REPORT NO.
UC SESM 74-16

STRUCTURES AND MATERIALS RESEARCH
DEPARTMENT OF CIVIL ENGINEERING

GIRDER-PC—A COMPUTER PROGRAM FOR DESIGN CHECKING OF PRESTRESSED CONCRETE BOX GIRDER BRIDGES

BY

G. H. POWELL

In cooperation with the State of California, Business and Transportation Agency, Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

DECEMBER 1974

COLLEGE OF ENGINEERING
OFFICE OF RESEARCH SERVICES
UNIVERSITY OF CALIFORNIA
BERKELEY CALIFORNIA

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle

GIRDER PC: A COMPUTER PROGRAM FOR DESIGN CHECKING OF PRESTRESSED CONCRETE BOX GIRDER BRIDGES

5. Report Date

September, 1974

6. Performing Organization Code

7. Author(s)

G. H. Powell

8. Performing Organization Report No.

UC SESM 74-16

9. Performing Organization Name and Address

Department of Civil Engineering
University of California
Berkeley, California 94720

10. Work Unit No.

11. Contract or Grant No.

HPR-1(12) D-4-53

12. Sponsoring Agency Name and Address

California Department of Transportation
Sacramento, California 95807

13. Type of Report and Period Covered

FINAL REPORT

14. Sponsoring Agency Code

RTA 13945 - 14648

15. Supplementary Notes

Prepared in cooperation with the State of California, Business and Transportation Agency, Department of Transportation and U. S. Department of Transportation, Federal Highway Administration

16. Abstract

A computer program for the design analysis of multi-span prestressed concrete bridge structures is described, the analysis and design checking procedures are explained, and a detailed user's guide is presented. The program is primarily applicable to cast in-situ prestressed concrete box girder bridges.

The girder must be straight in plan, and it must be possible to idealize any girder cross section as an equivalent I-section, otherwise the program is applicable to structures of a wide variety of geometrical shapes. The girder depth, width and slab thicknesses may vary arbitrarily along its length, vertical or inclined monolithic piers may be specified, and internal hinges and expansion joints may be included. Any number of prestressing cables may be specified, with arbitrary profiles and anchorages at any point. Some spans of the girder may be non-prestressed if desired.

Structural analyses for gravity load, live loads of a variety of types, support settlement, temperature differential between deck and soffit, and prestress forces are carried out independently, using the Direct Stiffness Method. The results of these analyses may then be combined arbitrarily, and design checks for working load flexure, ultimate moment capacity and shear capacity can be carried out at any specified sections.

A design described in the State of California Bridge Design Manual is checked, and an artificial example for illustration purposes is described.

17. Key Words

box girders, computers, concrete bridges, design, research, prestressed concrete

18. Distribution Statement

Unlimited

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	118	

Structures and Materials Research
Department of Civil Engineering
Division of Structural Engineering
and
Structural Mechanics

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Report to the Sponsors:
State of California
Department of Transportation
Division of Highways
and
U. S. Department of Transportation
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College of Engineering
Office of Research Services
University of California
Berkeley, California

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A design described in the state of California Bridge Design Manual is checked, and an artificial example for illustration purposes is described.

ACKNOWLEDGEMENT

Prepared in cooperation with the State of California, Business and Transportation Agency, Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration, under State of California Research Technical Agreement 13945-14648.

The assistance provided by G. Mancarti, R. Davis, D. Nix and E. Evans of the California Division of Highways is gratefully acknowledged.

TABLE OF CONTENTS

1. INTRODUCTION

1.1 SCOPE

GIRDER-PC is a computer program for the practical design analysis of prestressed concrete bridges which can be idealized as isolated girders or frames in two dimensions. The program is particularly applicable to post-tensioned concrete box girder bridges.

Within the limitations of the isolated girder idealization, the aim has been to produce a program which is applicable to bridges of virtually arbitrary geometry. Some features of the program are as follows.

(1) The girder may vary arbitrarily in depth and width along its length. The slab and web thickness may also vary arbitrarily.

(2) The piers may be vertical or inclined, and may be monolithically connected to the superstructure.

(3) Internal hinges and expansion joints are permitted.

(4) There may be any number of prestressing cables, following any paths and subjected to arbitrary stressing sequences. Friction loss computations are carried out.

(5) Although the program is intended primarily for prestressed concrete, it is also applicable to reinforced concrete girders and to structures with some spans of prestressed concrete and others of reinforced concrete.

(6) Gravity loads are determined automatically. Loadings due to foundation settlement and temperature differences between deck and soffit can be considered.

(7) Live load maximum effects for AASHO trucks, AASHO lane loads and arbitrary load trains can be determined automatically.

(8) Design checks may be carried out for working load flexure, shear capacity (including stirrup design) and ultimate moment capacity (including computation of required areas of non-prestressed reinforcement).

The program is not applicable to composite structures with precast girders and in-situ deck slabs. The isolated girder idealization does not permit the effects of torsion or lateral distribution of load to be considered.

The program can be used to carry out design checks on designs produced by other means, or can be used to design a structure by successive trials, the redesign decisions being made at each stage by the program user.

1.2 REPORT LAYOUT

Chapter 2 of this report is a self-contained user's guide for the computer program. The guide contains detailed notes and explanations, and hence is likely to be too bulky for everyday use by an experienced user. It is expected that users will produce their own abbreviated guides should they be needed.

Chapter 3 contains descriptions of two examples, to demonstrate the capabilities of the program and to provide guidance in preparing input data. Conclusions and recommendations for implementation of the program are contained in Chapter 4.

2. PROGRAM USER'S GUIDE

UNIVERSITY OF CALIFORNIA
Berkeley

Division of Structural Engineering
and Structural Mechanics

Computer Programming Series

IDENTIFICATION

GIRDER - PC: Computer program for structural analysis and design checking of prestressed concrete box girder bridges.

Programmed: G. H. Powell, University of California, Berkeley, 1973.

PURPOSE

The program determines the bending moments, shear forces and axial forces in prestressed concrete bridge girders subjected to static loads, support settlement, temperature loads, prestressing forces and movable live loads, and carries out design checks for working load flexural stress, shear strength and ultimate moment capacity.

The program is applicable primarily to post-tensioned, cast in-situ, box girder bridges, but can be applied to other types of girder also. The girder may vary in width, depth and wall thickness along its length. The girder may be supported on bearings or by monolithic pier structures which may be either vertical or inclined. Internal hinges and expansion joints can be considered.

Live load envelopes may be produced for AASHO loadings and for arbitrary trains of wheels. The design checking is carried out according to the practice of the California Division of Highways.

ASSUMPTIONS AND FEATURES

Details of the assumptions and idealization procedure are presented in the accompanying NOTES. The bridge is idealized as a plane frame, with closely spaced joints (nodes) along the girder. Axial and flexural deformations of the girder and supporting substructures are considered. The geometry of the structure is defined in terms of the node locations, supported nodes and support properties, internal hinge locations, and girder cross section dimensions. The cross section dimensions are specified span by span, independently of the node locations, so that it is not necessary to specify section properties for each girder member (element).

Prestressing cables may be of virtually any shape and may be subjected to arbitrary stressing sequences. Anchorages may be located anywhere in the girder, and any number of different cables may be specified. Friction loss calculations are carried out, and the prestress loads are applied to the girder by a rational procedure.

Live load envelopes for trucks and other load trains are obtained by stepping the loads across the girder, from node to node, and accumulating maximum effects. Envelopes for AASHO lane loading are produced by a similar procedure, with modifications to account for only a single point load for positive moments but two point loads for negative moments.

Checks of flexural stress, shear strength (including required stirrup spacings) and ultimate moment capacity (including required additional unprestressed reinforcement) can be made at any locations in the girder. Axial forces and bending moments for use in substructure design are computed.

Provision for data checking runs is made, to permit the input data to be reviewed prior to actual execution.

CAPACITY LIMITATIONS

There are virtually no capacity limitations associated with fixed array dimensions, because the data storage is allocated dynamically at execution time. The data is stored in blank COMMON, and the available storage can be changed by altering the following two statements in the short main program:

```
COMMON A(n)

MAXA = n
```

in which n = storage to be allocated. A value n = 10000 should be adequate for all except the longest and most complex bridges. The required storage is computed and printed by the program, so that changes can easily be made if the allocated storage is inadequate.

Two scratch files, TAPE 1 and TAPE 2, are required. The required file lengths are printed by the program during both execution and data checking runs, but data is written only during execution runs. The required file lengths, if important, can therefore be determined by making a data checking run. The required lengths are usually short. A length of 20000 words on TAPE 1 and 4000 words on TAPE 2 should be ample in most cases.

INPUT DATA

The following cards describe the structure geometry (Sections A through G), the loading (Sections H through K), and the design checking requirements (Sections L and M).

The units required for the input quantities should be carefully noted. Units of feet for length have been used consistently, kips for force, sq. ins for steel areas and ksi for stresses. Minor program modifications would be needed to convert to other units.

FORTRAN format conversions have been used throughout, and it is assumed that users are familiar with these conversions.

A. START CARD AND PROBLEM TITLE (A4,4X,18A4) - ONE CARD.

Columns	1 - 5:	Punch the word START
	6 - 8:	Blank
	9 - 80:	Problem title, to be printed with output.

B. STRUCTURE CONTROL INFORMATION (10I5) - ONE CARD

See NOTE 1 for geometric definition of structure.

Columns	1 - 5:	Number of spans.
	6 - 10:	Number of subspans.
	11 - 15:	Total number of nodes.
	16 - 20:	Number of "control" nodes for which coordinates are specified (NCJTS, see Section C1). All other nodes are interpolated automatically. See NOTE 2 for generation procedure.
	21 - 25:	Number of commands defining height of deck surface above reference plane (NGENP, See Section C2). Typically zero, so that deck surface becomes reference plane. See NOTE 3 for explanation.
	26 - 30:	Number of "simple" supports, for which translational and rotational stiffnesses are specified (NSSPS, See Section D1). See NOTE 4 for support properties.
	31 - 35:	Number of supporting substructures for which dimensions and other properties are specified. (NSUPS, See Section D2). See NOTE 5 for support properties.
	36 - 40:	Number of internal hinge links, for which translational and rotational stiffnesses are specified (NHINS, See Section D3). See NOTE 6 for hinge properties.
	41 - 45:	Largest number of cross section segments in any span of the girder. See Note following for explanation. If blank or zero, assumed to be 10.
	46 - 50:	Data checking code. Punch 1 if only a data checking run is required. Leave blank or punch zero if full execution is required.

Note: The maximum number of cross section segments is the largest of any of the values NTS, NBS, NWS and NDS specified in Section G(a). The largest value will commonly be NBS = 3 for an interior span. This number is required to permit the program to allocate core storage. The program will execute properly if a number equal to or larger than the maximum number of segments is specified, but will produce errors if a smaller number is entered. The default option of 10 should be generous for almost all applications, and may be used if desired. The only concern when specifying a larger value than the actual maximum is that an unnecessarily large amount of core will be allocated for storage of cross section data. However, this excess allocated core will usually be small.

C. NODE COORDINATE SPECIFICATION

C1. CONTROL NODE COORDINATES (2I5,F10.0) - NCJTS CARDS, ONE FOR EACH CONTROL NODE.

See NOTE 1 for generation procedure.

Columns 1 - 5: Control node number, in any sequence.

6 - 10: Offset node number, as follows.

(a) If the coordinate of the control node is being specified directly, leave blank or punch zero.

(b) If the location of this control node is being defined by its distance (positive or negative) from a previously defined node, punch the number of this node.

11 - 20: (a) X coordinate of node (ft) or

(b) X distance (ft) from the offset node. The coordinate origin may be anywhere.

C2. LOCATION OF REFERENCE PLANE (2I5,3F10.0) - NGENP CARDS, ONE FOR EACH COMMAND DEFINING HEIGHT OF DECK SURFACE ABOVE REFERENCE PLANE

Omit if NGENP is zero. Unless specified otherwise all nodes are assumed to be at the deck surface. No generation commands are needed if the deck surface is to be the reference plane, as will usually be the case. See NOTE 2 for explanation.

Columns 1 - 5: Number of node at left end of series of nodes covered by this command.

6 - 10: Number of node at right end of series.

11 - 20: Height (ft) of deck surface above reference plane at left end.

21 - 30: Height (ft) of deck surface above reference plane at point midway between the left and right ends.

31 - 40: Height (ft) of deck surface above reference plane at right end.

D. SPAN AND SUBSPAN DEFINITION (4I5,3F10.0) - ONE CARD FOR EACH SUBSPAN

Subspans must be numbered consecutively from left to right along the bridge. See NOTE 1 for explanation.

Columns	1 - 5:	Subspan number, in sequence.
	6 - 10:	Number of node at left end of subspan.
	11 - 15:	Number of node at right end of subspan.
	16 - 20:	Span number of which this subspan is a part.
	21 - 30:	Moment impact factor, F_M, for live loads applied in this subspan. See NOTE 7 for explanation.
	31 - 40:	Shear modifying factor, f, for live load shears at left end of subspan. If blank or zero, assumed to be 1.0.
	41 - 50:	Shear modifying factor, f, for live load shears at right end of subspan. If blank or zero, assumed to be 1.0.

E. MATERIAL PROPERTIES (5E10.0) - ONE CARD

Columns	1 - 10:	Young's modulus (short term loading) for concrete in girder, assumed to be the same for both prestressed and nonprestressed concrete (ksi).
	11 - 20:	Coefficient of thermal expansion for concrete in girder.
	21 - 30:	Density of concrete in girder (lb./cu.ft.). See Note below.
	31 - 40:	Young's modulus (short term loading) for concrete in substructures (ksi).
	41 - 50:	Reduction factor by which short term moduli are to be multiplied to obtain those for long term loading.

Note: Dead loads are computed by the program for the girder only. Substructure weights, if required, must be specified as point loads.

F. SUPPORTS AND INTERNAL HINGES

F1. SIMPLE SUPPORTS (2I5,4F10.0) - NSSPS CARDS, ONE FOR EACH SIMPLE SUPPORT.

Omit if NSSPS is zero. See NOTE 4 for description of support properties.

Columns 1 - 5: Simple support number (for identification purposes). Simple supports must be numbered in the sequence 1,2,3, etc, and entered in this same sequence.

6 - 10: Number of supported node.

11 - 20: Support stiffness (k/ft.) in horizontal (X) direction. Enter zero for a roller support.

21 - 30: Support stiffness (k/ft.) in vertical (Y) direction. If support is essentially rigid, a value of 10^{10} k/ft. is suggested.

31 - 40: Rotational stiffness of support (k.ft/radian). Enter zero for a typical bearing. Enter the flexural stiffness of the pier structure if this type of support is being used to approximate a supporting substructure.

41 - 50: Depth of supported point below reference plane (ft.). See NOTE 4 for significance of this distance.

F2. SUPPORTING SUBSTRUCTURES (2I5,2F10.0,3F5.0,2F10.0,2F5.0) - NSUPS CARDS, ONE FOR EACH SUBSTRUCTURE.

Omit if NSUPS is zero. See NOTE 5 for description of support properties.

Columns 1 - 5: Substructure number (for identification purposes). Substructures must be numbered in the sequence 1,2,3, etc., and entered in this same sequence.

6 - 10: Number of supported node.

11 - 20: Average cross section area (ft.^{2}), for computation of axial stiffness.

21 - 30: Reference value of cross section moment of inertia (ft.^{4}). For a uniform member this is the actual moment of inertia.

31 - 35: Flexural stiffness coefficient, k_{tt}, for top of substructure. E.g. 4.0 for a uniform member with a fixed base; 3.0 for a uniform member with a pinned base.

36 - 40: Flexural stiffness coefficient, k_{bb}, for base of substructure. E.g. 4.0 for a uniform member with a fixed base; 0.0 for a pinned base.

41 - 45: Flexural stiffness coefficient, k_{tb}, for carry-over effects between top and base. E.g. 2.0 for a uniform member with a fixed base; 0.0 for a pinned base.

46 - 55: Height of substructure (ft.) between base and top.

56 - 65: X-projection of substructure (ft.) between base and top, positive if top is to right of base.

66 - 70: X-eccentricity, e_X (ft.), between top of substructure and supported node, positive if supported node is to right of substructure top. A rigid block is assumed to connect the substructure to the node.

71 - 75: Y-eccentricity, e_Y (ft.), between top of substructure and supported node, positive if supported node is above substructure top.

F3. INTERNAL HINGES (2I5,4E10.0) - NHINS CARDS, ONE FOR EACH INTERNAL HINGE.

Omit if NHINS is zero. See NOTE 6 for description of hinge properties. Each hinge connects a pair of sequentially numbered nodes, which should have identical X coordinates.

Columns 1 - 5: Hinge number (for identification purposes). Hinges must be numbered in the sequence 1, 2,3, etc., and entered in this same sequence.

6 - 10: Number of node to left of hinge. The number of the node to the right of the hinge will be one larger.

11 - 20: Stiffness (k/ft.) of hinge for relative horizontal (X) displacement between the connected nodes.

21 - 30: Stiffness (k/ft.) of hinge for relative vertical (Y) displacement between the connected nodes. If the hinge is essentially rigid, a value of 10^{10} k/ft. is suggested.

31 - 40: Rotational stiffness (k. ft./radian) of hinge for relative rotation of nodes. This will typically be zero.

41 - 50: Depth of hinge center below reference plane (ft.). See NOTE 6 for significance of this distance.

G. CROSS SECTION DIMENSIONS - ONE SET OF CARDS, AS FOLLOWS, FOR EACH SPAN.

See NOTE 8 for dimensioning procedure. See NOTE 9 for similarity option.

G(a). CONTROL INFORMATION FOR SPAN (9I5) - ONE CARD.

Columns 1 - 5: Span number, in any sequence.

6 - 10: Number of top flange segments in span (NTS). Leave blank or punch zero to exercise similarily option.

11 - 15: Number of bottom flange segments (NBS).

16 - 20: Number of segments for definition of web thickness (NWS).

21 - 25: Number of segments for definition of girder depth (NDS).

26 - 30: Similar span number (plus or minus) for top flange (NSIMT). Leave blank or punch zero if similarity option for top flange is not exercised.

31 - 35: Similar span number for bottom flange (NSIMB).

36 - 40: Similar span number for web thickness (NSIMW).

41 - 45: Similar span number for girder depth (NSIMD).

G(b). TOP FLANGE DEFINITION (I5,5F10.0) - NTS CARDS, ONE FOR EACH TOP FLANGE SEGMENT.

Omit if NTS is zero (and hence NSIMT is not zero). Linear variations of flange thickness and width may be specified in each segment.

Columns 1 - 5: Segment number within span, in any sequence. Segments must be numbered in the sequence 1,2,3, etc. within each span, starting at the left end of the span.

6 - 15: Segment length, as follows.

(a) If greater than 1.0, assumed to be the actual segment length (ft.).

(b) If less than or equal to 1.0, assumed to be a proportion of the span length.

16 - 25: Effective flange thickness at left end of segment (ft.)

26 - 35: Effective flange thickness at right end of segment (ft.).

36 - 45: Effective flange width at left end of segment (ft.).

46 - 55: Effective flange width at right end of segment (ft.).

G(c). BOTTOM FLANGE DEFINITION (I5,5F10.0) - NBS CARDS, ONE FOR EACH SEGMENT.

Omit if NBS is zero (and hence NSIMB is not zero).

Enter data in same format as for top flange.

G(d). WEB THICKNESS DEFINITION (I5,3F10.0) - NWS CARDS ONE FOR EACH SEGMENT.

Omit if NWS is zero (and hence NSIMW is not zero). A linear variation of web thickness may be specified in each segment.

Columns 1 - 5: Segment number within span, in any sequence.

6 - 15: Segment length, as for top flange.

16 - 25: Web thickness at left end of segment (ft.).

26 - 35: Web thickness at right end of segment (ft.).

G(e). GIRDER DEPTH DEFINITION (I5,4F10.0) - NDS CARDS, ONE FOR EACH SEGMENT.

Omit if NDS is zero (and henge NSIMD is not zero). A constant, linear or quadratic variation of depth may be specified.

Columns 1 - 5: Segment number within span, in any sequence.

6 - 15: Segment length, as for top flange.

