ECONOMIC COMPARISON OF HIGHWAY BRIDGE TYPES FOR FIFTY TO ONE-HUNDRED FOOT SPANS

Thesis for the Degree of M. S.

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Shantilal Ambalal Patel

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ntilal Ambalal Patel

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of the requirements for

M.S. degree in Civil Engine ring

Carl L. Shermer
Major professor
Carl L. Shermer

ate May 5, 1954

ECONOMIC COMPARISON OF HIGHWAY BRIDGE TYPES

FOR FIFTY TO ONE-HUNDRED FOOT SPANS

By

Shantilal Ambalal Patel

A THESIS

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I. INTRODUCTION

Highway Bridge Engineering can be divided as follows:

A. Bridge location

B. Preliminary investigational work and stream study

C. Economic type selection

D. Detailed design

E. Construction

F. Maintenance and operation

In general it may be said that many of the highway bridges span from 50' to 100'. Therefore, it is the purpose of this thesis to make an economic study of highway bridge types for these spans including actual cost comparison.

It is needless to say that type selection is the most difficult but the most important feature of Bridge Engineering. Yet, the engineers in general have not quite appreciated the importance of correct type selection. As a result, millions of dollars have been wasted through improper type adaption due to unwarranted first costs or unnecessary maintenance expenses.

Correct type selection is, therefore, the very corner stone of economy. A failure to evaluate correctly the factors governing this problem may frequently result in waste many times greater than any saving anticipated from refinements in stress analysis and design.

II. FACTORS DICTATING TYPE SELECTION

It is necessary to discuss the general factors that govern the type selection for any span before discussing fifty to one-hundred-foot spans. The question of economy in the first cost, maintenance and renewal is naturally a major controlling consideration. It should be kept in mind that there are some factors other than economic considerations which operate to dictate type selection. These may be grouped as follows:

A. Stream behavior

B. Requirements of navigation

C. Traffic considerations.

D. Architectural features

This is not the place for an extended discussion of these, however, these will be introduced briefly.

A. Stream Behavior

Stream behavior in this thesis means the peculiar characteristics of the waterway during periods of high waters as regards erosion of bed and banks, lateral shifting of channels, carriage of drift, ice and debris, etc. These characteristics have direct influence on (1) main channel structure, (2) structural approaches, (3) approach embankment.

We know that $Q = VA$ where Q = discharge of stream in ft. sec. through the main channel.

A = Waterway area of main channel ft^2.

V = Velocity of flow in ft./sec.

Construction of any structure across the stream tends to obstruct the flow of water. This, naturally, affects the velocity of flow and causes erosion or deposition of bed. As a result this characteristic puts limit on minimum spacing of piers.

Meandering of stream in flood plains many times make it necessary to carry main channel construction clear across the flood plain, where otherwise, short span approach construction would be entirely adequate and much more economical. In such cases, stabilization of channel by means of river training works may sometimes be economical against long span construction over the entire flood plain.

This automatically dictates the structural approaches as well as the approach embankment.

The necessity to guard against the drift or ice dictates the vertical clearance, although extreme flood elevation alone may at times be the limiting factor. These factors often force the elimination of deck truss construction even though it is the most desirable.

Shallow truss construction, enough to provide required clearance, may sometimes be more economical than deck truss of ordinary depth which requires raising the general grade of the approaches.

B. Requirements of Navigation

The horizontal and vertical clearances required for the main channel span to fulfill navigation needs certainly limit the type selection. Generally moveable spans are used for this purpose, of course, the type of design for the moving leaves is dictated by water traffic. In many of the streams in this country even the flanking spans are held to a certain established minimum as regards these clearances by the United States War Department. Thus, they control the spacing of piers regardless of other considerations.

C. Traffic Considerations

In order to allow adequate sight distance at sharp curved approaches structure such as through steel truss construction should be avoided.

The ultimate purpose of bridge structure is to facilitate traffic movement. Hence thorough consideration should be given to (1) the direction of traffic movements over the span, (2) provision for the separation of slow and fast moving traffic.

Deck construction is much superior to any other type from a traffic standpoint as it satisfies all the above requirements. If the clearance requirements eliminate this type from consideration on proposed grade line, then, it becomes necessary to investigate the feasibility and cost of a new grade line modified so as to permit deck construction and also the feasibility of special shallow truss design.

D. Architectural Features

In certain cases architectural requirements play an important role in the selection of Bridge Structures. The following can be the order from the architectural point of view.

1. Masonry arch construction
2. Reinforced concrete deck construction
3. Deck truss or plate girder construction with a concrete deck.

4. Through truss or girder construction. If the alignment is such that the side elevation of structure is plainly visible from the approaching highway, more attention should be given to a type which gives a pleasing side elevation than if only the roadway is ordinarily visible. Natural scenic setting will also influence the type selection. Furthermore, the location of the bridge in reference to parks, pleasure resorts, etc. will influence the type selection.

In general, all such factors operate to place certain limits on type selection. These limits are generally in regard to the spacing of piers, type of approach construction, choice between deck and through construction, grades, clearances, roadway width, etc. In final analysis when the relative economy of the different types is determined, the unobstructed roadway should be given a great deal of consideration and should be chosen unless cost considerations are prohibitive.

III. TYPE SELECTION FOR 50' to 100' SPANS

Assuming all the factors that dictate the type selection are favorable to the use of a deck structure, then the present trend is to the type of construction using steel beams with concrete floors for these span lengths. Recent developments to provide mechanical shear developers have caused the discarding of the conventional steel construction. As a result state highway departments are designing composite steel bridges.

On the other hand, for 50' to 100' spans the conventional reinforced concrete construction cannot be used owing to its limitations in regard to span length. But recent development has brought about the use of prestressed concrete construction. This has increased the previous span limit. Hence, the prestressed concrete bridges can be built for these spans. How far this type of construction can compete with the present composite construction can now be found out by actual cost comparison. However, it is assumed that the sub-structure construction is the same in both the cases. Hence, it is now possible to determine cost differences for these two types of deck constructions for the 50' to 100' spans. Unit costs for the composite construction were supplied to the author by the Michigan State Highway Department, Lansing, Michigan. Unit costs of the prestressed construction were supplied by the specialized firms.

The order of the following discussion is:

A. Composite construction

B. Prestressed concrete construction

A. COMPOSITE CONSTRUCTION

1. GENERAL

a. Introduction

During the last twenty years composite construction has replaced the conventional steel construction in the field of highway bridges. This is because, in the conventional type of steel beam bridge with a reinforced concrete deck, each beam carries the entire load transmitted to it by the concrete slab. While in composite construction, adequate provision of shear connection between the beam and the slab has made the section composite and hence acts similar to the reinforced concrete T-beams.

The main advantages of composite construction over the conventional type of steel beam and slab construction are:

(i). Greater economy

(ii). Shallower construction

(iii). Greater stiffness

(iv). Greater live load factor of safety

b. Composite Beam

The composite beam consists of three essential parts:

(i). Steel section: this may be a rolled beam or a riveted or welded girder. Composite construction is more economical when the bottom flange of the steel section is larger than the top flange. This can be accomplished in the case of a standard rolled beam by welding a cover plate at the bottom.

(ii). Concrete slab: reinforced concrete slab is designed as in conventional construction. The slab may rest on the steel beam or on concrete haunch increasing the carrying capacity of the composite section due to the increased depth.

(iii). Shear connectors: to provide adequate bond between the concrete slab and the steel beam, mechanical shear connectors are provided. Among the many, the spirals are the most economical and adaptable. These are normally welded to the top flange of the steel beam and embedded in the concrete to transmit the longitudinal shear and to prevent the movement between the slab and the beam.

c. Methods of Erection

Two different methods of erection are in practice:

(i). No temporary supports are used under the steel beams during the pouring and setting of concrete slab. So steel section carries the dead load while the composite section carries the live load plus any dead load applied after the slab has set e.g. wearing surface.

(ii). Effective temporary supports are used under the steel beams during the pouring and setting of the concrete slab, hence the composite section carries both the dead and live loads.

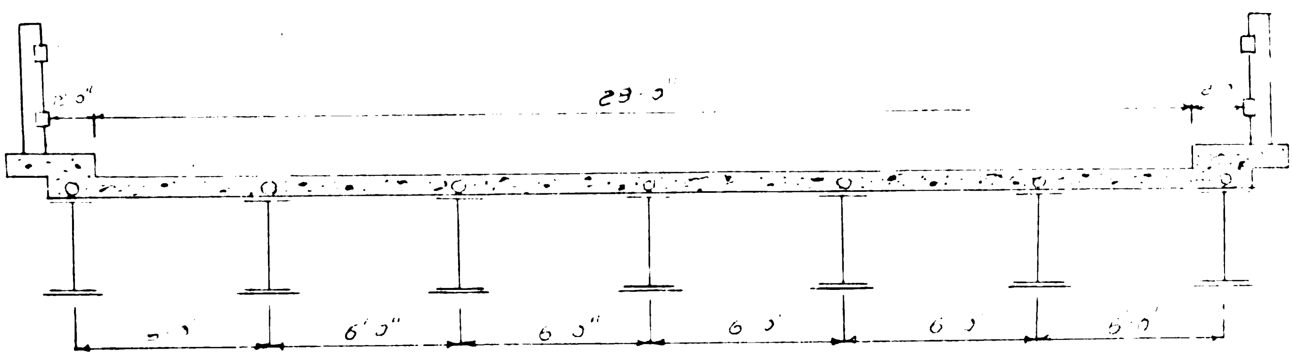
However, practice has proved the first method as more economical than the second, though the steel section required for the second method is smaller than that for the first method. This is because the cost of the intermediate supports frequently exceeds the saving in the steel.

In the following designs, all of the above main factors that bring about economy are taken into consideration.

The method of design of composite beams is based on the transformed section method used in the design of reinforced concrete beams. For the trial sections, the properties are used from the tables given in Reference No. 2.

d. Data

The general layout of the cross-section is as shown in the following Figure 1. The roadway is 28' wide and is flanked by two 2-ft. wide curb walks. The stringer spacing is kept 6 ft. c. to c. for all the spans. The slab is kept the same for all the spans.



CROSS-SECTION OF DECK

Fig. 1

Live load is H20-44

Future wearing surface is assumed to be 20 lbs. per square foot.

Load and design requirements adhere in general to Standard Specifications for Highway Bridges, sixth edition, 1953 adopted by AASHO.

e. Notations

kd	=	Distance from compression surface of slab to neutral axis in inches.
L	=	Span length in feet.
As	=	Cross-sectional area of steel section.
I'	=	Moment of inertia of steel section about its NA.
I	=	Moment of inertia of composite section about its NA.
S's	=	Tensile section modulus of the steel section.
Ss	=	Tensile section of modulus of the composite section.
n	=	Ratio of modulus of elasticity of steel to that of concrete (Es/Ec).
MDL	=	Bending moment due to dead load.
MLL	=	Bending moment due to live load.
Mws	=	Bending moment due to wearing surface.
Fsdl	=	Steel stress due to MDL. (Ksi).
Fsll	=	Steel stress due to MLL (Ksi).
Fsws	=	Steel stress due to Mws (Ksi).
VDL	=	Vertical shear due to dead load.
VLL	=	Vertical shear due to live load.
Vws	=	Vertical shear due to wearing surface.
H	=	Horizontal shear per unit length.
S	=	Pitch of spirals.

