delta_v} D \qquad (3)$$

3. A back-calculated value obtained by a rearrangement of the Iowa deflection formula, the use of the vertical deflection, and the assumptions that the bedding factor $K = 0.1$ and that W_c for the embankment condition is equal to the total weight of the soil above the pipe, i.e.,

$$E_v^* = 16.39\left[\frac{0.1\, H\, \gamma\, D}{\Delta_v} - \frac{EI}{R^3}\right] \qquad (4)$$

where γ is the density of the backfill material and R and EI are the radius and flexural stiffness of the corrugated pipe respectively.

The results of this study revealed that, for the case of the same backfill material and pipe configuration, the ratios of vertical to horizontal stresses and deflections are independent of fill height; however, the ratios are different for each model. Ratios for a backfill [height: 50 to 150 ft (15 to 45 m)] with an elastic soil modulus $E_s = 5{,}000$ psi (3450 kPa) and a 60-in.-diameter (1.5 m) pipe of 18 gauge with $2\frac{2}{3}$-in. × $\frac{1}{2}$-in. corrugations were

Ratio	Embankment Model	Test Cell Model
σ_v/σ_h	0.31	0.84
Δ_v/Δ_h	1.38	1.55

For both the embankment model and the test cell model, the stress ratios increased slightly and the deflection ratios decreased somewhat with a lower soil modulus [from 2,500 to 7,500 psi (17 000 to 52 000 kPa)] or stiffer pipe wall (a fourfold variation in EI). These changes were 10 percent for the stresses and 15 percent for the deflections over the range of variables indicated for both the embankment and test cell models. To check for size effects with the test cell model, a 36-in.-diameter (1 m) pipe of 16 gauge with 2⅔-in. × ½-in. corrugations was used. The overall stiffness factor (EI/R^3) of this pipe was within the range of the 60-in. (1.5 m) pipe. With a soil modulus E_s = 5,000 psi (34 500 kPa) and 50 ft (15 m) of fill above the pipe, a stress ratio of 2.12 and a deflection ratio of 1.13 were obtained.

The vertical stress, horizontal stress, vertical deflection, and horizontal deflection for the embankment model were all only slightly sensitive to fill height, but for the test cell model the increase was almost directly proportional to the fill height. Thus, it would appear that the use of the hydraulic rams to simulate fill height requires further study. The phenomenon of arching may not be properly modeled by the test cell.

For each case in which a 60-in. (1.5 m) pipe was used, the value of E′ (as defined by Eq. 2) for the embankment model was equal to the value of E′ obtained from the test cell model. For the 36-in. (1 m) pipe however, the value of E′ from the test cell model was substantially less than the value from the embankment model. For the 60-in. (1.5 m) pipe and all fill heights the value of E_v' (Eq. 3) for the test cell model was approximately 2.4 times the value of E_v' from the embankment model data. This ratio increased slightly with an increase in soil modulus. For the 36-in. (1 m) pipe, this ratio was approximately 1.5.

For the 60-in. (1.5 m) pipe over the range of fill heights and soil moduli, it was found that

$$E_v^* = C_1 E' \tag{5}$$

where $C_1 \approx 1.3$ for the test cell model, and C_1 varies as a function of the height of fill for the embankment model. The values of C_1 were substantially higher for the smaller pipe.

An alternate form of E′ can be defined as

$$E_v'' = \frac{P_v}{\Delta_v} D \tag{6}$$

where P_v is the vertical pressure that would be present if no pipe were in place. Thus, $P_v = H\gamma$ in the embankment case. For the 60-in. (1.5 m) pipe over the range of fill heights and soil moduli it was found that

$$E_v'' = C_2 E' \tag{7}$$

where $C_2 \approx 0.8$ for the test cell model, and C_2 varies as a function of the height of fill for the embankment model. The values of C_2 were substantially higher for the smaller pipe.

Thus, the assumption that E′ values from test cell data are a faithful representation of the true value of E′ for an embankment installation appears to be justified if, and only if, the true definition of E′ (Eq. 2) is used. Because of the elastic analysis limitation of the finite element model, additional data from actual full-scale installations are needed before any of the methods discussed are justified or rejected.

EFFECT OF PARAMETER VARIATION ON E′

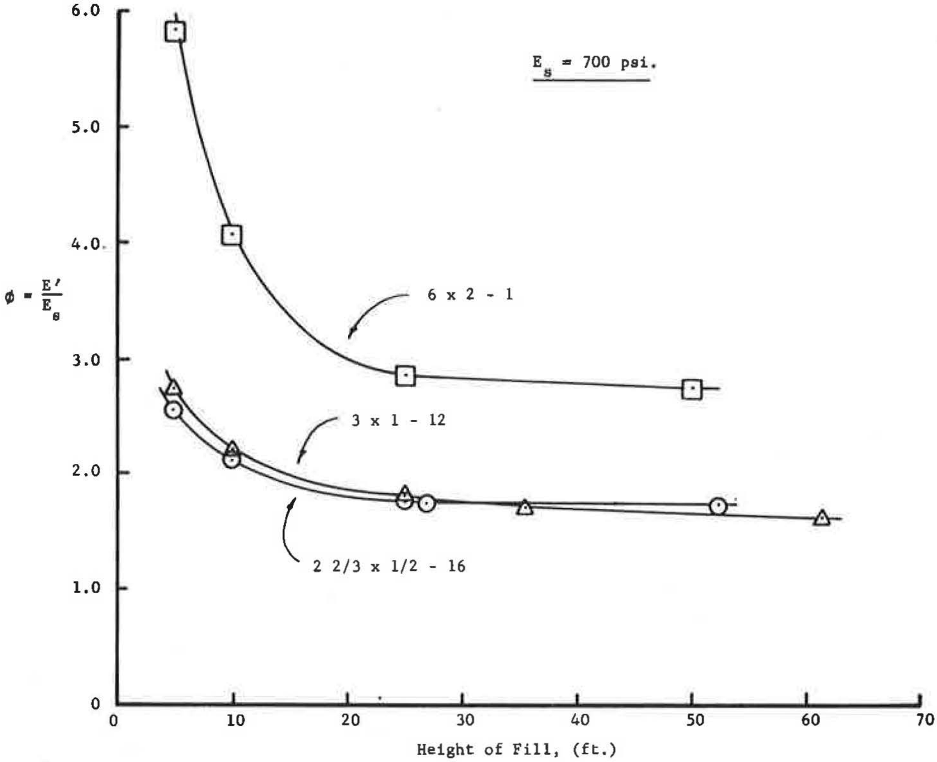
The finite element mathematical model of the embankment soil-culvert system was also used to study the behavior of a variety of different pipes in different soils and to evaluate the true value of E′ from Eq. 2. For the purposes of this study, the normal range of steel corrugated pipe (9) is considered (Table 1). The following corrugations were used:

Table 1. Geometric properties of corrugated steel pipe.

	Corrugation Configuration					
	$2^2/_3$ in. × $^1/_2$ in.		3 in. × 1 in.		6 in. × 2 in.	
Gauge	I (in.^{4}/in.)	ρ = A/I	I (in.^{4}/in.)	ρ = A/I	I (in.^{4}/in.)	ρ = A/I
16	0.00189	34.14	0.00866	8.57		
14	0.00239	33.78	0.01083	8.56		
12	0.00342	32.99	0.01546	8.41	0.0604	2.15
10	0.00453	32.07	0.02018	8.30	0.0782	2.13
8	0.00572	31.05	0.02508	8.18	0.0962	2.12
7					0.1078	2.11
5					0.1269	2.10
3					0.1462	2.08
1					0.1658	2.07

Note: 1 in. = 2.54 cm.

Figure 3. Variation of E′ with pipe stiffness and height of fill.



Corrugation Configuration (in.)	Gauge	Relative I
$2\frac{2}{3} \times \frac{1}{2}$	16	1.0
3×1	12	8.17
3×1	8	13.26
6×2	1	87.65

The diameter of the pipe was held constant at 5 ft (1.5 m). The unit weight of the soil was also held constant at 120 pcf (1920 kg/m^3); however, two values (6, 9) of elastic modulus for the soil were considered: E_s = 700 psi (4830 kPa) and E_s = 2,800 psi (19 300 kPa).

Figure 3 shows the results for the analysis of three different pipe configurations embedded in a soil that has a modulus of 700 psi (4830 kPa). The data are plotted as height of fill versus the nondimensional ratio, ϕ, which is defined as the ratio of E′ (from Eq. 2 and the results of the simulation) to the modulus of soil, E_s. If E′ were indeed a constant, these data should plot as horizontal straight lines and all of these lines would be coincident. As seen in Figure 3, a different curved line is obtained for each pipe stiffness. In addition it should be noted that two of the lines cross at a fill height of approximately 30 ft (9 m). One must conclude that E′ is not an inherent property of the elastic soil and that it appears to depend on the stiffness of the pipe.

Figure 4 shows the data in the form of the deflection ratio (i.e., Δ/D) versus height of fill. The soil modulus was again assumed to be constant and equal to 700 psi (4830 kPa). For purposes of comparison, the theoretical height of fill based on the elastic deflection as calculated by the Iowa deflection formula is also indicated on the diagram. These fill heights are for the condition that the deflection lag factor D_l = 1 and E′ = 700 and 1,400 psi (4830 and 9660 kPa). The deflection criterion was that the ratio Δ/D be equal to 0.050. Deflections from the finite element analysis are less than 0.020 for the theoretical design fill heights as evaluated by the Iowa formula, which used E′ equal to 700 psi (4830 kPa). The theoretical fill heights determined by the criterion of E′ equal to 1,400 psi (9660 kPa) yield displacements of less than 0.30 as determined from computer simulation. Thus, it appears that there is a lack of consistency between the finite element analysis for deflection and the results obtained by using the Iowa deflection formula.

Figure 5 shows the effect of a change in the value of E_s. The solid line indicates the results for E_s = 700 psi (4830 kPa) and the dashed line the results for E_s = 2,800 psi (19 300 kPa). A variation can be noted in which the relative location of the dashed and solid curves is changed and modified for different pipe configurations.

From the graphical displays of the results (Figs. 3 through 5), one can see that E′ is definitely not a function of the elastic properties of the soil, the fill height, or the geometry of the pipe system. Thus, it can be concluded that E′ is an empirical parameter and not a constant.

FIELD MEASUREMENT PROGRAM

In 1971, the Federal Highway Administration conducted a survey of 20 flexible pipe installations under high fills (2). Twenty-two installations, of which 20 had been in place for periods of from 10 to 20 years, were inspected. Both horizontal and vertical diameters were measured, generally at three separate locations along the pipeline, and deflection was calculated by subtracting the measured vertical diameter from the vertical diameter at time of installation. It was then possible to evaluate an estimate of E′ by means of a rearranged Iowa deflection formula, such as Eq. 4 (however, the deflection lag factor was assumed to be 1.25 rather than 1.0 as was used in Eq. 4). Because of the variability of construction techniques, it is highly questionable whether the value of E′ obtained by this method is even a reliable estimate of E_v^*.

It is convenient to start with a histogram of the results (Fig. 6) to analyze the E′ data as reported in the Colorado study. The observed mean is approximately 2,400 psi (16 500 kPa), and the observed median is approximately 2,000 psi (13 800 kPa). The skewness of the histogram is clear, and this makes the median the appropriate central

Figure 4. Variation of deflection with pipe stiffness and height of fill.

