Strengthening of Continuous-Span Composite Steel-Stringer Bridges

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On the basis of current bridge rating standards, many continuous-span composite steel-stringer concrete deck bridges in the United States are classified as deficient and in need of rehabilitation and strengthening, or replacement. Through several Iowa Department of Transportation research projects, methods of strengthening such bridges have been developed. Ways in which two of these strengthening procedures—post-tensioning and superimposed trusses—have been applied to actual bridges are described, and a design methodology is explained briefly. The strengthening systems were implemented and tested on two existing three-span bridges; both bridges were 45.72 m (150 ft) long and had roadway widths of 7.37 m (24 ft). From two analyses, it was determined that both bridges, when subjected to legal live loads, were overstressed in both the positive and negative moment regions. The bridges, instrumented for strain and deflection measurements, were loaded with trucks before and after strengthening to determine the effectiveness of the strengthening systems. To alleviate the flexural overstress in Bridge 1, a post-tensioning scheme was designed in which the positive moment regions of all beams were post-tensioned. This strengthening scheme reduced the overstresses in both the positive and negative moment regions. In Bridge 2, superimposed trusses were employed over the piers on the exterior stringers in addition to the post-tensioning of the positive moment regions of all beams. In both bridges, considerable end restraint was measured; it was also determined that the guardrails were making a structural contribution. The transverse and longitudinal distribution of post-tensioning forces is summarized, behavior changes are noted, and the effectiveness of both strengthening systems is discussed. A design methodology developed for practicing engineers for use in designing a strengthening system for a given continuous-span bridge will also be described briefly. Both strengthening schemes were determined to be cost-effective and practical techniques.

Of the large percentage of bridges in the United States classified as deficient and in need of rehabilitation or replacement, many are deficient because their load-carrying capacity is inadequate for today's traffic. Strengthening often can be a cost-effective alternative to replacing the bridges or posting them for restricted loads.

The concept of strengthening single-span composite steel-beam concrete deck bridges by post-tensioning has been developed through several Iowa Department of Transportation (Iowa DOT) research projects (*1–6*). Since the completion of the initial design manual (*5*) Iowa DOT has used the allowable stress design methodology for the post-tension strengthening of many single-span bridges.

As a result of the previous success with post-tension strengthening of single-span composite bridges, a research program for strengthening continuous-span composite bridges—similar to the program for single-span bridges—was undertaken. In Phase 1 (7) it was verified that continuous-span bridges can be strengthened by post-tensioning. In most continuous-span bridges, the desired stress reduction in the positive moment regions as well as in the negative moment regions can be obtained by post-tensioning only the positive moment regions. This finding was determined theoretically by using a finite element analysis and experimentally by testing various post-tensioning schemes on a 1/3-scale three-span continuous bridge model.

This paper presents the results of Phase 2 (*8,9*), in which two three-span continuous bridges were strengthened by post-tensioning and then tested. The primary goals of this phase of the study were to design and install the strengthening systems on continuous-span steel-beam concrete deck bridges, instrument the bridges for measuring deflections and strains, and document the bridges' behavior for a period after installation of the strengthening systems.

The final phase of the investigation, Phase 3 (*10*), has also been completed. The design methodology developed provides a procedure for determining the magnitude of forces required to strengthen a given continuous-span bridge. Finite element analysis and the experimental results from the two bridge tests described in this paper were used to formulate and calibrate the methodology. As a result of the complexity of the design procedure, a spreadsheet was developed to help engineers determine the strengthening forces required for a given bridge.

Description of Bridges

An advisory committee assisted in locating the two three-span continuous bridges selected for strengthening. For clarification, the bridges henceforth will be referred to as Bridge 1 (located in northwest Iowa) and Bridge 2 (located in central Iowa). The framing plan for the two bridges and the bridges' cross sections are shown in Figure 1. As illustrated, the bridges have a total length of 45.72 m (150 ft), and spans of 13.94 m (45 ft 9 in.), and center spans of 17.83 m (58 ft 6 in.). The four beams in these bridges are coverplated top and bottom near each of the two piers and are spliced at the center-span nominal dead-load inflection points. Except for a small difference in the size of diaphragms at the piers (Bridge 1 pier diaphragms are shown in Figure 1), the two bridges are identical.

Analysis of the bridges indicated that they required strengthening to reduce flexural overstresses as well as additional shear connectors to improve composite action. The strengthening system designed for Bridge 1 involved post-tensioning the positive moment regions of all beams (12 regions). In the end spans, post-tensioning tendons were positioned above the bottom beam flange; in the center span, as a result of large clearances, it was possible to position the tendons below the bottom beam flange. Tendon forces were applied to theoretically reduce all steel beam stresses to levels below operating load levels, thus removing the need for load posting; in all but a few isolated locations, steel beam stresses are actually below inventory stress levels. In total, 6427 kN (1,444 kips) of post-tensioning force was required to strengthen Bridge 1.

The post-tensioning system designed for Bridge 2 was similar to that used on Bridge 1 in that the positive moment regions of all beams (12 regions) were post-tensioned. In addition to the post-tensioning, however, Bridge 2 had superimposed trusses installed on the exterior stringers at the piers (Figure 2). At each of these locations, there are trusses on both sides of the beam web—eight superimposed trusses on the bridge. By post-tensioning the tendons in the trusses, upward forces are produced at the upper truss joints. The combination of trusses and post-tensioning of the positive moment regions makes it possible to reduce stresses the desired amount at all locations in the bridge. Clearance restrictions in Bridge 2 dictated that post-tensioning tendons be positioned above the bottom beam flange at all locations. The total theoretically required tendon forces, increased to account for potential losses, were 6311 kN (1,418 kips). The post-tensioning forces in the positive moment region were significantly smaller than those required to strengthen Bridge 1 [6427 kN (1,444 kips) for Bridge 1 versus 3338 kN (750 kips) for Bridge 2] because of the contribution of the superimposed trusses. In addition to the post-tensioning forces, Bridge 2 had 2973 kN (668 kips) of force applied to the trusses.