Apl = Cross-sectional area of plate.

Qpl = Static moment of cover plate about NA of composite section with n = 10.

Q'pl = Static moment of cover plate about NA of steel section.

Q"pl = Static moment of cover plate about NA of composite section with n = 30.

Q = Static moment of the effective concrete area about NA of composite section.

Fw = Shearing force transmitted per pitch (Kips).

Fsa = Allowable steel stress = 18 (Ksi).

2. FLOOR SLAB

a. Design

Distance between stringers c. to c. = 6 ft.

Bending Moment shall be calculated by the methods based on the Westergaard's theory of the distribution of loads and design of concrete section.

According to AASHO specifications, Art. 3.3.2. (c) case A, where main reinforcement is perpendicular to traffic and spans are 2-7 ft.

distribution of wheel loads:

$E = 0.6s + 2.5$

where s = effective span length which is the distance between edges of flanges plus 1/2 of the stringer flange width for slabs supported on stringers. AASHO specifications. Art. 3.3.2. (a)

Assuming 9" wide flange,

$s = 6' - 0" - 9" + 4.5" = 5.62'$

$E = 0.6s + 2.5$

$= 0.6 \times 5.62 + 2.5$

$= 3.37 + 2.5$

$= 5.87'$

Bending moment for continuous spans:

$M = \pm \dfrac{0.2 \times P_1 \times s}{E}$ where P_1 = Load on one wheel of single axle.

P_1 = 12,000 lbs. for H20 - 44 loading.

$M = \pm (0.2 \times 12{,}000 \times 5.62) \quad 5.87 \times 1000$

$= \pm 2.34$ k. ft.

Impact factor = $\frac{50}{L + 125}$ (L = s = 5.62')

$= \frac{50}{5.62} - 125$

= 38.4%

But max. allowed is 30%

Design M = ± 3.04 k. ft.

for 20,000/10/1200, and M = 3.04 k. ft. d = 4".

Minimum slab thickness = 4 + 1.5 = 5.5"

Use 7" thick slab.

Hence, d = 7 - 1.5 - 0.37 = 5.2".

Reinforcement:

$= A_s = \frac{M}{f_s j d} = \frac{3.04 \times 12}{20 \times .867 \times 5.2}$

= 0.405 sq. in./ft.

Provide 5/8" Ø Bars @ 7" c. to c. at bottom as well as at the top.

Distribution Reinforcement: reinforcement shall be placed in the bottom of all slabs normal to main steel to provide for lateral distribution of all loads. The amount shall be the percentage of the main reinforcing steel required for positive moment as given by the following formula:

$\% = 100 s^{\frac{1}{2}} = 100/5.62^{\frac{1}{2}} = 42.2\%$

Area of steel regd. = .422 x .402 = 0.17

Provide 1/2" Ø Bars @ 10" centers.

b. Quantity:

concrete/sq. ft. = $\frac{7}{12}$ x 1 = 0.583 cu. ft.

Positive steel/sq. ft. $= \frac{1.043 \times 12}{7} = 1.79$ lbs.

Negative steel/sq. ft. $= 1.79$ lbs.

Distribution steel/sq. ft. $= \frac{.668 \times 12}{10} = 0.8$ lbs.

c. **Cost:**

Per sq. ft. of slab:

concrete @ \$48/cu. yd. $= \frac{0.583 \times 48}{27} = \$1.04$

steel reinf. @ \$0.10/lb. $= (1.79 - 1.79 - 0.8) \times .10$

$= 3.38/.10 = 33.8$ cents

Cost of slab per sq. ft. = \$1.378

3. COMPOSITE BEAM

a. Design

The following example illustrates the detailed design of a typical composite beam. The span is 60' as simply supported.

Loading H20 - 44

Stringer spacing 6'

Slab thickness 7"

Wearing surface 20 lbs./sq. ft.

Live Load: (Lane load governs)

Distributed load/ft. of lane = 640 lbs.

Impact factor = $\frac{50}{60 + 125}$ = 27%

Impact load/ft. of lane = 174 lbs.

Total load/ft. of lane = 814 lbs.

Per stringer, load/ft. = $\frac{6}{10}$ x 814 lbs. = 488 lbs.

i.e. WLL = .488 k

Concentrated load for moment/lane = 18 k

Impact load = 4.86 k

Effective concentrated load for moment/lane = 22.86 k

Per stringer concentrated load P = 0.6/22.86

Concentrated load for shear/lane = 26 k

Impact load = 7 k

Effective concentrated load for shear/lane = 33 k

Per stringer concentrated load = 0.6 x 33

= 19.8 k

Live Load Moment $M_{LL} = \dfrac{.488 \times 60^2}{8} + \dfrac{13.7 \times 60}{4}$

$= 423$ k ft.

Dead load carried by steel section alone:

Wt. of slab $= \dfrac{7}{12} \times 150 \times 6 = 525$ lbs.

Steel section = 120 lbs.

Diaphrams, spirals, welds, = 20 lbs.

i.s. WDL = 665 lbs./ft.

Dead load Moment $M_{DL} = \dfrac{.665 \times 60^2}{8} = 299$ k. ft.

Dead load carried by Composite Section (n = 30): for the loads which are superimposed after the concrete has set, stresses shall be computed with n = 30 for superimposed deal load and with n = 10 for live load. This higher value of n is supposed to take care of the combined effect of dead load and plastic flow stresses in the concrete and steel.

Wt. wearing surface = 20 x 6 = 120 lbs./ft.

Wearing Surface Moment $M_{WS} = \dfrac{.120 \times 60^2}{8}$

$= 54$ k. ft.

Trial Design:

Wherever plastic flow is considered in the design of composite beams, its effect may be neglected in the trial design because of its small influence on the required section. Therefore, loads producing plastic flow are grouped with the live load in the trial design procedure.

Steel section modulus required for dead load

$$= S'sdl = \frac{MDL \times 12}{fs} = \frac{299 \times 12}{18} = 200 \text{ in}^3$$

Steel section modulus required for WLL + Wss

$$= Ssll = \frac{(423 + 54) \times 12}{18} = 477 \times 12$$
$$= 318 \text{ in}^3$$

Composite Design with Cover Plate:

To select the required composite section, try values from Trial Design Table V of Reference No. 2 for the section 27wf. @ 94.

$$\text{Total Steel area reqd.} = As = \frac{S'sdl}{S's/As} + \frac{Ssll}{Ss/As}$$

$$= \frac{200}{8.78} + \frac{318}{13 \times 0.978}$$

$$= 22.8 + 25.0 = 47.8 \text{ sq. in.}$$

Now 27 wf. @ 94 give steel area = 26.65 sq. in.

Therefore,

$$\text{Area of Plate} = Apl = \frac{As - 27.65}{2}$$

$$= \frac{47.8 - 27.65}{2} = 10.75 \text{ sq. in.}$$

Use a slightly larger cover plate than determined as above, since it was assumed in the trial design that the load carried by the composite section with n = 30 was carried by the section with n = 10.

Therefore use a 27 WF. @ 94 with a 12" x 15/15" (Apl = 11.25 sq. in.) cover plate at bottom.

Properties of section without cover plate (at supports).

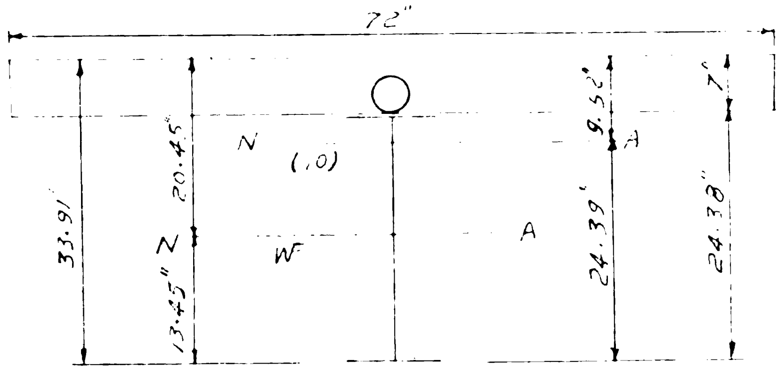


Fig 2

Steel Section:

Area = 27.65 sq. in.

$I'o$ = 3264 in^4.

$S'so$ = 243 in^3.

Composite Section (n = 10)

Area: A_1 = 7.2 x 7 = 50.4

A_2 = steel = <u>27.65</u>

Total area = 78.05 sq. in.

Statical Moment:

A_1 x 3.5 = 177

A_2 x 20.45 = <u>565</u>

Total = 742 in^3.

kd = 742/78.05 = 9.52" from top of slab.

Moment of Inertia:

$\frac{1}{12}$ x 7.2 x 7^3 = 205

50.4 x $(6.02)^2$ = 1,870

WF. 27 = 3,267

27.65 x $(10.93)^2$ = 3,220

Total = Io = 8,662 in^4

Modulus of Section Sso = 8,662/24.38 = 355 in^3

Composite Section (n = 30)

Area:

2.4 x 7 = 16.8

WF 27 = 27.65

Total area = 44.45 sq. in.

Statical Moment:

16.8 x 3.5 = 59

27.65 x 20.45 = 565

Total = 624 in^3

kd = 624 x 44.45 = 14" from top of slab.

Moment of Inertia:

$\frac{1}{12}$ x 2.4 x 7^3 = 68.4

16.8 x $(11.5)^2$ = 2,220.0

WF 27 = 3,267.0

27.65 x $(6.45)^2$ = 1,150.0

Total = I"o = 6,705.4 in^4

Modulus of Section = S"o = 6705.4/19.9 = 336 in^3

Properties of Section with cover plate (at mid-span)

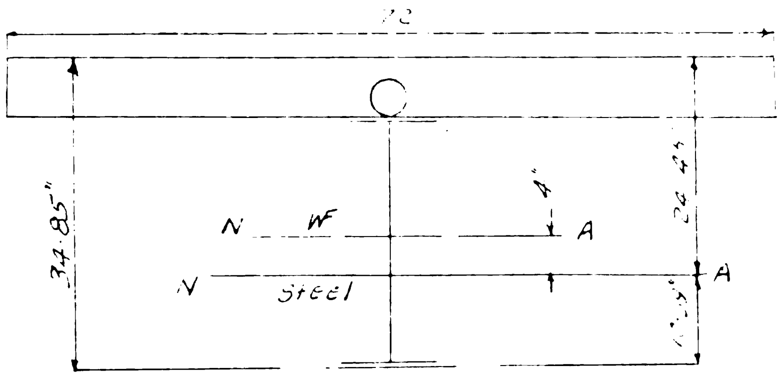


Fig. 3

Change in Neutral Axis due to adding a cover plate can be determined by the following formula:

Change in NA = (Area of plate) x (Distance from c.g. of plate to NA of section without plate) / (Equivalent steel area of entire section).