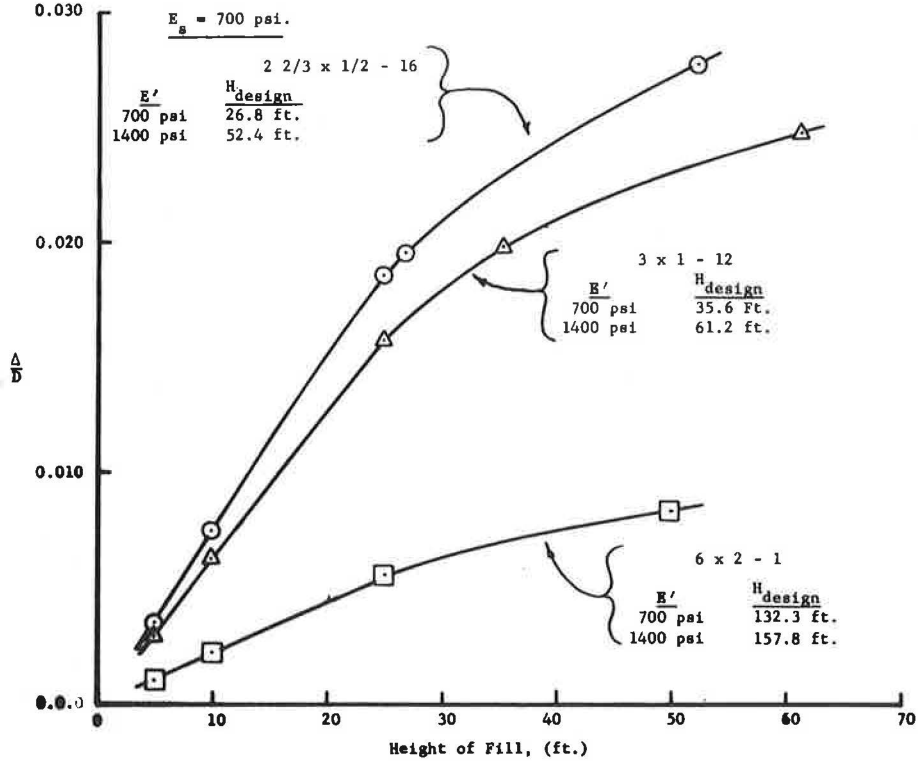


Figure 5. Effect of soil modulus and pipe stiffness on E'.

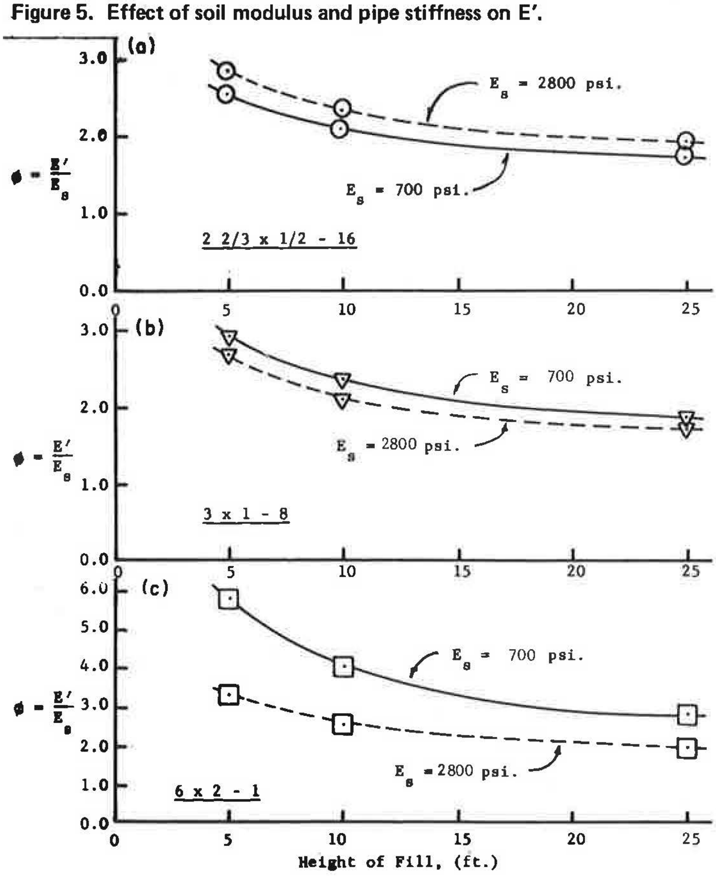


Figure 6. Histogram of observed values of E′.

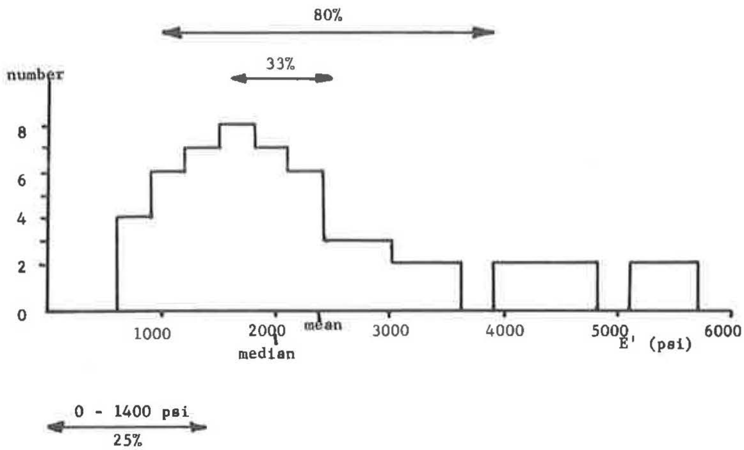
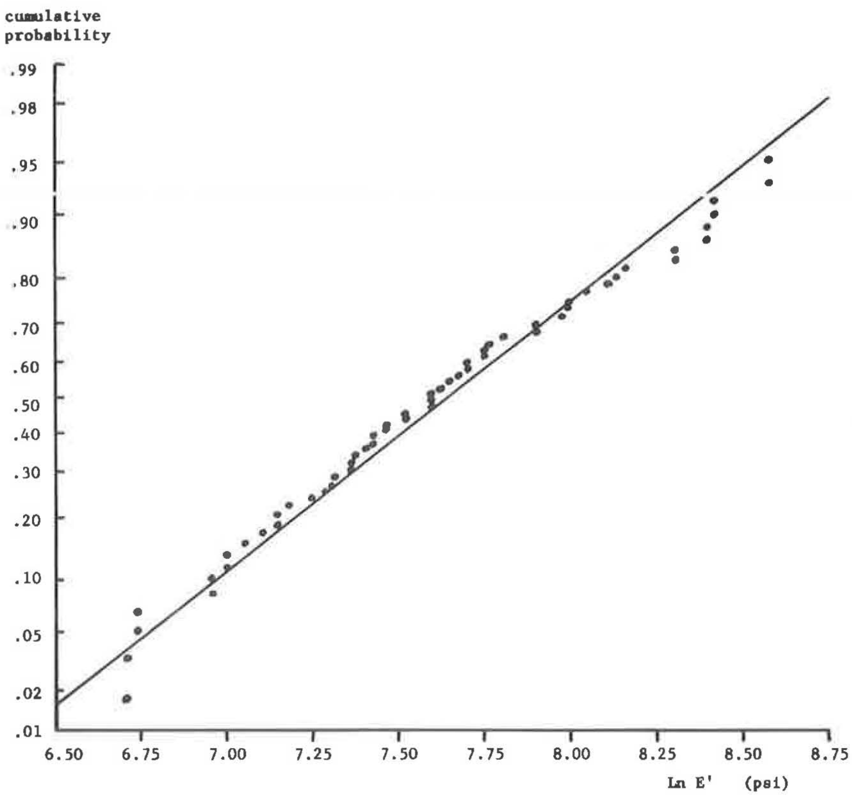


Figure 7. Fitted lognormal and observed values of E′.



value for computing probabilities. The median is the value that is exceeded 50 percent of the time. The histogram indicates a lognormal distribution for E′. A lognormal function was fitted to the observed parameters of E′, and these cumulative data (Fig. 7) and the cumulative data from the Colorado report are shown. The lognormal distribution gives excellent fit (Fig. 7). The data pass both the chi-square test and Kolmogorov-Smirnov test at all usual hypothesis test levels. The following probability statements that used the fitted lognormal distribution are thus meaningful estimates. Figure 6 shows various probability ranges, and, 50 percent of the time, a value of E′ below 2,000 psi (13 800 kPa) may be expected. Of the time values, 33 percent from 1,600 to 2,500 (11 000 to 17 250 kPa) may be expected, and this means 33 percent of the time values below 1,600 psi (11 000 kPa) may occur. Of the time values, 80 percent from 1,000 to 3,900 psi (6900 to 26 900 kPa) may be expected with values below 1,000 psi (6900 kPa), and this occurs 10 percent of the time. The lower quartile point is 1,400 psi (9700 kPa), and there is one chance in four of encountering a value below this.

The assumed load on the pipe in the Colorado report is H γ D, where H is height of overburden, γ is density of soil, and D is the pipe diameter. A previous study (3) of all data that could be located from full-scale field-type installations in which proper data were measured showed that, on the average, the actual load is less than H γ D. For those installations,

$$W_{true} = \frac{H \gamma D}{1.38} \tag{8}$$

If E′ values are computed for those installations, on the average

$$E_{true} = \frac{E'_{approx}}{1.58} \tag{9}$$

Therefore, to get a more accurate estimate of E′ when H γ D is used for vertical load requires that E'_{approx} be reduced. If the reduction is applied, for instance, to the median, it becomes 1,266 psi (8700 kPa) rather than 2,000 psi (13 800 kPa).

The use of vertical deflection rather than theoretically correct horizontal deflection tends to produce a somewhat smaller value of E′ (3). When horizontal deflection is used, the median of E′ would increase, on the average, from 1,266 to 1,450 psi (8700 to 10 000 kPa).

CONCLUSIONS

This mathematical analysis is based on a linear elastic finite element model. Although many useful and interesting results may be obtained from an elastic analysis (1), the fact remains that the soil-structure interaction is a nonlinear phenomenon. The exact solution to the problem is only obtained through actual full-scale field installations. Within this limitation, however, there are inherent similitude problems with the use of model studies or devices for simulating the full-scale situation, and any such investigations must also involve data from full-scale installations to establish the reliability of the modeling techniques. It has also been shown, through parameter variation in the finite element model, that E′ is apparently not an inherent soil property.

The Iowa deflection formula is exactly what the name implies, a formula for calculating the deflection of a flexible conduit. If additional data are required to establish a more statistically meaningful data base for the values of the coefficients appearing in the formula, then such studies should be conducted within the context of the basic definition of each coefficient. A simple juxtaposition of the terms or an application of the Iowa deflection formula for purposes other than it was originally intended can lead to erroneous data and questionable conclusions.

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FIELD TEST DEFLECTIONS OF REINFORCED PLASTIC MORTAR PIPE

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Two test sections of 30-in.-diameter (76 cm) reinforced plastic mortar (RPM) pipe were installed to replace two deteriorated open irrigation laterals. Twenty-nine 20-ft (6.1 m) sections of pipe were placed in one section in a dry trench with five different types of bedding. Pipe deflections were measured at four different times during the 16 months after installation. Results showed the following average initial vertical pipe deflections: (a) well-compacted material, 1 percent; (b) puddled natural earth, 4 percent; (c) loose sand, 6 percent; and (d) loose natural earth, 8 percent. Thirty 20-ft (6.1 m) pipe lengths were placed in the other section in a wet and unstable subgrade. Deflections of the bell end, spigot end, and center of the pipe were measured at 2, 5, and 12 months after installation. Results showed that the average initial vertical deflections were 0.7 percent at the spigot end, 1 percent at the bell end, and 2.3 percent at the center of the pipe. These field tests indicate that RPM pipe will deflect less than 5 percent if it is installed according to U.S. Bureau of Reclamation specifications.

•THE U.S. Bureau of Reclamation (USBR) is now emphasizing the use of closed conduits on its water distribution systems; this includes replacing deteriorated existing canals with pipelines. Pipelines reduce water losses, maintenance costs, and hazards to people and animals and permit the use of land over the conduit.

The Open and Closed Conduit Systems (OCCS) Committee of USBR evaluates different types of pipe and construction methods in an effort to reduce the high initial costs of installing pipelines. In 1967, USBR began investigating reinforced plastic mortar (RPM) pipe. Laboratory tests have indicated that RPM pipe could be an economical alternative to the types of pipe allowed under USBR specifications. In 1970, the OCCS Committee funded a special test program on the Yuma Project to evaluate the field performance of RPM pipe.