As noted previously, both bridges required additional shear connectors for composite action. Because the number of load cycles that had been applied to each bridge was unknown, the number of additional shear connectors required was based on ultimate strength. Double-nutted high-strength bolts 25.4 mm (1 in.) in diameter—essentially the same as those tested and employed in single-span bridges—were used as shear connectors. Added to each interior and exterior beam of the two bridges were 58 and 52 high-strength bolt shear connectors, respectively (220 per bridge).

Field Testing Program

Bridge 1 was strengthened and tested one summer and retested approximately 1 year later. Except for removing and reapplying the post-tensioning forces the second year, the testing program used each year was essentially

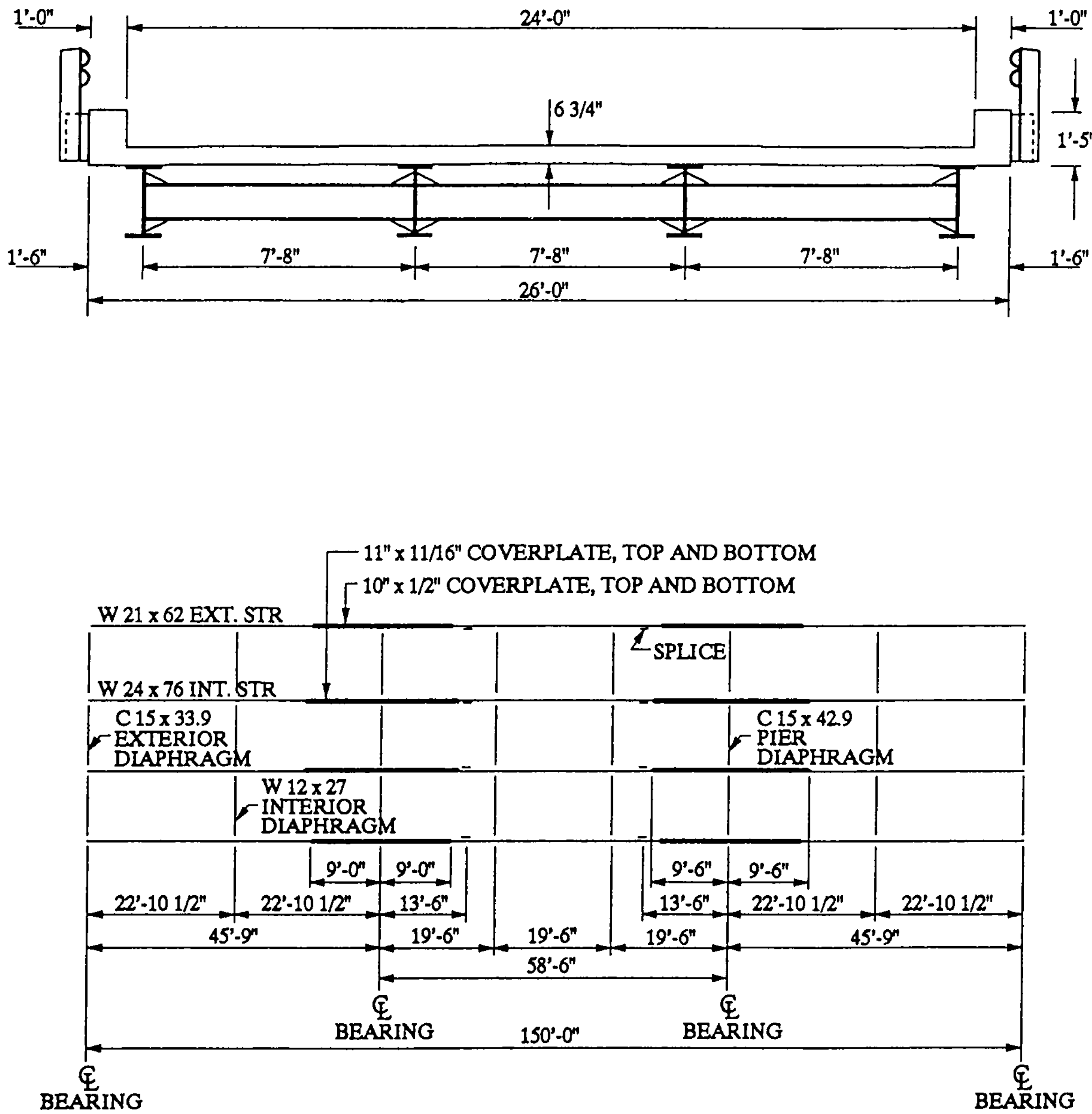


FIGURE 1 Cross section (*top*) and framing plan (*bottom*) of Bridges 1 and 2 (1 ft = 0.305 m).

the same. Bridge 2 was strengthened and tested a few years after Bridge 1. Both bridges were subjected to the following four loading conditions to determine their behavior, strains, and longitudinal and vertical displacements:

1. A heavily loaded truck at various predetermined locations on the bridge;
2. Various stages of the strengthening sequence—because all 12 beams of Bridges 1 and 2 and the trusses of Bridge 2 required post-tensioning, it was necessary to apply the strengthening forces in stages;
3. The same heavily loaded truck at the same locations, after strengthening of the bridges was completed, to determine the effectiveness of the strengthening systems; and
4. Two heavily loaded trucks at various predetermined locations on the bridge, to maximize the moments at various locations.

