THE DESIGN PROCEDURES OF CONTINUOUS BRIDGES WITH PRECAST, PRESTRESSED CONCRETE GIRDERS

by

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INTRODUCTION

In recent years, continuous highway bridges with precast-prestressed concrete girders have been used in many countries. The primary advantages of this type of bridge are the elimination of maintenance costs of deck joints and deck drainage onto the substructure and the structural economy of continuous design. Continuity also can improve the appearance and riding qualities of a bridge.

There are many different kinds of construction methods available to achieve continuity of precast-prestressed concrete bridge girders or slabs. In this report, continuity is achieved for live load plus impact moment by using non-prestressed reinforcement in the deck slab and in the diaphragms over piers.

The effects of shrinkage and creep between the precast girder and the cast-in-place deck slab are more important than in other types of construction of continuous prestressed concrete bridges. The Research and Development Laboratory of the Portland Cement Association completed a research program on this kind of bridge during 1960-1961. Then the results of their research provided available information for designing a continuous precast-prestressed concrete bridge considering the effects of shrinkage and creep.

The design procedures of simple span prestressed concrete bridges are presented in most prestressed concrete textbooks. Designing continuous precast-prestressed concrete girders has been mentioned by H. K. Preston[11] and T. Y. Lin[9], while Clifford L. Freyermuth[2] described the importance of the effects of shrinkage and creep in designing a continuous precast-prestressed concrete girder in his paper. But they did not state a proper and complete procedure for designing this kind of bridge. Therefore the main purpose of this report is to present design procedures for continuous precast-prestressed concrete bridges, including the influence of shrinkage and creep. A numerical example will be included to illustrate the procedures.

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2 Freyermuth, C. L., "Design of Continuous Highway Bridges with Precast, Prestressed Girders." Journal of Prestressed Concrete Institute, April 1969.

9 Lin, T. Y., "Design of Prestressed Concrete Structures," John Wiley and Sons Inc., N. Y. 1963, p.509-p.518.

11 Preston, H. K., "Practical Prestressed Concrete," McGraw-Hill Book Company Inc., N. Y. 1960.

Design Procedure

Before designing, assume that the general conditions such as loading pattern, span length, provision for future wearing surface and the width of roadway are known. The specifications for material and the method of construction will be defined, because these vary from one locality to another.

(I). Shape of Cross Section

The cross section that has the capacity to sustain the desirable service loading with least cost and ease of erection will be selected.

The type of section which is most efficient and economical in prestressed concrete bridges for any specific structure is a function of the following;

(1) Length of span

(2) Live and impact loads which are to be carried by the bridge

(3) Allowable stresses

(4) Feasibility of various construction types and procedures as controlled by the requirements of the bridge site.

For economy in concrete and steel, the principle is that the section should have sufficient kern distance to produce the necessary internal resisting couple. The effective way is to put concrete near the extreme fibers as much as possible. There are many different kinds of sections such as I shape, T section, hollow slab, and box shape that accomplish these requirements. These have their own advantages in particular cases.

To select a suitable section, the most important factors are the span length and the live load and impact. If the span is short, the dead load moment is small, which means that the M_G/M_T ratio is small, and the center of pressure at transfer may lie below the bottom kern point. Then tensile stress may occur in the top flange and high compression stress in the bottom flange. The section of a member must provide a relatively large bottom flange to resist the prestressing force during the period when live load is not being applied. In this case, solid slab, hollow slab, and beams with large top and bottom flanges will satisfy these conditions.[8] In long spans, the dead load moment is a large percentage of the total moment, so that the M_G/M_T ratio is relatively large, the center of pressure at transfer will lie above the top kern point. Then the bottom flange is not required to resist the prestressing force during the period in which the moving live load is not applied. In this case, the T section is always used.

Another factor that should be pointed out is the simplicity of framework. In plant production more complicated shapes can be used. Only when the forms can be reused many times would the I shape and other complicated shapes be considered economical for cast-in-place beams.

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8 Libby, J. R., "Prestressed Concrete Design and Construction," Ronald Press Co., N. Y. 1961, p.286.

(II) Properties of Sections

The shape of section to used can be decided from criteria outlined in the preceding paragraphs. Then assume the dimension of the cross section or pick a section from the available data, such as Standard AASHO-PCI prestressed concrete I beams or box beams for highways. Then use these dimensions of sections to calculate the properties of the sections.

First, calculate the properties of precast beams. The basic dimension and properties which must be determined are the area, the center of gravity and the moment of inertia about the centroid of the section. The other properties which are used to facilitate the computation of stresses can be determined from these basic properties.

Second, find the basic properties of composite section: the area, the center of gravity and the moment of inertia about the centroid of composite section. The concrete in the cast-in-place deck slab does not have the same elastic properties as the precast beam. The quality of the concrete used in deck slabs is not as good as that used in precast beams. For this reason, when computing the properties of composite sections one should use a transformed section. The transformed section to be used consists of the gross section of precast beam and a slab section having a depth equal to the actual slab and the effective width equal to the actual width of slab multiplied by the ratio of the elastic modulus of slab concrete to the elastic modulus of the concrete of the precast beam.

(III) Stresses due to dead loads

The dead load shall consist of the weight of the structure complete, including girder, cast-in-place deck slab, wearing surface, sidewalk, conduit, and diaphragms. In this kind of bridge, the construction sequence is to set in place the precast beam without shoring, and then the slab concrete and diaphragm cast over midspan, and then over the piers. For this reason, all the dead loads act on the precast girder as a simple beam action. These loads act uniformly along the beam, except the weight of diaphragm which acts as a concentrated load the beam.

The maximum moments at the center of the span, M_G, M_S, and M_{Dia}, due to the girder's weight, deck slab and diaphragm respectively can be calculated. Then the total maximum moment at center of the span due to dead load, M_{DL}, is expressed as follows,

$$M_{DL} = M_G + M_S + M_{Dia} \tag{1}$$

Then use the flexure formula to find the fiber stress due to the dead load on precast girder,

$$f_{DL}^{b,t} = \frac{M_{DL}c}{I_p} \tag{2}$$

where, $f_{DL}^{b,t}$ = Extreme fiber stresses

M_{DL} = The maximum bending moment due to dead load

c = The distance from neutral axis to the extreme fiber of precast girder,

I_p = The moment of inertia of the precast girder about its neutral axis

(IV) Stresses due to live load

The live load should consist of applied moving loads including vehicles, cars and pedestrians. From the general condition, the loading pattern acting on the bridge is known, In AASHO Sec. 1. 2. 5(A), there are two types of loading specified, the H loadings and HS loadings.

To aid in finding the maximum positive and negative moments due to live load, influence lines for them should be constructed. The writer assumes the reader is familiar with the technique of drawing influence lines and using them. In continuous girders, finding maximum moments at several critical points is necessary. These points are at the interior supports, the center of exterior spans. Therefore, the influence line for moment at these points should be completed. In some cases, the influence line for reactions at exterior supports and interior supports are needed.

After the influence lines are completed, determine the live load to be distributed on the girders from the AASHO Specification in Sec. 3, Division 1. Then according to the given loading pattern, the maximum positive and negative moments at these specified points can be computed by the arrangement of the loading condition stated in Sec. 1. 2. 8(C) AASHO.

In addition to the moment due to moving loads, the effects of dynamic loading, vibration and impact should be taken into consideration. In AASHO Specification, Sec. 1. 2. 12(C), the amount of this allowance or increment is expressed in the percentage of live load moment, and should be determined by the formula;

$$I = \frac{50}{L + 125} \tag{3}$$

where I = Impact factor, (maximum 30 percent)

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

Then add the moment due to moving load and the effect of impact to get the maximum live load moment at specified sections. That moment can be expressed as follows;

$$M_{LL} = M_{ML} (1 + I) \tag{4}$$

Under live load, the composite section composed of precast beam and deck slab acts as a continuous member. Therefore, the I_p in the equation (2) should become I_c, which is the moment of inertia of the composite section about the neutral axis of the transformed section. Then the stresses can be calculated by the following formula;

$$f_{LL}^{b,t} = \frac{M_{LL}c}{I_c} \tag{5}$$

where $f_{LL}^{b,t}$ = The extreme fiber stresses

M_{LL} = The maximum bending moment due to live load and impact

I_c = The moment of inertia of the composite girder about its neutral axis

(V) Designing the Prestressing Steel

In preliminary design of prestressed concrete sections for flexure is based on a knowledge of the internal C-T couple acting in the section. Then the required total effective prestress force F can be computed as following;[9]

$$F = \frac{M_T}{0.65\ h} \qquad (6)$$

where M_T = The total moment acts on the section

h = The depth of the section

The required prestressing force is governed by two controlling values of external moment; the total moment M_T acting on the section, which controls the stresses under the action of the working loads; and the girder load moment M_G, which determine the location of the c.g.s. and the stresses at transfer. Therefore, design the prestress force to limit the fiber stresses of the section to be within the allowable at transfer and under working load.

The basic concept of designing the prestressing force is same as in noncomposite section. One additional concept introduced for composite sections is the reduction of the moments on the composite section to equivalent moments on the precast portion. This is accomplished by the use of the ratio of the section moduli of two sections. Then the required prestressing force can be determined by the following steps:[9]

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9 Lin, T. Y., "Design of Prestressed Concrete Structures," John Wiley and Sons Inc., N. Y. 1963, p161 and p.185-p.187.

Step 1. Location of the center of gravity of reinforcing steel. For any given trial section, the c.g.s. should be located so that the precast girder will not be overstressed and has the optimum capacity in resisting the external moment. Therefore, the c.g.s. must be located as low as possible but not lower than given by the following value of eccentricity, Fig. (1),

$$e_o = k_b + e_1 + e_2 \tag{7}$$

where e_o = The distance between c.g.c. of precast girder and the c.g.s.

$k_b = \dfrac{I_p}{A_p c_t}$, the lower kern distance

$e_1 = \dfrac{f_t I_p}{c_t F_o}$, the eccentricity under the allowable tension stress on top fiber of precast girder at transfer

$e_2 = \dfrac{M_G}{F_o}$, the eccentricity to resist optimum external moment

f'_t = allowable tension on top fiber of precast girder at transfer

A_p = area of precast girder

c_t, c_b = distance to top or bottom fiber from c.g.c. of precast girder

$F_o = \dfrac{f_i}{f_e} F$, total prestressing force just after transfer

f_i = initial unit prestress in steel before transfer

f_e = effective unit prestress in steel after deducting losses

F = total effective prestress after deducting losses, the value can be from Eq. (6).

Another way to locate the center of gravity of prestressing steel is to assume the eccentricity of prestress and then check it. This procedure can reduce some calculations.

Step 2. Compute the equivalent moment on the precast girder. There is a moment M_c acting on the composite section which will produce stresses on the precast girder as shown in Fig. (2) and computed as follows:

$$f^t = \frac{M_c\ c_t''}{I_c}$$

$$f^b = \frac{M_c\ c_b''}{I_c}$$

where c_b'', c_t'' are distances to extreme fibers of the precast girder measured from c.g.c.' of the composite section.

Let $m_t = \dfrac{I_p/c_t}{I_c/c_t''}$, and $m_b = \dfrac{I_p/c_b}{I_c/c_b''}$

Then $$f^t = \frac{m_t M_c d_t}{I_p} = \frac{m_t M_c}{A_p k_b} \qquad (8a)$$

$$f^b = \frac{m_b M_c c_b}{I_p} = \frac{m_t M_c}{A_p k_t} \qquad (8b)$$

where k_t top kern distance of precast girder

From Eq. (8a) and (8b) M_c can be modified by m_t and m_b, so that it can be reduced to equivalent moments for computation based on the precast girder properties.

Step 3. Compute the amount of prestress required for moments as follows. If M_p = the total moment acting the precast girder, and f_b' = allowable tensile stress at bottom fiber, so that;

$$f_b' = -\frac{F}{A_p}\left(1 + \frac{e_o}{k_t}\right) + \frac{M_p}{A_p k_t} + \frac{m_b M_c}{A_p k_t}$$

Then, $$F = \frac{M_p + m_b M_c + f_b' k_t A_p}{e_o + k_t} \qquad (9a)$$

If $f_b' = 0$,

then $$F = \frac{M_p + m_b M_c}{e_o + k_t} \qquad (9b)$$

from which the required total prestress force F_o can be computed. If necessary, use this new value of F_o to revise the location of c.g.s.

Step 4. Compute the required prestressing steel area by the following formula;

$$A_s = \frac{F_o}{f_e} \qquad (10)$$

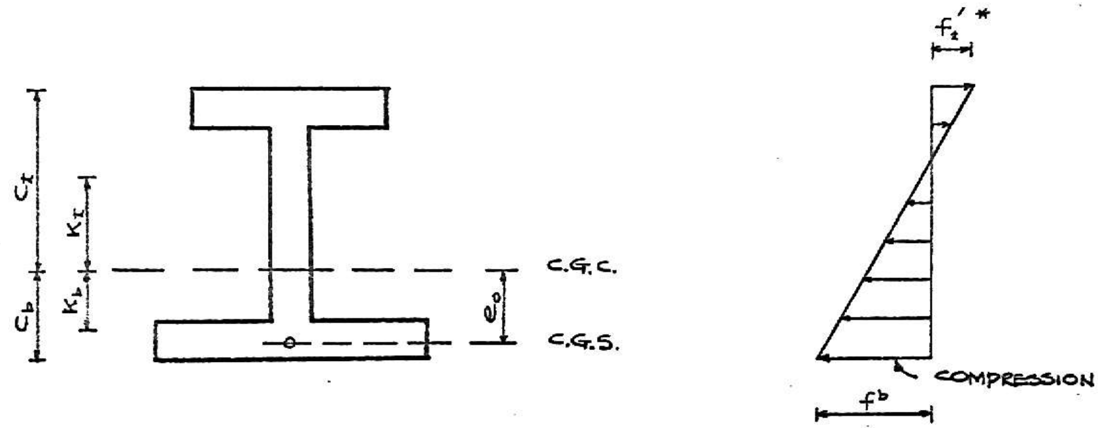
Then the number of strands to be used can be selected.

Step 5. Check the top and bottom fiber stresses at the center of the span at transfer, then

$$f^t = \frac{F_o}{A_p} - \frac{(F_o e_o - M_G)}{A_p\, k_b} \qquad (11a)$$

$$f^b = \frac{F_o}{A_p} + \frac{(F_o e_o - M_G)}{A_p\, k_t} \qquad (11b)$$

Step 6. Check the top and bottom fiber stresses at the



* f_b', f_t' allowable tensile stress in bottom fiber stress under working load and top fiber stress at transfer respectively

FIG. 1. PRECAST GIRDER, STRESS DISTRIBUTION AT TRANSFER

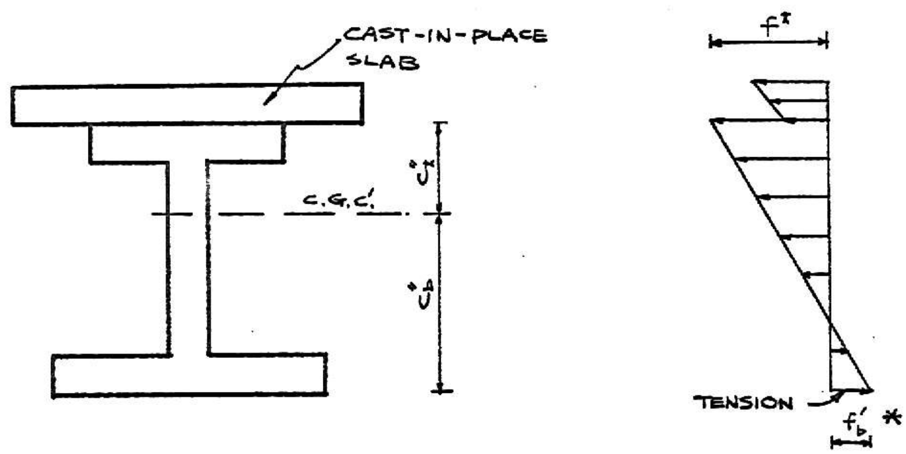


FIG. 2. COMPOSITE SECTION, STRESS DISTRIBUTION UNDER WORKING LOAD

center of the span under working load, since

$$f^t = \frac{F}{A_p} + \frac{M_p + m_t M_c - Fe_o}{A_p k_b} \tag{12a}$$

$$f^b = \frac{F}{A_p} - \frac{M_p + m_b M_c - Fe_o}{A_p k_t} \tag{12b}$$

(VI) Positive moment over piers due to creep and differential shrinkage

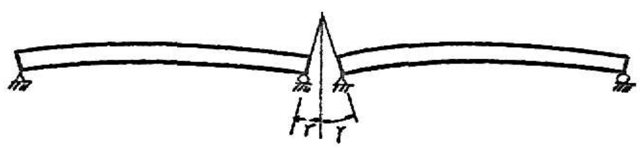
One of the unique features of the type of bridge under consideration is that positive moments are produced over piers due to creep in the precast prestressed girders as well as due to the effects of live loads in remote spans. These positive moments will be partially counteracted by negative moments resulting from differential shrinkage between the cast-in-place deck slab and the precast girders. The positive live load and impact moment over piers due to remote span can be calculated from previous steps. It is very difficult to measure these two effects, creep and shrinkage, separately in a structure, because the result measured is always the resultant of these two influences. For convenience in analysis, these two effects are considered separately.

(1) The effect of creep

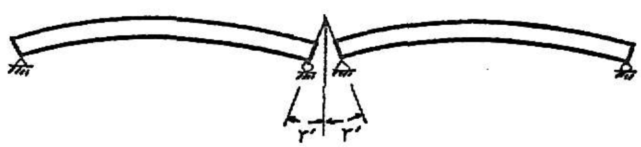
Since the creep is time dependant, with the more rapid deformation occuring during the early stages of loading, the amount of positive restraint moment produced by the prestressing force depends on the time when the continuity connection is made.

It also depends on the inherent creep potential of concrete mix, the exposure conditions, and the volume to surface ratio of the precast girder. The deformation and restraint moments induced in a two span continuous girder by creep due to prestressing force are illustrated in Fig. (3).

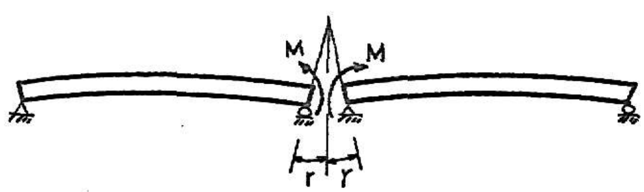
The most accurate method of obtaining creep data is by extrapolation from a number of test samples prepared in advance from the actual mix to be used in the precast girders. In most



(a). Initial deformation



(b). Final deformation if left as two simple spans.