DESCRIPTION OF PIPE AND TEST SITES

Reinforced Plastic Mortar Pipe

RPM pipe is a composite of polyester resin, silicate sand, and glass filament reinforcing. The pipe is built up in layers on a mandrel by a filament winding process modified to incorporate the sand into the process. The result is a pipe that is flexible and lightweight and that provides high tensile hoop strength. RPM pipe also is resistant to a wide variety of chemical solutions. The pipe is manufactured in standard 20-ft (6.1 m) lengths with bell-and-spigot, rubber-gasketed (O-ring) joints. The bell is fabricated as an integral part of the pipe on the mandrel during the winding process. The spigot is cast or molded on the outside of the pipe wall at the end of the pipe.

Results of cooperative laboratory studies on RPM pipe by private industry and the federal government have been published in two USBR laboratory reports (1 and 2). Another USBR laboratory report (3) compares the structural behavior of RPM pipe

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buried in a special laboratory test container with that of steel pipe and is one of a series of reports on flexible pipe behavior (4, 5, 6, 7, 8). This paper covers the field performance of 30-in. (76 cm) RPM pipe used in the rehabilitation of two open laterals on the Yuma Project and is based on two USBR reports (9 and 10).

Yuma Project, Reservation Division

The Reservation Division of the Yuma Project is one of the oldest reclamation developments on the Colorado River. Construction was started in 1905 and completed in 1909, and water was delivered from 1910 on. The canals and laterals have degraded significantly; seepage losses are high and unstable banks are common. As a result, frequent breaks in canal and lateral banks disrupt water service to the users.

The water for the project was originally diverted from Laguna Dam, but since 1948 water for the Reservation Division has come directly from the All-American Canal or the Yuma Main Canal that heads at the All-American Canal. With the water supply coming from this source, any rehabilitation of the project could be accomplished with a closed-conduit, full-pressure system. The normal high water surface of the All-American Canal at the turnouts to the Reservation Division is 30 to 40 ft (9 to 12 m) above the ground surface elevation of the farm areas.

The soils in the Reservation Division are mostly sandy silts with a widely varying groundwater table. RPM pipe was installed under both high and low groundwater conditions with no attempt to dewater for the high groundwater condition. Both natural and imported materials were used for the pipe bedding.

On the Toronto Lateral, the pipe was installed, in a dry trench with five different beddings, to examine the effects of various beddings on the pipe structural behavior. On the Apache Lateral, the pipe was laid below the water table to assess the problems of installing the pipe below water. In addition, on the Apache Lateral the deflections of the bell-and-spigot ends of the pipe were measured and compared to the deflections in the center of the pipe sections.

Toronto Lateral

Twenty-nine 20-ft (6.1 m) sections of 30-in.-diameter (76 cm) RPM pipe were installed on the Toronto Lateral on a dry subgrade in October 1970. The bell-and-spigot joint pipes were rated for 100-psi (689 kPa) internal pressure. The line included the necessary mitered bends and connections to existing structures.

The trench for the pipe had a 5-ft (1.5 m) bottom width with $\frac{3}{4}$:1 side slopes. The subgrade was approximately 9 ft (2.7 m) below the top of the lateral embankment and 5 ft (1.5 m) below the bottom of the old lateral. The native material was generally a sandy silt with some areas of clay. The in-place moistures and densities of the trench walls were determined in seven locations. The gradation, consistency limits, specific gravity, maximum density, and optimum water content were determined in the laboratory.

The pipe was laid downstream with the spigot in the downstream direction and was joined by hand leveling. Internal vertical and horizontal diameter measurements were made at selected locations in each pipe section. After placement, the pipe was filled with water to determine if it was watertight so that it could be kept on grade during the placing of the bedding. The pipe was watertight except for a small leak at the concrete mortar pipe closure that formed the mitered upstream bend.

There were five types of bedding used on this lateral:

1. 60 lin ft (18.3 m) of compacted sand, saturated and vibrated;
2. 240 lin ft (73.2 m) of loose sand;
3. 40 lin ft (12.2 m) of compacted natural earth, tamped;
4. 180 lin ft (54.9 m) of loose natural earth; and
5. 60 lin ft (18.3 m) of puddled natural earth.

In accordance with USBR specifications, the bedding material was placed to a depth of 0.7 times the outside diameter of the pipe. Before backfilling was put over the pipe, in-place density tests were made on each side of the pipe in the compacted natural

earth and in the compacted sand bedding. For the puddled natural earth and the loose natural earth, density tests were made after the backfill had been placed and the beddings had settled. These densities represent the material density after the initial pipe deflection and the change in density caused by the surcharge of the overlying backfill.

The initial pipe deflection values were made about 2 weeks after construction, and subsequent deflections were measured in March 1971 (at 4 months), August 1971 (at 10 months), and March 1972 (at 16 months).

The compacted sand bedding was placed by dumping in the imported sand, flooding the sand with water, and then vibrating this with concrete vibrators as shown in Figure 1. The loose sand bedding was prepared by dumping the sand in and then flooding the sand to settle it with no vibration. The compacted natural earth bedding was constructed by placing the soil in loose layers and compacting each layer with pneumatic tampers. The loose earth bedding was placed by dumping the earth into place and then flooding it to water-settle the material. The puddled natural earth bedding was constructed by flooding the trench with water, dumping the soil, and settling it by working it with shovels (Fig. 2). The backfill over the pipe was pushed into place with a dozer and then water-settled.

Apache Lateral

Thirty 20-ft (6.1 m) sections of 30-in. (76 cm) diameter RPM pipe were installed in the Apache Lateral in March 1971 on an unstable subgrade, 9 to 18 in. (23 to 46 cm) below the water table. The pipe had bell-and-spigot joints and was rated for 100-psi (689 kPa) internal pressure. Included in this line were a horticultural turnout and the necessary vertical bends and connections with existing structures.

The trench for the pipe had an approximate 6-ft (1.8 m) bottom width with $\frac{3}{4}$:1 side slopes. It was difficult to excavate the trench to grade because it was below the water table, and the sandy silt sluffed into the trench. The subgrade was approximately 10 ft (3 m) below the top of the lateral embankment. The trench was overexcavated and the pipe was immediately lowered into the trench and joined so that the pipe could be put on grade. The pipe was then brought to grade by filling beneath it, maneuvering it to grade, and weighting it with some backfill to prevent floating. No attempt was made to dewater the trench. The pipe was laid downstream with the bell end downstream. The pipe was joined easily in spite of the water and soil that covered the lower portion of the bell. Gaskets were checked as conditions permitted, but a watertight joint could not be ensured.

Internal vertical and horizontal diameter measurements were made at the bell and the spigot before the pipe was placed. Initial deflection measurements were made in May 1971 (at 2 months) and subsequently in August 1971 (at 5 months) and in March 1972 (at 12 months).

The pipe bedding was a natural silty sand (obtained by water-settling) that was placed to a depth of 0.7 times the outside diameter. The backfill was water-settled natural earth and placed to 4 ft (1.2 m) above the top of the pipe.

PIPE DEFLECTIONS UNDER LOAD

Flexible Pipe Behavior

The external soil load on a flexible pipe causes a decrease in the vertical diameter (ΔY) and an increase in the horizontal diameter (ΔX). The horizontal movement of the pipe into the soil bedding material develops a passive resistance that acts to help support the pipe. The resistance of the soil is affected by the type of soil and its density and moisture content. The higher the soil resistance, the less the pipe will deflect.

Several design procedures exist that can be used to predict the deflection of buried flexible pipe. The deflection depends on the soil load on the pipe, the strength of the pipe, the passive resistance of the bedding soil, and the time-consolidation rate (deflection-lag factor) of the bedding soil. In the Toronto Lateral installation, the pipe strength and the load on the pipe are the same for each pipe section, but the various bedding materials allow comparisons to be made of the passive resistance of each type of material and its deflection-lag factor.

Figure 1. Compacting sand bedding with internal vibrators, Toronto Lateral.

Figure 2. Dumping loose earth around RPM pipe, Toronto Lateral.

Table 1. Bedding condition for pipe, Toronto Lateral.

Material Type and Condition	Construction Method	Vertical Deflection (percent)	Bedding Type
Cohesionless sand			
Worst	Dumped and flooded	5.9	Loose sand
Best	Saturated and vibrated	0.7	Compacted sand
Cohesive natural earth			
Worst	Dumped and flooded	8.3	Loose natural earth
Intermediate	Puddled	4.2	Puddled natural earth
Best	Pneumatically tamped	-0.3	Compacted natural earth

Table 2. Pipe deflections, Toronto Lateral.

Bedding	Vertical Deflection (percent)	In-Place Soil Density
Compacted natural earth	0	Proctor density, 95 to 97 percent
Compacted sand	1	Relative density, 30 to 38 percent
Puddled natural earth	4	Not determined
Loose sand	6	Below laboratory minimum density
Loose natural earth	8	Proctor density[a], 90 to 93 percent

[a]Measured after backfill was placed over the pipe and water settled; this does not represent bedding densities at the time of initial pipe deflection.

Effectiveness of Beddings

There are two basic types of soils used for pipe bedding: cohesive (clay and silt) and cohesionless (sand and gravel). For cohesive bedding material, USBR specifications require that the material be compacted to a minimum of 95 percent of Proctor maximum dry density [determined in the laboratory with designation E-11 (11)]. For cohesionless bedding materials, the specifications require a minimum of 70 percent relative density. From the relative density method (11), the minimum field density is established as a percentage of the range between the minimum and maximum densities of the soil as determined by laboratory tests.

The five types of beddings in this installation ranged from the worst condition (dumped and flooded) to the best condition (compacted by mechanical methods). In addition, the puddled cohesive material provided an intermediate condition. Table 1 gives the condition of each bedding type and the resulting vertical deflection of the pipe.

Soil deformation at the sides of a flexible pipe depends mostly on the soil's compressibility, which depends on the type of soil and the degree of compaction. Well-compacted sands and gravels provide good support because they have a close, interlocked granular structure and because individual grains are relatively incompressible. Well-compacted cohesive soils are more compressible because their fine-particle structure combined with water films does not permit contact and interlocking of particles. (Deflection of the pipe on the Toronto Lateral was well under 5 percent when it was property installed with well-compacted bedding.)

Because of the granular structure of a cohesionless material, it is generally considered to be a better bedding material than cohesive material, and the results indicate this. The loose (dumped and flooded) sand bedding resulted in 25 percent less pipe deflection than the loose (dumped and flooded) natural earth. The compacted sand bedding resulted in slightly more deflection than the compacted natural earth. However, the compacted natural earth bedding densities met the specifications, whereas the compacted sand bedding densities did not. The sand, as indicated by the density test results, was compacted to only 30 to 38 percent relative density. The reasons for this unusually low density for saturated and vibrated sand are not known. However, for test purposes, it provided an additional density condition for comparison purposes. If the sand had been compacted to specifications, the deflections probably would have been about the same as for the pipe in compacted natural earth. That the intermediate condition, the puddled natural earth, gave better support to the pipe than did loose sand bedding is of particular interest.

Table 2 gives the various beddings according to decreasing effectiveness. The average deflection values are compared with the range of deflection values in Figure 3.

Modulus of Soil Reaction

In 1941, Spangler published a design procedure (12) for flexible pipe that still serves as the main design method. Spangler and Watkins (13) later modified the formula to include a more realistic value for the soil parameter. The modified Iowa formula is

$$\Delta X = D_1 \frac{KW\,r^3}{EI + 0.061\,e'r^3}$$

where

ΔX = horizontal deflection of the pipe, in inches;
D_1 = deflection lag factor to compensate for the time-consolidation rate of the soil, dimensionless;
K = bedding constant that varies with the angle of the bedding, dimensionless;
W = load on the pipe per unit length, in lb/lin in.;
r = pipe radius, in inches;
EI = pipe wall stiffness per unit length, in in.-lb; and
e' = modulus of soil reaction, in psi.

Rearranging the Iowa formula to find e' values from pipe deflection gives

$$e' = 16.39\left(\frac{D_1 KW}{\Delta X} - \frac{EI}{r^3}\right)$$

The term EI/r^3 is called the ring stiffness factor and incorporates all of the physical properties of the pipe in one term. Data furnished by the pipe manufacturer give the EI value of the pipe as 6,835 in.-lb (0.772 kJ) at 5 percent deflection. When there is a radius of 15 in. (38 cm), EI/r^3 becomes 2.03 psi or 2 psi (13.79 kPa) approximately. The initial deflection values will be used; therefore a deflection lag factor of 1.0 is used. The bedding constant ranges from 0.110 for a 0-deg bedding angle (line load in the bottom of the pipe) to 0.083 for a 90-deg (1.6 rad) bedding angle (full support under the bottom half of the pipe). Most investigators of flexible pipe behavior use a bedding constant of 0.1 as a typical value, and that will be used here.

