PRESTRESSED CONCRETE TRUSSED GIRDER

Thesis for the Degree of M. S.

MICHIGAN STATE COLLEGE

Manubhai N. Patel

1954

This is to certify that the

thesis entitled

Prestressed Concrete Trussed Girder

presented by

Manubhai N. Patel

has been accepted towards fulfillment
of the requirements for

M.S. degree in C.E.

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Date 5-14-54

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PRESTRESSED CONCRETE TRUSSED GIRDER

By

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A THESIS

Submitted to the School of Graduate Studies of Michigan
State College of Agriculture and Applied Science
in partial fulfillment of the requirements
for the degree of

MASTER OF SCIENCE

Department of Civil Engineering

Year 1954

THESIS

ACKNOWLEDGEMENT

The writer wishes to express his sincere appreciation to Dr. Richard H. J. Pian for his inspiration and his valuable guidance without which it would have been impossible to carry out the development of this thesis. He is also greatly indebted to Dr. Carl L. Shermer and Dr. G. C. Blomquist for their valuable suggestions. His heartfelt thanks are also expressed to Dr. and Mrs. E. A. Brand for reviewing the pages of this dissertation. He is also grateful to Professor T. Y. Lin of the Civil Engineering Department of the University of California, for his informative letter written from the University of Ghent, Ghent, Belgium.

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I. INTRODUCTION

The application of reinforced concrete in engineering structures and buildings began towards the end of the nineteenth century. Yet up to the present, steel construction rather than reinforced concrete has succeeded in holding the field of long span engineering structures. Some of the reasons for this may be accounted as follows:

(a). For long spans, the dead load of the structure becomes so great that the structure becomes uneconomical.

(b). Materials cannot be used efficiently in reinforced concrete members subjected to bending.

(c). The bond between steel and concrete under working stresses in steel requires adequate provision for sufficient surface of bars covered by a sufficient amount of concrete. Observations in practice indicate that the bond between concrete and steel is generally insufficient* when bars are more than one inch in diameter or when high working stresses are used.

The drawbacks to using the reinforced concrete in long span structures can be overcome by prestressed concrete. The principles and their applications are not new.

*August E. Komendant, "Prestressed Concrete Structures." page 2.

As far as is known, first attempts probably took place in Germany in about 1888.* These early efforts failed mainly because cements used at that time did not produce concrete of the quality necessary to withstand the high stresses developed.

In the present days, the concrete can be made in practice to such a high strength so as to give the crushing strength 8000 p.s.i. or more, by choosing the proper kinds of materials and by proper curing. Also the high strength steel available has made the prestressed concrete of extensive use in actual practice for many engineering structures.

The main purpose of this thesis is to study the possibility of spanning a space by providing the trussed concrete girder. Simply supported structures have been found economical for spans ranging from 100'-0" to 200'-0".** The weight of the structure is one of the important factors which leads to economy. By using a trussed girder, the dead weight of the structure is reduced. A span of 120 feet is chosen for study and the design is carried out so that permanent stresses are created prior to the application of loads on the structure. These stresses developed in the structure, when loads are applied, are within the limits.

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*John G. Hendrichson, "Prestressed concrete Pipe," Proceedings of the First United States Conference on Prestressed Concrete. August 14 to 16, 1951.

**August E. Komendant, "Prestressed Concrete Structures," page 24.

II. NOTATIONS

L = Length of member

A = Area of entire concrete section (Steel area not deducted)

d = Depth of section

b = Width of rectangular section

C.G.C. = Center of gravity of entire cross-section

C.G.S. = Center of gravity of steel area

Y_b, Y_t = Distance of bottom and top fibre to C.G.C. respectively

F_b, F_t = Stresses in bottom and top fibre respectively

I = Moment of inertia of the concrete section about C.G.C.

r = Radius of gyration of a about C.G.C.

e = Total rise of curved cable from the center of span to the interior support

c = Total rise of curved cable from the center of span to the exterior support

e_{u1s}, e_{u2s}, --- etc. = Eccentricity of the cable with respect to C.G.C. at supports

e_{u0c}, e_{u1c}, e_{u2c}, --- etc. = Eccentricity of the cable with respect to C.G.C. at the center of span

a_1 = Length from center of span to point of contraflexure or curved cable.

a_2 = Length from support to point of contraflexure of curved cable

P_i = Initial Prestress

P = Final Prestress

W_1 = Equivalent load produced by the continuous curved cable with curvature concave upwards.

W_2 = Equivalent load produced by the continuous curved cable with curvature concave downwards.

W_3 = Equivalent load produced by the cable with curvature concave downwards but the cable is discontinuous at that point.

U_0, U_1, U_2 --- = Top chord joints of the truss

L_0, L_1, L_2 --- = Lower chord joints of the truss

M_p = Moment due to eccentricity of prestressing force

M_d = Moment due to dead load of structure

M_L = Moment due to live load on structure

M^F = Restrained moment

III. GENERAL CONSIDERATIONS

The distinguishing principle in the design of prestressed concrete truss girders is the separation of main tension members and compression members. A conventional reinforced concrete beam is converted into a trussed girder with pretensioned tension members. Depending upon the magnitude of prestressing of tension members, the concrete members can be subjected to compressive stresses to such an extent that tensile stresses normally developed by loadings can be eliminated entirely if this is desired.

By using the floor slab as the top chord and designing the web members accordingly, the elaborate floor system, which is generally used in steel structures, can be eliminated. The stresses in the upper chord can be used in areas of positive moments as the prestressing loads for the floor slab.

In some cases the width of the web members equal to that of the truss may not be required to maintain the effective width. In these situations openings may be left in the members there by reducing the weight of the structure. This makes available the space for some reparations which are to be made in future or for carriage of some utilities through the structure.

Secondary stresses which are quite high in reinforced concrete trusses can be reduced to a smaller extent by prestressing the members. The prestressed truss members are relatively small. Moreover all the members being concrete slabs of a width equal to or less than the truss width, uniform distribution of the prestressed can be achieved.

Creep and shrinkage in such trusses will be of intense value. They occur gradually and may induce tremendous stresses in the members depending upon the ratio of stiffness of the members. It is necessary in such cases to get more or less uniform shrinkage all over the truss, so as to reduce stresses due to shrinkage. Because the different members will be under different prestress, there will be different amounts of creep. This may induce stresses in the structure. These stresses due to creep and shrinkage can be minimized by adjusting the prestress in all members as is necessary from time to time. This is accomplished by post tensioning the members. The post tensioning of the members to induce stresses in the structure can be achieved by means of hydraulic jacks supported against hardened concrete members, thus the tensioning is achieved simultaneously with the removal of the scaffolding.

IV. DESIGN PROCEDURE

A. Primary Stress Analysis

(a). Selection of truss type. The web members of a truss are generally designed to take expression or tension. The most suitable arrangement for a prestressed concrete truss girder is to design the diagonals to take tension and the verticles to take compression. By designing the members in this way, the bending moments due to the weight of the members are reduced to a great extent and also secondary stresses are less than in those trusses having diagonal members in compression and verticle members in tension.

In a simple span of prestressed concrete trusses the economical ratio of depth to span* varies from 1:10 to 1:15 and in continuous trusses from 1:15 to 1:20.

To reduce the stresses due to the deflection of a centrally loaded member, the length of the panel should not be more than twice the depth of the truss. To reduce secondary stresses, the depths of truss members should be smaller than their widths.

(b). Preliminary stress analysis. In a preliminary stress analysis the members are considered to be pinconnected at panèl points by frictionless pins, and the loads are

*August E. Komendant, "Prestressed Concrete Structures" page 124.

supposed to act at the panel points. The dead weight of the truss is assumed to be a certain amount for the analysis but should be revised according to the sections designed from the normal forces and other informations available. The normal stresses for dead loads can be obtained by the method of sections. In the analysis of a trussed girder which is acted upon by moving loads, influence lines can be used to determine maximum forces in every member.

(c). Truss as a rigid frame. In the prestressed trussed girders, where the floor slab is used as the top chord of the structure, the restrained moments due to their own weight as well as live loads occur at the joints. In such cases, the truss should be treated as a rigid frame, and the restrained moments should be determined at all joints. The stiffnesses for all members can be taken from the assumed sections of the members and center to center length of the members and should be revised each time the sections are revised. Three general conditions of loadings should be considered for such trusses.

(1). The truss is loaded with its own weight.

(2). The truss is loaded with live loads to produce maximum positive moment in the floor slab.

(3). The truss is loaded with live loads to produce maximum negative moments in the floor slab.

The primary stresses in all the members of the prestressed concrete trusses can be obtained from the normal forces and the computed moments and these should be used for choosing the final sections of all members. The stresses developed at all sections of a member by the normal forces and those by prestressing must be within limits.

(d). Design of members.

(1). Top chord. In many cases, the floor slab can be used as the top chord of a prestressed trussed girder. It may be designed as a prestressed continuous slab supported on the verticles. The approximate sections and the primary stresses being known, the profile of the cable, and its eccentricities at different sections for required prestressing force, can be computed. The sections are then checked to keep stresses within allowable limits. As a general principle of prestressed concrete, the stress condition assumed at the compression fibre is zero stress under the action of dead loads of the structure and the prestressing force on the structure. But it may, in some cases, limit the tensile fibre stress condition under the dead load and the prestressing force. The prestressing force is limited by the allowable compressive stresses in the concrete.

If the panels are very long, not more than three continuous spans should be prestressed together. Professor Magnel* has carried out some tests for friction losses in

*Gustave Magnel, "Prestressed concrete," Chapter IV, page 80.

prestressing and has found that it does not exceed five percent provided that there are a maximum of three spans and that prestressing is applied simultaneously. But if the panels are not very large, the top chord can be prestressed by a curved continuous cable without exceeding the friction limits.

