3. The proposed posting policy provides flexibility by allowing the engineer to perform a Level 2 analysis or inspect bridges at increased frequency.
4. The proposed posting policy protects the state's capital investment in its bridges. This is mainly a result of the protection from overloads provided by posting policy.
5. The proposed posting policy protects the public safety for the same reason as the aforementioned.

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Prescription for Steel Girder Bridge Rehabilitation

RAYMOND E. DAVIS, RAMIN RASHEDI, and KHOSROW KHOSRAVIFARD

ABSTRACT

Increases in design or permit live loads, coupled with material deterioration, currently require rehabilitation or replacement of many bridges. Current AASHTO live-load distribution criteria, originally developed for fully loaded structures, result in ultraconservatism when applied to overload vehicles occupying only one or two lanes. Indiscriminate use of these criteria may suggest needless rehabilitation. Application, even if time-consuming, of currently available, sophisticated, computerized analyses that treat such partial-width loadings accurately may effect significant economies by demonstrating structural adequacy. Plans for overlayment on a deteriorated concrete deck on steel girders at a California site and upgrading for a new live-load permit vehicle, based on AASHTO distribution criteria, will require auxiliary steel flanges, introduction of composite behavior and web posttensioning to carry increased dead and live loads. A grillage analysis with the CURVBRG computer program suggests questionable need for posttensioning. Further analyses with FINPLA and STRUDL finite-element programs demonstrate a method for assessing web stresses at the posttensioning brackets.

In 1975 the California Department of Transportation (Caltrans) upgraded its specifications for bridge loadings with the objective of producing initial designs commensurate with later ratings for overload (1). The revisions introduced a modular design vehicle called the Permit- or P-series vehicle, significantly longer and heavier than H-series vehicles used previously. Unlike H-series trucks, P-series vehicles are used singly or in conjunction with an HS20 vehicle in an adjacent lane, and load factors are significantly lower than H-series factors.

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As a result of the new, heavier loadings, a program was initiated in which existing bridges on the state highway extra legal load (SHELL) system are screened to determine structural adequacy. For steel girder bridges, this screening utilizes the CURVBRG program, written by Mondkar and Powell at the University of California, Berkeley (2), and implemented into the Caltrans operating system by the Structural Research Unit in 1974 (3). Bridges that are proven to be structurally inadequate in the screening process must be either replaced or rehabilitated, the latter frequently by applying posttensioning forces to webs or flanges, by introducing composite behavior, or both.

Mancarti has discussed (4) current plans for major rehabilitation of this nature at the Yuba Pass overhead and separation. This work involves a number of phases including: (a) replacement of bearings; (b) addition of transverse and longitudinal stiffeners; (c) addition of auxiliary bottom flanges; (d) removal and replacement of existing curbs and railings; (e) scalping a thin, upper layer of the deck, placement of shear connectors to introduce composite action, and subsequent thickening of the deck; and (f) addition of reinforced steel brackets and posttensioning tendons to the sides of the girder webs. AASHTO-specified live-load distribution was used in determining requirements for strengthening.

Mancarti noted that posttensioning forces would be significantly larger than any applied previously in this manner in California, resulting in a request that the Structural Analysis Unit investigate distributions of prestress forces into webs in the vicinity of the brackets. The study is detailed in this paper.

DESCRIPTION OF PROTOTYPE

Figure 1 shows a general plan elevation view of the left structure. The first (simple) span is already composite, Spans 2 through 4 are not. Plans call for partially remedying this situation in the suspended sections of Spans 2 and 4 by removing portions of the deck down to the upper steel flanges and affixing shear connectors to produce composite action. An enlarged elevation showing posttensioning details is shown in Figure 2. Figure 3 shows the posttensioning

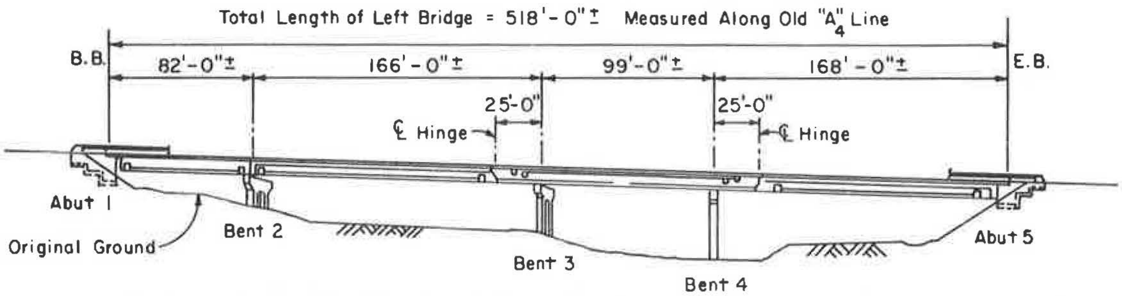


FIGURE 1 Elevation—Left bridge.

