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C O N T E N T S

TRENDS IN MODERN STEEL BRIDGE DESIGN

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Introduction

There have been many changes in the design of steel bridges during the past thirty years, changes in the types of structure, changes in methods of design, and changes in materials. Some of these changes are representative of trends which have appeared, been accepted, and then have become almost standard practice, such as the now generally accepted use of statically indeterminate design. Some of the changes are representative of continuing trends, such as the increasing use of welded construction, and of low alloy steels. As a starting point, it may be well to review, briefly, the changes in types of short-span bridges during the past thirty years.

In 1930, most steel bridges could have been classified, as to span and type, about as follows:

Spans less than 60 ft:	Simple stringer bridges
Spans 60 ft to 120 ft:	Deck trusses or girders, pony trusses, or through plate girders
Spans 120 ft to 150 ft:	Deck or pony trusses
Spans over 150 ft:	Deck or through trusses

These limits were not rigid, and there was considerable overlap. The bridges were nearly always simple spans, although a number of continuous truss spans had been built before 1930. Continuous stringer and plate girder bridges were coming into uso during the early 1930's and, by 1940, were common.

Today, a comparable classification would be about as follows:

Spans less than 80 ft:	Continuous stringer bridges, or simple stringer bridges of composite design
Spans 80 ft to 350 ft:	Deck plate girders, either riveted or welded

Spans over 350 ft: Through trusses

Again, the span limits are not rigid, and there is some overlap.

The most noticeable change to be noted in comparing the two classifications is the almost total disappearance of the short and medium-span through trusses which used to be so common. This change came about because of the decided preference for deck structures, with no obstructions above the roadway level, which developed about 1940. Today, short-span trusses are almost a thing of the past.

Another trend, which is still in progress, is the remarkable increase in the span lengths for which plate girder bridges are used. This increase has come about partly from the drift away from through structures, partly because of more economical methods of design, and to some extent, from the increased use of welding and of alloy steels.

Along with the changes in bridge types have come improved methods of design, improvement in fabricating facilities, a larger variety of steels with which to work, better erection facilities, and better fasteners. Let us look at a few of these trends in more detail.

Continuous Construction

Until about 1930, a great many bridge engineers objected to the use of statically indeterminate structures because they felt that any settlement of the supports would cause large and indeterminate overstresses in the principal load-carrying members. For a long time, these engineers would not consider the use of continuous construction unless the substructure was to be founded on rock, or on piles driven to rock. Today, few bridge engineers would give this point of view serious consideration, and a large proportion of our bridge structures are statically indeterminate.

Continuous bridges are considerably stiffer than simple bridges of comparable spans, but they do develop vibratory effects under the passage of heavy truck loads which, at times, may be objectionable. These effects are seldom noticeable to the drivers of the vehicles, and they have no bearing whatever upon the strength of the structure.

The saving in metal due to the use of continuity in design will vary widely, depending upon the ratio of dead to live load, the span lengths, and other factors, but for the ordinary three-span stringer bridge, the saving will average around 25 percent. This is too large an economic factor to be disregarded.

Composite Design

The use of composite design for short stringer spans has been increasing steadily for several years. Primarily, it furnishes a means of using the deck slab as a part of the top flange of the stringer. For simple spans, composite design will show a saving of 15 to 20 percent in weight of metal as compared with noncomposite designs, the amount of saving depending upon the span length and the ratio of dead to live load. This saving may be at least partly theoretical, however.

In a non-composite design, we assume that the deck slab is entirely free of the steel beam, and we design the steel beam to carry the whole load. Actually, under working loads, it is doubtful if the bond between the slab and the top flange of the steel beam will ever be broken. Unless, it is broken, we have a composite design for working loads, whether we figure it that way or not, and we have a stiffer structure than we would have it composite action had been taken into account in the design.

In composite design, the top flange of the steel beam is located close to the neutral axis of the composite section and, consequently, is never stressed to its full capacity. This has led to suggestions that a new series of rolled beams be developed, with one flange smaller than the other. It seems doubtful if this will ever be done, since the steel mills would have to set up manufacturing facilities for a whole new series of beams, and the market for them would not be large enough to justify the expense. However, such sections can be fabricated easily by welding and, in many cases, the welded beams would be lighter and more economical than the rolled beams.

Deck Plate Girders

One of the most amazing trends in recent years has been the progressive increase in span lengths of plate girder spans. Until about 1940, plate girder span lengths were limited from 100 to 120 ft. Since that time, the spans have been increased to 350 ft and more. About seven years ago, the Reclamation Bureau built a three-span continuous bridge across the Snake River in Wyoming with spans of 136-264-126 ft. Three or four years ago, the Quinnipiac River bridge on the Connecticut Turnpike was built, containing a continuous unit of 258-387-258 ft. These were both riveted structures. More recently, the Buffalo Bayou bridges, in Texas, were built with welded girders in continuous units of 198-270-198 ft. California now has a continuous welded girder bridge under construction with spans of 260-350-260 ft. This last bridge is particularly interesting, because three different grades of steel are being used in the fabrication of the girders.

One thing which has helped to promote the use of longer and longer plate girder spans has been the increasing use of longitudinal stiffeners, which permit cutting the web thickness for a given depth of girder in half. The longitudinal stiffeners are relatively light, and the metal saved in the web can be used much more efficiently in the flanges.

This matter of web plate thickness deserves more attention. The depth-to-thickness ratios in our specifications have been developed from rational buckling theories, and are based upon the yield point of the steel, modified by a factor of safety. They depend upon the compressive stresses in the web plate adjacent to the compression flange, and not upon the shear in the web, as is sometimes assumed. Some engineers are now advocating higher depth ratios, such as are commonly used in European practice. They believe that the present specifications ratios are too conservative, and that the girders would not fail even if the critical buckling stresses were to be carried well above the yield point of the steel. Extensive tests are now being conducted at Lehigh University to obtain more data on the subject. The results reported so far, however, are not conclusive. They have shown that the test girders do not fail until after the critical buckling stresses are far above the yield point of the steel but, at failure, the girders are so distorted that they would be useless in a practical structure. It is to be hoped that the final report will indicate how high our depth-to-thickness ratios can be raised safely, if they can be raised. If so, it should not be difficult to revise our present specifications.

Under present specifications, the depth from which the web thickness is determined is the clear depth between the toes of the flange angles for riveted girders, and the clear depth between flanges for welded girders. This means that, for a given web thickness, a riveted girder can be made from 8 to 16 inches deeper than a welded one, depending on the size of the flange angles. In places where headroom is not a governing factor, this gives the riveted girder an advantage.

It is difficult to predict what the maximum spans for plate girders will be in the future. It is probable, however, that the limit will be governed by shipping and erection limitations rather than by design limitations.

<u>Alloy Steels</u>

Nickel and silicon steels have been used in bridge work for many years, but cost considerations limited their use to long-span structures. A few years ago, so-called low-alloy steels were developed, and since 1948 the AASHO specifications have made provision for its use. Quite often, for any particular structure, low-alloy steel will prove to be more economical than carbon steel, even for short spans. However, there are several grades of low alloy steel, each of which has its own characteristics, and its own base price. For instance, a weldable low-alloy steel costs about one cent per pound more than one which would be satisfactory for riveted work. For any particular application, careful study is needed to select the proper grade of steel, and to be sure that it is properly used. The base prices of low-alloy steels vary from 25 to 45 percent above that for carbon steel, but the erected price seems to average about 10 to 15 percent above that of carbon steel.

The basic allowable stresses vary with the thickness of the material and are as follows:

For material less than 3/4 in. thick:	27,000 psi
For material 3/4 in. to 1-1/2 in thick:	24,000 psi
For material 1-1/2 in. to 2 in. thick:	22,000 psi

Obviously, it is advantageous to use material less than 3/4 in. thick whenever it is possible to do so.

At first glance, it would seem that a stress advantage of 22 to 50 percent can be obtained at an increased cost of only 10 to 15 percent. Unfortunately, there are many factors which erode the apparent stress advantage. Only the very lightest of the WF sections have flange thicknesses less than 3/4 in., so rolled beam spans have to be designed for the 24,000 psi working stress. For welded girders, it will often be found impossible to keep the flange thickness under 3/4 in., and for longer spans, it will be difficult to keep it below 1-1/2 in. Plate girders, either riveted or welded, will require web plates about 23 percent thicker than would be required for riveted girders of the same depth, resulting in a less efficient section. Often, live load deflection limitations will make it necessary to use larger beam sections than stress considerations would require. Columns of small slenderness ratios will have considerably more strength in low-alloy than in carbon steel, but as the slenderness ratios increase the advantage decreases, and for long, very slender columns the alloy steel has no stress advantage at all.

For riveted plate girders of moderate span length, it is nearly always possible to keep the thickness of all material under 3/4 in., and thus the basic 27,000 psi allowable stress can be used, while welded girders are nearly always limited to 24,000 psi or 22,000 psi. This gives the riveted girders an advantage which can not be overcome by welding economies.

With all of these factors, plus some others, to be considered, it becomes necessary to study each individual bridge carefully to determine whether or not alloy steel will be economical. A recent study of three continuous stringer bridges showed low-alloy steel to be the cheaper for two of them, but carbon steel was cheaper for the third.

Some mention should be made of the T-1 alloy steel which was developed by the U. S. Steel Corporation about six years ago. It is a very high-strength steel with a yield point of 90,000 psi, as compared to 33,000 psi for carbon steel, but it is also a high-priced steel. Its erected price will be from 1.5 to 1.6 times that for carbon steel. For certain special applications in long span bridges, it will be economical because of its high strength, but for short and medium spans, its price will preclude its use for some time to come.

It is likely that there will be a growing use of low-alloy steels for some years to come, but the trend would probably be accelerated if the mills could coordinate their operations so as to produce fewer grades. As it is, there are several grades which fail to meet the ASTM A-242 specification by small amounts, and which cause needless confusion when selecting a steel for a particular job.

Welding

There has probably been more controversy over the use of welding in bridge work during the past twenty years than over any other one thing. Until about twelve years ago, except for minor details, welding was prohibited by the AASHO specifications, although welding had been used in building work before that time, and a few welded bridges had been built.

Bridge engineers are inclined to be conservative, and many of them were skeptical about welding the ordinary carbon (A-7) steel. There was reason for this skepticism. The Lincoln Electric Company had, for many years, recommended that a good welding steel contain not more than 0.25 percent of carbon, nor more than 0.90 percent of manganese. A-7 steel has no specified limits for either of these elements and the carbon, especially, will often run several points above the limit recommended by the Lincoln Electric. Finally, about six years ago, a specification for a weldable steel was formulated and agreed upon by the interested agencies, including the American Welding Society, and the steel mills. This steel (ASTM A-373) is now generally accepted as a good welding steel, although the carbon content is only limited to 0.26 to 0.28 percent, somewhat above the Lincoln Electric recommendation. Its cost is from one-half cent to one cent per pound more than that of ordinary A-7 steel.

Until quite recently, many of the fabricating shops were not equipped to do welding on a major scale. There was little incentive for the bridge fabricating shops to so equip themselves as long as the AASHO specifications did not permit major welding. Now, however, this handicap has been largely overcome and most of the bridge shops are equipped to do first-class welding.

Welding inspection was pretty much of a rule-of-thumb operation until recently. The welding inspector needed to be a qualified welder himself and, even then, unless he could watch the welding operation continuously, there was no assurance that faulty welds, invisible on the surface of the completed work, would not occur. Radiographic methods of inspection have been developed, and are now in common use, which make it possible to inspect completed welds with confidence that hidden defects will be discovered.

Field welding is still a problem. The welder has to do his work under conditions much less favorable than those which obtain in the shop. Adequate inspection is also more difficult than it is in the shop. For this reason, many bridge engineers specify riveted or bolted field splices for welded girders. Some highway departments, whose programs are large enough to justify the expense, have set up their own welding engineering and inspection sections and are building completely welded bridges satisfactorily.

Welding of alloy steels can be done satisfactorily, but more care is required than for carbon steel. The Whiskey Creek bridge in California, which was mentioned earlier, is being built of three grades of steel. T-1 steel is being used for the most heavily stressed sections over the piers, low-alloy steel for the lower stressed sections around the quarter points, and A-373 steel for the positive moment areas in the middles of the spans. This arrangement makes it possible to keep a constant web depth and thickness throughout, and to keep the flange thickness nearly constant. The bridge is to be completely welded.

For short, simple-span, plate girders, comparative designs show that welded girders will weigh about 25 percent less than corresponding riveted girders, and will cost from 10 percent to 15 percent less than the riveted girders. It is to be expected that welding will continue to take a larger and larger share of bridge work for some time to come.

Fastening

While it has little to do with design, there has been a new development in field bolting during the past two years which may be of interest. We are all familiar with the extent to which high-tensile bolts have displaced rivets for field connections. The new bolt is a high-tensile bolt with a rivet instead of a bolt head. The shank is ribbed, with the ribs on a slight spiral around the body of the bolt. The ribs are knurled, so that the bolt is easier to drive than a Dardelet bolt. Tests have shown that, under fatigue loading, the new bolts provide higher joint strengths than the conventional bolts, since there is complete bearing around the bolt shank, while the usual high-tensile bolt depends upon the friction between the faying surfaces.