Although there were some small variations in the instrumentation used on the two bridges, the instrumentation on Bridges 1 and 2 was essentially the same. Instrumentation for the field tests consisted of electrical-resistance strain gauges (strain gauges), direct-current displacement transducers (DCDTs), dial gauges, and crack monitors. Strain gauges (two gauges per location) were mounted on the lower flanges of all beams near

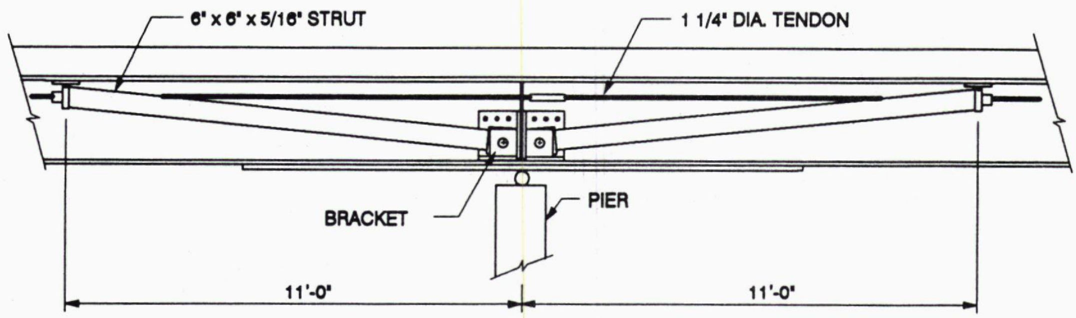


FIGURE 2 Superimposed truss system (1 ft = 0.305 m).

the centerline of each span and near the supports [0.38 m (15 in.) from the centerlines of the piers and end abutments]; thus, there were 14 strain gauges per beam. Strain gauges were also mounted on the tendons so that applied post-tensioning forces and changes in post-tensioning forces due to live load could be measured accurately. A few strain gauges were also mounted on the guardrails of both bridges.

Eight stages of post-tensioning were applied to Bridge 2. Post-tensioned forces were first applied to the trusses (Stages 1 and 2) and then to the beams (Stages 3–8) so that the applied strengthening forces were always applied symmetrically with respect to the centerline of the bridge. Because Bridge 1 only had post-tensioning of the beams, it was strengthened in six stages, which were similar to Stages 3–8 used on Bridge 2. Vertical deflections of the bridges (midspan of all beams) were determined using DCDTs. Longitudinal movements of the bridges relative to the supports were determined using dial gauges and crack monitors. The data from the strain gauges and DCDTs were recorded by a computer-controlled data acquisition system.

As noted, Bridge 1 was retested approximately 1 year after it was strengthened to determine any changes in its behavior and any loss in prestressing forces. Photographs of Bridges 1 and 2 are shown in Figure 3. Bridge 1, which had only post-tensioning of the positive moment regions, is shown in the top of the figure; the bottom illustrates Bridge 2, which was strengthened with post-tensioning in the positive moment regions and with superimposed trusses in the negative moment regions.

Test Results

Only a very limited portion of the results of this investigation is presented in this paper. Additional results on the strengthening and testing of Bridges 1 and 2 may be found in work by Klaiber et al. (*8,9*).

As previously noted, it was necessary to apply the strengthening forces to the two bridges in stages: six stages for Bridge 1, and eight stages for Bridge 2. Except for one location, forces slightly larger than theoretically required were applied to Bridge 1. In Bridge 2, the applied truss post-tensioning forces and a few of the applied beam post-tensioning forces were slightly less than the theoretical values. The effect of one post-tensioning stage on the post-tensioning forces in beams that had been post-tensioned previously was apparent in both bridges. This effect is more significant on beams in the same span and on beams in adjacent spans that are in line with those being post-tensioned. The greatest loss

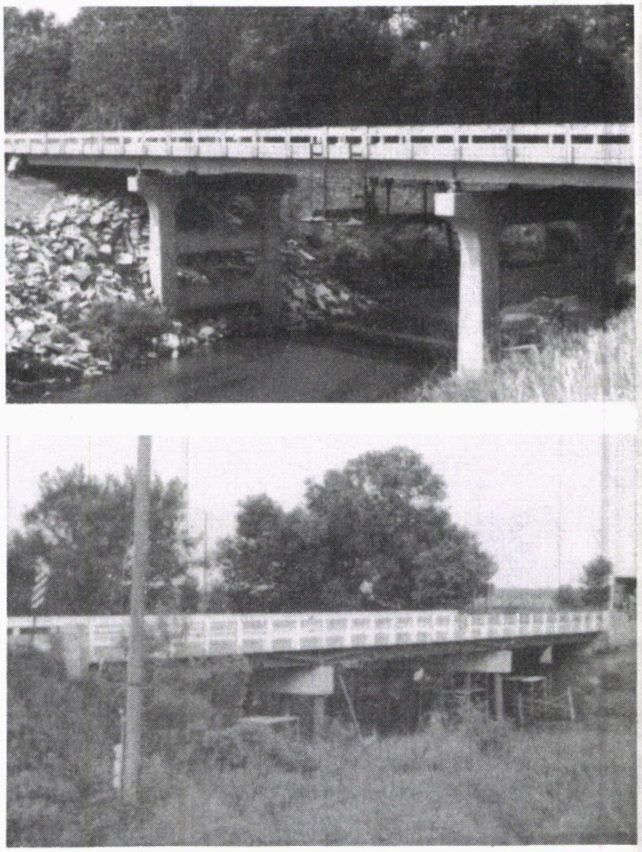


FIGURE 3 Photographs of strengthened bridges: *top*, Bridge 1; *bottom*, Bridge 2.

observed was 5.9 percent, and the greatest gain was 2.9 percent.