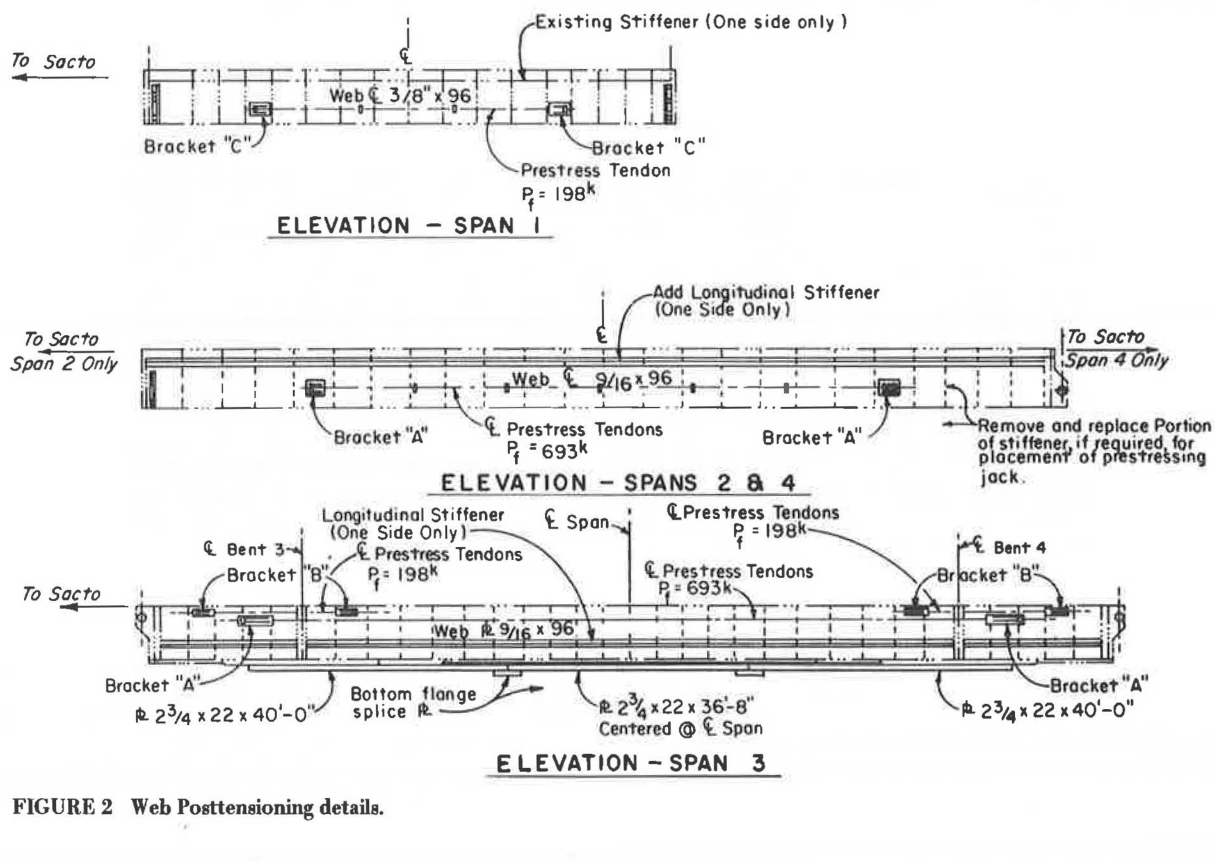
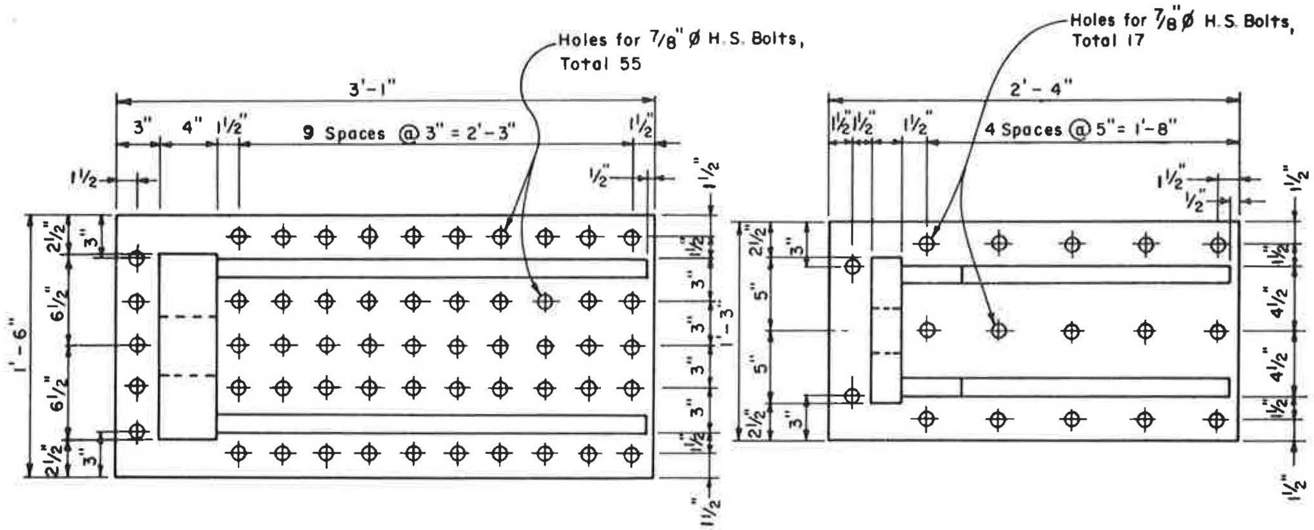


FIGURE 2 Web Posttensioning details.



ELEVATION – BRACKET "A"

ELEVATION – BRACKET "B" & "C"

FIGURE 3 Posttensioning brackets.

brackets. The section between hinges, including Span 3, is controlled by negative moment and will remain noncomposite.