(c). Final deformation and restraint moment if spans are made continuous after prestressing

FIG. 3. DEFORMATIONS AND RESTRAINT MOMENTS IN A TWO-SPAN CONTINUOUS BEAM CAUSED UNDER PRESTRESS FORCE

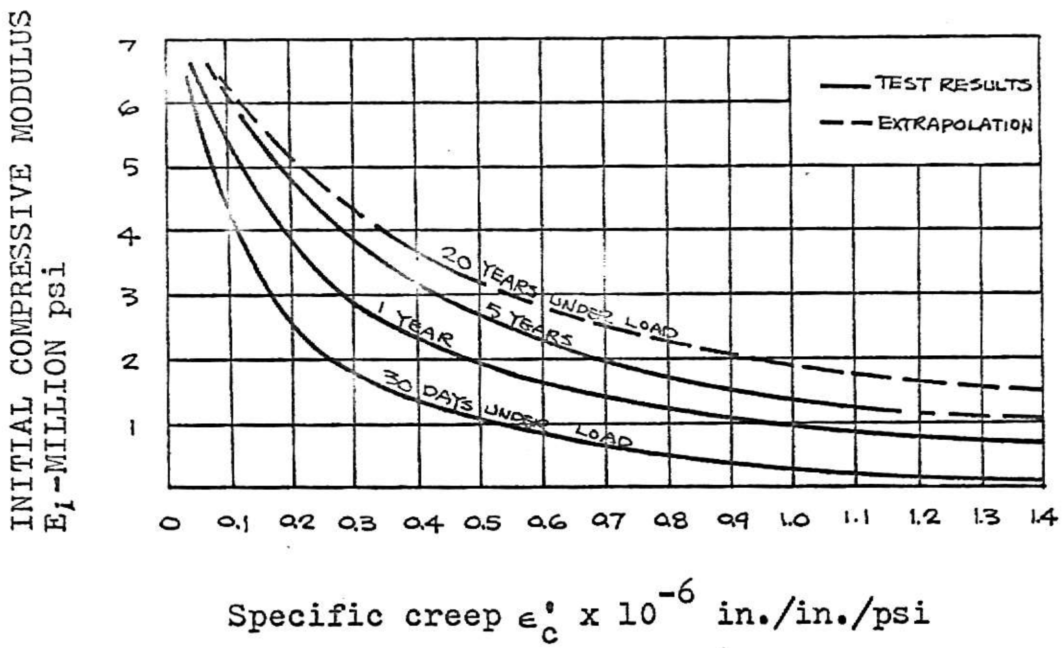


FIG. 4. PREDICTION OF BASIC CREEP FROM ELASTIC MODULUS

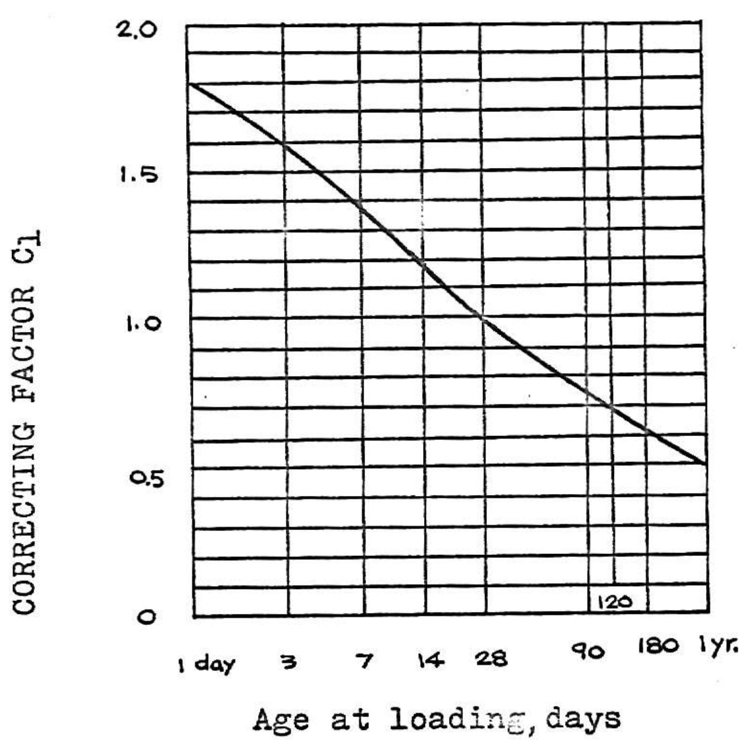


FIG. 5. CREEP VS. AGE AT LOADING

design work, it is sufficient to rely on the available research information. Such research[1,6] indicates that the basic creep value for loading at 28 days can be predicted from the elastic modulus from the curve in Fig. (4). For design purpose, the 20 year creep curve can be regarded as the ultimate creep. The ultimate creep value for loading at 28 days from Fig. (4) must be adjusted to account for the age when the girders are prestressed and for the volume/surface ratio of the girders. The correcting factor of creep due to the age at loading can be obtained from the curve in Fig. (5)[16], and the correcting factor of creep due to volume/surface ratio is shown in Fig. (6)[5]. Then ultimate creep can be determined as follows;

(a) Get the basic creep ϵ_c for loading at 28 days from Fig. (4).

(b) Find the correcting factor C_1 due to age at loading

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1 Fintel, Mark and Khan, Fazlur R., "Effect of Column Creep and Shrinkage in Tall Structures-Prediction of Inelastic Column Shortening," Portland Cement Association, Old Orchard Road, Skokie, Ill. 60076.

5 Hansen, T. C. and Mattock, A. H., "Influence of Size and Shape of Member on the Shrinkage and Creep of Concrete," Journal of the American Concrete Institute, Vol. 63, Feb. 1966. p.267-p.290.

6 Hickey, K. B., "Creep of Concrete Predicted from Elastic Modulus Tests," Report No. C-1242, United States Department of Interior, Bureau of Reclamation, Denver, Colo., Jan. 1968.

16 "Recommendations for International Code of Practice for Reinforced Concrete," published by the American Concrete Institute and the Cement and Concrete Association.

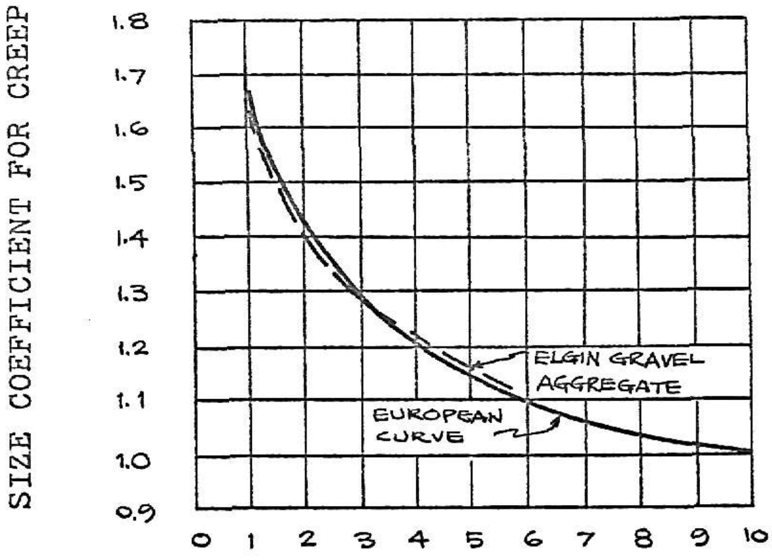

FIG. 6. CREEP VS. VOLUME-TO-SURFACE RATIO

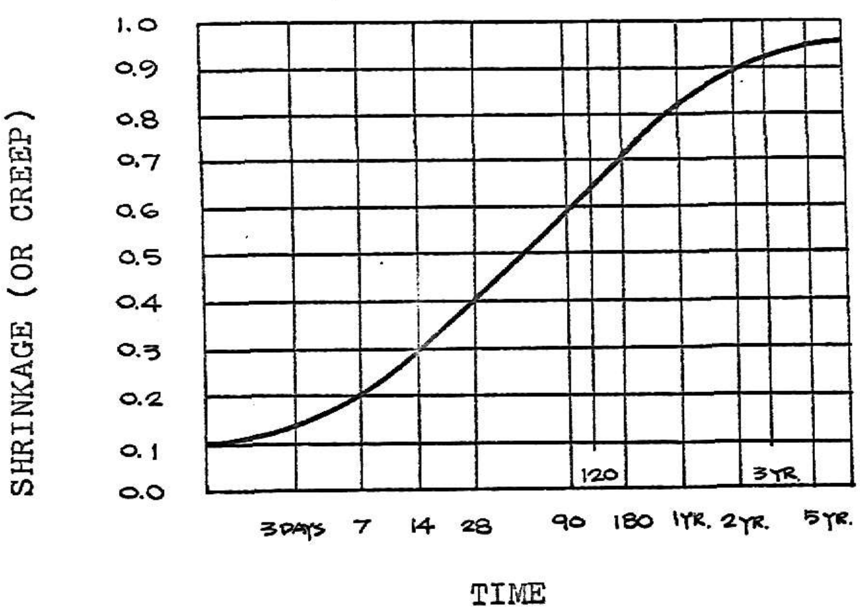

FIG. 7. PROPORTION OF FINAL SHRINKAGE
OR CREEP VS. TIME

from Fig. (5), and the correcting factor C_2 due to volume/surface ratio from Fig. (6).

(c) Then the ultimate creep is $\epsilon_c \times C_1 \times C_2$.

Then determine the percentage of ultimate creep which will occur after the continuity connection is made. This value can be obtained from Fig. (7)[3]. For example, if a connection were made at 28 days after casting the precast girder, about 40 percent of creep strains will have taken place. This can be found from Fig. (7). The remaining 60 percent is available to develop moments in the connection. These restraint moments due to creep under prestress and dead load, M_{rp} and M_{rdl} respectively, act on the continuous girders as shown in Fig. (3). These moments over each pier can be solved by any method of analysis of statically indeterminant structures, and the analyses are illustrated in Appendix IV. Therefore, the final restraint moment M_{RC} caused by the effect of creep on the precast girder after the creation of continuity will be expressed as follows;

$$M_{RC} = (M_{rp} + M_{rdl})\ (1 - e^{-\phi}) \tag{13}$$

where M_{RC} = The final restraint moment over piers due to the effect of creep under prestress and dead load

M_{rp} = The restraint moment due to the effect of creep under prestress

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3 Hansen, J. A., "Prestress Loss as Affected by Type of Curing," Portland Cement Association Development Department, Bulletin D75.

M_{rdl} = The restraint moment due to the effect of creep under dead load

e = The base of Naperian logarithms, 2.7183

ϕ = $\frac{\epsilon_c}{\epsilon}$, the ratio of creep strain per psi of stress to the elastic strain per psi of stress

If the value of ϕ is known, the value of $(1 - e^{-\phi})$ can be taken directly from Fig. (8).

(2) The effect of shrinkage

Shrinkage of the deck slab with respect to the precast girders causes negative restraint moments over piers that will reduce the creep restraint moment. The restraint moments due to differential shrinkage, m_{rs}, are applied to the continuous girder uniformly along its length, and can be expressed as follows;[11]

$$m_{rs} = \epsilon_s E_b A_b \, (e_2' + t_b/2) \tag{14}$$

where ϵ_s = differential shrinkage strain (as described below)

E_b = elastic modulus of the deck slab concrete

A_b = cross-section area of deck slab

$(e_2' + t_b/2)$ = distance between the mid-depth of the slab and the centroid of composite section

t_b = slab thickness

11 Mattock, A. H., "Precast-Prestressed Concrete Bridges 5. Creep and Shrinkage Studies," Portland Cement Association Development Department, Bulletin D46.

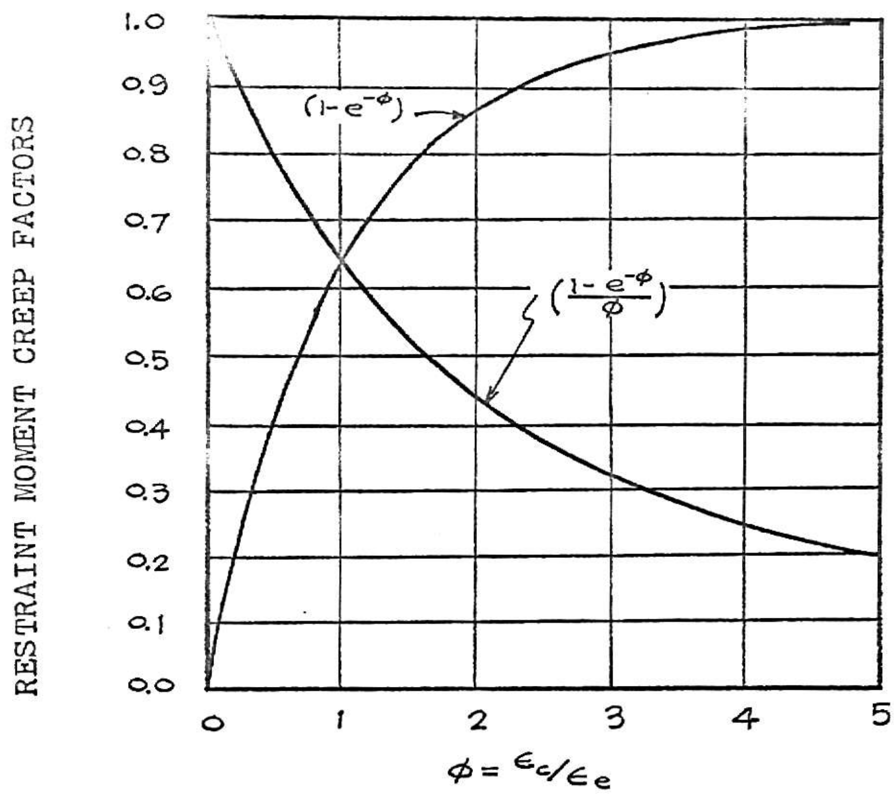

FIG. 8. VARIATIONS IN RESTRAINT MOMENT CREEP EFFECT FACTORS FOR BOTH CREEP AND SHRINKAGE

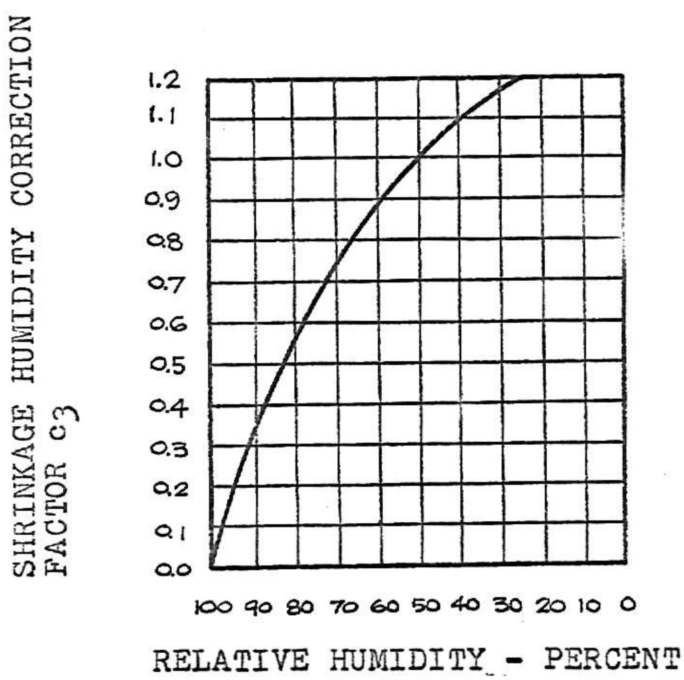

FIG. 9. SHRINKAGE HUMIDITY CORRECTION FACOTR

$$\epsilon_s = \frac{\epsilon_{su} \ T^{16}}{N_s + T} \tag{15}$$

ϵ_s = measured shrinkage strain at any time T in days

ϵ_{su} = ultimate shrinkage strain at T = infinity

N_s = $26.0 \ e^{0.36V/S}$

V/S = the volume / surface ratio of the member under consideration

In general, the way to get the value of ϵ_s in Eq. (14) is from equation 15. By testing specimens which were made from the mix to be used in the structure the final value of shrinkage can be projected by assuming that the shrinkage-time relationship can be represented by Eq. (15). If the test data are not available, then ϵ_s can be obtained by the following steps;

(a) Assume the ultimate shrinkage $\epsilon_{s(ult)}$ under exposure at 50 percent relative humidity is 0.6×10^{-3}.[3, 13]

(b) Find the shrinkage humidity correction C_3 from Fig.(9).

(c) Find the correction factor C_4 for the differential shrinkage between slab and girder for a given time lapse between the castings from Fig. (7).

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3 Hanson, J. A., "Prestress Loss as Affected by Type of Curing," Portland Cement Association Development Department, Bulletin D75.

13 "Design and Control of Concrete Mixtures," Portland Cement Association, Old Orchard Road, Skokie, Ill. 60076.

16 "Recommendations for International Code of Practice for Reinforced Concrete," Published by the American Concrete Institute and the Cement and Concrete Association.

(d) Then the value of ϵ_s is $C_3 \times C_4 \times \epsilon_{s(ult)}$, and can be substitute into Eq. (14) to calculate m_{rs}.

The restraint moments m_{rs} due to differential shrinkage are known. Then any convenient method for the analysis of statically indeterminant structures can be applied to find the restraint moments. M_{RS} can be computed by multiplying by the factor $(1 - e^{-\phi})/\phi$ for the effect of creep as given by the following,

$$M_{RS} = m_{rs}(1 - e^{-\phi})/\phi \tag{16}$$

Then the total positive moments over piers due to creep and shrinkage are given as follows;

$$M_R = M_{RC} + M_{RS} \tag{17}$$

(VII) Design positive moment connection

As determined in the P.C.A. Laboratories,[11] the positive moment may develop at the interior support sections of continuous girders. If a positive moment continuity connection is not provided, cracking may occur at the bottom of the diaphragm at these sections. However, the cracking, if any, will affect only the behavior of the girder at service load level. The positive restraint moment due to creep and shrinkage will not affect the ultimate load carrying capacity of the girders. For this reason, it is recommended that design be based on the lower

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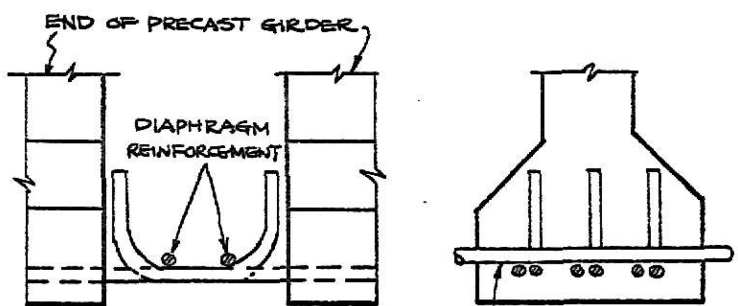
11 Mattock, A. H., "Precast-Prestressed Concrete Bridges 5, Creep and Shrinkage Studies," Portland Cement Association Development Department, Bulletin D46.

stress of 0.6 of the yield strength.

In the P.C.A. Development Department bulletin,[10] there are two kinds of connection details to resist the positive restraint moment. The detail considered more practical is shown in Fig. (10).

When designing the connections, there are serveral details that should be considered. First, the length of embedment required for hooked connection bars. This can be calculated by considering the bond to develop uniformly from the face of the beam around the bend to the end of the bar.[4] In Sec. 1. 5. 1.(D), AASHO, the allowable bond stress is 0.1 f'_c (Max. 350 psi).



Reinforcing bars with hook ends embedded in ends of precast girders

Diaphram reinforcement

ELEVATION

SECTION

FIG. 10. POSITIVE RESTRAINT MOMENT CONNECTION DETAILS

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4 Hanson, N. W. and Connor, Harold W., "Seismic Resistance of Reinforced Concrete Beam-Column Joints," Portland Cement Association Development Department, Bulletin D121.

10 Mattock, A. H., "Precast-Prestressed Concrete Bridges 3, Further Tests of Continuous Girders," Journal of the PCA Research and Development Laboratories, Vol. 2, No. 3, Sept. 1960.

Second, the distance from the inside face of precast girder to the inside face of the hook should be at least 12 times the bar diameter.