When the post-tensioning forces were removed from Bridge 1 during its retesting 1 year later, it was determined that the largest loss in post-tensioning force on one beam was 10.3 percent and that the largest gain on one beam was 3.8 percent. The average change was a 2.1 percent loss, which is slightly less than the 3.7 percent loss initially assumed in the design of the strengthening system.

Shown in Figures 4 and 5 are the bottom flange strains in an exterior beam (*top*) and an interior beam (*bottom*) resulting from the strengthening of Bridges 1 and 2, respectively. Also shown in these figures are the theoretical bottom flange beam strain variations (obtained from finite element analyses), assuming no restraint at the abutment supports. Except for a few locations, there is excellent agreement between the experimental and theoretical strains in both bridges. Apparent in both bridges is the presence of end restraint at the abutments. Although only the positive moment regions of Bridge 1 were post-tensioned, strain reduction in the negative moment regions is readily apparent. The magnitude of the post-tensioning forces applied to Bridge 1 was based on the desired strain reduction in the negative moment region. In other words, the positive moment regions in Bridge 1 are "overstrengthened." Since Bridge 2 had the superimposed trusses in the negative moment regions, it was possible to apply significantly smaller post-tensioning forces in the positive moment regions. The effect of the trusses is apparent when one compares the theoretical strain curves of the exterior beams in Bridges 1 and 2 [Figures 4 and 5 (*top*)].

In both bridges, a few strain gauges were installed on the guardrails. In some instances, guardrail strains of more than 50 microstrains/inch were recorded, which was a significant percentage of the beam strains that occurred when the load was near the guardrail strain gauges. In other words, the guardrails are carrying a portion of the applied truck loading.

Analysis

The analysis of continuous-span bridges due to the effect of vertical loads is addressed in the AASHTO *Standard Specifications for Highway Bridges.* Wheel load fractions are provided to aid the designer in determining the percentage of the vertical loads distributed to each of the bridge stringers.

Analysis of continuous-span bridges strengthened using post-tensioning and superimposed trusses presents a much more involved problem. The forces acting on the bridge in this case include axial forces and concentrated moments induced by the tendons at the various bracket locations, as well as vertical forces induced at the bearing points of the superimposed trusses. The lateral stiffness of the deck and the diaphragms results in the transfer of a significant portion of the strengthening forces from the strengthened stringer to other stringers. Forces and moments are transferred from one span to the others by the longitudinal continuity of the stringers and the deck. To the authors' knowledge, no practical procedures are available for computing the distribution of the previously described strengthening forces and moments throughout a given continuous-span bridge.

A finite element model was developed to analyze various bridges for different force conditions. Details of the model are provided by Klaiber et al. (*9*). The model was validated using the experimental data presented in this paper.

Development of Strengthening Design Methodology

The use of a finite element model for analyzing bridges under the effect of the forces from a strengthening sys-