It seems reasonable to expect that, eventually, it will be possible to make field joints with fewer of the new bolts than would be required for either rivets or conventional bolts. If so, considerable economy can result from the reduced size of splices and connections.

THE USE OF ELECTRONIC COMPUTERS IN BRIDGE DESIGN

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The process of performing mathematical computations has been most stable from the earliest history of mathematics to about 1900. Since that year the process has advanced from long-hand and logarithm to slide rule, desk calculator, and now to electronic computers. The biggest change or advancement is this last transition from desk calculator to electronic computer.

The electronic computer which increases man's ability to do mental work, is said to be one of the three most important technological developments to be included in the history of our age. The other two are:

Nuclear energy, which increases the amount of energy to do man's work.

Automation, which increases man's ability to use tools and his work productivity.

Some believe that of these three, the computer will bring the greatest benefit to man.

Bridge engineers have been relatively slow to use electronic computers compared to accountants and research scientists. These two groups most definitely have exploited computing equipment in their fields of work. However, the electronic computer is becoming accessible to engineers and many bridge engineers are becoming familiar with this process of computation. In a relatively short time, the electronic computer has become standard equipment in most of the State highway departments of the country. Many consulting engineers and universities are now using the electronic computer for engineering computations.

There are two general categories of computers: analogue and digital. The analogue computer measures some quantity or physical condition which is analogous to some continuous function and then relates the measurement to the unknown function. A slide rule is an analog computer, it measures distances and relates that distance to a number. A thermometer is another form of analog computer, it measures a height of fluid in a tube and relates the measurement of fluid height to a unit of temperature. If the computer measures analogous electrical quantities it is called an electronic analogue computer. The principal uses of the electronic analogue computer are in industrial and research processes. The analogue computer is very fast, but the results or answers are approximate depending on the validity of the analogy and the accuracy of the measurement.

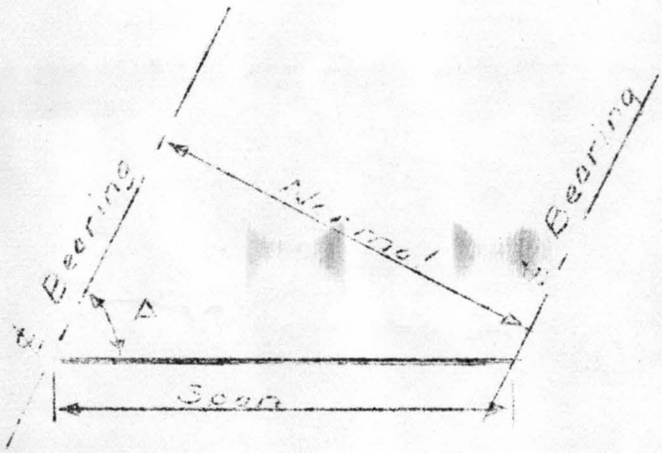
The digital computer as its name implies, works with digits. Long hand computation and desk calculators are forms of digital computers. If electronic circuity is employed to represent digits and perform arithmetic operations then the computer is called an electronic digital computer. A digital computer can solve any problem which can be expressed in mathematical form with practically any degree of precision desired.

Either type of electronic computer may be designed for special purpose or general usage. The general purpose electronic digital computer is the computer used in highway and bridge engineering computation. This type of computer is manufactured by a number of companies and is available in a variety of models. In size the range goes from a computer which occupies about five cubic feet of space to the large systems which may require about 3,000 square feet of floor space.

Computers are not yet "thinking machines", although they do have a surprising array of powers. They can "read", "write", do elementary arithmetic, compare, make yes or no choices and transfer information from one place to another. Computers can do these things far more accurately and much faster than human counterparts. The most important feature of a computer is its ability to carry out long sequences of arithmetic operations automatically. This is done through the use of a series of detailed step-by-step instructions for the computer to follow in solving the problem. The complete set of instructions for a particular type of problem is called a program.

The procedure followed by a computer operating under control or by direction of a program may be explained most clearly by comparison with a similar calculation carried out on a desk calculator. As an example, suppose that an engineer wishes to determine the span length between two skewed supports as shown in Figure 1. Given either by field measurement or by previous computations, would be the normal distance between the supports and the skew angle "A". The angle A is the angle which the center-line of support makes with the span dimension. The span length will be given by the formula presented in the figure.

This computation could be performed on a desk calculator for the engineer by another individual who is a specialist in computing. This "specialist" has very definite limitations in his abilities - he knows only how to operate a desk calculator, and to look up and transcribe numbers. In order that the



$$\text{Span} = \frac{\text{Normal}}{\text{Sin "A"}}$$

Figure 1

PROGRAM FOR SOLUTION OF BRIDGE SPAN LENGTH

Program

1. From data sheet write normal distance in space 1 on work sheet.
2. From data sheet write angle "A" in space 2 on work sheet.
3. In trigonometric function book look up sine of angle corresponding to number in space 2 and record sine function in space 3.
4. On desk calculator divide the number in space 1 by the number in space 3 and record the quotient in space 4.
5. From space 4 of work sheet write answer as span on original data sheet and give data sheet back to the engineer.

Work Sheet

1	44.00
2	30°-00
3	.5000
4	88.00

Data Sheet

Normal Distance	= 44.00
Angle A	= 30°-00
Span	=
Engineer:	

Figure 2

computing specialist can carry out this required calculation, it is necessary for the engineer to write out a program or complete set of instructions for all of the operations the specialist is to perform. Also, the engineer must supply data and a work sheet on which numbers can be recorded. The program and work sheet might look as shown in Figure 2.

The procedure followed to solve the problem in the above example is exactly the procedure which would be followed in an automatic computer solution. The limitations which were imposed on the computing specialist are analogous to the limitations of an automatic computer.

In order that this simple calculation might be performed by an automatic computer the engineer first would have to write out a series of detailed step-by-step instructions in a coded digit form which the computer could understand and feed them into the machine, usually by means of code punched cards or tape. The next step would be to prepare the basic data pertaining to the problem to be fed also into the machine and at the proper time in a manner similar to the means used for feeding the program to the machine. Finally, depressing the start button on the computer would cause the machine to execute the first instruction of the program and subsequently go through the same operations as done by the computer specialist and described before, but at a very much greater speed. The machine program which is the equivalent to the specialist's program, might appear somewhat as shown in Figure 3.

MACHINE PROGRAM

1. Read "NORM" into storage cell 1.

2. Read "A" into storage cell 2.

3. Use number in storage cell 2 as the independent function for a sub-program "SIN" and store the result in storage cell 3. First instruction of sub-program "SIN" is located in storage cell 100.

4. Transfer number from storage cell 1 to the arithmetic unit.

5. Divide the number in the arithmetic unit by the number in storage cell 3.

6. Transfer number from arithmetic unit to storage cell 4.

7. Print out number in storage cell 4.

8. Stop.

Figure 3

This example program demonstrates the basic characteristics of an automatic digital computer, namely; a facility to read, ability to transfer information from one place to anohher, ability to perform elementary arithmetic and a facility to write out results or answers. There is one other characteristic of the automatic computer which was previously mentioned but not demonstrated in this example, and that is an ability to select one of two choices based on a comparison of two records. The computer can make a choice by examining a result of some arithmetic operation or transfer operation for a negative or positive condition, or a test for a zero or non-zero condition, and then follow a path or certain sequence of instructions depending on the test condition.

The above example program, however, does not indicate the complexity of the problem involved in programming for the electronic digital computer. The preparation of the computer program is the principal restriction to widespread usage of electronic digital computers in bridge design work. To develop and check out a computer program for a complex structural design problem may take up to several months of concentrated effort.

The preparation of a program for any particular type of problem includes everything that is necessary to bring about a solution to the problem. The problem must be specifically defined, the method of solution selected and outlined, and the mathematical equations that are involved in the method of solution must be formulated. Then these mathematical equations must be reduced to basic arithmetic operations since the computer is limited to perform only addition, subtraction, multiplication and division.

The next step in the development of a computer program is to construct a detailed flow chart showing each of the arithmetic operations and logical decisions or choices to be made during the solution of the problem in proper relationship to all the others. The operations shown in the flow chart are then coded in the code language used by the particular computer on which the problem is to be solved.

The programming task through the construction of the detailed flow chart is best done by the engineer who is thoroutly familiar with the problem and the method of solution. The coding and checking out the program can be done by the engineer although this is not necessary. Usually, program coding work is done by a person who is thoroughly trained in the coding techniques and the operation of the computer to be used. Much work is being done by computer manufacturers along lines to have the computer itself do the coding of the program, and ultimately this part of the programming task will be a machine operation.

Computer programs that have been developed in the bridge engineering field cover a wide range of applications, covering both geometric and structural design. Some of them are quite simple, others are more complex. The geometric programs are basically problems in trigonometry. Nevertheless they can be quite complex and may involve a large amount of computations as in the case of multispan skewed bridges located on curved alignment. Problems covered by structural design programs range from the determination of internal stresses in a reinforced concrete column subjected to biaxial bending to the complete stress analysis for various external loading conditions on fixed arches. A number of programs have been developed for continuous beam bridge design. These programs include the computations of deflections, design constants, and influence line ordinates for moments and shears.

There is one program that will completely design a concrete-steel composite beam, a type of beam commonly used in bridge construction. In this type of structure, the concrete slab forming the roadway of the bridge and the steel beams supporting the slab are designed to act as a unit in carrying the loads which are applied on the bridge. Using this program, the computer determines the size of a standard rolled steel beam section and cover plate details as required for a particular design condition. The analysis and design by the program is made in accordance with the latest American Association of State Highway Officials Bridge Design Specifications. Included in this program which is fed into the storage facility of the computer, is detailed numerical information pertaining to 16 different steel rolled beam sections, the 16 most commonly used beams in bridge construction. These beams range in size from a 24 inch deep beam weighing 76 pounds per foot to a beam 36 inches in depth and weighing 280 pounds per foot. This steel beam information included with the program instructions relates to beam weight, depth, material thicknesses, cross sectional area, moment of inertia, etc. for each beam. All of this information was taken directly from a steel beam design handbook. This basic information for each beam is arranged in the storage unit of the computer according to the beam size by weight, much like a steel beam design handbook. For a given design condition, the computer will automatically select the smallest beam (24 WF 76) as an initial design choice. It will compute all of the structural properties of the composite section such as: area, moment of inertia, and section of moduli for various conditions of the concrete slab. It will then determine the design moments, dead load, superimposed dead load and live load. The live loading on the beam will consist of standard trucks or of lane loads which are given in AASHO Bridge Design Specifications. Two systems of loading are provided in the program, the H loadings and the H-S loadings. Stresses are calculated in the composite section corresponding to the bending moments. The calculated stress is automatically compared with allowable stresses for concrete and steel, if any of the beam material is overstressed, then a minimum size cover plate, 3/8-inches thick, is automatically considered attached to the bottom flange of the basic steel beam. The structural properties are then recomputed. Also recomputed are the bending moments and the internal working stresses of the composite section. If the beam is still overstressed, instructions in the program will automatically increase the cover plate thickness by 1/8-inch and again analyze the new beam section and check the computed stresses with the allowable stresses. An overstress condition at this point will cause the program to instruct the computer to increase the thickness of the cover plate by 1/8-inch increments up to a limiting maximum based on the flange thickness of the basic steel beam section involved or considered in the composite section. When the thickness of the cover plate exceeds the thickness of the beam flange section by 1/4 of an inch, the next larger size steel beam in computer storage is selected as the basic steel beam component in the composite section and the design analysis cycle is started again from the beginning. When a satisfactory stress condition is attained, the computer, guided by the program, will complete the composite beam design. It will compute deflections, determine the length of cover plate where one is required and determine the spacing of shear connectors at various points along the beam. At the conclusion of one design, if that particular design requires a cover plate attached to the steel beam section then the computer will automatically make a second complete design selecting the next larger steel beam section in storage, if available, than was used in the previous design. If the second design also requires a cover plate, a third design will be made by the computer selecting a still larger steel beam section, if available, than the previous design for an additional composite beam design. If the third design requires a cover plate and the beam section selected is not the largest steel beam section contained in the beam table in storage, the computer will discard the last design and make another design using a larger beam section than the previous one. It will automatically attempt to make one design where no cover plate is required on the bottom flange. All designs are tabulated in order by the weight of steel required in the design. Included in the tabulation will be the weight of steel on a lineal basis; the beam size, given by depth and weight; cover plate details, size and length; values of moments of inertia for checking purposes; design moments and their corresponding stresses; mid-span deflections; and shear connector spacings. Three complete designs are possible for a single design condition depending upon whether or not cover plates are required in the first or second design.

To illustrate the speed at which an electronic digital computer is able to perform arithmetic computations, a medium sized computer using this program is able to make a complete composite beam design for a bridge having a 70-foot span, loaded with the standard AASHO H-20 S-16 truck in seven or eight minutes. This design condition probably would require the computer to make the maximum of three complete designs because cover plates would be involved in the final designs.

As mentioned previously, nearly all of the State highway departments and a large number of bridge engineering consultants are using electronic computers in their routine engineering work. In addition, the Bureau of Public Roads, the U. S. Forest Service and many colleges and universities use computers on engineering work. All of these organizations have been and are actively engaged in the development of computer programs to assist in bridge design and related fields. In order to minimize duplication, users of the same commercial make of computer have joined together in user groups, and within these groups there is free exchange of completed programs. These groups meet periodically to discuss common problems of users concerned with computer operations and plan future program development work.