16 - 25: Overall girder depth at left end of segment (ft.).

26 - 35: Overall girder depth at right end of segment (ft.). If blank or zero, assumed to be equal to depth at left end.

36 - 45: Overall girder depth a midpoint of segment (ft.). If blank or zero, assumed to be average of depths at left and right ends (i.e. linear depth variation).

Repeat from card G(a) for all spans. It will frequently be possible to use the similarity option, and hence to omit some or all of cards G(b) through G(e) for most spans.

H. LOAD CONTROL INFORMATION (5I5) - ONE CARD

Columns 1 - 5: Number of static load cases (NDLO). See NOTE 10.

10: Temperature loading code, as follows. See NOTE 11.

(a) Leave blank or punch zero if temperature effects are to be ignored.

(b) Punch 1 if temperature effects are to be considered. The actual temperatures are specified as explained in Section M1.

11 - 15: Number of groups into which prestressing cables are divided (NCSETS). See NOTE 12.

16 - 20: Number of live load envelopes to be produced (NLLO). See NOTE 13.

21 - 25: Maximum number of parabolic or straight segments in any cable. If blank or zero, assumed to be 4 x (no. of spans) - 2. See Note below.

Note: The maximum number of cable segments is the largest value of NCSEGS (See Section J(b)(ii)). This number is required to permit the program to allocate core storage. The program will execute correctly if the actual maximum or any larger number is specified. Specification of an unnecessarily large number merely results in the allocation of an unnecessary amount of core, and the excess will normally be small.

The default option for a blank or zero input value is the number of segments in a typical cable extending over all spans (4 segments for each interior span, 3 for each end span).

I. STATIC LOADS - NDLO SETS OF CARDS, AS FOLLOWS, ONE FOR EACH STATIC LOAD CASE.

Omit if NDLO is zero. See NOTE 10 for explanation of procedures.

I(a). LOAD CASE CONTROL INFORMATION (6I5,13A4) - ONE CARD.

Columns 1 - 5: Load case number (for identification purposes). Static load cases must be numbered in the sequence 1,2,3, etc., and entered in this same sequence.

6 - 10: Number of regions for which girder self weights are to be calculated automatically (NAUTO).

11 - 15: Number of regions over which additional distributed loads are specified (NDLS).

16 - 20: Number of commands defining concentrated loads applied at nodes (NJLS).

21 - 25: Number of simple supports with specified base displacements (NSSDS).

26 - 30: Number of substructures with specified base displacements (NSDS).

31 - 80: Load case title, to be printed with output.

I(b). AUTOMATIC SELF WEIGHT REGIONS (2I5) - NAUTO CARDS, ONE FOR EACH REGION.

Omit if NAUTO is zero. See NOTE 10.2 for procedure. The complete girder may be specified as a single region.

Columns 1 - 5: Number of first subspan in region over which girder self weight is to be calculated by the program.

6 - 10: Number of last subspan in region.

I(c). ADDITIONAL DISTRIBUTED LOADS (2I5,3F10.0) - NDLS CARDS, ONE FOR EACH REGION.

Omit if NDLS is zero. See NOTE 10.3 for procedure. The load may be constant or may vary linearly or quadratically within any region. Regions may overlap as desired. Loads are applied vertically, positive downwards.

Columns 1 - 5: Number of node at left end of region.

6 - 10: Number of node at right end of region.

11 - 20: Distributed load intensity (k/ft.) at left end of region.

21 - 30: Distributed load intensity (k/ft.) at right end of region. If zero or blank, assumed to be equal to the intensity at the left end. May be left blank if the load is uniform.

31 - 40: Distributed load intensity (k/ft.) at midpoint of region. If zero or blank, assumed to be the average of the intensities at the left and right ends. May be left blank if the load is uniform or varies linearly.

I(d) CONCENTRATED NODAL LOADS (I5,3F10.0,2I5) - NJLS CARDS, ONE FOR EACH COMMAND

Omit if NJLS is zero. Each command may specify the load on a single node or a number of nodes with constant node number difference. A single node may appear in several commands if desired, the loads on such a node being added together.

Columns 1 - 5: Node number, or number of lowest numbered node in a series of nodes to which identical loads are to be added.

6 - 15: Load in horizontal (X) direction (kips, positive to right).

16 - 25: Load in vertical (Y) direction (kips, positive downwards).

26 - 35: Moment load (k. ft., positive clockwise).

36 - 40: Number of highest numbered node in series. Leave blank for a single node.

41 - 45: Node number difference between successive nodes in series. Leave blank for a single node.

I(e). SIMPLE SUPPORT DISPLACEMENTS (I5,3F10.0) - NSSDS CARDS, ONE FOR EACH DISPLACED SUPPORT.

Omit if NSSDS is zero. See NOTE 10.4 for procedure.

Columns 1 - 5: Identifying number (not node number) of simple support.

6 - 15: Horizontal (X) displacement of support (ft., positive to the right).

16 - 25: Vertical (Y) displacement of support (ft., positive downwards).

26 - 35: Rotation of support (radians, positive clockwise).

I(f). SUBSTRUCTURE BASE DISPLACEMENTS (I5,3F10.0) - NSDS CARDS, ONE FOR EACH DISPLACED SUBSTRUCTURE.

Omit if NSDS is zero. See NOTE 10.4 for procedure.

Columns 1 - 5: Identifying number (not node number) of substructure.

6 - 15: Horizontal (X) displacement at base of substructure (ft., positive to the right).

16 - 25: Vertical (Y) displacement at base of substructure (ft., positive downwards).

26 - 35: Rotation (radians, positive clockwise) at base of substructure.

J. PRESTRESSING CABLES - NCSETS SETS OF CARDS, AS FOLLOWS, ONE FOR EACH CABLE GROUP.

Omit if NCSETS is zero. See NOTE 12 for description of procedures.

J(a). CONTROL INFORMATION FOR GROUP (2I5,3F10.0,I5,F10.0,6A4) - ONE CARD

Columns 1 - 5: Cable group number (for identification purposes). Cable groups must be numbered in the sequence 1,2,3, etc., and entered in this same sequence.

6 - 10: Number of cables in group (NCABS, See Section J(b)).

11 - 20: Young's modulus of prestressing steel (ksi).

21 - 30: Curvature friction coefficient.

31 - 40: Wobble friction coefficient (per foot).

45: Creep loss code, as follows.

(a) Punch 1 if prestress loss is to be specified as a constant stress.

(b) Punch 2 if prestress loss is to be specified as a percentage of the steel stress at transfer.

46 - 55: Creep loss, as follows.

(a) If loss code = 1, punch loss in stress (ksi). Enter as a positive value.

(b) If loss code = 2, punch percent stress loss. Enter as a positive value.

56 - 79: Cable group name, to be printed with output.

J(b). CABLE DIMENSIONS AND STRESSING SEQUENCE - NCABS SETS OF CARDS, ONE FOR EACH CABLE IN GROUP.

J(b)(i). CABLE CONTROL INFORMATION (5I5,3F10.0,6A4) - ONE CARD.

Columns 1 - 5: Cable number within group (for identification purposes). The cables within each group must be numbered in the sequence 1,2,3, etc., and entered in this same sequence.

6 - 10: Number of first subspan spanned by cable (i.e. subspan at left end of cable).

11 - 15: Number of last subspan spanned by cable (i.e. subspan at right end of cable). The first and last subspan will be the same for a cable extending over only one subspan.

16 - 20: Number of straight or semi-parabolic segments making up cable profile (NCSEGS, See Section J(b)(ii)).

21 - 25: Number of jacking operations (NOPS, See Section J(b)(iii)). If blank or zero, the six "standard" operations as described in J(b)(iv) are assumed.

26 - 35: Depth of left anchorage below reference plane (ft.).

36 - 45: Depth of right anchorage below reference plane (ft.).

46 - 55: Cross sectional area of cable (sq. ins).

56 - 79: Cable name, to be printed with output.

Note: If the specified cable segment lengths do not sum to the lengths of the subspans over which the cable extends, and if the specified cable sags are inconsistent with the left and right anchorage depths, error messages will be printed and execution will be suppressed.

J(b)(ii). CABLE SEGMENT DIMENSIONS (2I5,3F10.0) - NCSEGS CARDS, ONE FOR EACH SEGMENT.

Columns 1 - 5: Segment number within cable (for identification purposes). The segments within each cable must be numbered from the left anchorage in the sequence 1,2,3, etc., and entered in this same sequence.

10: Shape code for segment, as follows.

(a) 1: straight.

(b) 2: semi-parabola with zero slope at right end.

(c) 3: semi-parabola with zero slope at left end.

(d) 4: part parabola with specified slope at right end.

(e) 5: part parabola with specified slope at left end.

11 - 20: Horizontal (X) projection of segment (ft.).

21 - 30: Vertical (Y) projection of segment (ft.), positive if the right end is below the left end, otherwise negative.

31 - 40: Leave blank for shape code = 1,2 or 3. Enter slope at right (or left) end if shape code is 4 (or 5). The slope is the tangent of the slope angle, positive for a downwards slope from left to right.

J(b)(iii). JACKING SEQUENCE (3I5,F10.0) - NOPS CARDS, ONE FOR EACH JACKING OPERATION.

Omit if NOPS is zero, and ender one card as in J(b)(iv). Each operation is assumed to be completed before the next operation is begun.

Columns 1 - 5: Operation number (for identification purposes). The jacking operations for each cable must be numbered in the sequence 1,2,3 etc., and entered in this same sequence.

10: End code, as follows

(a) 1: jacking at left anchorage.

(b) 2: jacking at right anchorage.

15: Operation code, as follows.

(a) 1: jacking force is increased (or decreased) to the total value specified in the following field.

(b) 2: jacking force is increased (or decreased) by the amount specified in the following field.

(c) 3: an extension (or draw-in) is imposed, equal to the amount specified in the following field.

16 - 25: Specified force (k), force increase (k, negative for a decrease) or extension (inches, negative for draw-in) at anchorage.

J(b)(iv). "STANDARD" JACKING OPERATIONS (I5,6F10.0) - ONE CARD, TO BE ENTERED ONLY IF NOPS IS ZERO.

Columns 1 - 5: End code, as follows.

(a) 1: jacking takes place at left anchorage first, followed by right anchorage.

(b) 2: jacking takes place at right anchorage first, followed by left anchorage.

6 - 15: Temporary jacking force at first anchorage (k, typically based on $0.8f'_s$). This is operation 1.

16 - 25: Jacking force at first anchorage after reduction to working value (k, typically based on $0.75f'_s$ or $0.7f'_s$). This is operation 2.

26 - 35: Anchorage draw-in at first anchorage (inches, input as a negative value). This is operation 3.

36 - 45: Temporary jacking force at second anchorage (k). This is operation 4.

46 - 55: Jacking force at second anchorage after reduction to working value (k). This is operation 5.

56 - 65: Anchorage draw-in at second anchorage (inches, negative). This is operation 6.

Repeat from Section J(b)(i) for each cable in group, then from Section J(a) for each group.

K. LIVE LOADS - NLLO SETS OF CARDS, AS FOLLOWS, ONE FOR EACH LIVE LOAD ENVELOPE.

Omit if NLLO is zero. See NOTE 13 for description of procedure.

K(a). ENVELOPE CONTROL INFORMATION (2I5,2X,17A4) - ONE CARD.

Columns 1 - 5: Envelope number (for identification purposes). Envelopes must be numbered in the sequence 1,2,3, etc., and entered in this same sequence.

6 - 10: Number of different live load cases (NLDS, See Section K(b)) contributing to this envelope.

11 - 12: Blank

13 - 80: Envelope title, to be printed with output.

K(b). LOAD SPECIFICATION - NLDS SETS OF CARDS, ONE FOR EACH LOAD CASE.

For all except arbitrary train loadings, each set will consist of only one card, as in K(b)(i). For arbitrary trains, additional cards are required to specify wheel loadings and axle spacings, as in K(b)(ii) and (iii).

K(b)(i). LOAD CASE CONTROL INFORMATION (2I5,F10.0,3I5,2F10.0) - ONE CARD.

Columns 1 - 5: Load case number within this envelope (for identification purposes). The load cases for each envelope must be numbered in the sequence 1,2,3, etc., and entered in this same sequence.

6 - 10: Loading type, as follows.

(a) 1: arbitrary train of wheel loads.

(b) 2: H10 truck loading.

(c) 3: H15 truck loading.

(d) 4: H20 truck loading.

(e) 5: HS15 truck loading.

(f) 6: HS20 truck loading.

(g) 7: H10 lane loading.

(h) 8: H15 lane loading.

(i) 9: H20 lane loading.

(j) 10: P3 truck loading.

(k) 11: P5 truck loading.

(l) 12: P7 truck loading.

(m) 13: P9 truck loading.

(n) 14: P11 truck loading.

(o) 15: P13 truck loading.

11 - 20: Loading magnitude, as follows.

(a) Loading type 1: scale factor to be applied to wheel loads specified in Section K(b)(ii). This will usually represent the number of wheel lines or load lanes.

(b) Other loading types: number of load lanes. Note that for truck loadings the number of load lanes is half the number of wheel lines.

21 - 25: Axle spacings, as follows.

(a) Loading type 1: Number of wheels in train. Maximum 40.

(b) Loading types 5 and 6: Number of different spacings between drive and trailer axles to be considered for HS20 trucks.

(c) Other loading types: zero or blank.

26 - 30: Principal axle identification, as follows.

(a) Loading type 1: Number of principal axle (i.e. axle load which is moved from node to node in stepping process). First axle = 1, next axle = 2, etc.

(b) Other loading types: Zero or blank

35: Direction reversal code, as follows.

(a) Loading types 7 through 9: Zero or blank.

(b) Other loading types: Punch 1 if train or truck travels across bridge from left to right only; punch 2 for travel from right to left only; punch 3 for travel in both directions.

36 - 45: Smallest axle spacing for HS trucks.

(a) Loading types 5 and 6: Smallest axle spacing (ft.) to be considered between drive and trailer axles.

(b) Other loading types: blank.

46 - 55: Largest axle spacing for HS trucks.

(a) Loading types 5 and 6: Largest axle spacing (ft.). Leave blank if only a single spacing is to be considered. For three or more spacings, the intermediate spacings are uniformly interpolated between the minimum and maximum spacings.

(b) Other loading types: blank.

K(b)(ii). LOAD TRAIN AXLE LOADS (8F10.0) - ONE OR MORE CARDS.

Enter for loading type 1 only. Enter as many cards as needed to specify all axle loads, eight to a card.

Columns 1 - 80: Eight fields, each F10.0. Enter axle loads (k), in sequence from front of train.

K(b)(iii). LOAD TRAIN AXLE SPACINGS (8F10.0) - ONE OR MORE CARDS.

Enter for loading type 1 only. Enter as many cards as needed to specify all axle spacings, eight to a card. The number of spacings is one less than the number of axles. <u>Omit</u> for a single axle.

Columns 1 - 80: Eight fields, each F10.0. Enter axle spacings (ft.), in sequence from front of train.

L. CONTROL INFORMATION FOR DESIGN CHECKING (7I5) - ONE CARD

Columns 1 - 5:	Number of load combinations for which checks of working load flexural stresses are required at prestressed cross sections (NWLF, see Section M1).
6 - 10:	Number of load combinations for which checks of shear capacity are required (NSHR, see Section M2).
11 - 15:	Number of load combinations for which checks of negative ultimate moment capacity are required at prestressed cross sections (NULTN, see Section M3).
16 - 20:	Number of load combinations for which checks of positive ultimate moment capacity are required at prestressed cross sections (NULTP, see Section M4).
21 - 25:	Number of load combinations for which required flexural steel areas are to be computed at non-prestressed cross sections using working load procedures (NRAW, see Section M5).
26 - 30:	Number of load combinations for which required flexural steel areas are to be computed at non-prestressed cross sections using ultimate moment procedures (NRAU, see Section M6).
31 - 35	Maximum number of checking sections specified for any load combination. If blank or zero, assumed to be 100. See Note below.

Note: The maximum number of checking sections is required to permit the program to allocate core storage. The program will execute correctly if the actual maximum or any larger value is specified. The default value of 100 should be adequate for most situations.

M1. CHECKS OF WORKING LOAD FLEXURE - NWLF SETS OF CARDS, AS FOLLOWS, ONE FOR EACH LOAD COMBINATION.

Omit if NWLF is zero. See NOTE 15 for checking procedure. See NOTE 14 for load combination and section location procedures.

M1(a). CONTROL INFORMATION (2I5,2F10.0,9A4,7A2) - ONE CARD.

Columns 1 - 5: Combination number (for identification purposes). Combinations must be numbered in the sequence 1,2,3, etc., and entered in this same sequence.

6 - 10: Number of commands defining locations of sections at which checks are to be made (NSEC, See Section M1(c)).

11 - 20: Allowable concrete stress in compression (ksi). Enter as a positive number. Computed stresses which exceed this value are marked by asterisks.

21 - 30: Allowable concrete stress in tension (ksi). Enter as a positive number.

31 - 80: Combination title, to be printed with output.

M1(b). COMBINATION FACTORS - ONE SET OF CARDS DEFINING COMBINATION FACTORS FOR ALL LOADING CASES.

M1(b)(i). STATIC LOAD COMBINATION FACTORS (5F10.0) - ONE OR MORE CARDS.

Enter as many cards as needed to specify NDLO combination factors, one for each static load case, five factors to a card. Omit if there are no static loads (NDLO is zero).

Columns 1 - 50: Five fields, each F10.0. Enter combination factor by which corresponding static load case is to be multiplied, in load case sequence. If a load case is to be ignored, leave corresponding field blank or punch zero.

M1(b)(ii). TEMPERATURE INCREASES (2F10.0) - ONE CARD

This card must be entered whether or not temperature loads have been ignored. The temperature is assumed to vary linearly with respect to girder depth, and to be constant along the bridge length. Enter a blank card or punch zero if temperature loads are not to be considered.

Columns 1 - 10: Temperature increase at deck level.

11 - 20: Temperature increase at underside of girder.

M1(b)(iii). CABLE GROUP COMBINATION FACTORS (5(I5,F10.0)) - ONE OR MORE CARDS

Enter as many cards as needed to specify data for NCSETS cable groups, five groups to a card. Omit if there are no cable groups (NCSETS is zero).

Columns 5: Loss code, as follows, for cable group 1.

(a) Punch zero or leave blank if prestress at transfer is to be considered.

(b) Punch 1 if prestress after losses is to be considered.

6 - 15: Factor by which computed prestress forces for cable group 1 are to be multiplied. This will usually be 1.0. Leave blank or punch zero if group is to be ignored.

20: Loss code for cable group 2.

21 - 30: Combination factor for cable group 2.

35, etc: Continue as above for each cable group.

M1(b)(iv). LIVE LOAD ENVELOPE COMBINATION FACTORS (5F10.0) - ONE OR MORE CARDS.

Enter as many cards as needed to specify NLLO combination factors, one for each live load envelope, five factors to a card. Omit if there are no live loads (NLLO is zero). Note that live load envelopes are not combined by addition, but by further enveloping (See NOTE 13).

Columns 1 - 50: Five fields, each F10.0. Enter combination factor by which corresponding live load envelope is to be multiplied, in numerical sequence. If an envelope is to be ignored, leave corresponding field blank or punch zero.

M1(c). LOCATIONS OF CHECK SECTIONS (2I5,7F10.0) - NSEC CARDS, ONE FOR EACH SECTION LOCATION COMMAND.

Columns 1 - 5: Command number (for identification purposes). Commands must be numbered in the sequence 1,2,3, etc., and entered in this same sequence.

6 - 10: Span number in which all sections defined by this command are located.

11 - 80: Seven fields, each F10.0, containing from zero to seven locations, as follows.

(a) If the first field is zero or blank, the check sections are automatically assumed to be the same as the analysis sections within the span. The remaining fields must also be blank.

(b) If the first field is not zero or blank, the fields up to the first zero or blank field define up to seven check section locations. If the specified value is larger than 1.0, it is assumed to be the distance of the section (ft.) from the left end of the span. If the value is less than or equal to 1.0, it is assumed to be the proportion of the span from the left end of the span to the section.