Another arrangement to reduce friction losses in prestressing of cables, developed with curved prestressing cables, consists of the use of overlapping intermediate anchorages by which the length of the prestressing unit is reduced. In this case the prestressing units are to be brought out from the slab and long stressing cavities large enough to provide space for building in the jacks are to be made in the structure. These cavities are then filled with unstressed concrete which is subjected to tensile stresses. The cavity shrinks away from the old concrete, thus making it nonmonolithic with the structure, and damaging stress concentration may occur. It is therefore advantageous to use a continuous curved cable to prestress a continuous span supported on columns.

The profile of the cable can be taken as a second degree parabola. It has been found that the moments vary only slightly when the curvature of the cable* is changed

*A.L.Parme and G.H.Paris, "Analysis of Continuous Prestressed Concrete Structures," Proceedings of the First United States Conference on Prestressed Concrete, August 14 to 16, 1951.

from parabolic to sinusoidal, thus a slight displacement of the cable from its exact profile will not materially alter the end moments.

The elastic behaviour of a curved cable which follows some sinusoidal path can be determined by the application of equivalent uniform loads of proper magnitude and distribution on the member. By distribution and balancing of restrained moments consisting of the primary moments due to loads and secondary moments due to the continuity of the cables over the supports, the actual moments existing at the supports are known and by the use of these moments at supports, the moments existing at any section of the structure can be computed. These moments can be used to check the stresses within the allowable limits.

(2). Web members. The web members are either in tension or compression, but to reduce the bending moments due to their own weight, it is a suitable arrangement to make the diagonals take tension and the verticles take compression.

(i) Diagonals. The bending moments on these members being small in comparison with the normal forces, the diagonals can be prestressed by pretensioning the cables axially. Prior to the loads on the structure, compressive stresses are induced in concrete so that the stresses remain within limits when the loads act on the structure.

(ii) <u>Verticles</u>. These being the compression members, they are designed as common reinforced concrete columns loaded axially.

(3). <u>Bottom chord</u>. The bottom chord of the prestressed concrete truss should be designed to take tension. It can be designed as a slab, the width of which is equal to the truss width and the length equal to the length of the truss. The slab is prestressed axially by pretensioning the cables so that the stresses are within limits before and after the loads are applied to the structure.

B. Secondary Stress Analysis

(a) <u>Secondary stresses due to the prestressing</u>. When the concrete members are prestressed, the lengths of the members are changed. The joints being rigid, the resistance of joints to free rotation produces secondary moments on the members. For computing the change in lengths of the members and the displacements of the joints, non-rigid connection of the members at the joints should be assumed. The displacements of the joints can be found by drawing Williot Diagram.

(b). <u>Secondary stresses due to normal forces</u>. Due to normal forces on all the members, the members will change in length and displacement of joints occur. The moments due to this can be found in the same way as in B (a).

Conventional reinforcement can be provided to take care of any tensile stresses which might occur in the concrete members.

V. DESIGN EXAMPLE

The vibrated concrete to be used in this example is to have a minimum compressive strength (f'_c) of 7000 p.s.i. and modulus of elasticity of $6 \cdot 3 \times 10^6$ p.s.i. Before allowance for creep and shrinkage is made, the allowable compression in extreme fibre is to be 2800 p.s.i. and the allowable tension in extreme fibre, 350 p.s.i. After allowance for creep and shrinkage is made, the allowable compression in extreme fibre is to be 2000 p.s.i. and the allowable tension in extreme fibre, 210 p.s.i. The allowable principle tensile stress, after allowance for creep and shrinkage is made, is to be 280 p.s.i.

The cables used for post-tensioning are Roebling galvanized prestressed-concrete strands. As these strands are fabricated from hot-dip galvanized wire, complete protection against corrosion without further treatment is assured. The allowable design load per strand is to be according to the manufacturer's allowable design load.

The loss in initial prestress allowed for creep and shrinkage is to be 15%.

A. Primary Stress Analysis

(a) Selection of truss type. A span of 120'-0" is covered by a prestressed concrete trussed girder as shown in figure 1. The girder is an 'N' type truss having six panels of 20'-0" each. Under the loads the diagonals and the bottom chord members are in tension while the top chord and the verticles are in compression. The girder is 20'-0" wide accomodating two traffic lanes, with an additional 3'-0" side walk on each side. It is designed to carry A.A.S.H.O. H·44 loading. The height of the girder is taken as 10'-0".

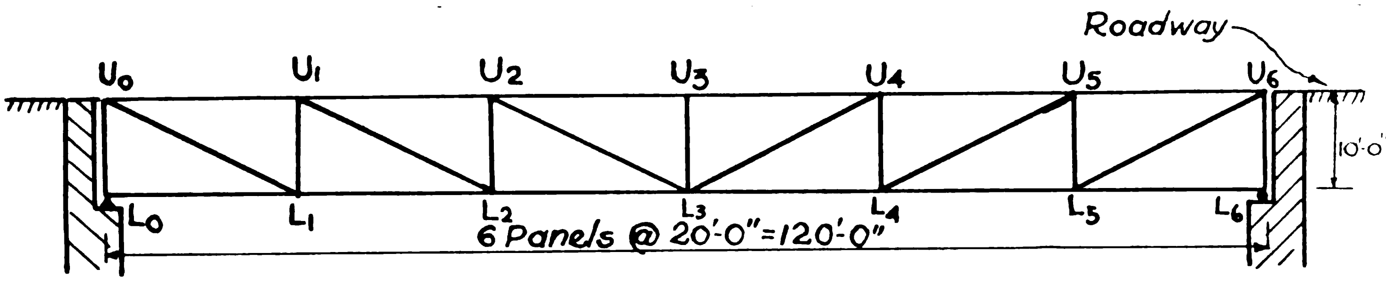


Figure-1

The top chord of the truss is a continuous concrete slab 20'0" wide and 120'-0" long and is used as the floor slab. The floor system, which is common to most of the concrete and steel structures of this type, is therefore eliminated. The bottom chord is also a continuous concrete slab 20'-0" wide and 120'-0" long. The diagonals and verticles are also slabs, but openings are left primarily to reduce the weight of the structure and to facilitate any type of repairs which may be necessary in the future.

(b). Preliminary stress analysis. For a preliminary analysis joints U_0, U_1, U_2 --- U_6, and L_0, L_1, L_2 --- L_6 are assumed to be pin connected.

The dead load of the structure, including surface finish, is assumed as 280 p.s.f.* The live load on the structure is taken as 640 lbs. per linear foot of load lane and a concentrated load of 18,000 lbs.** The sidewalk live load is assumed as 80 p.s.f. for a width of six feet and is treated as distributed load per foot run of the truss.

The impact factor for the live load is determined by the formula.***

$$I = \frac{50}{L + 125} \qquad \text{in which,}$$

I = impact factor (maximum 30%)

L = length in feet of the portion of the span which is loaded to produce the maximum stress in the member.

The dead load is assumed to act at panel points. With this dead load of 280 p.s.f., 75,000 pounds of concentrated load is assumed to act at the panel point. The stresses in all of the members are computed.

These stresses are used to assume the sections of the members. As the sections are changed twice, the dead load stresses are also changed in every case. After sufficient

*August E. Komendant, "Prestressed Concrete Structures," Figure 13, page 22.

** The American Association of State Highway Officials "Standard Specifications for Highway Bridges, 1953," Section 3.2.8., page 159.

*** The American Association of State Highway Officials "Standard Specifications for Highway Bridges, 1953," Section 323.2.12., page 184.

trials, the sections shown in Table I are found out and the dead load stresses are changed accordingly. The dead load of the structure, found from these sections, is 432,000 lb., say 225 p.s.f. The panel load is 90,000 lb. distributed equally between the upper and lower joints of the panel. Thus a concentrated load of 45,000 lb. acts at each joint. The normal forces on all members are as shown in Table I.

To compute the maximum normal stresses on the members of the truss, influence lines for shear in the panel and moment at the joints are drawn. The truss is loaded with uniform live load of 1280 lb. per linear foot and a concentrated load of 58,000 lb., in such a way as to produce maximum normal stress in the members. The stresses, due to live loads increased by the required value of impact factor, are shown in Table I. The maximum normal stresses due to the sidewalk live loads are also computed in the same way, but no impact is taken into consideration. They are shown in Table I.

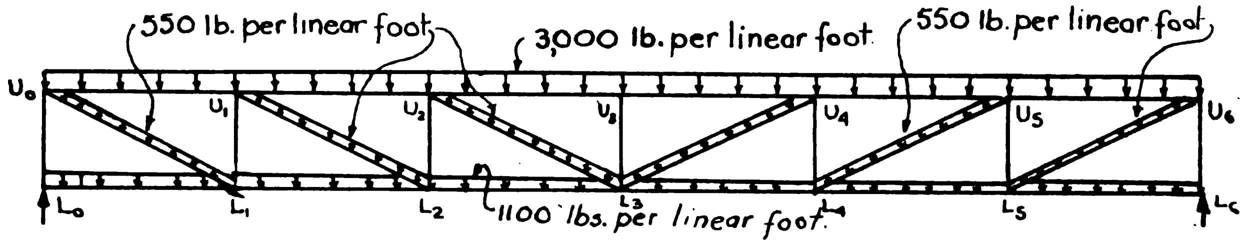
(c). Truss as a rigid frame. The loads on the floor slab produce restrained moments at the rigid joints. These moments are computed for three conditions of loading on the structure, as shown in Figure 2.