Third, the removal of the end girder form after prestressing should be considered. It may be required that the connection bars be cast in the beam straight and then field bent after removal of the end form.

Then the required steel area of the connection can be calculated by the ordinary reinforced concrete formula;

$$A_s' = \frac{M_R}{f_s' \, jd} \tag{18}$$

where A_s' = area of conventional tensile steel

f_s' = allowable tensile stress in conventional steel

j = ratio of distance between controid of compression and centroid of tension to the depth d

d = distance from extreme compressive fiber to centroid of tensile force

(VIII) Design Negative Moment Reinforcement

Since most of the dead loads are carried by the precast beam acting as a simple beam, the negative moments over piers due to live load plus impact are sustained by the composite section. Although this negative moment is computed by the elastic theory, the reinforcement can be designed to meet the ultimate strength requirements with the specified minimum load factors, which are 2.5 for live load and impact and 1.5 for dead load. The reason is that the effect of initial precompression due to prestress in the precast girders can be neglected in the negative moment calculation of ultimate strength if the maximum precompression stress is less than $0.4f'_c$ and the continuity reinforcement is less than 1.5 percent.[7]

It will be found that the depth of compression block will be less than the thickness of bottom flange of the precast girder. For this reason, the required negative moment reinforcement can be determined by assuming the beam as a rectangular section with a width equal to the width of the bottom flange of the girder. Therefore, the flexure formula for ultimate strength design from Sec. 1601 ACI Building Code 1963 may be used to compute the required steel area as follows;

$$M_u = \alpha[bd^2 f'_c q\,(1 - 0.59q)] = \alpha A'_s f'_y \left(d - \frac{a}{2}\right) \qquad (19)$$

where M_u = ultimate resisting moment

α = capacity reduction factor, equals 0.9

b = width of compression face of flexure member

f'_c = compressive strength of concrete at 28 days

$$q = \frac{p'f'_y}{f'_c}$$

$$p' = \frac{A_s}{bd}$$

$$a = \frac{A'_s f'_y}{0.85 f'_c b}$$

7 Kaar, P. H., Kriz, L. B. and Hognestad, E., "Precast-Prestressed Concrete Bridges 1. Pilot Test of Continuous Beams," Portland Cement Association Development Department, Bulletin D34.

The reinforcement ratio, p', shall not exceed 0.75 of the ratio, p_b, which produces balanced conditions at ultimate strength given by;

$$p_b = \frac{0.85 k_1 f'_c}{f'_y} \quad \frac{87{,}000}{87{,}000 + f'_y} \tag{20}$$

where k_1 = a factor which shall be taken as 0.85 for strength f'_c, up to 4000 psi and shall be reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi.

(IX) Ultimate Strength

After completing the girder flexural design, the ultimate flexural capacity of the girder should be computed at the critical sections. First, consider the section of midspan of the girder. From Sec. 1. 6. 10. AASHO, if the neutral axis lies within the flange, then the ultimate flexural strength will be assumed as;

$$M'_u = A_s f_{su} d \left(1 - 0.6 \frac{p f_{su}}{f'_c}\right) \tag{21}$$

where A_s = area of main prestressing tensile steel

f_{su} = average stress in prestressing steel at ultimate load

$p = \frac{A_s}{bd}$, ratio of prestressing steel

M'_u = ultimate flexural strength

If the neutral axis falls outside flange, then the ultimate strength will be assumed as;

$$M_u' = A_{sr}f_{su}d\left(1 - 0.6\,\frac{A_{sr}f_{su}}{b'df_c'}\right) + 0.85f_c'(b-b')t(d-0.5t) \qquad (22)$$

where $A_{sr} = A_s - A_{sf}$, the steel area required to develop the ultimate compressive strength of the web of a flange section

$A_{sf} = 0.85f_c'\ (b-b')\ t/f_{su}$, steel area required to develop the ultimate compressive of the overhanging portion of the flange

t = the thickness of the flange

b = width of flange of flanged member or width of rectangular member

b' = width of web of a flanged member

Second, consider the sections over piers. Since these sections are designed by the reinforced concrete theory, Eq. (19) can be used without the capacity reduction factor, α, to calculate the ultimate flexural strength at these section. This moment can be expressed as;

$$M_u' = bd^2f_c'q\ (1-0.59q) = A_s'f_y'\left(d- \frac{a}{2}\right) \qquad (23)$$

(X) Compute the reaction over piers and design shear steel

Since the precast girders are acting as simple beams under dead load, then the reaction caused by dead load can be found easily by statics. The computation of reactions due to live load and impact that can make use of influence lines by arranging the load condition to get the maximum reaction over piers. The ultimate shear will be governed by the following formula by using the load factor from AASHO Sec. 1. 6. 6.;

$$V_{ult} = 1.5\ V_{DL} + 2.5\ V_{LL} \tag{24}$$

where V_{ult} = ultimate shear due to dead load and live load including impact

V_{DL} = shear due to dead load

V_{LL} = shear due to live load and impact

Then the effective ultimate shear is the ultimate shear minus the shear carried by the strand at the end, which can be expressed as follows;

$$V_u = V_{ult} - V_s \tag{25}$$

where V_u = effective shear due to specified ultimate load and effect of prestressing

V_s = shear carried by the strands

Then the required web reinforcement can be calculated by the formula from Sec. 1. 6. 13, AASHO.

$$A_v = \frac{(v_u - V_c)s}{2f_y'\ jd} \tag{26}$$

where A_v = area of web reinforcement at spacing s, placed perpendicular to the axis of the member, which shall not be less than 0.0025b's

V_c = 0.06 $f_c'b'jd$, but not more than 180b'jd (assuming j = 7/8)

s = longitudinal spacing of web reinforcement

The spacing of web reinforcement shall not exceed three-fourths the depth of the member and shall provide transverse reinforcement across the bottom flanges.

(XI) Compute the camber or deflection

From Sec. 202.2 of Tentative Recommendations for Prestressed Concrete,[14] camber and deflection may be design limitations and should be investigated for both short and long time effects. In fact, it is very difficult to compute a precise deflection or camber, because there are two difficulties that cannot be overcome. One is the determination of the value of E_c, flexural modulus of elasticity of concrete, the other is the estimation of the effect of creep on deflections.

According to the Sec. 203.3 in Tentative Recommendations for Prestressed Concrete, for the deflection or camber under short time loading the value of E_c can be obtained by assuming E_c is 1,800,000 plus 500 times the cylinder strength at the age considered. Also, for deflection associated with dead load, prestress and live loads sustained for a long time may be computed on the assumption that the corresponding concrete strains are increased as a result of creep.

By following the above-referenced specification, the deflection or camber can be computed by the conjugate-beam method at various loading stages. First, the deflections due to girder's weight, then the camber due to prestressing force, and then the

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14 ACI-ASCE Joint Committee 323, "Tentative Recommendations for Prestressed Concrete," Journal of the American Concrete Institute, Vol.29, Jan. 1958.

additional deflection due to the weight of slab and diaphragm and finally the live load and impact. The final camber equals the summation of these deflections.

DESIGN EXAMPLE

In this design example, the interior girder of a four-span continuous precast-prestressed concrete bridge will be considered. Before designing the general information of the bridge and design data will be introduced as follows;

a. The cross section and elevation of the bridge are as shown in Fig. (11) and Fig. (12) respectively.

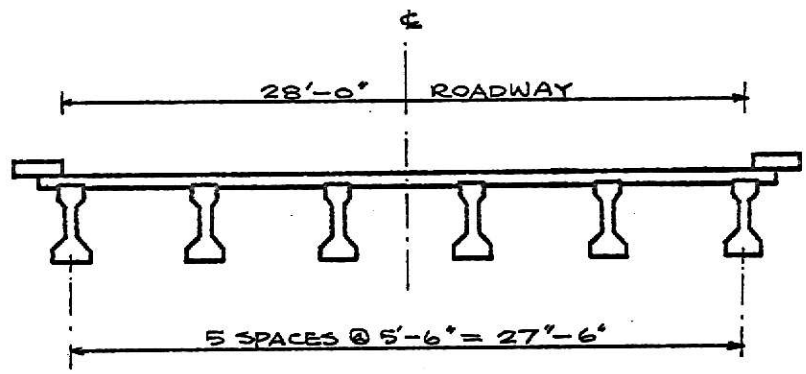


FIG. 11. CROSS SECTION OF BRIDGE

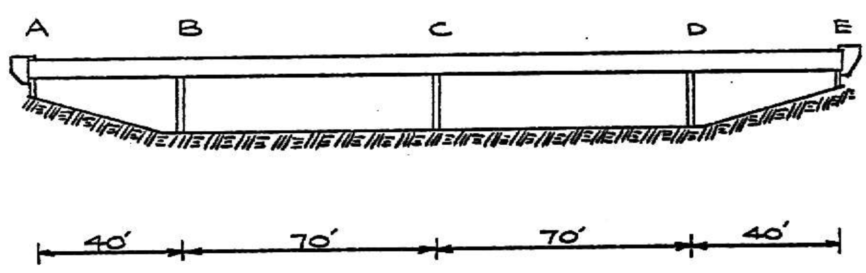


FIG. 12. ELEVATION OF BRIDGE

b. Loading;

Live load : AASHO Loading H20-40

Weight of concrete : 150 lb/cu.ft.

Wearing surface : 2 in. thickness of concrete

c. Specifications;

The structural design is essentially based AASHO Specifications for Highway Bridges, 1969, 10th edition, and use several portions of ACI-ASCE Joint Committee 323 recommendations for Prestressed Concrete, 1958, and ACI Building Code Requirement for Reinforced Concrete (ACI 318-63).

d. Prestressing steel;

Use 7/16 in. 7-wire strands, with $A_s = 0.1089$ in.^{2} per strand, $f_i = 175{,}000$ psi, $f_e = 140{,}000$ psi, $f_s = 250{,}000$ psi.

e. Reinforcing steel; intermediate grade $f_y' = 40{,}000$ psi.

f. Precast concrete; $f'_{ci} = 5{,}000$ psi, $f_c' = 6{,}000$ psi.

g. In-place slab concrete; $f_c' = 4{,}000$ psi.

All the design procedures will be follow the procedures described before, and will illustrate every step in the follows;

(I). Shape of cross section;

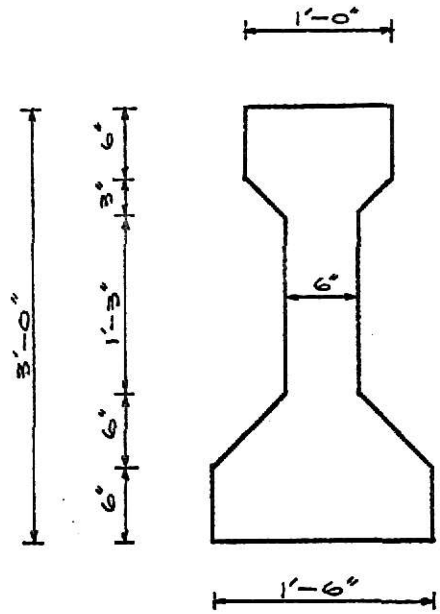
In this example, the type II of I beam of AASHO-PCI Standard will be used. The thickness of slab is assumed 6.5 in. and the dimensions of the section are shown in Fig. (13).

(II). Properties of section

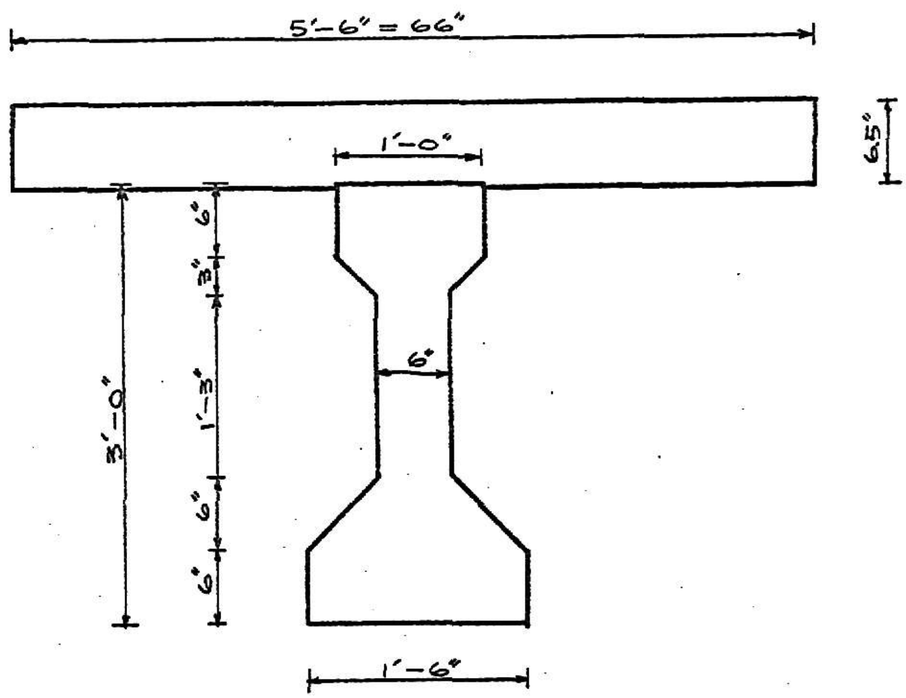
Precast section $A_p = 369$ in.^{2}

$c_b = 15.83$ in., $c_t = 20.17$ in.

$I_p = 50{,}980$ in.^{4}



a. SECTION OF PRECAST GIRDER



b. SECTION OF COMPOSITE SECTION

FIG. 13. CROSS SECTION OF GIRDER

Then the sectional moduli are;

$Z_t = 50{,}980/20.17 = 2{,}530$ in.^{3}

$Z_b = 50{,}980/15.83 = 3{,}220$ in.^{3}

weight per unit foot = 369x150/144 = 384 lb/ft

Composite section

Because the modulus of elasticity in precast girder and slab are different, the modulus of elasticity for precast and slab should be computed separately according to Sec. 203.2 Tentative Recommendations for Prestressed Concrete as follows;

For slab $E_b = 1{,}800{,}000 + 500\text{x}4{,}000 = 3{,}800{,}000$ psi

For precast girder

$$E_c = 1{,}800{,}000 + 500 \times 6{,}000 = 4{,}800{,}000 \text{ psi}$$

Ratio = 3,800,000/4,800,000 = 0.792

Then taking moments about the axis through the top fiber of the slab, to find the section properties;

Section	A	y	Ay	Ay^2	I_o
66 x 6.5 x 0.792	339	3.25	1,101.	3,580.	1,195.
precast girder	369	26.67	9,850.	263,580	50,980.
	708		10,951.		52,175.

$I = 266{,}580 - 52{,}175 = 318{,}755$ in.^{4}

$c_t'' = 10{,}953/707 = 15.48$ in. $c_b'' = 42.5 - 15.48 = 27.02$ in.

$I_c = 318{,}755 - 708(15.48)^2 = 149{,}155$ in.^{4}

Then the sectional moduli are;

Z_t = 149,155/15.48 = 9,650 in.^{3}

Z_b = 149,155/27.02 = 5,745 in.^{3}

(III) Stresses due to dead loads;

Compute the moments at center of the span due to girder's weight,

Ext. span M_G = 1/8x384x(40)^2 = 86,500 ft-lb

Int. span M_G = 1/8x384x(70)^2 = 235,200 ft-lb

Compute the moment at center of the span due to the weight of slab and wearing surface,

weight of slab and wearing surface is;

(6.5 + 2) x 66 x 150/144 = 584.3 lb/ft

Ext. span M_s = 1/8 x 584.3 x (40)^2 = 116,900 ft-lb

Int. span M_s = 1/8 x 584.3 x (70)^2 = 358,000 ft-lb

From the AASHO-PCI Standards the diaphragms are at the center of the span. The dimensions of diaphragms are 8 in x 2 ft-6 in x 5 ft-6 in, therefore the weight of the diaphragm is

2.5 x 5.5 x 2/3 x 150 = 1,375 lb

Computing the moments at the center of the span,

Ext. span $M_{Dia.}$ = 1/4 x 1,375 x 40 = 1,375 ft-lb

Int. span $M_{Dia.}$ = 1/4 x 1,375 x 70 = 24,100 ft-lb

Then the total moments and stresses at the center of the span of the precast girder due to dead load as follows;

Ext. span M_{DL} = 86,500 + 116,900 + 13,750 = 217,150 ft-lb

$$f^b_{DL} = 217,150 \times 12 \,/\, 3220 = 809 \text{ psi}$$

$$f^t_{DL} = 217,150 \times 12 \,/\, 2,530 = 1,030 \text{ psi}$$

Int. span $M_{DL} = 235,200 + 358,000 + 24,100 = 617,100$ ft-lb

$$f^b_{DL} = 617,100 \times 12 \,/\, 3,220 = 2,300 \text{ psi}$$

$$f^t_{DL} = 617,100 \times 12 \,/\, 2,530 = 2,925 \text{ psi}$$

(IV). Moments due to live load

From Sec. 1. 3. 1. (B) AASHO, the distribution factor of live load is S/5.5 ,

$$D.F. = 5.5 \,/ (5.5 \times 2) = 0.5$$

Then compute the impact factor by Eq. (3),

Ext. span; +M, I = 50/125+40 = 0.303, use 0.30

-M, I = 50/125+55.5 = 0.27

Int. span +M, I = 50/125+70 = 0.256

-M, I = 50/125+70 = 0.256

By using the influence lines, the maximum live load moments per lane are as follows;

Ext. span,

+M near midspan

$M = 99.856966 \times 2 \times 0.64 + 8.505965 \times 18 = 281$ ft-k

-M over pier

$M = -198.334 \times 2 \times 0.64 - 3.0597802 \times 18 - 7.1451 \times 18$

$= -437.5$ ft-k

Int. span

+M near midspan

$M = 170.0631 \times 2 \times 0.64 + 11.02275 \times 18 = 415.8$ ft-k

-M over pier

$M = -248.1423 \times 2 \times 0.64 - 5.793148 \times 18 \times 2 = -526.5$ ft-k

Total moments due to live load and impact are as follows;

Ext. span $+M_{LL} = 281 \times 0.5 \times 1.30 = 182.5$ ft-k

$-M_{LL} = -437.5 \times 0.5 \times 1.27 = -278$ ft-k

Int. span $+M_{LL} = 415.8 \times 0.5 \times 1.256 = 261$ ft-k

$-M_{LL} = -526.5 \times 0.5 \times 1.256 = -331$ ft-k

(V) Design the prestressing steel

Ext. span, assuming the c.g.s. is 4 in. above the bottom fiber of precast girder, and from the previous steps, the following data can be computed;

$M_p = 217.15$ ft-k, $M_G = 86.5$ ft-k, $M_c = 182.5$ ft-k

$$m_t = \frac{50980}{20.17} \Big/ \frac{149155}{15.48} = 0.263$$

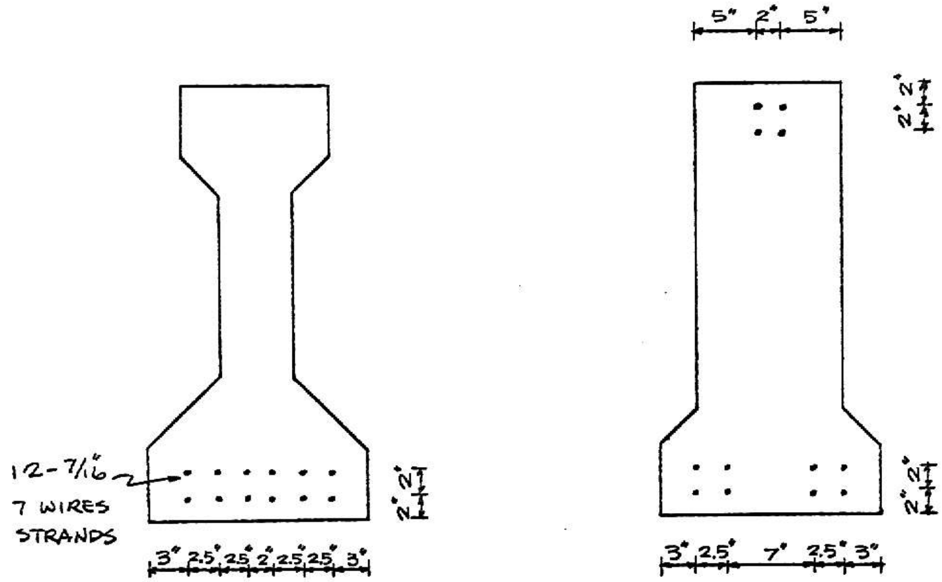
$$m_b = \frac{50980}{15.83} \Big/ \frac{149155}{27.02} = 0.583$$

$$k_b = \frac{50980}{369 \times 20.17} = 6.86 \text{ in.}$$

$$k_t = \frac{50980}{369 \times 15.83} = 8.72 \text{ in.}$$

Using Eq. (9b), to find F.

$$F = \frac{(217.15 - 0.583 \times 182.5) \times 12}{11.83 + 8.72} = 189 \text{ k}$$



MIDSPAN END

FIG. 14. ARRANGEMENT OF RPESTRESSING STEEL OF SPAN AB

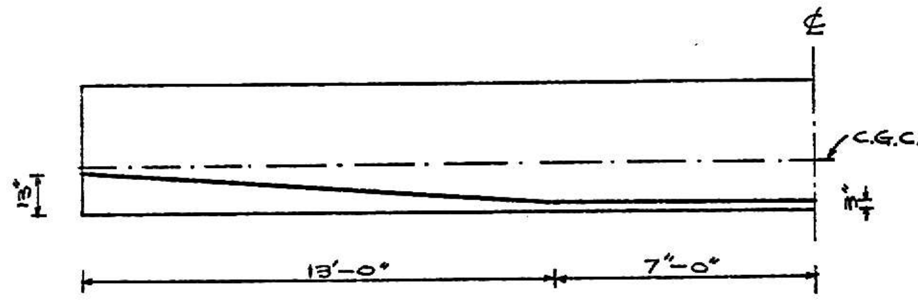


FIG. 15. PATH OF C.G.S. OF SPAN AB

$F_o = 175/140 \times 189 = 236.3$ k

$A_s = 189/140 = 1.35$ in.^{2} Min. steel $0.003 \times 369 = 1.107$ in.^{2}

Use 12 strands

The arrangement of the prestressing steel is shown in Fig. (14). Check the eccentricity of prestress, e_o.