M2. CHECKS OF SHEAR CAPACITY - NSHR SETS OF CARDS, AS FOLLOWS, ONE FOR EACH LOAD COMBINATION

Omit if NSHR is zero. See NOTE 16 for checking procedure. See NOTE 14 for load combination and section location procedures.

M2(a). CONTROL INFORMATION (2I5,3F10.0,10A4) - ONE CARD

Columns 1 - 5: Combination number (for identification purposes), in sequence 1,2,3, etc.

6 - 10: Number of commands defining locations of sections at which checks are to be made (NSEC).

11 - 20: Shear stress which may be carried by concrete (ksi). Typically this is $.06f'_c$ psi, but not exceeding 0.18 ksi.

21 - 30: Yield stress of stirrup steel (ksi).

31 - 40: Area of one stirrup (sq. in) for computation of required stirrup spacing.

41 - 80: Combination title, to be printed with output.

M2(b). COMBINATION FACTORS

Enter combination factors as in Section M1(b).

M2(c). LOCATIONS OF CHECK SECTIONS

Enter section location commands as in Section M1(c).

M3. CHECKS OF NEGATIVE ULTIMATE MOMENT CAPACITY - NULTN SETS OF CARDS, AS FOLLOWS, ONE FOR EACH LOAD COMBINATION.

Omit if NULTN is zero. See NOTE 17 for checking procedure. See NOTE 14 for load combination and section location procedures. Sections with zero negative moment will be ignored.

M3(a). CONTROL INFORMATION (2I5,5F10.0,5A4) - ONE CARD.

Columns	1 - 5:	Combination number (for identification purposes) in sequence 1,2,3, etc.
	6 - 10:	Number of commands defining locations of sections at which checks are to be made (NSEC).
	11 - 20:	Crushing strength of concrete (ksi).
	21 - 30:	Yield strength of prestressed steel (ksi).
	31 - 40:	Yield strength of unprestressed (mild) steel required to augment moment capacity (ksi).
	41 - 50:	Cover to centroid of mild steel (in).
	51 - 60:	Minimum depth ratio for effective prestressing cables. To avoid considering cables which are too near the compression flange, all cables with distances from the compression flange less than this ratio of the total section depth will be ignored in the ultimate moment capacity computation. Punch zero to include all cables.
	61 - 80:	Combination title, to be printed with output.

M3(b). COMBINATION FACTORS

Enter combination factors as in Section M1(b). Note, how ever, that bending moments due to prestress are ignored for calculating required moment capacities. Hence, the cable group combination factors serve only to scale the specified cable areas up or down for the calculation of available moment capacity.

M3(c). LOCATIONS OF CHECK SECTIONS

Enter section location commands as in Section M1(c).

M4. CHECKS OF POSITIVE ULTIMATE MOMENT CAPACITY - NULTP SETS OF CARDS, ONE FOR EACH LOAD COMBINATION

Omit if NULTP is zero. See NOTE 17 for checking procedure. See NOTE 14 for load combination and section location procedures. Sections with zero positive moment will be ignored.

M4(a). CONTROL INFORMATION

Enter as for negative ultimate moment, Section M3(a).

M4(b). COMBINATION FACTORS

Enter combination factors as in Section M1(b).

M4(c). LOCATIONS OF CHECK SECTIONS.

Enter section location commands as in Section M1(c).

M5. FLEXURAL STEEL AT NON-PRESTRESSED SECTIONS BY WORKING LOAD PROCEDURE - NRAW SETS OF CARDS, AS FOLLOWS, ONE FOR EACH LOAD COMBINATION

Omit if NRAW is zero. See NOTE 18 for computation procedure. See NOTE 14 for load combination and section location procedure.

M5(a) CONTROL INFORMATION (2I5,5F10.0,5A4) - ONE CARD

Columns	1 - 5:	Combination number (for identification purposes), in sequence 1, 2, 3, etc.
	6 - 10:	Number of commands defining sections at which computations are to be made (NSEC).
	11 - 20:	Allowable stress in concrete (ksi).
	21 - 30:	Allowable stress in flexural reinforcement (ksi).
	31 - 40:	Modular ratio (Young's modulus of steel divided by that of concrete).
	41 - 50:	Cover to centroid of flexural steel at top of girder (in).
	51 - 60:	Cover to centroid of flexural steel at bottom of girder (in).
	61 - 80:	Combination title, to be printed with output.

M5(b) COMBINATION FACTORS

Enter combination factors as in Section M1(b). Factors for cable groups must be included, because prestressing forces may influence the bending moments at non-prestressed sections.

M5(c) LOCATIONS OF SECTIONS

Enter section location commands as in Section M1(c). Sections with prestressed cables having nonzero cable group combination factors will be ignored.

M6. FLEXURAL STEEL AT NON-PRESTRESSED SECTIONS BY ULTIMATE MOMENT PROCEDURE - NRAU SETS OF CARDS, AS FOLLOWS, ONE FOR EACH LOAD COMBINATION

Omit if NRAU is zero. See NOTE 19 for computation procedure See NOTE 14 for load combination and section location procedure.

M6(a) CONTROL INFORMATION (2I5,5F10.0,5A4) - ONE CARD

Columns	1 - 5:	Combination number (for identification purposes), in sequence 1, 2, 3, etc.
	6 - 10:	Number of commands defining sections at which computations are to be made (NSEC).
	11 - 20:	Crushing strength of concrete (ksi).
	21 - 30:	Yield stress of flexural reinforcement (ksi).
	31 - 40:	Strength reduction factor (ϕ, typically 0.90).
	41 - 50:	Cover to flexural steel at top of girder (in).
	51 - 60:	Cover to flexural steel at bottom of girder (in).
	61 - 80:	Combination title, to be printed with output.

M6(b) COMBINATION FACTORS

Enter combination factors as in Section M1(b). Factors for cable groups must be included, because prestressing forces may influence the bending moments at non-prestressed sections.

M6(c) LOCATIONS OF SECTIONS

Enter section location commands as in Section M1(c). Sections with prestressed cables having nonzero cable group combination factors will be ignored.

N. NEXT PROBLEM

Data for as many problems as desired may be processed in a single computer run. Enter data for each problem in turn, beginning with Card A.

O. TERMINATION (A4) - ONE CARD TO TERMINATE DATA DECK

Columns 1 - 4: Punch the word STOP

OUTPUT DATA

A. STRUCTURE GEOMETRY DATA

The following geometry data are printed.

(1) Echo of structure control information.

(2) Echo of control node coordinates.

(3) Coordinates of all nodes.

(4) Echo of reference plane cards, if any.

(5) Height of deck surface above reference plane at each node. This is omitted if the reference plane lies in the deck surface.

(6) Echo of span and subspan definition data.

(7) The length of each span and the first and last node in each span. This helps to provide a check on the data.

(8) Echo of material properties.

(9) Echo of data for simple supports, substructures and internal hinges.

(10) Echo of input data for cross section dimensions.

(11) Cross section dimensions as stored by the program. This helps to provide a check on the data.

(12) The total depth, depth from the deck surface to the section centroid, area, moment of inertia and section moduli for top and bottom fibers at each analysis section. This helps to provide a check on the data.

(13) The length of each element and the slope (downwards to the right positive) of the centroidal axis for each element. This helps to provide a check on the data.

B. COMPUTED RESULTS

The results printed for static, thermal and prestressing loads include the following. The output sequence is indicated in the next section.

(1) Vertical deflections at each node (ft, downwards positive).

(2) Axial force (k, tension positive), bending moment (k.ft, sagging positive) and shear force (k, positive for clockwise shear couple on girder element) at each analysis section.

(3) Horizontal reaction (k, positive acting to the right on the support), vertical reaction (k, positive downwards on the support), moment reaction (k.ft., positive clockwise on the support), horizontal deformation (ft, positive if supported point moves to right relative to support base), vertical deformation (ft, positive if supported point moves downwards relative to support base), and rotational deformation (radians, positive if supported point rotates clockwise relative to support base) for each simple support.

(4) Moment at top (k.ft., positive acting clockwise on the substructure), moment at bottom (k.ft, clockwise positive) and axial force (k, tension positive) for each substructure.

(5) Horizontal force transmitted by hinge link (k, positive with girder in tension), vertical force transmitted by link (k, positive for positive girder shear), moment transmitted by link (k. ft, sagging positive), horizontal deformation of hinge (ft, positive in tension), vertical deformation (ft, positive if left node moves upwards relative to right node), and rotational deformation (radians, positive if left node rotates clockwise relative to right node) for each internal hinge.

The results printed for each live load envelope include the maximum positive and negative values of each of the quantitites printed for static loadings, except as follows.

(a) The girder axial forces are the values occurring at the same time as the maximum positive and negative bending moments. These forces are not necessarily the maximum forces.

(b) The substructure results include the maximum effects plus the axial forces corresponding to the maximum and minimum top and bottom moments, and the top and bottom moments corresponding to the maximum and minimum axial forces.

In all cases the maximum is the numerically largest positive value, and the minimum is the numerically largest negative value. The results are not printed for data checking runs.

C. LOAD DATA AND RESULTS PRINTOUT

The following load data and results are printed.

(1) Echo of load control information

(2) Echo of static load data.

(3) Loads on each node for each static load case. This helps provide a check on the data.

(4) Results for each static load case, as explained in the preceding section.

(5) Results for unit temperature increase at the deck level and for unit increase at the girder soffit. These results are computed and printed only if temperature loadings are to be applied.

(6) Echo of input data for each cable group and each cable. The cable slopes and the depths from the reference plane to the cable are also printed at the left and right ends of each cable segment. This helps to provide a check on the data.

(7) Echo of jacking data, and the cable forces at the fifth points of each cable segment after each jacking operation.

(8) Results for prestressing loads at transfer and after losses.

(9) Echo of input data for each live loading case, and the results for each envelope.

D. DESIGN CHECKING

The following design checking data and results are printed.

(1) Echo of checking control information.

(2) The following data for each load combination:

(a) Echo of strength data and load case combination factors.

(b) Results for girder, supports and hinges, including maximum values as for a live loading envelope.

(c) Echo of commands defining checking sections.

(d) For a data checking run, the location of each checking section.

(3) For checks of working load flexure the following values are printed at each checking section:

(a) Section location.

(b) Maximum moment, corresponding axial force, and stresses (ksi, compression positive) in top and bottom fibres.

(c) Minimum moment, corresponding axial force, and fiber stresses.

(d) If any fiber stress exceeds the allowable value in tension or compression, it is marked with asterisks.

(4) For checks of shear strength the following values are printed at each checking section.

(a) Section location.

(b) Maximum shear (k).

(c) Shear carried by concrete (k).

(d) Required stirrup area (sq. in/ft). If the minimum area governs, this area is marked with asterisks.

(e) Stirrup spacing (inches).

(5) For checks of ultimate moment capacity the following values are printed at each checking section:

(a) Section location.

(b) Area of prestressed steel (sq. in).

(c) Depth to centroid of prestressed steel (ft).

(d) Required moment capacity (k. ft).

(e) Moment capacity with no added mild steel (k. ft).

(f) Mild steel area required to augment moment capacity (sq. ins).

(g) Maximum permissible moment, based on a compression failure (k. ft). If this moment is less than the required moment, it is marked with asterisks.

E. STORAGE REQUIREMENTS AND FILE LENGTHS

The blank COMMON lengths required at each of four stages are printed. These stages are (1) static and thermal effects calculation, (2) prestressing effects calculation, (3) live load effects calculation, and (4) design checking.

The storage lengths required on scratch files TAPE1 and TAPE2 are also printed.

F. ERROR MESSAGES

A number of self-explanatory messages are printed by the program. Some errors are not fatal, and cause the run to be converted to a data checking run, so that an attempt is made to process the remaining input data. Other errors are fatal and cause the execution to terminate immediately.

NOTES

The following notes are referenced in the user's guide, and serve to explain the assumptions and procedures used in the idealization, analysis and design checking.

This section may also be read independently of the user's guide, to provide information on the capabilities of the computer program.

NOTE 1. GEOMETRIC DEFINITION OF STRUCTURE

Figure N.1(a) shows a bridge structure incorporating a variety of different features, and Fig. N.1(b) shows the idealized analysis model for this structure. The idealization procedure is sufficiently general to accommodate girders of virtually any configuration, is convenient for both computation and data storage, and is believed to be sufficiently simple for practical use.

The girder must first be divided into <u>spans</u> and <u>subspans</u>. The spans will always be the actual bridge spans, between supports. Subspans <u>must</u> terminate at the following discontinuities:

(1) Any support (i.e. the end of any span).

(2) Any internal hinge or expansion joint.

(3) Any prestressing cable anchorage.

Subspans may be terminated at any other point, if desired. Although it is not essential, it is advisable to terminate subspans at discontinuities of the following type, in order to obtain accurate results:

(1) Points at which large concentrated static loads are applied to the girder.

(2) Harp points in prestressing cables.

For analysis, the subspans must be subdivided into <u>elements</u>, which are separated by <u>nodes</u>. A subspan may consist of only a single element if desired. The idealized structure is analyzed as a finite element assemblage, with three degrees of displacement freedom at each node (X, Y and rotational displacements).

For calculation of the element stiffnesses, each element is assumed to be of constant cross section, with the section properties computed at the element midpoint. Fig. N.2 demonstrates that this assumption introduces little error in girders with variable cross section, provided each span contains approximately 10 elements. Nevertheless, it is advisable to place nodes at points where there are substantial discontinuities in the girder section. It is also necessary to provide several elements per span in order to obtain accurate live load envelopes when load trains are stepped across the span, as indicated in NOTE 12. For practical analysis, a minimum of 10 elements per span is recommended.

It is also important to note that when stress or strength computations are made, the actual section dimensions at the checking sections are used, not those at the element midpoint. That is, the element midpoint properties are used only for forming the structure stiffness and hence calculating the bending moment, shear force and axial force variations along the girder. These variations are not very sensitive to changes in the element stiffnesses.

At each node within a subspan an analysis section is established. Where a node is common to two subspans, two analysis sections are created, one immediately to the left of the node, in the first subspan, and one immediately to the right of the node, in the second subspan. The analysis sections serve two purposes, as follows.

(1) The values of bending moment, shear force and axial force are computed and printed at each analysis section. Two analysis sections are set up at nodes which are common to two subspans because substantial discontinuities of bending moment, shear force and axial force may occur at such nodes.

(2) The location of the cross section centroid is determined at each analysis section, and hence the slopes of the element centroidal axes between nodes are determined. These slopes are taken into account in forming the element stiffnesses, so that arching effects in the girder may be taken into account. However, the printed values of shear force and axial force are for vertical cross sections of the girder, rather than sections which are normal to the centroidal axis, although any differences should be small.

When checks of stress or strength are made, the checking sections may be specified to be anywhere along the girder. The actions at each checking section are determined by linear interpolation between the analysis sections, and the actual cross section dimensions at the checking section are computed. The checking sections are assumed to be vertical, rather than normal to the centroidal axis.

Spans, subspans, nodes, elements and analysis sections are all numbered sequentially along the bridge, beginning with 1 at the left end.

NOTE 2. SPECIFICATION OF NODE LOCATIONS

The node locations are defined by their X coordinates. The coordinate origin will typically be at the left end of the bridge, but may be at any other point. The nodes must be numbered sequentially along the bridge, starting with node 1 at the left end.

It will generally be necessary to specify the locations of only a few "control" nodes, and to allow the program to generate the coordinates of all remaining nodes. After the control node coordinates have been defined, the program identifies each set of undefined nodes, and interpolates them uniformly between the control nodes preceding and following the set. Thus, in a simple bridge it is necessary to define only the nodes at the span ends, and all other nodes will be equally spaced within the spans. If all spans are of equal length in such a case, it becomes necessary to specify only the first and last nodes, all others being generated. If internal hinges, discontinuous prestressing cables, etc., are present, more control nodes will usually be needed.

Control nodes coordinates may be specified absolutely, or as offsets from some other node. The use of offset nodes avoids the need to add up span lengths to compute control node coordinates within the second and subsequent spans.

NOTE 3. REFERENCE PLANE

For analysis purposes, all nodes are assumed to lie on a straight line, termed the reference plane. This plane is also the X coordinate axis.

If the girder has insignificant vertical curvature, as will generally be the case, it will be reasonable to ignore this curvature. It will then usually be convenient to position the reference plane in the deck surface, and no commands will be needed to define the reference plane location.

If the girder top flange has substantial vertical curvature, as may be the case if the program is being used to analyze a structure other than a typical bridge, then a reference plane must be formally established. The specified heights of the deck surface above the reference plane then define the top flange profile.

The error involved in ignoring vertical curvature is illustrated in Fig. N.3. The effects of the straight prestressing cable are clearly different on the two beams, so that the true top flange profile should be taken into account. However, if the girder curvature is small in comparison with the cable curvature, as will usually be the case in bridges, there will be little error in assuming that the deck is level.

NOTE 4. "SIMPLE" SUPPORTS

Each simple support restrains a supported node, as indicated in Fig. N.4. The support stiffness is defined by

$$\begin{Bmatrix} F_X \\ F_Y \\ M \end{Bmatrix} = \begin{bmatrix} k_X & & \\ & k_Y & \\ & & k_R \end{bmatrix} \begin{Bmatrix} \delta_X \\ \delta_Y \\ \theta \end{Bmatrix} \qquad \text{(N4.1)}$$

in which F_X, F_Y, M are the X, Y and moment reactions, respectively; δ_X, δ_Y, θ are the X, Y and rotational displacements of the support center relative to the support base; and k_X, k_Y, k_R are horizontal, vertical and rotational stiffnesses to be specified by the program user.

A simple support will typically represent a hinged or sliding bearing. Hinged bearings will be assigned both vertical (Y) and horizontal (X) stiffness, and sliding bearings vertical stiffness only. Neither would normally be assigned any rotational stiffness.

Simple supports can also be used to approximate vertical piers which are cast monolithically with the girder, typically by assigning appropriate vertical and rotational stiffnesses and ignoring any horizontal stiffness. If a pier can develop significant horizontal reactions, there may be significant interaction between horizontal force and bending moment in the pier, and it should be idealized more accurately as a substructure support.

The depth of the support center below the supported node (i.e. reference plane) may be assumed to be zero. However, this depth may have two influences, as follows.

(1) If a support develops significant horizontal reaction, then the effect of this reaction at the node is both a horizontal force and a moment. The effect may be particularly significant where arching action is developed by specifying horizontally stiff supports at significant distances below the girder centroidal axis, although in practice it would be rare to rely on such arching action.

(2) The program prints out the horizontal deformation of the support, which for a sliding bearing is the amount of sliding displacement. This displacement is equal to the horizontal displacement at the supported node plus the rotation at the node multiplied by the depth to the support. The node rotation effect may be significant, and hence it may be desirable to take the true location of the support center into account. Because specification of the support center location requires negligible additional effort, it is probably desirable to specify this location in all cases.

It is usual in analysis, to assume that supports are rigid. In fact, no support can be truly rigid. For example, a block of concrete 50 inches square and 200 inches high with a Young's modulus of 4000 ksi has an axial stiffness $EA/L = 5 \times 10^4$ k/in. Supports with stiffnesses of this order of magnitude will be rigid for practical purposes. Numerical sensitivity problems may result if very large stiffnesses (for example, 10^{20} k/in.) are specified to simulate "rigid" supports.

NOTE 5. SUBSTRUCTURE SUPPORTS

Substructure supports are intended for the representation of vertical or inclined piers, whether or not they are cast monolithically with the superstructure. Fig. N.5 illustrates the geometry of a substructure support of general type.

The substructure may be of variable eross section, and may be pin connected at either its top or its base. Because of the wide range of shapes which are possible, it is impractical to compute the substructure stiffness within the program, and the user is required to specify stiffness coefficients. These coefficients apply for that part of the substructure between its top and base. The eccentricities from the supported node at the top are taken into account automatically by the program, assuming that the diaphragm acts as a rigid block. If desired, stiffness coefficients accounting for these eccentricities could be specified, in which case the top of the substructure would be at the node and the eccentricities would be zero.

The axial stiffness of the substructure is computed as EA_{av}/L, where A_{av} is the average, or effective, area of the member, E = Young's modulus, and L = Length of substructure member. Ideally, the effective area should account for deformations of the diaphragm, footing and soil. However, it is probable that the superstructure behavior will be insensitive to changes in the axial stiffness of the substructure, so that precision is not essential.