PROPOSED METHOD OF ANALYSIS

Available Programs

The current repertoire of programs available to the Structural Analysis Unit includes no single code, with the possible exception of STRUDL, capable of treating the entire problem. However, the following codes contain facilities that can be used to significant advantage in the analysis.

CURVBRG

This program provides a powerful tool for treatment of steel-plate girder bridges with concrete slabs. It is one of the few available programs for curved girder analysis that treats warping torsion. The program is not restricted to curved girders but may treat straight ones or those with angle points in plan.

This code treats articulation (hinging) of girders and uses a generator that greatly facilitates application of live loads to the structure. Output comprises envelopes of stresses, moments, shears, displacements, and so on. Construction stages may be specified so that scaling factors can be applied to physical properties of component materials that differ from stage to stage.

Load trains of wheel reactions at proper spacings may be applied and stepped along any user-specified lines of nodes. Various load cases can be superposed as load combinations with simultaneous application of multiplying factors that may include impact factors, load factors, and so on.

Use of a very simple grillage model minimizes the expense of program usage, even for large and complex structures; however, formulating input can be tedious, and determination of representative construction stages can be complex, in a problem such as this one. The analysis does not treat loads acting in horizontal planes, and, therefore, the important posttensioning forces. Nonetheless, CURVBRG will serve a useful function in the analysis.

CURVBRG's compilers have established favorable comparisons between experimental and program results from a number of curved girder model tests. In addition, field tests of six lightly instrumented plate girder structures by Caltrans' Office of Structures Maintenance, a literature search, and subsequent CURVBRG analyses of tested structures by the Structural Analysis Unit have yielded some excellent correlations between computed and measured strains (details to be published).

FINPLA

FINPLA (5) uses a finite-element analysis to treat prismatic folded-plate structures with eccentric plate and beam elements. The program was extensively tested by the senior author in 1968 on a steel box girder bridge over the Sacramento River at Bryte Bend (6,7) where excellent correlations were established between program-predicted strains and those measured by 1,200 strain-gauge circuits attached to steel elements or embedded in the composite, concrete deck slab. In these tests, the program's power to analyze steel box girder behavior was amply demonstrated, and the influences of longitudinal and transverse stiffening elements, diaphragms, frame elements, and so on, were considered. These capabilities strongly suggest use of FINPLA in this study, especially because it can readily treat forces in horizontal planes such as web posttensioning.

As far as is known, the code was not designed for analysis of open-web type structures and has not previously been used as such. It cannot treat the structure's articulation directly and there is no live-load generator, therefore, no convenient method of ascertaining critical live-load positioning.

STRUDL

STRUDL provides a finite element capability and undoubtedly is capable of treating the whole program but would be somewhat cumbersome in the total analysis of a structure and loadings as complex as these. More details of the program's use are provided.

General Plan of Analytical Procedure

The ultimate plan of approach involved using all three programs according to the following steps.

1. Make initial stress analysis using CURVBRG. To assess complete stress fields, begin from inception of bridge construction in 1961, considering all pertinent construction stages.
2. Because CURVBRG is a relatively inexpensive program to run, examine all potentially critical live-load conditions, including HS-series trucks, HS-series lane loadings, and P-series trucks in whatever combinations design specifications require. The factoring facility in the section on load combination permits convenient consideration of unfactored and factored (ultimate load) cases.
3. Compare critical load stresses with allowable stresses to confirm need for posttensioning.
4. Use CURVBRG to determine which type of live loading produces critical envelope stresses. Because program output comprises envelopes of stress maxima without preserving vehicle positions producing such maxima, make a guess pertinent to two or three potentially critical positions, and rerun CURVBRG with these input as single loadings to determine vehicle types and positions producing stresses comparable in magnitudes to envelope stresses. Record output displacements of each girder at hinges for dead and critical live loads for the longitudinal section from Bent 2 to the first hinge point. In the analysis of the longitudinal section between hinges, including Span 3, use output girder shears from CURVBRG to determine reactions on the cantilevered girders for subsequent input to FINPLA in lieu of displacements.
5. Establish input for FINPLA stiffness matrices using output displacements from CURVBRG for Span 2 up to the first hinge as boundary conditions at hinges. Note that separate stiffness matrices are required for dead and live loadings. For the section between hinges, include hinge reactions in the load vector.
6. Input dead and critical live loadings to FINPLA in conjunction with appropriate stiffness matrices.
7. Substructure Spans 1, 2, and 3 to FINPLA blocks containing posttensioning brackets and bounded by transverse sections spaced at 48 in. for the small brackets, and 52 in. for the large brackets.
8. Determine global X- and Z-displacement fields at FINPLA sections comprising substructure boundaries over web depths and across substructure (block) widths.
9. Use the POLYFIT program to establish polynomial regression functions for displacement fields in the FINPLA global X- and Z-directions at each of four boundaries for each substructured mesh.
10. Subdivide the two substructures into finite-element meshes with internal element dimensions comparable in size to bolts securing posttensioning brackets to the webs, using STRUDL CSTG constant-strain triangles and isoparametric IPLQ quadrilaterals with two (x and y) degrees of freedom at each node.
11. Subject substructure meshes to three, plane stress finite-element analyses with STRUDL:
   a. First, designate nodes at four boundaries as rigid supports, assume the posttensioning force to be distributed equally to each bolt, apply as uniformly distributed pressures on faces of small elements (1-in. squares) adjacent to holes.
   b. Second, apply no posttensioning forces but allow boundary nodes to displace as determined by FINPLA in Steps 8 and 9, because of dead, live, and impact loadings and posttensioning forces. Use POLYFIT regression functions established through a relatively small number of FINPLA nodes distributed over FINPLA section (web) depths and FINPLA block widths to assess displacements at all nodes comprising substructure mesh boundaries.
   c. Third, combine principal stress fields from a and b to assess total stress fields resulting from application of posttensioning forces on a mesh with boundaries free to move in accordance with restraints imposed by the surrounding web.

