Figure 24. Example of treated-timber bridge.

Like any new type of construction or material, contractors new to timber bridges may bid unrealistically high when bidding the first few times. With a little experience, however, the cost of treated timber becomes consistently equal to, or usually less than, other bridge materials.

Recently, a consulting engineering firm offered data on total in-place cost comparisons on approximately 200 county and/or township bridges built over a period of three years. Treated-timber bridges averaged about $\$30/ft^2$. Their closest competitor--concrete Quad-T--was about 10 percent more. Other types of bridges were even higher. And, incidentally, most of those timber bridges were of the longitudinal deck design.

We do not have to remind you of the tremendous need for bridge replacement or that the need for new bridges will cost the taxpayer millions of dollars. Never has there been a time when economy was more important. Many of those bridges are in rural areas on county, township, or municipal roads and at sites where simple, multiple short-span, and economical bridges are ideal. With treated-timber bridges, there is an opportunity to have some of the best of all worlds (Figures 23 and 24).

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Live Load Distribution in Concrete Box-Girder Bridges

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Traditional methods for designing bridges that reduce the significant parameters affecting distribution of live loads to a single entity (e.g., stringer spacing or deck width) represent archaic oversimplifications. They are held over from the precomputer era and result in a spectrum of designs that range from ultraconservative to those that would be unsafe but for generous safety factors. Development of such distribution factors has usually been based on the assumption that all lanes on a structure are loaded with design vehicles, and such design methods become particularly meaningless when used in conjunction with hybrid loadings such as California's Permit-series, which comprises a single, heavy rating vehicle in combination with a single H-series design vehicle. Sophisticated analytical tools developed in the postcomputer era can provide very exact designs (perhaps more exact than warranted by live load specifications), but these tools are too cumbersome for use in a production environment. Presented here is an alternate, intermediate design method that combines relative exactness with a shortcut design approach that employs nomographic analysis for traditional designs and influence-line analysis for hybrid loadings.

For many years, the concrete box-girder bridge has enjoyed special popularity on California's freeway network. Prior to 1959, design of such structures for live load was based on a distribution-factor approach in which individual I-sections were assumed loaded with S/5 wheel lines of a standard H-series vehicle, where S is the spacing (in feet) between centerlines of webs.

In 1959, California design engineers, who appreciated the large torsional rigidity of the closed box section, suggested to the American Association of State Highway Officials (AASHO) a change in this distribution factor to S/7. Sophisticated techniques for analyzing such structures were unavailable at the time, and the recommendation had little scientific basis; nonetheless, the new specification was tentatively adopted, contingent on California's agreeing to embark on a research project to study box-girder load-distribution phenomena.

The research program began in 1960 with field testing of the Harrison Street Undercrossing (1,2), a 34-ft-wide structure that had a single span of 80 ft. The cross section comprised four cells spaced at 7 ft 3 in and provided a live load distribution of S/5 = 1.450 wheel lines according to the earlier specification and S/7 = 1.036 wheel lines according to the revised specification.

Field testing entailed heavily instrumenting the structure with strain- and deflection-measuring devices and running a Euclid truck across the span in 13 transverse positions while internal strains and deflections were recorded. The two-axle test vehicle was heavily ballasted with reinforcing bar ingots to 57 kips. Tests were run in three phases: with and without intermediate diaphragms and after addition of 3-ft-wide barrier curbs and rails.

Analytic techniques for data reduction involved plotting of individual strains as functions of transverse position of the test vehicle and, subsequently, hypothetical placement of more than one vehicle on these strain plots for superposition of strains and conversion to stresses and stress integration in individual I-sections for determination of stringer moments. These stringer moments were compared with computed moments on the span due to a single wheel line of the test vehicle to permit assessment of an S/D factor for hypothetical combinations of test vehicles critical for each stringer.