The direct exchange of computer programs between users of different makes or models of computers is not practical because each make or model of computer uses a unique machine language or code. There are at least a dozen different makes or models of electronic digital computers in daily use doing computational work in the bridge engineering field.

To overcome this difficulty of program exchange between users of different computers, the Federal Highway Administrator has established a computer program library in the Division of Development of the Bureau of Public Roads. This library serves as a central point for the receipt and distribution of computer programs developed for use in the highway and bridge fields. Programs received from various computer users for the program library are converted to a library form, a form in which the program is expressed in English and mathematical terms representing the sequence of operations involved in the solution of the problem so that it can be readily coded for use with any computer. In addition, Library programs are developed within the Division of Development of the Bureau of Public Roads, either directly or through cooperative projects with State highway departments and educational institutions or research organizations. All programs in library form are made available to the State highway departments and other computer users. About 325 electronic computer programs have been received to date for the library. The Bureau of Public Roads program library supplements the several computer user groups and makes possible complete interchange of programs regardless of the make or model of computer used. A library memorandum is issued periodically to provide information on programs received and those available in library form. The most recent library memorandum, number 7 and dated February 1960, lists 321 programs. Of the total number listed, 135 programs are in the field of highway location and design including earthwork computations, 115 are in the field of structural design, 13 in traffic, 5 in soils, 6 in hydraulics, 10 in administrative areas, and 37 on miscellaneous subjects. Twenty-seven of these programs have been converted to library form and a number of other programs are in the process of being converted. Ultimately the program library will include programs covering all the common problems in highway and bridge engineering. Some of the programs received by the library pertaining to bridge engineering are listed in the appendix.

The general tendency in developing a computer program for a structural engineering problem has been to follow the analytical procedures which have been developed in the past few decades for efficient manual solutions. Some of these procedures include crude approximations, short cuts or rule of thumb methods which may not be equally efficient in a machine solution as in manual solutions. Because the computer does mathematical operations so rapidly, length and complexity of computations are not important in an analytical solution.

The computer offers a completely new approach to calculation. It also offers an opportunity for a return to basic design principles and opens the way for the development of new or different methods of analyzing bridge or structural design problems. The electronic digital computer is a powerful tool for the bridge engineer and will become even more powerful in the future. It is important to bear in mind, however, that it is only a calculating machine. The thinking that is necessary in the solution of an engineering problem must be done by the engineer.

The purpose of this paper has been to indicate briefly how an electronic digital computer works and how it may be used in bridge design. In conclusion, some of the advantages to be gained by bridge engineers through the use of the computer will be reviewed.

Even though automatic computing devices are expensive, there is an economical advantage to be gained in the cost of performing computation. Calculations done by automatic computers quite often cost less than the same work done by manual methods. The high speed of the computer makes this economy possible. The American Bridge Division of the United States Steel Corporation which is using an electronic computer for machine solutions of many bridge engineering problems reports that the use of the electronic computer saves 75 percent of the cost of equivalent calculations performed manually.

A second advantage in the use of computers is increased productivity of the engineers working with them. The use of the computer relieves the engineer of much routine computations and practically doubles the time that the engineer can devote to truly professional level work. Moreover, the reliability of the machine calculations further increases his productivity by eliminating check calculations. Because of the checking procedures incorporated into the program for engineering problems the results are considered as reliable as the input data to the program.

A third advantage or benefit in the use of the computer is that it will enable the engineer to solve many problems that heretofore he could not have solved chiefly because of the complexities in these problems. This benefit has not yet been fully realized by bridge engineers, but in the near future engineers will be using the computer to gain this advantage.

A fourth benefit through the use of the computer in bridge design is that it is able to produce more economical designs leading to lower construction costs. The use of the computer in speeding up engineering computations will provide more time for economic comparison of alternate designs and alternate materials. It is estimated that structural requirements for the Nation's highways will cost in excess of $30 billion. About 375,000 bridges are needed. A saving of only one percent on the Nation's bridge requirements would amount to $300 million. Such savings are possible through more extensive evaluations of alternate designs, particularly on large structures.

The Texas Highway Department reports the following:

> "Savings in engineering costs and construction costs are being realized by the use of the IBM 650 computer in designing bridges. The Houston Urban Office has been utilizing the computer located in the Austin headquarters on continuous-beam designs problems. By so doing they have been able to refine their designs to an extent not normally possible when using engineering manpower to make the time-consuming computation.

The computer was used in designing the top structure for a four-level interchange in Houston carrying IH 610 over the Southwest Freeway (US59) and other connecting roadways. The State estimated that design refinements on this particular structure resulted in construction savings of approximately 100,000 pounds of structural steel. Savings in design costs were estimated at 33-1/3 percent."

The computer's use in bridge design is also valuable in checking bridge designs for obtaining maximum economy. Recently a highway department was observed checking a bridge design which was submitted by an outside firm. The specified beams in the proposed bridge superstructure appeared to be unnecessarily heavy. Formerly, because of the lack of design engineers, the highway department was not in a position to check the detailed computations and would in a case of this kind send the plans back to the firm for further investigation. The State now has an electronic computer and in this case, an independent design computation was made in about ten minutes which revealed that a considerably lighter beam would be adequate for the design loads. This one item involved a difference in cost of $13,000.

The benefits thus far gained for bridge engineers through the use of the electronic computer in bridge design indicate that there are possible future advantages to be gained through continued use. Each successful application of the computer points or leads to another application. The computer possibilities in bridge design seem to be limited only by our ingenuity in adapting them to our needs.

Appendix

The following is a partial list of programs pertaining to bridge design which are included in the Bureau of Public Roads Electronic Computer Program Library.

1. Continuous Steel Beam Bridge Design
2. Computation of Continuous Beam Characteristics
3. Geometric of Skewed Bridges on Circular Curves
4. Analysis of Rectangular Reinforced Concrete Columns
5. Concrete-Steel Composite Beam Analysis and Design
6. Moment Distribution and Influence Line Calculations
7. Analysis of Fixed Arches, Frames and Rings
8. Bridge Pier Analysis
9. Prestressed Concrete Girder Design
10. Beam Deflections
11. Reinforced Concrete Box Culvert Design
12. Circular Reinforced Concrete Column Analysis
13. Five Span Continuous Bridge Analysis
14. Composite Welded Steel Girder Analysis
15. Dead Load Plus Live Load Bending Moments
16. Retaining Wall Design
17. Pier Cap Design
18. Investigation of Reinforced Concrete Beam
19. Computation of Pile Loads
20. Steel Reinforcing Bar Schedule Computation

STREAMLINING OF BRIDGE SUBSTRUCTURE

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Introduction

Bridge substructure, serving two independent purposes, is designed structurally to support a deck and hydraulically to discharge a flood. This paper is concerned only with the hydraulic design.

Hydraulic design must dispose of four hazards, viz: (1) underscour of foundations for piers and abutments, (2) erosion around and behind abutments, (3) large static and dynamic forces acting on the substructure (and possibly the superstructure), and (4) complications caused by detrital flow. The last is too often overlooked or under-estimated, with the result that most of our catastrophic failures are charged to entrapment or drift on the bridge or fluidity of granular alluvial materials.

Until recently, the conventional bridge was located normal to the stream, regardless of angle of approach highway. Being narrow and designed for light loads, it was cheaper than high approach embankments, so that channels were seldom constricted at abutments. Piers were traditionally massive to buck currents, ice and drift. Only on the greatest bridges were piers protected by sterlings or cutwaters.

Modern highway demands have changed all this. Alignments are sequences of long tangents and flat curves controlled remotely from stream crossings. Bridges are located simply by intersecting these long lines with each channel enroute, regardless of large skews and horizontal curvature imposed on structures. Deck widths have grown with width and number of lanes. Live loads have grown enormously. Divided highways have spawned parallel bridges. Automation has demanded modular design. Substructure has had to satisfy these impositions without damming the underlying channels.

Channel Impairment

The need for streamlining is forcefully demonstrated by a review of deficiencies contributing to channel impairment. These range from size, shape and spacing of substructure units to lack of transitions from natural to constricted channel.

Notorious is the massive pier with a plane upstream face, often supported on a still broader base which is exposed by scour during floods. Such piers collect drift, adding to the effective obstruction of the channel. Currents are deflected sharply, generating turbulence with downward components that impinge on and scour the bed. Modern demands would worsen the obstruction because of longer piers for wider bridges, skew alignment and twin piers for parallel bridges.

Skew impairment may follow from skew alignment of the bridge on skew currents in irregular channels. Plate A shows a scour hole 120 ft in diameter and 15 ft deep around Pier 21 of the bridge over Feather River on US 99 carved by skew currents in an overflow area adjoining a main channel. Heavy drift or pile bents add to distress caused by skew.

Underestimate of drift leads to false economies and consequent impairment. Short spans often prove eloquently just how long drift really is. As soon as one spar straddles two piers or strands on one wide one, a drift jam begins to collect. The mass of such material carried barely awash is seldom realized until intercepted (Plate B). Since most of our washouts are charged to drift, this factor is of utmost importance.

Column bents impair channels more than piers because of the greater number of obstructive elements creating turbulence and the opportunity for entrapment of drift between colums of the bents. Designs based only on structural economy produce a veritable forest of piles - a forest which may almost become "impenetrable" if the stream meanders to a diagonal course under the bridge (Plate C). For framed bents, scour often exposes pedestals much thicker and more obstructive than the frames.

Impairment of vertical clearance becomes more common with demand for sight distance on the highway, eliminating "humped" grades at bridges. The effect of contact of a moving water surface with bridge superstructure can be expressed by reference to vertical velocity curves as a deceleration at the surface and an acceleration of the strength (thread of maximum velocity). The strength is deflected downward as well, generating a current which impinges on and scours the bed somewhere downstream. This effect is pronounced if drift is intercepted by the superstructure.

A comparatively new form of impairment has been observed in recent construction of pier bases or pile footings below grade. Conventionally such footings were constructed in cofferdams or neat excavations, with little disturbance of the rest of the bed. Contractors have found it economical to excavate one big glory hole encompassing a series of footings, controlling underflow with well points around the perimeter. Backfill of such holes is less resistant to scour than the natural bed. We are currently studying the alternatives of restricting such excavation or specifying a more resistant backfill.

Classical Streamlining

The classical concession to streamlining is the "cutwater," although its primary function was often protection of the pier from pressure by ice or impact of vessels. Sterlings served the same purpose without much contribution to streamlining. As the name implied, cutwaters were pointed nosings built with or superimposed on the upstream face of the pier. The leading edge was generally vertical or battered like the face, but a few were given a flatter slope for special purposes.

Streamlining of bents was generally added when need was demonstrated by pile-up of drift in a flood. The familiar cylinder piers were joined by a partition or web wall to make the equally familiar dumbbell pier. Timber pile and framed bents were sheathed with plank. Filler walls were cast of concrete to join steel or concrete piling. Hydraulic efficiency was greatly increased by such expedients.

Shaping of piers was infrequent and empirical until Nagler's work in 1914(1). Although his backwater formula was questioned, his measurement of relative efficiency of pier shapes still provides a valuable design tool. Separating his combinations, the coefficients of discharge for pier noses and tails have been tabulated:

	Coefficient of discharge for shape as a:		
Pier-end Shape	Nose	Tail	Both
Bullet	.963	.969	.934
Thick-fish	.952	.974	.928
Half-round	.957	.964	.923
Thin-fish	.955	.965	.922
45° cutwater	.953	.963	.916
90° cutwater	.935	.957	.893
Square end	.900	.950	.861

Hence the bullet-nose fish-tail pier was an ideal shape. For symmetry, the bullet shape could be used at both ends. The fish-tail shape is not as practical for a second choice

(1) Obstruction of Bridge Piers to the Flow of Water, F. A. Nagler, ASCE Trans. Vol. LXXXII, p. 334.

as the popular half-round, which is nearly as good. Later investigations by others of flow at higher velocity confirmed the bullet-nose and found an elongated bullet-tail nearly equal to and more practical than the fish-tail.

Modern Streamlining

2-Column Bent - The greatest gain in modern streamlining has been in the field of greatest need - multiple short-span bridges for which pile column bents had been most economical. The most obvious improvement was a reduction in number of columns to two. Heavier columns and a stronger cap were required, but rigid framing minimized the additional materials.

1-Column Bent - More surprising was the further reduction of the bent to a single column. Not only does the single-column bent reduce the number of obstructions to stream flow, but its axis can depart moderately from the axis of the cap so as to align the column with the flow and its cap with the superstructure frame. The combination is particularly adapted to curvilinear skew crossings (Plate D).

Thin Pier - For normal crossings, the thin pier is cheaper than the single-column bent, and very practical when longitudinal forces can be carried by superstructure to an abutment. The hydraulic efficiency is obvious in situ and in formula. Only for very high velocity is it necessary to shape the nose and tail (Plate E). Parallel bridges on straight channels can be built with piers in parallel without much additional resistance to flow.

Partitions - A series of obstructions along a streamline add their contributions to backwater. When this is objectionable, or if drift may be trapped between successive obstructions, a partition or web wall is a logical improvement. The general idea is not new, but application to parallel bridges has proved an interesting extension of the practice.