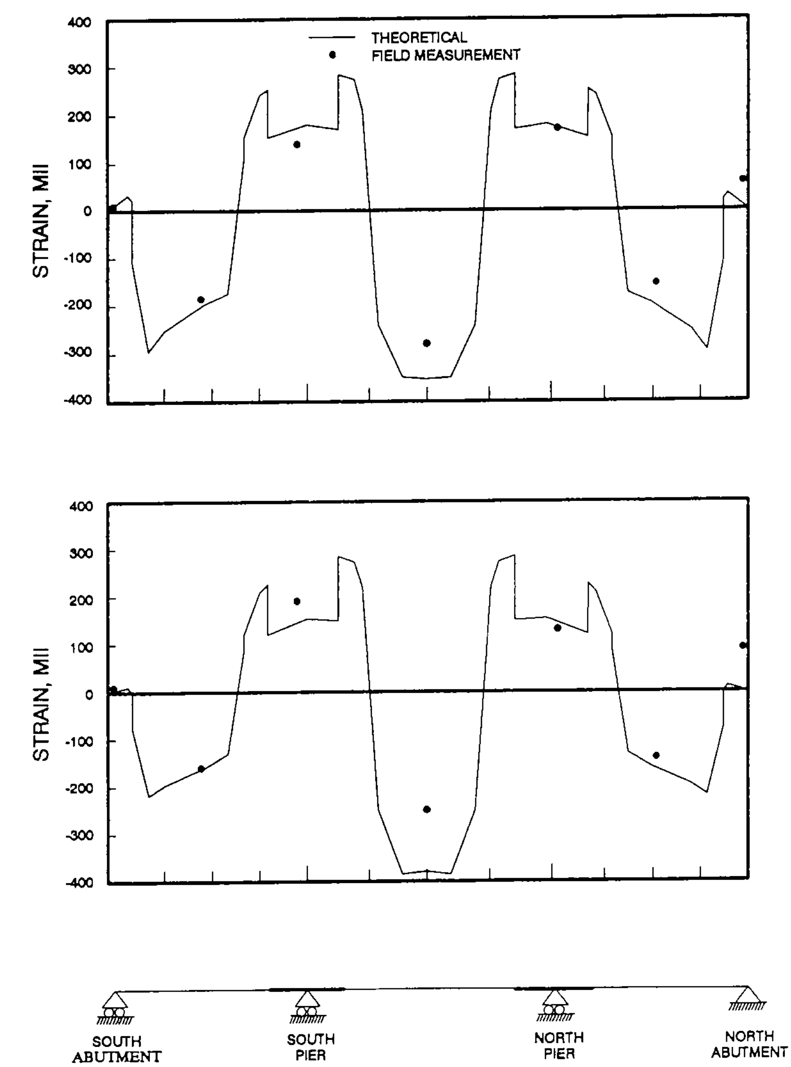


FIGURE 4 Bridge 1 bottom flange beam strains resulting from strengthening (all six stages applied): *top*, exterior stringer; *bottom*, interior stringer.

tem requires access to a large computer, a finite element solution package, and pre- and postprocessing programs. To simplify the design process for a typical continuous-span composite bridge, the authors developed a simplified design methodology for use by the practicing engineer. The development of the design methodology is explained briefly in the following paragraphs.

The design methodology is based on dividing the strengthening system into a number of separate schemes. In each scheme, the post-tensioning forces (or superimposed trusses) were applied so that symmetry was maintained. When designing a strengthening system, the designer can add a number of these schemes together to obtain the desired stress reduction at the various locations on the bridge. The possible strengthening schemes A through E are shown in Figure 6.

A representative example of the axial force and moment diagrams on the bridge stringers, as well as on the full bridge, due to strengthening Scheme A (post-tensioning of the exterior stringers of the end spans) is shown in Figure 7. These results were obtained from the finite element model developed; no vertical scale has been provided in Figure 7, as the comparison is independent of the magnitude of the strengthening forces. Note that the critical sections that have been identified (four for force distribution and six for moment distribution). The number and location of critical sections vary for the five schemes. For Scheme B, there are also four force and six moment critical sections. For Schemes C and D, there are three force and four moment critical sections. Because Scheme E applies only moment, there are only five critical moment sections.

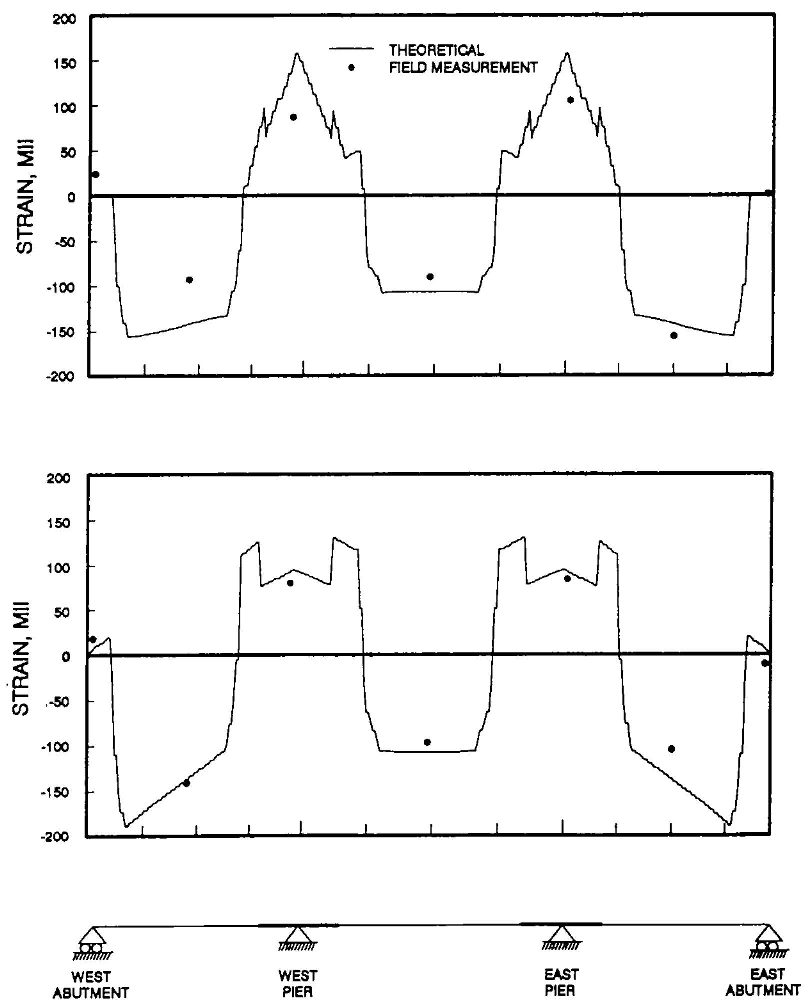


FIGURE 5 Bridge 2 bottom flange beam strains resulting from strengthening (all eight stages applied): *top*, exterior stringer; *bottom*, interior stringer.

For the development of the stringer force and moment diagrams on the stringers without using finite element analysis, several approximations were made to various force and moment diagrams that resulted from the finite element analysis. The first approximation is that the moment on the total bridge section at any section can be determined by analyzing the bridge as a continuous two-dimensional beam. The strengthening forces on the idealized beam are taken equal to the total strengthening forces on all bridge stringers, and the beam moment of inertia at any location is taken equal to the total moment of inertia of the bridge section at this location.