The load, W, in lb/lin in., is assumed to be the weight of the column of soil over the pipe. W is then found from

$$W = \gamma_{wet} \times h \times D$$

where

γ_{wet} = wet soil density,
h = backfill depth, and
D = pipe diameter, in inches.

The dry density of the backfill soil is about 89 lb/ft^3 (1425 kg/m^3). If one used a water content of 30 percent, the wet backfill density would be 115 lb/ft^3 (1842 kg/m^3). The backfill depth averages about 4.5 ft (1.4 m) or 54 in. W then becomes

$$(115\ \text{lb/ft}^3)\ \frac{1}{1{,}728}\ \text{in.}^3/\text{ft}^3\ (54\ \text{in.})\ (D) = 3.59\ \frac{\text{lb}}{\text{in.}^2}\ (D)$$

Substituting these values into the rearranged Iowa formula gives

$$e' = 16.39\left(\frac{1.0 \times 0.1 \times 3.59}{\Delta X/D} - 2\right)$$

$$e' = 16.39\left(\frac{0.359}{\Delta X/D} - 2\right)$$

With percent deflection for $\Delta X/D$

$$e' = 16.39\left(\frac{35.9}{\Delta X/D - \%} - 2\right)$$

where $\Delta X/D - \%$ is the percent horizontal deflection of the pipe.

Table 3 gives the e′ values calculated for the various bedding conditions. Although the modulus of soil reaction, e′, increases over 100 times for the natural earth after compaction, it is consistent with the Iowa formula. Figure 4 shows a plot of the pipe deflection for various e′ values for a pipe under 4 ft (1.2 m) of backfill. Pipe deflections are very small for well-compacted beddings that have high e′ values. Poor compaction gives low e′ values and high pipe deflections.

The data from this study also support the recent statement (14) by Spangler that e′ is a semiempirical factor and values should be chosen by using experience and judgment.

Deflection Lag Factor

The deflection lag factor, D_1, in the Iowa formula compensates for the time-consolidation rate of the soil at the sides of the pipe. Initial consolidation of the soil takes place soon after a load is applied. The soil will continue to consolidate with time, and the pipe will continue to deflect over a long time period.

Figure 3. Range and averages of percentage of vertical deflections (measured November 18, 1970), Toronto Lateral.

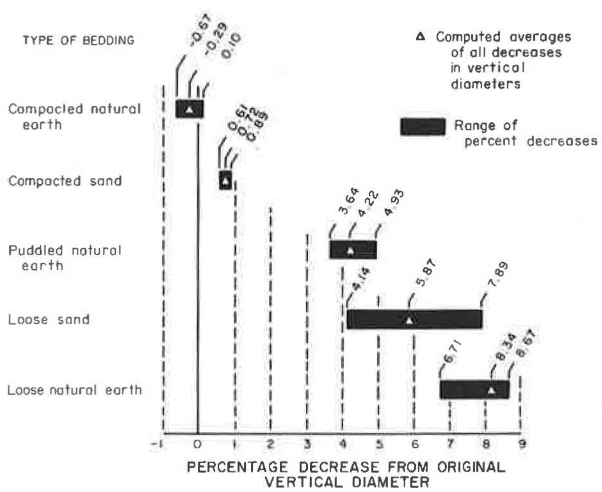
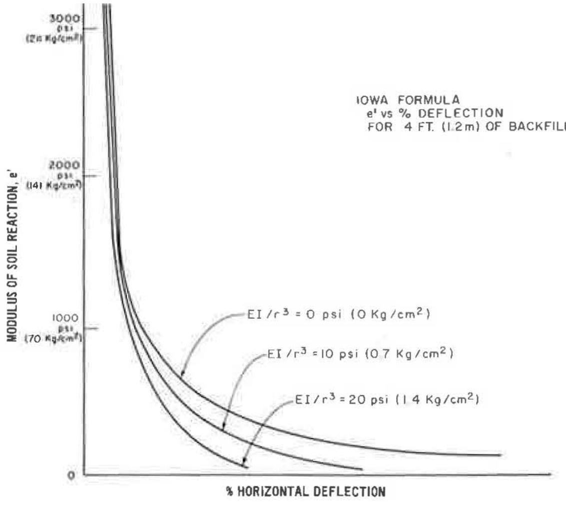


Table 3. Modulus of soil reaction values, Toronto Lateral.

Bedding	Average $\Delta X/D$ (percent)	e′, psi
Loose natural earth	7.8	45
Loose sand	5.1	83
Puddled natural earth	3.5	157
Compacted sand	0.6	948
Tamped natural earth	0.1[a]	5,851

Note: 1 psi = 6894.757 Pa.
[a]Maximum deflection measured.

Figure 4. Percentage of horizontal deflection versus e′ from Iowa formula for 4 ft (1.2 m) of backfill.



Spangler had recommended 1.5 as a maximum value for the deflection lag factor, although the time-consolidation rate for soils varies widely with the soil type and its density and moisture content and should be determined from laboratory tests.

The increases in pipe deflection with time are given in Table 4. The pipe bedded in the loose materials showed less percentage increase than the pipe in the well-compacted beddings. Apparently, most of the deflection of a pipe bedded in loose material occurs immediately, and time effect is very small.

Average pipe deflections for each of the bedding conditions, the modulus of soil reaction, and the time lag are given in Table 5.

DEFLECTIONS OF JOINTS

Only one type of soil and bedding condition was used on the Apache Lateral. Deflection measurements were made on the bell end, the center of the pipe, and the spigot end in 29 of the pipe sections and were averaged to evaluate the difference in deflection between the center of the pipe and the stiffer ends of the pipe.

The initial deflection measurement (2 months after construction) showed that average vertical deflections were, for the spigot end, 0.7 percent; for the center of the pipe, 2.3 percent; and for the bell end, 1.0 percent. These average deflection values are compared with the range of deflection values in Figure 5.

The bell end of the pipe deflected vertically 50 percent more than the spigot end and 100 percent more than the spigot end horizontally. The center of the pipe deflected vertically 350 percent more than the spigot end and 450 percent more than the spigot end horizontally.

The measurements of the original diameters were made on March 10, 1971. The first readings after backfilling were made on May 20 to 21, 1971 (at 2 months), August 26 to 30, 1971 (at 5 months), and on March 8 to 9, 1972 (at 12 months). The increases in the deflection values are given in Table 6.

Over the 10-month period, the vertical deflections of the spigot end increased 13 percent, of the center of the pipe 9 percent, and of the bell end 30 percent. Because the center of the pipe showed the least increase in deflection, the maximum difference in deflection between the pipe joint ends and the center of the pipe occurred right after installation.

ACKNOWLEDGMENTS

The extra effort of the personnel of the Yuma Project in proposing and carrying out this test program is gratefully acknowledged. T. H. Moser was project manager, and construction was directed by D. Krull and J. Durnell. H. G. Metzger, formerly of the Yuma Project, originally compiled and reported the data.

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Table 4. Deflection-lag values, Toronto Lateral.

Bedding	Diameter	Date of Measurement March 1971	Aug. 1971	March 1972
Compacted sand	Vertical	1.34	1.44	1.50
	Horizontal	1.21	1.32	1.43
Loose sand	Vertical	1.00	1.05	1.05
	Horizontal	1.03	1.00	1.04
Compacted natural earth	Vertical	Too variable to evaluate		
	Horizontal	Too variable to evaluate		
Loose natural earth	Vertical	1.04	1.09	1.10
	Horizontal	1.06	1.06	1.08
Puddled natural earth	Vertical	–	1.04	1.06
	Horizontal	1.08	1.19	1.22
Loose natural earth[a]	Vertical	1.08	1.08	1.09
	Horizontal	1.03	1.04	1.06

Note: Increase in deflection from November 1970 readings.

[a]The loose natural earth was placed in two reaches separated by the puddled natural earth section.

Table 5. Summary of results, Toronto Lateral.

Bedding	Initial Vertical Deflection (percent)	Initial Horizontal Deflection (percent)	Modulus of Soil Reaction (psi)	Deflection Lag Factor Over 16 Months
Compacted natural earth	-0.3	0.1	5,851	–
Compacted sand	0.7	0.6	948	1.50
Puddled natural earth	4.2	3.5	157	1.06
Loose sand	5.9	5.1	83	1.05
Loose natural earth	8.3	7.8	45	1.10

Note: 1 psi = 6894.757 Pa.

Figure 5. Range and averages of percentaging vertical deflections (measured May 1971), Apache Lateral.

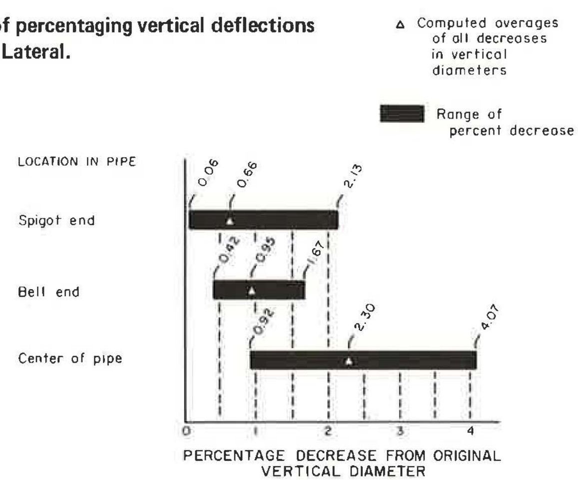

Table 6. Deflection-lag values, Apache Lateral.

Location in Pipe	Diameter	Date of Measurement Aug. 1971	March 1972
Spigot end	Vertical	0.99	1.13
	Horizontal	1.02	1.24
Center	Vertical	1.03	1.09
	Horizontal	1.05	1.12
Bell end	Vertical	1.13	1.30
	Horizontal	1.04	1.19

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TEST PROGRAM FOR EVALUATING DESIGN METHOD AND STANDARD DESIGNS FOR PRECAST CONCRETE BOX CULVERTS WITH WELDED WIRE FABRIC REINFORCING

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This paper describes the results of a test program developed to verify the design method and the standard designs for precast concrete box culverts reinforced with welded wire fabric (1). The design equations used to calculate expected test results and the structural analysis used to determine test loads equivalent to design loads are included. The test results are evaluated by comparison with the required design and ultimate loads. The evaluations verify that the design method and the standard designs are adequate and result in satisfactory designs. The results also show that the equations for determining the maximum wire spacing for crack control are conservative.

•THIS REPORT summarizes the results of a test program developed to verify the design method and standard designs for precast concrete box culverts reinforced with welded wire fabric (1). The results are compared with test strengths calculated by using the proposed design method and with required equivalent design and ultimate loads for prototype culvert designs.

NOTATION

These notations will be used throughout the paper:

- a = distance between test loads = 0.25 Si;
- a_n = depth of stress block in ultimate strength design for section n;
- A_s = area of reinforcing steel per unit width;
- AS1 = area of reinforcing steel per unit width in outside layer walls, and top and bottom slabs;
- AS2 = area of reinforcing steel per unit width in inside layer, top slab;
- AS3 = area of reinforcing steel per unit width in inside layer, bottom slab;
- b = width of unit strip (12 in.);
- C-Len = length of outside steel in top and bottom slabs to theoretical cutoff point plus anchorage length;
- C_1, C_2 = constants in various equations;
- d = depth from extreme compression fiber to centroid of tension reinforcement;
- d_1, d_2, d_3 = depth from extreme compression fiber to centroid of tension reinforcement at locations 1, 2, and 3 (Fig. 3, load arrangement);
- D-Load = total test load per ft of culvert divided by inside span, Si, in ft;
- D.T. = diagonal tension failure;
- F = flexural failure;

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f'_c = compressive strength of concrete, psi;
f_s = stress in reinforcement at service loads;
f_{su} = ultimate tensile strength of reinforcing steel;
f_y = yield strength of reinforcing steel;
M_u = ultimate design moment;
P_b = steel ratio, A_s/bd, for balanced ultimate flexural failure;
P = load on test specimen;
P_{cr} = total load on test specimen at first visible crack;
P_{des} = total test load that produces structural behavior in test specimen equivalent to effect of design earth cover;
P_u = ultimate test load;
$P_{u\ des}$ = total ultimate test load that is equivalent to required ultimate strength with design earth cover;
$P_{udt\ calc}$ = total load on test specimen calculated to cause ultimate diagonal tension failure;
$P_{udt\ test}$ = total load on test specimen in diagonal tension failure;
$P_{uf\ calc}$ = total load on test specimen calculated to cause ultimate flexural failure;
$P_{uf\ test}$ = total load on test specimen in ultimate flexural failure;
$P_{0.01\ calc}$ = total load on test specimen calculated to cause 0.01-in. crack;
$P_{0.01\ test}$ = total load on test specimen at 0.01-in. crack;
s_ℓ = spacing of longitudinal wires;
Si = span, between inside faces of side walls;
t_b = distance from centroid of tension steel to outermost concrete tension fiber;
W = weight of box culvert per unit width (12 in.); and
w = uniformly distributed load on prototype culvert.