Steel Section:

$$NA = \frac{11.25 \times 13.92}{38.9} = 4.0''$$

Moment of inertia:

27 WF section	=	3,267
2765×4^2	=	432
11.25×9.92^2	=	1,120
Total = I'	=	4,819 in^4

Modulus of Section = S's = 4819/10.39 = 472 in^3

Composite Section (n = 10)

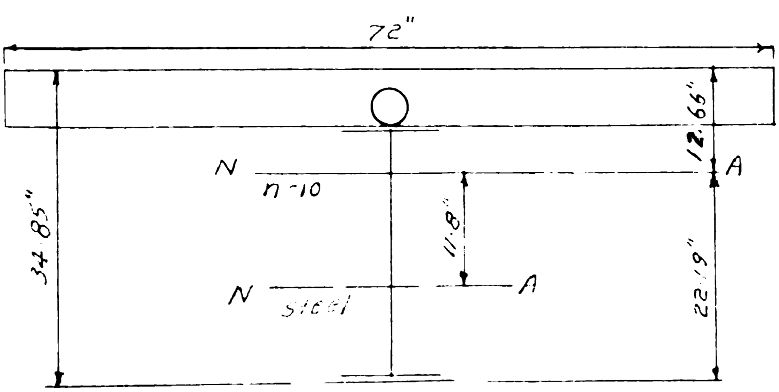


Fig. 4

NA (By adding slab to steel section) =

$$\frac{50.4 \times 20.96}{893} = 11.8''$$

Moment of Inertia

$\frac{1}{12} \times 7.2 \times 7^3 = 205$

$(50.4) \times (9.16)^2 = 4{,}230$

Steel I' = 4,863

$38.9 \times (11.8)^2 = \underline{5{,}420}$

Total = I = 14,718 in^4

Modulus of Section Ss $= \frac{14718}{22.19} = 663\ in^3$

Composite Section (n = 30)

NA (By adding slab to steel section) $= \frac{16.8 \times 20.96}{55.7} = 6.3"$

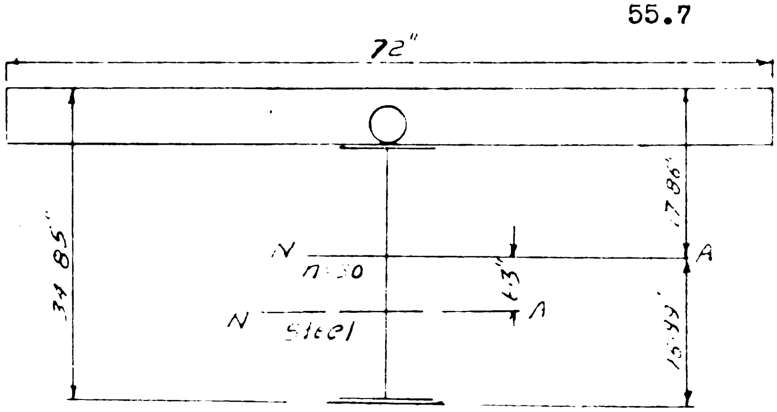


Fig 5

Motion of Inertia:

$\frac{1}{12} \times 2.4 \times (7)^3 = 68.4$

$16.8 \times (14.36)^2 = 3{,}480.0$

Steel I = 4,863.0

$38.9 \times (6.3)^2 = \underline{1{,}540.0}$

Total = I" = 9,951.4 in^4

Modulus of Section = S"s $\frac{9951.4}{16.99} = 585\ in^3$

Grouping together:

Section	Moment of Inertia in^4	NA	Section Modulii
Steel	$I' = 4{,}819$	24.46" 10.39"	$S's = 472\ in^3$
Composite (n = 10)	$I = 14{,}718$	17.86" 22.19"	$Ss = 663\ in^3$
Composite	$I'' = 9{,}951.4$	17.86" 16.99"	$S''s = 585\ in^3$

Unit stresses at center:

		Bottom of steel	Top of steel
FSDL	$= \frac{299 \times 10.39 \times 12}{4819} =$	7.85 k/in^2	13.9
FSLL	$= \frac{423 \times 12 \times 22.19}{14718} =$	7.75 k/in^2	2.0
Fsws	$= \frac{54 \times 12 \times 16.99}{9951.4} =$	1.11 k/in^2	0.77
	Total =	16.71 k/in^2	16.67 k/in^2

Top of Concrete:

Due to L.L. $\frac{432 \times 12 \times 12.66}{10 \times 14718} = 0.436\ k/in^2$

Due to W.S. $\frac{54 \times 12 \times 17.86}{30 \times 9951.4} = .039\ k/in^2$

Total $= 0.575\ k/in^2$

All stresses are within the safe allowable stresses. Hence, the section is safe.

Length of Cover Plate:

$$= L' = L\sqrt{1 - \frac{Sso}{Ss}}$$

$$= 60\sqrt{1 - \frac{355}{663}} = 60\sqrt{(1 - .535)}$$

$$= 40.9'$$

Use 42' cover plate.

End weld on cover plate:

The end force in cover plate $= A_{pl} \times F_{sa} \times \dfrac{S_{so}}{S_s}$

$$= 11.25 \times 18 \times 355/663$$

$$= 113.5 \text{ kips.}$$

Required length of 5/16" weld to transmit this end force

$$\text{only} = \frac{113.5}{2.21} = 50 \text{ in.}$$

Welds along sides of cover plate:

To find horizontal shear per inch at the end of plate:

At 9' from left support

$$V_{DL} = 0.665\,(30 - 9) = 0.665 \times 21 = 14 \text{ k.}$$

$$V_{LL} = 0.488 \times \frac{51}{60} \times \frac{51}{2} + \frac{19.8 \times 51}{60} = 10.6 + 16.8 = 27.4 \text{ k.}$$

$$V_{WS} = 0.120\,(30 - 9) = 0.120 \times 21 = 2.52 \text{ k.}$$

Then horizontal shear per inch can be given as:

$$H = \frac{V_{DL}\ Q'_{pl}}{I'} + \frac{V_{LL}\ Q_{pl}}{I} + \frac{V_{WS}\ Q''_{pl}}{I_1''}$$

$$= \frac{14(11.25 \times 9.92)}{4819} + \frac{27.4(11.25 \times 21.72)}{14718} + \frac{2.52(11.25 \times 16.52)}{9951.4}$$

$$= 0.324 + 0.456 + 0.047$$

$$= 0.827 \text{ k/in or } 0.413 \text{ k/in per side.}$$

Try 1/4" intermittent welds, length of each being = 1 1/2"

Then strength = $1.77 \times 1.5 = 2.65$ k.

Therefore:

$$\text{spacing} = \frac{2.65}{0.413} = 6.5''$$

By AWS specifications the minimum weld length = 1-1/2" and the maximum clear spacing = 14 x thickness of thinner part joined = 14 x 11/16 = 9.63"

Maximum center to center spacing = 11.13". Therefore, the spacing could be increased to 11", near the center of the span.

Spiral Shear Connectors:

Use 5/8" ⌀ bars with 4-1/2" mean coil diameter.

This gives 7" - (4-1/2 + 5/8) = 1-7/8" cover over spirals.

At end supports:

$$V(DL + WS) = (0.665 + 0.12) \times 30 = 23.5 \text{ k.}$$

$$VLL = 0.488 \times 30 + 19.8 = \underline{34.4} \text{ k.}$$

$$\text{Total shear } 57.9 \text{ k.}$$

Spiral pitch,

$$S = \frac{Fw\, I}{VQ} = \frac{11.04 \times 8662}{57.9(50.4)(6.02)}$$

At 10 ft. from the support:

$$V(DL + WS) = 0.785\ (30 - 10) = 15.7 \text{ k.}$$

$$VLL = 0.488 \times \frac{50}{60} \times \frac{50}{2} + 19.8 \times \frac{50}{60} = \underline{26.7} \text{ k.}$$

$$\text{Total shear} = 42.4 \text{ k.}$$

As the cover plate exists in this zone, the properties of the section with cover plate will be used.

Spiral pitch,

$$S = \frac{11.04 \times 14718}{42.4 \times 50.5 \times 9.16} = 8.32\text{", say } 8\text{"}$$

At 20 ft. from support:

$$V(DL + WS) = 0.785 \times (30 - 20) = 7.85 \text{ k.}$$

$$VLL = 0.488 \times \frac{40}{60} \times \frac{40}{2} + 19.8 \times \frac{40}{60} = 19.7 \text{ k.}$$

Total shear = 27.55 k.

Spiral pitch,

$$S = \frac{11.04}{27.55} \times \frac{14718}{(50.4)(9.16)}$$

$$= 12.8"$$

say 12-1/2"

Spiral	Pitch	Spiral length	Weight
S_0	5-1/2"	10' - 9"	29.8 lbs.
S_{10}	8"	10' - 9"	21.9 lbs.
S_{20}	12-1/2"	10' - 9"	17.2 lbs.
		Total =	68.9 lbs.

Average weight of spiral per foot of beam $= \frac{68.9}{30} = 23$ lbs.

Live load deflection:

The theoretical live load deflection is computed by using n = 8 and by considering the change in section due to adding a cover plate. However, sufficiently close results are obtained by using n = 10 and the properties of the section at mid-span only.

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{36\ PL^3}{EI}$$

$$= \frac{22.5 \times .488 \times 60^4}{29,000 \times 14,718} + \frac{36 \times 13.7 \times 60^3}{29,000 \times 14,718}$$

$$= .332 + .25$$

$$= 0.582 \text{ in.}$$

$$\frac{\Delta}{L} = \frac{0.582}{60 \times 12} = \frac{1}{1,230}$$

This is safe because maximum deflection is allowed up to $\frac{1}{800}$ of span.

Factor of Safety:

Factor of safety is given for composite construction based on the bottom flange stresses, which governs the design.

Minimum yield point stress	=	33 ksi
Dead load stress	=	8.96 ksi
Therefore, stress available for LL	=	24.04 ksi

$$\text{Factor of Safety} = \frac{\text{stress available for LL}}{\text{LL stress}}$$

$$= \frac{24.04}{775} = 3.1$$

Diaphragm:

The diaphragms are generally tentatively provided for the better distribution of the live load to the various girders. They also make the structure more rigid to withstand unknown forces such as traction or sudden braking. Hence the diaphragm shall consist of two Ls 3"x3"x 3/8" with a 3/8" plate. The depth of the diaphragm shall be such as to provide 4" clearance from the top flange of the section and 3" to 4" from the bottom flange. The maximum spacing shall not exceed 20 ft. Hence provide two diaphragms, each having two Ls 3"x3"x 3/8" with 19"x 3/8" plate.

b. Quantity:

(i) Structural steel:

6 Nos. 27" WF. @ 94 for 60 ft.	= 94x6x60	=	33,840.0	lbs.
6 Nos. 12"x 15/16" plate 42' long	= 38.3x6x42	=	9,651.6	lbs.
4Ls 3"x3"x 3/8" - 30' long	= 7.2x4x30	=	864.0	lbs.
2 Nos. 19"x 3/8" plate - 30' long	= 24.2x2x30	=	1,452.0	lbs.
	Total	=	45,807.6	lbs.