From Fig. (14), $e_o = 3$ in., therefore the magnitude of F and the required prestressing steel area should be recomputed as followings;

$F = 323.45 \times 12 \ / \ 21.55 = 180$ k

$F_o = 175 \times 180 \ / \ 140 = 225$ k

$A_s = 180 \ / \ 140 = 1.285$ in.^{2}, use 12 strands.

Find the center of gravity of prestressing steel at the end of the girder and draw the path of the c.g.s. of the girder as shown in Fig. (15). Taking moments about the bottom fiber,

$4 \times 33 = 132$

$8 \times 3 = \underline{24}$

$156 \div 12 = 13$ in. above the bottom fiber

Check the stresses at the end of the girder,

$$f^t = -\frac{225}{369} + \frac{222 \times 2.58}{2530} = -0.61 + 0.23 = -0.38 \text{ ksi}$$

$$f^b = -\frac{225}{369} - \frac{225 \times 2.58}{3220} = -0.61 - 0.18 = -0.79 \text{ ksi}$$

Check the stresses at transfer at the center of the span,

$$f^t = -\frac{225}{369} + \frac{225 \times 12.83 - 86.5 \times 12}{369 \times 6.86} = -0.61 - \frac{1853}{369 \times 6.86}$$

$= -0.61 + 0.731 = +0.121 \text{ ksi} < 6\sqrt{f'_{ci}} = 424 \text{ psi (tension)}$

$$f^b = -\frac{225}{369} - \frac{1853}{369 \times 8.72} = -0.61 - 0.576 = -1.186 \text{ ksi}$$

$$< 0.6\, f'_{ci} = 3.0 \text{ ksi}$$

Check the stresses under working load at the center of the span by Eq. (12a) and Eq. (12b),

$$f^t = -\frac{180}{369} - \frac{217.15 \times 12 - 0.263 \times 12 - 180 \times 12.83}{369 \times 6.86}$$

$$= -0.488 - \frac{870}{369 \times 6.86} = -0.488 - 0.344 = -0.832 \text{ ksi}$$

$$< 0.4\, f'_c = 2.4 \text{ ksi}$$

$$f^b = -\frac{180}{369} + \frac{870}{369 \times 8.72} = -0.488 + 0.27 = -0.218 \text{ ksi}$$

compression o.k.

Int. span, assuming the c.g.s. is 4 in. above the bottom fiber of precast girder, and from previous steps, the following data can be computed;

$M_p = 617.1$ ft-k, $M_G = 235.2$ ft-k, $M_c = 261$ ft-k

$m_b = 0.583$, $m_t = 0.263$

$k_b = 6.86$ in., $k_t = 8.72$ in.

Using Eq. (9b) to find F

$$F = \frac{(617.1 - 0.583 \times 261) \times 12}{11.83 + 8.72} = 769.1 \times 12 / 20.55 = 449 \text{ k}$$

$$F_o = 175 \times 449 / 140 = 560.5 \text{ k}$$

$A_s = 449/140 = 3.205$ in.^{2} $\quad 3.205/0.1089 = 29.4$

Use 30 strands.

The arrangement of the prestressing steel is shown in Fig. (16). Check the eccentricity of prestress, e_o.

From Fig. (16), e_o = 4.027 in. above the bottom fiber, it's close enough, so no revision is needed. Then find center of gravity of prestressing steel at the end of the girder and draw the path of c.g.s. of the girder as shown in Fig. (17).

Taking moments about the bottom fiber to locate the c.g.s. at the end of the girder,

$$12 \times 29 = 348$$
$$18 \times 4 = \underline{72}$$
$$420 \div 30 = 14 \text{ in. above the bottom fiber}$$

Check the stresses at ends at transfer,

$$f^t = -\frac{560.5}{369} + \frac{560.5\text{x}1.83}{2530} = -1.52 + 0.406 = -1.014 \text{ ksi}$$

$$f^b = -\frac{560.5}{369} + \frac{560.5\text{x}1.83}{3220} = -1.52 - 0.319 = -1.839 \text{ ksi}$$

Check the stresses at transfer at midspan by Eq. (11a) and Eq. (11b),

$$f^t = -\frac{560.5}{369} + \frac{560.5 \times 11.83 - 235.2 \times 12}{369 \times 6.86}$$

$$= -1.52 + \frac{6,640 - 2,820}{369 \times 6.86} = -1.52 + \frac{3,820}{369\text{x}6.86}$$

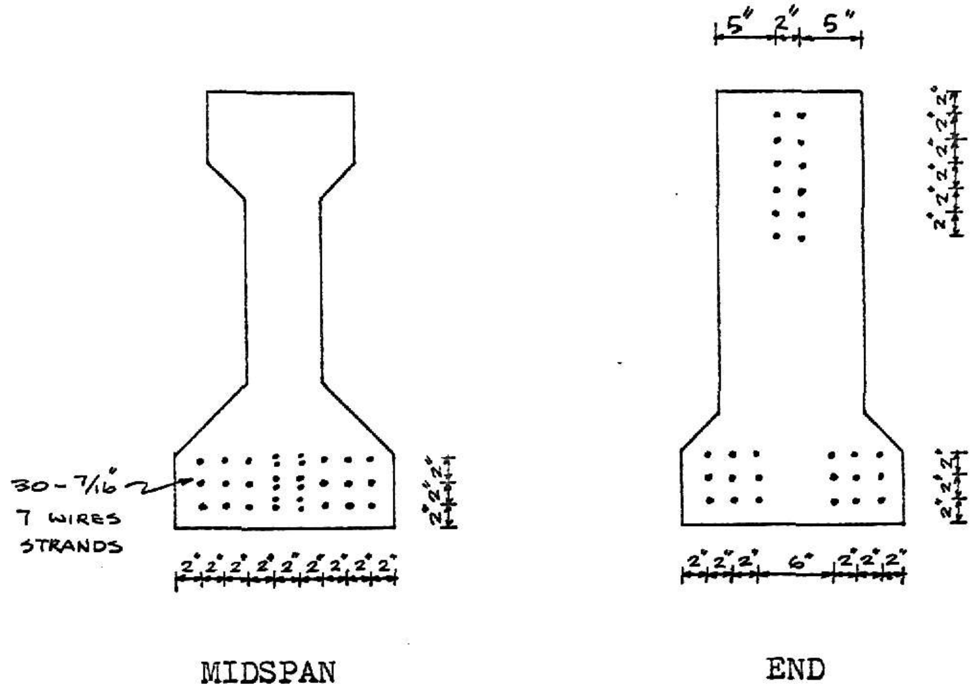


FIG. 16. ARRANGEMENT OF PRESTRESSING STEEL OF SPAN BC

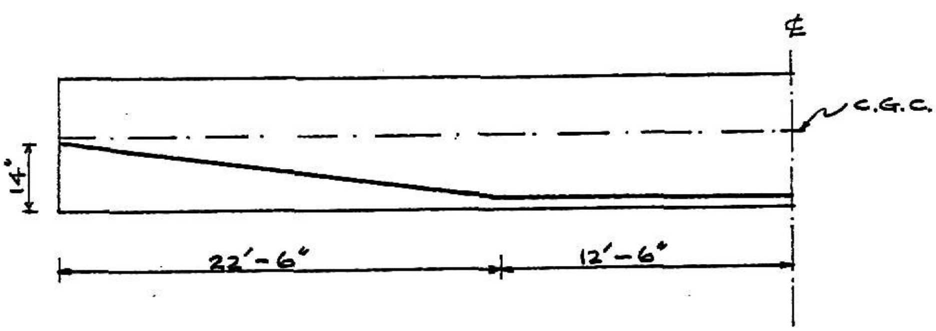


FIG. 17. PATH OF C.G.S. OF SPAN BC

$$= -1.52 + 1.51 = -0.01 \text{ ksi} \quad \text{compression o.k.}$$

$$f^b = -\frac{560.5}{369} - \frac{3{,}820}{369 \times 8.72} = -1.52 - 1.188 = -2.708 \text{ ksi}$$

$$< 3.0 \text{ ksi} \quad \text{o.k.}$$

Check the stresses under working load at midspan by Eq. (12a) and (12b);

$$f^t = -\frac{449}{369} - \frac{617.1 \times 12 - 0.267 \times 261 \times 12 - 449 \times 12}{369 \times 6.86}$$

$$= -12.15 - \frac{2{,}834}{369 \times 6.86} = -1.215 - 1.12 = -2.335 \text{ ksi}$$

$$< 2.4 \text{ ksi o.k.}$$

$$f^b = -\frac{449}{369} + \frac{2{,}834}{369 \times 8.72} = -1.215 + 0.881 = -0.334 \text{ ksi}$$

compression o.k.

(VII). The moments over piers due to creep and shrinkage

a. due to creep

The final restraint moments M_{rc} without adjustment for creep effect at supports, can be known from appendix IV p. 76, and p. 80, as follows;

At support B, $M_{rc} = -327.15 + 507 = 175.85$ ft-k

C, $M_{rc} = -452.93 + 822 = 369.07$ ft-k

Evaluating the creep factors as followings;

(1). The modulus of elasticity at strand release = 4.3×10^6 psi, from Fig. (4) the specific creep at 20 years is 0.3×10^{-6} in/in/psi.

(2). For release of the prestress force at 2 days, the value of specific creep is adjusted by Fig. (5) to;

$$1.66 \times 0.30 \times 10^{-6} = 0.498 \times 10^{-6} \text{ in/in/psi}$$

(3). The volume/surface ratio of Type II I-beam is 3.4 from AASHO-PCI Standards data, and from Fig. (6), the creep volume/surface ratio correction factor is 1.25. So the specific creep is adjusted as follows;

$$0.498 \times 10^{-6} \times 1.25 = 0.623 \times 10^{-6} \text{ in/in/psi}$$

(4). From Fig. (7), at 28 days 0.4 of the creep has occurred, leaving 0.6 to produce restraint moment, that is,

$$0.623 \times 10^{-6} \times 0.6 = 0.3735 \times 10^{-6} \text{ in/in/psi}$$

(5). The value of o for the structure with a connection made at 28 days would be;

$$\phi = 0.3735 \times 10^{-6} \times 4.8 \times 10^{6} = 1.79$$

(6). $\phi = 1.79$ is known, then from Fig. (8) can get;

$$(1 - e^{-\phi}) = 0.85$$

$$\frac{1 - e^{-\phi}}{\phi} = 0.47$$

Therefore the final moment M_{RC} at supports can be computed as follows;

At support B, $M_{RC} = 175.85 \times 0.85 = 149.5$ ft-k

$$C,\ M_{RC} = 369.07 \times 0.85 = 314 \text{ ft-k}$$

b. due to shrinkage

From appendix V p. 83, the final restraint moments M_{RS} at supports are;

$$\text{At support B, } M_{RS} = 1.216\ m_{rs} \left(\frac{1 - e^{-\phi}}{\phi} \right)$$

$$C,\ M_{RS} = 0.892\ m_{rs} \left(\frac{1 - e^{-\phi}}{\phi} \right)$$

$$\text{where } m_{rs} = 0.0006 \times 0.4 \times 1.0 \times 3.8 \times 10^6 \times 66 \times 6.5 \times 12.23$$

$$= 4{,}795{,}000 \text{ in-lb} = 399 \text{ ft-k}$$

Assume the condition of the continuity connections made 28 days after prestress release, exposure at 50 percent relative humidity and ultimate shrinkage is 0.0006 in/in. Substituting the value of m_{rs} and $\frac{1-e^{-\phi}}{\phi}$ into the expression of M_{RS}, then

$$\text{At support B, } M_{RS} = 1.216 \times 399 \times 0.47 = 228 \text{ ft-k}$$

$$C,\ M_{RS} = 0.892 \times 399 \times 0.47 = 167.4 \text{ ft-k}$$

(VIII). The positive moments over piers and design the positive connection

By summing the restraint moments due to creep and shrinkage in previous the step and the live load and impact from appendix VI p. 90, then the max, positive moments at supports are as follows;

$$\text{At support B, } M = 149.5 - 228 - (39.456 \times 2 \times 0.64 + 1.842 \times 18) \times 0.5 \times 1.3$$

$$= 149.5 - 228 - 54.4 \times 0.5 \times 1.3 = -23.5 \text{ ft-k}$$

$$C, \; M = 314 - 167.4 - (21.587 \times 2 \times 0.64 + 0.831 \times 18)$$
$$\times 0.5 \times 1.256 = 314 - 167.4 - 26.3 = 172.9 \text{ ft-k}$$

In this particular case, there is no positive moment at support B, therefore design the positive moment connection for support C only. Use intermediate grade reinforcement with an allowable stress $= 0.6\, f'_y = 24$ ksi, then required area of steel is;

$$A'_s = 172.9 \times 12/24 \times 7/8 \times 39.5 = 2.5 \text{ in.}^2$$

Use 6 - #6 bars, provided $A'_s = 2.64$ in.^{2}

To find the stress in the steel,

first compute the ratio of moduli of elasticity n,

$$n = \frac{2{,}900{,}000}{w^{3.5} 3.3\sqrt{f'_c}} = \frac{2{,}900{,}000}{(150)^{1.5} 3.3\sqrt{6{,}000}} = 6.49$$

$$p' = 2.64/66 \times 39.5 = 0.001025, \quad p'n = 0.00665$$

$$k = \sqrt{2pn - (pn)^2} - pn = \sqrt{0.0133 - (0.00665)^2} - 0.00665$$
$$= 0.1155 - 0.0133 = 0.10885$$

$$j = 1 - \frac{k}{3} = 0.96705$$

$$f_s = 172.9 \times 12/2.64 \times 0.96705 \times 39.5 = 20.6 \text{ ksi} < 24 \text{ ksi o.k.}$$

Designing the embedment length

for the allowable bond stress $= 0.1\, f'_c = 400$ psi > 350 psi,

so use 350 psi.

Assume the distance from the end face of the precast girder to the inside face of the hook = 12 bar diameter = $12 \times 6/8 = 9$ in.

Min. hook radius = 3x6/8 = 2.25 in.

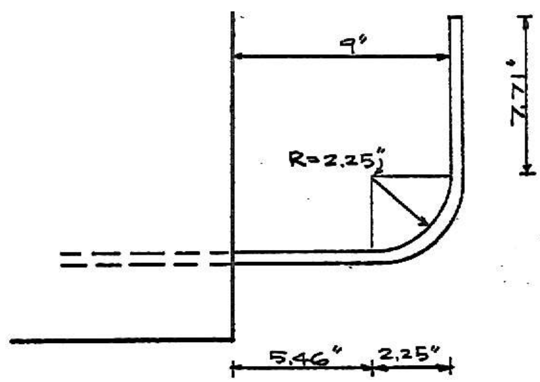
Req. embedment length = 206,000 x 0.6/350 = 2.36 = 15 in.

1/4 bend = 3.54 in.

15 - 3.54 - 6.75 = 4.71 in.

Use 7.71 in. extension, gives 18 in. embedment.

To ensure that bars extend into the part of the beam where the prestress force is effective, and to avoid termination all connection bars at one point, extend 2 bars 2 ft. into the beam, and extend the other 4 bars 3 ft. 6 in. into the beam.



(IX) Design of negative reinforcement

From the previous step, the moments over the piers due to live load can be determined,

At support B, M = (-198.334 x 0.64 - 3.0597802 x 18 - 7.145 x 18) x 0.5 x 1.27

= -437.5 x 0.5 x 1.27 = -278 ft-k

C, M = (248.147 x 2 x 0.64 - 5.793 x 18 x 2) x 0.5 x 1.256 = -526.5 x 0.5 x 1.256

= -331 ft-k

Design the negative moment reinforcement at support C; the ultimate negative moment is

$M_u = 2.5 \times 331 = 828$ ft-k

Then use the ordinary ultimate design procedures.

Assume a = 4.9 in.

$A_s' = 828 \times 12/0.9 \times 40 \times (39.25 - 2.45) = 7.5$ in.^{2}

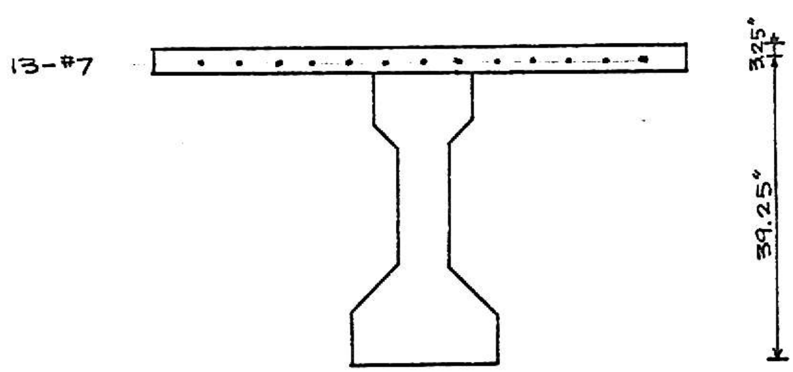
then, $a = 7.5 \times 40/0.85 \times 4 \times 18 = 4.91$ in.(it's close enough.)