DESIGN METHOD

The proposed design method (1) covers both ultimate strength and service load criteria. Standard culvert dimensions have been suggested (1), and a range of culvert strengths were obtained by varying the area of flexural reinforcement without the use of shear reinforcement.

The design method essentially follows the 1971 ACI code (2) and conforms to the 1972 Interim AASHO specification (3) with two relatively minor deviations. At present the design method limits the maximum steel ratio to 0.75 P_b instead of 0.50 P_b as required (3); however, none of the standard designs presented (1) has steel in excess of 0.50 P_b. The design method does not limit the maximum service load stress to 36,000 psi (3). Instead, it uses a crack control criterion to limit service load stress for reinforcing.

Crack widths are limited to 0.01 in. at the service or design load on the culvert. The required crack control is obtained by limiting the spacing of wires in the welded wire fabric reinforcing. For a maximum crack width of 0.01 in.:

$$f_{s_{0.01}} \leq \frac{65}{\sqrt[3]{t_b^2\, s_\ell}} + 5 \qquad (1)$$

Stresses calculated with Eq. 1 are lower (i.e., more conservative) than would be obtained from a similar relation (2).

TEST PROGRAM

The test program was designed to evaluate and verify the design method and standard designs that have been suggested (1).

Test Specimens

Design requirements that were established for the test culverts are given in Table 1. Three sizes were selected to represent small, intermediate, and large spans and three

designs for each size were selected to represent the lowest, intermediate, and highest heights of cover. The highest height is at or just above the design limit of diagonal tension strength for the standard wall thickness and concrete strength. Because test loads can be related more easily to field design loads, which do not include a concentrated surface load, the standard designs without truck load are used for the comparison of test results with design loads.

The arrangement of reinforcing suggested for standard culvert designs and used for test specimens and the nomenclature used in this report are shown in Figure 1. Various dimensional parameters that determine the structural behavior of the test culverts were measured for each test specimen.

Note that, because of a clerical error prior to manufacture, 2 × 6—0.5/7 fabric was called for instead of 3 × 6—0.5/7 fabric for the exterior wall reinforcing of the 6 × 4 - 2A and the 6 × 4 - 2B specimens; this resulted in 50 percent excess outside reinforcing. Furthermore, 0.5 wire instead of 1.5 wire was furnished for the inside reinforcing of the top slab, which provided 17 percent excess inside reinforcing. The bottom slab has approximately the correct reinforcing, and this portion of the culvert governs its 0.01-in. crack strength and ultimate diagonal tension strengths—the two parameters that define the design limit of these culverts.

Material Control Tests

Control tests were carried out to determine significant structural properties of steel and concrete materials in the specimens. Measured steel strengths were well in excess of the 75,000-psi minimum ultimate strength requirement (ASTM A 185).

Concrete mixes were designed by the manufacturers to meet the nominal design compressive strength of 5,000 psi. Concrete compressive strengths in the actual specimens were measured by tests on both standard cylinders and cores cut from the wall of the culverts after the test. They were representative of average strengths expected for typical 5,000-psi design mixes in commercial precasting plants.

Test Procedure

The arrangement of loads used for test specimens is shown in Figure 2. It produces approximately the same ratio of positive moment (tension on the inside of the culvert) in the top and bottom slabs of the test specimen at midspan to shear at a distance d (out from the end of the haunch) as is produced by the uniformly distributed earth load on the top and bottom slabs of the buried culvert. These two structural parameters are the most significant parameters that govern the field strength of box culverts.

Test load was recorded at the occurrence of the first 0.01-in. crack, and ultimate failure and crack patterns were sketched for all specimens. After testing, the concrete covering the reinforcing was broken off at critical locations and the depth of cover measured.

Test Results

Test results are given in Table 2 for each specimen. The insides of the top and bottom slabs are subject to tension over much of their length, and crack spacing was equal to or less than the 8-in. spacing of longitudinal wires. In many cases, there were 2 cracks per longitudinal wire space in the central region of maximum bending moment. The test load also causes tension in the outside of the side walls. No cracks were observed in the outside surface at the ends of the top and bottom slabs although tension existed.

ANALYSIS OF TEST RESULTS

Evaluation of Design Method for Limiting Crack Width

The 0.01-in. crack strengths obtained in the tests are compared with the corresponding calculated strengths for the test load arrangement, and they provide an evaluation and verification of the design method for limiting crack width. The load needed to

Table 1. Requirements for test culverts.

Culvert Size (ft × ft)	Wall Thick-ness (in.)	Design Earth Cover: Interstate Truck Load (ft)	Design Earth Cover: No Truck Load[a] (ft)	Test Box Culvert Mark	Nominal Reinforcing Area: AS1 (in.²/ft)	Nominal Reinforcing Area: AS2 (in.²/ft)	Nominal Reinforcing Area: AS3 (in.²/ft)	Distance Between Test Loads (in.)	P_{des}[b] (lb/ft)	$P_{u\ des}$[c] (lb/ft)	Ultimate Load, Calculated Test[d]: Flexural (lb/ft)	Ultimate Load, Calculated Test[d]: Diagonal Tension (lb/ft)
8×4	8	8	12	8×4-8A 8×4-8B	0.301	0.301	0.301	24	7,110	11,950	13,400	20,800
	8	2	18	8×4-2A 8×4-2B	0.516	0.410	0.410	24	10,660	17,930	21,600	20,800
	8	18	21	8×4-18A 8×4-18B	0.516	0.516	0.516	24	12,440	20,920	24,200	20,800
6×4	7	10	14	6×4-10A 6×4-10B	0.173	0.239	0.239	18	6,020	9,750	10,600	18,100
	7	2	21	6×4-2A 6×4-2B	0.273[e]	0.351[e]	0.325	18	9,030	14,620	16,100	18,100
	7	22	24	6×4-22A 6×4-22B	0.295	0.410	0.410	18	10,320	16,700	19,100	18,100
4×4	5	4	12	4×4-4A 4×4-4B	0.135	0.135	0.135	12	3,090	5,470	7,100	12,200
	5	18	20	4×4-18A 4×4-18B	0.135	0.239	0.239	12	5,160	9,120	9,900	12,200
	5	2	28	4×4-2A 4×4-2B	0.186	0.325	0.325	12	7,220	12,770	13,600	12,200

[a] Used for design loading condition with unit weight of fill equal to 120 pcf.
[b] Total test load that produces same midspan bending moment in test culvert as the design earth cover at 120 pcf produces in a buried culvert. The effect of side pressure equal to $\frac{1}{3}$ the top pressure is included in determining the proper equivalent bending moment for test specimen. The bending moment in the test culvert is for loads and supports as arranged in Figure 2.
[c] 1.5 times the total weight of 120 pcf of earth cover (with no truck) between critical shear points located at distance d out from haunch.
[d] Based on f_{su} = 75,000 psi and f'_c = 5,000 psi.
[e] 0.410 in.²/ft actually provided because wrong style fabric called for.

Figure 1. Arrangement and nomenclature for standard box culverts.

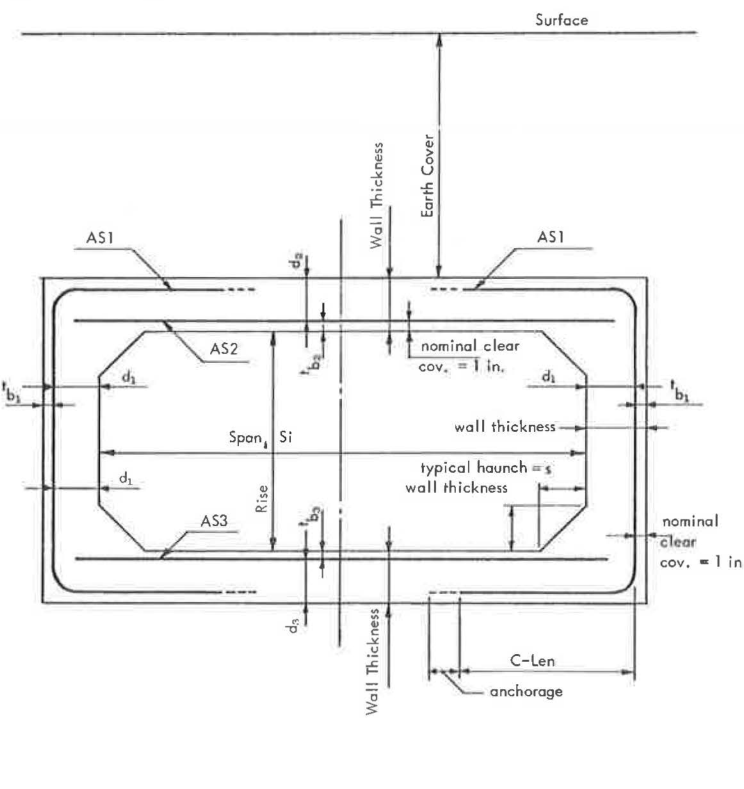


Figure 2. Test loading arrangement.

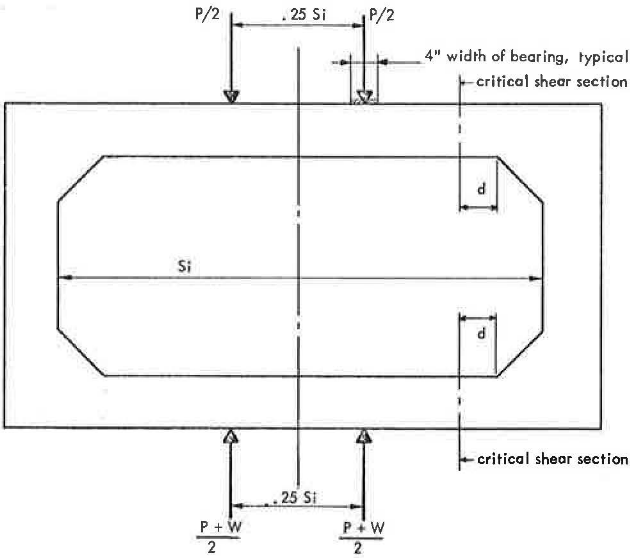


Table 2. Test results.