To verify this assumption, several actual bridges were analyzed using the finite element analysis and the idealized beam model; the results from the two analyses were then compared. An example of this comparison (for Scheme C) is shown in Figure 8. As illustrated, the total moments along the bridge obtained by both methods are very close. In analyzing a number of bridges strengthened with the different schemes, it was determined that the difference between the moments computed using the two methods did not exceed 7 percent at critical locations.

Another approximation (made for each of the strengthening schemes) was that the force and moment diagrams for the individual stringers were idealized by straight line segments between the critical sections. The locations of the critical sections were selected to describe accurately the actual diagrams. This straight-line idealization allows the designer to reconstruct the axial force and moment diagrams along the stringers once the magnitudes of force and moment are known at these critical sections.

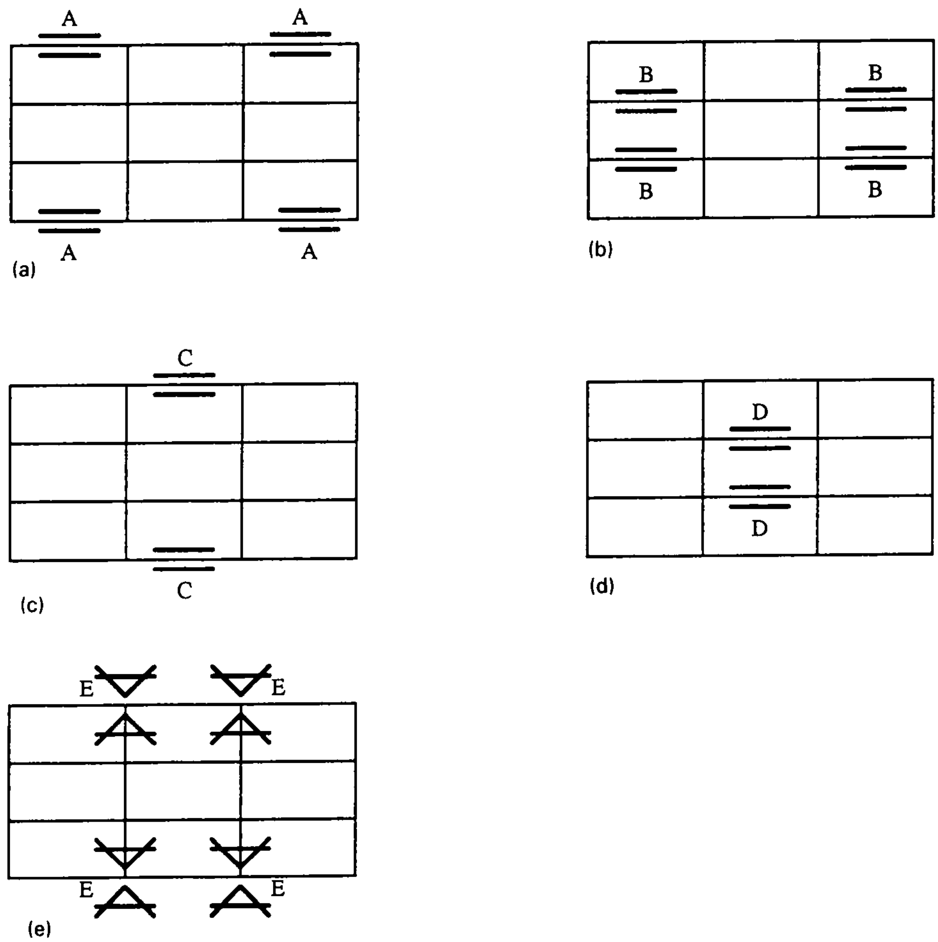


FIGURE 6 Various locations of post-tensioning and superimposed trusses: (*a*) Scheme A: post-tensioning end spans of exterior stringers; (*b*) Scheme B: post-tensioning end spans of interior stringers; (*c*) Scheme C: post-tensioning centerspans of exterior stringers; (*d*) Scheme D: post-tensioning centerspans of interior stringers; (*e*) Scheme E: superimposed trusses at piers of exterior stringers.

Definition of Force and Moment Distribution Fractions

In recognition of the complexity of finite element analysis, a simplified approach was developed that uses force and moment fractions to distribute the strengthening forces to the various stringers. The force (or moment) distribution fractions at the critical sections are defined as follows:

Force (or moment) fraction at (*i*)

$$= \frac{\text{axial force (or moment) in strengthened stringer at } (i)}{\text{total axial force (or moment) on bridge at } (i)}$$

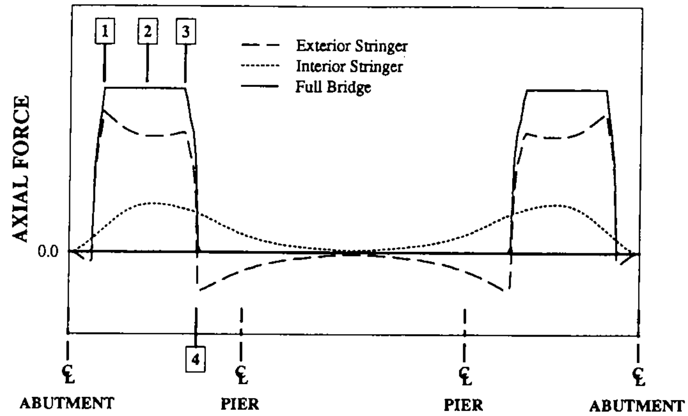
where (*i*) indicates the critical section.