(ii) Spiral shear developers for 6 beams 60' long =

2.3 x 6 x 60 = 828 lbs.

c. Cost:

(i) Structural steel (fabrication, erection, painting) @

$0.135 per lb. = 45,807.6 x .135 = $6,184.03

(ii) Spiral shear developers @ $0.50 per lb. = 828 x 0.50 = $414.00

(iii) 7" slab 28" x 60' @ $1.378 per sq. ft. = $2,320.00

Therefore, total cost of deck = $8,918.03

B. PRESTRESSED CONCRETE CONSTRUCTION

1. GENERAL

a. Introduction

Conventional reinforced concrete enjoys an enviable position and can be used successfully under most circumstances. But it has also its limitations in regard to span, load or height. But recent development has brought the prestressed concrete construction into practice. This breaks down previous limitations on spans and loads.

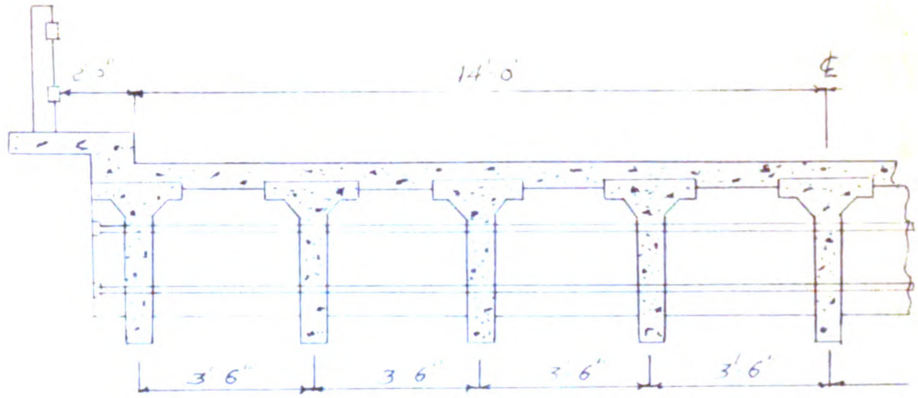
Moreover prestressed designs have resulted in great savings of material. The prestressing tends to cancel the tensile stresses in the concrete and so the compressive stresses that occur over the entire cross-section tend to prevent the occurrence of cracks. The arrangement of reinforcement is also simplified considerable, because the diagonal tension is considerably less than in the conventional design.

All of these advantages over the conventional concrete construction have made the prestressed concrete construction economical.

With this brief introduction the prestressed decks for the same spans and road width as before will be designed.

b. Data:

The general layout of the cross-section of the deck is as shown in the following Figure No. 6. The roadway is 28' wide and flanked by two 2' wide curbs.



HALF CROSS-SECTION OF DECK

Fig. 6

The deck consists of prestressed girders spaced 3.5 ft. on centers. The cross-section of the girder is of T-shape. The conventional T-shape appears to be particularly advantageous from the viewpoint of simple form construction.

The cast in place concrete slab is 4 in. thick above the girder flanges and 7 in. thick between them. Stirrups are extended above the top surface of the flanges and are spliced in the slab concrete. This has enabled it to assume composite action between slab and girder.

The girders are post-tensioned by cables comprising a group of nontwisted wires with ends anchored mechanically. The cables are placed in metal tubing with 1-1/4" outside diameter. The tubes are finally filled with grout so as to provide sufficient bond.

c. Specifications:

Load and design requirements shall adhere in general to

Standard Specifications for Highway Bridges, sixth edition, 1953, adopted by AASHO. But as AASHO specifications have not considered prestressed concrete, these will be supplemented by the "Design Criteria for Prestressed Concrete", published by the Bureau of Public Roads, on March 10, 1952.

Live load is H-20-44.

Concrete:

Compressive strength f'c = 4,000 psi

Initial allowable compression in extreme fiber = 2,000 psi

Initial allowable tension in extreme fiber = 160 psi

Final allowable compression in extreme fiber = 1,600 psi

Final allowable tension in extreme fiber = 0 psi

Allowable tension in bottom fiber at cracking load = 600 psi

cracking load shall be = 1.0 DL + 2.0 LL.

Ultimate load shall not be less than 2.25 (DL + LL) or (1.0 DL + 3.5 LL) whichever is greater.

Allowable principle tensile stress at design load = 160 psi

Allowable principle tensile stress at ultimate load = 300 psi

Minimum web reinforcement of 3/8" diameter stirrups spaced at a distance not more than half the depth of girder shall be provided.

Steel:

Tensile strength of the steel wire = 250,000 psi

Allowable initial prestress = 150,000 psi

Allowable final prestress = 0.85 (150,000) = 127,500 psi

The size of wire shall be between and including 0.192 and 0.276 in diameter.

2. FLOOR SLAB

a. Design

As mentioned before, the slab thickness is tentatively provided 7 in. between the girders.

It will be noticed that clear distance between flanges is 1' 6" for all spans and the width of the flange is 2 ft. for all girders.

Therefore, $s = 1.5 + \frac{1}{2}(2) = 2.5'$

and $E = 0.6s + 2.5 = 0.6 \times 2.5 - 2.5$

$= 1.5 + 2.5 = 4$ ft.

Moment $= \pm\ 0.2\ \frac{P_1}{E}\ S$ AASHO 3.3.2. (c) Case A

$= \pm\ \frac{0.2 \times 12{,}000 \times 2.5}{4}$ (Wheel load P_1 = 12,000 lb.)

$= \pm$ 1500 ft. lbs. = = 18,000 in lbs.

Main Reinforcement $= \pm\ \frac{18{,}000}{.866 \times 5.2 \times 20{,}000}$

$= \pm$ 0.20 sq. in.

This is believed to be too little reinforcement, hence provide 5/8" diameter bars @ 12" c. to c. at top as well as bottom arbitrarily.

Distribution reinforcement: 50 percent of bottom steel

$= \frac{1}{2} \times 0.34 = 0.17$ sq. in.

Hence, provide $\frac{1}{2}$" diameter bars @ c. to c. = 12".

b. Quantity:

Per foot of 28 ft. wide roadway slab:

(i) Concrete: $28 \times \frac{4}{12} + 8 \times 1.5 \times \frac{3}{12} = 12.33$ cu. ft.

(ii) Steel:

Positive steel	=	28 x 1.043	=	28.1 lbs.
Negative steel	=	28 x 1.043	=	28.1 lbs.
Distribution steel	=	29 x 0.668	=	19.4 lbs.
		Total steel	=	75.6 lbs.

c. Cost:

Concrete @ $46.00 per cu. yd.	=	12.33 x 46	=	$21.00
Steel reinf. @ 10¢ per lb.	=	75.6 x 0.10	=	7.56
Therefore, cost per ft. of 28 ft. wide slab			=	$28.56

3. PRESTRESSED GIRDER

a. Design

The girder is simply supported with a span of 60 feet.

The section is selected with dimensions shown in the following Figure No. 7:

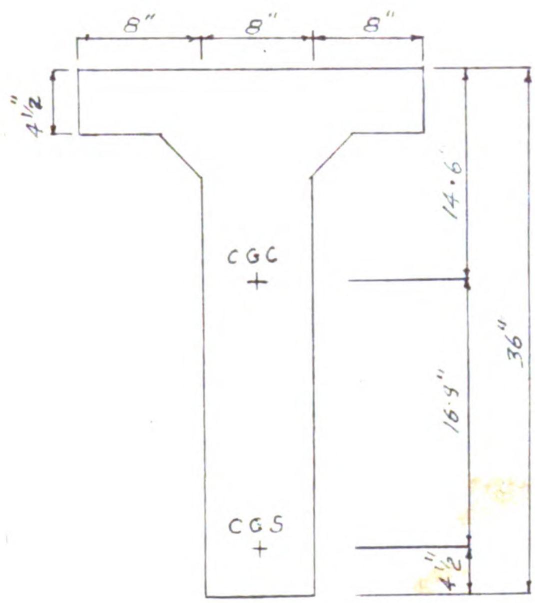


Fig. 7

Symbols:

C.G.C. = C.G. of concrete

C.G.S. = C.G. of steel

A. = Area

Yt = Distance from top to C.G.C. (+)

Yb = Distance from bottom to C.G.C. (-)

e = Distance from C.G.S. to C.G.C. (-)

I = Moment of Inertia

r^2 = I/A

Properties of Gross Concrete Section:

Area			Statical Moment		
A_1 = 16 x 4.5	=	72	A_1 x 2.25	=	162
A_2 = 8 x 36	=	288	A_2 x 18	=	5,200
A_3 = 3 x 3	=	9	A_3 x 5.5	=	49.5
A	=	369 sq. in.	Total	=	5,411.5

$$Yt = 5411.5 \times \frac{1}{369} = 14.6 \text{ in.}$$

$$Yb = Yt - 36 = -21.4 \text{ in.}$$

$$e = Yb - 4.5 = -16.9 \text{ in.}$$

Moment of Inertia:

$$I: \frac{1}{12} \times 16 \times (4.5)^3 = 122$$

$$\frac{1}{3} \times 8 \times (14.6)^3 = 8,260$$

$$\frac{1}{3} \times 8 \times (21.4)^3 = 26,200$$

$$72 \times (12.35)^2 = 10,800$$

$$9 \times (9.1)^2 = \underline{746}$$

$$I = 46,128 \text{ in}^4.$$

$$r^2 = I/A = 46,128/369 = 125 \text{ in}^2.$$

Girder Stresses:

$$\text{Girder weight} = 369 \times \frac{150}{144} \times 60 = 23,000 \text{ lbs.}$$

$$\text{Moment} = \frac{12}{8} \times 23,000 \times 60 = 2,080,000 \text{ in lbs.}$$

$$\text{Stress} = \frac{2,080,000}{46,128}(+14.6) = +660 \text{ psi.}$$

$$\text{and} = \frac{2,808,000}{46,128}(-21.4) = -920 \text{ psi.}$$

Use 62 wires, 0.196 in diameter.

Steel area = $62 \times 0.03 = 1.86 \text{ in}^2$.

Therefore, initial prestress, $P = 1.86 \times 150,000 = 278,500$ lbs.

This will produce stresses in the girder.

$$f = \frac{P}{A}\left(1 + \frac{e}{r^2} \times y\right)$$

$$= \frac{278,500}{369}\left(1 + \frac{-16.9}{125} \times +14.6\right)$$

$$= \frac{278,500}{369}(-0.97) = -732 \text{ psi.}$$

and at bottom

$$f = \frac{278,500}{369}\left(1 + \frac{-16.9}{125} \times -21.4\right)$$

$$= \frac{278,500}{369}(3.88) = +2,920 \text{ psi.}$$

Allowing for creep and shrinkage of concrete, final stresses due to prestressing will be, at top 0.85(-732) = -625 psi.

at bottom 0.85(2,920) = +2,480 psi.

When prestress comes in effect, the girder deflects upward, the girder tries to deflect downward due to its own weight.