Box Culvert Mark	Design Earth Cover: Interstate Truck Plus Earth Cover (ft)	Design Earth Cover: No Truck (ft)	Test for First Visible Crack Load: P (lb/ft)	Test for First Visible Crack Load: D-Load (psf)	Test for 0.01-In. Crack Load: Top (lb/ft)	Test for 0.01-In. Crack Load: Bottom (lb/ft)	Test for 0.01-In. Crack Load: Minimum D-Load (psf)	Test for Ultimate Load, Flexure: P_{uf} (lb/ft)	Test for Ultimate Load, Flexure: D-Load (psf)	Test for Ultimate Load, Diagonal Tension: P_{udt} (lb/ft)	Test for Ultimate Load, Diagonal Tension: D-Load (psf)	Type of Failure Observed
8×4-8A	8	12	5,500	690	9,250[a]	11,000	1,160	17,860	2,230			F
-8B			6,500	815	11,300[a]	13,000	1,420	17,230	2,150			F
-2A	2	18	6,000	750		14,000[a]	1,760	29,690	3,710			F
-2B			7,000	880	12,300[a]	15,500	1,540			22,520	2,820	D.T.
-18A	18	21	7,500	940	13,000[a]	15,500	1,630			20,890	2,610	D.T.
-18B			7,000	880	15,000	13,500[a]	1,700			24,490	3,060	D.T.
6×4-10A	10	14	6,500	1,090		9,500[a]	1,590	16,100	2,680			F
-10B			6,800	1,135		9,500[a]	1,590	15,000	2,500			F
-2A	2	21	8,300	1,390		14,500[a]	2,430			19,400	3,230	D.T.
-2B			6,800	1,135		10,500[a]	1,760	25,250	4,210	25,250	4,210	F, D.T.
-22A	22	24	8,810	1,475		15,000[a]	2,510			25,680	4,280	D.T.
-22B			7,300	1,220	12,500[a]	14,500	2,090			21,150	3,530	D.T.
4×4-4A	4	12	5,300	1,335	6,700[a]	6,700[a]	1,690	8,980	2,245			F
-4B			4,300	1,085	6,000[a]	7,000	1,510	8,440	2,110			F
-18A	18	20	5,800	1,465	7,000[a]	7,000[a]	1,770	13,150	3,290			F
-18B			5,500	1,385	8,000[a]	8,500	2,010	13,170	3,290			F
-2A	2	28	5,000	1,265	7,800[a]	7,800[a]	1,980			14,080	3,520	D.T.
-2B			5,300	1,335	8,500[a]	8,500[a]	2,140	19,300	4,830			F

[a]Lowest test 0.01-in. crack load.

produce a 0.01-in. crack in the top or bottom slab under the test load arrangement may be determined from the following equations, which are based on the maximum bending moments shown in Figure 3.

$$P_{0.01\ calc} = C_1 A_s\, d\, f_{s_{0.01}} - C_2 W \tag{2}$$

where

C_1 = 139 for 4 × 4 culverts, 107 for 6 × 4 culverts, and 89 for 8 × 4 culverts and
$C_2 \approx 0.9$ for 0.01-in. crack in bottom slab and 0.2 for 0.01-in. crack in top slab,

or

$$f_{s_{0.01}} = \frac{65}{\sqrt[3]{t_b^2\, s_\ell}} + 5 \tag{1}$$

or f_y, whichever is less.

Values for $P_{0.01\ calc}$ were calculated for all test culverts by using Eqs. 1 and 2 and the actual measured values for wall thickness, concrete cover thickness, and steel area. Test and calculated 0.01-in. crack loads are given in Table 3.

For the entire 18 test specimens of the culvert test program, the average $P_{0.01\ test}/P_{0.01\ calc} = 1.29$. The coefficient of variation is 34 percent, and the standard deviation is 0.43. If the two specimens with the lowest steel areas for each size culvert are excluded from the statistical analysis, for the 12 remaining specimens the average $P_{0.01\ test}/P_{0.01\ calc} = 1.08$. The coefficient of variation is 14 percent, and the standard deviation is 0.15. Figure 4 graphically shows this comparison.

The test results show that the design equations give a low (i.e., very conservative) estimate of 0.01-in. crack strength for lightly reinforced culverts. This probably occurs because, for these structures, the 0.01-in. crack strength is not much greater than the first visible crack strength and the concrete flexural strength between cracks significantly reduces the average stress in the reinforcement. A similar phenomenon was observed in pipe tests (7), and the semiempirical equation for a 0.01-in. crack strength of pipe (7) contains a term reflecting the contribution of flexural concrete strength between cracks. This term is most significant for lightly reinforced pipe.

Excluding the lightly reinforced test specimens, the correlation between test and calculated 0.01-in. crack strength is good, and the coefficient of variation is typical of statistical variation for 0.01-in. crack strength obtained in many pipe tests (7, 8).

Evaluation of Design Methods for Ultimate Strength

The ultimate strengths that were obtained in the test culverts are compared with the corresponding calculated strengths in both flexure and diagonal tension for the test load arrangement. This provides an evaluation and verification of the design methods.

The ultimate flexural strength is determined by using two assumptions from the plastic theory of reinforced concrete behavior:

1. The critical flexural sections are underreinforced. At the sections of maximum bending moment, welded wire fabric reinforcing can be stressed to its ultimate tensile strength without failure of concrete in compression.
2. The ductility of the underreinforced sections of maximum bending moment with welded wire fabric reinforcing causes flexural failure to occur by tensile rupture of the steel reinforcing at one or more sections of maximum bending moment. This occurs only after plastic hinges have formed at the sections of maximum bending moment at the bottom midspan and sidewall exteriors, or top midspan and sidewall exteriors. The shear and bending moment diagrams are shown in Figure 5 for the test load arrangement.

The first assumption is the standard basis for calculating ultimate bending strength (2, 3) except that, for welded wire fabric reinforcing, the ultimate strength of the steel

Figure 3. Load, shear, and bending moment diagrams for test load.

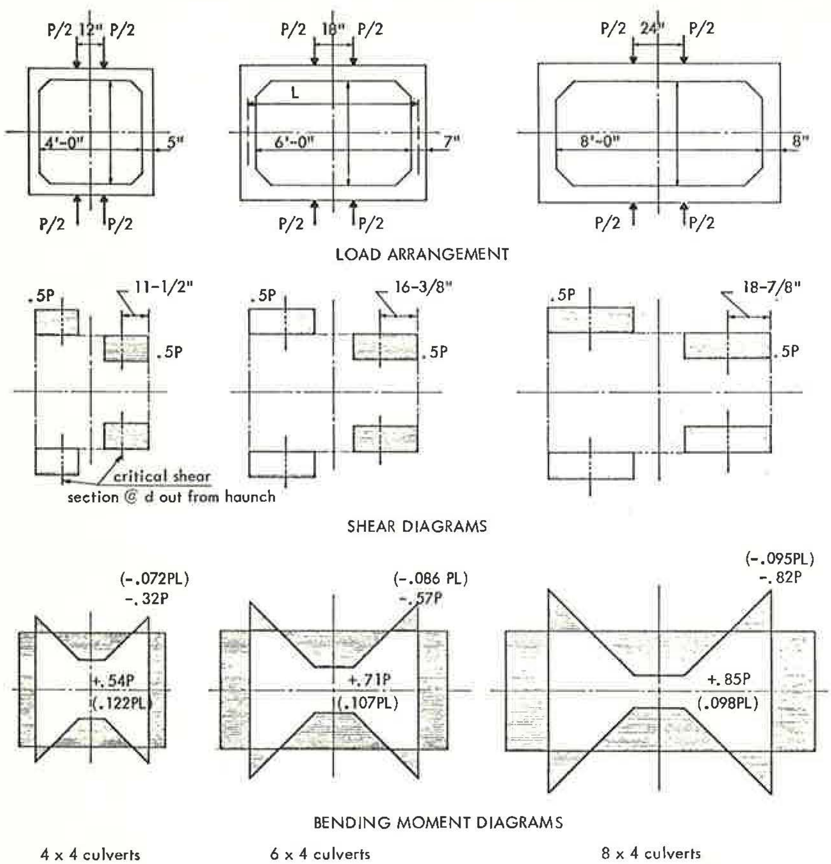


Table 3. Comparison of test and calculated 0.01-in. crack strengths.

Box Culvert Mark	Test $P_{0.01\ test}$ (lb/ft)	Test $f_{s0.01}$[a] (ksi)	Calculated Top $P_{0.01\ calc}$ (lb/ft)	Calculated $f_{s2\,0.01}$[b] (ksi)	Calculated Bottom $P_{0.01\ calc}$ (lb/ft)	Calculated $f_{s3\,0.01}$[b] (ksi)	$\frac{P_{0.01\ test}}{P_{0.01\ calc}}$
8×4-8A	9,250 T[c]	54.6	7,840	46.7	6,570	49.4	1.18
-8B	11,300 T	62.5	9,430	52.7	6,400	49.4	1.19
-2A	14,000 B[c]	61.8	10,540	43.9	10,950	50.3	1.29
-2B	12,300 T	48.6	12,360	48.8	9,410	46.2	1.00
-18A	13,000 T	41.8	15,650	50.0	12,550	47.2	0.83
-18B	13,500 B	50.1	13,940	45.9	13,442	50.0	1.00
6×4-10A	9,500 B	71.4	8,743	54.8	5,830	48.2	1.62
-10B	9,500 B	69.9	7,640	49.7	6,220	49.7	1.53
-2A	14,500 B	73.9	14,900	54.4	10,640	56.4	1.36
-2B	10,500 B	55.8	11,610	45.4	10,640	56.4	0.99
-22A	15,000 B	61.1	14,410	53.8	12,510	52.1	1.20
-22B	12,500 T	49.4	11,090	44.0	12,510	52.1	1.13
4×4-4A	6,700 B	97.0[d]	2,760	41.8	2,740	47.2	2.45
-4B	6,000 T	87.9[d]	2,700	41.3	2,020	41.8	2.22
-18A	7,000 B	56.5	6,220	49.8	7,770	62.0	0.90
-18B	8,000 T	59.7	7,090	53.1	5,740	51.3	1.13
-2A	7,800 B	51.6	6,690	40.4	6,940	46.6	1.12
-2B	8,500 B	52.0	6,600	41.3	8,380	51.4	1.01

[a]Test $f_{s0.01}$ is the calculated reinforcing stress in the top or bottom slab, whichever governs, for the test 0.01-in. crack load.
[b]Calculated $f_{s0.01}$ is the reinforcing stress (Eq. 2) that determines $P_{0.01\ calc}$.
[c]T = top and B = bottom. They denote slab location where first 0.01-in. crack occurred.
[d]Yield strength of wire equals 78.5 ksi.

Figure 4. Comparison of test and calculated 0.01-in. crack strengths.

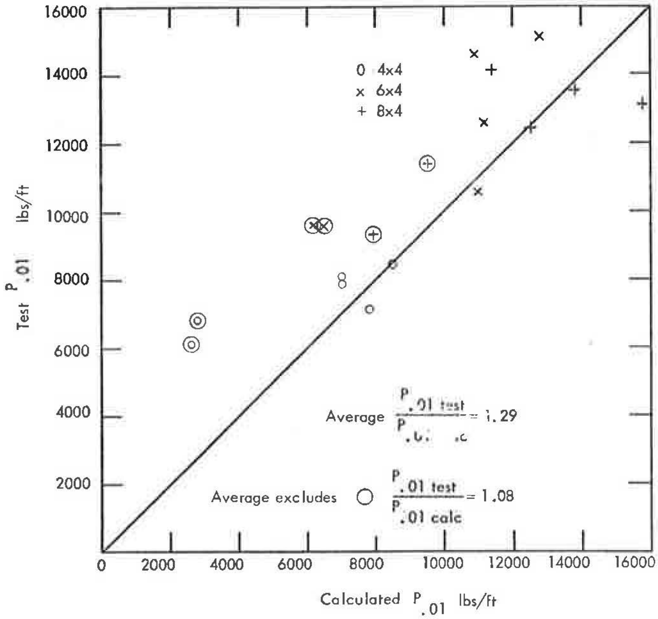
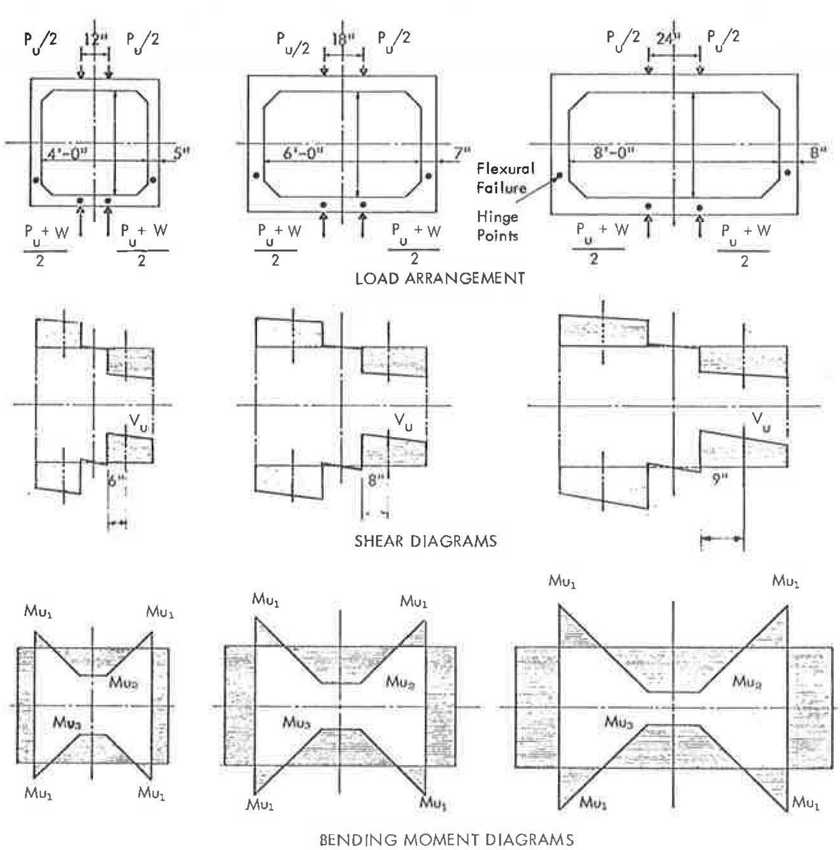


Figure 5. Load, shear, and bending moment diagrams for ultimate test load.



is used instead of the yield strength. This assumption is confirmed by the tensile rupture of reinforcing attained in the box culvert test specimens that failed in flexure. It is also confirmed by the results of many pipe tests (7). The ultimate tensile strength is used only for comparison of calculated and measured strengths in the test program and is not used for design of standard culverts. The more conservative yield strength is used as the maximum reinforcing strength for ultimate strength design of the standard box culverts.