The flexural stiffness of the substructure member is computed as

$$\begin{Bmatrix} M_t \\ M_b \end{Bmatrix} = \frac{EI_r}{L} \begin{bmatrix} k_{tt} & k_{tb} \\ k_{tb} & k_{bb} \end{bmatrix} \begin{Bmatrix} \theta_t \\ \theta_b \end{Bmatrix} \qquad \text{(N5.1)}$$

in which M_t, M_b = moments at substructure top and bottom, respectively; θ_t, θ_b = rotational deformations (i.e. rotations relative to the member axis) at top and bottom, respectively; I_r = reference moment of inertia, which is any convenient reference value; and k_{tt}, k_{bb} and k_{tb} are stiffness coefficients. For example, for a uniform member with assumed fixity at the footing,

$$\begin{Bmatrix} M_t \\ M_b \end{Bmatrix} = \frac{EI}{L} \begin{bmatrix} 4 & 2 \\ 2 & 4 \end{bmatrix} \begin{Bmatrix} \theta_t \\ \theta_b \end{Bmatrix} \qquad \text{(N5.2)}$$

in which I = actual moment of inertia.

The computation of k_{tt}, k_{bb} and k_{tb} should take into account both cross section variations and the degree of fixity at the footing. However, the slope of the substructure and the eccentricities at the diaphragm are taken into account by the program.

NOTE 6. INTERNAL HINGES

Each internal hinge connects a pair of sequentially numbered nodes, as indicated in Fig. N.6, by a link with user-specified stiffnesses. The link stiffness is defined by

$$\begin{Bmatrix} F_{XH} \\ F_{YH} \\ M_H \end{Bmatrix} = \begin{bmatrix} k_X & & \\ & k_Y & \\ & & k_R \end{bmatrix} \begin{Bmatrix} \delta_{XH} \\ \delta_{YH} \\ \theta_H \end{Bmatrix} \qquad \text{(N6.1)}$$

in which F_{XH}, F_{YH}, M_H are the X force, Y force and moment, respectively, transmitted by the hinge at the hinge center; δ_{XH}, δ_{YH}, θ_H are the relative X displacement, Y displacement and rotation, respectively between the connected nodes; and k_X, k_Y and k_R are specified horizontal, vertical and rotational stiffnesses.

A typical expansion joint will have $k_X = k_R = 0$, and k_Y equal to some large value. It should be noted that hinge connections in practice can not be completely rigid. To avoid numerical difficulties, stiffnesses no larger than 10^{10} k/ft. and 10^{15} k. ft./rad are recommended.

A typical internal hinge will have k_X and k_Y large, with $k_R = 0$. Some situations may arise in which k_R is nonzero.

As with a simple support, eccentricity effects associated with horizontal forces and displacements may be significant if the hinge center is above or below the reference plane. Hence, provision is made for specification of the depth of the connection centerline below the reference plane.

NOTE 7. LIVE LOAD IMPACT

As described in NOTE 12, the maximum effects produced by trains of loads are computed by (a) stepping the loads across the bridge, (b) analyzing the structure for each load location, and (c) accumulating results envelopes. That is, influence lines are not used. The maximum effects produced by AASHO lane loading are computed by a similar although somewhat more complex procedure.

Unfortunately, the live load impact requirements specified by AASHO appear to be designed for use with influence lines, and are difficult to apply when loads are stepped across the structure. Further, the rationale behind the AASHO impact formula appears to have been lost in antiquity.

When loads are stepped across a structure, the simplest and most rational procedure to account for impact is to multiply the load on any node by a factor which depends on either the position of the node or the position of some representative point in the load train. Studies of weights dropped from a height onto a spring will show that the maximum dynamic force developed in the spring decreases as the spring stiffness decreases. On this basis, smaller impact factors might be expected for loads applied near midspan, where the structure is flexible, than near the supports, where it is stiff. In fact the problem is more complex than this, because it involves the dynamic interaction of the load train with the bridge. Also, it might be logical to reduce the impact factors for long trains because of the small probability that all wheels will exert their maximum dynamic loads simultaneously. The development of a rational approach to the selection of impact factors for use when loads are stepped across the structure would therefore appear to require considerable research. Until such a procedure is available, the following approach has been adopted in the program.

(1) For each subspan, a moment impact factor must be specified. When the loads are applied to the structure in the analysis, the magnitude of the load on any node is multiplied by the moment impact

factor for the subspan in which the node lies. With the AASHO impact formula, the moment impact factor, F_M, given by

$$F_M = 1 + \frac{50}{L_M + 125}$$

in which L_M = loaded length (ft.) for moment computation, selected as follows:

(a) For all subspans except cantilever subspans, L_M = length of the span containing the subspan.

(b) For cantilever subspans, L_M = subspan length.

This procedure should give moment envelopes very close to those obtained by the procedure recommended in the California Manual of Bridge Design Practice [1].

(2) For the calculation of maximum shear at any analysis section, a different impact factor, F_S, is implied [1], given by

$$F_S = 1 + \frac{50}{L_S + 125}$$

in which L_S = loaded length for maximum shear, which is equal to the larger of the distances from the analysis section to the two adjacent supports. The difference between F_M and F_S is taken into account in the analysis by means of a modifying factor, f, given by

$$f = \frac{F_S}{F_M}$$

which varies with the analysis section location. During the analysis, the live load shears computed at each analysis section are scaled by the factor f for that section. The factor f is required to be specified at each end of each subspan, and is assumed to vary linearly along the subspan. A typical calculation for f is shown in Fig. N.7.

It can be seen that f wil be 1.0 for analysis sections near the span ends, and is unlikely to differ much from 1.0 at any section. Hence, it may be reasonable in analysis to assume a value of 1.0. If so, the fields containing f in the input data may be left blank.

NOTE 8. CROSS SECTION DIMENSIONS

Figure N.8(a) shows the cross sectional shape assumed by the program at any section. Fig. N.8(b) shows a cross section of the type more likely to occur in practice. The sample calculation in Fig. N.8 demonstrates how the equivalent dimensions for the assumed cross section can be computed. Fillets have been ignored in this example, but can easily be included.

With this procedure, the cross section moment of inertia will be slightly in error, because the moments of inertia of the equivalent flanges about their own centroidal axes will not be identical to those of the actual flanges. However, these flange inertias contribute comparatively little to the inertia of the complete cross section, and hence any errors will be small.

If the cross section area and moment of inertia are known, rather than the section dimensions, equivalent dimensions can be computed as follows.

$$(1) \quad t_t = t_b = b_w = 10^{-6} \qquad \text{(N8.1)}$$

$$(2) \quad d = 2\sqrt{I/A} \qquad \text{(N8.2)}$$

$$(3) \quad b_t = b_b = 0.5 \times 10^{6} \times A \qquad \text{(N8.3)}$$

These formulae assume an exceedingly thin walled I beam with very wide flanges. Obviously, stress and strength checks can not be carried out with this type of input.

The cross section dimensions are specified span by span, independently of the node locations, the top flange, bottom flange, web and girder depth being specified separately for each span.

The top flange, for example, may be subdivided into several segments along the span length, where the flange thickness and width are permitted to vary linearly within each segment. A new segment must be created at each discontinuity in width or thickness, and several segments may be needed for complex variations. The bottom flange is specified similarly.

Typically, a single segment will be sufficient for the top flange, because its section is usually constant, and three segments will be required for the bottom flange, to account for the uniform thickness portion near midspan and the flares at each end. Although the dimensions can usually be specified without considering the node locations (and hence the data need not be modified if the node locations are changed), it may be advisable to place a node at large discontinuities in the cross section, so that the girder element stiffnesses are computed accurately (see NOTE 1).

The web dimensions and girder depth are specified similarly, segment by segment. The girder depth may be constant or may vary linearly or quadratically in any segment. There need be no relationship between the segment subdivisions for the top flange, bottom flange, web and girder depth.

Where the flange dimensions and/or girder depth vary, the equivalent cross section dimensions will typically be computed at the segment ends, and then linear variations of flange width and thickness will be assumed. This linear variation will usually be adequate to ensure that the section properties of the assumed cross section are sufficiently close to the actual section properties at all points. However, for girders with substantial cross section variations it may be advisable to compare the assumed and actual section properties at the segment midpoints, and to subdivide into shorter segments if necessary.

Any specified segment lengths should sum to the span length. If the sum differs from the span length by more than 2%, a warning message is printed, and the segment lengths are all scaled, by the same factor, to give the correct sum. Execution of the problem is not interrupted.

NOTE 9. SIMILARITY OPTION

Some or all properties of one span will often be identical to, or the mirror image of, the properties of some other span. If so, it is not necessary to re-specify the similar data.

If, for example, the top flange of span 3 is the same as that of span 2, which has already been specified, then the number of top flange segments, NTS, for span 3 may be input as zero, and the similar span number, NSIMT, input as 2. The top flange data for span 3 must then be omitted, and the program uses the data already available for span 2.

This option may be exercized for the top flange, bottom flange, web and girder depth separately or in any combination. Also it is not necessary for the span lengths of similar spans to be identical, but only for the segments to be the same proportions of the span lengths in both spans. This is because the program stores the segment lengths as proportions of the span length rather than as actual distances.

The option can also be applied where the dimensions in one span are the mirror image, with respect to distance along the span, of those in a previously specified span. In this case the similar span number is set to <u>minus</u> the number of the similar span.

The section geometry data as input and as finally stored by the program are both printed, so that a check can be made on whether the similarity option has been used correctly.

NOTE 10. STATIC LOADS

10.1 Load Options

Static loads are all assumed to be of long duration, so that the long term elastic moduli are used. This affects only the computed displacements and the stresses due to support displacement.

A static load case can consist of any combination of point loads on the nodes, distributed loads along the girder length, and loads due to support displacement. There is no limit to the number of static load cases which may be specified.

10.2 Automatic Dead Load Computation

The program will compute the girder self weight automatically if required. The calculation is carried out element by element, the weight of an element being the product of the element length, the average of the cross section areas at the element ends, and the concrete density. The element weight is then applied to the structure as statically equivalent point loads on the adjacent nodes, assuming a linear variation of cross sectional area along the element (i.e., not necessarily one half at each node). Corrections to the shear forces computed for these nodal loads are made to allow for the fact that the loads are actually distributed over the element lengths.

Typically, automatic self weight computations will be specified for the complete length of the bridge (i.e. there will be only a single self weight region, from the first to the last subspan). However, for greater flexibility the program permits these computations to be carried out for several separate regions should the user desire.

10.3 Additional Distributed Loads

Additional distributed loads will typically be specified to account for surfacing, guardrails, etc., which are not included in the automatic self weight calculations. However, the girder self weight could also be included by specifying it as a distributed load if desired. Diaphragms can be represented as point loads or "smeared" and represented as distributed loads.

Distributed loads will typically be uniform along the girder, but provision is made for linearly or quadratically varying loads to be specified. The loading is applied element by element, assuming a uniform load on any affected element equal to the load intensity at the element midpoint. One half of the load on the element is then applied as a point load on each of the two adjacent nodes. Again, corrections to the computed shears are made to allow for the fact that the loads are distributed over the elements.

If distributed loads are specified in overlapping regions, the total load is the sum of the separate loads.

10.4 Support Displacement

In effect, a simple support consists of a series of three independent springs, with stiffnesses k_X, k_Y and k_R respectively, connecting the support center (not necessarily the supported node) to a rigid base, as shown in Fig. N.4. This is essentially a correct idealization of an actual bearing, the spring deformations corresponding to the deformations of the bearing structure.

When displacements, are specified at a simple support, they are assumed to be applied to the rigid base, not directly to the supported point. Again, this is essentially a correct idealization of the manner in which support displacements are transmitted to an actual structure. If, say, the stiffness k_X is very large, then the X spring will deform very little, and the displacement produced at the support center will be essentially identical to the base displacement. The spring force will be printed by the program, and will be the reaction induced by support displacement. On the other hand, if k_X is zero, any X displacement at the base will simply deform the spring, without inducing any force. The spring deformation is printed with the computer output, and would correspond in this case to the movement at a sliding bearing. If the support spring has a stiffness which is comparable with that of the girder, it will both deform significantly and develop a significant reaction. Both the spring deformation and the displacement at the support center will then be significantly different from the specified base displacement. In all cases, the sum of the spring deformation and the displacement at the support center will equal the specified base displacement.

The procedure for specified settlements at the base of a substructure is similar. The base of the substructure is subjected to the specified displacements. These displacements plus the substructure deformations are then equal to the displacements at the substructure top. The axial forces and bending moments developed in the substructure are printed by the program.

NOTE 11. THERMAL LOADS

The program will compute both thermal deformations and the forces due to restraint of thermal expansion. However, the analysis is limited to simple distributions of temperature rise which are (a) constant with respect to length along the girder at the deck level and girder soffit, and (b) vary linearly with respect to depth at any cross section. Hence, localized thermal expansion effects can not be considered, although it would not be difficult to extend the program if a demand existed.

Analyses for two basic thermal loadings are first carried out by the program, namely (a) unit temperature rise at the deck level only, and (b) unit temperature rise at the girder soffit only. Any values of deck level and girder soffit temperatures may then be specified for a load combination (See NOTE 13), and the thermal effects obtained by simple superposition of the two basic cases.

NOTE 12. PRESTRESSING CABLES

12.1 Cable Groups

The prestressing cables may be divided into groups if desired, or specified as a single group. The analysis sequence is as follows:

(1) The cable profile and stressing sequence is read for the first cable in the group. Friction loss calculations are carried out, to obtain cable forces.

(2) The loads exerted on the girder by this cable are determined.

(3) Steps 1 and 2 are repeated for each cable in the group.

(4) The bending moments, shear forces, etc., produced by the prestressing loads on the girder are determined for the group, both at transfer and after losses.

(5) Steps 1 through 4 are repeated for the next cable group.

When load combinations are specified (see NOTE 13) separate scaling factors can be specified for the effects produced by each cable group. Hence, the influence of changing the prestress in some of the cables while keeping the remaining cables constant can be explored. Also, some cable groups can be deleted entirely if desired, by specifying zero scaling factors.

12.2 Cable Profile, Stressing Procedure

The procedure for determining friction losses is based on the well known formula

$$F_2 = F_1 e^{\pm(\mu\theta + kL)} \tag{N12.1}$$

in which F_2 = force at point 2 in cable; F_1 = force at point 1, a distance L from 2; μ = coefficient of friction between cable and duct; k = wobble factor; and θ = change in slope of cable, assumed to be uniformly distributed over length L. The sign of the exponent varies depending on whether the force increases or decreases from point 1 to point 2. If a cable is harped, and is tensioned after placing in its final position, the change in cable tension across the harp point can be obtained from Eq. N12.1, with θ = change in cable slope at point and $L = 0$. Although it would be rare for friction loss of this type to occur, the situation is considered in the analysis. Pretensioned cables should be assigned zero friction coefficients, so that the friction losses are computed to be zero.

Each cable must be specified to consist of a number of straight or parabolic segments, and each segment is further subdivided by the program into ten equal subsegments. Eq. N12.1 is applied in turn to each subsegment, beginning at the jacking end of the cable, where the cable force is known. If the cable is initially unstressed, the process is carried through to the opposite anchorage. However, if the cable has been subjected to a prior jacking operation, the influence of a change in jacking force may extend over only a part of the cable length.

The program permits any sequence of jacking operations to be specified, including application of both increases and decreases in cable force. The cable extension for each operation is also computed. However, only the cable extension is calculated, and the influence of shortening in the concrete girder is not included in this part of the analysis.

Provision is also made for cable extensions to be specified, rather than changes in cable force. When an extension is specified, an iterative procedure is used to calculate the change in jacking force which produces the specified extension. This option is necessary for considering the effects of specified amounts of anchorage draw-in. Again, however, concrete deformations are not considered.

Any cable may be subjected to an arbitrary stressing sequence, as indicated in the user's guide. The stressing operations are assumed to take place consecutively, not concurrently. Hence, if the order of stressing operations is important, the stressing operations should be specified to simulate the actual operations as closely as possible.

The volume of input data defining the stressing operations will be reduced substantially if the "standard" operations recognized by the program are used. These operations consist of stressing to a temporary level (typically 0.8 f's), slackening the prestress to the working level (0.75 f's or 0.7 f's), and then allowing for anchorage draw-in, first at the left anchorage and then at the right anchorage.

Creep losses in the prestressing steel may be specified either as reduction by a specified stress (typically 25 ksi) or reduction by a specified percentage of the steel stress at transfer. It is possible that a more sophisticated creep loss analysis could be added to the program.

12.3 Girder Loads from Prestressing Cables

When a cable passes through a curved duct in a girder, it transmits forces to the girder which are both normal and tangential to the duct. The normal force at any point depends on the cable force and duct curvature, and the tangential force on the friction between the cable and duct. In addition, the cable exerts forces on the girder at the anchorages. The complete set of forces which any cable exerts on a girder, including the anchorage forces and the normal and tangential forces along the duct, must form a self-equilibrating system. In the analysis of the girder, it is not, in general, correct to ignore the tangential forces. These forces may be of significant magnitude and may act substantially above or below the girder axis, effectively loading the girder with a distributed moment loading, and possibly contributing significantly to the girder bending moments.

The procedure used in the computer program is illustrated in Fig. N.9. For the purposes of calculating friction losses, the cable is assumed to consist of smoothly curved segments, with continuously varying cable forces (Figs. N.9 (a) and (b)). However, for the purposes of calculating the cable loads on the girder, each subsegment is assumed to be straight, with a constant cable force (Fig. N.9 (c)). This is equivalent to assuming that the normal and tangential load transfer between the cable and girder occurs only at the subsegment ends. With this assumption, it is easy to calculate the transferred load at each subsegment end, as shown in Fig. N.9 (d). The difference between the actual distributed load and the assumed point loads is small for practical purposes.

The calculated loads act at the ends of the cable subsegments, and not, in general, at the girder nodes. It is assumed for analysis that loads are applied to the girder only at the nodes, except that corrections to the shear forces are made to account for the actual load location. The loads at the ends of the cable subsegments are transferred to the girder nodes by the procedure shown in Fig. N.9 (e).

Although harped cables will be rare in bridge structures, it may be noted that at harp points substantial concentrated loads may be applied to the girder. Hence, to avoid difficulties with differences in shear force to the left and right of such points, it is recommended (see NOTE 1) that girder subspans be terminated at such points. If this is not done, only one shear force will be computed at the corresponding analysis section, and errors may result. Also, if a harp point lies between two girder nodes, errors in bending moment values may result.

Subspans are required to terminate at cable anchorages for a similar reason, namely that the concentrated anchorage loads can produce discontinuities in the bending moment, axial force and shear force in the girder, and it is therefore desirable to compute separate values immediately to the left and right of such loads. The form of the program input data prevents cable anchorages from being placed at points other than subspan ends. However, the program carries out no check on the location of harp points, and the user must decide whether to terminate a subspan at each such point.

Elastic shortening effects in the concrete during the stressing operations are ignored in the friction loss calculation. However, these effects are taken into account when the prestress loads are applied to the girder. Hence, axial displacements due to prestress are computed, and prestress reductions in the concrete due to restraint of axial deformation by the supports are calculated, together with the forces developed in the supports. The cable forces are assumed to comprise constant loads on the girder for this part of the analysis.

For considering the effects of prestress at transfer the short term Young's modulus for the girder material is assumed, whereas for prestress after losses the long term modulus is assumed. As in the other loading conditions, the change in assumed modulus affects the computed displacements only, and not the stresses within the structure.

NOTE 13. MOVING LIVE LOADS

13.1 Load Trains

Analyses can be carried out for moving live loads of a variety of types, including AASHO H and HS trucks, AASHO lane loads, California "P" vehicles and arbitrary load trains specified by the user.

For the truck and arbitary train loadings, the envelopes of maximum positive and negative bending moment, shear force and axial force are determined. A principal axle in the train is identified (for H and HS trucks the drive axle, for P vehicles the second axle and for arbitrary trains an axle specified by the program user). The load corresponding to that axle is automatically applied at each node of the girder in turn, with the remaining loads positioned to maintain the specified axle spacings. For each position of the principal axle, a set of nodal displacements, bending moments, axial forces and shear forces is determined. If the positive (or negative) value of any effect exceeds (or is less than) the previously determined maximum (or minimum) value of the effect at any node, the new maximum (or minimum) is retained. Hence, the envelope values are determined.