The 28-ft roadway between the 3-ft-wide barrier curbs permitted two lanes under the specifications current at that time, and application of two hypothetical test vehicles to strain plots produced a distribution factor of about S/8. A 1-ft-wide standard barrier curb on the same basic superstructure, which had a 32-ft-wide roadway, 3 traffic lanes, and 3 hypothetical vehicles on the strain plots, indicated a value of about S/5.5 with no change in the value of S. A distribution factor based only on stringer spacing was inadequate and would require, at least, inclusion of the number of traffic lanes as an additional parameter.

Tests of a 1:3.78 scale, reinforced concrete model were conducted at the University of California, Berkeley, by Scheffey concurrently with tests of the full-scale prototype. The model was loaded with a single, concentrated force, but it was demonstrated in the analysis of the data by Davis and others (3,4) that its behavior closely paralleled that of the prototype.

Subsequent to completion of initial field tests and analyses, a long-term study was initiated jointly by the California Division of Highways and the University of California, Berkeley, to develop analytical techniques for assessment of box girder structural behavior. This phase of the work resulted from a request for methods of assessing shears in skewed concrete box girders. It soon became evident that behavior of the skew box was complex and that analysis of such structures would follow a lengthy progression of development of analyses of geometrical configurations characterized by simpler behavior.

Concurrent development of the electronic computer and its adoption by the engineering discipline permitted application of established analytical techniques of numerical complexity that had previously precluded such use. In 1966, Scordelis (5) published the first program (MULTPL) to be developed as part of this research contract, which was for analysis of simply supported box-girder bridges employing a direct stiffness application of folded-plate techniques to closed box sections. Initial application of the folded-plate analysis to the Harrison Street Undercrossing field-test data had demonstrated that the method could predict moment distributions with precision.

In 1967, Scordelis (6) described three additional programs for box-girder analysis: (a) MUPDI, a folded-plate analysis; (b) SIMPLA, a finite-segment approach; and (c) FINPLA, a finite-element analysis. A second-generation MUPDI (MUPDI2) permitted application of folded-plate analysis to continuous structures that contained rigid, intermediate diaphragms and provided automatic moment integration for individual stringers. The MULTPL and MUPDI programs require simple support conditions at abutments, while SIMPLA can analyze structures with fixed (built-in) conditions at end supports.

The S/7 distribution factor adopted in 1959 continued to be used in California except for a slight modification in the late 1960s, which was to a "whole width"/7 factor.

In 1969, Scordelis, Davis, and Lo (7) noted that an S/7 distribution factor for the Harrison Street Undercrossing would produce 7.25/7 = 1.036 wheel lines per interior girder and W_e x 1.036/S = 6.125 x 1.036/7.25 = 0.875 wheel lines on each exterior girder for a total of 4.86 wheel lines on the structure. Such a design would be conservative for 3-ft barrier curbs and two lanes of traffic, and it would be unconservative for 1-ft barrier curbs and three lanes of traffic.

In 1970, Sanders (8), who used data from a study by Scordelis of 200 California bridges and the Scordelis programs, estimated a range of live load distribution factors of S/5.7 to S/8.3.

In 1969, Scordelis and Meyer (9) published an exhaustive study of wheel-load distribution in concrete box-girder bridges and developed a formula that included parameters thought to influence load distributions; i.e., span between supports, span between inflection points, number of lanes, cell width, and number of cells. The formula provides a ratio (α) of the number of wheel lines taken by a girder to the number of wheel lines taken by the girder in a rigid structure; the latter factor may be calculated as the ratio of the stiffness of a given girder to the stiffness of the cross section. (The formula will not be repeated here, since the original publication may be consulted by interested readers.)

In 1974, California adopted a load factor design and introduced a special Permit-series live loading to reconcile discrepancies between design and rating criteria (10). The special loading stipulated a series of P-series vehicles of greater weight and size than the H-series trucks with one such vehicle in a given lane of a bridge and a single H-series vehicle in an adjacent lane.

Prior to 1974, American Association of State Highway and Transportation Officials (AASHTO) specifications stipulated traffic lanes of variable widths that divided the roadway between curbs equally. In that year, the specifications were changed to provide 12-ft-wide lanes with no partial traffic lanes.