The second assumption is based on the observed behavior of pipe that is reinforced with welded wire fabric and that fails in flexure (6, 7, 8). Some of the box culverts that failed in flexure showed the same behavior, namely, rupture or near rupture of both inner reinforcing at the midspan and outer reinforcing at sidewall locations at the time of failure.

The load to produce ultimate flexural failure for the test load arrangement is determined from the following equations, which are based on the ultimate flexural theory and the assumptions explained above.

$$P_{uf\ calc} = \frac{4(M_{u3} + M_{u1})}{(0.75\ Si + 2)} - C_s\ W \tag{3}$$

$$M_{un} = A_{sn} f_{sun}\ (d_n - a_n/2) \tag{4}$$

$$a_n = \frac{f_{sun}\ A_{sn}}{10.2\ f_c'} \tag{5}$$

Calculate for sections with n = 3 and 1, or 2 and 1 of Figure 1 (i.e., use of values of A_{s1}, f_{su1}, d_1, and a_1 for section 1) where $C \simeq 0.9$ for bottom slab failure and 0.2 for top slab failure.

Values for $P_{uf\ calc}$ were calculated for all test culverts by using Eqs. 3, 4, and 5 and the actual measured values for wall thickness, concrete cover thickness, steel area, steel ultimate tensile strength, and concrete ultimate compressive strength (based on cores). Test and calculated ultimate flexural loads are given in Table 4 for culverts that failed in flexure, and Figure 6 graphically shows this comparison.

For the 10 test specimens that failed in flexure, the average $P_{uf\ test}/P_{uf\ calc} = 1.03$. The coefficient of variation and the standard deviation are both 6 percent.

The load calculated to produce ultimate diagonal tension (shear) failure for the test load arrangement is determined from the following equation:

$$P_{udt\ calc} = 48\ d_n\ f_c - C_4\ W \tag{6}$$

where $C_4 \approx 0.9$ for bottom slab failure and 0.2 for top slab failure.

Values for $P_{udt\ calc}$ were calculated (by using Eq. 5) for (a) all test culverts, (b) actual measured values for wall thickness, (c) cover thickness, and (d) concrete compressive strength. Test and calculated ultimate diagonal tension loads are compared in Table 4 for culverts that failed in diagonal tension, and Figure 7 graphically shows this comparison.

For the 8 test specimens that failed in diagonal tension, the average $P_{udt\ test}/P_{udt\ calc} = 1.02$. The coefficient of variation and the standard deviation are both 12 percent. Both comparisons (tested and calculated for flexure and shear failure) show excellent correlations and are typical of other pipe tests (7, 8).

Evaluation of Standard Box Culvert Designs

The test results may be used for a direct evaluation of standard culvert designs (1) by determining the test loads that represent the equivalent design earth load in the test arrangement and the required ultimate test load in the test arrangement.

Shear and bending moment diagrams are shown in Figure 8 for uniformly distributed vertical pressure on the top slab and reaction on the bottom slab and are shown in Figure 9 for a uniformly distributed lateral load on each side equal to ⅓ the vertical

Table 4. Comparison of test and calculated ultimate strengths.

Box Culvert Mark	Flexural Failure $P_{uf\ test}$ (lb/ft)	$P_{uf\ calc}$ (lb/ft)	$\frac{P_{uf\ test}}{P_{uf\ calc}}$	Diagonal Tension Failure $P_{udt\ test}$ (lb/ft)	$P_{udt\ calc}$ (lb/ft)	$\frac{P_{udt\ test}}{P_{udt\ calc}}$
8×4-8A	17,860	16,050	1.11		34,430[a]	
-8B	17,230	15,780	1.09		21,582	
-2A	29,690	28,200	1.05		20,120	
-2B		27,750		22,520	21,520	1.05
-18A		31,800		20,890	21,590	0.97
-18B		31,830		24,490	22,860	1.07
6×4-10A	16,100	15,380	1.05		22,170	
-10B	15,000	15,390	0.98		23,360	
-2A		28,690		19,400	23,420	0.83
-2B		28,990		25,250	23,950	1.05
-22A		26,690		25,680	21,260	1.21
-22B		26,700		21,150	23,530	0.90
4×4-4A	8,980	9,800	0.92		14,900	
-4B	8,440	9,011	0.94		14,260	
-18A	13,150	12,680	1.04		16,350	
-18B	13,170	12,730	1.03		14,670	
-2A		17,290		14,080	12,790	1.10
-2B	19,300	18,510	1.04		14,670	

[a]Abnormal core strength causes high calculated results.

Figure 6. Comparison of test and calculated ultimate flexural strengths.

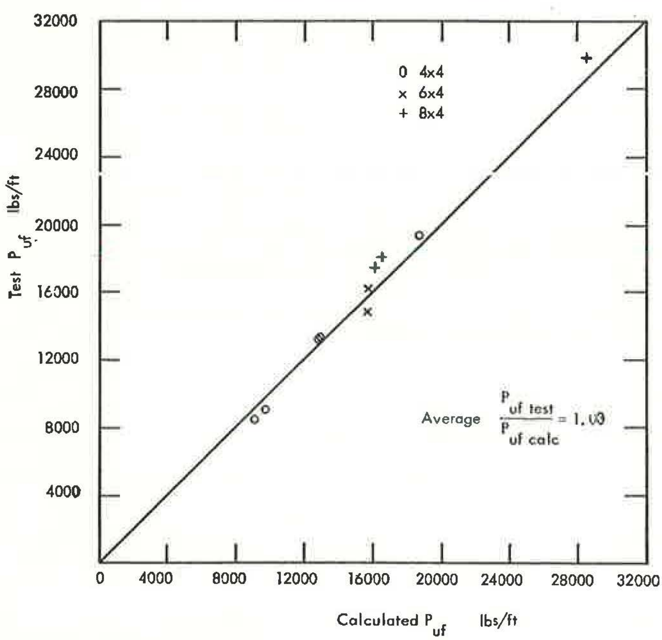


Figure 7. Comparison of test and calculated ultimate diagonal tension strengths.

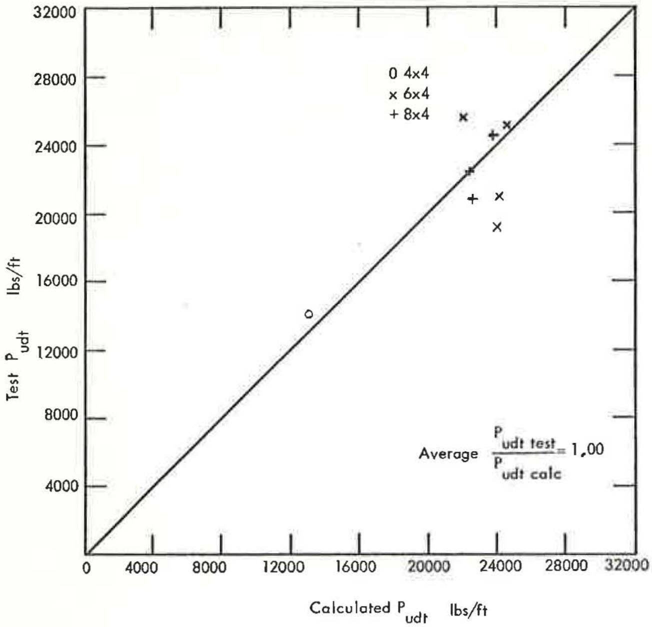


Figure 8. Load, shear, and bending moment for culvert subjected to uniform vertical earth load.

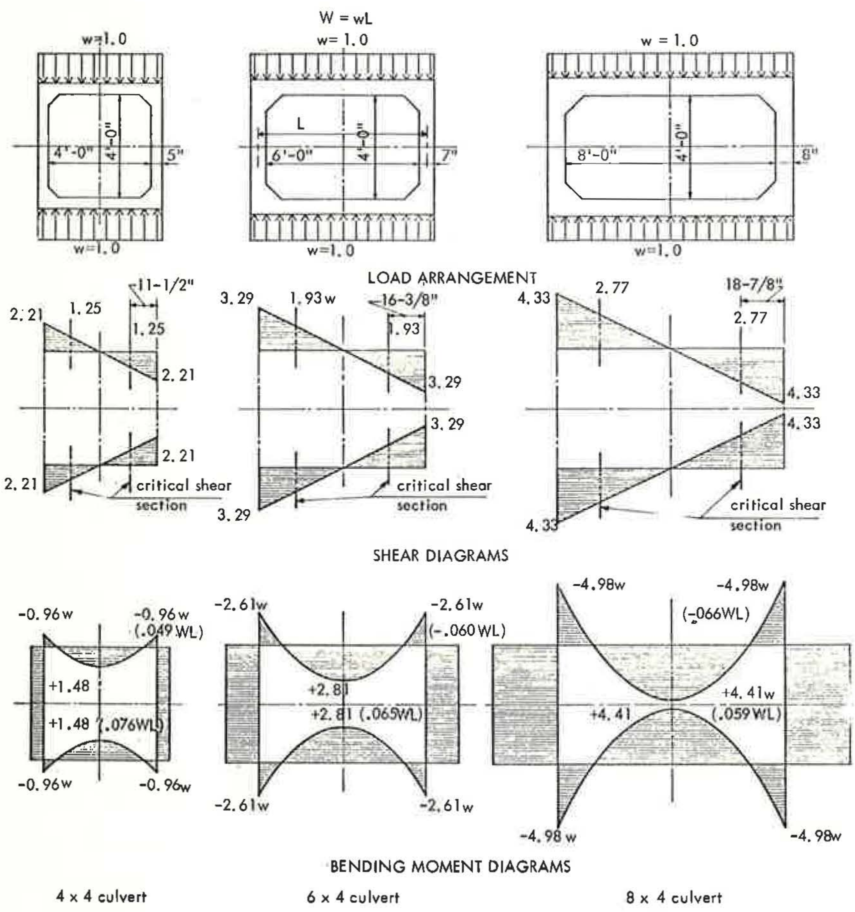


Figure 9. Load, shear, and bending moment for culvert subjected to uniform lateral earth load.