The moments, thus computed, are distributed at all joints of the truss. The final moments on the members, TABLE - I

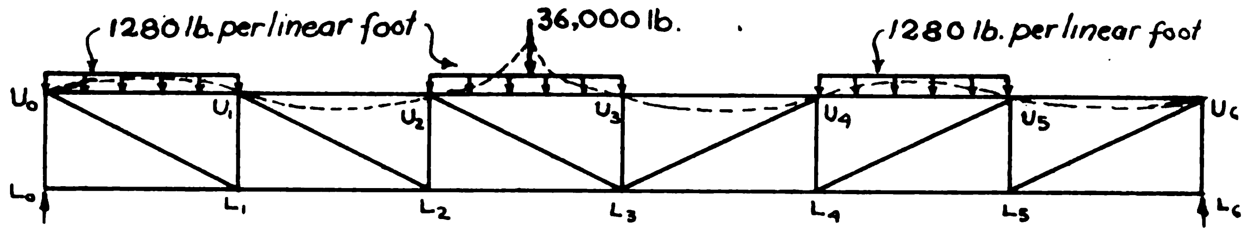
Normal Stresses and Properties of Members (Half Truss)

\- Compression
$\not+$ Tension

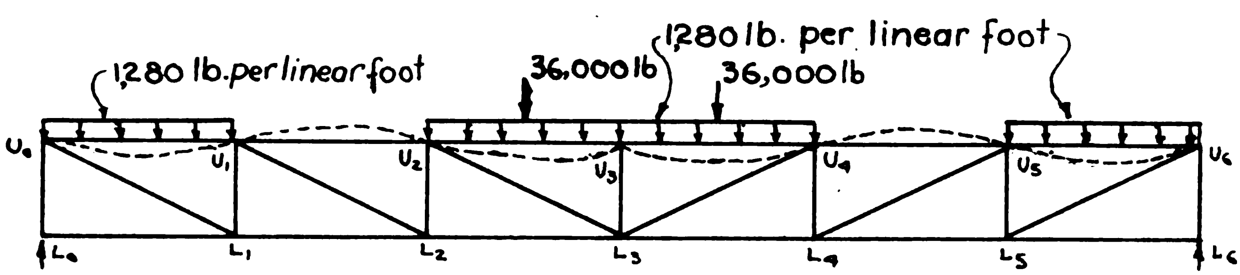
Member	Normal force due to dead load in kips.	Normal force due to side-walk live-load in kips.	Normal force due to truck live load, including impact, in kips.	Length of member in inches 'L'	Depth of member in inches 'd'	Effective width of member in inches	Openings provided in inches	Moment of Inertia of the section per foot width in in.^{4} 'I'	Relative Stiffness $\frac{I}{L}$ [Average]
U_0-U_1	-450.00	-36.00	-226.00	240	7	240	---	344	1.430
U_1-U_2	-720.00	-57.50	-362.00	240	7	240	---	344	1.430
U_2-U_3	-810.00	-64.80	-407.00	240	7	240	---	344	1.430
U_0-L_0	-247.50	-21.80	-136.00	120	5	120	120	125	0.520
U_1-L_1	-180.00	-18.00	-114.00	120	5	120	120	125	0.520
U_2-L_2	- 88.50	-11.50	- 79.50	120	5	120	120	125	0.520
U_3-L_3	- 45.00	- 6.48	- 51.50	120	5	120	120	125	0.520
L_0-L_1	-----	-----	-----	240	$4\frac{1}{2}$	240	---	91	0.354
L_1-L_2	$\not+$450.00	$\not+$36.00	$\not+$226.00	240	$4\frac{1}{2}$	240	---	91	0.354
L_2-L_3	$\not+$720.00	$\not+$57.50	$\not+$362.00	240	$4\frac{1}{2}$	240	---	91	0.354
U_0-L_1	$\not+$503.00	$\not+$40.20	$\not+$254.00	270	$4\frac{1}{2}$	144	96	91	0.220
U_1-L_2	$\not+$302.00	$\not+$25.70	$\not+$177.50	270	$4\frac{1}{2}$	120	120	91	0.170
U_2-L_3	$\not+$100.50	$\not+$14.50	$\not+$115.00	270	$4\frac{1}{2}$	120	120	91	0.170



(i) Dead load acting on structure.

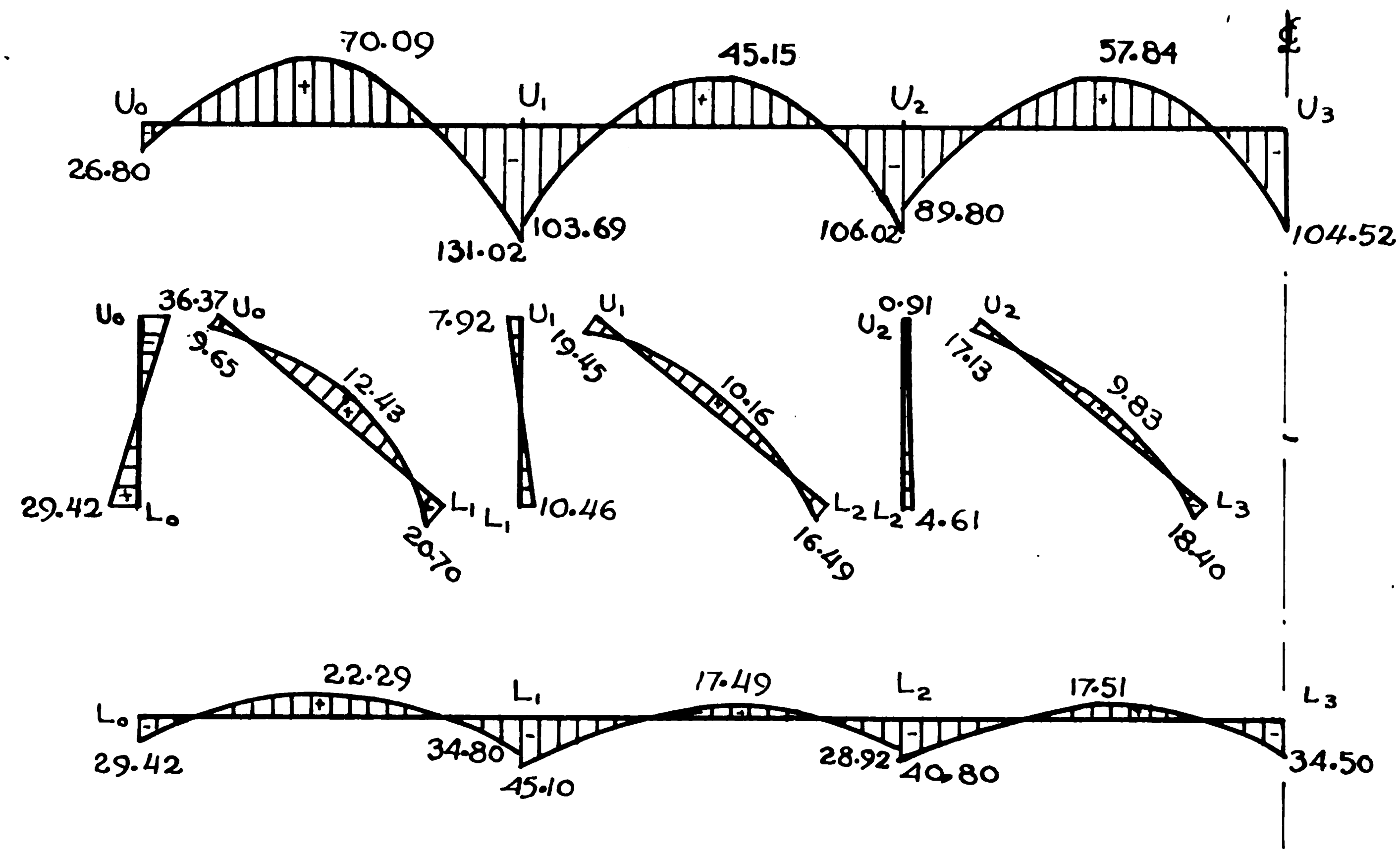


(ii) Live load acting on structure to produce maximum +ve moment at the centre of span U_2-U_3



(iii) Live load acting on structure to produce maximum -ve moment at U_3

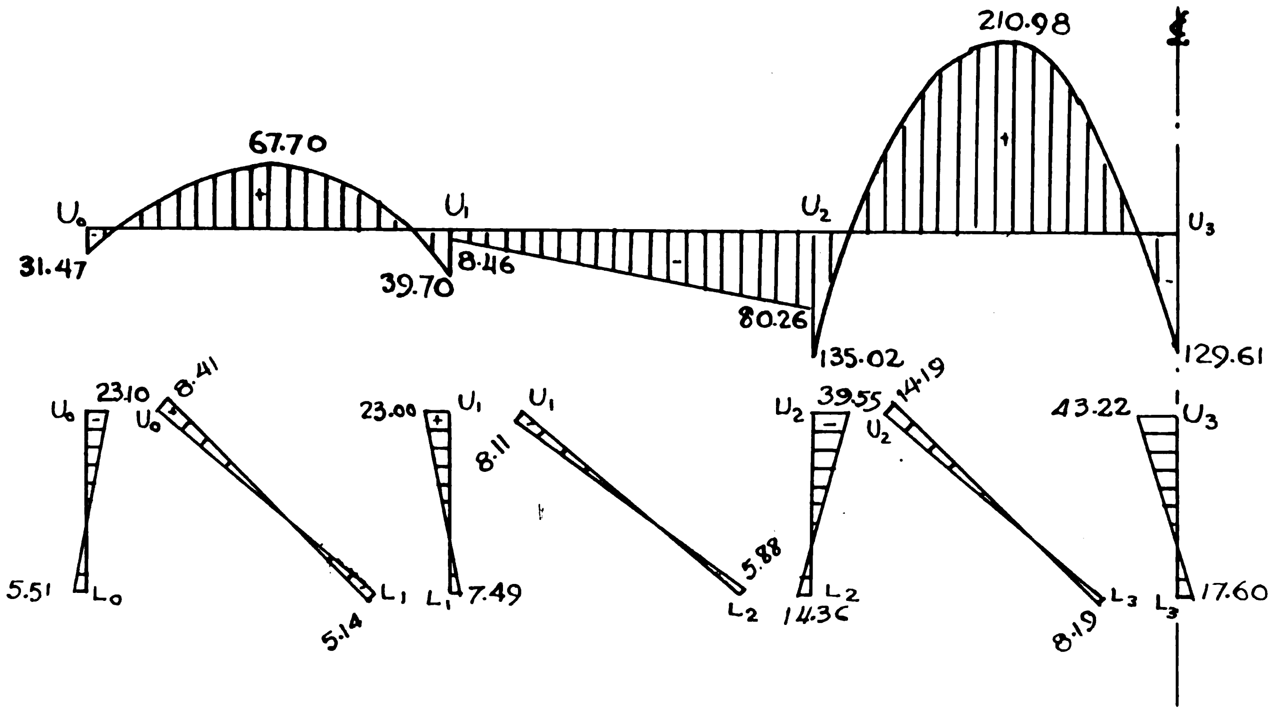
Figure - 2.