The Scordelis-Meyer formula soon became obsolete for the following reasons:

1. The derivation was based on lane widths of 10-16 ft. The 1974 change in specifications significantly altered the basic assumptions.
2. The formula recognizes a difference between wheel-line distributions to exterior and interior girders but assumes vertical exterior webs. Most California box girders designed in the past decade have used sloping or curved exterior webs, where stiffnesses of exterior and first interior girders will be affected, and a correct formula must include separate α-factors for exterior, interior, and first interior girders.
3. The formula recognizes a difference in wheel-line distributions at midspan and interior supports. Based on proof of small error, distributions throughout the positive-moment region were derived by assuming that all wheel reactions were concentrated at midspan. It appears that this same assumption was made for support moments, and this condition may not be critical.
4. Since Permit-series vehicles are much longer than HS-20 trucks, some reassessment of validity of the first assumption mentioned in 3 above is necessary.
5. The Scordelis formula was based on the assumption that all lanes were loaded with H-series trucks. The same assumption used in conjunction with heavier Permit-series loadings would result in ultraconservative designs. It is difficult to visualize the application of any formula to hybrid loadings, which are comprised of mixed-vehicle modes. The Office of Bridge Maintenance, California Department of Transportation (Caltrans), frequently rates bridges for vehicle configurations that differ from P- or H-trucks; i.e., any proposed design tool should be capable of treating any loading.
6. A very significant deficiency in the formula is its unwieldiness for a production environment. Significant effort was expended by the Caltrans Research Unit that investigated methods of application, such as nomographs, parametric curves, etc.; the number of significant parameters precludes use of such devices.

OBJECTIVES OF RESEARCH

The current project was initiated to overcome the above-mentioned problems and incorporate into the design the results of box-girder studies in California. A need has been identified for the production of design methods sufficiently flexible to permit treatment of any reasonably specified loading.

RESEARCH PROCEDURES

Development of a load-distribution formula proved to be infeasible. The Scordelis formula was too unwieldy for production design because of the number of parameters. An even worse situation could now be expected, since (a) a new parameter--the web slope--would be added, (b) a third equation (for first interior girders) would be added to two equations now required for separate distributions to exterior and interior girders, and (c) three additional equations would be required for distribution at supports. A distribution formula for hybrid (e.g., P-series) loadings would be difficult to develop.

It was decided to present the distribution concept in the format of parametric influence lines, which have ranges of parameters typical of the majority of designs for any arbitrary loading, and simplified nomographs for application to H-series loadings.

A particular advantage of the influence-line approach is its applicability to unusual loadings. A vehicle of one variety can be placed at a critical location on an influence line and percentages of total moment due to that vehicle distributed to each stringer can be read from the plot. The process may be repeated for vehicles of other types and stringer moments due to various vehicles accumulated.

Some disadvantages are also evident, such as the following:

1. Precedent almost demands inclusion of a distribution formula for every basic structure type in the AASHTO specifications. It is difficult to predict how engineers will react to substitution of a booklet of parametric influence lines for such a formula.
2. The multiplicity of parameters required limitations on parametric ranges and use of mean values within these ranges. Some error will occur in designs where parametric values depart from mean values.
3. Interpolation will be required when parametric values fall between plotted ones. Extrapolation may not be safe for unusual parametric values that are outside plotted ranges. A sophisticated analysis by means of such programs as MUPDI and SIMPLA may be required in such cases.
4. The major disadvantage lies in limitations of applicability. Subsequent to development of Scordelis' formula, work on box-girder analysis has proceeded at the University of California and at Caltrans. Models have been constructed and analytical techniques developed to assess behavior of curved and skewed box girders.

The finite-element program CELL was developed by Willam and Scordelis (11,12). Tests by Aslam and Godden (13) of small aluminum models that had varying skews demonstrated that the program could assess skew box behavior with precision. Nix (14) used the program to study behavior of a heavily skewed box-girder railroad structure and noted significant diminutions of longitudinal stringer moments. Wallace (15,16) made 51 parameter studies and demonstrated analytically that total stringer moments could be reduced as much as 44 percent for a 45° skew and 70 percent for a 60° skew.