Provision is made in the program for load trains to be stepped across from left to right only, from right to left only, or in both directions. If stepping from both directions is specified, two separate analyses are carried out with the principal axle on each node, the second analysis with the axle positions reversed relative to the principal axle. For HS loadings, a series of different spacings between the drive and trailer axles may be specified by the user. For each spacing a new train of loads is established, and a separate series of analyses carried out.

Because the only load locations considered are those with the principal axle at the nodes, it is important to subdivide each span into a substantial number of elements to ensure accuracy. A certain amount of experiment and experience will be necessary to select appropriate element subdivisions. However, because the computer costs are likely to be small, it is probably wise to be generous in subdividing the spans. As indicated in NOTE 1, no fewer than 10 elements should be specified in any span, and more should be used if convenient.

13.2 AASHO Lane Loading

The computational procedure for the AASHO lane loading is different from that for a train of loads. Two special features must be taken into account, namely (a) the need to position the distributed loads to produce the maximum effects of each type, and (b) the need to apply one point load for positive bending moments and two for negative bending moments.

The maximum effects due to the distributed load are determined by applying, on each element in turn, a "patch" of distributed load, and determining the bending moments, shear forces and axial forces produced by this load. If any action is positive, it is added to an array which accumulates positive values, and if negative to an array which accumulates negative values. After forces have been applied to all elements, the arrays contain the maximum effects for the distributed load, considering all possible patterns of loading.

The AASHO lane loading also requires consideration of a point load for calculating maximum positive bending moments, a pair of point loads in adjacent spans for calculating maximum negative bending moments, and a point load of different magnitude for calculating shear forces.

The envelope values of shear force, axial force and positive bending moment are easily determined by the load train stepping procedure previously described, using a train consisting of only a single load of appropriate magnitude. However, the determination of negative bending moment envelopes is complicated by the need to consider two point loads in different spans, and is carried out as follows.

(1) Separate negative moment envelopes are calculated and stored for single point loads travelling across each span of the girder in turn.

(2) At each node, the negative moment values from the different envelopes are compared, and the two numerically largest values found.

(3) These two values are added together. The resulting moment is the largest negative value produced at the node by point loads in two different spans. These two loads are not necessarily applied in adjacent spans, although this will almost always be the case.

After the maximum effects envelopes for the distributed and point load parts of the lane loading have been determined, they are added together to produce the final envelopes.

13.3 Moment-Force Interaction

The design of a girder or substructure cross section for bending strength will depend on both the bending moment and the axial force on the section. This introduces a complication with moving live loads, because the proportions of bending moment and axial force will vary with the load location, and it may not be obvious which load location is critical. Further, it may not be reasonable to envelope bending moment and axial force separately, and then to design for the maximum values of each. This is because the two maxima are not necessarily produced by the same live load, and hence do not necessarily occur simultaneously.

Fortunately, the axial forces produced on girder cross sections by live loading can be expected to be small, and will commonly be zero. Hence, in the calculation of maximum live load effects for the girder, it is assumed that the bending moments are of primary importance, and these are enveloped as described previously. Axial forces, however, are not enveloped in this way. Instead, for each maximum bending moment, the value of the axial force occurring at the same time as the maximum moment is retained. The printed live load envelopes therefore show these corresponding axial forces, rather than the maximum axial forces.

For substructures the situation is different, because the live load axial forces are likely to be more substantial. Also, maximum substructure moments are likely to be produced by loading patterns which are quite different from those producing maximum axial forces. Nevertheless, live load effects on substructures are likely to be relatively small in comparison with the total effects produced by all loadings, so that precise live load calculations are not essential. The following procedure is followed, and should provide ample information for design purposes.

(a) The largest bending moments at the substructure top and bottom are determined by the enveloping procedure, and for each maximum the corresponding axial force is retained (i.e. the axial force produced by the same loading pattern).

(b) The largest axial forces in the substructure are determined by the enveloping procedure, and the corresponding top and bottom bending moments are retained.

It may be noted that it may also be desirable to consider interaction of shear force and bending moment in the design for shear strength. A procedure similar to that used for moment-force interaction could be included in the program, with maximum bending moments being retained together with the corresponding shears, and maximum shears being retained together with the corresponding moments. However, current bridge design practice for checking shear strength does not require consideration of moment-shear interaction, and hence in the current program the maximum effects alone are retained.

NOTE 14. LOAD COMBINATIONS, CHECKING SECTIONS

The results of the analyses for static, thermal, prestress and live loadings are stored by the program and are available to be combined in any manner. A load combination is established by specifying the following data:

(a) A combination factor for each static load case.

(b) Temperature changes at the deck and soffit levels.

(c) A combination factor for each prestressing cable group, and an indication of whether the cable forces at transfer or after losses are to be considered.

(d) A combination factor for each live load envelope.

A zero scale factor indicates that the corresponding loading is to be ignored. To produce the combined effects, the bending moments, shear forces, axial forces and support reactions for the static, thermal and prestress load cases are multiplied by their corresponding combination factors, and added together. The live load envelopes are similarly multiplied by the combination factors. However, these envelopes are not added, but are further enveloped to obtain the maxima for all envelopes. These maxima are then added to the combined results for static, thermal and prestress loadings to give the results for the load combination.

The girder actions are printed at each analysis section for each load combination. A series of cross sections at which stress or strength checks are to be made can then be defined. These checking sections may be positioned at any locations within any of the girder spans. Where a checking section lies between two analysis sections, the program determines the checking section actions by linear interpolation.

To reduce the effort required to specify checking section locations, the program contains a default option in which all of the analysis sections in any span automatically become checking sections.

NOTE 15. CHECKS OF WORKING LOAD FLEXURE

Flexural stresses are computed using the formula

$$F = \frac{M}{S} + \frac{P}{A} \qquad \text{(M15.1)}$$

in which F = stress; M = bending moment; S = section modulus for top or bottom fiber of section; P = axial force (force corresponding to maximum moment used for live load, see NOTE 13.3); and A = section area.

At each checking section, the top and bottom fiber stresses for maximum and minimum moments are computed and printed. If the load combination includes no live loads, the maximum and minimum moments will be equal.

Maximum allowable stresses may be specified if desired. Any computed stresses which exceed the allowable values are marked by asterisks in the computer output.

Typically, the checking sections for working load flexure will be specified to correspond to the analysis sections, using the default option for checking section locations (See NOTE 14). However, any other section locations can be specified if desired.

NOTE 16. CHECKING FOR SHEAR STRENGTH

The maximum positive and negative shear forces are computed at each checking section. The shear force with the largest absolute value, V_u, is used to check the shear strength. Interaction between bending moment and shear is not considered.

The design checking is based on the following procedure [1, 2, 3].

(1) The effective depth of the cross section is assumed to be

$$jd = 0.9d \tag{N16.1}$$

in which d is permitted by the AASHO specification to be the larger of (a) the depth from the compression fiber to the centroid of prestressing steel and (b) 0.8h, where h = total depth of section. The assumption

$$d = 0.8h \tag{N16.2}$$

is made. This is conservative for cases where d exceeds 0.8h, and is in accordance with usual practice [1].

(2) The shear stress permitted on the concrete alone must be specified by the user. This is typically

$$f_v = 0.06f'_c \text{ , but } \not> 180 \text{ psi} \tag{N16.3}$$

in which f'_c = crushing strength of concrete (psi). The 180 psi value will generally govern. Hence, the shear force permitted on the concrete alone is

$$V_c = f_v \, b_w \, jd \tag{N16.4}$$

in which b_w = web thickness.

(3) The yield stress of the stirrup steel, f'_y , must be specified by the user. The required shear area, A_v , per unit length of girder, is given by

$$A_v = \frac{(V_u - V_c)}{2f'_y \, jd} \qquad (N16.5)$$

but not less than

$$A_v = 0.0025 \, b_w \qquad (N16.6)$$

(4) The area per stirrup, A'_s , must be specified by the user. The stirrup spacing is then

$$s = \frac{A'_s}{A_v} \qquad (N16.7)$$

NOTE 17. ULTIMATE MOMENT CAPACITY

Separate checks are required to be specified for negative and positive ultimate moment capacity. Typically only negative moments will be considered, at sections near the supports.

The checking procedure is believed to be consistent with usual practice [1, 2, 3]. The moment capacity of the section with no additional mild (nonprestressed) steel reinforcement is computed, and if the required capacity exceeds this value, the area of mild steel required to augment the capacity is determined. The maximum permissible bending moment is also printed, corresponding to the steel percentage which would cause a primary compression failure. The checking procedure is as follows.

(1) Those prestressing cables which cross the checking section are determined. The steel area and the depth from the compression fiber to the steel centroid are calculated. To permit elimination of cables which may be close to the compression fiber, the program permits a minimum depth proportion to be specified. Any cables which are located so that the depth of the cable is less than this proportion of the total depth are ignored. This provision makes only a crude allowance for complex cable configurations in which cables may pass close to both the top and bottom flanges at a single section, and its use is not recommended except to obtain rough estimates of strength in such cases. The formulae for computation of ultimate moment capacity are not necessarily applicable to such cable configurations, and the checking procedure incorporated into the program should be used with caution.

(2) The stress in the prestressing steel at ultimate moment, f_{su}, is computed as

$$f_{su} = f'_s \left(1 - 0.5 \frac{pf'_s}{f'_c}\right) \tag{N17.1}$$

in which f'_s = ultimate strength of prestressing steel; f'_c = crushing strength of concrete; and

$$p = \frac{A_s}{bd} \tag{N17.2}$$

in which A_s = area of prestressing steel; b = width of compression flange (assumed to be fully effective); and d = depth to centroid of prestressing steel. Eq. N17.2 assumes bonded prestressing cables.

(3) The area of mild steel, A_s', is assumed to be zero to initiate an iterative solution process. The depth to the neutral axis assuming this axis lies in the flange, d_{nf}, is computed from

$$d_{nf} = 1.4\, d\, \frac{p f_{su} + p' f_y'}{f_c'} \tag{N17.3}$$

in which f_y' = yield stress of mild steel; and

$$p' = \frac{A_s'}{bd} \tag{N17.4}$$

(4) If d_{nf} is less than the flange thickness, the moment capacity of the section is computed as the sum of two parts, namely M_{up} = moment capacity of prestressing steel, and M_{um} = additional moment capacity for unit area of mild steel. These are given by

$$M_{up} = A_s\, f_{su}\, (d - 0.43\, d_{nf}) \tag{N17.5}$$

and

$$M_{um} = f_y'\, (d_m - 0.43\, d_{nf}) \tag{N17.6}$$

in which d_m = depth to centroid of mild steel.

(5) If d_{nf} exceeds the flange thickness, the neutral axis lies in the web. The moments M_{up} and M_{um} are then computed as follows.

$$A_{sf} = \frac{0.85 f_c' \,(b - b_w)\, t}{f_{su}} \tag{N17.7}$$

in which A_{sf} = prestress steel area required to develop the strength of the flange; b_w = web thickness; and t = flange thickness.

$$A_{sr} = A_s - A_{sf} \tag{N17.8}$$

in which A_{sr} = prestress steel area available to develop strength of web. A_{sr} may have a negative value in later steps of the iteration, when A_s' is not zero.

$$p_w = \frac{A_{sr}}{b_w d} \tag{N17.9}$$

$$p_w' = \frac{A_s}{b_w d} \tag{N17.10}$$

$$d_n = 1.4d \, \frac{p_w \, f_{su} + p_w' \, f_y'}{f_c'} \tag{N17.11}$$

in which d_n = depth to neutral axis.

$$M_{up} = f_{su} \, A_{sf} \, (d - 0.5t) + A_{sr} \, f_{su} \, (d - 0.43 \, d_n) \tag{N17.12}$$

$$M_{um} = f_y' \, (d_m - 0.43 \, d_n) \tag{N17.13}$$

(6) If this is the first iteration, then M_{up} is the ultimate moment capacity with zero mild steel, and this value is saved for subsequent printing.

(7) An estimate of the required mild steel area is computed as

$$A_s' = \frac{M - M_{up}}{M_{um}} \tag{N17.14}$$

in which M = required ultimate moment capacity, which is the moment computed for the specified checking section and load combination, but ignoring the bending moments due to prestress.

If A_s' is within either one percent or 0.5 sq. ins. of the area from the previous iteration, the process is assumed to have converged. If not, the iteration is repeated from step 3. If convergence is not obtained after 10 cycles, a warning message is printed and the iteration ceases.

(8) The maximum permissible moment capacity is computed as the smaller of

$$\text{(a)} \quad 0.25 f_c' \, b d^2 \qquad \text{(N17.15)}$$

and

$$\text{(b)} \quad 0.25 f_c' \, b_w \, d^2 + 0.85 f_c' \, (b - b_w) \, t \, (d - 0.5t) \qquad \text{(N17.16)}$$

If this moment is less that the required moment capacity, it is marked with asterisks in the computer output.

It is important to note that Eqs. N17.15 and N17.16 assume that any mild steel is placed at the level of the prestressed steel centroid. Hence, for cross sections where the prestressing cables are not near the tension flange, these equations may underestimate the maximum moment capacity. If the computer printout indicates insufficient maximum capacity at any cross section, this should be interpreted only as a warning, not as a definite indication of under-design. If the depths to the prestressed and non-prestressed steel centroids are significantly different at such a section, the maximum moment capacity should be checked using more refined procedures. Such procedures may be incorporated into the computer program at a later date.

NOTE 18. WORKING STRESS COMPUTATIONS AT NON-PRESTRESSED CROSS SECTIONS

Required steel areas in the top and bottom flanges are computed by an iterative process. If the required tension steel area is less than that for a balanced design, then compression steel areas are ignored. If the required tension steel exceeds that for a balanced design, the required area of compression steel is also determined. The equations used are those commonly applied in working stress design. The iterative procedure is as follows.

(1) Assume for the first cycle of iteration that the concrete stress is the maximum allowable. Because the steel area will be selected to be a minimum, the steel stress will always be the maximum allowable. Hence

$$kd = \frac{nf_c}{nf_c + f_s} d \tag{N18.1}$$

in which d = depth to reinforcement; kd = depth to neutral axis; f_s = allowable steel stress; f_c = concrete stress, initially equal to the allowable; and n = modular ratio.

(2) For these stresses, compute the resisting moment, M_R. If kd < t, where t = flange thickness, then

$$M_R = 0.5\ f_c\ bkd\ (d - 0.33kd) \tag{N18.2}$$

in which b = flange width. If kd > t, then

$$\begin{aligned} M_R = {} & \frac{(kd - t)}{kd} f_c\ (b - b_w)\ t\ (d - 0.5\ t) \\ & + 0.5\ f_c\ b_w\ kd\ (d - 0.333\ kd) \\ & + 0.5\ \frac{t}{kd}\ f_c\ (b - b_w)\ t\ (d - 0.333\ t) \end{aligned} \tag{N18.3}$$

in which b_w = web thickness.

At the first cycle, the value of M_R is the balanced moment capacity. This value is retained for subsequent printing. If the bending moment on the cross section exceeds the balanced capacity, the procedure moves to step 5 to compute the required amounts of tension and compression reinforcement. If the bending moment is less than the balanced capacity, the procedure continues with step 3.

(3) Modify the assumed concrete stress, f_c, to f_{cm}, as follows

$$f_{cm} = \frac{M}{M_R} f_c \tag{N18.4}$$

in which M = moment on cross section. If f_{cm} differs from f_c by no more than 2 percent, proceed to step 4. Otherwise put $f_c = f_{cm}$ and repeat from step 1.

(4) After convergence the concrete force, F_c, is computed as

$$\begin{aligned} F_c &= \frac{(kd - t)}{kd} f_c (b - b_w) t \\ &+ 0.5 f_c b_w kd \\ &+ 0.5 \frac{t}{kd} f_c (b - b_w) t \end{aligned} \tag{N18.5}$$

Hence, the required steel area, A_s, is given by

$$A_s = \frac{F_c}{f_s} \tag{N18.6}$$

(5) If the cross section moment exceeds the balanced capacity, the tension steel area, A_s, is the sum of A_{s1} and A_{s2}, where A_{s1} is given by Eqs. N18.5 and N18.6 and A_{s2} is given by

$$A_{s2} = \frac{(M - M_R)}{f_s (d - d')} \tag{N18.7}$$

in which d' = depth to compression reinforcement. The effective stress in the compression reinforcement, f_{sc}, is then given by

$$f_{sc} = (2n - 1) \frac{(kd - d')}{kd} f_c \tag{N18.8}$$

in which a modular ratio 2n is implied for the compression steel. Hence, the area of compression reinforcement, A_s', is

$$A_s' = \frac{f_s}{f_{sc}} A_{s2} \tag{N18.9}$$

NOTE 19. ULTIMATE MOMENT COMPUTATIONS AT NON-PRESTRESSED CROSS SECTIONS

Required steel areas for positive and negative moment are computed by an iterative process. The cross section is assumed to be singly reinforced in each case, any steel in the compression flange being ignored. Axial forces on any cross section are assumed to be small, and are ignored. The iterative procedure is as follows.

(1) Assume initially that the compression stress block extends just through the slab thickness. That is, assume

$$a = t \qquad \text{(N19.1)}$$

in which a = stress block depth; and t = flange thickness. The concrete force, F_o, for this stress block depth is then

$$F_o = 0.85f'_c bt \qquad \text{(N19.2)}$$

in which b = flange width; and f'_c = concrete crushing strength. The resisting moment, M_R, is therefore

$$M_{Ro} = \phi\, F_o\, (d - 0.5t) \qquad \text{(N19.3)}$$

in which d = depth to flexural steel; and ϕ = strength reduction factor (typically 0.9). If the moment on the cross section, M, is less than M_{Ro}, then the stress block lies entirely in the flange, and the iterative procedure of step 2 is used. If M exceeds M_{Ro}, then the stress block extends into the web and the procedure of step 3 is followed.

(2) Let F_n be the concrete force at iteration n, where F_o is given by Eq. N19.2. Estimate this force from

$$F_n = \frac{M}{M_R} F_{n-1} \qquad \text{(N19.4)}$$

Hence, the estimated stress block depth, a_n, is

$$a_n = \frac{F_n}{0.85 f'_c b} \tag{N19.5}$$

and the resisting moment is

$$M_{Rn} = \phi \, F_n \, (d - 0.5 \, a_n) \tag{N19.6}$$

The iterative procedure is repeated from Eq. N19.4 until F_n converges to within 2 percent of F_{n-1}. The required flexural steel area, A_s, is then given by

$$A_s = \frac{F_n}{f'_y} \tag{N19.7}$$

in which f'_y = steel yield stress.

(3) Again, let F_n be the concrete force at iteration n, where F_o is given by Eq. N19.2, and estimate a new value from

$$F_n = \frac{M}{M_R} F_{n-1} \tag{N19.8}$$

Divide F_n into F_w, developed in the web, and F_f, developed in that part of the flange outside the web. F_f is constant, and given by

$$F_f = 0.85 f'_c \, (b - b_w) \, t \tag{N19.9}$$

Hence, at iteration n the force F_{wn} is given by

$$F_{wn} = F_n - F_f \tag{N19.10}$$

The estimated stress block depth is therefore

$$a_n = \frac{F_{wn}}{0.85 f'_c b_w} \tag{N19.11}$$

and the resisting moment is

$$M_{Rn} = \phi\, F_f\,(d - 0.5t) + \phi\, F_{wn}\,(d - 0.5\, a_n) \qquad \text{(N19.12)}$$

The iterative procedure is repeated from Eq. N19.8 until F_n converges to within 2 percent of F_{n-1}. The required flexural steel area is then

$$A_s = \frac{F_f + F_w}{f'_y} \qquad \text{(N19.13)}$$

(4) The amount of flexural steel may not exceed 75 percent of the "balanced" amount, at which the concrete strain reaches .003 at the same time that the steel reaches its specified yield stress.