BENDING MOMENT DIAGRAM FOR DEAD LOAD ON STRCTURE.

[Moments in k.ft.]

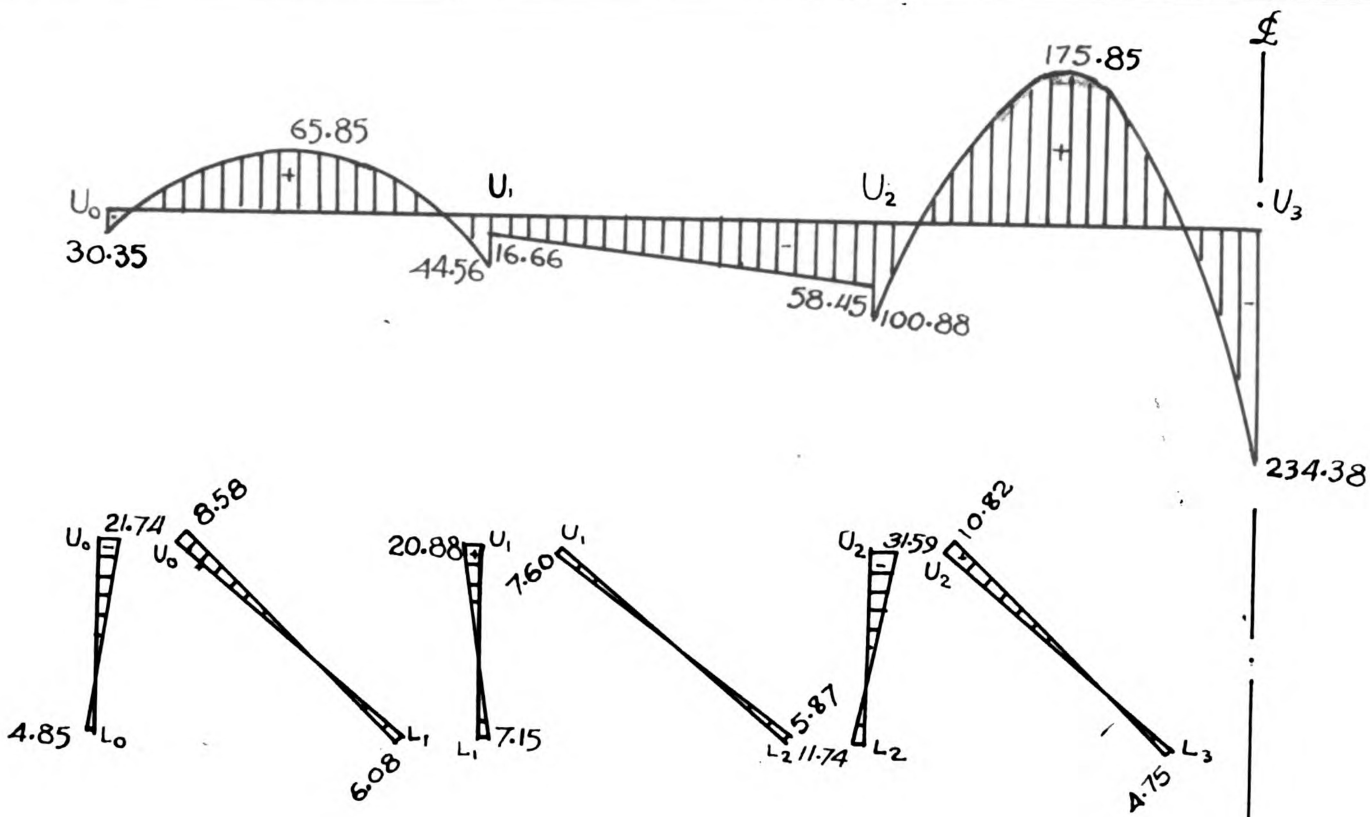
Figure - 3.



BENDING MOMENT DIAGRAM FOR
LOADING CONDITION TO
PRODUCE $+M_{max}$

Bending moments on bottom chord members are not shown.
The moments are in k ft

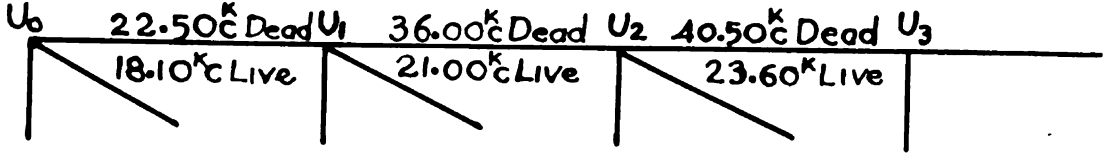
Figure 4



BENDING MOMENT DIAGRAM FOR
LOADING CONDITION TO
PRODUCE $-M_{max}$.

Bending moments on Bottom chord members are not shown.
The moments are in K.ft.

Figure 5



U_0
22.50^{K}c Dead
U_1
36.00^{K}c Dead
U_2
40.50^{K}c Dead
U_3
18.10^{K}c Live
21.00^{K}c Live
23.60^K Live

Figure-6

after distribution for all the three loading conditions, are shown in Figure 2. The moment diagrams are drawn as shown in Figures 3, 4, 5.

(d). Design of Members.

1. Top chord. The floor slab is used as the top chord of the prestressed concrete trussed girder. It is a continuous prestressed slab supported on verticles as shown in figure 6. The prestressing of the slab is achieved by post-tensioning the continuous curved cable which passes through the cross-section of the slab. In any cross-section, the prestressing operation induces not only the moment P • e, but at the same time a secondary bending moment. This is due to the fact that the deformations, created by the prestressing, create external reactions,* and hence external moments, which have nothing to do with the moments caused by the external load that may exist at the moment of prestressing. These secondary bending moments can be taken into account if the slab is loaded with equivalent uniform loading of proper magnitude and direction, hence the design of top chord is separated into two parts:

(i) Eccentricities of the cable are first found without taking into consideration the secondary moments due to prestressing.

*G. Magnel, "Significant features of Prestressed Concrete," Canadian Conference on Prestressed Concrete. 1954.

(ii) Stresses at all sections are checked to be within the limits when the primary stresses, due to normal forces and external moments; and secondary stresses, due to prestressing, are taken into consideration.

(i) Eccentricity of cable over support and at center of each span:

M_d Max. = -131,020 x 12 in. lb.

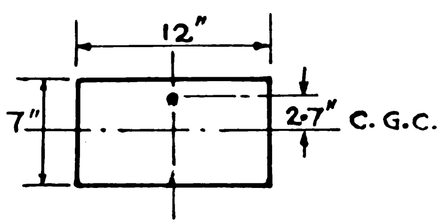
Average moment per foot width of slab = -6,551 x 12 in. lb.

From figure 7.

$I = \frac{1}{12} \times 12 \times 7^3 = 344 \text{ in.}^4$

$A = 84 \text{ in.}^4$

$\therefore r^2 = \frac{344}{84} = 4.10 \text{ in}^2$



Section at Support

Figure-7

e_u = 3.50
-0.50 cover
-0.30 1/2 diameter of cable
2.70 in.

Maximum dead load compressive stress due to direct compression = $\frac{40,500}{84}$ = 482 p.s.i.

Minimum dead load compressive stress due to direct compression = $\frac{22,500}{84}$ = 268 p.s.i.

$\therefore$ When dead load is acting on truss:

$$\frac{P}{A} - \frac{P \cdot e \cdot Y_b}{I} + \frac{M_d Y_b}{I} + 482 = 0$$

$$\therefore P\left[1 - \frac{e \cdot Y_b}{r^2}\right] + \frac{M_d \cdot Y_b}{r^2} + 482 \times 84 = 0$$

$$\therefore P\left[1 - \frac{2 \cdot 7 \times 3 \cdot 5}{4 \cdot 10}\right] + \frac{6,551 \times 12 \times 3 \cdot 5}{4 \cdot 10} + 40,500 = 0$$

$$\therefore P = 82,700 \text{ lbs.}$$

Allowing 15% losses, P_i = 97,500 lbs. Under this prestress, stresses are:

$$F_b = \frac{97,500}{84} - \frac{97,500 \times 2 \cdot 7 \times 3 \cdot 5}{344} + \frac{6,551 \times 12 \times 3 \cdot 5}{344} + 482$$

= 248 p.s.i. tension

Ft = 3632 p.s.i. compression, which is not allowable.

$\therefore$ The design is governed by allowable compressive stresses of 2000 p.s.i.

$$P\left[1 + \frac{e \cdot Yt}{r^2}\right] - \frac{M_d \cdot Yt}{r^2} + 482 = 2000$$

$$\therefore P = 59,000 \text{ lbs.}$$

Allowing 15% losses, P_i = 69,500 lbs.

$$\therefore Ft = \frac{69,500}{84} + \frac{69,500 \times 2 \cdot 7 \times 3 \cdot 5}{344} - \frac{6551 \times 12 \times 3 \cdot 5}{344} + 482$$

= 2579 p.s.i. compression.

Though this is initial stress, and it may come to allowable limit after time, it is higher,

$\therefore$ Use P_i = 62,000 lbs.

$$F_t = \frac{62,000}{84} + \frac{62,000 \times 2 \cdot 7 \times 3 \cdot 5}{344} - \frac{6551 \times 12 \times 3 \cdot 5}{344} + 482$$

= 2,120 p.s.i. compression

F_b = 320 p.s.i. compression

After some time P_i may reduce by about 15% and thus come to P = 52,600 lbs. Under this condition,

F_t = 1758 p.s.i. compression

F_b = 458 p.s.i. compression.