Davis (17) designed a 1:2.82 scale, reinforced concrete model with 45° skewed supports to be tested at the University of California, Berkeley (18-21), for further verification of CELL as applied to skewed concrete box girders. With redistribution of shears and diminution of moments, influence coefficients developed for structures on orthogonal supports will not apply to structures with heavy skews.

Influence surfaces might solve problems introduced by skew supports, but they are tedious to use and of questionable applicability to structures of configurations that differ from those for which they were developed. Each surface provides a value of a parameter at only one location. The volume of plots required to cover the range of parameters precludes their use. No simplistic tool can be developed that will cover the entire range of parameters that may be anticipated.

DEVELOPMENT OF INFLUENCE LINES

SIMPLA2 was used to establish data for input to the influence-line plotter program. Seven studies were made to determine realistic, transverse distributions of longitudinal girder moments for 60-, 100-, and 120-ft spans that have 3, 4, and 6 cells of widths at 7 and 7.5 ft.

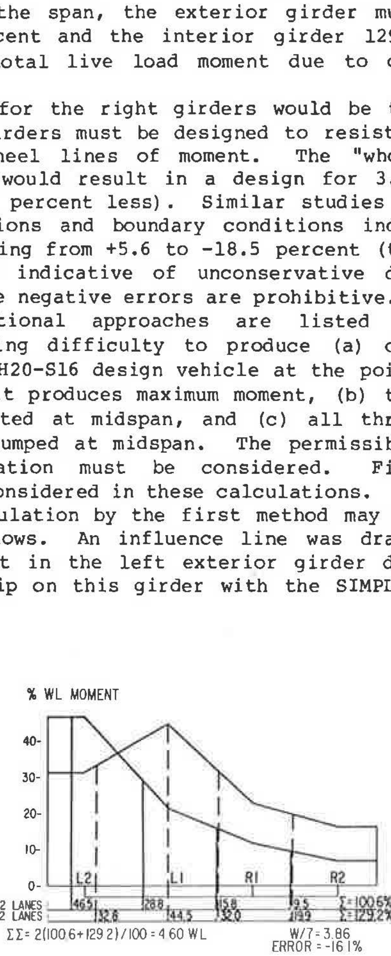
Figure 1 depicts transverse distributions of percentages of total girder moments for one wheel line of an H20-S16 truck on a 60-ft span box girder, which has three 7-ft-wide cells, a 27-ft-wide deck, and fixed simple-support conditions for left exterior and first interior girders. Percentages of total longitudinal moment taken by these two girders are calculated just below the cross section. With two vehicles on the span, the exterior girder must resist 100.6 percent and the interior girder 129.2 percent of the total live load moment due to one wheel line.

Distributions for the right girders would be the same. The four girders must be designed to resist a total of 4.60 wheel lines of moment. The "whole width"/7 concept would result in a design for 3.86 wheel lines (16.1 percent less). Similar studies of other cross sections and boundary conditions indicated errors ranging from +5.6 to -18.5 percent (the negative sign is indicative of unconservative designs). The large negative errors are prohibitive.

Three computational approaches are listed in order of increasing difficulty to produce (a) one wheel line of an H20-S16 design vehicle at the point in a span where it produces maximum moment, (b) the center wheel located at midspan, and (c) all three wheel reactions lumped at midspan. The permissible error of computation must be considered. Five structures were considered in these calculations.

A typical calculation by the first method may be described as follows. An influence line was drawn for midspan moment in the left exterior girder due to a load of 1 kip on this girder with the SIMPLA2

Figure 1. Calculation of error in use of S/7 formula.



program. Influence lines were drawn for moment in this girder due to loads on the other girders and for total moment in the cross section due to a 1-kip load in any transverse location.

A plastic overlay, which shows reactions for one wheel line of the design vehicle, was moved along the influence lines, and ordinates were multiplied by wheel reactions and summed until a maximum total moment was obtained. Use of just one wheel line ensured maximum possible moment due to a vehicle (since the longitudinal position for maximum moment in, say, girder A with girder A loaded is not necessarily that for maximum moment in girder A with girder B loaded).