For example, thin piers can be joined with a web wall framed to both without intermediate support, or cantilevered from one if there is a hazard of differential settlement of footings. In several cases new thin piers have been connected to old filled-in multi-column bents.

Partitions are nearly indispensable if parallel bridges cross a bending channel, or if lodgement of heavy drift is a hazard (Plate F).

Sloping Cutwaters - For drift-laden streams, sloping cutwaters have proved very effective. Drift is heaviest on the rising stage just before the peak. If waterlogged drift lodges crosswise on a vertical pier or pile, friction between log and pier exceeds the small residual buoyancy so that water rises above it. Successive lodgements build a drift jam, deflecting water laterally and downward with a probability of scour from the downward current.

However, if the leading edge of the pier is a sloping cutwater or "fin", the pressure of water on the drift assists the residual buoyancy in lifting the drift as the stage rises. This serves three favorable objectives, (1) the obstruction below water surface is reduced, (2) the lifting and translation of the drift may upset its balance until it dislodges, and (3) following drift will not override and jam the stranded drift, but deflect with the lateral current around the drift.

Proper slope of cutwater is a matter of opinion, as well as objective. California practice favors a 1:1 slope for two reasons: (1) economy, in that its shortness seldom requires additional foundation, and (2) self-maintenance, in that drift slides down on the falling stage and some loses balance and floats away (Plate G). Other agencies use slopes as flat as 2:1, presumably to increase assurance of lifting the drift, but certainly at much extra cost for both fin and foundation.

Warped Wings - For culverts as well as closed-abutment bridges, the approach and retreat of a stream follow transitions from trapezoidal to rectangular and back to trapezoidal section. Unless streamlined, the transitions may be very vulnerable to erosion. The warped wingwall serves this purpose admirably and, being a ruled surface, can be formed with straight centering in one direction. If length is carefully proportioned to velocity, drift will follow the wing in proper attitude and both scour and backwater are controlled (Plate H).

Varied Skew - Another expedient for curved-skew bridges is shown on Fig. 3. Modular units of superstructure are developed by uniform deflection of pier axes from curve radii. This varies the skew of piers to currents, the greatest skew being set in the slowest flow. Curvature has been exaggerated to emphasize the principle; for an actual bridge over Sacramento River, the skew was small and had little effect on streamlined thin piers (Plate I).

Closure

The important streamlining techniques are expensive, so that the hydraulic engineer is frequently in conflict with economy-minded structural engineers. In such conflict, it is significant that the structural engineer has precise design specifications and the hydraulic engineer relies on qualitative factors. Thus, if the hydraulic engineer recommends 80-ft spans and the structural engineer determines 50 ft as the economical spacing of piers, he may ask if spans can be cut to 60 ft, or even 70 or 75. There is a temptation to compromise to avoid replying, "I think 80 is right, but I can't prove it."

Use of various types of pilihg is another invitation to conflict. Fig. 4 shows schematically the relative turbulance around round, square and H piles and the comparative difficulty of streamlining the bents, - during construction or after the first flood. When the hydraulic engineer recommends a thin-wall pier and the structural engineer demonstrates the relative economy of pile bents and then resists adding sheathing or filler walls, there is no reliable rule or formula for insisting on streamlining.

Use of the word "designer" was avoided because to many it connotes the structural engineer alone, disregarding the earlier and more general design of the waterway by the hydraulic engineer. Under ideal conditions these two designers should work together, or, better yet, the designer should be a hybrid engineer, deriving the HY from HY-draulic and the BRID from BRIDge.

Acknowledgments

The pictorial examples and preparation of slides for the oral presentation were selected and produced by W. F. Johnson, M.ASCE, Senior Bridge Engineer, on the staff of the writer in the Special Studies Section of the California Division of Highways.

SCOUR DUE TO IMPAIRED CLEARANCE

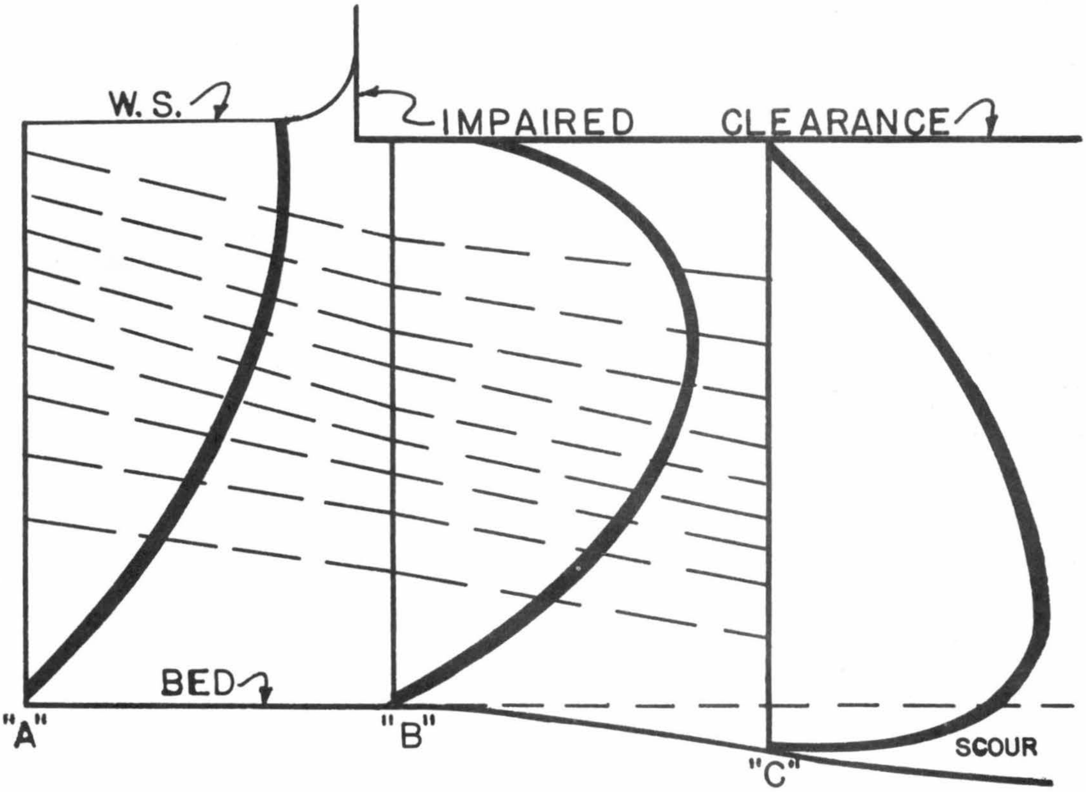


FIG. I. SCHEMATIC DIVISION OF VERTICAL VELOCITY CURVES AT SECTIONS "A", "B" & "C" INTO DECILES OF DISCHARGE TO SHOW DOWNWARD DEFLECTION OF CURRENT AT "B", WITH MOMENTUM IMPINGING THE CURRENT ON THE BED AT "C", CAUSING SCOUR.

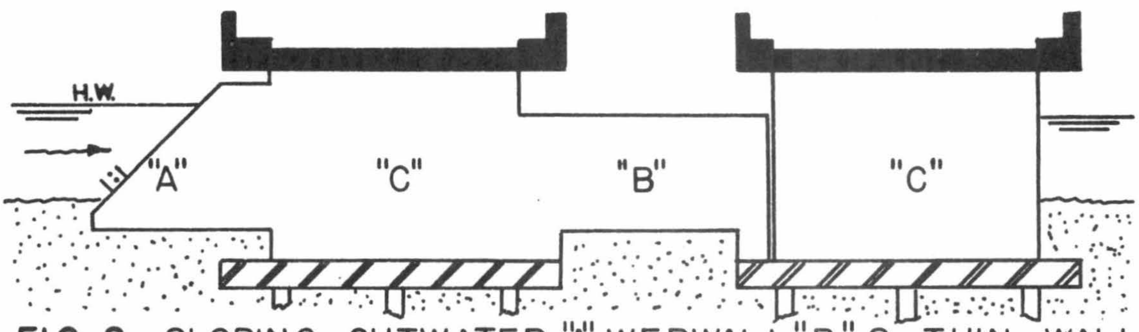


FIG. 2. SLOPING CUTWATER "A", WEBWALL "B" & THIN-WALL PIERS "C" USED TO STREAMLINE PARALLEL BRIDGES.

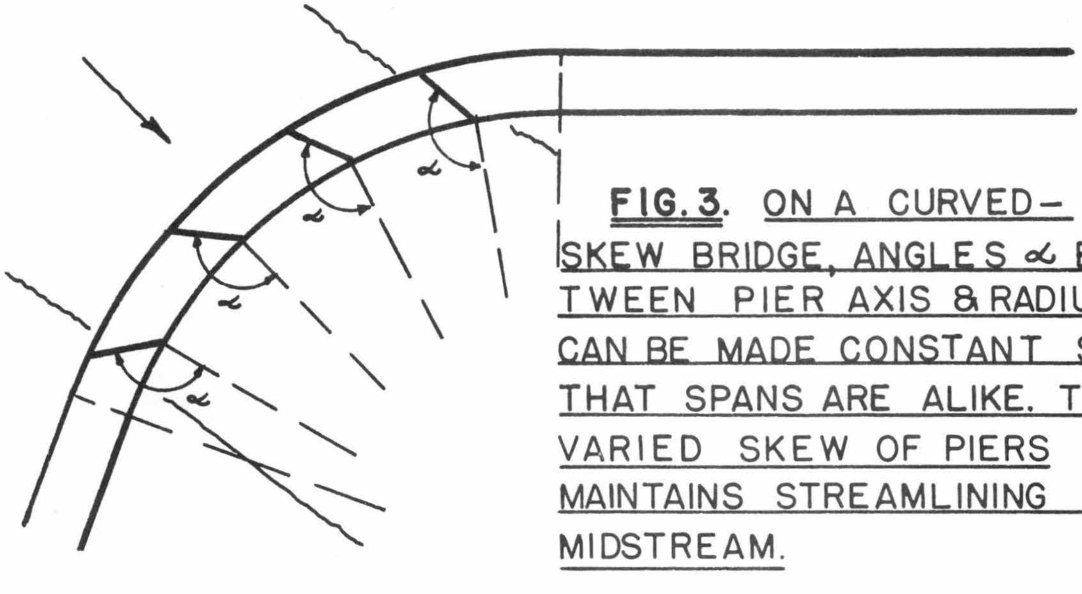


FIG. 3. ON A CURVED-SKEW BRIDGE, ANGLES α BETWEEN PIER AXIS & RADIUS CAN BE MADE CONSTANT SO THAT SPANS ARE ALIKE. THE VARIED SKEW OF PIERS MAINTAINS STREAMLINING IN MIDSTREAM.