When live load is acting on the truss:

$M_{L\,max}$ at support is -274,380 x 12 in. lb. and hence average moment per foot width is -11,720 x 12 in. lb.

$$\therefore \begin{matrix} F_b \\ \text{or} \\ F_t \end{matrix} = \frac{11,720 \times 12 \times 3.50}{344} = \begin{matrix} 1,430 \text{ p.s.i. compression} \\ \\ 1,430 \text{ p.s.i. compression} \end{matrix}$$

$\therefore$ When dead load and live load are acting on the truss; the final stresses under P_i are:

F_b = 320 + 1430 = 1,750 p.s.i. compression

F_t = 2120 - 1430 = 690 p.s.i. compression

and under P_i are:

F_b = 458 + 1430 = 1,880 p.s.i. compression

F_t = 1758 - 1430 = 328 p.s.i. compression.

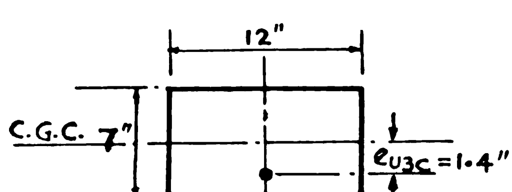
$\therefore$ Eccentricity of the parabolic cable at the support is kept 2.7 in. and P_i = 62,000 lb.

At center of span.

When dead load is acting on truss:

M_d max. = + 70,030 x 12 in. lb.

$\therefore$ Average moment per foot width = 3,504 x 12 in. lb.



Section at Center

Figure-8

$P_1 = 62{,}000$ lb..

$$\frac{P_1}{A} + \frac{P \cdot e_{u2c} \cdot Y_b}{I} - \frac{M_d \cdot Y_b}{I} + 482 = 2000$$

$$\frac{62{,}000}{84} + \frac{62{,}000 \cdot x\ e_{u2c} x\ 3 \cdot 5}{344} + \frac{3504\ x\ 12}{344} + 482$$

$\therefore\ e_{u2c} = 902 = 1 \cdot 43$ lb. in.

Use $e_{u2c} = 1 \cdot 4$ lb. in.

$$\therefore F_t = \frac{62{,}000}{84} - \frac{62{,}000\ x\ 1 \cdot 4\ x\ 3 \cdot 5}{344} + \frac{3504\ x\ 12}{344} + 482$$

$= 460$ p.s.i. compression

$F_b = 1980$ p.s.i. compression.

When P = 52,600 lbs.

$$F_t = \frac{52{,}600}{84} - \frac{52{,}000\ x\ 1 \cdot 4\ x\ 3 \cdot 5}{344} + \frac{3504\ x\ 12}{344} + 482$$

$= 481$ p.s.i. compression

$F_b = 1{,}737$ p.s.i. compression.

When live load is acting :

$M_{L\ max.} = 210{,}980\ x\ 12$ in. lb.

Average moment per foot width $= 10{,}549\ x\ 12$ in. lb.

$$\begin{array}{l} F_t \\ \text{or} \\ F_b \end{array} = \frac{10{,}549\ x\ 12\ x\ 3 \cdot 5}{344} = \begin{array}{l} 1290 \text{ p.s.i. compression} \\ \\ 1290 \text{ p.s.i. tension.} \end{array}$$

$\therefore$ When dead load and live load are acting on the truss, the final stresses under P_1, are:

$F_t = 460 + 1290 = 1{,}750$ p.s.i. compression

$F_b = 1980 - 1290 = 690$ p.s.i. compression.

TABLE II

Stresses in Slab Sections
(Without taking into consideration the secondary stresses due to prestressing).

	Due to		Line	Stresses Concrete in p;s.i.		Prestressing Force in lb. per ft. width of slab
				Top F_t	Bottom F_b	
At support where e = 2.7 in.	Initial Prestress		1	2,438C	962T	62,000
	Normal force due to dead load		2	482C	482C	62,000
	Normal force due to live load		3	200C	200C	62,000
	Dead load moment		4	800T	800T	62,000
	Live load moment		5	1,430T	1,430C	62,000
	Combined after dead-load	1+2+4	6	2,120C	320T	62,000
	Combined after live-load	1+2+3+4+5	7	320C	1,310C	62,000
	Final Prestress		8	2,076C	824T	52,600
	Combined after dead-load	8+2+4	9	1,758C	458C	52,600
	Combined after live-load	8+2+3+4+5	10	528C	2,088T	52,600
At center of span where e = 1.4 in.	Initial Prestress		11	144T	1,620C	62,000
	Normal force due to dead load		12	482C	482C	62,000
	Normal force due to live load		13	200C	200C	62,000
	Dead load moment		14	122C	122T	62,000
	Live load moment		15	1,290C	1,290T	62,000
	Combined after dead-load	11+12+14	16	460C	1,980C	62,000
	Combined after live load	11+12+13+14+15	17	1,950C	890C	62,000
	Final Prestress		18	123T	1,377C	52,600
	Combined after dead load	18+12+14	19	381C	1,737C	52,600
	Combined after live load	18+12+13+14+15	20	1,871C	647C	52,600

and under P are:

F_t = 481 + 1290 = 1,771 p.s.i. compression

F_b = 1737 - 1290 = 447 p.s.i. compression.

$\therefore$ Eccentricity at the support is 3.7 in. and at the center of each span 1.4 in.

The results of the stresses are tabulated below in Table II.

(ii)

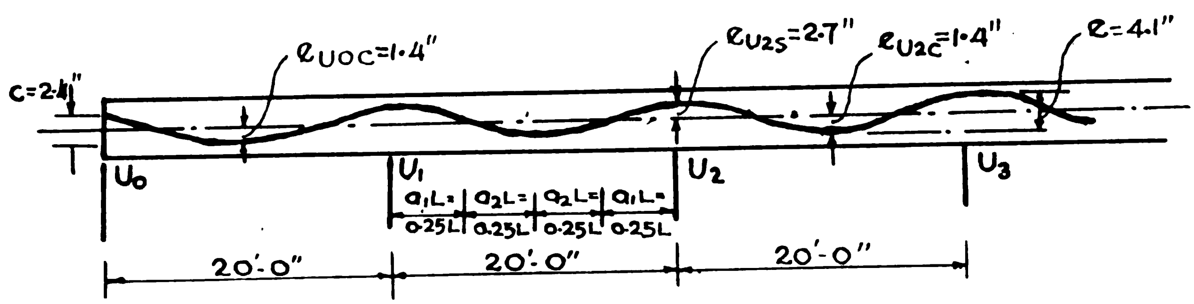
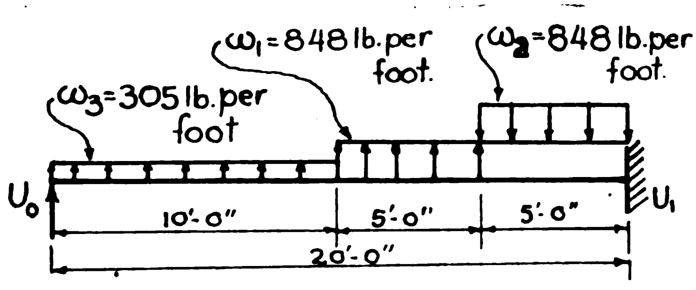


Figure - 9

The profile of the cable is a second degree parabola. The point of contraflexture for the cable is taken as 1/4th span length of each panel.

Secondary moments due to prestressing are induced at the supports. These moments, together with primary moments, can be determined by application of uniform loads of proper magnitude as shown in figures 10, 11. For end span:



Equivalent Loading on U_0-U_1.

Figure-10

P_o = 63,000 lb.

e = 4.1 in.

c = 2.46 in.

$a_1 = a_2 = 0.25$

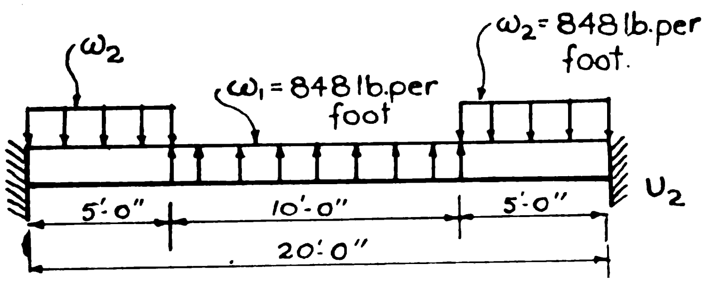
$$W_3 = \frac{8P_i \times C^*}{L^2} = \frac{8 \times 62{,}000 \times \frac{2\cdot 46}{12}}{20^2} = 305 \text{ lb. per linear foot.}$$

$$W_1 = \frac{4P_i \times e}{a_1 L^2} = \frac{4 \times 62{,}000 \times \frac{4\cdot 10}{12}}{0\cdot 25 \times 20^2} = 848 \text{ lb. per linear foot.}$$

$$W_2 = \frac{4P_i \times e}{a_2 \cdot L^2} = 848 \text{ lb. per linear foot}$$

$$\therefore M^F_{U_1U_0} = 12{,}055 \times 12 \text{ in. lb.}$$

Similarly analysing the intermediate span:



$$M^F_{U_1U_2} = M^F_{U_2U_1} = 10{,}680 \text{ in. lb.}$$

Equivalent Loading on U_1-U_2

Figure-11

These moments are distributed at all the top chord joints. The moment at the joint U_3 is 10,680 x 12 in. lb. and at U_2 is 10,508 x 12 in. lb.