Percentages of total moment taken by each girder were computed for successive transverse positions of the wheel line. (These percentages need not sum to 100; they are maximum percentages of total moment taken by a girder as functions of transverse position of the wheel line).

In the second set of calculations, midspan influence ordinates and those 14 ft on either side of midspan were multiplied by corresponding wheel reactions. These calculations required no bracketing and less effort than the former ones.

Third, the moment percentages were calculated as the ratios of influence ordinates at midspan for individual girder moments to those for the total section based on all wheel reactions being lumped at midspan. The differences between percentages resulting from the first and second computational methods were inappreciable and those between the first and third were prohibitively large (results of the third calculation were always conservative).

The first method of computation would produce the most accurate distribution factors but would require drawing all longitudinal influence lines and tedious bracketing. Because the second method did not introduce substantial error, moment percentages for each girder were obtained directly from the SIMPLA2 output with wheel reactions input in known positions.

Transverse distributions of moment at supports were investigated independently. It would seem most reasonable to assess such distributions for vehicle positions that produce maximum negative moments at supports. Two methods of computation were considered: (a) longitudinal influence lines were established for each girder with SIMPLA2 and an H20-S16 wheel-line overlay placed on these lines by trial and error until a maximum moment was obtained, and (b) the influence coefficient was calculated only for a single hypothetical maximum moment location and multiplied by the lumped value of three wheel reactions of one wheel line of the H20-S16 vehicle. The second mode of calculation entails much less effort; however, error due to the approximation must be within permissible limits.

Errors in moment percentages calculated by the second, less-exact method were as large as 8.9 percent and always unconservative. An assumed, permissible unconservative error of 4 percent suggested a uniform increase of 12 percent in the exterior girder support moment percentages for the idealized vehicle and an 8 percent increase in interior girder percentages in loaded girders only.

Additional calculations were made of positive moments at locations 7 and 14 ft from midspan. Moment percentages for positive-moment regions were compared with those previously calculated; there were no significant differences in moment percentages.

These studies were based on the H20-S16 design vehicle. The P-series vehicles are much longer, and some reduction in moment distribution coefficients may be expected.

Separate runs were made by using the SIMPLA2 program for P-loads, and moment percentages were compared by means of a reduction factor. These factors were plotted for short spans, and an approximate lower bound was established as a conservative estimate of maximum allowable reduction factors for short spans. Equations for this lower bound are as follows. For short spans (60 to 120 ft), the equations are given below:

Girder	Midspan	Support
Exterior	$R = (L - 48)/[600(1 + K)]$	$R = 0$
Interior	$R' = R/2$	$R' = 0$

In these equations, R and R' are the reduction factors, K is the exterior girder slope factor, and L is the span length (in feet).

The reduction factor for P-loads for short spans peaks at a span length of 120 ft at 12 percent for exterior girders and 6 percent for interior girders. For spans that exceed 120 ft, the reduction factor decreases with increasing span because the controlling alternative lane loading covers the whole span and is longer than the P-13 truck. Reduction factors are listed below for long spans (120 to 270 ft), where R, R', L, and K have the same significance as above:

Girder	Midspan	Support
Exterior	$R = (L - 240)/[1000(1 + K)]$	$R = 0$
Interior	$R' = R/2$	$R' = 0$

An evaluation made by the Caltrans Structures Loads Committee suggested that the new, relatively precise method be used as the design for more realistic trucks for which bridges are rated than for the "artificial" P-13 truck. Investigation indicated that the 13-axle truck would always be more critical than the vehicles for which such ratings are usually made.