The depth to the neutral axis in this case is given by

$$kd = \frac{.003\, E_s}{(.003\, E_s + f'_y)}\, d \qquad \text{(N19.14)}$$

Hence, the depth of the stress block, a_b, is

$$a_b = \beta_1\, kd \qquad \text{(N19.15)}$$

in which β_1 is an experimentally determined factor given by

$$\beta_1 = 0.85 - 0.05\, \frac{(f'_c - 4000)}{4000} \qquad \text{(N19.16)}$$

but not exceeding 0.85, where f'_c is in psi.

For a_b less than the flange thickness, t, the balanced steel area, A_{sb}, is given by

$$A_{sb} = \frac{0.85 f'_c\, b\, a_b}{f'_y} \qquad \text{(N19.17)}$$

For a_b greater than the flange thickness, A_{sb} is given by

$$A_{sb} = \frac{0.85 f'_c (b - b_w)\, t}{f'_y} + \frac{0.85 f'_c\, b_w\, a_b}{f'_y} \qquad \text{(N19.18)}$$

The value 0.75 A_{sb} is printed by the program, with a warning if A_s exceeds this value.

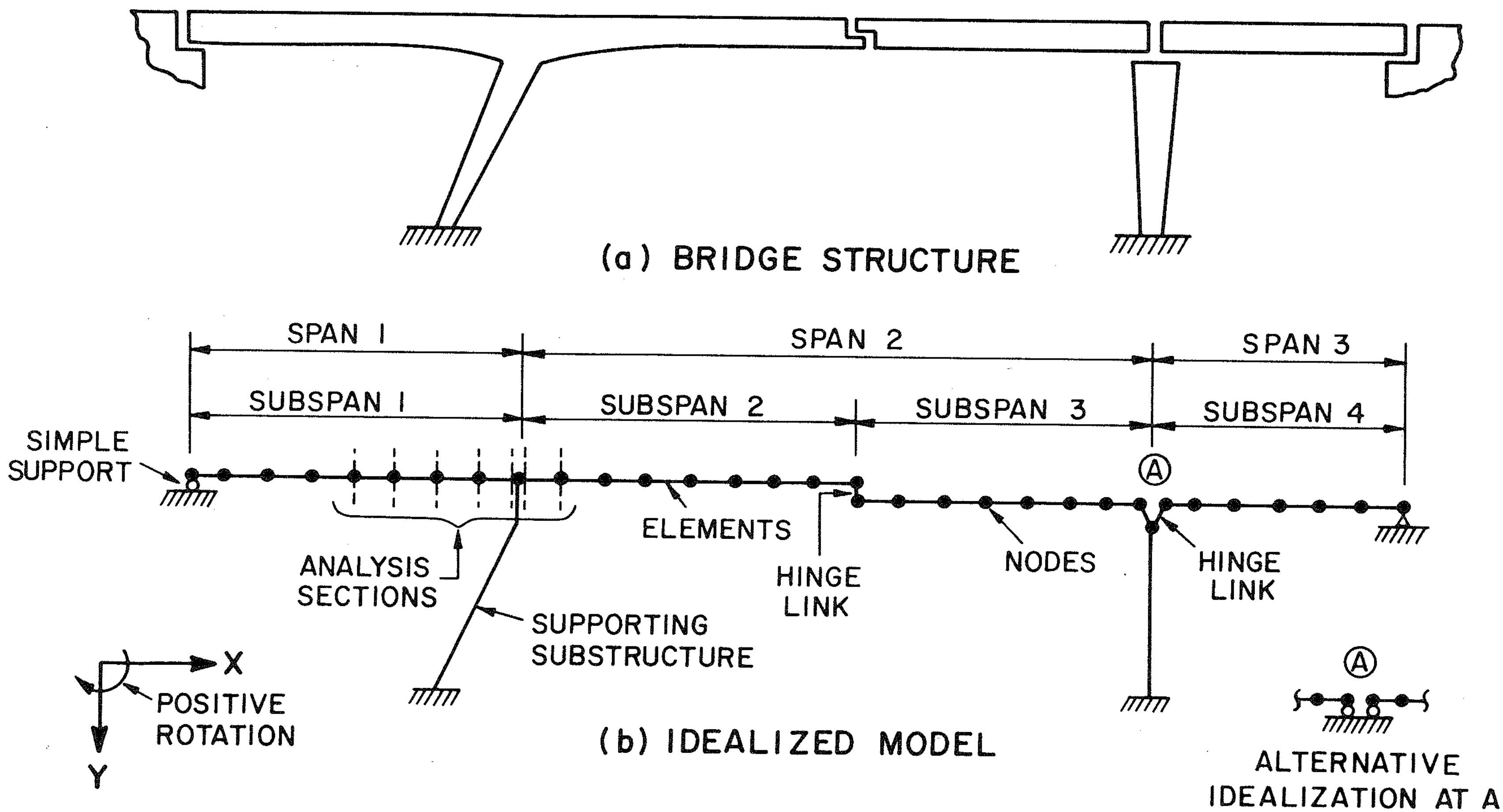
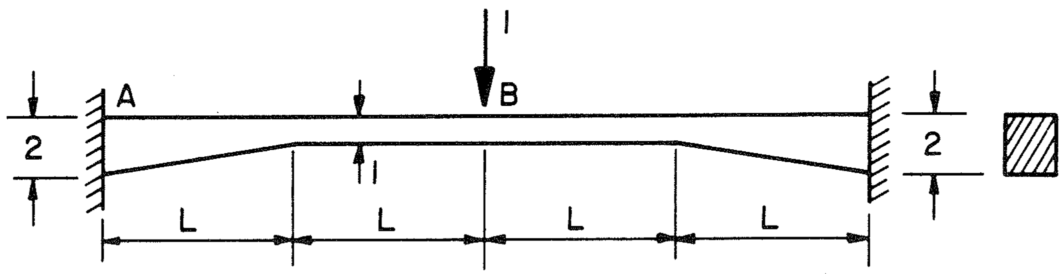
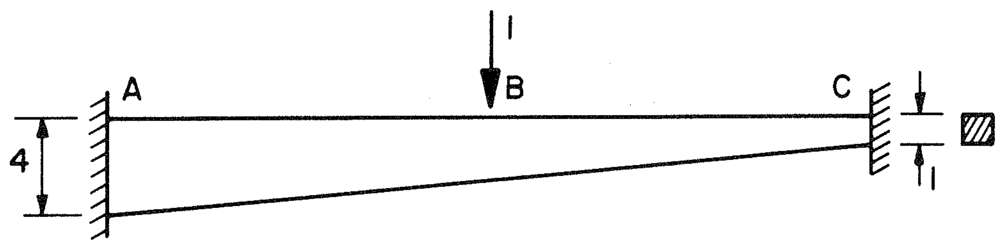


FIG. N.1 IDEALIZATION AND TERMINOLOGY



NO. OF ELEMENTS	M_A	M_B
4	-6.36	3.64
6	-6.18	3.82
8	-6.36	3.64
10	-6.30	3.70
EXACT	-6.36	3.64



NO. OF ELEMENTS	M_A	M_B	M_C
4	-9.92	4.09	-1.91
6	-10.24	4.07	-1.62
8	-10.39	4.05	-1.51
10	-10.46	4.04	-1.45
EXACT	-10.57	4.03	-1.37

FIG. N.2 INFLUENCE OF NUMBER OF ELEMENTS ON BENDING MOMENT ACCURACY

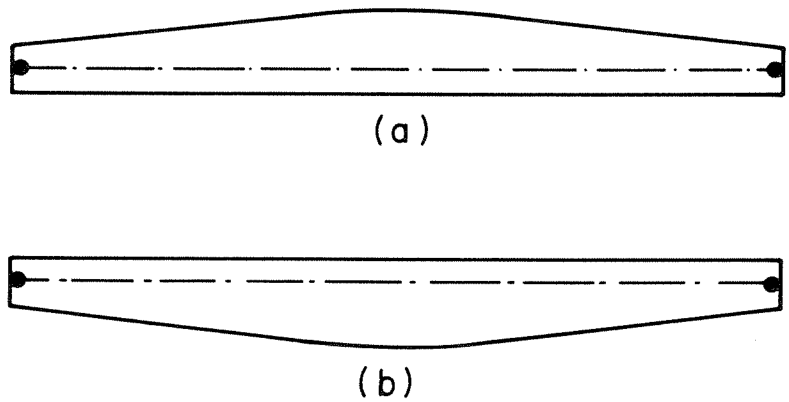


FIG. N.3 SIGNIFICANCE OF TOP FLANGE CURVATURE

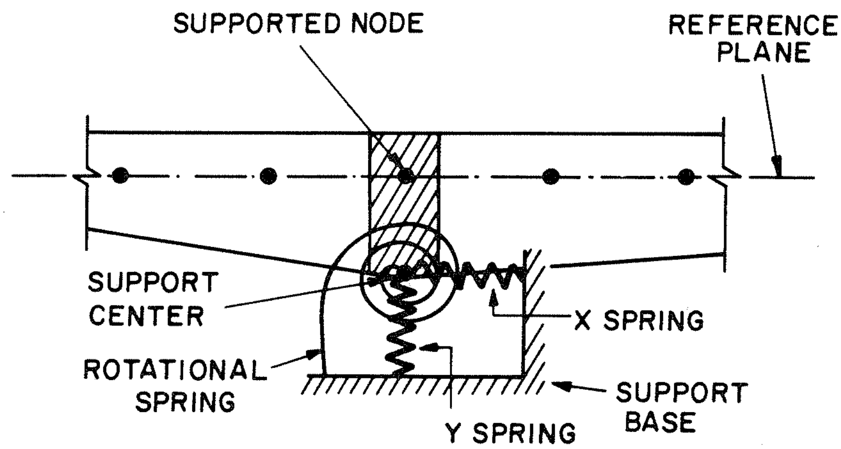


FIG. N.4 SIMPLE SUPPORT

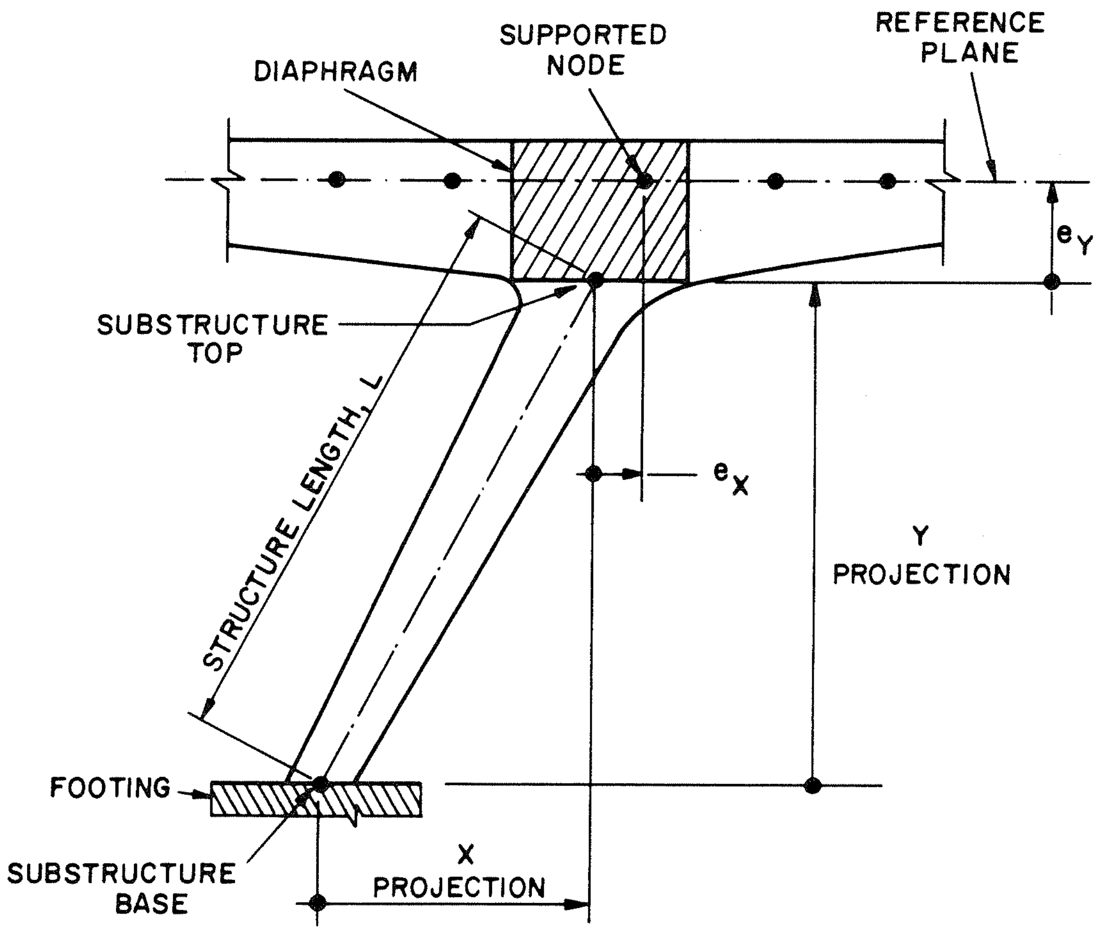


FIG. N.5 SUBSTRUCTURE DIMENSIONS

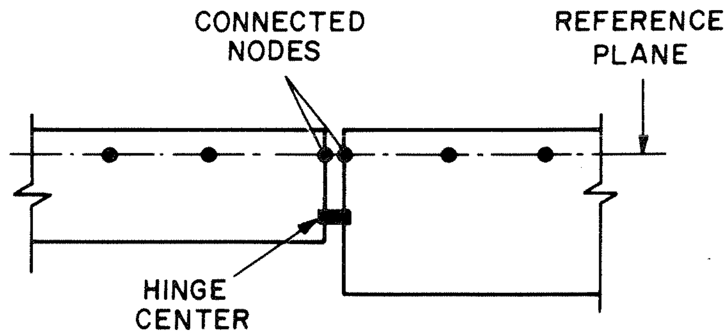
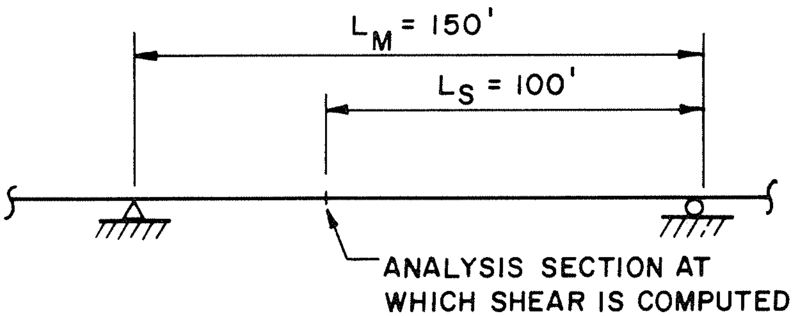
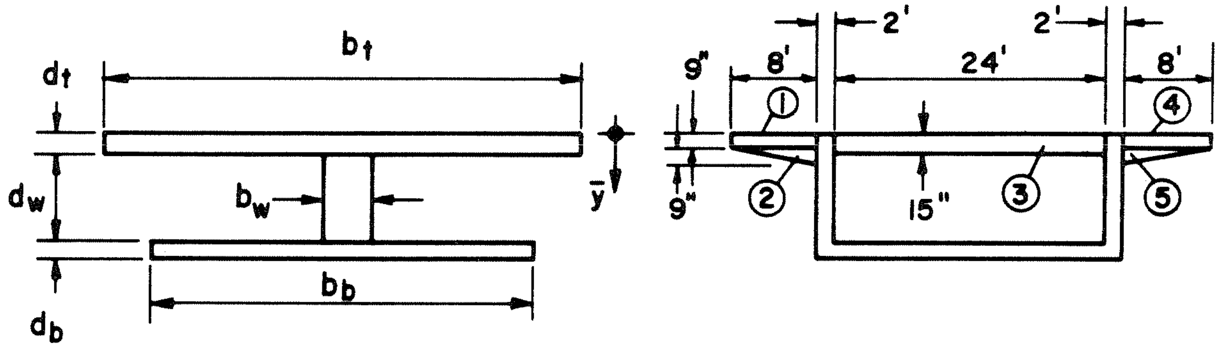


FIG. N.6 INTERNAL HINGE



$$\left.\begin{aligned} F_M &= 1 + \frac{50}{150+125} = 1.18 \\ F_S &= 1 + \frac{50}{100+125} = 1.22 \end{aligned}\right\} \quad f_s = \frac{F_S}{F_M} = 1.03$$

FIG. N.7 MODIFYING FACTOR FOR LIVE LOAD SHEAR CALCULATION



(a) ASSUMED SECTION (b) ACTUAL SECTION

REGION	AREA (ft^2)	$\bar{y}$ (ft)	$A\bar{y}$ (ft^3)
1	6.0	.375	2.25
2	3.0	1.0	3.00
3	30.0	.625	18.75
4	6.0	.375	2.25
5	3.0	1.0	3.0
	Σ = 48.0		Σ = 29.25

$\bar{y} = \frac{29.25}{48.0} = 0.609$ ft. Hence $t_t = 2\bar{y} = 1.218$ ft.

Hence $b_t = \frac{48.0}{1.218} + b_w = 39.41 + 4.0$

$= 43.41$ ft.

FIG. N.8 SECTION DIMENSIONS AND SAMPLE CALCULATION FOR EQUIVALENT TOP FLANGE

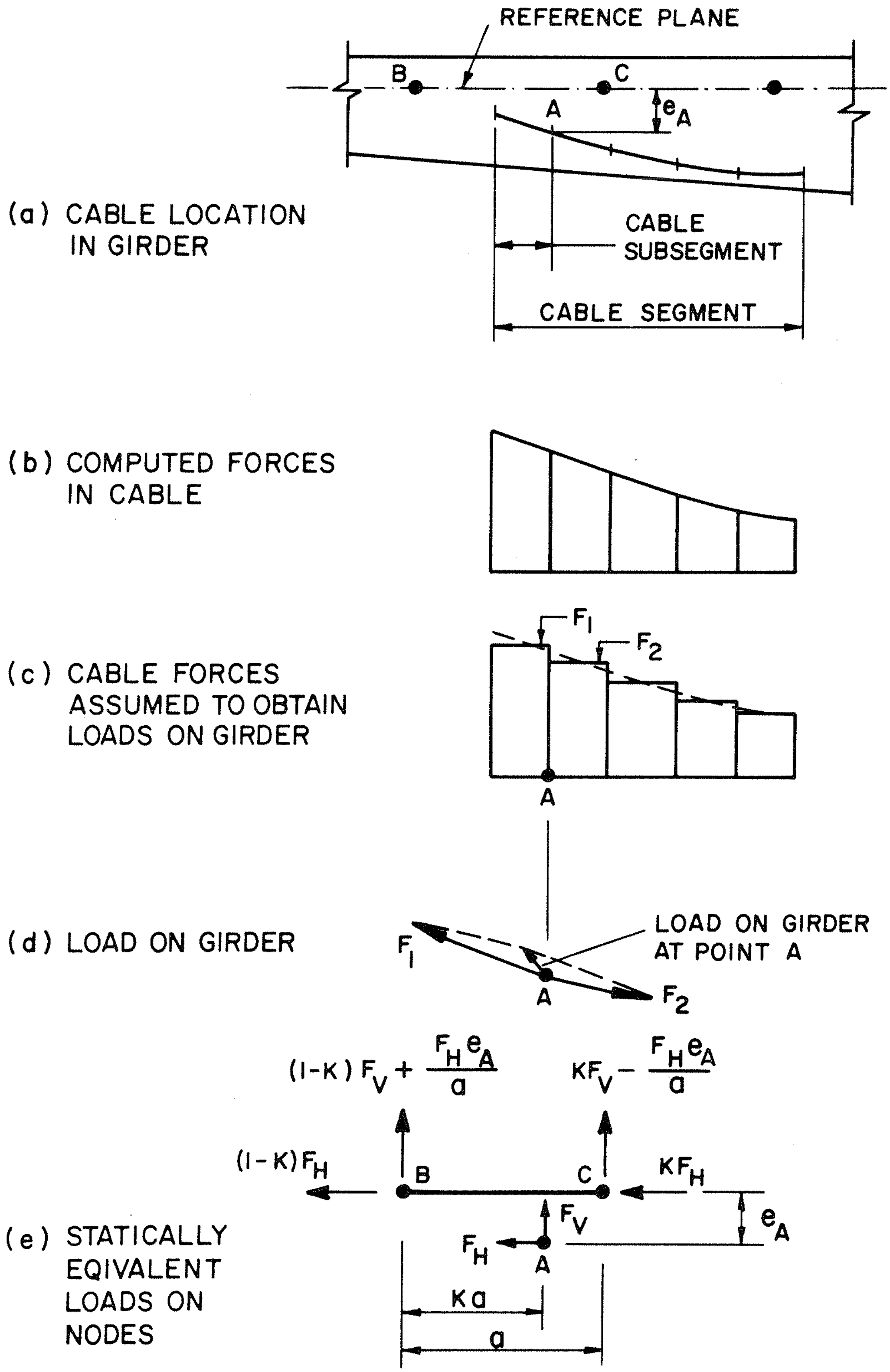


FIG. N.9 PRESTRESS LOADS ON GIRDER

3. EXAMPLES

3.1 TWO SPAN BRIDGE FROM DESIGN MANUAL

Sample calculations for the design of a two-span continuous prestressed box girder bridge are included in Chapter 7 of the California Manual of Bridge Design Practice [1]. The layout and a typical section of the bridge are shown in Fig. 3.1, and the input cards for the GIRDER-PC program are listed in Table 3.10. For the analysis, bottom flange flares extending over one tenth of each span from the bent centerline have been assumed. The jacking force of 7830 k is that determined in Reference 1. Only selected pages from the computer output are included in this report, because the complete output is bulky. Duplicate data decks and the complete output will be included when copies of the program deck are distributed.