DETAILS OF ANALYTICAL PROCEDURES

CURVBRG Analysis

Despite CURVBRG's adaptability, analyses for various construction stages can be complex. The program permits use of successive construction stages to simulate, for example, erection of steel girders, pouring a concrete deck slab, placement of barrier curbs, application of live loads, and so on. Elastic properties (moduli, Poisson's ratios) and densities of each component material are input separately. In general, augmentation of concrete slab density is required because internal computation of selfweight (dead load) of deck elements will be based on thicknesses and effective widths input for girder section properties, and these dimensions may vary significantly from actual ones. Density augmentation may also be used to account for dead loads of steel stiffeners, cross-frames, wind bracing, and so on, which would not otherwise be included in dead-load computations for the steel girders.

Contributions to structural behavior of various component materials during different construction stages may be assessed by internal application of scaling factors used to reduce moduli. Typical input for material properties may be those given in Table 1. Note the requirement for unit consistency.

TABLE 1 Typical Input for Material Properties in Deck Construction

Material	Elastic Modulus (ksi)	Poisson's Ratio	Density (k/in.^{3})
Steel	29,000	0.285	0.0002836
Concrete (composite)	3,000[a]	0.180	0.0000868[b]
Concrete (noncomposite)	1	0.05	0.0000868[b]

[a]More accurate figures may be obtained from American Concrete Institute formulas.
[b]Concrete densities will usually be augmented as noted previously, and different material types will be required for exterior and interior girders because of differing augmented densities.

Because of stress relaxation in the concrete deck as a result of creep, it is customary to reduce the elastic modulus of deck concrete, usually by a factor of 4, for long-term loads such as barrier curbs, overlays, wearing surfaces, and so on.

The program accounts for effects of transverse slab strips on load distribution and production of force coupling in the slab, diaphragms, wind bracing, and so on; therefore, it will usually be necessary to specify two separate types of concrete for a noncomposite deck slab that contributes to transverse, but not to longitudinal, strength. Table 2 may be cited as typical input for the aforementioned materials in a simple case.

TABLE 2 Typical Input for Materials for Deck Construction

Material	Construction Stage	Activity	Scaling Factor
Steel	1	Erect bare steel	1
	2	Pour wet concrete deck slab	1
	3	Place barrier curbs	1
	4	Apply line loads	1
Deck concrete	1	—[a]	0
	2	—[a]	0
	3	—[a]	0.25
	4	—[a]	1

[a] Activities 1-4 for deck concrete same as those for steel.

With the Table 2 factors in mind, behavior of the prototype with the following construction stages should be considered.

• Stage 1: Erect bare steel (1961). The steel is given a scaling factor of 1, the 9 5/8-in. concrete deck, 0.

• Stage 2: Pour 9 5/8-in. deck slab (1961). Scaling factors are the same as in Stage 1 because concrete is still noncomposite in all spans for this load.

• Stage 3: Place existing barrier curbs (1961). Steel scaling factor is 1, deck concrete is 0.250 for long-term load in Span 1 and 0 in noncomposite Spans 2, 3, and 4.

• Stage 4: Place auxiliary steel flanges in Span 3, shear connectors in Spans 2 and 4 (1985). The dead loads of these flanges are carried by the steel girders only. Concrete has a scaling factor of 0. The scaling factor for existing steel is 1, for new steel, 0, and a separate material must be specified. For all subsequent stages, the auxiliary flange steel will be characterized by a scaling factor of 1; however, the new and existing steel flanges will be working at different stress levels, the existing flanges exhibiting stresses caused by loads of original components, the new flanges, those of components removed or added in subsequent stages.

• Stage 5: Remove 2 in. of deck and existing barrier curbs. Span 1 is now acting compositely with a 7 5/8-in. slab, and loadings are really short-term, and concrete scaling factors are somewhat questionable. In Spans 2, 3, and 4, the concrete deck is noncomposite, the scaling factor for concrete, 0.

• Stage 6: Add 6 3/8 in. of concrete deck in Span 1. In Span 1, the existing 7 5/8-in. deck is acting compositely to support dead load of the additional 6 3/8 in. of wet, strengthless concrete. Loading is long term and a scaling factor of 0.250 should logically be used for the 7 5/8-in. layer and 0 for the 6 3/8-in. layer. There are now two layers of concrete in the deck acting at different stress levels, the upper essentially unstressed and the lower stressed as a result of dead load of preexisting components plus the new layer. Actually, as a result of shrinkage, the upper layer will in time develop tensile stresses in itself, and the lower steel flanges will develop compressive stresses in the existing concrete layer and upper steel flanges.

• Stage 7: Add 6 3/8 in. of wet concrete to Spans 2, 3, and 4. Neither the 7 5/8-in. nor 6 3/8-in. layers of concrete acts compositely at this time; concrete scaling factor is 0, steel factor is 1, and the dead load of concrete is carried only by the steel girders.