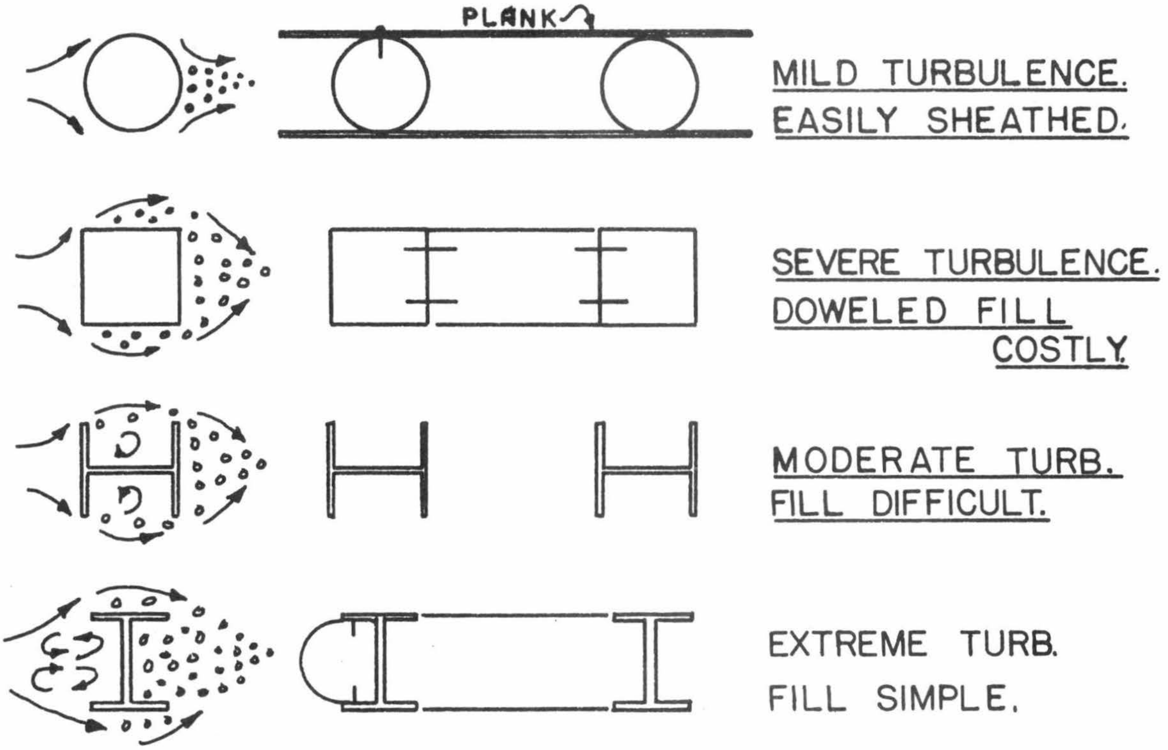
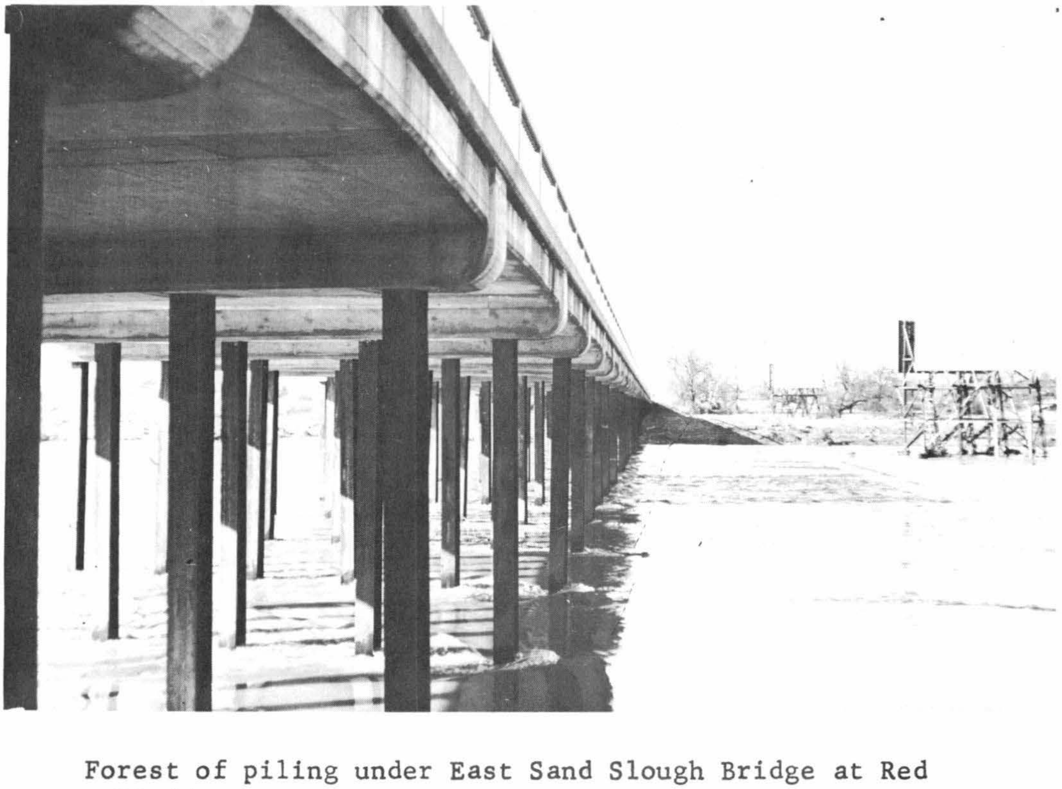


FIG. 4. RELATIVE TURBULENCE & COMPARATIVE COST OF STREAMLINING PILE BENTS.

PLATE A

Scour around Pier 21 of Feather River Bridge at Yuba City due to cross current in overflow channel.

PLATE C

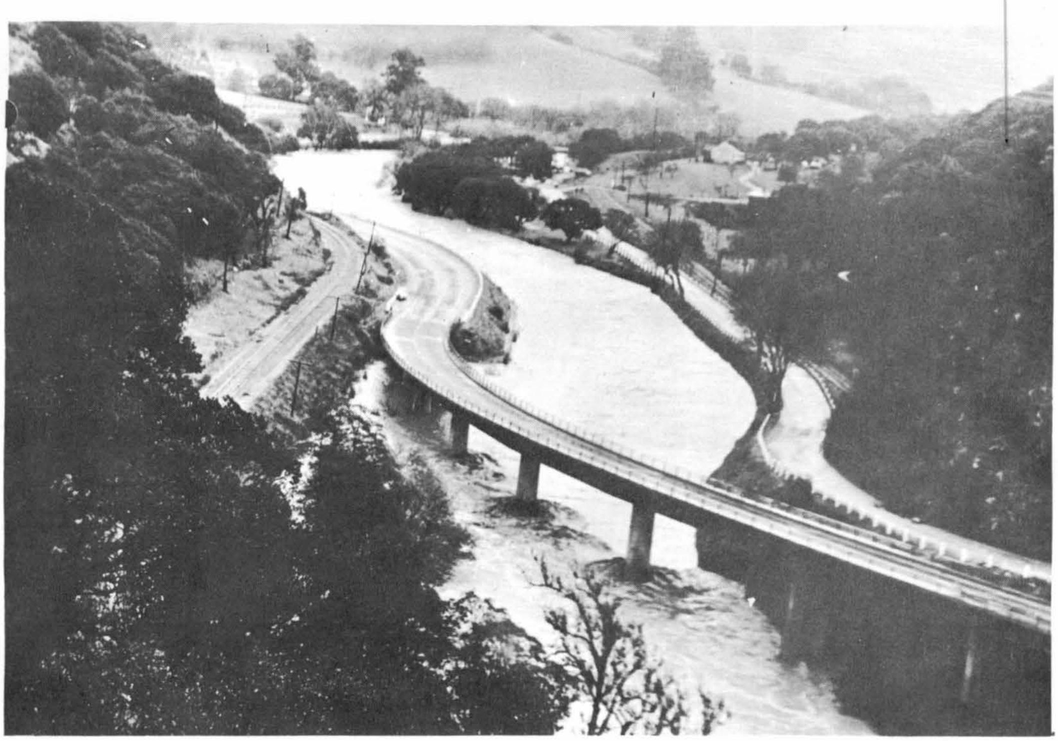


Forest of piling under East Sand Slough Bridge at Red Bluff.

PLATE B

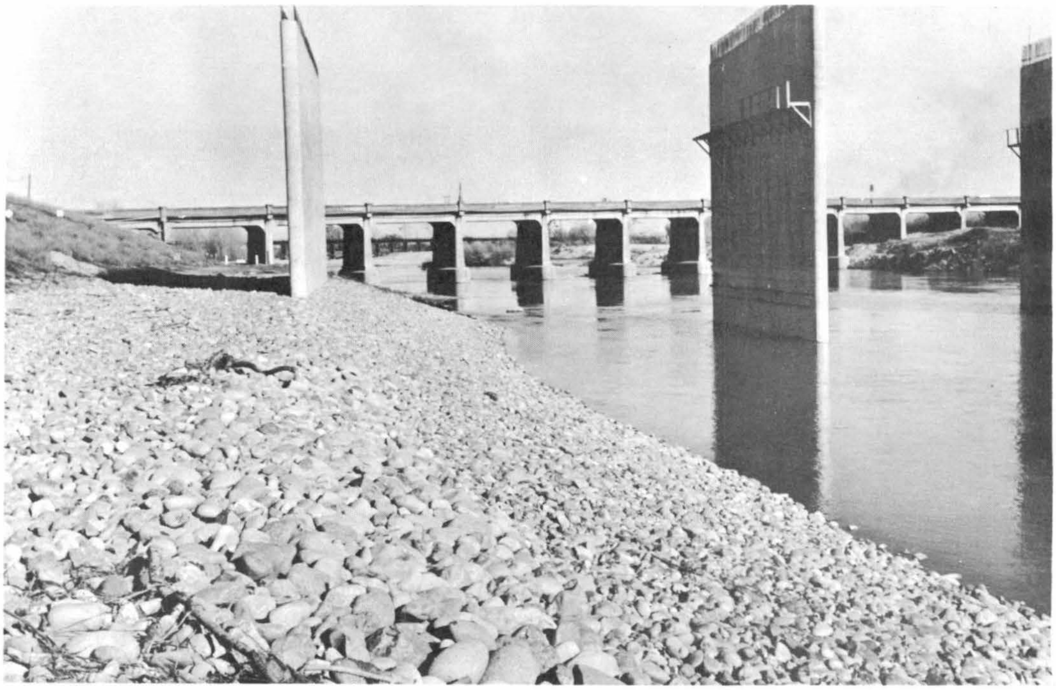
Drift on abutment pier of Mad River Bridge near Arcata;
pile-trestle approach destroyed by drift.

PLATE D



Single-column bents streamline the skew-curved Alameda Creek Bridge near Fremont

PLATE E

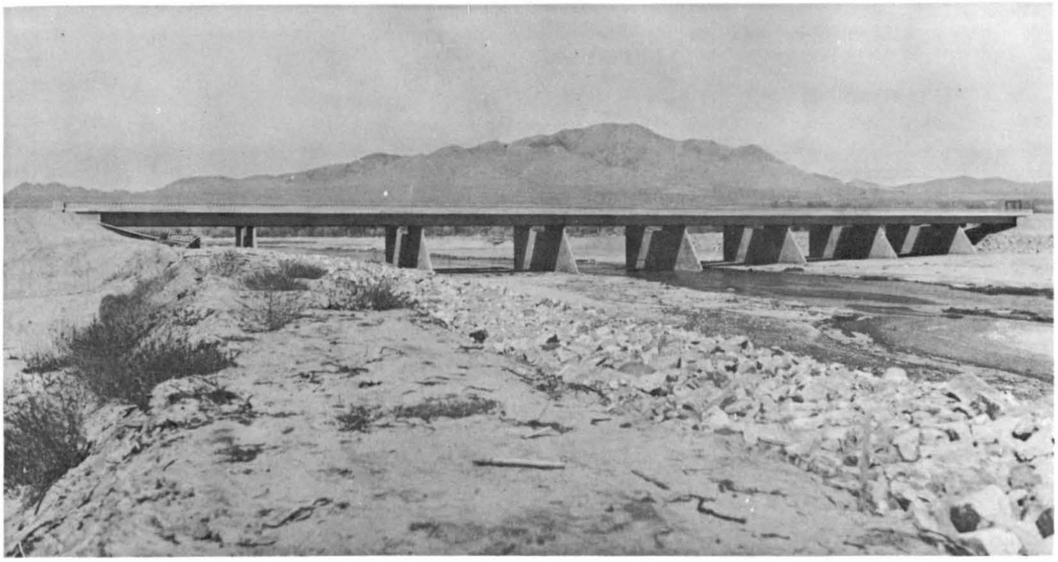


New thin-wall piers of Yuba River Bridge at Marysville compared to massive piers of old bridge

PLATE F

Web walls between parallel bridges over San Francisquito Creek at Palo Alto

PLATE G

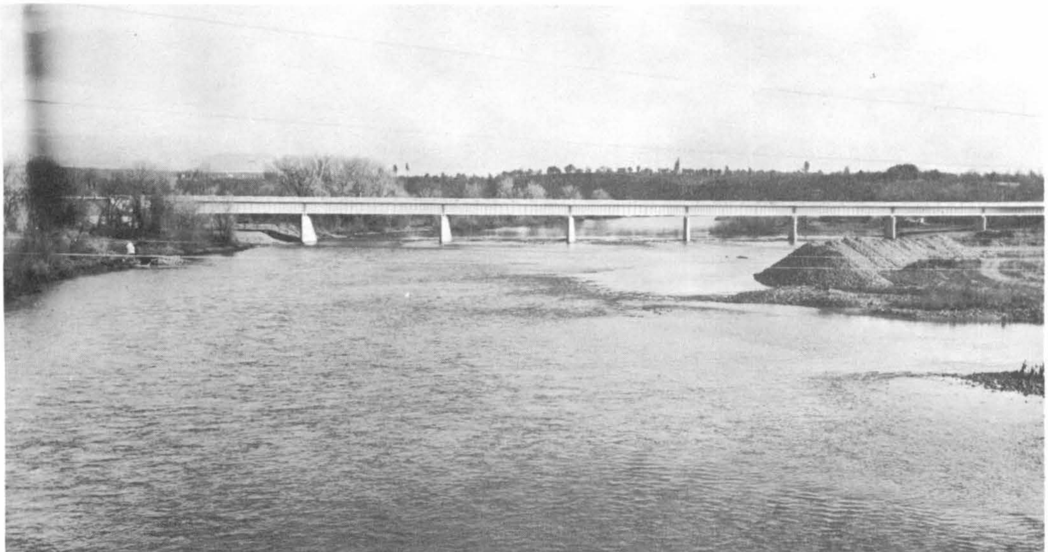


Web walls and sloping cutwaters streamline parallel bridges over Mojave River at Victorville.

PLATE H

Stream lining of double-box culvert with warped wingwalls and half-round nosing.

PLATE I



Varied-skew piers on skew-curved bridge over Sacramento River at Redding (see Fig. 3).

THE DESIGN AND CONSTRUCTION OF CONCRETE BRIDGES

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The design and construction of concrete bridges might well be the subject of an entire seminar. I will attempt to touch briefly on some of the most important points.

The Bureau of Public Roads has estimated that the structural requirements for the nation, both on and off the Federal Aid System, will cost in excess of 32 billion dollars. About 375,000 structures are needed. This includes over 45,000 on the Interstate System, over 130,000 on the remaining Federal Aid Systems, and nearly 200,000 off the Federal Aid Systems. Approximately 90 percent of these will have spans of less than 100 feet, which indicates the selection of concrete construction. The appearance, adaptability, low first cost, and long time economy of continuous concrete bridges has resulted in their acceptance, as first choice, by many of the state highway departments.