Use 13 - #7 bars, providing $A_s' = 7.8$ in.^{2}

Check the steel percentage,

$$p_b = \frac{0.85 \times 0.85 \times 4.0}{40} \quad \frac{87}{87 + 40} = 0.0537$$

$p' = 7.8/18 \times 39.25 = 0.01105 < 0.0537 \times 0.75 = 0.04025 < p_b$ o.k.



(X) Ultimate Strength

Ext. span, the ultimate strength of midspan;

Use Eq. (21),

$A_s = 12 \times 0.1089 = 1.308$ in.^{2}, $f_{su} = 250$ ksi

$p = 1.308/66 \times 0.792 \times 39.5 = 0.0063$

$M'_u = 1.308 \times 250 \times 39.5 \times (1 - 0.6 \times 0.0063 \times 250/6)$

$= 1.308 \times 250 \times 39.5 \times (1 - 0.16) = 10.860$ in-k $= 905$ ft-k

$> 217.15 \times 1.5 + 182.5 \times 2.5 = 325 + 456 = 781$ ft-k

Int. span, the ultimate strength of midspan;

$A_s = 30 \times 0.1089 = 3.27$ in.^{2}

$p = 3.485/66 \times 0.792 \times 38.5 = 0.01625$

$M'_u = 3.27 \times 250 \times 38.5 \times (1 - 0.6 \times 0.001625 \times 250/6)$

$= 3.27 \times 250 \times 38.5 \times (1 - 0.04) = 30.200$ in.-k$=2,515$ ft-k

$> 617.1 \times 1.5 + 261 \times 2.5 = 925 + 652.5 = 1577.5$ ft-k

The ultimate strength over piers, use following equations;

$$M'_u = A'_s f'_y (d - \frac{a}{2})$$

$A'_s = 7.8$ in.^{2}, $f'_y = 40$ ksi, $d = 39.25$ in.

$a = 7.8 \times 40 / 0.85 \times 4 \times 18 = 4.9$ in.

$M'_u = 7.8 \times 40 \times (39.25 - 2.45) = 1.148$ in-k $= 955$ in-k

$> 2.5 \times 331 = 828$ ft-k o.k.

(X) Design of shear steel

Ext. span;

Reaction due to dead load;

girder's weight	0.384 20	= 7.68
slab	0.5843 20	= 11.686
diaphragm	1.375 1.5	= 2.063
		21.429 k

The max. reactions due to live load and impact, can be obtained under the loading condition shown in Fig. (18) and Fig. (19) for support A and B respectively;

Reaction at support A, $40 \times 1.27 \times 0.5 = 25.4$ k

B, $38 \times 1.27 \times 0.5 = 24.1$ k

Then shear diagram for span AB will be shown in Fig. (20).

To calculate the ultimate shear at support B to design the shear steel,

$$V_{ult} = 24.1 \times 2.5 + 20.054 \times 1.5 = 90.40 \text{ k}$$

Shear carried by strand can be computed by followings;

$$\sqrt{30^2 + (13 \times 12)^2} = \sqrt{25{,}200} = 158.8 \text{ in.}$$

$$V_s = 60 \times 30 / 158.8 = 11.31 \text{ k}$$

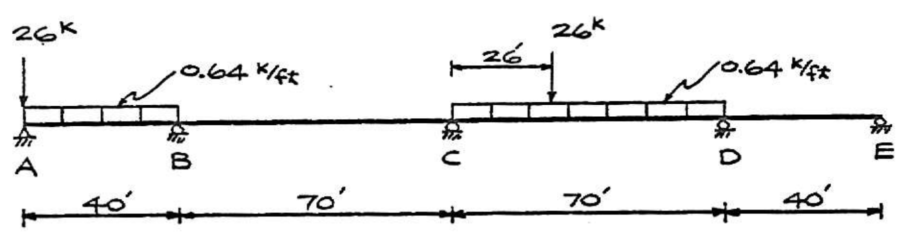


FIG. 18. MAX. REACTION AT SUPPORT A

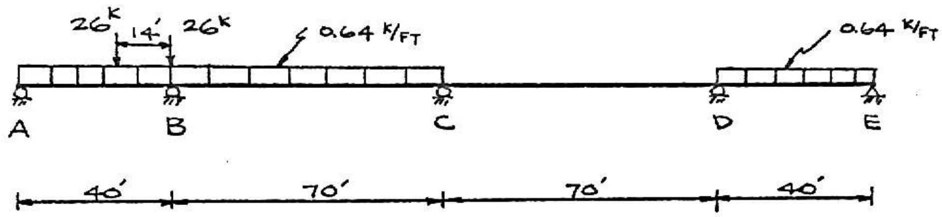
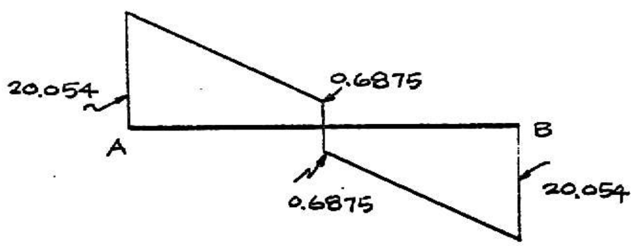
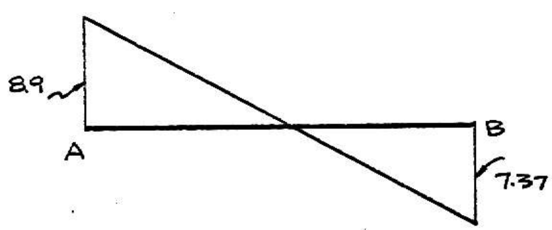


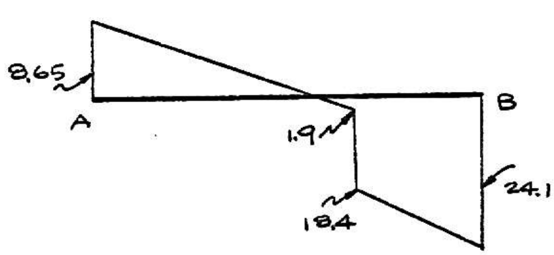
FIG. 19. MAX. REACTION AT SUPPORT B.



(a). DUE TO DEAD LOAD



(b). DUE TO LIVE LOAD AND IMPACT FROM FIG. (18) LOADING CONDITION (Loadsx1.27x0.5)



(c). DUE TO LIVE LOAD AND IMPACT FROM FIG. (19) LOADING CONDITION (Loadsx1.27x0.5)

FIG. 20. SHEAR DIAGRAM OF SPAN AB

The effective ultimate shear $V_u = 90.40 - 11.31 = 79.09$ k

use #4 bars, $A_v = 0.4$ in.^{2}

$V_c = 180 x b' x\ jd = 180 \times 6 \times 7/8 \times 39.5 = 37.3$ k

$jd = 7/8 \times 39.5 = 34.6$ in.

$s = 2 \times 0.4 \times 40 \times 34.6/79.09 = 24.4$ in.

Min. spacing $3/4 x 39.5 = 29.6$ in.

or $0.4/0.0025 = 26.7$ in.

use #4 bars at 24 in. c.c. spacing

Check the horizontal shearing stress at top fiber of precast girder between slab and girder under live load and impact by the following equation;

$$v = \frac{VQ}{I_c b}$$

$Q = 66 \times 6.5 \times (15.48 - 3.25) \times 0.792 = 4{,}160$ in.^{3}

$v = 60.25 x 4{,}160 x 1{,}000/149{,}155 x 12 = 140 \text{ psi} < 225 \text{ psi}$

According Sec. 212.3.3. Tentative Recommendations for Prestressed Concrete, the allowable bond stress is 225 psi for using additional steel ties. Therefore, #4 ties should be placed at 12 in. spacing in outer quarter of the span.

Int. span;

Reaction due to dead load;

Girder's weight	$0.384 \times 35 = 13.43$
slab	$0.5843 \times 35 = 20.41$
diaphragm	$1.375 \times 1.5 = 2.063$
	35.903 k

The max. reactions due to live load and impact can be obtained under the loading condition shown in Fig. (19) and Fig. (21) for supports B and C respectively;

Reaction at support B, $(25 + 26) \times 1.256 \times 0.5 = 32$ k

C, $46.5 \times 1.256 \times 0.5 = 29.8$ k

Or, $(27.15 + 26) \times 1.256 \times 0.5 = 33.4$ k

The shear diagram for span BC shown in Fig. (22).

To calculate the ultimate shear at support B to design the shear steel,

$$V_{ult} = 34.528 \times 1.5 + 29.8 \times 2.5 = 51.7 = 74.5 = 126.2 \text{ k}$$

Shearing force carried by the strand can be computed as follows;

$$\sqrt{(25)^2 + (22.5 \quad 12)^2} = \sqrt{625 + 72,900} = \sqrt{73,525} = 271 \text{ in.}$$

$$V_s = 179.5 \times 25 \ / \ 271 = 16.5 \text{ k}$$

The effective ultimate shear $V_u = 126.2 - 16.5 = 109.7$ k

use #4 bars, $A_v = 0.4$ in.^{2}

$$V_c = 180 \times 6 \times 7 \ / \ 8 \times 38.5 = 36.4 \text{ k}$$

$$jd = 7/8 \times 38.5 = 33.7 \text{ in.}$$

$$s = 2 \times 0.4 \times 40 \times 33.7 \ / \ (109.7 - 36.4) = 14.74 \text{ in.}$$

Use #4 bars at 12 in. c.c. spacing

Check the horizontal shearing stress at top fiber of pre-cast girder between slab and girder under live load and impact by following formula;

$$v = \frac{VQ}{I_c b}$$

$$Q = 66 \times 6.5 \times (15.48 - 3.25) \times 0.792 = 4,160 \text{ in.}^3$$

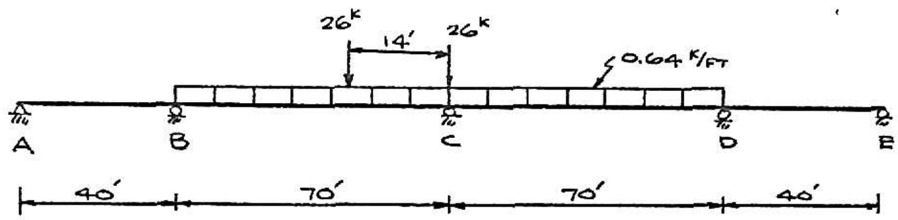
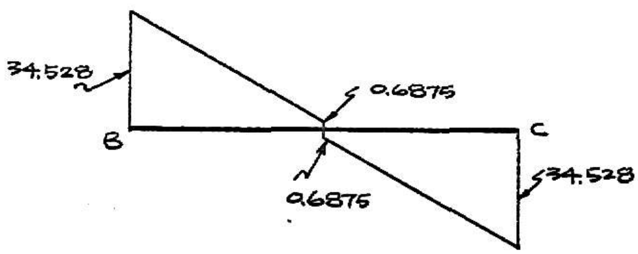
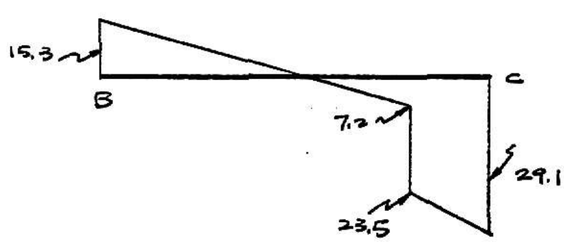


FIG. (21). MAX. REACTION AT SUPPORT C.



(a) DUE TO DEAD LOAD

(b) DUE TO LIVE LOAD AND IMPACT FROM FIG. (19) LOADING CONDITION (Loadsx1.256x0.5)



(c) DUE TO LIVE LOAD AND IMPACT FROM FIG. (21) LOADING CONDITION (Loadsx1.256x0.5)

FIG. (22). SHEAR DIAGRAM OF SPAN BC

$v = 75.4 \times 4{,}160 \times 1{,}000 / 149{,}155 \times 12 = 173$ psi < 225 psi

So, use #4 bars at 12 in. spacing will be satisfied.

(XI) Compute the camber

a. due to dead load of the girder.

At the time prestressing, $f'_{ci} = 4{,}000$ psi, so $E_c = 3.8 \times 10^6$ psi, then the deflection can be found as follows;

Ext. span

$$\Delta_G = \frac{5wl^4}{384EI_p} = 5 \times 384 \times (40)^4 \times (12)^3 / 384 \times 3.8 \times 10^6 \times 50{,}980$$

$$= 0.114 \text{ in. (downward)}$$

Int. span;

$$\Delta_G = 5 \times 384 \times (70)^4 \times (12)^3 / 384 \times 3.8 \times 10^6 \times 50{,}980 = 1.07 \text{ in.}$$

(downward)

b. due to eccentricity of the prestressing force

Ext. span

Moment due to prestress at end is

$M = 2.83 \times 225 = 637{,}000$ in-lb

at center is $M = 12.83 \times 225 = 2{,}890{,}000$ in-lb

The moment diagram constructed as shown in Fig. (23), and M/EI diagram also in Fig. (23). Then use conjugate beam method ro find the deflection at the center of the span;

part	A	X	AX
I	15.3×10^7	120	18.35×10^9
II	27.0×10^7	136	30.65×10^9
III	18.9×10^7	42	7.95×10^9
	61.1×10^7		56.95×10^9

$$\Delta_p = (61.1x10^7x20x12 - 56.95x10^9)/3.8x10^6x50{,}980$$
$$= 0.465 \text{ in. (upward)}$$

Net camber = 0.465 - 0.114 = 0.351 in. (upward)

Int. span

Moment due prestress at end is

$M = 1.83 \times 560.5 \times 10^3 = 1{,}010{,}000$ in-lb

at center is $M = 11.83x56.5x10^3 = 6{,}640{,}000$ in-k

The moment diagram constructed as shown in Fig. (24), and M/EI diagram is also in Fig. (24). Then use conjugate beam method to find deflection at the center of the span;

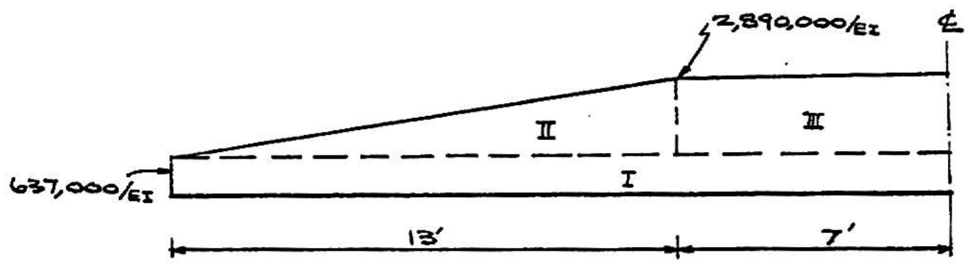


FIG. 23. M/EI DIAGRAM OF SPAN AB DUE TO PRESTRESS

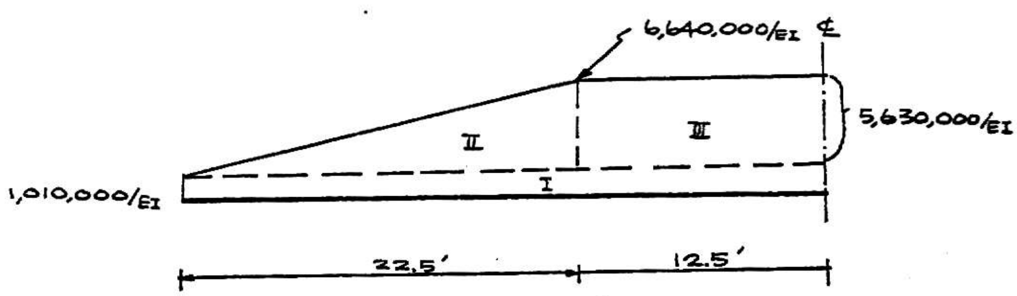


FIG. 24. M/EI DIAGRAM OF SPAN BC DUE TO PRESTRESS

Part	A	X	AX
I	235×10^6	210	494×10^8
II	760×10^6	240	$1{,}825\times10^8$
III	844×10^6	75	632.5×10^8
	$1{,}839\times10^6$		$2{,}951.5\times10^8$

Then deflection at center is,

$$\Delta_p = (1{,}839\times10^6\times35\times12 - 2{,}951.5\times10^8)/3.8\times10^6 \quad 50{,}980$$
$$= 2.46 \text{ in. (upward)}$$

Net camber = 2.46 - 1.07 = 1.39 in. (upward)

c. due to the weight of slab and diaphragm

When the slab and diaphragm are poured, the strength of the concrete in the girder will 6,000 psi, and $E_c = 4.8\times10^6$ psi. Then compute the deflection at the center due to weight of slab are;

Ext. span;

$$\Delta_{ss} = 5\times584.3\times(40)^4\times(12)^3/384\times4.8\times10^6\times50{,}980$$
$$= 0.1375 \text{ in. (downward)}$$

due to diaphragm

$$\Delta_{dia.} = 1{,}375\times(40\times12)^3/48\times4.8\times10^6\times50{,}980$$
$$= 0.013 \text{ in. (downward)}$$

Net camber = 0.351 - 0.1375 - 0.013 = 0.213 in. (upward)

Int. span;

$$\Delta_{ss} = 5\times584.3\times(70)^4(12)^3/384\times4.8\times10^6\times50{,}980$$

$= 1.29$ in. (downward)

$\Delta_{dia.} = 1,375 \times (70 \times 12)^3 / 48 \times 4.8 \times 50,980 = 0.07$ in. (downward)

Net camber $= 1.39 - 1.29 - 0.07 = 0.03$ in. (upward)

After the slab is poured, the net camber in the girder are 0.213 in. and 0.03 in. for exterior and interior span respectively. Since all the stresses in the beam at this time will be sustained permanently, the camber will gradually increase owing to creep in the concrete. If the beam were acting independently, camber could be expected to increase 100 to 300 percent from Sec. 203.3 Tentative recommendations for Prestressed Concrete. Actually the beam and slab now form a composite structure in which the stress in the slab is zero. The tendency of the beam to increase its camber is resisted by the composite section, so that the resultant increase is lower than would be expected if the girder were acting alone. We shall assume that the camber growth beyond the point would be 150 per cent if the beam were acting independently. Therefore, the camber growth will computed by followings;

Ext. span

$1.5 \times 0.213 \times 50,980 / 149,155 = 0.1093$ in.

$\therefore$ Total final camber $= 0.213 - 0.1093 = 0.3223$ in. (upward)

Int. span

$1.5 \times 0.03 \times 50,980 / 149,155 = 0.0153$ in.

$\therefore$ Total final camber $= 0.03 + 0.0153 = 0.0453$ in. (upward)

d. due to live load and impact

For this is a continuous girder, the max. positive moment is not exactly at center of the span, but it is near the mid-span. So in computing the max. camber or deflection, for convenience in computation, the deflection at center of the span will be assumed the max.