• Stage 8: Place new barrier curbs. This loading is long term. All spans now have a 14-in. deck but Span 3 remains noncomposite with a scaling factor of 0. Concrete in Spans 1, 2, and 4 is composite with a scaling factor of 0.250.

• Stage 9: Apply live loads. These are short-term loadings. Deck concrete in Spans 1, 2, and 4 has a scaling factor of 1, and in noncomposite Span 3 a scaling factor of 0.

Output CURVBRG stresses for the critically loaded girders in Spans 1, 2, and 3 are plotted in Figures 4, 5, and 6. Except in Span 1, these stresses are produced in exterior girders under influence of ec-

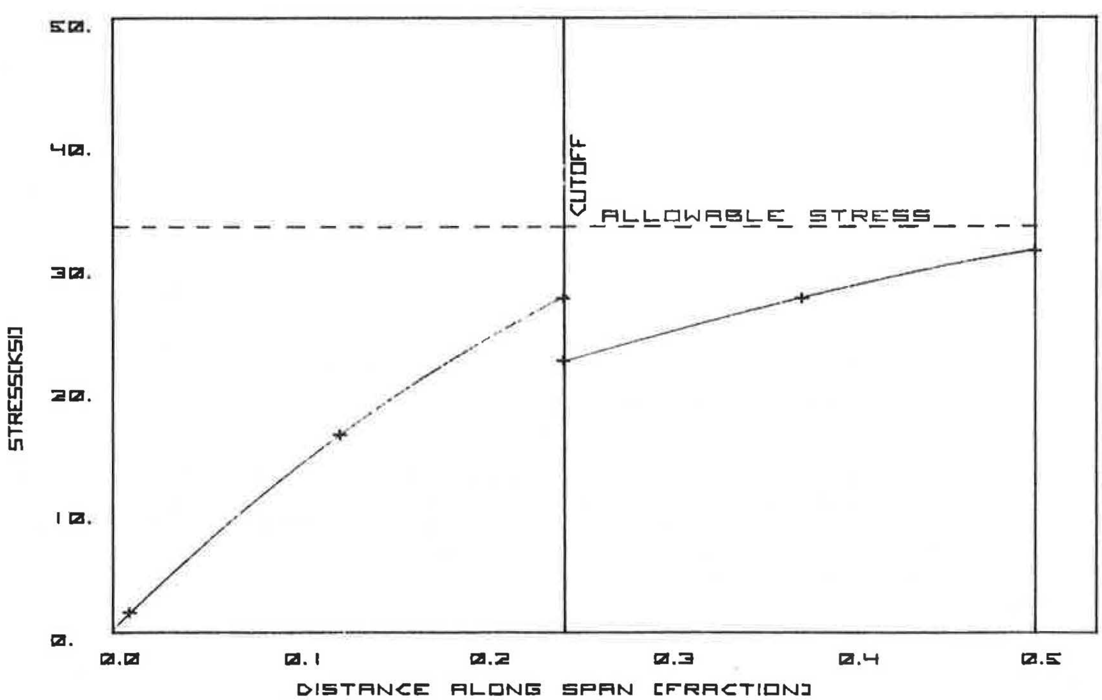


FIGURE 4 CURVBRG stresses Yuba Pass Overhead—Span 1.

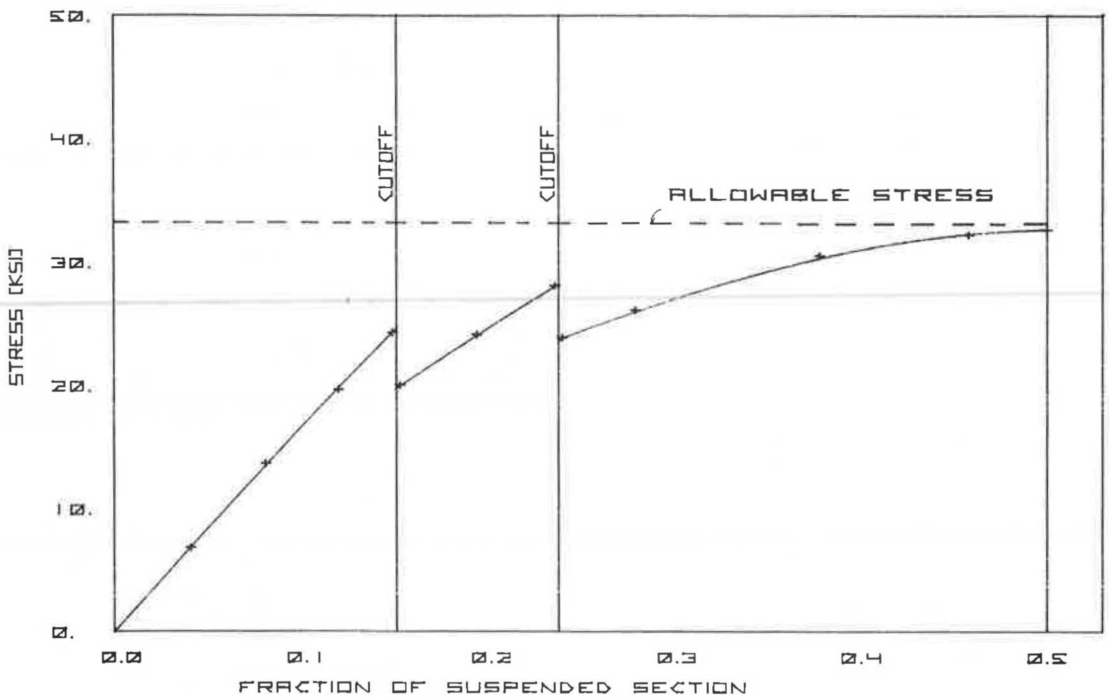


FIGURE 5 CURVBRG stresses Yuba Pass Overhead—Span 2 (suspended section).