Therefore, resultant stresses will be as follows:

	Prestress	Girder	Combined
Initial	-732	+660	-72 psi
	+2,920	-920	-20,000 psi
Final	-625	+660	+35 psi
	+2,480	-920	+1,560 psi

Stresses due to slab:

After the girders are erected they will first carry the slab. Slab moment, then, will reduce the compressive stress in the bottom fiber.

$$\text{Slab weight} = \left(\frac{1}{3} \times 3.5 + \frac{1}{4} \times 1.5\right) 150 = 231 \text{ lbs./ft.}$$

$$\text{Moment} = \frac{12}{8} \times 231 \times 60^2 = 1{,}250{,}000 \text{ in lbs.}$$

$$\text{Stress} = \frac{1{,}250{,}000}{46{,}128} \times (14.6) = +401 \text{ psi at top.}$$

$$\text{and} = \frac{1{,}250{,}000}{46{,}218} \times (-21.4) = -590 \text{ psi at bottom}$$

Stresses in girder, now, will be as follows:

Before Slab	Due to Slab	Combined
+ 35	+ 401	+ 436 psi.
+1,560	- 590	+ 970 psi.

Bending stresses in Composite Section:

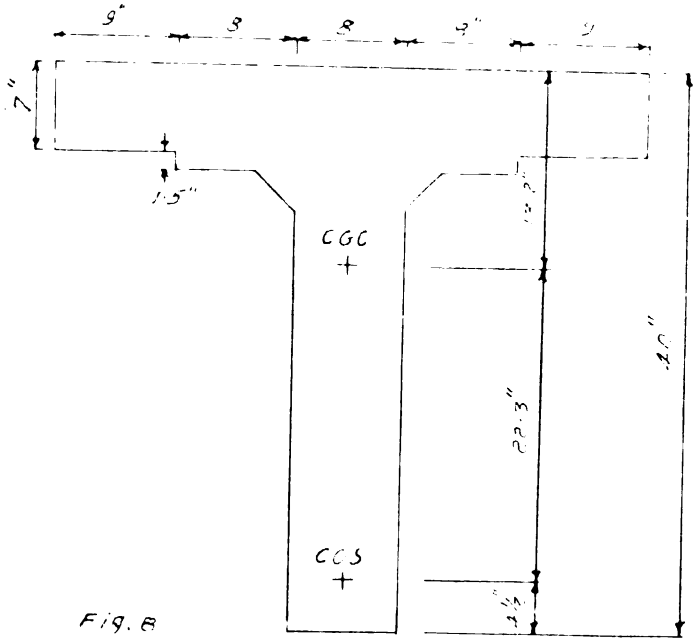


Fig. 8

After the slab sets, it acts integrally with the girder. Hence any subsequent load stresses will be calculated with the assumed composite section as shown in Figure 8.

Properties of gross section:

	Area			Statical Moment	
A_1	= 18 x 7	= 126	A_1 x 3.5	=	440.0
A_2	= 16 x 8.5	= 136	A_2 x 4.25	=	577.5
A_3	= 8 x 40	= 320	A_3 x 20	=	6,400.0
A_4	= 3 x 3	= 9	A_4 x 9.5	=	85.5
A_5	= 1.86 x 7	= 13.0	A_5 x 35.5	=	462.0
	A	= 604.0 sq. in.	Total	=	7,965.0 in^2.

Therefore,

$$Yt = 7965/604 = +13.2 \text{ in.}$$

$$Yb = Yt - 40 = -26.8 \text{ in.}$$

$$3 = Yt - 4.5 = -22.3 \text{ in.}$$

Moment of Inertia:

$$I: \quad \frac{1}{12} \times 18 \times 7^3 = 515$$

$$\frac{1}{12} \times 16 \times (8.5)^3 = 820$$

$$\frac{1}{3} \times 8 \times (13.2)^3 = 6{,}150$$

$$\frac{1}{3} \times 8 \times (26.8)^3 = 51{,}200$$

$$126 \times (9.7)^2 = 11{,}850$$

$$136 \times (8.95)^2 = 10{,}900$$

$$9 \times (3.7)^2 = 123$$

$$13 \times (22.3)^2 = \underline{6{,}440}$$

$$I = 87{,}998 \text{ in}^4.$$

Stresses due to live load:

From Appendix 'A', AASHO specifications, 1953, page 285, for 60' span.

Live Load Moment = 555 k. ft. = 6,650,000 in lbs.

27 percent impact = 1,800,000 in lbs.

Therefore, total Live Load Moment per lane = 8,450,000 in lbs.

Fraction of moment to each girder

$$= \frac{s}{2 \text{x} 5.0} = \frac{3.5}{10} = 0.35$$

Design L.L. moment = 0.35 x 8,450,000

= 2,960,000 in lbs.

Stress at top of slab $= \dfrac{2{,}960{,}000}{87{,}998}(13.2) = +\ 445$ psi.

At top of girder = 2,960,000(9.2) = + 310 psi.

At bottom of girder = 2,960,000(-26.8) = - 910 psi.

Final stress analysis:

	Before L.L. Applied	L.L. Stress	Combined
At top of slab	0	+ 445	+ 445 psi.
At top of girder	+ 436	+ 310	+ 746 psi.
At bottom of girder	+ 970	- 910	+ 60 psi.

The girder is quite safe as no tensile stress occurs in the final analysis.

Checking of stresses at Cracking Load:

Cracking load = 1.0 DL + 2.0 LL

This means 100 percent over load.

Live Load Moment = 2x2x8,450,000 = 3,360,000 in lbs.

Curb and railing = $\frac{2\times12\times400\times60^2}{8}$ = 43,320,000 in lbs.

Surfacing = $\frac{12\times20\times28\times60^2}{8}$ = $\underline{3,020,000}$ in lbs.

Total moment on 9 girders = 40,950,000 in lbs.

Therefore, moment per girder = 4,560,000 in lbs.

Stress at top of slab = $\frac{4,560,000}{87,998} \times 13.2 = + 680$ psi.

Stress at top of girder = $\frac{4,560,000}{87,998} \times 9.2 = + 475$ psi.

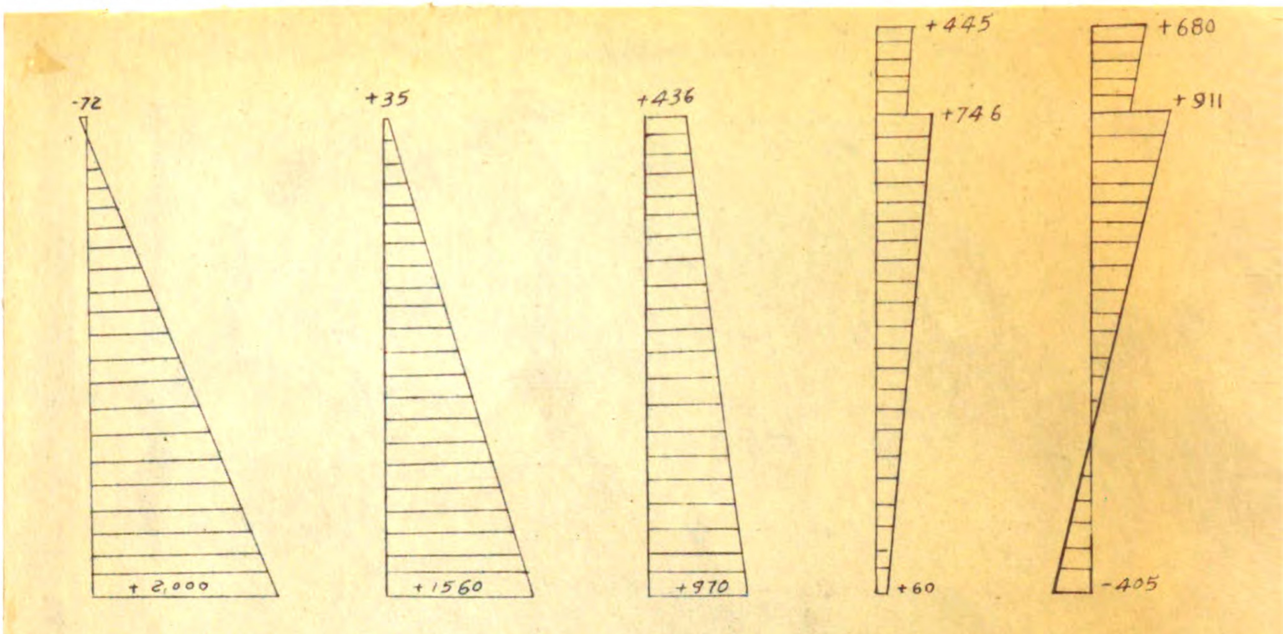
Stress at bottom of girder = $\frac{4,560,000}{87,998} \times -26.8 = -1,375$ psi.

Therefore, stresses due to 1.0 DL + 2.0 LL will be as follows:

	Due to D.L.	Due to 2.0 LL	Combined
At top of slab	0	+ 680	+ 680 psi.
At top of girder	+ 436	+ 475	+ 911 psi.
At bottom of girder	+ 970	-1,375	- 405 psi.

The tensile stress at the bottom fiber is less than permissible, hence design at cracking load is also safe.

Stress diagrams for different stages are shown in the following Figure No. 9:



Initial Prestress + Girder | Final Prestress + Girder | Prestress + Girder + Slab | Prestress + D.L. + L.L. | Prestress + D.L. + 2L.L.

Fig. 9

Ultimate Load

Ultimate resisting moment, Mu = As fu jd

where As = steel area

fu = Ultimate Stress = 250,000 psi.

jd is moment - arm.

Assume j = 0.9

Then Mu = (9x1.86)250,000 x 0.9 x 35.5 = 133,000,000 in lbs.

Total girder and slab moment = 9 x 3,330,000 = 29,970,000 in lbs.

Moment due to curb, railing surfacing, = 7,340,000 in lbs.

Total D.L. Moment = 37,310,000 in lbs.

Total L.L. Moment = 16,900,000 in lbs.

D.L. + L.L. = 54,210,000 in lbs.

Ultimate Resisting Moment, Mu	=	133,000,000 in lbs.
Subtracting D.L. Moment	=	- 37,310,000 in lbs.
Balance left for L.L.	=	95,690,000 in lbs.

Ultimate load factors:

$$= \frac{133,000,000}{54,210,000} \text{ (D.L. + L.L.)} = 2.28 \text{ (D.L. + L.L)}$$

$$\text{also } \frac{95,690,000}{16,900,000} \text{ (L.L.)} = 5.65 \text{ (L.L.)}$$

These are within allowable limits. Hence, the design is also safe at Ultimate Loading.

Eccentricity at Support:

As moment at the support is zero, to make top fiber stress equal to zero,

$$e = -r^2/y_t$$

$$= -125/14.6 = -8.55"$$

The profile of the c.g. of the steel is assumed to be parabola for the beam.

Principle stresses:

The shearing stress v in an uncracked concrete section is maximum at the centroid. This can be given by, $v = VQ/bI$.

Referring to Figure No. 7, Q, the statical moment of section on either side of centroid taken about that point

$$= 21.4 \times 8 \times 10.7 = 1,830 \text{ in}^3.$$

Now shear at the support = $\frac{1}{2}$(wt. of girder + wt. of slab)

$$= \tfrac{1}{2}(23,000 + 231 \times 60) = 18,430 \text{ lbs.}$$

Vertical component of prestress in wires,

$$= -237,000 \times 2 \times 8.55/12 \times 30 = -11,250 \text{ lbs.}$$

Therefore, $V = 18,430 - 11,250 = 7,180$ lbs.