RECOMMENDATIONS FOR IMPLEMENTATION

The four different approaches developed for the design of box-girder bridges are given below:

1. There is nomographic analysis for structures that have 3 to 8 cells, spans of 60 to 270 ft, cell widths of 7 to 15 ft, exterior web slopes of 0 (vertical) to 1 (1:1), depth-to-span ratios commensurate with ordinary reinforcement (60- to 150-ft spans) and prestressed (80- and 270-ft spans), and with the critical number of lanes loaded with H20-S16 design trucks. Specified reduction factors (0.90 for three lanes, 0.75 for four or more lanes) are included. The method will not apply to partial loadings for mixed-vehicle modes.
2. There is influence-line analysis for the same parameters as above but that includes provisions for partial and hybrid (mixed-vehicle) modes and rating for special vehicles.
3. There is programmed analysis, which permits automated application with interpolation of the influence-line analysis.
4. There is also the application of sophisticated computer codes developed by Scordelis for special uses (e.g., unicellular or bicellular spine beams, boxes with more than eight cells, heavily skewed structures, structures with arbitrary plan geometry, composite steel and concrete boxes, etc.).

Figure 2 depicts a typical structure cross section to which these methods might be applied. The report by Davis and others (22) provides a detailed study of such an application. The following dimensions are pertinent: span (L) = 80 ft, number of cells (N) = 4, cell width (CW) = 8.5 ft, (W_e) =

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Design of a Skew, Reinforced Concrete Box-Girder Bridge Model

RAYMOND E. DAVIS

A 1:2.82 scale model of a two-span, continuous, reinforced concrete box-girder bridge, which has supports skewed at 45°, was constructed and tested at the University of California, Berkeley. The cross section and significant dimensions were similar to those of two previously tested models, one straight on orthogonal supports and one curved on radial supports. The objective of the research was to compare behavior of the three models and to verify an analytically predicted diminution of longitudinal stringer moments that result from skewing supports. All three models were designed by the California Department of Transportation. Because traditional design criteria make no provision for skewed supports, the skew model was designed by means of a sophisticated finite-element computer code called CELL. Girder moments proved to be significantly less than those in the orthogonally supported model and had a 19 percent reduction in the main longitudinal reinforcing steel. Distribution of girder shears was changed significantly from that of the model on normal bearings. As a basis for implementation, this paper discusses some features of the skew model design process.

For many years, the California Department of Transportation (Caltrans) has been interested in anomalies that characterize the structural behavior of reinforced concrete box-girder bridges with skewed supports. Initially, interest was centered on effects of skew on girder shears. Excessive cracking of webs observed at obtuse corners suggested enhancement of girder reactions that had commensurate increases in diagonal tension.

Complexities in the analysis of skew boxes restricted early efforts toward mitigation of observed excessive web cracking to establishment of curves for augmentation of exterior and first interior girder shears at obtuse corners of such boxes. (Traditionally, skewed boxes in California have been designed as structures of the same spans on orthogonal supports and detailed with skewed supports.) Curves for shear augmentation were established with little scientific basis and furnished, at best, only estimates.

A request in 1959 by design management for a more definitive study of this problem initiated a protracted study of reinforced concrete cellular structures performed jointly by Caltrans' Structural Research Unit and the University of California, Berkeley. The research effort included tests of full-scale prototypes and small and large-scale models. Structures of increasing complexity were studied on a progressive basis, as follows: (a) simple span boxes without diaphragms on normal supports; (b) simple span boxes with rigid intermediate diaphragms, or continuous boxes without intermediate diaphragms on normal supports; (c) continuous boxes with intermediate diaphragms, which consider effects of bent and diaphragm flexibility; (d) curved boxes with radial supports; (e) nonprismatic boxes; (f) skewed boxes; (g) prestressed boxes; and (h) composite concrete and steel boxes. Analytic methods employed in the development of computer codes by the University of California relied heavily on the folded-plate theory and finite-strip, finite-segment, and finite-element methods.

A valuable computer code developed as part of the research effort employs a finite-element analysis to assess behavior of cellular structures of arbitrary plan geometry. This program, called CELL, was first used within Caltrans to analyze a heavily skewed, and curved, box-girder bridge to carry rail traffic and to assess the influence of intermediate diaphragms on that behavior. The program has been used in studies of boxes of varying skews and aspect ratios to establish functional relations between skew angle and shear augmentation factors. Estimated curves of such factors previously used by Caltrans were proved to be unconservative.

A serendipitous result of these studies was the