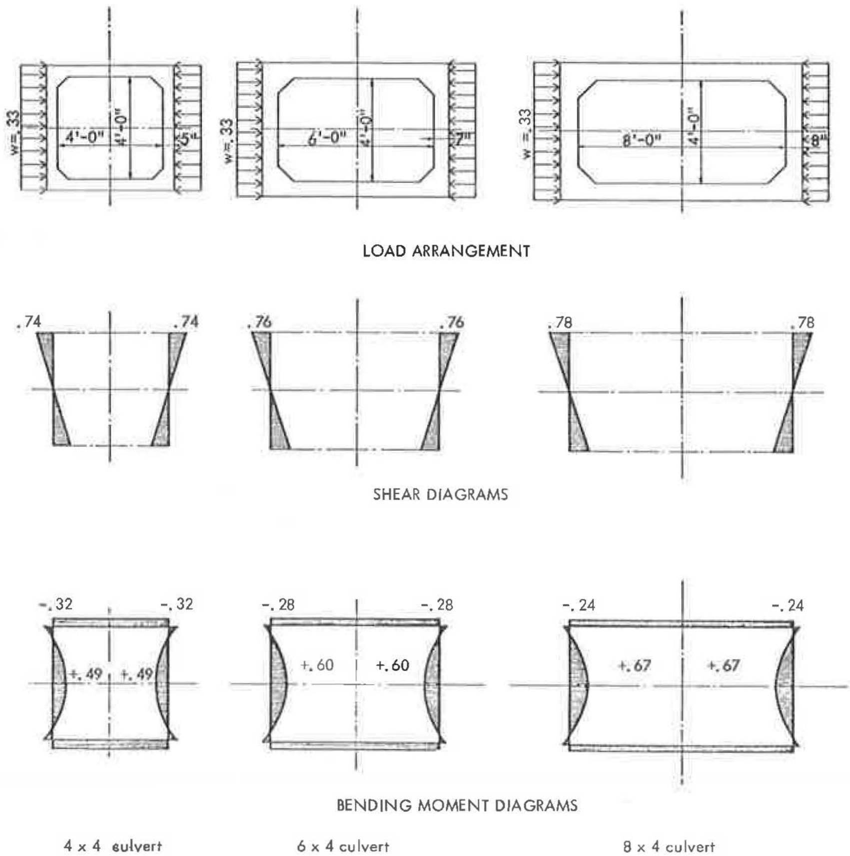


Table 5. Comparison of test and design loads.

Box Culvert Mark	Design Earth Cover: Interstate Truck Load (ft)	Design Earth Cover: No Truck Load[a] (ft)	P_{des}[b] (lb/ft)	$P_{0.01\ test}$ (lb/ft)	$\frac{P_{des}}{P_{0.01\ test}}$	$P_{u\ des}$[c] (lb/ft)	$P_{u\ test}$ (lb/ft)	$\frac{P_{u\ des}}{P_{u\ test}}$	Minimum Calculated Ultimate Load[d]: $P_{uf\ calc}$ (lb/ft)	Minimum Calculated Ultimate Load[d]: $P_{udt\ calc}$ (lb/ft)	Type of Failure Observed	$\frac{P_{u\ test}}{P_{uf\ calc}}$	$\frac{P_{u\ test}}{P_{udt\ calc}}$	P_{cr} (lb/ft)	$\frac{P_{cr}}{P_{des}}$
8×4-8A	8	12	7,110	9,300	1.31	11,950	18,000	1.51	13,400	20,800	F	1.34	–	5,500	0.77
-8B				11,300	1.59		17,400	1.46			F	1.30		6,500	0.91
-2A	2	18	10,660	14,000	1.31	17,930	30,000	1.67	21,600	20,800	F	1.39	–	6,000	0.56
-2B				12,300	1.15		22,800	1.27			D.T.		1.10	7,000	0.66
-18A	18	21	12,440	13,000	1.05	20,920	21,000	1.00	24,200	20,800	D.T.		1.01	7,500	0.60
-18B				13,500	1.08		24,800	1.19			D.T.		1.19	7,000	0.56
6×4-10A	10	14	6,020	9,500	1.58	9,750	16,100	1.65	10,600	18,100	F	1.52	–	6,500	1.08
-10B				9,500	1.58		15,000	1.54			F	1.42	–	6,800	1.13
-2A	2	21	9,030	14,500	1.61	14,620	19,500	1.33	16,100	18,100	F, D.T.		1.07	8,300	0.92
-2B				10,500	1.16		25,500	1.74			F, D.T.		1.41	6,800	0.75
-22A	22	24	10,320	15,000	1.45	16,700	25,800	1.54	19,100	18,100	D.T.		1.43	8,800	0.85
-22B				12,500	1.21		21,400	1.28			D.T.		1.18	7,300	0.71
4×4-4A	4	12	3,090	6,800	2.20	5,470	9,100	1.66	7,100	12,200	F	1.28	–	5,300	1.71
-4B				6,000	1.94		8,600	1.57			F	1.21	–	4,300	1.39
-18A	18	20	5,160	7,000	1.36	9,120	13,200	1.45	9,900	12,200	F	1.33	–	5,800	1.12
-18B				8,000	1.55		13,200	1.45			F	1.33	–	5,500	1.07
-2A	2	28	7,220	7,800	1.08	12,770	14,200	1.11	13,600	12,200	D.T.		1.16	5,000	0.69
-2B				8,500	1.18		19,500	1.53			F	1.43	–	5,300	0.73
Average					1.41			1.44				1.36	1.19		

[a] Used for design loading condition with unit weight of fill equal to 120 pcf.

[b] Total test load that produces the same midspan bending moment in the test culvert as the design earth fill height produces in a buried culvert. Load from design fill height is taken as the weight of a column of 120 pcf earth having same width as culvert. The effect of side pressure equal to ⅓ the top pressure is included in determining the proper equivalent bending moment for test specimen. The bending moment in the test culvert is for loads and supports as arranged in Figure 2.

[c] 1.5 times the total weight of 120 pcf earth cover (with no truck) between critical shear points located at distance d out from haunch.

[d] Based on f_{su} = 75,000 psi and f'_c = 5,000 psi.

pressure. The midspan bending moments in Figure 3 for test loads and in Figures 8 and 9 for field loads are equated to obtain the test load, which is equivalent to the design earth fill height.

Design earth fill heights and P_{des} are given in Table 5 for each test culvert. These equivalent design loads are compared with $P_{0.01\ test}$ in the table. The average $P_{0.01\ test}/P_{des} = 1.41$. All test culverts exhibited a higher test 0.01-in. crack load than the test load that produces the same maximum slab bending moment as the design earth fill height. A graphical comparison of $P_{0.01}$ and P_{des} is shown in Figure 10.

The required minimum ultimate load for the design earth fill height is the test load that equals 1.5 times the weight of a column of 120-pcf earth extending between the critical shear sections on each side of the top slab. The critical shear section is at distance d into the slab from the edge of the haunch on each side. The spacing of test loads (Fig. 2) was established to obtain the same ratio of midspan positive moment to shear at the critical section in the test specimen as the ratio that occurs in a similar buried culvert. The moment in the buried culvert is the slab midspan positive moment caused by uniform vertical loads only. Thus, the ultimate load for design earth fill height produces both the same shear at a critical section d out from the haunches and the same midspan positive moment in the test culvert as 1.5 times the design earth fill height produces in the buried culvert (not accounting for the effect of side pressure).

Test loads equivalent to $P_{u\ des}$ are given in Table 5 for each test culvert and compared with $P_{u\ test}$. The average $P_{u\ test}/P_{u\ des} = 1.44$. All test culverts had a higher test ultimate load than the required $P_{u\ des}$. A comparison of $P_{u\ test}$ and $P_{u\ des}$ is graphically shown in Figure 11.

CONCLUSIONS

The test program results verify that the proposed design method provides satisfactory designs for precast concrete box culverts within the range of earth fill heights and culvert dimensions used for standard designs (1). They also provide a direct verification of the adequacy of nine standard designs, which cover the range of strength and dimensions of proposed standard designs.

The results show that there is additional reserve ultimate strength capacity of at least 15 percent above the 1.5 ultimate strength load factor used for standard designs that are governed by flexural ultimate strength. This is because the ultimate tensile strength of the steel is developed before flexural failure occurs. Because the standard designs with welded wire fabric reinforcing are based on a maximum steel stress equal to the 65,000-psi minimum yield strength of the wire and the wire has a specified minimum ultimate tensile strength of 75,000 psi, the additional ultimate flexural capacity of the culverts is at least the ratio of these strengths times the design ultimate flexural capacity.

This additional reserve capacity is not available for those designs near the upper end of the design fill heights because their strength is governed by diagonal tension failure; however, the test results show that the proposed design method provides the 1.5 specified load factor.

The test results also show that the proposed equations for determining the maximum wire spacing for crack control give increasingly conservative results as reinforcing reduced toward the low end of the range of proposed heights of earth fill. This occurs because, for these designs, the tensile strength of the concrete between cracks contributes a significant resistance to flexural deformation between cracks and reduces the crack width. However, for more heavily reinforced designs, the contribution of concrete tensile strength between cracks is much less significant. From a practical point of view, the design method already shows that wire spacing is not a critical parameter in lightly reinforced designs; therefore, the conservatism does not result in a penalty in practical design. Further research would probably show that a modified 0.01-in. crack equation for pipe (7, 8), which takes into account tensile resistance of concrete between cracks, would probably provide a better correlation of test and calculated 0.01-in. crack strength. However, such a development is not necessary for practical design of box culverts.

Figure 10. Comparison of test loads and design loads.

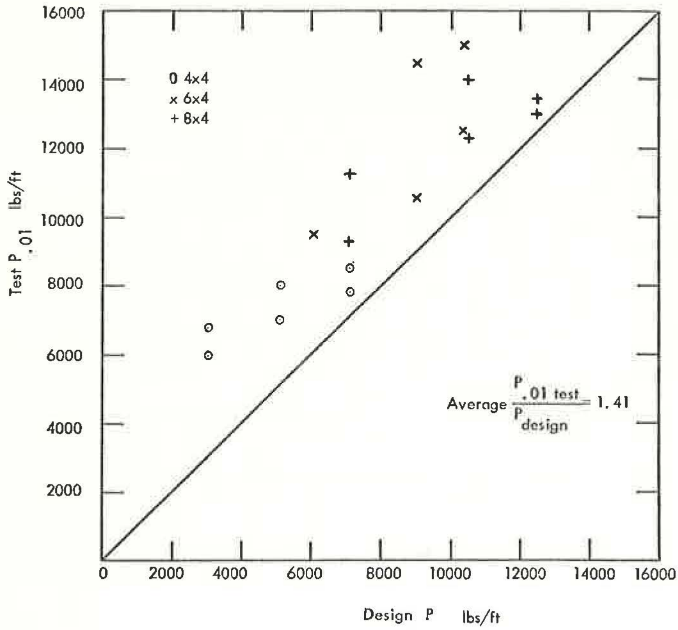
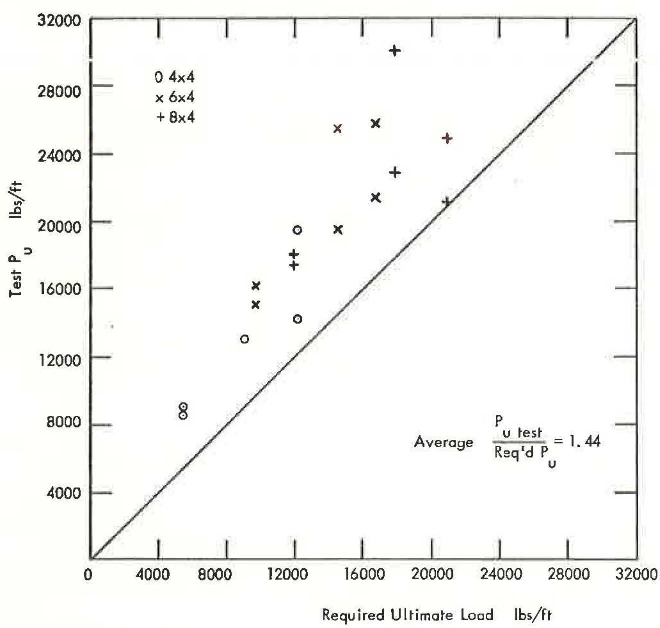


Figure 11. Comparison of test loads and required ultimate load.



Both the ultimate strength and crack control design methods proposed (1) for box culvert design are based on the current ACI code for reinforced concrete design, which is widely accepted without verification of specific applications by tests.

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