Ext. span

Consider the loading condition as shown in Fig. (25), the moments at supports are computed,

$$M_B = -158.8 \text{ ft-k}, \quad M_C = -149.8 \text{ ft-k}, \quad M_D = -203.3 \text{ ft-k}$$

Then the M/EI diagram of span AB can be constructed as in Fig. (26) and use conjugate beam method to find the deflection at midspan by parts as in the following;

$$\delta_1 = 5 \text{x} 640 \text{x} (40)^4 \; (12)3/384 \text{x} 4.8 \text{x} 10^6 \text{x} 149{,}155$$

$$= 0.00516 \text{ in. (downward, due to uniform load)}$$

$$\delta_2 = 18{,}000 \text{x} (40 \text{x} 12)^3/48 \text{x} 4.8 \text{x} 10^6 \text{x} 149{,}155$$

$$= 0.058 \text{ in. (downward, due to concentrate load)}$$

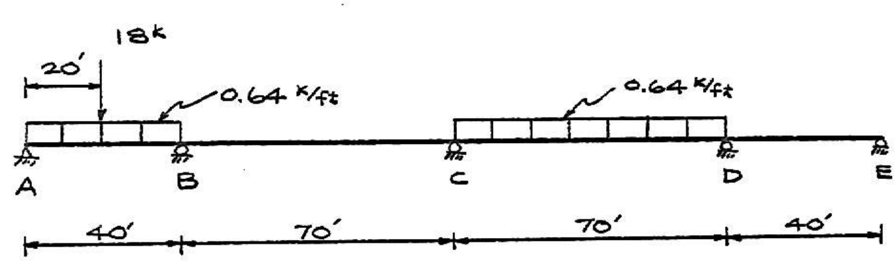


FIG. 25. MAX. DEFLECTION AT MIDSPAN OF SPAN AB

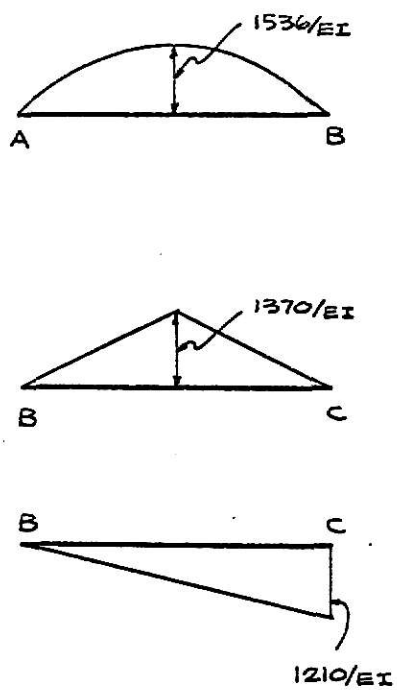
$$\delta_3 = 1{,}210 \times 1{,}000\ (40 \times 12)^2 / 16 \times 4.8 \times 149{,}155 = 0.0244 \text{ in.}$$

(downward, due to end moment)

Total deflection at the center of span AB is

$$\Delta_{LL} = 0.0516 - 0.058 - 0.0244 = 0.134 \text{ in. (downward)}$$

Net camber = 0.3223 - 0.134 = 0.1883 in. (upward)



(a). DUE TO UNIFORM LOAD

(b). DUE TO CONCENTRATE LOAD

(c). DUE TO END MOMENT

FIG. 26. M/EI DIAGRAM DUE LIVE LOAD IN SPAN AB

Int. span

Consider the loading condition as shown in Fig. (27), the moments at supports are computed,

$M_B = -321.83$ ft-k, $M_C = -236.2$ ft-k, $M_D = 27.83$ ft-k

Then the M/EI diagram of span BC can be constructed as in Fig. (28) and the conjugate beam method can be used to find the deflection at midspan by parts as in the following;

$$\delta_1 = 5\text{x}640\text{x}(70)^4\text{x}(12)^3/384\text{x}4.8\text{x}10^6\text{x}149{,}155$$

$$= 0.484 \text{ in. (downward, due to uniform load)}$$

$$\delta_2 = 18{,}000\text{x}(70\text{x}12)^3/48\text{x}4.8\text{x}10^6\text{x}149\text{m}155$$

$$= 0.312 \text{ in. (downward, due to concentrate load)}$$

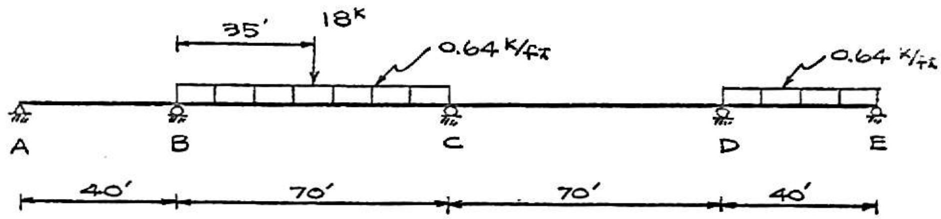
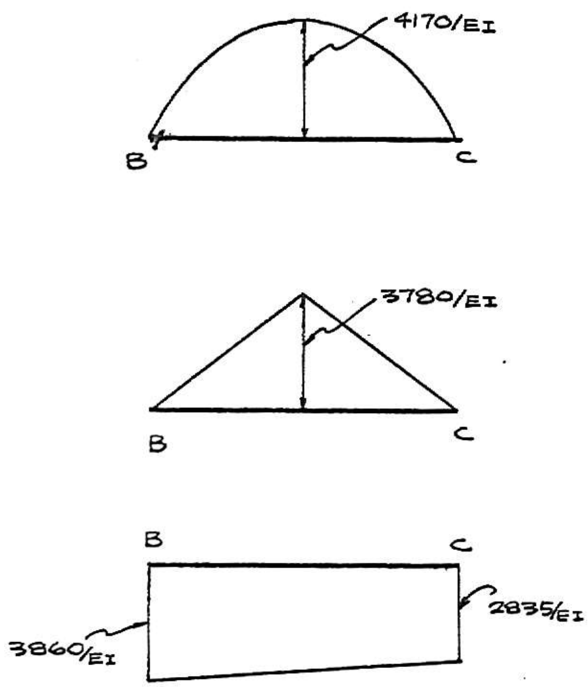


FIG. 27. MAX. DEFLECTION AT MIDSPAN OF SPAN BC



(a). DUE TO UNIFORM LIVE LOAD

(b). DUE TO CONCENTRATE LOAD

(c). DUE TO END MOMENTS

FIG. 28. M/EI DIAGRAM DUE TO LIVE LOAD OF SPAN BC

$\delta_3 = (2{,}835{,}000 - 386{,}000)\ (70x12)^2/16x4.8x10^6x149{,}155$

$= 0.412$ in. (upward, due end moment)

Total deflection at the center of span BC is

$\Delta_{LL} = 0.484 - 0.312 - 0.412 = 0.3687$ in. (downward)

Net camber $= 0.0453 - 0.384 = -0.3387$ in. (downward)

CONCLUSION

1. The effects of creep and differential shrinkage are very important to this type of bridge. The positive restraint moments over piers do not reduce the ultimate load carrying capacity, but will cause cracks over piers without the connection.

2. To date, the research on this subject is limited to studies conducted by P.C.A. Development and Research Laboratory on a two-span precast prestressed concrete bridge. Much research is still to be done on this type of bridge for controlling the influence of shrinkage and creep.

3. In this report, the design of end bearing and other details such as end block, pick up hook, etc. are not mentioned, because these details are different from one job to another. In practical design, however, these should be taken into account.

4. In recent years, the digital computer has been widely used, this kind of design work can be written into a program to eliminate the tedious calculations.

ACKNOWLEDGMENT

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APPENDIX II - NOTATIONS

A_b = cross-section area of deck slab

A_p = area of precast girder

A_s = area of main prestressing tensile steel

A_s' = area of conventional tensile steel

A_{sf} = area of steel required to develop the ultimate compressive strength of the overhanging portion of the flange

A_{sr} = the steel area required to develop the ultimate compressive strength of the web of a flange section

A_v = area of web reinforcement at spacing s, placed perpendicular to the axis of the member, which shall not be less than 0.0025b's

b = width of compression force of the flexure member

b' = width of web of flanged member

c = the distance from neutral axis to the extrme fiber of precast girder

c_t, c_b = distance to top or bottom fiber from c.g.c. of precast girder

c_t'', c_b'' = distance to top or bottom fiber from c.g.c'. of precast girder

d = distance from extreme compressive fiber to centroid of tensile force

E_b, E_c = elastic modulus of the deck slab or precast girder concrete

e = the base of Naperian logarithms, 2.7183

e_o = the distance between c.g.c. of precast girder and the c.g.s.

e_1 = the eccentricity under the allowable tension stress on top fiber of precast girder at transfer

e_2 = the eccentricity to resist optimum external moment

$(e'_2+t_b/2)$ = distance between the mid-depth of the slab and centroid of composite section

F = required total effective prestress force after deducting losses

F_o = total prestressing force just after transfer

$f^b, {}^t, f^b, f^t$=extreme fiber stresses

f'_b = allowable tensile stress at bottom fiber

f'_c = compressive strength of concrete at 28 days

f'_{ci} = ultiamte unit stress in concrete, at time of transfer

f_e = effective unit prestress after deduting losses

f_i = initial unit prestress in steel before transfer

f_s = ultimate unit stress in prestressing steel

f'_s = allowable tensile stress in conventional steel

f_{su} = average stress in prestressing steel at ultimate load

f'_t = allowable tension on top fiber precast girder at transfer

f'_y = yield strength of conventional steel

h = the depth of the section

I = impact factor (max. 30 percent)

I_c = the moment of inertia of the composite section about its neutral axis

I_p = the moment of inertia of the precast girder about its neutral axis

j = ratio of diatance between controid of compression and centroid of tension to the depth d

k_b, k_t = the top or bottom distance

k_1 = a factor which shall be taken as 0.85 for strength f'_c, up to 4,000 psi and shall be reduce continuously at a rate of 0.05 for each 1,000 psi of strength in excess of 4,000 psi

L = Length in feet of the portion of the span which is loaded to produce the max. moment in the member

M_c = any moment acting on the composite section

M_{DL} = the max. bending moment due to dead load

M_{Dia} = the max. moment at the center of the span due to diaphragm

M_G = the max. moment at the center of span due to girder's weight

M_{LL} = the max. bending moment due to live load and impact

M_{ML} = the max. bending moment due to live load

M_p = the total moment acting the precast girder

M_R = the total positive moments over piers due to creep and shrinkage

M_{RC} = the final restraint moment over piers due to the effect of creep under prestress and dead load

M_{rdl} = the restraint moment due to the effect of creep under dead load

M_{rp} = the restraint moment due to the effect of creep under prestress

M_{RS} = the total restraint moment due to shrinkage

m_{rs} = restraint moment due to differential shrinkage

M_T = the total moment acts on the section

M_u = ultimate resisting moment

M'_u = ultimate flexural strength

n = ratio of modulus of elasticity

p = ratio of prestressing steel

p' = ratio of non-prestressing steel

s = longitudinal spacing of web reinforcement

t = the thickness of the flange

t_b = slab thickness

V_c = $0.06f'_c b'jd$, but not more than $180b'jd$ (assuming $j = 7/8$)

V_{DL} = shear due to dead load

V_{LL} = shear due to live load and impact

V_s = shear carried by the strands

V_u = effective shear due to specified ultimate load and effect of prestressing

V_{ult} = ultimate shear due to dead load and live load including impact

V/S = the volume/surface ratio of the member under consideration

Z_b, Z_t = sectional modulus

α = capacity reduction factor, equals 0.9

γ, γ' = angle of rotation

Δ = deflection of beam

o = the ratio of creep strain per psi of stress to the elastic strain per psi of stress

ϵ_c = basic creep strain

ϵ_s = differential shrinkage strain

ϵ_{su} = ultimate shrinkage strain at T = infinity

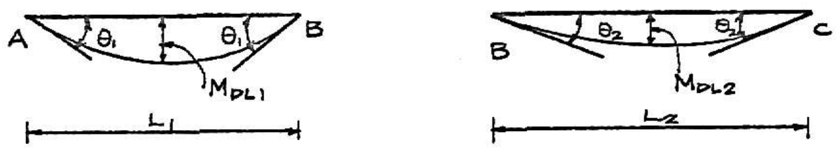
APPENDIX IV TABLE OF FIGURES

APPENDIX IV

Derivation of general equation for creep restraint moment

1. Dead load creep restraint moment

Using the conjugate beam method to find the slope at ends,

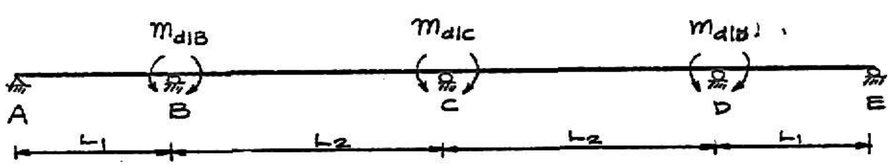


In span AB, $\theta_1 = M_{DL1}\ L_1/3EI$

span BC, $\theta_2 = M_{DL2}\ L_2/3EI$

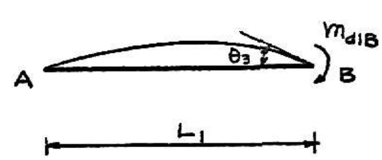
where $M_{DL1} = w_1 L_1^2/8$, and $M_{DL2} = w_2 L_2^2/8$, and assume EI is constant.

Apply moments m_{dl} to return beam ends to horizontal as shown in the Figure below;



Ext. span

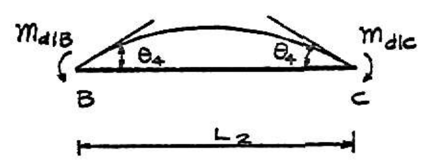
$\theta_1 = \theta_3$



$-m_{dlB} L_1/3EI = M_{DL1}\ L_1/3EI$

$m_{dlB} = -M_{DL1}$

Int. span



$\theta_2 = \theta_4$

$$+M_{DL2}L_2/3EI = -m_{d1B}L_2/3EI - m_{d1C}L_2/6EI$$

$$m_{d1B} = m_{d1C}$$

$$m_{d1B} = m_{d1C} = -2M_{DL2}/3$$

Distribute the restraint moemtns m_{dl} by moment distribution, then compute the final dead load restraint moments M_{rdl} due to creep (not reduced by the creep effects factor).

For example,

$M_{DL1} = 217.15$ ft-k, $m_{d1B} = 217.15$ ft-k

$M_{DL2} = 617.1$ ft-k, $m_{d1C} = -411$ ft-k

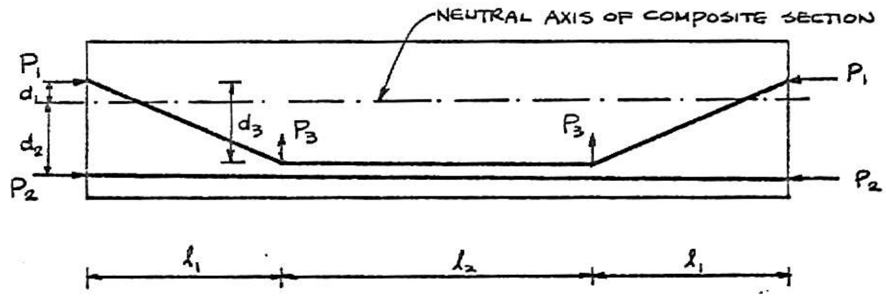
Then use moment distribution method;

	A	B		C		D		E
D.F.	0.	0.568	0.432	0.5	0.5	0.432	0.568	0.
FEM		-217.15	-411.	-411.	-411.	-411.	-217.5	
Bal.		-110.	83.85			83.85	-110.	
C.O.				-41.93	-41.93			
Σ M	0.	-327.15	-327.15	-452.93	-452.93	-327.15	-327.15	0.

The final restraint moments are;

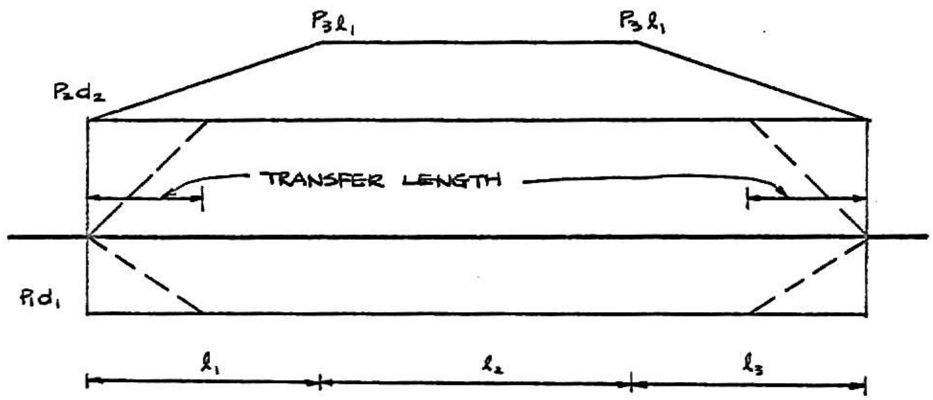
$M_{r1B} = -327.15$ ft-k, $M_{r1C} = -452.93$ ft-k

2. Prestress creep restraint moment



The creep under the prestress force depends on the location of the various forces with respect to the center of gravity of the composite section. The tendons will be replaced in the calculations by the equivalent forces acted on the member.

The moment diagram by parts can be shown as follows;

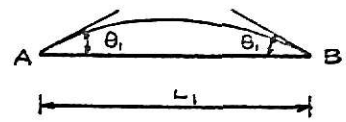


Strictly speaking, the moments should vary from zero to full value along the transfer length (dotted lines). For convenience, the rectangular moment diagram will be used in calculation;

Using the conjugate beam method, the slopes due to prestress at the ends are as follows;

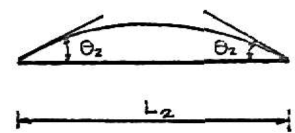
Ext. span

$$\theta_1 = \frac{p_1 d_1 L_1}{2EI} - \frac{p_2 d_2 L_1}{2EI} - \frac{p_3 l_1^2}{2EI} - \frac{p_3 l_1 l_2}{2EI}$$

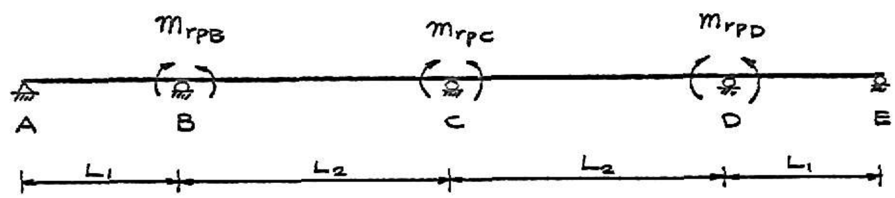


Int. span

$$\theta_2 = \frac{p_1 d_1 L_2}{2EI} - \frac{p_2 d_2 L_2}{2EI} - \frac{p_3 l_1^2}{2EI} - \frac{p_3 l_1 l_2}{2EI}$$



Moments m_{rp} to return beam ends to horizontal are as shown in the figure below;

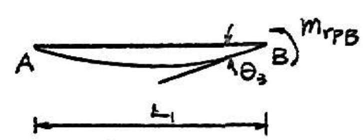


Ext. span

$\theta_1 = \theta_3$

$\theta_1 = m_{rpB} L_1/3EI$

$m_{rpB} = -3EI\theta_1/2L_1$



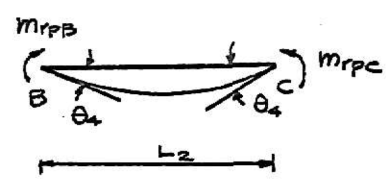
Int. span

$\theta_4 = m_{rpB} L_2/3EI - m_{rpC} L_2/6EI$

$m_{rpB} L_2/2EI$

$\theta_2 = \theta_4$

$m_{rpB} = m_{rpC} = -2EI\theta_2/L_2$



Distribute the restraint moments m_{rp} by moment distribution, then the final prestress restraint moments M_{rp} due to creep (not reduced by the creep effects factor) can be computed.