Some comparisons of results from the Bridge Design Manual and from the GIRDER-PC program are shown in Tables 3.1 through 3.5. The agreement is seen to be close except for the deflections for prestress loading. The GIRDER-PC deflections are believed to be more accurate.

Typical pages of computer output from the GIRDER-PC analysis are shown in Table 3.6 through 3.9.

3.2 ILLUSTRATIVE EXAMPLE

The bridge shown in Fig. 3.2 has been selected for illustration purposes only, and is not intended to represent a real structure. The bridge includes an inclined pier, an expansion joint, variable depth, and discontinuous prestressing. It is subjected to gravity load, thermal loads, support settlement, and a variety of live load conditions. Design checks for working load flexure, shear capacity, and ultimate moment

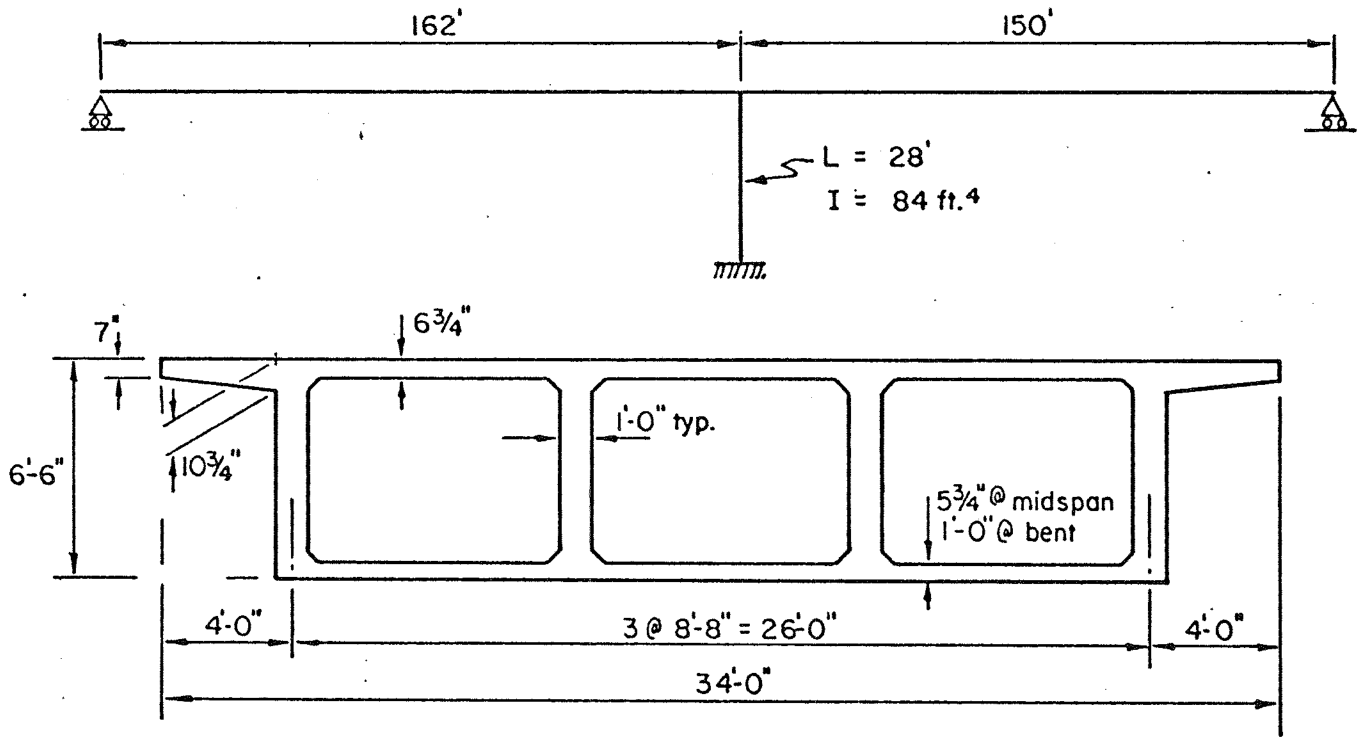


FIG. 3.1 BRIDGE FROM MANUAL OF BRIDGE DESIGN PRACTICE

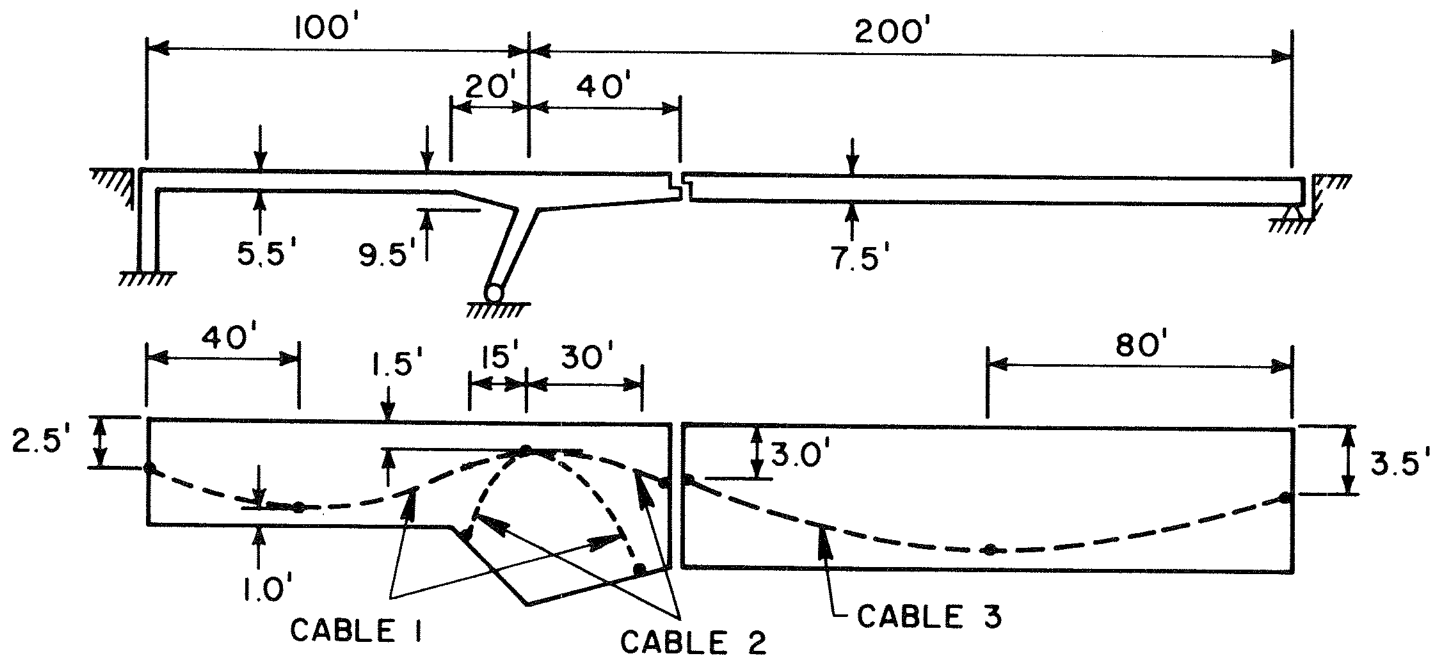


FIG. 3.2 ARTIFICAL EXAMPLE-ELEVATION AND CABLE PROFILE

capacity are carried out for the prestressed structure. The prestressing cables in span 1 and the cantilever part of span 2 are then ignored, and required reinforcement areas are determined assuming this part of the structure to be of reinforced concrete construction.

This example exercises most features of the program, and is included for illustration purposes. A copy of the input data deck and the computer output for this example will be included when copies of the computer program are distributed to prospective users.

It may also be of interest for users to confirm that equilibrium is satisfied for static loads on this structure. Because the displacement method of structural analysis is used in the program, an equilibrium check provides useful confirmation of the accuracy of the analysis. When equilibrium checks are being carried out, it is important to note that the girder cross section stress resultants printed by the program are for vertical sections through the girder, and are referred to the cross section centroid for each section. The input cards for this example are listed in Table 3.11.

TABLE 3.1

TWO SPAN BRIDGE EXAMPLE. COMPARISON OF MOMENTS
FROM BRIDGE DESIGN MANUAL AND GIRDER-PC

Moments are in k.ft.

Location	Loading	Manual	GIRDER-PC
0.4 point, span 1	DL	16206	16260
	DL + LL	21746	22166
Left of pier	DL	-27194	-27664
	DL + LL	-34562	-35090
Right of pier	DL	-25287	-26918
	DL + LL	-32193	-34173
0.6 point, span 2	DL	13105	12660
	DL + LL	18281	17950

TABLE 3.2

TWO SPAN BRIDGE EXAMPLE. COMPARISON OF PRESTRESS CABLE FORCES FROM BRIDGE DESIGN MANUAL AND GIRDER-PC

Cable forces are proportions of jacking force, including friction losses, anchorage draw-in and relaxation losses.

Location	Manual	GIRDER-PC
Left anchorage	0.763	0.771
0.8 point, span 1	0.818	0.817
Center pier	0.783	0.782
0.2 point, span 2	0.818	0.815
Right anchorage	0.757	0.767

TABLE 3.3

TWO SPAN BRIDGE EXAMPLE. COMPARISON OF WORKING LOAD STRESSES FROM BRIDGE DESIGN MANUAL AND GIRDER-PC

Stresses are in psi, compression positive for DL + LL + prestress after losses. All points are in span 1

Location in Span	Top Fiber		Bottom Fiber	
	Manual	GIRDER-PC	Manual	GIRDER-PC
0	745	753	745	752
0.1	1057	1072	369	364
0.2	1285	1287	99	109
0.3	1376	1397	-2	-16
0.4	1382	1407	5	-15
0.5	1292	1320	131	112
0.6	1145	1177	335	304
0.7	983	1016	558	522
0.8	687	625	939	1007
0.9	490	460	1159	1187
1.0*	106	-77*	1106	1234*

*Values from GIRDER-PC are theoretical peaks assuming a knife edge support, whereas values from Manual include reduction for pier thickness. The support centerline would normally not be chosen as a checking section for GIRDER-PC

TABLE 3.4

TWO SPAN BRIDGE EXAMPLE. COMPARISON OF ULTIMATE MOMENT CAPACITIES FROM BRIDGE DESIGN MANUAL AND GIRDER-PC

The design check is for a loading of 2.5 LL + 1.5 DL

Location	Item	Manual	GIRDER-PC
At pier*	Applied moment (k.ft.)	54070	54686
	Resisting moment with no mild steel (k.ft.)	46643	46205
	Mild steel required (sq.in.)	23.87	26.55
0.9 point, span 1	Applied moment (k.ft.)	31440	32614
	Resisting moment with no mild steel**(k.ft.)	29576	30156

*GIRDER-PC check at 3 feet from pier centerline. Design Manual check at pier centerline with reduced bending moment.

**Values are maximum permissible capacities in both Design Manual and GIRDER-PC, and redesign of the bottom flange flare is indicated in both cases. The GIRDER-PC output also indicates underdesign for the assumed flare in span 2, as shown in Table 3.9

TABLE 3.5

TWO SPAN BRIDGE EXAMPLE. COMPARISON OF DEFLECTIONS
FROM BRIDGE DESIGN MANUAL AND GIRDER-PC

Deflections are in feet, at midspan of span 1. The long term elastic modulus (3500 ksi x 0.333) is assumed.

Loading	Manual	GIRDER-PC
DL	.558	.568
Prestress after losses	-.477	-.397
DL + prestress	.081	.171

TABLE 3.6

CROSS SECTION PROPERTIES COMPUTED BY PROGRAM

CROSS SECTION PROPERTIES

SPAN NO.	SUBSPAN NO.	NODE NO.	SECTION NO.	TOTAL DEPTH (FT)	DEPTH TO N.A. (FT)	AREA (SQ.FT.)	INERTIA (FT**4)	SECTION MODULI (FT**3) TOP	BOTTOM
1	1	1	1	6.500	2.890	55.78	346.16	119.77	95.90
		2	2	6.500	2.890	55.78	346.16	119.77	95.90
		3	3	6.500	2.890	55.78	346.16	119.77	95.90
		4	4	6.500	2.890	55.78	346.16	119.77	95.90
		5	5	6.500	2.890	55.78	346.16	119.77	95.90
		6	6	6.500	2.890	55.78	346.16	119.77	95.90
		7	7	6.500	2.890	55.78	346.16	119.77	95.90
		8	8	6.500	2.890	55.78	346.16	119.77	95.90
		9	9	6.500	2.890	55.78	346.16	119.77	95.90
		10	10	6.500	2.890	55.78	346.16	119.77	95.90
		11	11	6.500	2.890	55.78	346.16	119.77	95.90
	2	11	12	6.500	2.890	55.78	346.16	119.77	95.90
		12	13	6.500	2.890	55.78	346.16	119.77	95.90
		13	14	6.500	2.890	55.78	346.16	119.77	95.90
		14	15	6.500	2.890	55.78	346.16	119.77	95.90
		15	16	6.500	2.890	55.78	346.16	119.77	95.90
		16	17	6.500	2.890	55.78	346.16	119.77	95.90
		17	18	6.500	2.890	55.78	346.16	119.77	95.90
		18	19	6.500	2.890	55.78	346.16	119.77	95.90
		19	20	6.500	2.890	55.78	346.16	119.77	95.90
		20	21	6.500	3.178	61.74	394.18	124.02	118.67
		21	22	6.500	3.396	67.76	426.97	125.74	137.53
2	3	21	23	6.500	3.396	67.76	426.97	125.74	137.53
		22	24	6.500	3.178	61.74	394.18	124.02	118.67
		23	25	6.500	2.890	55.78	346.16	119.77	95.90
		24	26	6.500	2.890	55.78	346.16	119.77	95.90
		25	27	6.500	2.890	55.78	346.16	119.77	95.90
		26	28	6.500	2.890	55.78	346.16	119.77	95.90
		27	29	6.500	2.890	55.78	346.16	119.77	95.90
		28	30	6.500	2.890	55.78	346.16	119.77	95.90
		29	31	6.500	2.890	55.78	346.16	119.77	95.90
		30	32	6.500	2.890	55.78	346.16	119.77	95.90
		31	33	6.500	2.890	55.78	346.16	119.77	95.90
	4	31	34	6.500	2.890	55.78	346.16	119.77	95.90
		32	35	6.500	2.890	55.78	346.16	119.77	95.90
		33	36	6.500	2.890	55.78	346.16	119.77	95.90
		34	37	6.500	2.890	55.78	346.16	119.77	95.90
		35	38	6.500	2.890	55.78	346.16	119.77	95.90
		36	39	6.500	2.890	55.78	346.16	119.77	95.90
		37	40	6.500	2.890	55.78	346.16	119.77	95.90
		38	41	6.500	2.890	55.78	346.16	119.77	95.90
		39	42	6.500	2.890	55.78	346.16	119.77	95.90
		40	43	6.500	2.890	55.78	346.16	119.77	95.90
		41	44	6.500	2.890	55.78	346.16	119.77	95.90

TABLE 3.7

PRINTOUT FOR CHECK OF WORKING LOAD FLEXURE

STRESSES AT CHECKING SECTIONS, LOAD COMBINATION 2 - DL + LL + PRESTRESS AFTER LOSSES

SECT. NO.	SPAN NO.	PROP. OF SPAN	DIST. ALONG SPAN (FT)		MOMENT (K.FT)	AX. FORCE (K)	STRESSES (KSI) TOP	BOTTOM
1	1	.0000	.00	MAX. MOMENT	2.08	-6044.98	.753	.752
				MIN. MOMENT	1.79	-6044.98	.753	.752
2	1	.0500	8.10	MAX. MOMENT	2947.46	-6057.09	.925	.541
				MIN. MOMENT	1472.01	-6057.09	.839	.648
3	1	.1000	16.20	MAX. MOMENT	5431.39	-6081.24	1.072	.364
				MIN. MOMENT	2654.56	-6081.24	.911	.565
4	1	.1500	24.30	MAX. MOMENT	7457.67	-6105.25	1.193	.220
				MIN. MOMENT	3552.95	-6105.25	.966	.503
5	1	.2000	32.40	MAX. MOMENT	9031.01	-6129.11	1.287	.109
				MIN. MOMENT	4170.91	-6129.11	1.005	.461
6	1	.2500	40.50	MAX. MOMENT	10156.47	-6152.82	1.355	.031
				MIN. MOMENT	4512.15	-6152.82	1.028	.439
7	1	.3000	48.60	MAX. MOMENT	10839.47	-6176.39	1.397	-.016***
				MIN. MOMENT	4580.32	-6176.39	1.035	.437
8	1	.3500	56.70	MAX. MOMENT	11085.78	-6199.79	1.415	-.031***
				MIN. MOMENT	4379.05	-6199.79	1.026	.455
9	1	.4000	64.80	MAX. MOMENT	10901.48	-6223.04	1.407	-.015***
				MIN. MOMENT	3911.91	-6223.04	1.002	.491
10	1	.4500	72.90	MAX. MOMENT	10293.04	-6246.13	1.374	.032
				MIN. MOMENT	3182.43	-6246.13	.962	.547
11	1	.5000	81.00	MAX. MOMENT	9261.34	-6257.63	1.316	.108
				MIN. MOMENT	2188.17	-6257.63	.906	.621
12	1	.5000	81.00	MAX. MOMENT	9260.89	-6288.38	1.320	.112
				MIN. MOMENT	2187.76	-6288.38	.910	.624
13	1	.5500	89.10	MAX. MOMENT	8021.54	-6304.63	1.250	.204
				MIN. MOMENT	1140.61	-6304.63	.851	.702
14	1	.6000	97.20	MAX. MOMENT	6695.24	-6336.74	1.177	.304
				MIN. MOMENT	157.25	-6336.74	.798	.778
15	1	.6500	105.30	MAX. MOMENT	5294.39	-6368.07	1.100	.409
				MIN. MOMENT	-754.43	-6368.07	.749	.847
16	1	.7000	113.40	MAX. MOMENT	3789.67	-6398.64	1.016	.522
				MIN. MOMENT	-1628.63	-6398.64	.702	.915
17	1	.7500	121.50	MAX. MOMENT	2423.80	-6408.23	.938	.622
				MIN. MOMENT	-2228.52	-6408.23	.669	.959

TABLE 3.7 (CONTD.)