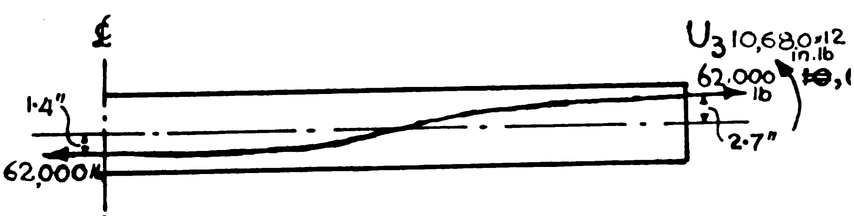


Figure-12

From figure 12 the moment at the center of the intermediate span U_2-U_3 is given by

$$-62{,}000 \times \frac{1\cdot 4}{12} \times 12 + \left(\frac{10{,}680 + 10{,}508}{2} - 62{,}000 \times \frac{2\cdot 7}{12}\right) \times 12$$

$$= -10{,}631 \times 12 \text{ in. lb.}$$

Final stresses at section at support U_3:

When P_i = 62,000 lb.

$$F_t = \frac{P_i}{A} + \begin{array}{l}\text{Compression}\\ \text{due to moment}\\ \text{due to } P_i\end{array} - \frac{M_d \cdot Y_t}{I} - \frac{M_L \cdot Y_t}{I}$$

$$+ \begin{array}{l}\text{Compression due to}\\ \text{normal force in the}\\ \text{member.}\end{array}$$

$$= \frac{62{,}000}{84} + \frac{10{,}680 \times 12 \times 3\cdot5}{344} - \frac{6{,}551 \times 12 \times 3\cdot5}{344}$$

$$- \frac{11{,}720 \times 12 \times 3\cdot50}{344} + 482 + 200$$

= 490 p.s.i. Compression.

F_b = 2150 p.s.i. Compression   2000

This is due to initial prestress, and therefore it is allowable.

Final Stresses at middle section of span U_2-U_3

$$F_t = \frac{P_i}{A} - \begin{array}{l}\text{Tension due to}\\ \text{moment due to}\\ P_i\end{array} + \frac{M_d \cdot Y_t}{I} + \frac{M_L \cdot Y_t}{I}$$

$$+ \begin{array}{l}\text{Compression due to}\\ \text{normal force in the}\\ \text{member.}\end{array}$$

$$= \frac{62{,}000}{84} - \frac{8928 \times 12 \times 3\cdot5}{344} + \frac{3{,}504 \times 12}{344} + \frac{10{,}549 \times 12 \times 3\cdot5}{344}$$

= 1604 p.s.i. compression.

F_b = 1100 p.s.i. compression.

Final stresses at support and centre of span are tabulated in Table III.

When P = 52,600, W_3 = 258 lb. per linear foot; W_1 = 740 lb. per linear foot; W_2 = 740 lb. per linear foot, and

$M^F_{U_1-U_0} = 10,200 \times 12$ in. lb.

$M^F_{U_1-U_2} = M^F_{U_2-U_1}$ --- $= 9,000 \times 12$ in. lb.

These moments are distributed at all the top chord joints.

The moments at the joint $U_3 = 9,050 \times 12$ in. lb. and at U_2 are 8,900 x 12 in. lb.

Moment at center of intermediate span =

$$-52,600 \times \frac{1\cdot 4}{12} \times 12 + \left(\frac{9,050 + 8,900}{2} - 52,600 \times \frac{2\cdot 7}{12}\right) \times 12$$

$= -8,965 \times 12$ in. lb.

Final stresses at support and center of the span are tabulated in table No. III.

2. Design of web members:

(i) Diagonals:

$\underline{U_0-L_1}$

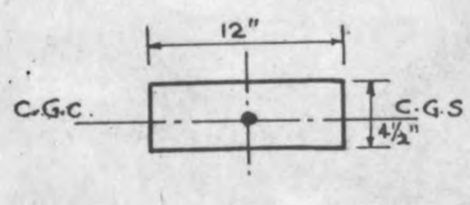


Figure 13

The slab of the diagonal is prestressed by cables passing through C.G.C. of the section of the member. Opening provided in this diagonal is eight feet.

$-M_d$ max. $= 20,700 \times 12$ in. lb.

$+ M_d$ max. $= 12,430 \times 12$ in. lb.

$-M_L$ max. $= -6,080 \times 12$ in. lb.

$+M_L$ max. = Negligible.

TABLE III

Final Stresses in Slab Sections

	Due to		Line	Stresses Concrete in p.s.i. Top F_t	Bottom F_b	Prestressing Force in lb. per ft. width of slab
	Initial Prestress		1	2,038C	762T	62,000
	Normal force due to dead load		2	482C	482C	62,000
	Normal force due to live load		3	200C	200C	62,000
	Dead load moment		4	800T	800C	62,000
	Live load moment		5	1,430T	1,430C	62,000
	Combined after dead load	1+2+4	6	1,720C	520C	62,000
	Combined after live load	1+2+3+4+5	7	480C	2,150C	62,000
	Final Prestress		8	1,725C	575T	52,600
	Combined after dead load	8+2+4	9	1,407C	707C	52,600
	Combined after live load	8+2+3+4+5	10	177C	2,337C	52,600
	Initial Prestress		11	350T	1,830C	62,000
	Normal force due to dead load		12	482C	482C	62,000
	Normal force due to live load		13	200C	200C	62,000
	Dead load moment		14	122C	122T	62,000
	Live load moment		15	1,290C	1,290T	62,000
	Combined after dead load	11+12+14	16	1140C	2,100C	62,000
	Combined after live load	11+12+13+14+15	17	1,604C	1,100C	62,000
	Final Prestress		18	475T	1,725C	52,600
	Combined after dead load	18+12+14	19	89C	2,085C	52,600
	Combined after live load	18+12+13+14+15	20	1,579C	935C	52,600

At Joint

Under dead load,

$$F_t \text{ or } F_b = \frac{1,725 \times 12 \times 2 \cdot 25}{91} = \begin{array}{l} 510 \text{ p.s.i. tension} \\ 510 \text{ p.s.i. compression} \end{array}$$

Under live load,

$$F_t \text{ or } F_b = \frac{6,080 \times 2 \cdot 25}{91} = \begin{array}{l} 298 \text{ p.s.i. Tension} \\ 298 \text{ p.s.i. Compression} \end{array}$$

Under Normal force;

$$\text{Dead load}: \frac{503,000}{54 \times 12} = 776 \text{ p.s.i. Tension}$$

$$\text{Live load}: \frac{294,000}{54 \times 12} = 454 \text{ p.s.i. Tension}$$

Maximum Stresses due to dead load, live load, and normal forces are;

$$F_b = + 510 + 298 - 776 - 454 = 422 \text{ p.s.i. tension.}$$

$$F_t = -510 - 298 - 776 - 454 = 2038 \text{ p.s.i. tension.}$$

$$\therefore P = 2038 \times 54 = 110,000 \text{ lb.}$$

$$\therefore P_1 = 129,000 \text{ lb.}$$

The resultant stresses in the member are tabulated in Table IV.

5 Nos. 0.600 in. diameter Roebling cable initially stressed to 26,000 lb. each are used per foot width of slab. Final stress in each cable is 22,000 lb.

U_1-L_2

Section of the diagonal is the same as that of diagonal U_0-L_1 as shown in Figure 13. The opening provided is 10 feet.

$-M_{d\ max.} = -19,490 \times 12$ in. lb.

$+M_{d\ max.} = +10,180 \times 12$ in. lb.

$-M_{L\ max.} = -\ 8,180 \times 12$ in. lb.

$+M_{L\ max.}$ = Negligible.

At Joint,

Under dead load,

$$F_t \text{ or } F_b = \frac{1,949 \times 12 \times 2\cdot25}{91} = \begin{array}{l} 580 \text{ p.s.i. Tension.} \\ 580 \text{ p.s.i. Compression.} \end{array}$$

Under live load,

$$F_t \text{ or } F_b = \frac{818 \times 12 \times 2\cdot25}{91} = \begin{array}{l} 242 \text{ p.s.i. Tension.} \\ 242 \text{ p.s.i. Compression.} \end{array}$$

Under Normal Force,

$$\text{Dead load} = \frac{302,000}{54 \times 10} = 560 \text{ p.s.i. Tension.}$$

$$\text{Live load} = \frac{203,000}{54 \times 10} = 376 \text{ p.s.i. Tension.}$$

Maximum Stresses due to dead load, live load and normal forces are:

$F_b = +\ 580 + 242 - 560 - 376 = 144$ p.s.i. tension.

$F_t = -\ 580 - 242 - 560 - 376 = 1,758$ p.s.i. tension.

$\times\ 54 =$

$\therefore\ P = 1,758 \times 54 = 95,000$ lb.

$\therefore\ P_1 = 112,000$ lb.

The resultant stresses in the member are tabulated in Table IV.

5 Nos. 0·600 in. diameter Roebling cable initially stressed to 22,400 lb. each are used per foot width of the slab. Final stress in each cable is 19,000 p.s.i.

$\underline{U_2 - L_3}$

Section of the diagonal is the same as that of diagonals U_0-L_1 and U_2-L_3 as shown in figure 18. The opening provided is 10 feet.

$-M_{d\ max.} = 18,400 \times 12$ in. lb.

$+ M_{d\ max.} = 9,830 \times 12$ in. lb.

$-M_{L\ max.} = 8,190 \times 12$ in. lb.

$+M_{L\ max.}$ = Negligible.

At Joint,

Under dead load,

$$F_t \text{ or } F_b = \frac{1840 \times 12 \times 2{\cdot}25}{91} = \begin{array}{l} 545 \text{ p.s.i. Tension} \\ 545 \text{ p.s.i. Compression} \end{array}$$

Under live load,

$$F_t \text{ or } F_b = \frac{819 \times 12 \times 2{\cdot}25}{91} = \begin{array}{l} 243 \text{ p.s.i. Tension} \\ 243 \text{ p.s.i. Compression} \end{array}$$

Under normal force,

$$\text{Dead load} = \frac{100,500}{54 \times 10} = 186 \text{ p.s.i. Tension.}$$

$$\text{Live load} = \frac{129,500}{54 \times 10} = 240 \text{ p.s.i. Tension}$$

∴ Maximum stresses due to dead load, live load and normal forces are:

TABLE IV.