So that regression formulas for the force and moment fractions could be developed, several bridges were modeled and analyzed using the finite element model mentioned previously. The 2,400 bridges analyzed included standard Iowa DOT bridges and nonstandard bridges.

All bridges were analyzed with the tendons positioned at an elevation of 88.9 mm (3½ in.) above the top surface of the bottom flange. The effect of changing the elevation of the tendons within a range of 76.2 to 127.0 mm (3 to 5 in.) was investigated and found to have a minimal effect on the distribution fractions. Thus, the force and moment fractions determined in this investigation are valid for any elevation above the bottom flange in this range.

In each of the 2,400 analyses, force and moment fractions were determined at the critical sections using the finite element results. These values were used in developing the design distribution fractions at these sections.

A sensitivity study was conducted to determine the parameters that significantly affected the force and



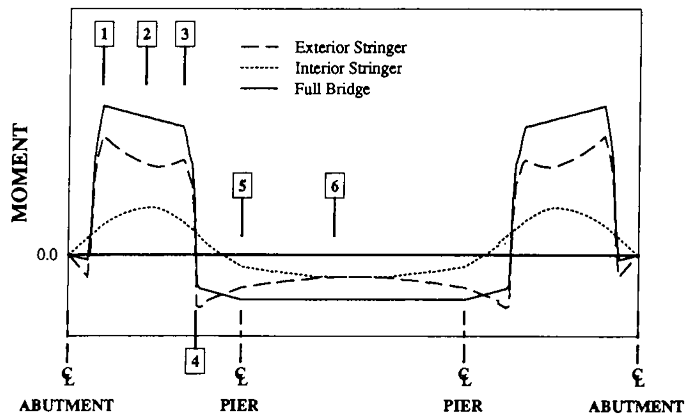


FIGURE 7 Locations of distribution fractions, Scheme A: *top*, axial force diagram; *bottom*, bending moment diagram.

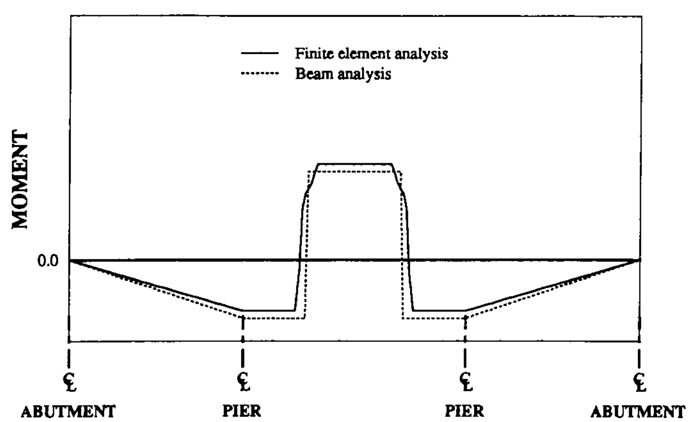


FIGURE 8 Total moments on bridge section, Scheme C.

moment fractions. These parameters included bridge length, angle of skew, ratio of end span to centerspan length, deck thickness, stringer spacing, stringer moments of inertia (composite and noncomposite), and the ratio of the post-tensioned portion of the span to the span length for the various strengthening schemes. To simplify the force and moment formulas, the bridge variables were included as dimensionless parameters.

Limits have been provided for the variables and for the force and moment fractions computed using the regression formulas. Variables and the computed force and moment fractions of the Iowa standard V12 and V14 series bridges are well within the established limits. For bridges with measurements that vary significantly from those of the standard bridges, the formulas do not give accurate force and moment fractions. In these cases it is strongly recommended that a finite element analysis be performed to determine the axial forces and moments in the bridge stringers.

As described, several approximations have been made to provide a simplified procedure for determining the response of the bridge to the strengthening system