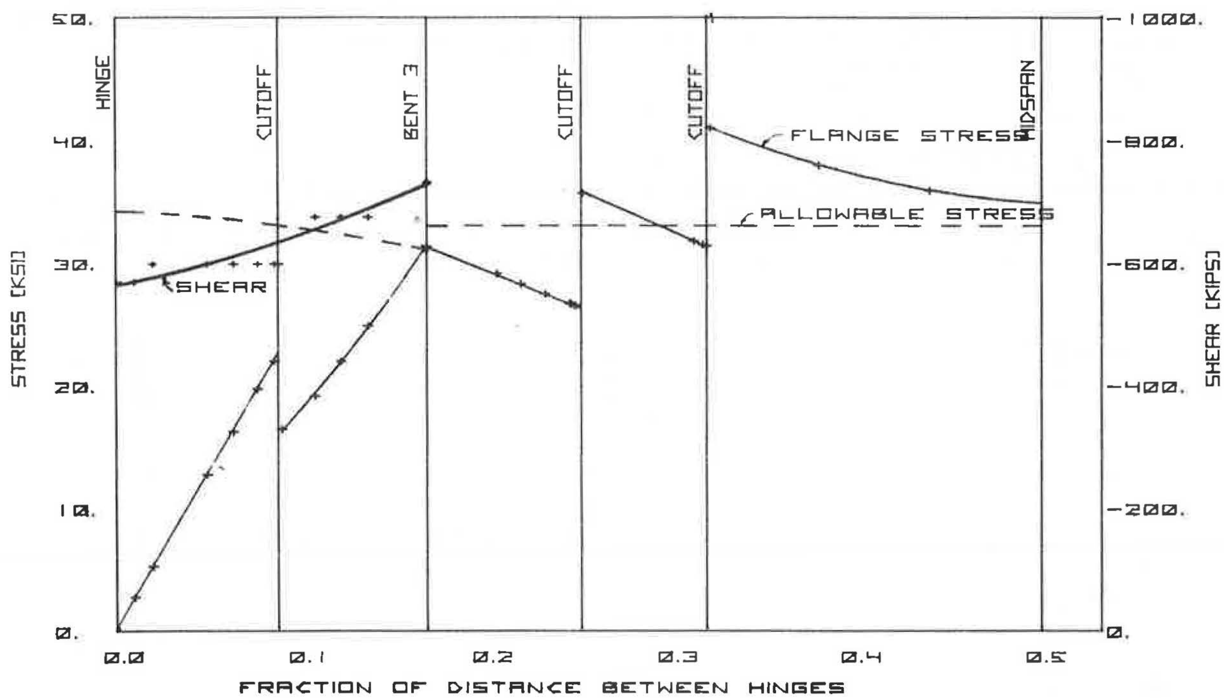


FIGURE 6 CURVBRG stresses Yuba Pass Overhead—Span 3 and cantilevers.

centrically placed P-series trucks, with H-series trucks in the adjacent lane, under which conditions total stresses from factored loads would be less than the 33-ksi yield stress in Spans 1 and 2 and at Bent 3. Stresses exceeding yield will occur at top flange cutoff points in Span 3.

FINPLA Analysis

The second phase of analysis utilized the finite element program, FINPLA, the primary use of which was to determine local effects of dead and live loads and general effects of posttensioning forces in the form of differential displacements at boundaries of blocks containing posttensioning brackets.

Loadings were of three types:

1. Dead loads of plate elements were expressed as regular surface loads (P/Plate area), which are calculated easily as products of plate thicknesses and material densities.
2. Loads of barrier curbs, longitudinal stiffeners, and other loads distributed longitudinally were

treated as distributed line loads, in kips per inch, acting at appropriate nodal joints between designated sections.

3. Loads of transverse stiffeners, wheel reactions, and so on, were treated as concentrated nodal joint loads, which, in the program, are entered as distributed line loads with the same beginning and end sections.

STRUDL Analysis

The total posttensioning force was divided equally among the bolts and applied as uniform pressures acting on faces of elements adjacent to the holes.

STRUDL output plots for a large bracket in Span 3 are shown in Figures 7 and 8, the former for major principal (maximum tensile) and the latter for minor principal (maximum compressive) stresses due to unfactored prestress forces and factored dead, live, and impact loads, respectively. Plots of the large bracket for Span 3 demonstrate that localized stresses significantly in excess of 33-ksi yield stresses are calculated in the vicinity of the bolts, some as large as 44 ksi. Major tensile stresses are concentrated along the column of bolts nearest the bracket edge in a direction opposed to that of the posttensioning force, while major compressive stresses are concentrated on the opposite row of bolts.