Stresses in Diagonals

	Due to		Line	Stresses Concrete in p.s.i. Top F_t	Bottom F_b	Prestressing Force in lb. per ft. width of slab
U_0-L_1 : e = 0 — At joint	Initial Prestress		1	2,380C	2,380C	129,000
	Normal force due to dead load		2	776T	776T	129,000
	Normal force due to live load		3	454T	454T	129,000
	Dead load moment		4	510T	510C	129,000
	Live load moment		5	298T	298C	129,000
	Combined after dead load	1+2+4	6	1,094C	2,114C	129,000
	Combined after live load	1+2+3+4+5	7	342C	1,958C	129,000
	Final Prestress		8	2,038C	2,038C	110,000
	Combined after dead load	8+2+4	9	752C	1,772C	110,000
	Combined after live load	8+2+3+4+5	10	0	1,616C	110,000
At center of member	Initial Prestress		11	2,380C	2,380C	129,000
	Normal force due to dead load		12	776T	776T	129,000
	Normal force due to live load		13	454T	454T	129,000
	Dead load moment		14	310C	310T	129,000
	Live load moment		15	---	---	129,000
	Combined after dead load	11+12+14	16	2,070C	1,294C	129,000
	Combined after live load	11+12+13+14	17	1,616C	840C	129,000
	Final Prestress		18	2,038C	2,038C	110,000
	Combined after dead load	18+12+14	19	1,672C	952C	110,000
	Combined after live load	18-12-13-14	20	1,218C	498C	110,000

TABLE IV (Continued)

e=0	Near joint	Initial Prestress		1	2,070C	2,070C	112,000
		Normal force due to dead load		2	560T	560T	112,000
		Normal force due to live load		3	376T	376T	112,000
		Dead load moment		4	580T	580C	112,000
		Live load moment		5	242T	242C	112,000
		Combined after dead load	1+2+4	6	930C	2,090C	112,000
		Combined after live load	1+2+3+4+5	7	312C	1,956C	112,000
		Final Prestress		8	1,758C	1,758C	95,000
		Combined after dead load	8+2+4	9	618C	1,778C	95,000
		Combined after live load		10	Θ	1,644C	95,000
U_1-L_2	At center of member	Initial Prestress		11	2,070C	2,070C	112,000
		Normal force due to dead load		12	560T	560T	112,000
		Normal force due to live load		13	376T	376T	112,000
		Dead load moment		14	300C	300T	112,000
		Live load moment		15	---	---	112,000
		Combined after dead load	11+12+14	16	1,810C	1,110C	112,000
		Combined after live load	11+12+13+14	17	1,534C	810C	112,000
		Final Prestress		18	1,758C	1,758C	95,000
		Combined after dead load	18+12+14	19	1,498C	898C	95,000
		Combined after live load	18+12+13+14	20	1,122C	934C	95,000

TABLE IV (Continued)

e = 0	At Member Joint	Initial Prestress		1	1,430C	1,430C	77,400
		Normal force due to dead load		2	186T	186T	77,400
		Normal force due to live load		3	240T	240T	77,400
		Dead load moment		4	545T	545C	77,400
		Live load moment		5	243T	243C	77,400
		Combined after dead load	1+2+4	6	699C	1,789C	77,400
		Combined after live load	1+2+3+4+5	7	358C	1,792C	77,400
		Final Prestress		8	1,276C	1,276C	65,400
		Combined after dead load	8+2+4	9	485C	1,575C	65,400
		Combined after live load	8+2+3+4+5	10	0	1,578C	65,400
U_0-L_2	At Center of Member	Initial Prestress		11	1,430C	1,430C	77,400
		Normal force due to dead load		12	186T	186T	77,400
		Normal force due to live load		13	240T	240T	77,400
		Dead load moment		14	292C	292T	77,400
		Live load moment		15	---	---	77,400
		Combined after dead load	11+12+14	16	1,536C	952C	77,400
		Combined after live load	11+12+13+14	17	1,290C	712C	77,400
		Final Prestress		18	1,216C	1,216C	65,400
		Combined after dead load	18+12+14	19	1,422C	738C	65,400
		Combined after live load	18+12+13+14	20	1,182C	498C	65,400

$F_b = + 545 + 243 - 186 - 240 = 360$ p.s.i. Compression

$F_t = - 545 - 243 - 186 - 240 = 1216$ p.s.i. Tension

$\therefore P = 1216 \times 54 = 65,400$ lb.

$\therefore P_i = 77,400$ lb.

The resultant stresses are tabulated in table No. IV.

3 Nos. 0·600 in. diameter Roebling cable initially stressed to 25,800 lb. each are used for a foot width of slab.

Final stress in the cable = 21,800 lb.

(ii) Verticles:

These are the vertical slabs with 10 feet openings.

The combined moment due to dead load, and live load is at the join U_0,

$-M_{d\ max.} = 3,637 \times 12$ in. lb. per foot width.

$-M_{L\ max.} = 2,173 \times 12$ in. lb. per foot width.

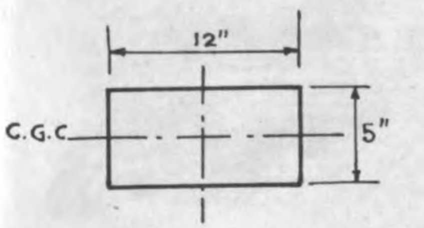


Figure-14

Normal forces,

Dead load = 24,750 lb. per foot width.

Live load = 15,780 lb. per foot width.

$A = 60$ in.^{2}

$I = \frac{1}{12} \times 12 \times 5^3 = 125$ in.

$$F_t = \frac{40,500}{60} - \frac{5,810 \times 12 \times 2.5}{125}$$

$$= 675 - 1,390 = 715 \text{ Tension.}$$

This is the stress due to rigidity of the joint. It can be accounted for by increasing the section at the end by providing haunches. These haunches also make the joints rigid. Furthermore, this is the stress due to moment at the center line of the upper slab. If the depth is taken into consideration, the stress reduces. Some reinforcing steel may be provided at every joint to take care of this tensile force.

$F_b = 675 + 1420 = 2095$ p.s.i. compression.

3. Bottom chord:

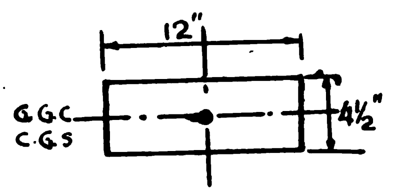


Figure 15

The bottom chord is a continuous slab prestressed by cables passing through C. G. C. of the section of the member.

The portion of the member L_1-L_2-L_3-L_4-L_5 prestressed by one uniform prestressing force, while the remaining portion by a different prestressing force.

<u>L_1-L_2-L_3-L_4-L_5</u>

$$I = \frac{1}{12} \times 12 \times 4.5^3 = 91 \text{ in.}^4$$

$$A = 54 \text{ in.}^2$$

$-M_{D\ max.} = -45{,}100 \times 12$ in. lb.

$= -\ 2{,}255 \times 12$ in. lb. per foot width of slab.

$+M_{D\ max.} = 17{,}500 \times 12$ in. lb.

$= +\ 876 \times 12$ in. lb.

M_L = Neglected

At joint,

Under dead load,

$$\begin{matrix} F_t \\ \text{or} \\ F_b \end{matrix} = \frac{2{,}255 \times 12 \times 2.25}{81} = \begin{matrix} 765 \text{ p.s.i. Tension} \\ \\ 765 \text{ p.s.i. Compression} \end{matrix}$$

Under Normal force,

$$\text{Dead load} = \frac{720{,}000}{54 \times 20} = 667 \text{ p.s.i. Tension.}$$

$$\text{Live load} = \frac{419{,}000}{54 \times 20} = 388 \text{ p.s.i. Tension.}$$

P_1 = 129,000 lb. per foot-width of slab.

P = 110,000 lb. per foot width of slab.

The final stresses in the member are tabulated in Table V.

L_0-L_1; L_5-L_6:

$-M_{d\ max.} = -34{,}800 \times 12$ in. lb.

$= 1{,}740 \times 12$ in. lb. per foot width of slab.

$+M_{d\ max.} = 22{,}290 \times 12$ in. lb.

$1{,}114 \times 12$ in. lb. per foot width of slab.

P_1 = 64,500 lb. per foot width of slab.

P = 55,000 lb. per foot width of slab.

TABLE V.

Stresses in Bottom Chord.

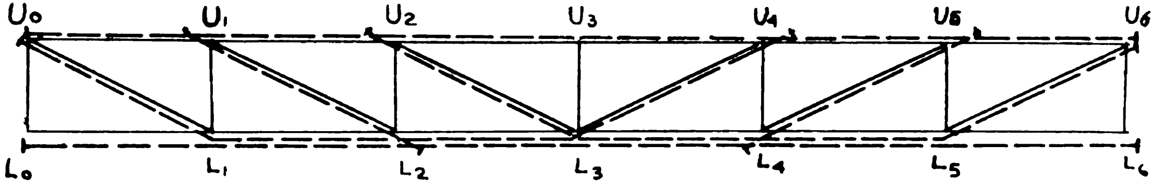
		Due to		Line	Stresses Concrete in p.s.i. Top F_t	Stresses Concrete in p.s.i. Bottom F_b	Prestressing Force in lb. per ft. width of slab
e = 0	Near Joint	Initial Prestress		1	1,190C	1,190C	51,600
		Normal force due to dead load		2	---	---	64,500
		Normal force due to live load		3	---	---	64,500
		Dead load moment		4	517T	517C	64,500
		Live load moment		5	---	---	64,500
		Combined after dead load	1+2+4	6	673C	1,707C	64,500
		Combined after live load	1+2+3+4+5	7	673C	1,707C	64,500
		Final Prestress		8	1,020C	1,020C	44,000
		Combined after dead load	8+2+4	9	503C	1,537C	55,000
		Combined after live load	8+2+3+4+5	10	503C	1,537C	55,000
L0-L1	At center of member	Initial Prestress		11	1,190C	1,190C	64,500
		Normal force due to dead load		12	---	---	64,500
		Normal force due to live load		13	---	---	64,500
		Dead load moment		14	332C	332T	64,500
		Live load moment		15	---	---	64,500
		Combined after dead load	11+12+14	16	1,522C	858C	64,500
		Combined after live load	11+12+13+14+15	17	1,522C	858C	64,500
		Final Prestress		18	1,020C	1,020C	55,000
		Combined after dead load	18+12+14	19	1,352	778C	55,000
		Combined after live load	18+12+13+14+15	20	1,352	778C	55,000