Hence, $V = VQ/bI = 7,180 \times 1,830/8 \times 46,128 = 35.5$ psi.

From AASHO Specifications, Appendix 'A' for 60 ft. span, maximum shear at the support = 45.2 k.

Therefore, shear with impact = $1.27 \times 45.2 = 57.5$ k.

Hence, Design shear = $.35 \times 57.5 = 20.1$ k.

Referring back to Figure No. 8, Q will be,

$$26.8 \times 8 \times 13.5 + 13 \times 22 = 2,860 + 290 = 3,150 \text{ in}^3.$$

Therefore, $V = 20,100 \times 3,150/8 \times 87,998 = 90$ psi.

Hence, total $V = 35.5 + 90 = 125.5$ psi.

The prestress force creates a horizontal compressive stress at the centroid of concrete.

This has intensity, $S_X = P/A$

$$= 0.85 \times 278,500/369 = 642 \text{ psi.}$$

Now, the stresses V and S_X produce a principal tensile stress which at the support at the centroid can be calculated by the well known formula, $S_t = \frac{1}{2}(\sqrt{4v^2 + S_X^2} - S_X)$

$$= \frac{1}{2}(\sqrt{4 \times 125.5^2 + 642^2} - 642)$$

$$= \frac{1}{2}(688 - 642) = 23 \text{ psi.}$$

Shearing stress at ultimate load will be,

(1) $2.25\ (DL + LL) = 2.25 \times 125.5 = 287$ psi.

(2) $DL + 3.5\ LL = 125.5 + 3.5 \times 90 = 350.5$ psi.

Hence, principle tensile stress, when $v = 350.5$ will be,

$$\frac{1}{2}(\sqrt{4 \times 350.5^2 + 642^2} - 642)$$

$$= \frac{1}{2}(950 - 642) = 154 \text{ psi.}$$

These are within limits, hence design is safe.

Deflection:

The deflection of prestressed girders can be computed by the formula, $D = \frac{5}{48} \frac{M_{max} L}{EI}$, when load is uniformly distributed on the span length, L, and the moment of inertia, I, is constant throughout the entire span,

$$D = \frac{5 \times 16,900,000 \times (60 \times 12)}{48 \times 3.5 \times 10 \times 87,988 \times 9}$$

$$= 0.33 \text{ in.} = 1/2,180 \text{ span}$$

This is much less than allowable live load deflection, hence safe.

Diaphragms:

These are provided tentatively as follows:

Provide 2 Nos. 24" x 9" section with two cables each having an area of 0.40 sq. in.

Quantity:

Concrete 2 (2x.75x24) = 72 cu. ft.

Cables 4 (30 x 1.41) = 169 lbs.

Cost:

Concrete @ \$46.00 per cu. yd. = $\frac{72 \times 46}{27}$ = \$123

Cables @ 60¢ per lb. = 169.2 x .6 = 102

Total = \$225

b. Quantity for 9 prestressed beams:

(i) Concrete: $9 \times \frac{369 \times 60}{144}$ = 1,380 c. ft.

(ii) Cable: 9 x 60 x 6.44 = 3,480 lbs.

c. Cost of deck:

(i) Concrete @ \$3.00 per cu. ft. = 1,380 x 3 = \$4,140

(ii) Cables (includes fitting, prestressing, anchoring)

@ 60 cents per lb. = 3480 x 0.6 = \$2,088

(iii) Transportation up to 100 miles

@ \$4 per ton = 103.5 x 4 = 414

(iv) Erection @ \$10 per ton = 103.5 x 10 = 414

(v) Slab 28' wide @ \$28.56 per foot = 60 x 28.56 = 1,713

(vi) Diaphragms 225

Total = \$9,615

IV. RESULTS

The costs of the deck for the two different types of construction are tabulated in the following tables with necessary details. Also, the total costs are plotted on the following graph paper for handy comparison. The cost analysis for 50', 70', 80', 90' and 100' spans are given in Appendix.

TABLE I

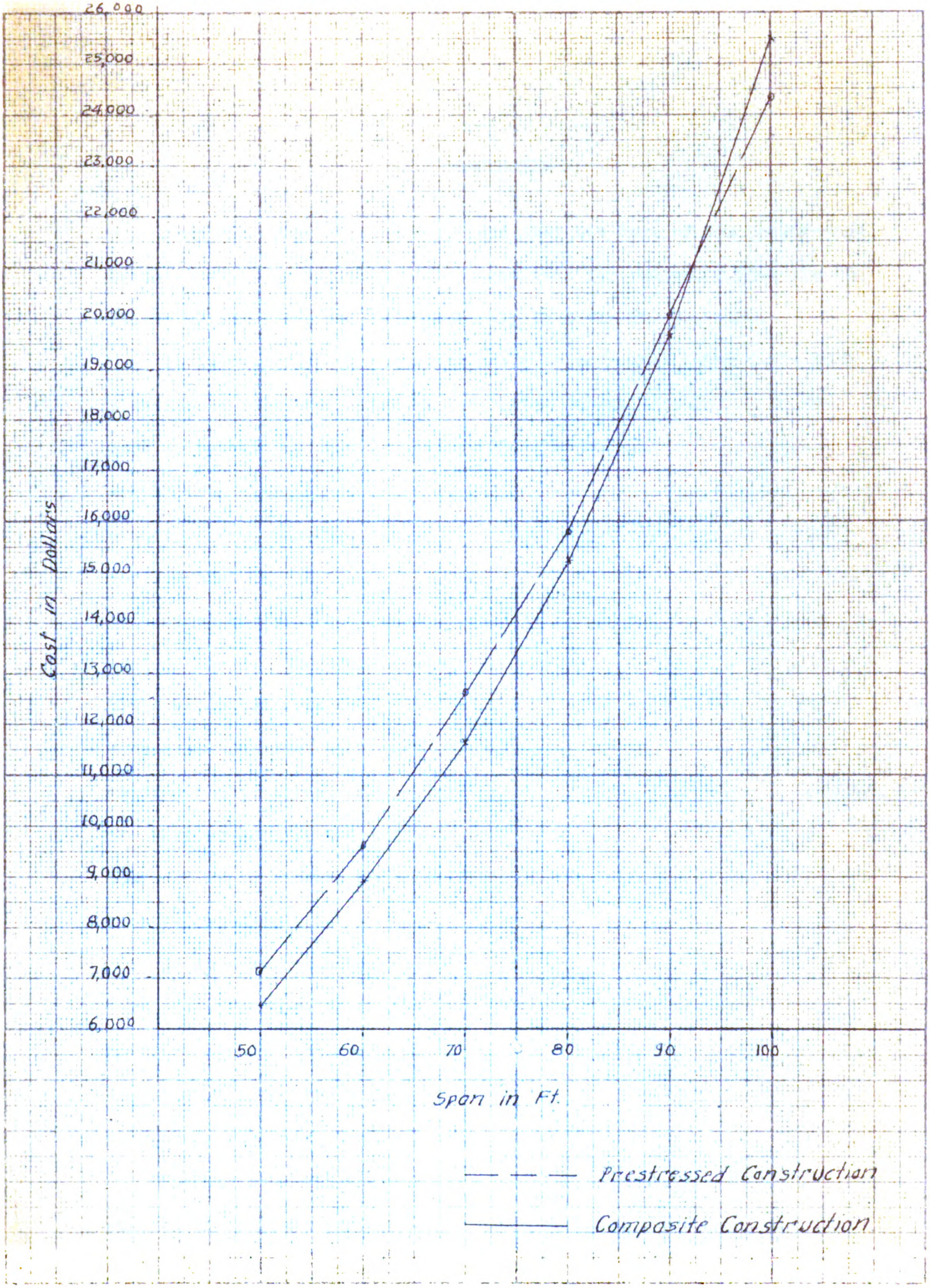
COMPOSITE CONSTRUCTION

Span Length	Structural Steel @ $0.135 per pound		Spiral Steel @ $0.50 per pound		7" Thick Slab @ $1.378 per sq. foot		Total Cost in Dollars
ft.	Quantity lbs.	Cost $	Quantity lbs.	Cost $	Quantity sq. ft.	Cost $	
50	30,946.8	4,178	780	390	1,400	1,929	6,497
60	45,807.6	6,184	828	414	1,680	2,320	8,918
70	62,955	8,499	915.6	458	1,960	2,701	11,658
80	85,548	11,549	1,041.6	521	2,240	3,087	15,157
90	115,380	15,576	1,317.6	659	2,520	3,475	19,710
100	155,276.4	20,962	1,380	690	2,800	3,859	25,511

TABLE II

PRESTRESSED CONCRETE CONSTRUCTION

Span	Prestressed Girders							Slab 28' Wide @ $28.56 per foot	Cost of Diaphragms	Total Cost
	Concrete $3 per cu. ft.		Cables @ $0.60 per lb.		Weight	Transportation Up to 100 Miles @ $4 per ton	Erection @ $10 per ton			
ft.	Quantity cu. ft.	Cost $	Quantity lbs.	Cost $	tons	$	$	$	$	$
50	1,000	3,000	2,420	1,452	75.0	$ 300	750	1,428	205	7,135
60	1,380	4,140	3,480	2,088	103.5	414	1,035	1,713	225	9,615
70	1,825	5,475	4,840	2,904	137.0	548	1,370	1,999	367	12,663
80	2,320	6,960	6,200	3,720	174.0	696	1,740	2,285	398	15,799
90	2,930	8,790	8,450	5,070	220.0	880	2,200	2,570	571	20,081
100	3,550	10,650	10,800	6,480	266.2	1,065	2,662	2,856	612	24,325


26,000
25,000
24,000
23,000
22,000
21,000
20,000
19,000
18,000
17,000
16,000
15,000
14,000
13,000
12,000
11,000
10,000
9,000
8,000
7,000
6,000
Cost in Dollars
50
60
70
80
90
100
Span in Ft.
Prestressed Construction
Composite Construction

V. CONCLUSION

From the graph it can be noticed that the cost of the prestressed bridge deck is only little higher that that for composite type for 50 ft. to 90 ft. spans and is appreciably less for 100 ft. span.

It is essential in comparing costs to note that labor costs have always been relatively high in the United States in relation to the cost of materials. Hence, savings of materials due to prestressed construction has proved it economical in Europe but not in this country. From the cost analysis of prestressed concrete, it can be noticed that the unit cost for concrete is relatively very high. This is because the contractors have not developed the labor saving methods and machinery. Therefore, to achieve economy the theoretical possibilities or new type or construction like this must lead to improved production methods. This can be achieved by making the process commerically attractive. For this there should be standard lengths, widths and bearing details for the prestressed bridge units that will meet the needs of state highway bridges. By this, development of mass production or assembly line can be set up.

If this can be achieved, the prestressed construction will surely turn out to be more economical than the present composite construction. However, the maintenance charges can be eliminated by prestressed construction. Also, the relative overload capacities of the prestressed bridges is also very high.

Looking from all these points, it will not be out of place to suggest the highway officials to encourage the contractors to develop the fabrication and erection methods suited to American conditions, to use prestressed concrete for bridge construction.