One of the major problems has been the lack of readily applied design procedures for continuous bridges, particularly those with a variable moment of inertia. Design methods have been developed by the Portland Cement Association, with charts and formulas for the use of designers. Graphical methods have been used by some designers. In our office we have developed a method of the design of continuous bridges by the use of coefficients.

(a) The design coefficients are computed for commonly used span ratios, such as 1 - 1.25 - 1.25 - 1, for a four-span structure. We have found that coefficients can be used with sufficient accuracy for three, four, five, or any number of spans.

(b) A unit moment is applied at one side of the center pier, and distributed. This procedure is repeated at the first pier. This gives a series of moment distribution coefficients from which the final moments at the piers are computed for fixed end moments. (Values of fixed end moments are available from tables of properties of haunched beams.)

(c) Each span is loaded with a unit uniform load, and the final moments at the piers are tabulated. Each span is loaded with a unit concentrated load at each of the tenth points in span one and span two. The final moments at the piers are tabulated.

(d) A unit concentrated load is placed at each of the tenth points in span 1, and the moments and shears at each tenth point are tabulated. These values are actually ordinates for influence lines for moments and shears at the tenth points in span 1. The same procedure is repeated for span 2.

(e) To use the design coefficients it is only necessary to multiply the coefficients for moment and shear by wl^2 or Pl for moments and by wl or P for shear, to obtain actual moments and shears.

(f) Steel area diagrams are drawn and the cutoff points for reinforcing steel are shown.

I would like to make this explanation with reference to the design of continuous concrete structures. It has been my experience that in general most designers go into too much refinement. We have found that for structures of 400 feet total length, no provision is necessary for rockers or other types of expansion devices. Steel pile abutments are used, and in the design of the superstructure it is assumed that the dock is simply supported at the abutments and piers. The piers are integral with the deck, and in the design of the substructure, the effects of shrinkage, temperature, longitudinal forces, lateral forces, and in the case of bridges on a curve, centrifugal forces are taken into account. Probably some economies could be achieved by reduction in moments in the superstructure, by including the stiffness of the piers in the design. But it is my opinion that the most economical structure is the one that has been thoroughly studied with respect to the construction methods.

Various types of continuous concrete bridges are available depending on span lengths. Economical span ratios range from 1 - 1.25 to 1 - 1.4 depending on haunch ratio.

(a) Continuous concrete slab bridges are economical for spans of approximately 50 feet. Common spans are 30-40-30 and 36-48-36. This type of structure has been bid as low as $7.20 per square foot.

(b) Continuous concrete girder bridges with either two or four girders are economical for spans of approximately 100 feet for the center span. In fact the two-girder type is very economical, the price for a recent 44-55-44 structure was $5.32 per square foot, and for 75-105-75 as low as $8.53 per square foot.

(c) Continuous hollow girder concrete bridges are economical for spans in excess of 100 feet. Many of the grade separation structures on the Interstate System are continuous hollow girder structures. Typical of this type are spans of 56-70-70-56 to spans of 88-110-110-88 at a cost of from $9 to $10 per square foot. There are several advantages to the use of hollow girder structures, particularly for grade separation structures, smooth surface on the bottom, uniform depth with the resulting simplification of formwork, less dead load weight, pleasing appearance. Other advantages are the ease with which single column piers can be used, and skew crossings accommodated with a minimum of complications. With hollow girder structures the pier beam can be completely concealed, thus reducing the headroom required for minimum clearance. Hollow girders with spans of 200 foot length have been built.

A good example of continuous concrete box girder construction is a recently completed grade separation structure on the Interstate just west of Topeka over U. S. 40, on a 3 degree curve, superelevated with spans of 88-110-110-88.

(d) One of the most interesting developments in bridge design and construction is the use of hollow slab spans, which have either round or square voids formed with cardboard tubes. This enables the designer to utilize the simplicity and economy of the slab type with longer spans and reduced dead load. You might call these junior box girder bridges. The Lawrence Paper Company at Lawrence, Kansas has done a great deal of research and development along this line, with square or rectangular voids available from 12 inches square to 32 inches by 39 inches. These voids have an egg crate type of unit inside to support the weight and pressure of the concrete, and end caps or collars. They are fabricated to order and shipped knocked down for ease of handling and storage, and are readily assembled on the job.

(e) No talk on bridge design would be complete without mentioning the widespread use of prestressed concrete for bridges and grade separation structures. Time will not permit me to go into the many advantages of prestressed concrete construction. The Prestressed Concrete Institute and the American Association of State Highway Officials have developed standards for the shape and dimensions of prestressed concrete beams, which has done much to simplify the problems of the designer and the manufacturer.

I would like to review briefly the history of continuous concrete bridges in the State of Kansas. The first continuous concrete girder bridge was built in 1933. The first continuous concrete hollow girder structure was built in 1949. Continuous hollow girder structures with total lengths of 485 feet (60-5 @ 73-60). Mr. E. S. "Ted" Elcock, State Bridge Engineer, is responsible for the development of continuous concrete structures in Kansas. A reduced modulus of elasticity is used for the computation of stresses due to temperature change and shrinkage, as well as the computation of dead load deflections. With the exception of large stream crossings, continuous concrete-structures are generally more economical in Kansas.

During the past three years our firm has designed more than 125 bridges and most of these have been continuous concrete structures. I will include a tabulation of some of the bridges designed in our office giving the span lengths, cost per lineal foot, and cost per square foot, and I might add that this cost is based on the curb to curb roadway width, and includes the complete substructure, handrail and other items normally included in bridge construction.

In conclusion, I might mention the use of the electronic computer in the design of continuous concrete bridges. Our office has worked out a number of computer programs for the design of continuous bridges. A list of programs is included in my manuscript, so I will only mention a few of them: the determination of the elastic properties of beams with a variable moment of inertia, the determination of influence ordinates, etc. One of the programs which has been a real time saver is the geometrics of parallel skewed bridges on concentric curves. The computer will do in two days what it formerly took eight man weeks of design time to do with electric calculating machines. This relieves the designer of many routine computations, and leaves his mind free to work with some of the real engineering problems.

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BRIDGE DATA SHEET

Rev. 1-20-60

	TYPE OF STRUCTURE		ROADWAY	LOADING	SKEW	COST	COST/LIN. FT.	COST/SQ. FT.
INTERSTATE: KANSAS US 50, FRANKLIN COUNTY								
	83-104-104-83	Cont. R/C Box Girder	28	H20-S16		$ 99,856.95	$265.22	$ 9.47
Twin	36-48-36	Cont. R/C Slab	44	H20-S16	20°	92,288.90	376.14	8.55
Twin	42-58-58-42	Cont. R/C Girder	41	H20-S16	20°	138,029.50	340.53	8.31
Twin	45-63-45	Cont. R/C Girder	30	H20-S16		75,619.70	243.15	8.11
	61-76-76-61	Cont. R/C Box Girder	24	H15		50,519.60	182.71	7.61
Twin	36-38-36	Cont. R/C Slab	40	H20-S16	14°	71,323.40	290.93	7.27
Twin	128-160-160-128	Cont. Plate Girder	30	H20-S16		470,286.24	405.89	13.53
	54-68-68-54	Cont. R/C Box Girder	24	H15		44,527.10	180.64	7.53
Twin	36-48-48-36	Cont. R/C Slab	30	H20-S16		79,371.70	232.76	7.76
	70-97-97-70	Cont. R/C Box Girder	24	H15		69,532.90	206.64	8.61
Twin	80-112-80	Cont. R/C Girder	30	H20-S16		175,342.70	319.39	10.65
	82-102-102-82	Cont. R/C Box Girder	24	H15		75,853.20	204.73	8.53
Twin	55-77-55	Cont. R/C Girder	30	H20-S16	30°	94,934.85	249.98	8.33
	61-76-76-61	Cont. R/C Box Girder	24	H15		53,744.43	194.37	8.10
	61-76-76-61	Cont. R/C Box Girder	26	H15		60,642.23	219.32	8.44
	68-85-85-68	Cont. R/C Box Girder	24	H15		66,031.61	214.04	8.92
	90-113-113-90	Cont. R/C Box Girder	24	H15		95,268.91	233.22	9.72
	57-72-72-57	Cont. R/C Box Girder	26	H15		55,220.90	211.98	8.15
Twin	30-40-30	Cont. R/C Slab	40	H20-S16		59,041.80	288.01	7.20
	58-73-73-58	Cont. R/C Box Girder	28	H20-S16		68,566.71	259.23	9.26
	70-97-97-70	Cont. R/C Box Girder	24	H15		71,029.78	211.08	8.80
INTERSTATE: TOPEKA BYPASS, SHAWNEE COUNTY								
	88-110-110-80	Cont. R/C Box Girder	30	H20-S16		138,087.50	346.52	11.55
	70-88-70	Cont. R/C Box Girder	30	H20-S16		71,098.40	308.45	10.28
	59-74-59	Cont. R/C Box Girder	30	H20-S16		54,663.00	281.04	9.37
Twin	67-84-84-67	Cont. R/C Box Girder	30	H20-S16		175,238.20	287.75	9.59
Twin	49-61-61-49	Cont. R/C Hollow Slab	30	H20-S16		124,105.00	278.89	9.30
	56-70-70-56	Cont. R/C Box Girder	28	H20-S16	15°	66,066.10	259.49	9.27
Twin	56-70-70-56	Cont. R/C Box Girder	28	H20-S16	15°	127,408.60	250.21	8.94
	61-76-76-61	Cont. R. C. Box Girder	30	H20-S16		77,760.50*	281.04	10.04
	54-68-68-54	Cont. R/C Box Girder	28	H20-S16		63,314.70	256.85	9.17
2 @	40-56-40	Cont. R/C Deck Girder	40&44	H20-S16		86,452.90	312.10	7.43
Twin	50-63-63-50	Cont. R/C Hollow Slab	30	H20-S16	41°	127,094.75	277.15	9.24
Twin	42-52.5-52.5-42	Cont. R/C Hollow Slab	30	H20-S16	22°	96,407.30	251.45	8.38
Twin	38-48-48-38	Cont. R/C Hollow Slab	30	H20-S16	4°	81,228.05	232.75	7.76
Twin	72-90-90-72	Cont. R/C Box Girder	30	H20-S16		180,558.15	276.51	9.22
	2 - 14x12x54	R. C. Box		H20-S16	30°	16,146.40	458.44	9.75
	2 - 14x12x26	R. C. Box		H20-S16		9,655.40	316.57	12.16
	57-71-71-57	Cont. R/C Box Girder	28	H20-S16	18°	64,103.60	247.86	8.85
	54-68-80-54	Cont. R/C Box Girder	28	H20-S16		66,573.35	257.54	9.20
	52-65-65-52	Cont. R/C Hollow Slab	38'6"	H20-S16	19°	73,944.15	312.46	8.12
KANSAS US 40, WALLACE COUNTY								
	48-60-48	Cont. R/C Hollow Slab	28	H20-S16	45°	47,887.00	300.42	10.73
	45-6 @ 63-45	Cont. R/C Girder	28	H20-S16		102,666.40*	218.21	7.79
	36-48-36	Cont. R/C Slab	28	H20-S16		30,605.60*	249.84	8.92
	45-3 @ 56.5-45	Cont. R/C Hollow Slab	28	H20-S16		67,529.60*	257.75	9.21
KANSAS US 36, MARSHALL & NEMAHA COUNTIES								
	36-48-36	Cont. R/C Slab	44	H20-S16		47,364.40	386.65	8.79
	50-70-50	Cont. R/C Girder	28	H20-S16		39,988.40	231.82	8.28
KANSAS K 213, RILEY COUNTY								
	45-63-45	Cont. R/C Deck Girder	28	H20-S16		39,000.00*	250.80	8.96
	30x23x150	R. C. Arch		H20-S16		50,000.00*	1,538.00	10.25
COLORADO BRIDGES								
	45-12 @ 63-45-57-80-57	Cont. R/C Deck Girder	30	H20-S16		306,538.20	293.62	8.16
	60-87-73-56	Cont. R/C Deck Girder	30	H20-S16		79,500.00	285.15	9.50
Twin	7 @ 50'	R/C Deck Girder	30	H20-S16		107,000.00	300.14	10.00
					Total	$4,685,018.76		