TABLE V (Continued)

e = 0	Near Joint	Initial Prestress		1	2,380C	2,380C	129,000
		Normal force due to dead load		2	417T	417T	129,000
		Normal force due to live load		3	242T	242T	129,000
		Dead load moment		4	765T	765C	129,000
		Live load moment		5	---	---	129,000
		Combined after dead load	1+2+4	6	1,198C	2,728C	129,000
		Combined after live load	1+2+3+4+5	7	956C	2,586C	129,000
		Final Prestress		8	2,050C	2,050C	110,000
		Combined after dead load	8+2+4	9	968C	2,398C	110,000
		Combined after live load	8+2+3+4+5	10	728C	2,158C	110,000
L_1-L_2	At center of span	Initial Prestress		11	2,380C	2,380C	129,000
		Normal force due to dead load		12	417T	417T	129,000
		Normal force due to live load		13	242T	242T	129,000
		Dead load moment		14	---	---	129,000
		Live load moment		15	138T	138C	129,000
		Combined after dead load	11+12+14	16	1,825C	2,101C	129,000
		Combined after live load	11+12+13+14+15	17	1,588C	1,859C	129,000
		Final Prestress		18	2,050C	2,050C	110,000
		Combined after dead load	18+12+14	19	1,495C	1,771C	110,000
		Combined after live load	18+12+13+14+15	20	1,258C	1,529C	110,000

TABLE V (Continued)

e = 0	Near joint	Initial Prestress		1	2,380C	2,380C	120,000
		Normal force due to dead load		2	667T	667T	128,000
		Normal force due to live load		3	388T	388T	129,000
		Dead load moment		4	605T	605C	128,000
		Live load moment		5	---	---	123,000
		Combined after dead load	1+2+4	6	1,108C	2,318C	129,000
		Combined after live load	1+2+3+4+5	7	720C	1,930C	129,000
		Final Prestress		8	2,050C	2,050C	110,000
		Combined after dead load	8+2+4	9	778C	2,08[illegible]C	110,000
		Combined after live load	8+2+3+4+5	10	390C	1,600C	110,000
L_3-L_2	At center of span	Initial Prestress		11	2,380C	3,790C	128,000
		Normal force due to dead load		12	667T	667T	128,000
		Normal force due to live load		13	388T	388T	119,000
		Dead load moment		14	300C	200T	120,000
		Live load moment		15	---	---	128,000
		Combined after dead load	11+12+14	16	1,977C	1,455C	128,000
		Combined after live load	11+12+13+14+15	17	1,585C	1,065	120,000
		Final Prestress		18	2,050C	2,070C	110,000
		Combined after dead load	18+12+14	19	1,843C	1,188C	110,000
		Combined after live load	18+12+13+14+15	20	1,255C	1,735C	110,000



Arrangement of cables.

[Dotted lines cables]

Figure - 16

The final stresses in the member are tabulated in Table No. V.

5 Nos. 0.600 in. diameter Roebling cables per foot width provided in diagonals U_0-L_1 and U_6-L_5 are carried over through the members L_1-L_2; L_2-L_3; L_3-L_4; L_4-L_5, and initially stressed to 28,000 lb. each. For the remaining width of eight feet, 5 nos. 0.600 in. diameter Roebling cables are provided in the all bottom chord members.

The layout of the cables is shown in figure ~~no~~. 16.

B. Secondary Stresses.

a. Secondary stresses due to prestressing. The prestressed members of the truss are post-tensioned members. When the prestressing force is applied on the member, the length of the member changes. Because the joints are rigid, secondary stresses are developed in the truss. The change in lengths of the members is tabulated in table No. VI. Williot diagram is drawn to compute relative deflections of all members. The moments on the members, due to these deflections, are computed. They/are distributed at all joints. The maximum moment, produced by the rigidity of joints, is 4.4% of the maximum moment for which the top chord is designed. Also the secondary moment produces compression at the top fibre, while the designed moment produces tension at the top of the support section. In the bottom chord, the variation is from 30% maximum to 4.5% minimum of the

TABLE VI

Relative Deflections due to Prestress

Member	Prestressing force P per ft. width of slab in lbs.	Area, A per ft. width in in.^{2}	Length of member, L in in.	$\frac{P.L}{A\ E} = \delta L$ in in.	Δ (from Williot diagram) in in.	$\frac{6EK\Delta}{L}$ in lb.
U_0-U_1	-52,600	84	240	0.0284	0.339	-87,600
U_1-U_2	-52,600	84	240	0.0234	0.203	-51,500
U_2-U_3	-52,600	84	240	0.0234	0.066	-15,700
L_0-L_1	-44,000	54	240	0.0310	0.330	-21,700
L_1-L_2	-110,000	54	240	0.0775	0.233	-12,750
L_2-L_2	-110,000	54	240	0.0775	0.068	- 3,800
U_0-L_0	---	60	120	---	0.105	-17,200
U_1-L_1	---	60	120	---	0.095	-15,600
U_2-L_2	---	60	120	---	0.055	- 8,700
U_3-L_3	---	60	120	---	0	0
U_0-L_1	-110,000	54	270	0.0872	0.390	-12,000
U_1-L_2	-95,000	54	270	0.0754	0.210	- 5,240
U_2-L_3	-65,400	54	270	0.0530	0.050	- 1,190

designed moment. In this case also, the moment is of the opposite sign. On the diagonals, the variation of the secondary moments is from 8% to 2% of the designed moment. On the verticles, the maximum variation is 30% of the designed moment. Secondary stresses, due to prestressing, are therefore not taken into account.

b. Secondary stresses due to Normal forces. Due to the normal forces, the members change in length. As the joints are rigid, secondary moments are introduced on the members. The change in lengths of the members is computed as shown in table no. VII. Williot diagram is drawn from these changes in lengths of members. From this diagram, relative deflections of the members are measured. Considering these deflections, restrained moments are computed and distributed at all joints. In the top chord, the maximum secondary moment is 3.7% of the design moment. In the bottom chord, the maximum secondary moment is 29% of the maximum design moment. The maximum secondary moment in the diagonals is 15% of the maximum design moment. The maximum secondary moment in the verticles is 13% of the maximum designed moment. Secondary stresses, due to normal forces, are therefore not taken into consideration.

TABLE VII

Relative Deflections Due to Normal Forces

Members	Loads S per ft. width of slab in in.	Area 'A' per foot width in in.^{2}	Length 'L' of member in inches	$\delta L = \frac{SL}{AE}$	Δ	$\frac{6EK\Delta}{L}$ in lb.
U_0-U_1	-35,600	84	240	-0.0161	0.403	+91,000
U_1-U_2	-56,970	84	240	-0.0253	0.294	+66,300
U_2-U_3	-64,090	84	240	-0.0282	0.110	+24,800
L_0-L_1	---	54	240	---	0.404	+24,600
L_1-L_2	+35,600	54	240	+0.0250	0.299	+16,700
L_2-L_3	+56,970	54	240	+0.0395	0.119	+ 6,150
U_0-L_0	-40,530	60	120	-0.0128	0.134	+22,000
U_1-L_1	-31,200	60	120	-0.0099	0.119	+19,500
U_2-L_2	-17,900	60	120	-0.0057	0.067	+11,000
U_3-L_3	-10,288	60	120	-0.0032	0	0
U_0-L_1	+66,430	54	270	+0.0525	0.410	+12,600
U_1-L_2	+51,530	54	270	+0.0410	0.300	+ 7,150
U_2-L_3	+23,000	54	270	+0.0182	0.107	+ 2,540

VI CONCLUSION

By inducing prestress in the members of the prestressed concrete trussed girder, the sections of the members are small. This reduces the weight of the structure, which is one of the important factors leading to more economical construction cost. The reduction in the depth of the members reduces their stiffnesses and the reduction in stiffnesses of the members reduces secondary stresses in the structure.

By using the floor slab as the top chord of the trussed girder, the normal stresses in the top chord can be used as the prestressing force. Uniform distribution of the prestress is also achieved as all the members of the prestressed concrete trussed girder are slabs.

Shrinkage and creep are two important factors which produce the uncertainty of the stress condition in the structure. These can be minimized by post tensioning of the members.

This type of prestressed concrete trussed girders can be more economical for heavy loads.

REFERENCES

1. August E. Komendant, "Prestressed Concrete Structures," page 1-21, McGraw-Hill Book Company, Inc., 1952.

2. Ibid, page 124-139

3. A. L. Parme and G. H. Paris, "Analysis of Continuous Prestressed Concrete Structures," page 195, Proceedings of the First United States Conference on Prestressed Concrete, August 14-16, 1951.

4. Clarence W. Dunham, "The Theory and Practice of Reinforced Concrete," page 382, McGraw-Hill Book Company, Inc. 1953.

5. Fritz Leonhardt, "Continuous Prestressed Concrete Beams," page 617, Journal of the American Concrete Institute, March 1953.

6. Gustav Magnel, "Prestressed Concrete," London Concrete Publications Limited, 1950.

7. Gustav Magnel, "Significant features of Prestressed Concrete," F-1 - F-12, Canadian Conference on Prestressed Concrete, 1954.

8. John G. Hendrickson, "Prestressed Concrete Pipe," page 21, Proceedings of the First United States Conference on Prestressed Concrete, August 14-16, 1951.

9. T. Y. Lin and A. C. Scordelis, "Selection and Design of Prestressed Concrete Beam Sections," Title No. 50-11, Journal of the American Concrete Institute, V.25, No. 3, November 1953.

10. The American Association of State Highway Officials, "Standard Specifications for Highway Bridges," 1953.