APPENDIX A

COMPOSITE CONSTRUCTION FOR 50' - 70' - 80' - 90' 100' SPANS

The composite beams for the spans 50' - 70' - 80' - 90' - 100' are designed on the same lines as that for 60' span. Hence, only the important properties of the sections are mentioned hereafter.

1. FOR 50' SPAN

a. Design

Span length = 50' Truck load governs. MLL = 344 kft.

MDL = 206 kft. Mws = 37.5 kft.

Provide 24" WF @ 76 + 12" x 3/4" cover plate 33' long.

Properties of section without cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	$I'o = 2096.4\ in^4$.	18.96" 11.95"	$S'so = 175.4\ in^3$.
Composite (n = 10)	$Io = 5996.4\ in^4$.	8.25" 22.66"	$Sso = 265\ in^3$.
Composite (n = 30)	$I''o = 4449.8\ in^4$.	12.30" 18.61"	$S''so = 238.5\ in^3$

Properties of section with cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	$I' = 3080.4\ in^4$.	21.53" 10.0 "	$S's = 308\ in^3$.
Composite (n = 10)	$I = 9375.4\ in^4$.	10.23" 21.30"	$Ss = 440\ in^3$.
Composite (n = 30)	$I'' = 6628.8\ in^4$.	15.03" 16.50"	$S''s = 402\ in^3$.

Unit Stresses at center of span: (in ksi)

Stress	Bottom of Steel	Top of Steel	Top of Concrete
Due to DL	8.00	11.60	0.000
Due to LL	8.85	1.35	0.427
Due to WS	1.10	0.56	0.033
Total	17.95	13.41	0.460

Spiral shear connectors:

Using 5/8" Ø bar with 4-1/2" mean coil diameter:

Spiral	Pitch	Length	lbs./ft.	Total wt. in lbs.
S_0	5"	10.75'	3.14	33.8
S_{10}	8"	10.75'	2.12	23.0
S_{20}	14"	5.50'	1.49	8.2
			Total	65.0 lbs.

Weight of spiral per foot of beam = 2.6 lbs.

Live load deflection = 1/1,260 of span

Live load factor of safety = 2.7

Provide 2 diaphragms, each having 2 Ls 3" x 3" x 3/8" with 16"x3/8" plate.

b. Quantity

(i) Structural Steel:

6 No.s 24" WF. @ 76 for 50' = 76 x 50	=	22,800 lbs.
6 Nos. 12" x 3/4" plate 33' long = 30.6x6x33	=	6,058.8 lbs.
4 Ls 3" x 3" x 3/8" - 30' long = 7.2x4x30	=	864.0 lbs.
2 Nos. 16" x 3/8" - 30' long = 20.4 x 2 x 30	=	1,224.0 lbs.
Total	=	30,946.8 lbs.

(ii) Spiral shear developers for 6 beams 50' long

= 2.6x6x50 = 780 lbs.

c. Cost

(i) Structural steel (fabrication, erection, painting)
@ $0.135 per lb. = 30,946.8 x .135 = $ 4,177.82

(ii) Spiral shear developers @ 50¢ per lb. = 780x .50 = 390.00

(iii) 7" slab 28' x 50' @ $1.378 per sq. ft. = 1,929.20

Hence, total cost of deck = $6,497.02

2. FOR 70' SPAN

a. Design

Span length = 70' Stringer spacing 6' c. to c.

Slab thickness = 7" Lane load governs

WLL = .483 k/ft. Impact factor = .256

Conc. load for moment = 13.36 K

Conc. load for shear = 19.6 K

MLL = 530 K ft. MDL = 430 K ft. Mws = 73.5 K ft.

Provide 30" WF @ 108 with 13-1/2" x 1" cover plate 50' long.

Properties of section without cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	I'o = 4,461 in^4.	21.91" 14.91"	S'so = 299 in^3.
Composite (n = 10)	Io = 11,356 in^4.	9.46" 27.36"	Sso = 414 in^3.
Composite (n = 30)	I"o = 8,239 in^4.	15.20" 21.62"	S"so = 402 in^3.

Properties of section with cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	I' = 6,718 in^4.	26.49" 11.33"	S's = 594 in^3.
Composite (n = 10)	I = 19,503 in^4.	14.39" 23.43"	Ss = 832 in^3.
Composite (n = 30)	I" = 13,253 in^4.	20.25" 17.57"	S"s = 754 in^3.

Unit stresses at the center of span (in ksi):

Stress	Bottom of Steel	Top of Steel	Top of Concrete
Due to DL	8.70	14.95	0.000
Due to LL	7.65	2.41	0.498
Due to WS	1.17	0.50	0.045
Total	17.52	17.86	0.543

Spear Shear Connectors:

Using 5/8" ⌀ bar with 4-1/2" mean coil diameter:

Spiral	Pitch	Length	Lbs./ft.	Total weight in lbs.
S_0	5"	10.75'	2.67	28.5
S_{10}	8"	10.75'	2.12	23.0
S_{20}	11"	10.75'	1.71	18.4
S_{30}	18"	5.5 '	1.31	6.7
			Total	76.6 lbs.

Weight of spiral per foot of beam = 2.18 lbs.

Live load deflection = 1/1,190 of span

Live load factor of safety = 3.28

Provide 3 diaphragms each having 2 Ls 3"x3"x3/8" with 22"x3/8" plate.

b. Quantity

(i) Structural steel:

6 Nos. 30" WF @ 108 for 70' =	108x6x70	= 45,360 lbs.
6 Nos. 13-1/2" x 1" cover plate 50' long =	45.9x50x6	= 13,770 lbs.
6 Ls 3" x 3" x 3/8" - 30' long =	7.2x6x30	= 1,296 lbs.
3 Nos. 22" x 3/8" plate 30' long =	28.1x3x30	= 2,529 lbs.
	Total	= 62,995 lbs.

(ii) Spiral shear developers for 6 beams 70' long

= 2.18 x 6 x 70 = 915.6 lbs.

c. Cost

(i) Structural steel (fabrication, erection, painting)

@ \$0.135 per lb. = 62,955x.135 = \$8,498.95

(ii) Spiral shear developers @ 50¢ per lb. = 915.6x.50 = 457.80

(iii) 7" slab 28' x 70' @ \$1.378 per sq. ft. = 2,700.80

Hence, total cost of deck = \$11,657.55

3. FOR 80' SPAN

a. Design

Span length = 80' Stringer spacing 6' c. to c.

Slab thickness = 7" Lane load governs

WLL = .476 k/ft. Impact factor 0.244

Conc. load for moment 13.44 k.

Conc. load for shear 19.40 k.

MLL = 648.8 k. ft. MDL = 600 k. ft. Mws = 96 k. ft.

Provide 33" WF. @ 130 with 14-1/2" x 1-1/8" cover plate 57' long.

Properties of section without cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	I'o = 6,699 in^4.	23.55" 16.55"	S'so = 404.8 in^3.
Composite (n = 10)	Io = 15,644 in^4.	12.10" 28.0 "	Sso = 560.0 in^3.
Composite (n = 30)	I"o = 11,472 in^4.	17.4 "	S"so = 505.0 in^3.

Properties of section with cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	I'o = 11,094 in^4.	28.55" 12.68"	S's = 875 in^3.
Composite (n = 10)	I = 27,819 in^4.	16.55" 24.68"	Ss = 1,120 in^3.
Composite (n = 30)	I"o = 19,242 in^4.	22.63" 18.60"	S"s = 1,030 in^3.

Unit stresses at the center of span (in ksi).

Stress	Bottom of Steel	Top of Steel	Top of Concrete
Due to DL	8.22	13.9	0.000
Due to LL	7.10	2.98	0.482
Due to WS	1.12	0.94	0.046
Total	16.44	17.82	0.528

Spiral shear connectors:

Using 5/8" Ø bar with 4-1/2" mean coil diameter:

Spiral	Pitch	Length	Lbs./ft.		Total weight in lbs.
S_0	5"	10.75'	3.14		33.8
S_{10}	7"	10.75'	2.36		25.4
S_{20}	16"	10.75'	1.40		15.1
S_{30}	25"	10.75'	1.15		12.3
				Total	86.6 lbs.

Weight of spiral per foot of beam = 2.17 lbs.

Live load deflection 1/1,130 of span.

Live load factor of safety = 3.34

Provide 3 diaphragms each having 2 Ls 3"x3"x 3/8" with 25"x 3/8" plate.

b. Quantity

(i) Structural steel:

6 Nos. 33 WF. @ 130 for 80' = 130 x 6 x 80 = 62,400 lbs.

6 Nos. 14-1/2" x 1-1/8" cover plate 57' long
= 55.5 x 6 x 57 = 18,981 lbs.

6 Ls 3" x 3" x 3/8" - 30' long = 7.2x6x30 = 1,296 lbs.

3 Nos. 25" x 3/8" plate 30' long = 31.9x3x30 = 2,871 lbs.

Total = 85,548 lbs.

(ii) Spiral shear developers for 6 beams 80' long
= 2.17 x 6 x 80 = 1,041.6 lbs.

c. Cost

(i) Structural steel (fabrication, erection, painting)
@ $0.135 per lb. = 85,548 x .135 = $11,548.98

(ii) Spiral shear developers @ 50¢ per lb. = 1,041.6x .50 = 520.80

(iii) 7" slab 28' x 80' @ $1.378 per sq ft. = 3,086.72

Hence, total cost of deck = $15,156.50

4. FOR 90' SPAN

a. Design

Span length = 90'	Stringer spacing 6' c. to c.
Slab thickness = 7"	Lane load governs
WLL = 0.474 k/ft.	Impact factor = 0.232

Conc. load for moment = 13.3 k

Conc. load for shear = 19.2 k

MLL = 779 k. ft. MDL = 810 k. ft. Mws = 121 k. ft.

Provide 36" WF @ 160 with 15" x 1-1/4" cover plate 60' long.

Properties of section without cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	I'o = 10,470 in^4.	25.08" 18.08"	S'so = 579.1 in^3.
Composite (n = 10)	Io = 22,375 in^4.	14.26" 28.90"	Sso = 773.0 in^3.
Composite (n = 30)	I"o = 16,389 in^4.	19.70" 23.46"	S"so = 697.0 in^3.

Properties of section with cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	I'o = 15,570 in^4.	30.18" 14.23"	S's = 1,090 in^3.
Composite (n = 10)	Io = 36,495 in^4.	18.18" 26.23"	Sso = 1,390 in^3.
Composite (n = 30)	I"o = 25,268.4 in^4.	24.94" 19.47"	S"so = 1,300 in^3.

Unit stresses at the center of span (in ksi):

Stress	Bottom of Steel	Top of Steel	Top of Concrete
Due to DL	8.92	14.21	0.000
Due to LL	6.72	2.72	0.465
Due to WS	1.11	0.95	0.047
Total	16.74	17.88	0.512

Spiral shear connectors:

Using 5/8" Ø bar with 4-1/2" mean coil diameter:

Spiral	Pitch	Length	Lbs./ft.	Total wt. in lbs.
S_0	5"	10.75'	3.14	33.8
S_{10}	7"	10.75'	2.36	25.4
S_{20}	10"	10.75'	1.82	19.6
S_{30}	15"	10.75'	1.44	15.5
S_{40}	23"	5.5 '	1.21	6.6
			Total	100.9 lbs.