*Estimated Cost

BURGWIN AND MARTIN
Consulting Engineers

408 WEST SEVENTH STREET
TOPEKA, KANSAS

BRIDGE DATA SHEET

Rev. 1-20-60

TYPE OF STRUCTURE		ROADWAY	LOADING	SKEW	COST	COST/LIN. FT.	COST/SQ. FT.
ALLEN COUNTY, KANSAS							
40-50-40	Cont. R/C Girder	24	H15		$ 19,825.59	$149.63	$ 6.23
40-50-40	Cont. R/C Girder	24	H15	30°	19,719.87	148.39	6.18
48-3 @ 60-48	Cont. R/C Girder	24	H15		39,872.20	143.17	5.97
44-55-44	Cont. R/C Girder	24	H15		18,572.70	127.65	5.32
CHEROKEE COUNTY, KANSAS							
88-112-112-88	Cont. Steel Girder	24	H15		121,022.96*	300.00	12.50
CRAWFORD COUNTY, KANSAS							
3-12x10x28	R/C Box	28	H20		9,364.00	240.10	8.58
28-35-28	Cont. R/C Girder	24	H15		17,245.80	184.45	7.69
FRANKLIN COUNTY, KANSAS							
88-112-112-88	Cont. Steel Girder	24	H15		117,491.10*	291.30	12.14
36-45-36	Cont. R/C Girder	24	H15		19,546.30	163.57	6.82
GEARY COUNTY, KANSAS							
128-170-170-128	Cnnt. Steel Girder	26	H15-S12		255,682.45	426.55	16.41
GREENWOOD COUNTY, KANSAS							
48-60-60-48	Cont. R/C Girder	24	H15		34,919.65	159.82	6.66
32-40-32	Cont. R/C Girder	24	H15		17,445.00	163.80	6.83
32-40-32	Cont. R/C Girder	24	H15		17,167.80	161.20	6.72
36-45-36	Cont. R/C Girder	24	H15		17,140.35	143.43	5.98
48-3 @ 60-48	Cont. R/C Girder	24	H15		54,097.50*	194.25	8.09
JEFFERSON COUNTY, KANSAS							
48-60-48	Cont. R/C Girder	24	H15		26,419.00	166.68	6.95
48-2 @ 60-48	Cont. R/C Girder	24	H15		35,265.25*	161.39	6.72
32-40-32	Cont. R/C Girder	24	H15		19,240.00	180.66	7.53
28-35-28	Cont. R/C Girder	24	H15		16,532.90	176.82	7.37
2-14x12x26	R/C Box	26	H20		10,336.90	293.50	11.29
40-50-40	Cont. R/C Girder	24	H15		23,000.00*	173.58	7.23
JOHNSON COUNTY, KANSAS							
55-77-55	Cont. R/C Girder	24	H15	30°	43,940.10	231.40	9.64
40-50-40	Cont. R/C Girder	24	H15		22,315.42	168.42	7.02
30-40-30	Cont. R/C Slab	24	H15		30,148.95	294.14	12.26
40-50-40	Cont. R/C Girder	24	H15		19,262.90	145.38	6.06
32-40-32	Cont. R/C Girder	24	H15		16,419.90	154.18	6.42
48-60-48	Cont. R/C Girder	24	H15		24,726.70	156.00	6.50
KEARNY COUNTY, KANSAS							
28-3 @ 35-28	Cont. R/C Girder	24	H15		35,891.10	219.52	9.15
LINN COUNTY, KANSAS							
75-105-75	Cont. R/C Deck Girder	24	H15		59,009.70*	229.16	9.55
POTTAWATOMIE COUNTY, KANSAS							
40-50-40	Cont. R/C Girder	24	H15		25,312.20	191.06	7.96
SHAWNEE COUNTY, KANSAS							
40-50-40	Cont. R/C Girder	24	H15	30°	24,957.50*	187.81	7.83
WILSON COUNTY, KANSAS							
62.5-75-62.5	Cont. Steel Beam	26	H15		49,835.10	245.09	9.43
85-100-85	Cont. Steel Girder	26	H15		65,439.90	239.26	9.20
85-100-85	Cont. Steel Girder	24	H15		72,701.15	265.82	11.08
75-105-75	Cont. R/C Deck Girder	24	H15		54,657.00	212.26	8.84
75-105-75	Cont. R/C Deck Girder	24	H15		52,688.00	204.61	8.53
				Total	1,507,212.94		
			Grand	Total	6,192,231.70		

*Estimated Cost

A QUARTER CENTRY OF BRIDGES

King Burghardt
Regional Bridge Engineer
Bureau of Public Roads
Denver, Colorado

A quarter century of bridges. I chose this topic with the expectation to trace briefly our developments in the art and science of highway bridge engineering. I have encountered an embarassing over-supply of data. To compress twenty-five years of experience into some twenty-five minutes requires drastic cutting and pruning. Personal experiences, sad, happy, funny or serious as well as the contributions of so many of my associates and good friends, must stand aside. Reference to the few unusual or somewhat spectacular structures, which I still view with satisfaction, must yield to the overall picture. For in this somewhat arid Region, it is the prosaic, routine, run-of-the-mill structure that represents the bulk of our efforts and bridge expenditures.

In January of the depression year of 1931, I accepted employment as Structural Designer in the Bridge Department of the Colorado State Highway Department. It was near the start of a new era in American highways. A transition from the "get us out of the mud" period to the "better roads" movement. It started a decade of ever increasing highway budgets where the usual Federal-State ratios were greatly augmented with various relief funds. Locally, it was a decade where a few strips of concrete pavement and many miles of gravel roads became an integrated system of hard surfaced highways.

In the Bridge Department, it was a period of mass production. For the choice of bridge types, the treated timber pile trestle received first consideration. Then followed the concrete on steel stringer spans, with low or through steel trusses reserved for the longer span crossings. Such selection followed prevailing nation-wide practice.

A typical bridge of the thirties had a concrete deck of either twenty-four or thirty-foot roadway with curbs and concrete handrails. Regionally, except in Wyoming, closed, concrete cantilever abutments were favored. Either solid concrete wall or multi-column piers were provided generally. At the start of the period, the design live load was usually H15 for basic unit stresses of 16,000 pound steel and 650 pound concrete. Later revisions to the design specifications increased these working stresses to some degree but retained the H15 live load. Typical unit prices showed treated timber at ninety dollars, concrete between fifteen and eighteen dollars, five cents for steel and four cents for bars.

The decade marked the introduction and gradual acceptance of continuous, statically indeterminate members in highway bridge design. The development of the concrete rigid frame span caused wide-spread interest. One of the four States of our Region adopted this type as their most prevalent crossing. The remaining three, after some trial installations, used it sparingly. Our design practices were improving slowly but our choice of bridge types remained rather routine. In general, our pre-war bridge philosophy produced a rather unimaginative, but serviceable and fairly economical structure. Their counterparts are wide-spread throughout the Region.

Then came the war! Highway construction nearly stopped. Most highway personnel, I included, took an extended leave for other activities. Those remaining made future designs and plans for the necessary recovery of a war-abused roadway system. National defense highways received intensive study at this time, and the Federal-aid Act of 1944 authorized study for establishment of a system of Interstate highways.

Near the close of the war, I returned to the bridge business. This time it was with the Bureau of Public Roads, or the Public Roads Administration, as it was temporarily named. I soon learned that new era of highway construction was upon us. We were entering the age of the super highway. Innovations appeared in droves. As these new ideas have become an operating part of our technical language of today, very little explanation is considered necessary. We are all familiar with H-S truck loads, shoulder width short bridges, transition spirals, interstate live loads, prestressed concrete, composite stringers, T-piers, pipe piles, drilled piles or caissons, single hinge continuous spans, concrete box girders, ramps, speed change lanes, bridge clearances and railings and each item expanded former concepts and practices.

With the close of the war, construction costs soared. We bridge men faced an uncomfortable period when our fellow workers were convinced that our extravagant bridges were depriving them of miles and miles of finished roads. In our defense, we studied and practiced economies with intensity. Post-war shortages also served to open our eyes to new materials and practices. At a time of very slow deliveries and high prices for structural steel, prestressed concrete appeared and proved itself to be highly competitive pricewise. Composite stringers provided decreased costs for steel construction, and welded plate girders show economies for longer spans. A return to the very old fashioned use of cast-in-place concrete slabs or T-beams, particularly with continuous spans, produced some quite economical bridges. Concrete box girders are quite new to this Region although several bridges of this type are ready to go to contract. I entertain the happy opinion that each of the States of Region Nine is thoroughly economy minded - that each studies comparative costs and selects bridge types and materials with a desire to provide the most economical bridge suitable to the site.

For my evaluation of today's progress in bridge construction, I consider it fair to say that we are showing more versatility in types and materials. Today's bridges are stronger, more economical of material, more skillfully designed and their continuous spans provide at least a few less bumps over expansion joints. Through the years we have shown some growth in the comparatively young science of highway bridge design, but possibilities for continued growth are readily apparent.

We need better foundation data. Superstructure failures of bridges designed under any edition of the AASHO Specifications are extremely rare. But when a footing becomes undermined, as it does occasionally, failure is automatic. On the other hand, how many times has piling been used just to play it safe? Foundation investigation by modern methods is becoming a more exact science. A modest investment in geologist-engineers and their necessary equipment should show substantial returns in more economical and better designed footings.

We need better hydrologic data. Many of the available records are buried without ready access to the busy Bridge Engineer. The compilation and processing of old and current discharge records is a specialty best handled by trained statisticians outside of the Bridge Department. Several of our States are getting valuable flood data through the cooperation of the United States Geological Survey. In addition, a Bridge Department needs men proficient in hydraulics to utilize the available data.

We need increasing thought to the training of our young Bridge Engineers. Occasionally we meet one of those rather rare individuals who can see through a maze of mathematical formulae and visualize a practical structure. Such men deserve all of the encouragement, help, and guided practical experience available.

We need to practice human engineering. Many a difficult bridge problem has reached a simple solution through an informal conference with the roadway department. An attitude of friendly mutual respect between State and Bureau pays dividends. We require tolerance in accepting new ideas and materials, maintaining a balance between complete indifference and over enthusiastic acclaim. For example, the modern T-pier is an excellent structural member for certain sites, but it is far from the universal answer to all pier problems.

This tracing of our recent progress in highway bridge design naturally poses the question, "Where do we go from here." My personal crystal ball gives only faint light and shadowy images. I expect to see highway bridges become still wider, stronger and with more skews, kinks and bends than ever. I anticipate continued invasion of the principles of limit or ultimate design theory in our bridge specifications, to the eventual replacement of the present elastic theories. We may be sure that electronic computers will take over many routine, time consuming duties in the bridge office. I expect to see new materials of construction and improved forms of existing materials continue to find their places in bridges. And, finally, I hope that a 1985 review of our new bridges of today will contain as many complimentary phrases as we now may apply to those bridges of the thirties.

A CONTRACTOR'S VIEWS ON BRIDGE DESIGNS

James W. Lawrence
Engineer
Lawrence Construction Company
Littleton, Colorado

Today, Gentlemen, I have been asked to present to you a Contractor's View Point on bridge design. I am associated with the Lawrence Construction Company. Bridge construction is our specialty. To present a contractor's view point on a subject such as this, the matter of economics cannot be overlooked. Since you, as design engineers, are looking for the most feasible design for a given structure, the consideration of economics is important to you. In other words, you are interested in getting the most for your dollar. Today, I will try to present to you some of the problems incurred by a contractor when building to the many different designs that are most commonly used today.

Our experience has been that there are four basic designs commonly used today. They all are primarily of a concrete substructure with steel beams, concrete girders, a concrete slab, or pre-stressed girders. Let us start the discussion with sub-structures.

Let us consider the foundations or footings first. Suppose a bridge is to be constructed over a river, and the design calls for the piers to be set on shale or rock. Many designs call for pads under the columns and a curtain wall between the columns. Usually the curtain wall will start about three to five feet above the tops of the pads. This design is very common and is pleasing to the eye. However, it may be very costly. It is mandatory to excavate the entire length of the pier. If the excavation is wet, the problem of dewatering is made greater by the additional area required for the curtain wall. If the excavation should require tight sheeting, the additional cost of sheeting the area could be quite costly. Also, the curtain wall must be shored up. Many field engineers require that twenty-one days pass before the shoring can be removed. Also, many field engineers will not permit the contractor to leave his shoring in instead of backfilling around it. This, then, entails tying up the sheeting for twenty-one days after the pour has been made; or, if it is not sheeted, the contractor must continue to dewater the excavation for twenty-one days. If the pumps are removed and then put back again twenty-one days later, generally you will have lost the excavation and will have to dig out around the shoring again. All this is costly. If the curtain wall were lowered to the elevation of the footings, no shoring would be required, and after the forms were stripped off, the pumps and sheeting could be pulled and moved to the next excavation, thus saving money. Also, whenever an excavation is wet and requires extensive dewatering, the efficiency of the contractor's organization drops fast as compared to a dry hole. With the labor market as it is today, skilled craftsmen are hard to come by. Many of them are so independent they will not put on a pair of boots and work in a wet hole. Therefore, the simpler the design can be made for a wet excavated hole, the more satisfactory it will be for everyone. If it is at all possible, make the curtain wall the same width as the columns, or make it a simple wall up to that elevation where esthetic or other values may take over. The volume of concrete and reinforcing steel saved by designing for only what is necessary may cost you a great deal more than the savings.

Many pier designs have piling under the pads. Also, sometimes these piling are battered. I am sure that most of you gentlemen are acquainted with the conventional types of pile drivers used in this area. It is practically impossible to batter a piling with a swing of the crane. The swing breaks on the machine are not heavy enough to hold when swinging to one side or the other to gain batter. Therefore, the pile driver either must back up or move ahead to gain the batter. On most conventional rigs, only a 1 in 12 batter can be achieved by moving ahead. Therefore, consider moving backwards. If there is not enough room between piers or between piers and abutments to back up to achieve the desired batter, a great deal of fancy preparation is required along with additional rigging. This cost may not reflect in the piling bid, as the quantity of piling may vary. However, this cost will be in the bid somewhere. Consider this when you design batter piling. Sometimes it may be cheaper to drive extra vertical piling. On some designs, especially those on abutments with piling under them, we have found the footings to meander as required to place the structure on them. We firmly believe that the additional forming expense and the additional reinforcing steel fabrication and placing costs do not justify the savings that would be made if the footing was poured in mass over the abutment area. Years ago, as a general rule, materials were nearly 60 to 80 percent of the job costs. Today, with high-priced and inefficient workmen, the pattern has reversed itself. For instance, the cost of all concrete materials may run $14.50 per cu. yd. while the mean bid price for bridge concrete will run between 50 and 60 dollars. When we estimate a project, we carefully examine the square footage of forming to be done and the size and total length of reinforcement bars to be placed.