Several factors should be noted in connection with these plotted stresses:

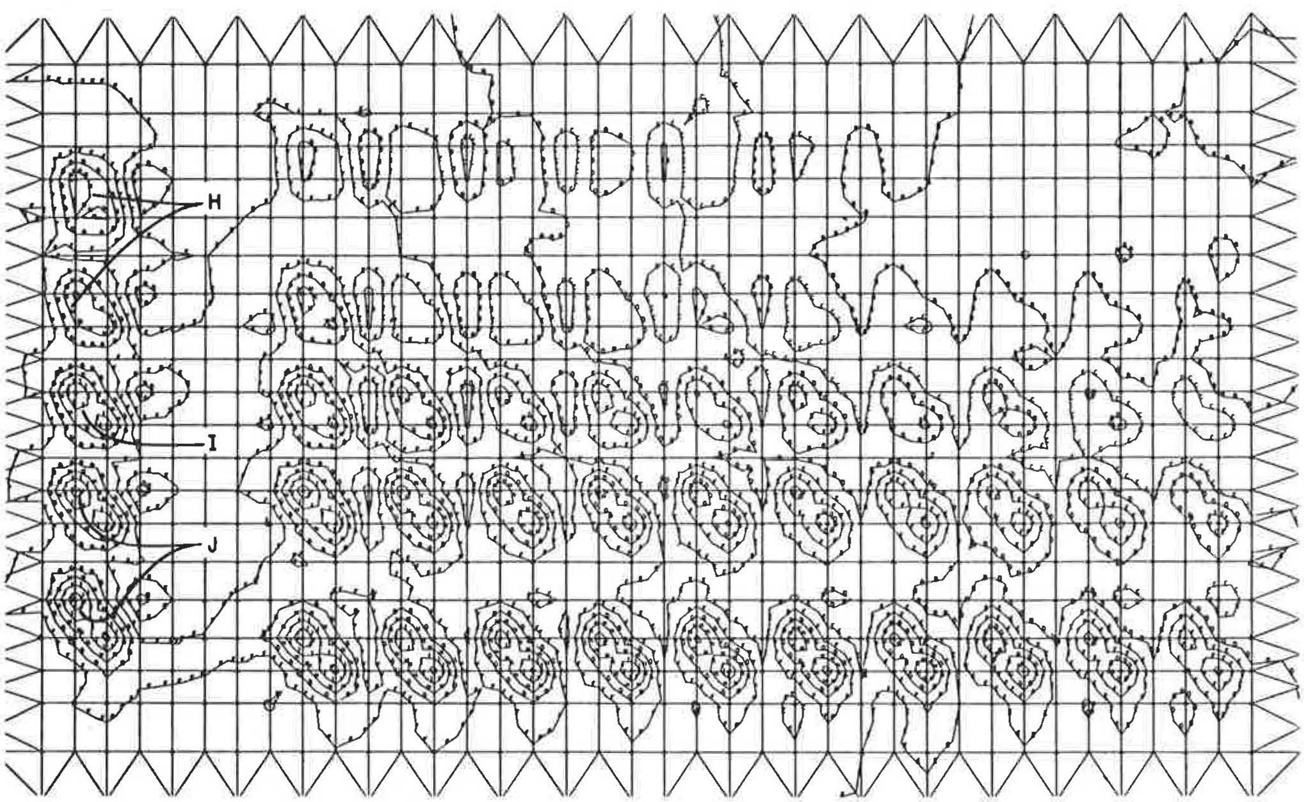
1. No attempt has been made to include effects of nonlinear material behavior. Except for the influence of strain hardening, steel cannot be expected to sustain stresses much in excess of yield stress; therefore, the yield stress contour might be expected to spread over a broader area than indicated by the plots.

2. Whereas the designer specified the highest level of friction bearing between brackets and webs, included in this analysis was the tacit assumption that the bearing was frictionless and that the posttensioning force was distributed equally to the bolts, with no beneficial effects from clamping. If clamping had been assumed to be 100 percent efficient in the analysis, it would have been reasonable to spread the prestress force over the entire bracket depth, almost certainly eliminating the large concentrations of stress, but not necessarily eliminating widespread yielding along bracket boundaries.

3. Plots included here are for the most critical combinations of loadings, with live loading included, and are, therefore, not necessarily the most critical without live loading in all cases. Therefore, the girder analyzed in all cases has been that with maximum tensile stresses due to dead, live, and impact loadings. Tensile stresses on the un-prestressed end of the bracket should be the largest that can be expected in any girder, but compressive stresses at the other end of the bracket would be less than those if live loads were not included.

Potential Design Revisions in Span 3

CURVBRG results indicate need for posttensioning only in Span 3, where calculated posttensioning force required to bring CURVBRG calculated overstress within the yield stress of the web steel is 398 kips, significantly less than the 693-kip value indicated by the AASHTO load distribution specifications.



H = 30.8 ksi , I = 35.2 ksi , J = 39.6 ksi   Contour Interval = 4.4 ksi

FIGURE 7 Major principal stress.