For example,

Ext. span

$p_1 = 60$ k, $p_2 = 120$ k, $p_3 = 36.68x60/13x12 = 14.1$ k,

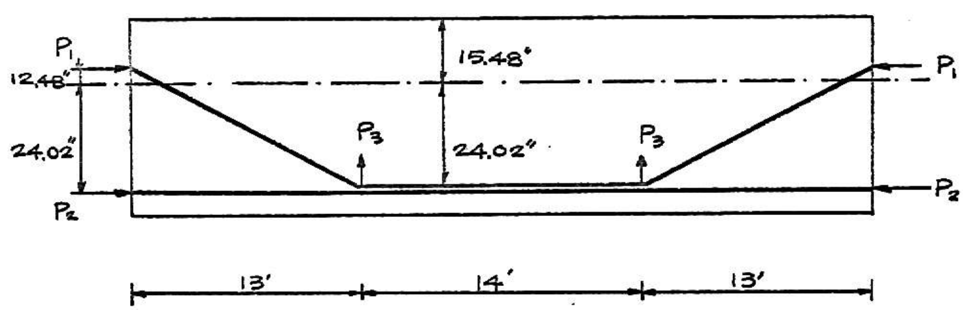
$$\theta_1 = \frac{60x12.48x40x12}{2EI} - \frac{120x24.02x40x12}{2EI}$$

$$- \frac{14.1x(13x12)^2}{2EI} - \frac{14.1x13x13x(12)^2}{2EI}$$

$$= \frac{1}{2EI}(359{,}000 - 1{,}427{,}000 - 343{,}000 - 343{,}000)$$

$$= 1{,}754{,}000/2EI$$

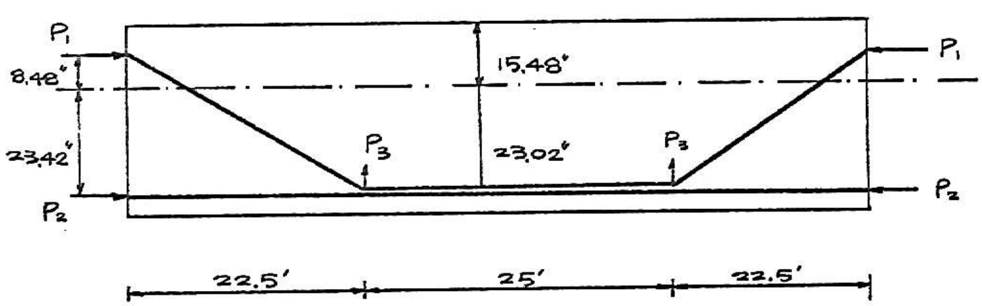
$$m_{rpB} = - \frac{3 \text{ x } EI}{2x40x12} \quad \frac{1{,}754{,}000}{2EI} = 2{,}760 \text{ in-k}$$



Int. span

$p_1 = 179.5$ k, $p_2 = 269.5$ k,

$p_3 = (31.5/22.5x12) \text{ x } 179.5 = 21.4$ k



$$\theta_1 = \frac{179.5\text{x}8.48\text{x}70\text{x}12}{2EI} - \frac{269.5\text{x}23.01\text{x}70\text{x}12}{2EI}$$

$$- \frac{21.4\text{x}(22.5\text{x}12)^2}{2EI} - \frac{21.4\text{x}22.5\text{x}(12)^2}{2EI}$$

$$= \frac{1}{2EI}(1{,}28{,}000 - 5{,}200{,}000 - 1{,}560{,}000 - 1{,}730{,}000)$$

$$= \frac{1}{2EI}(-7{,}210{,}000)$$

$$m_{rpC} = \frac{2EI}{70\text{x}12} \times \frac{7{,}210{,}000}{2EI} = 8{,}600 \text{ in-k}$$

Then use moment distribution method;

	A		B		C		D		E
DF	0.	0.568	0.432	0.5	0.5	0.432	0.568		0.
FEM		2760	8600	8600	8600	8600	2760		
BAL		3320	-2520			-2520	3320		
CO				1260	1260				
ΣM	0.	6080	6080	9860	9860	6080	6080		0.

The final restraint moments are;

M_{r1B} = 6080 in-k = 507 ft-k

M_{r1C} = 9860 in-k = 822 ft-k

Summing up the M_{rd1} and M_{rp}, and multiplying the value of $(1 - e^{\phi})$, then the final restraint moments due to creep M_{RC} can be computed.

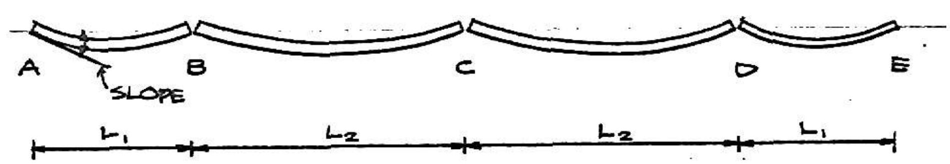
APPENDIX V

Derivation of general equation for restraint moment due to shrinkage.

As previously discussed, the restraint moments due to differential shrinkage between the deck slab and the precast beam are uniform along the composite section. The moment is expressed as following;

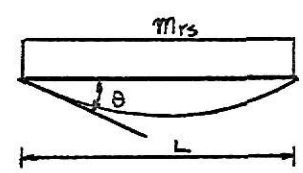
$$m_{rs} = \epsilon_s E_b A_b (e_2' - \frac{t_b}{2})$$

Assuming the bridge were composed of four simple spans, these moments will cause bending as shown below;



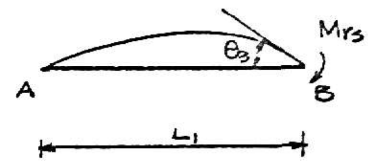
Therefore the slope at the ends of the simple span beams, can be obtained by the conjugate beam method as follow;

$$\theta = \frac{m_{rs} L}{2EI}$$



Then apply moments M_{rs}, at both sides of support B, C and D to return the beam ends to horizontal;

Ext. span



$$\theta_3 = \frac{M_{rs}L_1}{3EI}$$

$$\theta_3 = \theta$$

$$\frac{M_{rs}L_1}{3EI} = \frac{m_{rs}L_1}{2EI}$$

$$M_{rs} = 3\ m_{rs}/2$$

Int. span

$$\theta_4 = \frac{M_{rs}L_2}{2EI}$$

$$\theta_4 = \theta$$

$$M_{rs} = m_{rs}$$

Then use the moment distribution method to distribute the restraint moments M_{rs}, and the final shrinkage moment M_{RS} at supports B, C and D can be computed by multiplying by the value of $(\frac{1-e^{-\phi}}{\phi})$.

For example,

In exterior span M_{rs} is 1.5 m_{rs}, interior is m_{rs}.

	A	B	B	C	C	D	D	E
DF	0.	0.568	0.432	0.5	0.5	0.432	0.568	0.
FEM		1.5	1.0	1.0	1.0	1.0	1.5	
BAL		-0.284	0.216			0.216	-0.284	
CO				-0.108	-0.108			
Σ M	0.	1.216	1.216	0.892	0.892	1.216	1.216	0.

Final moment at B and D are 1.216 m_{rs} $\left(\frac{1-e^{-\phi}}{\phi} \right)$ and the final moments at C is 0.89 m_{rs} $\left(\frac{1-e^{-\phi}}{\phi} \right)$

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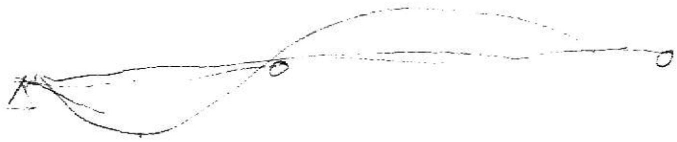
APPENDIX VI

```
C PROGRAM 1
C USING MOMENT DISTRIBUTION TO FIND THE ORDINATES OF INFLUENCE
C LINES FOR NEGATIVE MOMENT AT 0.75 AND AT C, MC, AT EVERY TWO FEET
  DIMENSION SPAN(10),FEM(20),DF(20),DISM(20),PM(20),RL2(20)
  DIMENSION PN(20)
  READ 1,NSPAN
1 FORMAT (I3)
  READ 2, (SPAN(J),J=1,NSPAN)
2 FORMAT (4F10.2)
  N=2*(NSPAN+1)
  READ 3, (DF(I),I=1,N)
3 FORMAT (10F7.4)
  SPINV=2.
  TL=0.
  DO 15 J=1,NSPAN
  TL=TL+SPAN(J)
15 CONTINUE
  PT=TL/SPINV
  M=PT
  UNIT=1.
  J=1
  SUM=SPAN(J)
13 DO 12 I=1,N
12 FEM(I)=0.
  A=SPINV*UNIT
5 IF(A-SUM) 6,6,4
6 I=2*J
  B=SPAN(J)-(SUM-A)
  FEM(I)=-B*((SUM-A)**2)/SPAN(J)**2
  FEM(I+1)=(B**2)*(SUM-A)/SPAN(J)**2
  GO TO 10
4 J=J+1
  SUM=SUM+SPAN(J)
  GO TO 5
10 DO 100 JJ=1,20
  DO 20 I=1,N,2
20 DISM(I)=-(FEM(I)+FEM(I+1))*DF(I)
  DO 30 I=2,N,2
30 DISM(I)=-(FEM(I)+FEM(I-1))*DF(I)
  DO 40 I=3,N,2
40 FEM(I)=FEM(I)+DISM(I)+DISM(I-1)*0.5
  N1=N-2
  DO 50 I=2,N1,2
  L=I+1
50 FEM(I)=FEM(I)+DISM(I)+DISM(L)*0.5
100 CONTINUE
```

```
      L=UNIT
      IR=J
      IF((SUM-A)-SPAN(J)/2.) 21,21,22
   21 RL2(IR)=(SUM-A)/SPAN(J)
      GO TO 23
   22 RL2(IR)=1.-(SUM-A)/SPAN(J)
   23 J1=J
      DO 115 I1=1,NSPAN
      PN(I1)=0.
  115 PM(I1)= .
      PM(J1)=RL2(IR)*SPAN(J)/2.
      DO 109 J2=1,NSPAN
      I=2*J2
  109 PN(J2)=(FEM(I)-FEM(I+1))/2.
      DO 110 K=1,NSPAN
  110 PM(K)=PM(K)+PN(K)
      PUNCH 7,L,FEM(4),FEM(6),PM(1),PM(2)
    7 FORMAT (I3,4F14.7)
      IF(UNIT-PT) 51,52,52
   51 UNIT=UNIT+1.
      GO TO 13
   52 CONTINUE
      STOP
      END
```

```
C PROGRAM 2
C USING THE OUTPUT OF PROGRAM 1.TO REARRANGE THE ORDINATES OF
C INFLUENCE LINES FOR MB AND MC AND FIND INFLUENCE LINES FOR
C POSITIVE MOMENTS AT MIDSPAN OF AB AND BC
  DIMENSION K(120),B(120),C(120),D(120),E(120)
  DO 10 I=2,111
10 READ 1, K(I),B(I).C(I).D(I),E(I)
 1 FORMAT (I3,4F14.7)
  K(1)=0
  B(1)=0.
  C(1)=0.
  D(1)=0.
  E(1)=0.
  PUNCH 3
 3 FORMAT (37HTHE ORDINATES OF INFLUENCE LINE OF MB)
  PUNCH 2,(B(I),I=1,111)
 2 FORMAT (5F14.7)
  PUNCH 4
 4 FORMAT (37HTHE ORDINATES OF INFLUENCE LINE OF MC)
  PUNCH 2, (C(I),I=1,111)
  PUNCH 5
 5 FORMAT (35HTHE ORDINATES OF POSITIVE MOMENT M1)
  PUNCH 2, (D(I),I=1,111)
  PUNCH 6
 6 FORMAT (35HTHE ORDINATES OF POSITIVE MOMENT M2)
  PUNCH 2, (E(I),I=1,111)
  SUMB=0.
  DO 11 I=1,56
11 SUMB=SUMB+B(I)
  DO 50 I=92,111
50 SUMB=SUMB+B(I)
  SUMC=0.
  DO 12 I=22,91
12 SUMC=SUMC+C(I)
  SUMD=0.
  DO 13 I=1,21
13 SUMD=SUMD+D(I)
  DO 60 I=56,91
60 SUMD=SUMD+D(I)
  SUME=0.
  DO 14 I=22,56
14 SUME=SUME+E(I)
  DO 70 I=92,111
70 SUME=SUME+E(I)
  SUMF=0.
  DO 20 I=56,91
20 SUMF=SUMF+B(I)
```

```
      SUMG=0.
      DO 30 I=1,21
   30 SUMG=SUMG+C(I)
      SUMH=0.
      DO 40 I=91,111
   40 SUMH=SUMH+C(I)
      SUMG=SUMG+SUMH
      PUNCH 7
    7 FORMAT (//48HTHE MAXIMUM NEGATIVE MOMENTS DUE TO UNIFORM LOAD)
      PUNCH 8
    8 FORMAT (22H         MB                  MC//)
      PUNCH 9, SUMB,SUMC
    9 FORMAT (2F14.7)
      PUNCH 15
   15 FORMAT (//48HTHE MAXIMUM POSITIVE MOMENTS DUE TO UNIFORM LOAD)
      PUNCH 16
   16 FORMAT (22H         M1                  M2//)
      PUNCH 17, SUMD,SUME
   17 FORMAT (2F14.7)
      PUNCH 18
   18 FORMAT (//46HPOSITIVE MOMENTS OVER PIERS DUE TO UNIFORM LOAD)
      PUNCH 19
   19 FORMAT (23H         MBP                 MCP//)
      PUNCH 21,SUMF,SUMG
   21 FORMAT (2F14.7)
      STOP
      END
```



5.7

THE ORDINATES OF INFLUENCE LINE OF MB

0.0000000	-.3974129	-.7888498	-1.1683343	-1.5298903
-1.8675419	-2.1753127	-2.4472271	-2.6773081	-2.8595802
-2.9880670	-3.0567925	-3.0597802	-2.9910547	-2.8446395
-2.6145582	-2.2948349	-1.8794936	-1.3625586	-.7380524
0.0000000	-1.1442372	-2.1734751	-3.0920489	-3.9042942
-4.6145471	-5.2271429	-5.7464166	-6.1767045	-6.5223422
-6.7876644	-6.9770084	-7.0947077	-7.1451000	-7.1325187
-7.0613009	-6.9357818	-6.7602969	-6.5391821	-6.2767725
-5.9774044	-5.6454125	-5.2851330	-4.9009010	-4.4970525
-4.0779231	-3.6478481	-3.2111632	-2.7722040	-2.3353064
-1.9048055	-1.4850371	-1.0803355	-.6950400	-.3334825
0.0000000	.3019982	.5728064	.8136453	1.0257355
1.2102970	1.3685504	1.5017166	1.6110158	1.6976682
1.7628946	1.8079155	1.8339515	1.8422228	1.8339498
1.8103534	1.7726538	1.7220713	1.6598268	1.5871405
1.5052332	1.4153248	1.3186364	1.2163881	1.1098004
1.0000939	.8884889	.7762064	.6644663	.5544894
.4474961	.3447067	.2473421	.1566224	.0737682
0.0000000	-.0637430	-.1176794	-.1623253	-.1981969
-.2258102	-.2456815	-.2583266	-.2642623	-.2640042
-.2580685	-.2469716	-.2312294	-.2113580	-.1878737
-.1612926	-.1321309	-.1009046	-.0681299	-.0343230
0.0000000				

THE ORDINATES OF INFLUENCE LINE OF MC

0.0000000	.1079336	.2142443	.3173090	.4155044
.5072077	.5907956	.6646452	.7271333	.7766369
.8115329	.8301984	.8310100	.8123448	.7725798
.7100918	.6232578	.5104546	.3700593	.2004488
0.0000000	-.2319745	-.4925215	-.7778027	-1.0839807
-1.4072166	-1.7436731	-2.0895122	-2.4408954	-2.7939852
-3.1449434	-3.4899318	-3.8251129	-4.1466485	-4.4507003
-4.7334307	-4.9910016	-5.2195750	-5.4153126	-5.5743768
-5.6929294	-5.7671326	-5.7931464	-5.7671384	-5.6852648
-5.5436900	-5.3385758	-5.0660840	-4.7223765	-4.3036157
-3.8059635	-3.2255817	-2.5586325	-1.8012777	-.9496795
0.0000000	-.9496802	-1.8012790	-2.5586342	-3.2255837
-3.8059659	-4.3036185	-4.7223796	-5.0660872	-5.3385792
-5.5436934	-5.6852682	-5.7671418	-5.7931519	-5.7671360
-5.6929328	-5.5743801	-5.4153157	-5.2195778	-4.9910042
-4.7334332	-4.4507026	-4.1466505	-3.8251148	-3.4899334
-3.1449448	-2.7939862	-2.4408963	-2.0895129	-1.7436736
-1.4072171	-1.0839809	-.7778029	-.4925215	-.2319746
0.0000000	.2004488	.3700595	.5104549	.6232582
.7100924	.7725806	.8123459	.8310113	.8301999
.8115347	.7766388	.7271355	.6646472	.5907976
.5072096	.4155062	.3173105	.2142455	.1079343
0.0000000				

THE ORDINATES OF POSITIVE MOMENT M1

		2		4
0.0000000	.8012933	1.6055745	2.4158320	3.2350538
4.0662278	4.9123422	5.7763846	6.6613442	7.5702081
8.5059650	7.4716018	6.4701060	5.5044708	4.5776785
3.6927193	2.8525812	2.0602521	1.3187199	.6309734
0.0000000	-.5721181	-1.0867367	-1.5460232	-1.9321454
-2.3072716	-2.6135692	-2.8732058	-3.0883495	-3.2611682
-3.3938291	-3.4885010	-3.5473566	-3.5725466	-3.5662559
-3.5306470	-3.4678874	-3.3801450	-3.2695876	-3.1383829
-2.9886990	-2.8227031	-2.6425635	-2.4504476	-2.2485235
-2.0369590	-1.8239217	-1.6055705	-1.3861001	-1.1676515
-.9524013	-.7425174	-.5401674	-.3475194	-.1567410
0.0000000	.1509988	.2864026	.4068218	.5128665
.6051471	.6842735	.7508564	.8055058	.8468317
.8814448	.9039550	.9169729	.9211084	.9169718
.9051735	.8863236	.8610323	.8299100	.7935668
.7526132	.7076590	.6593148	.6081908	.5548970
.5000439	.4442416	.3881005	.3322307	.2772425
.2237461	.1723517	.1236696	.0783103	.0368836
0.0000000	-.0318710	-.0588389	-.0811615	-.0990970
-.1129035	-.1228390	-.1291615	-.1321292	-.1320001
-.1290323	-.1234839	-.1156129	-.1056773	-.0939354
-.0806450	-.0660644	-.0504514	-.0340644	-.0171612
0.0000000				