And while I am on the subject of estimating, please remember, gentlemen, all of the information you have regarding soil conditions and drainage areas, where applicable, would be appreciated greatly by the contractor if it is passed on to him. Show the wet line if it is applicable or state that it is not applicable if the wet condition does not exist. Show the high-water elevation used in the design along with the low-water data. All of this information would be a great help to the contractor. It is my belief that the design engineer is not receiving enough <u>accurate</u> information prior to his design study. For instance, on one project recently completed, piling was called for under the pads of the piers. Upon examination, we found the most beautiful sand and gravel you could ever desire. We tried to drive piling but could not get a penetration over four feet. It was impossible to drill holes, because the excavation was quite wet. If this information previously had been given to the design engineer, I am sure he would not have called for piling.

While we are talking of piling, gentlemen, please consider strongly the soil conditions when determining the type of piling to be driven. The three most common types of piling being used today are timber, "H" beam, and steel pipe. Do not design for pipe or timber when you know a large boulder condition exists. Always use "H" piling when such is the case. Recently, there has been much talk among bridge people that steel pipe piling cannot be driven in many cases. I assure you, gentlemen, that steel pipe piling can be driven wherever you can drive timber piling and in the majority of the cases where you can drive "H" beams.

There is one other thing I would like to bring to your attention regarding pile driving. A definite refusal should be shown instead of leaving this to the discretion of the field engineer. Most contracting agencies use a formula for determing refusal, but they also state, "As determined by the engineer." Gentlemen, I actually have broken or damaged piling by overdriving them. If this condition exists, what good are your designs? We have driven 12BP53 with a No. 1 Vulcan hammer from a refusal 3/4 in. in the last 20 blows, to absolute refusal in the last 20 blows. Often I have wondered what the bottom of that piling would look like when we had bent the exposed portion of the piling. The same situation has occurred while driving tubular piling. I think that additional education for the design people in the field force as to what refusal "is" would be very constructive. Of course, a piling may be broken or damaged by the contractor if he does not know what he is doing. Many times this happens when tubular steel or timber piling is used. The piling must be hit with the hammer along the vertical axes of the piling, or you are asking for trouble.

One other thing concerning timber piling: you people, as the contracting agency, are paying good money for the materials being furnished. The contractor fully understands this. Therefore, make the inspections very rigid and insist on high quality material. Once we condemned three car-loads of piling that already had been approved by the contracting agency. From previous experience, we knew that these piling could not be driven satisfactorily. There is no justification for inferior materials in bridge construction.

Now let us go back to sub-structure design. With the ever increasing requirements for separation structures, a design pleasing to the eye is mandatory. The use of round columns is becoming very popular. Our experience has proved that round columns are much cheaper to construct than are other types. However, we now have a yard full of round column forms, and just last week I had to order another size round column form. Now we have five different diameter column form sets. Recently we finished a bridge project in which there were three bridges to be constructed, all of which were separation structures. One of the bridges had 20 in. round columns, one had 24 in. round columns, and one had 24 in. square columns. This entailed making or purchasing three different size and shape forms. The lengths of the columns did not vary greatly. Had the size and shape of the columns been standardized, we could have saved the contracting agency at least $1500. Within one project, gentlemen, try to standardize, if at all possible, on one design. Most contractors are equipped for forming round columns with metal forms. If it becomes necessary to put a tie or any other type of projection through a column, use a square or tapered column. Round metal forms must be handled very carefully in order that their shape may be maintained. Should it become necessary to cut an insert or provide any other type block-out in the steel column form, the standard form would be ruined and this increased cost would have to be charged against the job. Most square or tapered columns are still made of wood, and the number of their uses is limited. Therefore, the cost of cutting an insert into a wood column form would not be too great, because the cost of the forms generally are charged directly to the job. If a steel piling bent is designed to be incased with concrete, consider the clearance to be given between the form and the piling. Once we had a steel pile bent to encase which was wrapped with wire mesh. There was a 3/4 in. clearance called for between the piling and the form. Yet, with a standard concrete mix, we were to use 11/2 in. aggregate. We tried to pour this, but after placing 1/2 CY. of concrete, our form was full, when the required quantity was 4-1/2 CY. Two weeks were required to obtain approval to use 3/4 in. rock and to change the mix ratio. This, of course, was costly. While speaking of obtaining approval for a change, this brings to mind a very delicate subject.

Everyone makes mistakes. The eraser on my pencil always wears out before the lead. Whenever a mistake is discovered or a change is found to be necessary, the design people could help the contractor a great amount by promptly making a decision. If nothing else, give the contractor a rough sketch and let him proceed from there. This is far better than waiting for time consuming revisions and new drawings. If a revision becomes necessary, be sure to make your authority known. On one project we were constructing, the design required a pier to rest on rock. There were three pads under the columns. When we excavated, we found that two of the pads would be resting on solid rock, and that the other pad would be resting on "bubbling sand." The chief bridge engineers were summoned to look over the situation. Their instructions were to go ahead with the original design, because, as they said, the sand apparently had sufficient bearing capacity for the construction of the pad and the columns. Immediately after they left the job, the Resident Engineer on the job informed us that these "so-called experts" were not running the job, and that he would not allow us to build on that sand formation. Some three weeks and $5,000.00 later we poured the pad on the sand as originally instructed. To this day we have received no extra compensation for this Resident Engineer's folly. Be sure to make your authority known to those who would be inclined to take unreasonable advantage of their own authority.

When designing a cap that is not integral with the superstructure, be sure to give adequate dimensions regarding the placement of the anchor bolts, especially on a skewed bridge. I believe there is too much confusion among field personnel over the placement of anchor bolts.

Now let us consider some of the different problems a contractor might be confronted with in superstructure design. It is my firm opinion that the design engineer does not consider completely enough the existing field conditions when he designs a bridge. For instance, we recently observed two projects, both requiring separation-type structures. It was necessary that traffic be maintained under or around both of these structures. On the project that required traffic maintenance under the structure, the design specified a concrete-beam type of structure. At the other location the traffic was to be detoured around the new structure. This bridge was designed with steel beams and a concrete deck. Had the designs for these two structures been reversed, a great saving of money could have been realized. The structure with the concrete-girder design, of course, required exterior shoring. To shore up a bridge and to maintain traffic is quite a job in itself. Using here structural-steel beam and concrete-deck design, the bridge could have been built without disrupting traffic. At the job where the structural-steel design was used, traffic was closed under the bridge. Why? All this means increased costs.

I would like to talk about these various designs, considering each type individually. When working with structural steel, the matter of connections comes up. Recently, some designs have specified that the parapet wall be poured intregally with the abutment. Be sure to design your end diaphrams so that they may be connected easily. With only 6-in. clearance or less between the end diaphram and the parapet wall, it is extremely difficult in some designs to connect the end diaphrams. If the parapet walls are to be poured after the steel is in place, be sure to give the contractor enough room to get his forms in place. The contractor is working in tight quarters here anyway, and all of the additional room you can give him will be appreciated. A good deck can be obtained very easily on a steel-girder design. However, when a composite design is used, be sure that you know what is going to happen after the shores are removed or after the deck is poured. If the result is a rough deck, the contractor may spend hours and hours grinding away on the concrete surface. As contractors, we tend to shy away from some of the composite designs. Of course, if a rough deck is obtained, to the contractor falls the blame.

Recently, there has been quite a trend to concrete-girder bridges. Also, there has been a great deal of criticism about the end results; namely, that of rough decks. Gentlemen, you have no one to blame for a rough deck on a concrete-girder type bridge but yourselves. So long as the contracting agency allows the construction industry to do such sloppy work, again, they have no one else to blame but themselves. Certainly, shoring a concrete-girder bridge is not child's play. But as you all know, many of the shoring methods used in the past and which still are being used today invite trouble. You people, as design engineers, should require from the contractor a detailed plan of how the structure is to be shored, and then, you should follow this up in order to insure that this shoring plan is being adhered to. Recently, we observed two bridges being built side by side. The first bridge settled 3 in. during construction. Did the contracting agency stop the contractor from using the same shoring methods on the second pour? No!! The other bridge also went down 3 in. As design engineers, you can stop this unnecessary settlement. By using sound engineering principles and a little common sense, a concrete-girder bridge can be shored so that any settlement will be negligible. There is no excuse for excessive settlement or rough decks on concrete-girder bridges if your contractor is conscientious. Additional education for your field forces also would help.

On a concrete-girder bridge, the common practice lately, especially in separation structures is to require that the bottom of the girders receive a "rubbed" finish. Now, it is usually 21 days before the bottom form of a beam can be stripped. By this time the concrete has gained such strength that it is extremely difficult to achieve a rubbed finish. Now I ask you -- who sees the bottom of these beams? As a result, you are getting nothing but a painted surface of cement grout. If you are interested in effecting a saving, here is one place you may start. Insist that the contractor use panel boards or some similar type form, and you will achieve far more for much less. Diaphrams in a concrete-girder bridge completely interrupt the continuity of form work. If it is at all possible, have your designers stay away from as many diaphrams and other protruding members as is possible. When you design your girder reinforcement, think of the manner in which this reinforcement is to be placed. We had twin bridges to build several years ago wherein girder stirrups were of the closed type. This meant threading the heavy moment bars through all of the secondary reinforcement. Gentlemen, if you want some real fun, just try threading a 60 ft long No. 11 bar through the deck steel and through closed stirrups. If it will do the job, a single open stirrup would be appreciated.

Whenever a construction joint is mandatory, a little architectural treatment of the joint will help the contractor achieve a much nicer looking job. A chamfer strip here or there does not cost much but does enhance the beauty of the structure. So much for concrete-girder construction.

Now, let us talk about concrete slab construction. About the only thing that a contractor really is concerned about in this form of construction is the shoring. When you are designing either for concrete slabs or for concrete girders, check the existing conditions. A savings may be effected through the use of mud sills. However, adequate bearing for mud sills may prove to be non-existant, and the contractor may have to resort to piling. Possibly a steel girder or a prestressed concrete girder design may prove to be more economical.

Let us talk about prestressed girders. Definite progress in prestressed concrete design has been made in the last few years. Uniformity and standardization in design has reduced the cost tremendously. However, there are a few things which could be improved upon from a contractor's point of view. If possible, when you design, allow for either pre-tensioning or post-tensioning procedures. Consider just how these girders are to be erected. On one job a few years back I am sure that a structural steel design would have been much more economical even though the material cost would have been greater. Do not forget that on long spans these girders soon become very heavy and the method of erecting them soon may become so expensive that the additional cost overrides any gain that might have been realized over some other method of design. When designing the end diaphrams, consider the forming problems. Allow room to build and to strip the forms. To date, control of camber and alighment on some pre-tensioned girders has been very poor. Allow for this in your design. With more experience, the contracting profession undoubtedly will be able to produce better results. One governmental agency gives the contractor the use of its testing facilities in order to promote uniformity in design. This is a great help. Yet, there are still many variations in design. One designer will specify 4,000 psi before release of the tensioning wires or before post-tensioning may occur. Another will call for 3,500 psi. Another will call for post-tensioning from both ends of the girder over a given length. The next designer will not. So, to say the least, the contractor, understandably, may be confused. Remember this - the sooner you give the contractor permission to release tension or to begin post-tensioning, the smaller will be the number of casting beds required, thus reducing the cost per girder. The prestressed concrete girder has many of the advantages of structural steel to the contractor, but the weight of these girders often requires specialized equipment. Since prestressed concrete is a relatively new building material, the more the design forces can work hand-in-hand with the contractor, the lower the costs will become.

Now, let us consider curbs and handrails. From a contractor's point of view, in order to achieve a better job, that design which allows the curb to be poured separately from the deck is a great advantage. Pouring the curb separately allows the contractor time in which to properly align and finish the curb and the sidewalk. As to the design of the handrail, the fewer parts requiring field fabrication, such as welding, grinding, etc., the better. This will keep costs down.

In conclusion, Gentlemen, I wish to bring out one fact which I believe has long been overlooked. This is, that as technology increases in bridge design, so must the contractor increase his studies in order to keep abreast of these advancements. This is necessary if the contractor is to remain in business very long. The art of bridge building is coming to the front more and more. Because of this, some contractors are becoming specialists in this field. It is my firm opinion that the contracting agencies would receive better construction, more value for their dollar, and would have less expense overall if they would separate their structures from grading work, paving, and stabilization work and let their bridge work in separate contracts. Then there would be a chance for more and better cooperation between the design forces and the contractor since each would be dealing with someone who spoke his own language. This would be far better than attempting to work with some general contractor who 10 percents the job and sub-lets to the cheapest bidder.

Gentlemen, in this state, most of the bridge contractors hold to these same opinions. Why not give them a try?

REGISTRATION

Name	Official Connection	City & State
1. Adnan, Haji Siraj	Consulting Engineer	Denver, Colo.
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