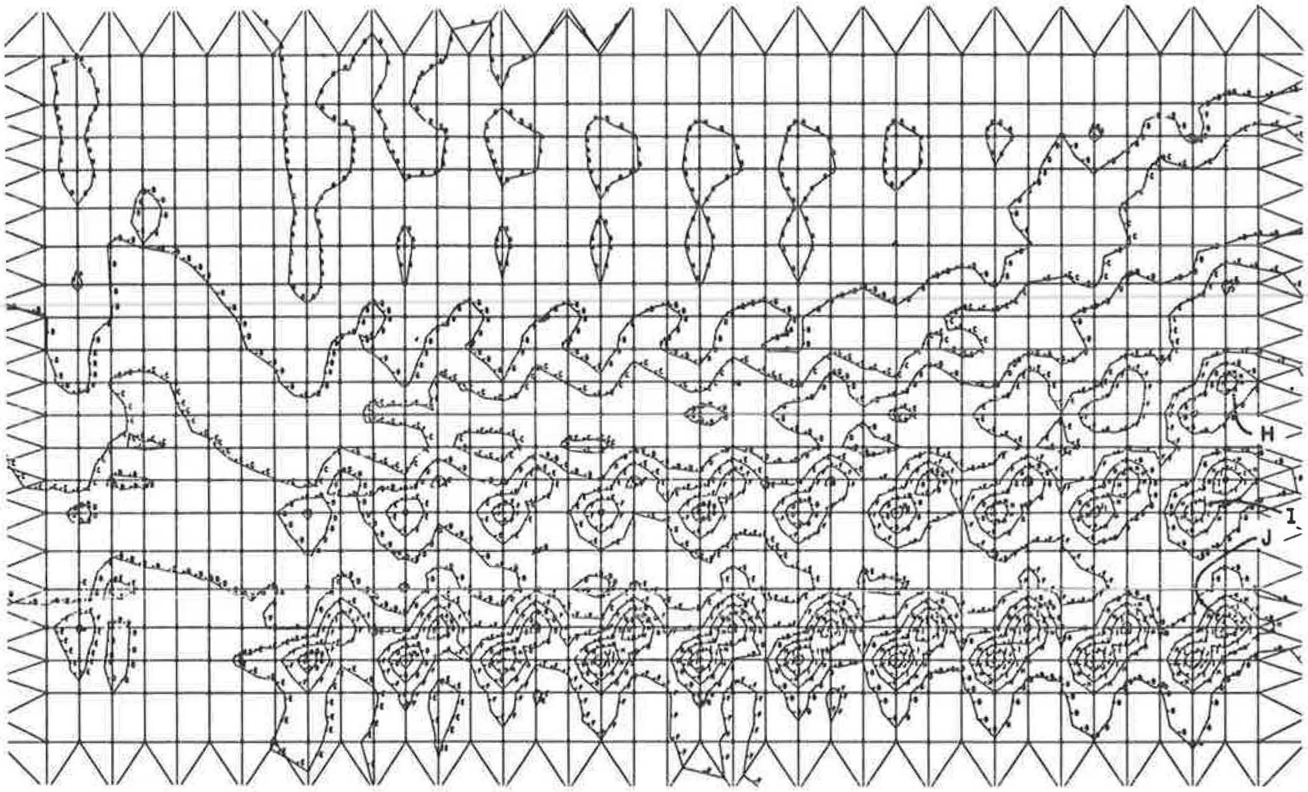


FIGURE 8 Minor principal stress.

SUMMARY AND CONCLUSIONS

Theoretical analyses (8) were made of the Yuba Pass overhead and separation, an existing plate girder and concrete deck structure, to assess need for posttensioning girder webs as recommended by the Caltrans' Office of Structures and Design, in conjunction with other rehabilitation for recent Caltrans live-load revisions and to remedy a deteriorated deck. Three main programs were used in the analyses, two of which (CURVBRG and FINPLA) emanated from the Caltrans research project Analysis, Design and Behavior of Highway Bridges (9).

CURVBRG, a grillage analysis developed for curved (or straight) plate girder bridges was used to

1. Assess maximum stresses resulting from complex construction phases from initial construction through repair phases exclusive of posttensioning;
2. Determine design vehicle types and locations for production of maximum stresses due to live load; and
3. Compute boundary (hinge) displacements or reactions, as required, for input to the second program, FINPLA.

FINPLA, a program previously used in analyses of cellular, folded plate structures, was used to produce relatively coarse finite-element analyses of the structure for dead, posttensioning, and critical live loadings to assess displacement fields at boundaries of small, substructured areas surrounding posttensioning brackets.

Substructured bracket areas were finally subdivided into finite-element meshes and analyzed by a third program, STRUDL, in three phases to assess principal local web stresses resulting from

1. Application of posttensioning forces with zero boundary displacements;
2. FINPLA output displacements at mesh boundaries due to dead, live, impact, and posttensioning forces; and
3. Combined posttensioning forces and boundary displacements.

CURVBRG analyses indicated subyield web stresses in Spans 1, 2, and 4, and at Bent 3 for factored load conditions without posttensioning. Stresses somewhat in excess of yield were computed at flange cutoff points in Span 3; however, the required posttensioning force was about one-half of that indicated by the use of AASHTO live-load distribution specifications in Caltrans' Office of Structures and Design.

The STRUDL finite-element analyses indicated localized stresses as high as 44 ksi (yield stress, 33 ksi) in the vicinities of bolts affixing brackets to webs.

RECOMMENDATIONS FOR IMPLEMENTATION

The authors recommended that all Caltrans designs for new structures or rehabilitation of existing structures that employ steel girders be reviewed with CURVBRG, with due consideration for all phases of construction to determine stresses, especially those resulting from partial-width live loads (e.g., Permit-series or combined P- and H-series loadings), for which AASHTO load distribution criteria may be highly conservative. Significant savings may be realized in new designs, and expensive rehabilitation of existing structures may be avoided by use of the program.

Use of CURVBRG in an analysis with complex construction procedures is illustrated and methods used to determine principal local web stresses are demonstrated. However, before future, extensive analyses are performed, the establishment of guidelines pertinent to stresses that will be considered excessive is recommended.

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