THE ORDINATES OF POSITIVE MOMENT M2

0.0000000	-.1447393	-.2873021	-.4255119	-.5571920
-.6801661	-.7922575	-.8912899	-.9750864	-1.0414707
-1.0882661	-1.1132963	-1.1143844	-1.0893544	-1.0360294
-.9522329	-.8357883	-.6845193	-.4962495	-.2688017
0.0000000	.3118966	.6670032	1.0650746	1.5058655
1.9891199	2.5145928	3.0820353	3.6912021	4.3418373
5.0336960	5.7665330	6.5400910	7.3541260	8.2083900
9.1026350	10.0366080	11.0100630	$\boxed{11.0227510}$	10.0744230
9.1648310	8.2937260	7.4608570	6.6659780	5.9088390
5.1891913	4.5067859	3.8613743	3.2527082	2.6805371
2.1446142	1.6446894	1.1805146	.7518404	.3564185
0.0000000	-.3238406	-.6142356	-.8724936	-1.0999231
-1.2978332	-1.4675326	-1.6103299	-1.7275341	-1.8204538
-1.8913977	-1.9386746	-1.9665934	-1.9754628	-1.9665914
-1.9412861	-1.9008615	-1.8466266	-1.7798741	-1.7019305
-1.6149987	-1.5176877	-1.4140060	-1.3043624	-1.1900657
-1.0724247	-.9527481	-.8323445	-.7125229	-.5945918
-.4798602	-.3696370	-.2652303	-.1679495	-.0791031
0.0000000	.0683529	.1261899	.1740646	.2125304
.2421407	.2634491	.2770070	.2833738	.2830970
.2767321	.2648326	.2479519	.2266435	.2014609
.1729575	.1416867	.1082021	.0730572	.0368053
0.0000000				

```
THE MAXIMUM NEGATIVE MOMENTS DUE TO UNIFORM LOAD
       MB                 MC

  -198.3340000   -248.1472300

THE MAXIMUM POSITIVE MOMENTS DUE TO UNIFORM LOAD
       M1                 M2

     99.8569660    170.0632100

POSITIVE MOMENTS OVER PIERS DUE TO UNIFRM LOAD)
       MBP                MCP

     39.4554640     21.5867960
```

SCALES; ABSCISSAS 0 10 FT 20 FT

ORDINATES 2 FT-K, 1 FT-K, 0 FT-K

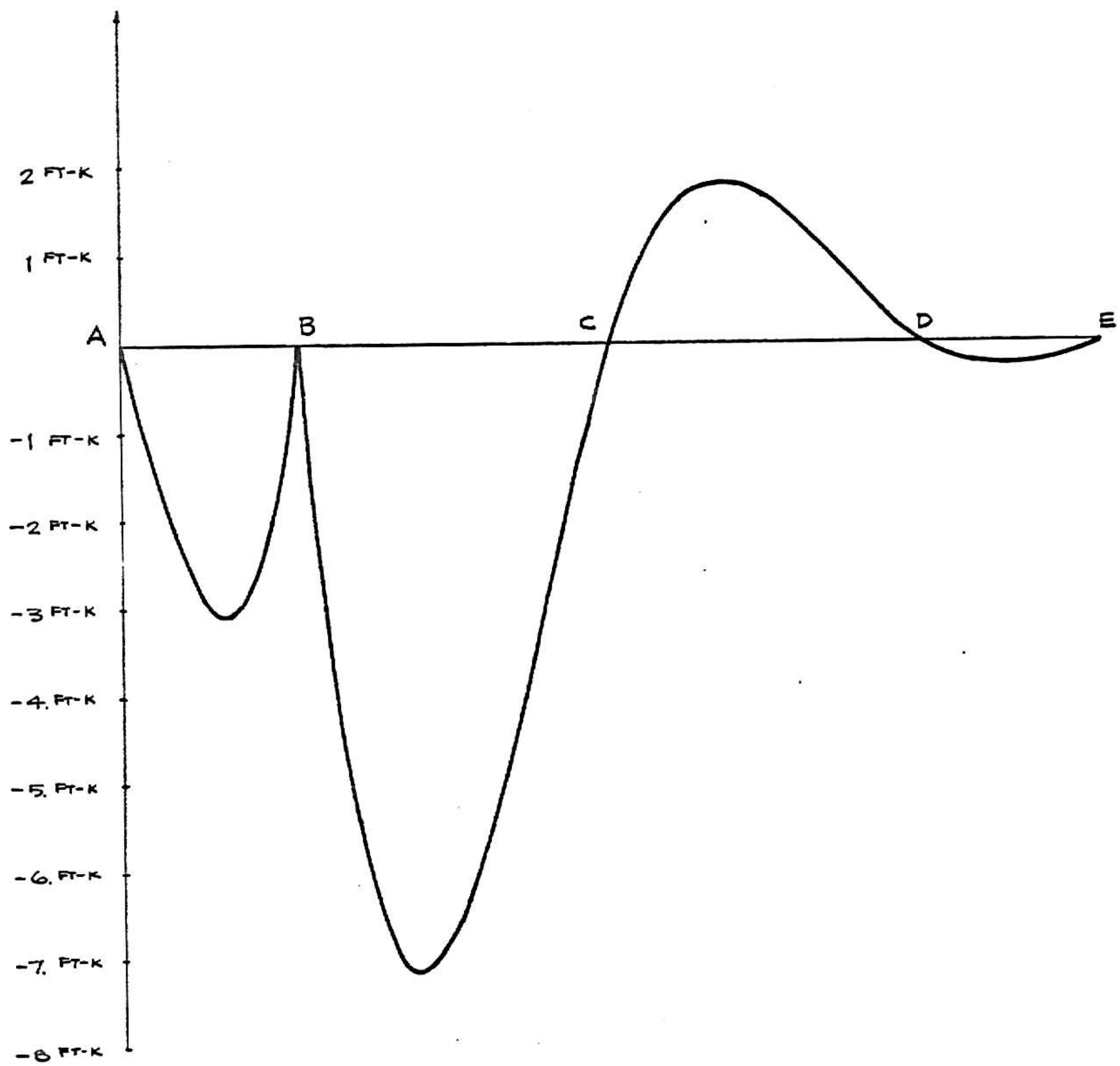


FIG. 29. INFLUENCE LINE FOR MOMENT AT SUPPORT B, MB.

SCALES; SAME AS FIG. 29

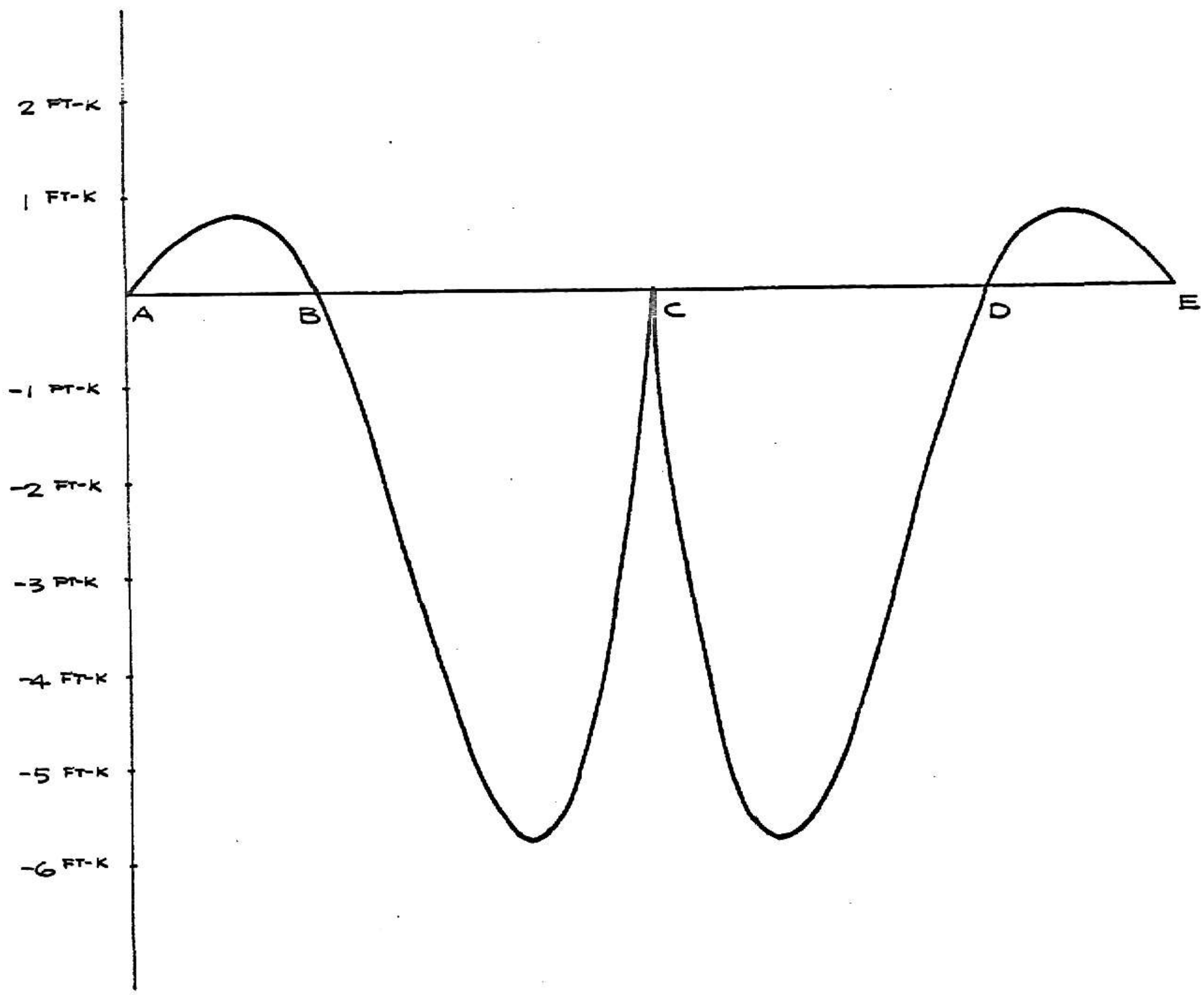


FIG. 30. INFLUENCE LINE FOR MOMENT AT SUPPORT C, MC.

SCALES; SAME AS FIG. 29.

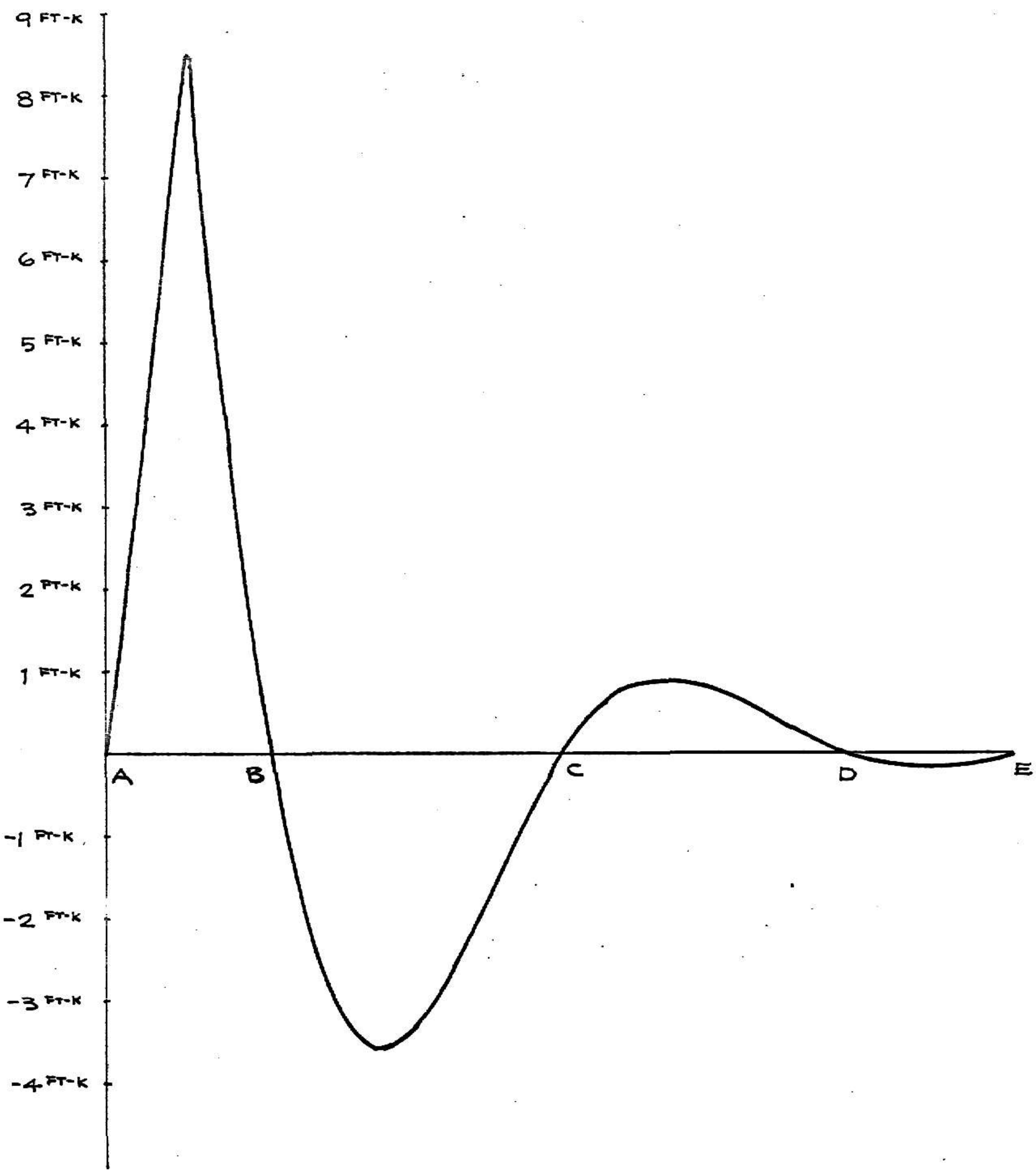


FIG. 31. INFLUENCE LINE FOR POSITIVE MOMENT AT CENTER OF SPAN AB, M1.

SCALES; SAME AS FIG.29.

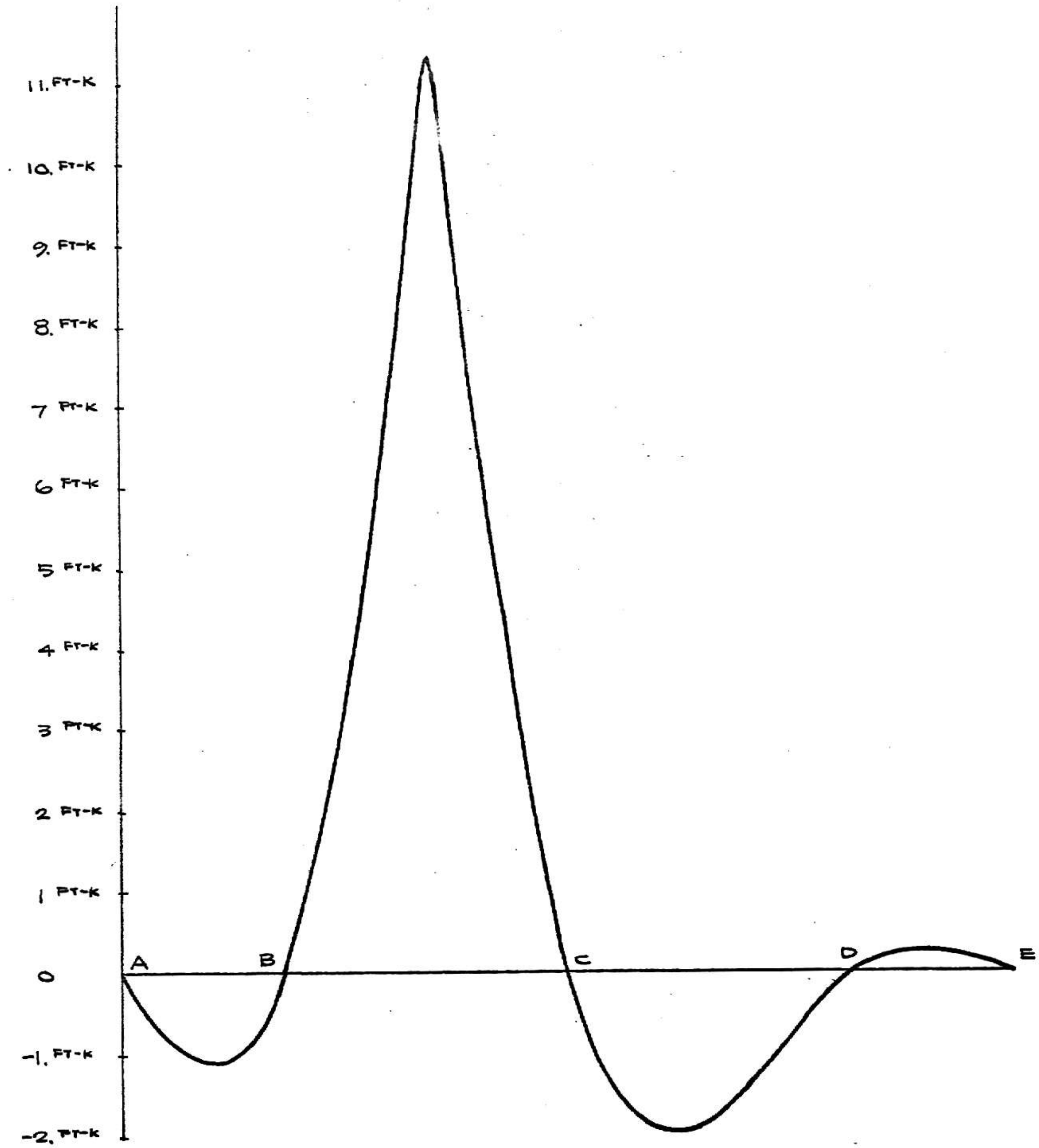


FIG.32. INFLUENCE LINE FOR POSITIVE MOMENT AT CENTER OF SPAN BC, M2.

THE DESIGN PROCEDURES OF CONTINUOUS BRIDGE WITH PRECAST, PRESTRESSED CONCRETE GIRDER

by

MING-LEE CHANG

Diploma, Taipei Institute of Technology, 1963

A MASTER'S REPORT

submitted in partial fulfillment of the

requirement for the degree

MASTER OF SCIENCE

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1970

In recent years, continuous highway bridges with precast-prestressed concrete girders have been used in many countries. The primary advantages of this type of bridge are the elimination of maintenance costs of deck joints and deck drainage onto the substructure and the structural economy of continuous design. Continuity also can improve the appearance and riding qualities of a bridge.

There are many different kinds of construction methods available to achieve continuity of precast-prestressed concrete bridge girders or slabs. In this report, continuity is achieved for live load plus impact moment by using nonprestressed reinforcement in the deck slab in the diaphragms over piers.

The effects of shrinkage and creep between the precast girder and the cast-in-place deck slab are more important than in other types of construction of continuous prestressed concrete bridges. The Research and Development Laboratory of the Portland Cement Association completed a research program on this kind of bridge during 1960-1961. Then the results of their research provided available information for designing a continuous precast-prestressed concrete bridge considering the effects of shrinkage and creep.

The main parts of this report are the design procedures for continuous precast-prestressed concrete bridges, including the influence of shrinkage and creep, and a design example of a four-span continuous bridge to illustrate the procedures.