18	1	.8000	129.60	MAX. MOMENT	1136.27	-6384.11	.861	.713
				MIN. MOMENT	-2933.99	-6384.11	.625	1.007
19	1	.8500	137.70	MAX. MOMENT	108.47	-6345.25	.796	.782
				MIN. MOMENT	-4067.49	-6345.25	.554	1.085
20	1	.9000	145.80	MAX. MOMENT	-732.77	-6293.77	.741	.837
				MIN. MOMENT	-5572.83	-6293.77	.460	1.187
21	1	.9500	153.90	MAX. MOMENT	-1261.44	-6219.02	.629	.773
				MIN. MOMENT	-7208.09	-6219.02	.296	1.121
22	1	1.0000	162.00	MAX. MOMENT	-5422.88	-6175.48	.333	.907
				MIN. MOMENT	-12849.03	-6175.48	-.077***	1.282
23	2	.0000	.00	MAX. MOMENT	-4688.85	-6159.58	.372	.868
				MIN. MOMENT	-11943.86	-6159.58	-.028***	1.234
24	2	.0500	7.50	MAX. MOMENT	-1372.97	-6203.88	.621	.778
				MIN. MOMENT	-7328.69	-6203.88	.287	1.127
25	2	.1000	15.00	MAX. MOMENT	-1566.15	-6280.48	.691	.895
				MIN. MOMENT	-6560.94	-6280.48	.402	1.257
26	2	.1500	22.50	MAX. MOMENT	-1378.71	-6334.04	.709	.888
				MIN. MOMENT	-5825.59	-6334.04	.451	1.210
27	2	.2000	30.00	MAX. MOMENT	-941.51	-6370.56	.739	.861
				MIN. MOMENT	-5357.11	-6370.56	.483	1.181
28	2	.2500	37.50	MAX. MOMENT	-192.81	-6383.38	.784	.809
				MIN. MOMENT	-5168.98	-6383.38	.495	1.169
29	2	.3000	45.00	MAX. MOMENT	685.44	-6364.39	.832	.743
				MIN. MOMENT	-4906.58	-6364.39	.508	1.148
30	2	.3500	52.50	MAX. MOMENT	1780.14	-6332.22	.892	.659
				MIN. MOMENT	-4307.99	-6332.22	.539	1.100
31	2	.4000	60.00	MAX. MOMENT	2865.69	-6300.50	.951	.577
				MIN. MOMENT	-3594.20	-6300.50	.576	1.045
32	2	.4500	67.50	MAX. MOMENT	3972.17	-6267.86	1.011	.493
				MIN. MOMENT	-2730.80	-6267.86	.622	.978
33	2	.5000	75.00	MAX. MOMENT	5087.62	-6251.30	1.073	.410
				MIN. MOMENT	-1725.78	-6251.30	.678	.903
34	2	.5000	75.00	MAX. MOMENT	5088.05	-6220.19	1.069	.406
				MIN. MOMENT	-1725.38	-6220.19	.674	.899
35	2	.5500	82.50	MAX. MOMENT	6091.79	-6208.72	1.126	.332
				MIN. MOMENT	-695.87	-6208.72	.733	.823
36	2	.6000	90.00	MAX. MOMENT	6769.73	-6185.67	1.163	.280

TABLE 3.7 (CONTD.)

				MIN. MOMENT	147.33	-6185.67	.779	.759
37	2	.6500	97.50	MAX. MOMENT	7122.26	-6162.42	1.180	.251
				MIN. MOMENT	807.54	-6162.42	.814	.709
38	2	.7000	105.00	MAX. MOMENT	7143.27	-6138.98	1.178	.247
				MIN. MOMENT	1281.22	-6138.98	.839	.672
39	2	.7500	112.50	MAX. MOMENT	6826.92	-6115.35	1.157	.267
				MIN. MOMENT	1564.78	-6115.35	.852	.648
40	2	.8000	120.00	MAX. MOMENT	6167.68	-6091.54	1.116	.312
				MIN. MOMENT	1654.56	-6091.54	.854	.639
41	2	.8500	127.50	MAX. MOMENT	5160.28	-6067.53	1.055	.382
				MIN. MOMENT	1546.86	-6067.53	.845	.643
42	2	.9000	135.00	MAX. MOMENT	3799.78	-6043.35	.973	.477
				MIN. MOMENT	1237.93	-6043.35	.824	.663
43	2	.9500	142.50	MAX. MOMENT	2081.51	-6018.99	.870	.599
				MIN. MOMENT	723.96	-6018.99	.791	.697
44	2	1.0000	150.00	MAX. MOMENT	1.90	-6006.76	.748	.748
				MIN. MOMENT	1.63	-6006.76	.748	.748

*** INDICATES OVERSTRESS

TABLE 3.8

PRINTOUT FOR SHEAR DESIGN CHECK

SHEAR DESIGN AT CHECKING SECTIONS, LOAD COMBINATION 1 - 1.5DL + 2.5LL + PRESTRESS AFTER LOSSES

SECT. NO.	SPAN NO.	PROP. OF SPAN	DIST. ALONG SPAN (FT)	MAX SHEAR (VU)(K)	CONC. SHR. (VC) (K)	REQD. AREA (SQ.IN/FT)	STIRRUP SPAC. (IN)
1	1	.0000	.00	980.28	485.22	1.44 **	20.42
2	1	.0500	8.10	883.41	485.22	1.44 **	20.42
3	1	.1000	16.20	770.94	485.22	1.44 **	20.42
4	1	.1500	24.30	660.80	485.22	1.44 **	20.42
5	1	.2000	32.40	553.03	485.22	1.44 **	20.42
6	1	.2500	40.50	447.69	485.22	1.44 **	20.42
7	1	.3000	48.60	344.83	485.22	1.44 **	20.42
8	1	.3500	56.70	244.49	485.22	1.44 **	20.42
9	1	.4000	64.80	-257.67	485.22	1.44 **	20.42
10	1	.4500	72.90	-357.72	485.22	1.44 **	20.42
11	1	.5000	81.00	-446.87	485.22	1.44 **	20.42
12	1	.5000	81.00	-446.90	485.22	1.44 **	20.42
13	1	.5500	89.10	-521.45	485.22	1.44 **	20.42
14	1	.6000	97.20	-596.57	485.22	1.44 **	20.42
15	1	.6500	105.30	-672.10	485.22	1.44 **	20.42
16	1	.7000	113.40	-681.32	485.22	1.44 **	20.42
17	1	.7500	121.50	-758.29	485.22	1.44 **	20.42
18	1	.8000	129.60	-837.84	485.22	1.44 **	20.42
19	1	.8500	137.70	-918.59	485.22	1.44 **	20.42
20	1	.9000	145.80	-1002.41	485.22	1.44 **	20.42
21	1	.9500	153.90	-1475.23	485.22	1.76	16.68
22	1	1.0000	162.00	-1920.14	485.22	2.56	11.51
23	2	.0000	.00	1828.53	485.22	2.39	12.29
24	2	.0500	7.50	1373.92	485.22	1.58	18.58
25	2	.1000	15.00	836.18	485.22	1.44 **	20.42

TABLE 3.8 (CONTD.)

26	2	.1500	22.50	816.74	485.22	1.44 **	20.42
27	2	.2000	30.00	750.79	485.22	1.44 **	20.42
28	2	.2500	37.50	687.15	485.22	1.44 **	20.42
29	2	.3000	45.00	626.01	485.22	1.44 **	20.42
30	2	.3500	52.50	636.59	485.22	1.44 **	20.42
31	2	.4000	60.00	575.82	485.22	1.44 **	20.42
32	2	.4500	67.50	515.31	485.22	1.44 **	20.42
33	2	.5000	75.00	455.23	485.22	1.44 **	20.42
34	2	.5000	75.00	455.20	485.22	1.44 **	20.42
35	2	.5500	82.50	379.34	485.22	1.44 **	20.42
36	2	.6000	90.00	291.46	485.22	1.44 **	20.42
37	2	.6500	97.50	205.32	485.22	1.44 **	20.42
38	2	.7000	105.00	-289.75	485.22	1.44 **	20.42
39	2	.7500	112.50	-380.49	485.22	1.44 **	20.42
40	2	.8000	120.00	-473.56	485.22	1.44 **	20.42
41	2	.8500	127.50	-568.90	485.22	1.44 **	20.42
42	2	.9000	135.00	-666.47	485.22	1.44 **	20.42
43	2	.9500	142.50	-766.20	485.22	1.44 **	20.42
44	2	1.0000	150.00	-848.90	485.22	1.44 **	20.42

** INDICATES MINIMUM AREA GOVERNS.

TABLE 3.9

PRINTOUT FOR CHECK OF NEGATIVE ULTIMATE MOMENT CAPACITY

MOMENT CAPACITIES AT CHECKING SECTIONS, LOAD COMBINATION 1 - 1.5DL + 2.5LL

SECT. NO.	SPAN NO.	PROP. OF SPAN	DIST. ALONG SPAN (FT)	EFF. CABLE AREA (SQ.IN)	EFF. DEPTH (FT)	MOM. CAPAC. REQD (K.FT)	CAPAC. WITH NO MILD STEEL	MILD STEEL REQD (SQ.IN)	MAX. PERMISS. MOMENT (K.FT)
1	1	.8000	129.60	38.67	2.99	11707.	20611.	0.	17801.
2	1	.8250	133.65	38.67	3.30	16493.	23598.	0.	20356.
3	1	.8500	137.70	38.67	3.66	21279.	26954.	0.	23294.
4	1	.8750	141.75	38.67	4.03	26947.	30601.	0.	26578.***
5	1	.9000	145.80	38.67	4.42	32614.	34451.	0.	30156.***
6	1	.9250	149.85	38.67	4.78	39082.	40405.	0.	38947.***
7	1	.9500	153.90	38.67	5.04	45549.	44339.	4.15	47407.
8	1	.9815	159.00	38.67	5.22	54686.	46205.	26.55	56753.
9	2	.0200	3.00	38.67	5.22	53381.	46206.	22.35	56371.
10	2	.0500	7.50	38.67	5.04	45682.	44339.	4.69	47407.
11	2	.0750	11.25	38.67	4.78	39957.	40405.	0.	38947.***
12	2	.1000	15.00	38.67	4.42	34232.	34451.	0.	30156.***
13	2	.1250	18.75	38.67	4.03	29218.	30600.	0.	26578.***
14	2	.1500	22.50	38.67	3.66	24204.	26954.	0.	23294.***
15	2	.1750	26.25	38.67	3.30	19976.	23598.	0.	20355.
16	2	.2000	30.00	38.67	2.99	15747.	20611.	0.	17801.

*** INDICATES INSUFFICIENT CAPACITY

TABLE 3.10

INPUT DATA FOR EXAMPLE FROM MANUAL OF BRIDGE DESIGN PRACTICE

```
START   TWO SPAN BRIDGE FROM 1973 MANUAL OF BRIDGE DESIGN PRACTICE.
    2     4   .41     3     0     2     1     0     2     0
    1     0    0.
   21     0  162.
   41    21  150.
    1     1   11      1   1.174    1.0        1.057
    2    11   21      1   1.174    1.057      1.0
    3    21   31      2   1.181    1.0        1.058
    4    31   41      2   1.181    1.058      1.0
 3500.      .000006     150.       3500.      .333
    1     1    0.           100000.  0.         6.5
    2    41    0.           100000.  0.         6.5
    1    21  100.          84.       4.    4.    2.    28.        0.        0.    2.89
    1     1    2      1     1     0     0     0     0
    1   1.0        .628       .628       33.347    33.347
    1   0.9        .500       .500       26.696    26.696
    2   0.1        .500      1.017       26.696    26.944
    1   1.0        4.0        4.0
    1   1.0        6.5
    2     0    0      0     0     1    -1     1     1
    1     0    1      1     0
    1     1    1      0     0     0  GIRDER SELF WEIGHT + 0.3K/FT FOR DIAPHS. ETC.
    1     4
    1    41    0.3
    1     1  27000.     .25         .0002       1   25.        SINGLE CABLE ONLY
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TABLE 3.10 (CONTD.)

```
   1    1    4     6     0   2.89     2.89      38.67
   1    2   81.0        2.490
   2    3   64.8       -3.304
   3    2   16.2       -0.826
   4    3   15.0        0.826
   5    2   60.0        3.304
   6    3   75.0       -2.49
   1  7830.    7830.      -0.625    7830.     7830.     -0.625
   1    1   AASHO HS20 LANE LOADING ONLY, 2.43 LANES
   1    9   2.43
   2    1    1     0    50
   1    2   1.925      0.0       DL + PRESTRESS AT TRANSFER
  1.0
   .0      0.0
        1.0
   .0
   1    1
   2    2
   2    2   1.925      0.0       DL + LL + PRESTRESS AFTER LOSSES
  1.0
   .0      0.0
   1    1.0
  1.0
   1    1
   2    2
   1    2   .180       60.        2.45     1.5DL + 2.5LL + PRESTRESS AFTER LOSSES
  1.5
   .0      0.0
   1    1.0
  2.5
```

TABLE 3.10 (CONTD.)

1	1							
2	2							
1	4	3.5	270.	60.	4.0	0.0	1.5DL + 2.5LL	
1.5								
.0		0.0						
1	1.0							
2.5								
1	1	.80	.825	.85	.875	.90	.925	.95
2	1	159.						
3	2	3.						
4	2	.05	.075	.10	.125	.15	.175	.20
STOP								

TABLE 3.11

INPUT DATA

```
START     ARTIFICIAL EXAMPLE. TESTING MOST PROGRAM FEATURES.
    2    5   30    5    1    1    2    1    2    0
    1    0   0.
    9    0   80.
   21    9   60.
   22   21   0.
   30   22   160.
    1   30   2.5       2.5        2.5
    1    1   10     1   1.22
    2   10   13     1   1.22
    3   13   19     2   1.15
    4   19   21     2   1.15
    5   22   30     2   1.15
  3500.     .000006    150.       3500.      .333
    1   30   1000.    100000.     0.         5.0
    1    1   100.      85.        4.    4.   2.    20.       0.        0.   3.
    2   13   100.      85.        3.    0.   0.    20.       7.5       0.   7.
    1   21   0.       100000.     0.         0.5
    1    1    1    1    2    0    0    0    0
    1   1.0       0.63       0.63      34.0       34.0
    1   1.0       0.5        0.5       26.0       26.0
    1   1.0       4.0        4.0
    1   80.       5.5
    2   20.       5.5        9.5
    2    0    0    0    2    1    1    1    0
    1   40.       9.5        7.5
    2   160.      7.5
    2    1    2    7    4
    1    1    1    2    0    0      SELF WEIGHT
    1    5
    1   30   0.3
   13   0.        30.        0.
   21   0.        10.        0.         22    1
    2    0    0    0         1   3 INCH SETTLEMENT AT CENTER PIER
    2   0.        0.25       0.
    1    2   27000.    0.25       .0002       1    25.     CABLES UP TO EXP. JT.
    1    1    3    4    3    0.        5.5       25.     L ABUT TO PIER+30FT
    1    2   40.       2.0
    2    3   50.       -2.5
    3    2   10.       -0.5
    4    3   30.       6.5
    1    1    1   6000.
    2    1    2   -600.
    3    1    3   -0.625
    2    2    4    2    3   4.0       0.5       25.     PIER-15FT TO EXP. JT.
    1    2   15.       -5.0
    2    3   40.       1.5
```

TABLE 3.11 (CONTD.)

```
  1    2    1   6000.
  2    2    2   -600.
  3    2    3   -0.625
  2    1   27000.   0.25      .0002      2    15.     CABLE IN SUSP. SPAN
  1    5    5    2    0    0.5      1.0       40.     EXP. JT. TO R ABUT.
  1    2   80.       3.5
  2    3   80.      -3.0
  1   10000.   9000.     -0.625    10000.     9000.      -0.625
  1    1      HS20 TRUCK LOADING, 2.5 LANES
  1    6   2.5          2     0     3    14.       30.
  2    2     HS20 TRUCK AS ARBITRARY TRAIN, 2.5 LANES
  1    1   5.0          3     2     3
 4.       16.       16.
14.       14.
  2    1   5.0          3     2     1
 4.       15.       16.
14.       30.
  3    1     HS20 LANE LOADING, 2.5 LANES
  1    9   2.5
  4    2    HS20 TRUCK AND LANE LOADING, 2.5 LANES
  1    6   2.5          2     0     3    14.       30.
  2    9   2.5
  5    1    P13 TRUCK LOADING, 2.0 LANES
  1   15   2.0          0     0     1
  6    1      P13 TRUCK AS 7 WHEEL TRAIN TO CHECK
  1    1   2.0          7     2     1
26.     48.       48.       48.       48.       48.       48.
18.     18.       18.       18.       18.       18.
  7    1      P13 TRUCK AS 13 WHEEL TRAIN TO STUDY DIFFERENCE
  1    1   2.0         13     2     1
26.     24.       24.       24.       24.       24.       24.       24.
24.     24.       24.       24.       24.
16.     4.        14.       4.        14.       4.        14.       4.
14.     4.        14.       4.
  3    1    1    1    1    1
  1    2   1.925    0.        DL + PRESTRESS AT TRANSFER
1.0       0.
0.        0.
     1.0        0    1.0
0.        0.        0.        0.        0.
0.        0.
  1    1
  2    2
  2    2   1.925    0.        DL + HS20 + TEMP + P AFTER LOSSES + SETTLEMENT
1.0       1.0
100.      0.
  1   1.0        1    1.0
0.        0.        0.        1.        0.
0.        0.
  1    1
  2    2
  3    2   1.925    0.         DL + P13 + P AFTER LOSSES
1.0       0.
0.        0.
  1   1.0        1    1.0
0.        0.        0.        0.        1.
0.        0.
```

TABLE 3.11 (CONTD.)

```
     1     1
     2     2
     1     2    0.18      60.       2.45      1.5DL +2.5HS20 + P AFTER LOSSES
   1.5        0.
   O.         0.
     1    1.0          1    1.0
   O.         0.       O.        2.5       O.
   O.         0.
     1     1
     2     2
     1     2    5.0      270.       60.       4.0         O.         1.5DL + 2.5HS20
    1.5         O.
     .          O.
     1    1.0          1    1.0
   O.         0.       O.        2.5       O.
   O.         0.
     1     1    0.7      0.75       0.8       0.85       0.9        0.95       97.
     2     2    3.       5.         10.       15.        20.        30.
     1     2    5.0      270.       60.       4.0        O.         1.5DL + 2.5HS20
    1.5         O.
     .          O.
     1    1.0          1    1.0
   O.         0.       O.        2.5       O.
   O.         O.
     1     1
     2     2
     1     2    1.6       24.       8.        4.         4.         DL + HS20 + P
   1.0          O.
    O.          O.
          .0           1    1.0
   O.         O.       O.        1.        O.
   O.         O.
     1     1
     2     2    3.        10.       20.       30.        40.
     1     2    4.0       60.       0.9       4.0        4.0        1.5DL + 2.5HS20 + P
    1.5         O.
    O.          O.
          .0           1    1.0
   O.         O.       O.        2.5       O.
   O.         O.
     1     1
     2     2    3.        10.       20.       30.        40.        50.
 STOP
```

4. CONCLUSION - PROGRAM IMPLEMENTATION

The GIRDER-PC program could be a useful tool in the design of prestressed concrete bridges. The program is limited to structures which can be idealized as an isolated girder or frame, but is otherwise very general. The program executes rapidly, and produces a detailed design check for the bridge girder. It is applicable to conventional load factor (ultimate load) design as well as working load design.

The program designs shear reinforcement areas and the areas of non-prestressed reinforcement required to augment ultimate moment capacity, but otherwise does not perform any design synthesis. Nevertheless, experience has shown that a designer can use the program to refine a rough initial design to a final design in only a few iterations at low cost.

Versions of the program for CDC 6400/6600/7600 series and IBM 360/370 series computers are available for immediate distribution. The program uses standard FORTRAN IV language, and should be easy to implement on any machine with adequate core storage. A core capacity of approximately 40k words is required.

The program has been tested on a variety of example structures, and at the time of writing has been used as an aid in the design of two major bridges. It is believed to be error-free, although no warranty to this effect is made.

This report is believed to constitute adequate documentation. The program is coded straightforwardly, with ample comment statements, so that modifications should not be unduly difficult to carry out.

REFERENCES

1. "Manual of Bridge Design Practice," State of California Business and Transportation Agency, Department of Public Works, Division of Highways, Third Edition, 1971.