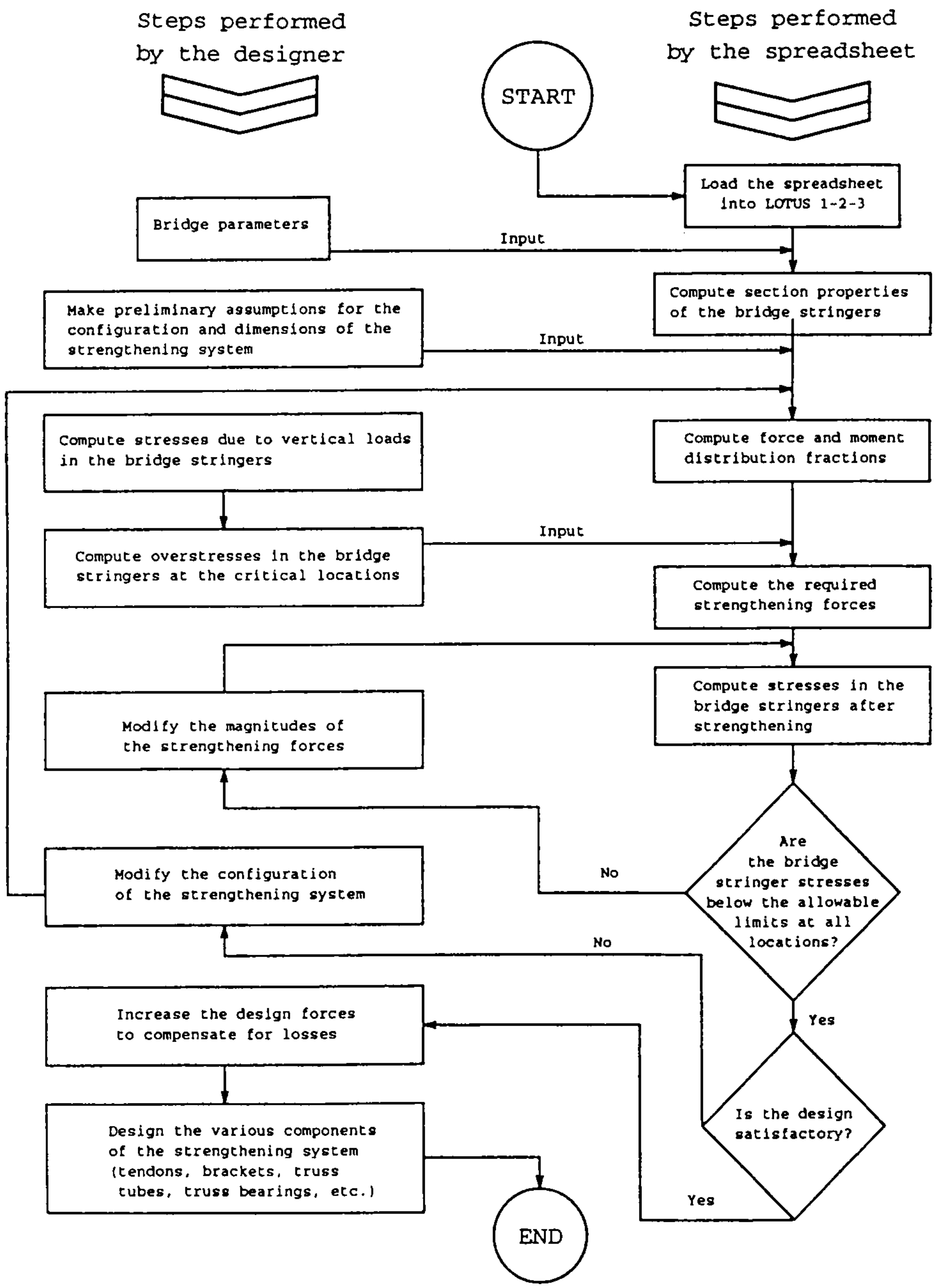


FIGURE 9 Design procedure for strengthening system.

and for designing the required strengthening system. Although the errors resulting from these approximations are small, their collective effect might be significant in some cases. There are several potential sources of error in the design methodology developed: analysis of a given bridge as continuous beams with variable moments of inertia, idealization of force and moment fractions, force and moment fractions, and post-tensioning losses.

Because of the complexity of the design procedure and the large number of formulas, it is difficult to account for the errors in the regression formulas using the error limits corresponding to each formula. Thus, it is recommended to increase all strengthening forces by a conservative 8 percent. The designer, however, needs to check that the stringer stresses based on the original strengthening forces and on the increased strengthening forces are both within the allowable limits.

Recommended Design Procedure

The various steps required in the design of a strengthening system for a typical continuous-span composite bridge are described briefly. For a detailed explanation of the design process, the reader is referred to Klaiber et al. (*10*).

A Lotus 1-2-3 spreadsheet was developed to assist the engineer with designing the required strengthening system. With each section of the spreadsheet, a "Help" area has been provided for guidance. The spreadsheet calculates the required strengthening forces and provides the designer with the final stress envelopes of the bridge stringers. Figure 9 illustrates the steps of the design procedure—those steps that are completed by the spreadsheet and those that must be completed by the designer.

Listed in the following is the procedure for determining the configuration of the strengthening system and the tendon forces required to strengthen a given three-span continuous bridge. Steps 1, 3, 4, and 5 must be completed by the designer; all the other steps, which tend to be more complex and time-consuming, are performed by the spreadsheet.

1. Determine section properties of the exterior and interior stringers for the following sections: (a) steel beam, (b) steel beam with coverplates, (c) composite stringer (steel beam and deck), and (d) composite stringer with coverplates (steel beam, coverplates, and deck).
2. Determine all loads and load fractions for exterior and interior stringers for (a) dead load, (b) long-term dead load, and (c) live load and impact.
3. Compute the moments and stresses in the exterior and interior stringers due to (a) dead load, (b) long-term dead load, and (c) live load and impact.
4. Compute the overstresses at the critical section locations to be removed by strengthening.
5. Make an initial assumption on the strengthening schemes required for tendon lengths and bracket locations. These values are used to compute the initial force and moment fractions.
6. Determine the post-tensioning forces and the vertical truss force that produce the desired stress reduction at the critical sections.
7. Check the final stresses in the exterior and interior stringers at various sections along the length of the bridge; one should especially check the stresses at the coverplate cut-off points, bracket locations, and truss bearing points.
8. Increase the strengthening design forces by 8 percent to account for post-tensioning time losses and errors due to approximations in the design methodology.

Summary and Conclusions

Field tests have been performed on two strengthened bridges to determine the effectiveness of the post-tensioning and superimposed truss concepts. The strengthening system that was designed and installed behaved generally as predicted from analytical results. A finite element model simulating the bridge and the strengthening system was validated from the field test results of the two bridges. A design methodology using this model was developed so that practicing engineers can design a strengthening system for similar continuous-span bridges. Both strengthening schemes were determined to be practical, cost-effective strengthening techniques. The design methodology that uses a computer spreadsheet is relatively simple to use and provides the required strengthening forces.

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The opinions, findings, and conclusions expressed herein are those of the authors and not necessarily those of Iowa DOT or the Highway Research Advisory Board.