Weight of spiral per foot of beam = 2.44 lbs.

Live load deflection = 1/1090 of span

Live load factor of safety = 3.4

Provide 4 diaphragms each have 2 Ls 3"x3"x 3/8" with 28"x 3/8" plate.

b. Quantity

(i) Structural steel:

6 Nos. 36 WF. @ 160 for 90' = 160 x 6 x 90 = 86,400 lbs.

6 Nos. 15" x 1-1/4" plate 60' long = 22,968 lbs.

8 Ls 3" x 3" x 3/8" - 30' long = 7.2x8x30 = 1,728 lbs.

4 Nos. 28" x 3/8" plate 30' long = 35.7x4x30 = 4,284 lbs.

Total 115,380 lbs.

(ii) Spiral shear deveopers for 6 beams 90' long

= 2.44x6x90 = 1,317.6 lbs.

c. Cost

(i) Structural steel (fabrication, erection, painting)

@ \$0.135 per lb. = 115,380 x .135 = \$15,576.30

(ii) Spiral shear developers @ 50¢ per lb. = 1317.6x.50= 658.80

(iii) 7" slab 28' x 90' @ \$1.378 per sq. ft. = 3,475.00

Hence, total cost of deck = \$19,710.10

5. FOR 100' SPAN

a. Design

Span length = 100' Stringer spacing 6' c. to c.

Slab thickness = 7" Lane load governs

WLL = 0.469 k/ft. Impact factor = 0.222

Conc. load for moment = 13.2 k.

Conc. load for shear = 19.1 k.

MLL = 915 kft. MDL = 1,100 kft. Mws = 150 kft.

Provide 36" WF. @ 245 with 18-1/2" x 1" cover plate 60' long.

Properties of section without cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	I'o = 16,092 in^4.	25.03" 18.03"	S'so = 892.5 in^3.
Composite (n = 10)	Io = 30,047 in^4.	16.20" 26.86"	Sso = 1,120 in^3.
Composite	I"o = 22,450.4 in^4.	20.90" 22.16"	S"so = 1,012 in^3.

Properties of section with cover plate:

Section	Moment of Inertia	NA	Section Modulii
Steel	I'o = 21,132 in^4.	28.81" 15.25"	S'so = 1,385 in^3.
Composite (n = 10)	Io = 42,057 in^4.	19.76" 24.30"	Sso = 1,730 in^3.
Composite (n = 30)	I"o = 30,260.4 in^4.	24.85" 19.21"	S"so = 1,580 in^3.

Unit stresses at the center of span (in ksi).

Stress	Bottom of Steel	Top of Steel	Top of Concrete
Due to DL	9.51	13.60	0.000
Due to LL	6.35	3.30	0.516
Due to WS	1.14	1.03	0.048
Total	10.70	17.93	0.564

Spiral shear connectors:

Using 5/8" Ø bar with 4-1/2" mean coil diameter:

Spiral	Pitch	Length	Lbs./ft.	Total wt. in lbs.
S_0	5"	10.75'	3.14	33.8
S_{10}	6"	10.75'	2.67	28.8
S_{20}	9"	10.75'	1.96	21.1
S_{30}	12"	10.75'	1.62	17.4
S_{40}	19"	10.75'	1.32	14.2
			Total	115.3 lbs.

Weight of spiral per foot of beam = 2.3 lbs.

Live load deflection = 1/956 of span.

Live load factor of safety = 3.51.

Provide 4 diaphragms each having 2 Ls 3"x3"x3/8" with 28"x3/8" plate.

b. Quantity

(i)	6 Nos. 36 WF. @ 245 for 100' = 245 x 6 x 100	= 147,000 lbs.
	6 Nos. 18-1/2" x 1" plate 60' long = 62.9x6x60	= 2,264.4
	8 Ls 3" x 3" x 3/8" - 30' long = 7.2 x 8 x 30	= 1,728.0
	4 Nos. 28"x3/8" plate 30' long = 35.7 x 4 x 30	= 4,284.0
	Total	155,276.4 lbs.

(ii) Spiral shear developers for 6 beams 100' long
= 2.3 x 6 x 100 = 1,380 lbs.

c. Cost

(i) Structural steel (fabrication, erection, painting)
@ \$0.135 per lb. = 155,276.4 x .135 = 20,962.26

(ii) Spiral shear developers @ 50¢ per lb. = 1380 x 0.50 = 690.00

(iii) 7" slab 28' x 100' @ \$1.378 per sq. ft. = 3,858.40

Hence, total cost of deck, \$25,510.66.

APPENDIX B

PRESTRESSED CONCRETE CONSTRUCTION FOR 50' - 70' - 80' - 90'- 100' SPAN

1. FOR 50' SPAN

a. Design

30" deep T section: 24" wide flange 4-1/2" thick. Web thickness = 8 in.

Gross properties: Area = 321 sq. in. S.M. = 3,811.5 in^3.

Yt = 11.9", Yb = -18.1", e = -13.6", I = 27,361 in^4.

Girder Moment = 1,250,000 lbs. in.

Provide 52 wires 0.196" dia., area = 1.56 sq. in.

Slab Moment = 866,000 lbs. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 635	- 545	-	- 90 psi.
	- 2830	- 830	-	- 2000 psi.
Final	- 540	- 545	- 376	- 381 psi.
	- 2410	- 830	- 574	- 1006 psi.

Composite Section Properties: Area = 553.92 sq. in.

S.M. = 6,045 in^3., Yt = 10.9", Yb = -23.1", I = 54,283 in^4.

Max. L.L. Moment with 28.6 percent impact = 6,890,000 lbs. in.

per lane. Hence, design L.L. Moment = 2,410,000 lbs. in.

Stress @	Before L.L.	L.L.	Combined
Top of slab	0	- 485	- 485 psi.
Top of girder	- 381	- 306	- 687 psi.
Bottom of girder	-1,006	-1002	- 4 psi.

Cracking Moment = 3,623,000 lbs. in.

Stress @	Before C.M.	C.M.	Combined
Top of slab	0	+ 726	+ 726 psi.
Top of girder	+ 381	+ 502	+ 883 psi.
Bottom of girder	+1,006	-1,540	- 536 psi.

Ultimate R.M.: Mu = 2.46 (DL + LL) or D.L. + 4.96 L.L.

L.L. Deflection = $\frac{1}{2,000}$ span.

Diaphragms:

Provide 2 Nos. 20" x 9" section with two cables each having an area of 0.40 sq. in. at the total cost of $205.

b. Quantity for 9 prestressed beams

(i) Concrete: 9x50x321/144 = 1,000 cu. ft.

(ii) Cable: 9x50x5.38 = 2,420 lbs.

c. Cost of deck

(i)	Concrete @ $3 per cu. ft. = 1,000x3	= $3,000
(ii)	Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 2,420x 0.6	= 1,452
(iii)	Transportation up to 100 miles @ $4 per ton = 75 x 4	= 300
(iv)	Erection @ $10 per ton = 75 x 10	= 750
(v)	Slab 28' wide @ $28.56 per ft. = 50 x 28.56	= 1,428
(vi)	Diaphragms	= 205

Hence, total cost of deck = $7,135

2. FOR 70' SPAN

a. Design

42" deep T section: 24" wide flange 4-1/2 thick. Web thickness = 8 in.

Gross Properties: Area 417 sq. in., S. M. = 7,261.5 in^3.

Yt = 17.4", Yb = -24.6", e = -20.1", I = 73,492 in^4.

Girder Moment = 3,200,000 lbs. in.

Provide 74 wires @ 1.96" dia., area = 2.22 sq. in.

Slab Moment = 1,700,000 lbs. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 790 + 3030	+ 756 -1,030	- -	- 34 +2,000
Final	- 671 + 2580	+ 756 -1,030	+ 402 - 570	+ 487 + 980

Composite Section Properties: Area = 656.76 sq. in.

S.M. = 10,298 in^3., Yt = -15.7", Yb = -29.3", I = 135,185 in^4.

Max. L.L. Moment with 25.65 percent impact = 10,774,000 lbs. in. per lane. Hence, design L.L. Moment = 3,780,000 lbs. in.

Stress @	Before L.L.	L.L.	Combined
Top of slab	0	+ 422	+ 422 psi.
Top of girder	+ 487	+ 328	+ 815 psi.
Bottom of girder	+ 980	- 850	+ 130 psi.

Cracking Moment = 5,920,000 lbs. in.

Stress @	Before C.M.	C.M.	Combined
Top of slab	0	+ 690	+ 690 psi.
Top of girder	+ 487	+ 515	+1,002 psi.
Bottom of girder	+ 980	-1,330	- 350 psi.

Ultimate R.M.: Mu. = 2.46 (D.L. + L.L.) or D.L. + 6.12 L.L.

L.L. Deflection = $\frac{1}{2,260}$ span.

Diaphragms:

Provide 3 Nos. 28" x 9" section with two cables each having an area of 0.40 sq. in. at the total cost of $367.

b. Quantity

(i) Concrete: 9x70x417/144 = 1,825 cu. ft.

(ii) Cable: 9x70x7.68 = 4,840 lbs.

c. Cost of deck

(i) Concrete @ $3 per cu. ft. = 1,825 x 3 = $5,475

(ii) Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 4,840 x 0.6 = $2,904

(iii) Transportation up to 100 miles @ $4 per ton = 137 x 4 = $548

(iv) Erection @ $10 per ton = 137 x 10 = $1,370

(v) Slab 28' wide @ $28.56 per foot = 70x28.56 = $1,999

(vi) Diaphragms = $ 367

Hence, total cost of deck = $12,663

3. FOR 80' SPAN

a. Design

48" deep T section: 24" wide flange 4-1/2" thick. Web thickness = 8 in.

Gross properties: Area = 465 sq. in., S. M. = 9,411.5 in^3.

Yt = 20.2", Yb = -27.8", e = -23.3", I = 104,162 in^4.

Girder Moment = 2,220,000 lbs. in.

Provide 86 wires 0.196" dia., Area = 2.58 sq. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 917	+ 900	-	- 17 psi.
	+3,240	-1,240	-	+ 2,000 psi.
Final	- 780	+ 900	+431	+ 581 psi.
	+2,760	-1,240	-594	+ 926 psi.

Composite Section Properties: Area = 705.06 sq. in.

S.M. = 12,758 in^3., Yt = 18.1", Yb = -33.9", I = 190,000 in^4.

Max. L.L. Moment with 24.4 percent impact = 13,330,000 lbs. in. per lane. Hence, design L.L. Moment = 4,650,000 lbs. in.

Stress @	Before L.L.	L.L.	Combined
Top of slab	0	+ 444	+ 444 psi.
Top of girder	+ 581	+ 346	+ 927 psi.
Bottom of girder	+ 926	- 830	+ 96 psi.

Cracking Moment = 7,360,000 lbs. in.

Stress @	Before C.M.	C.M.	Combined
Top of slab	0	+ 702	+ 702 psi.
Top of girder	+ 581	+ 507	+1,088 psi.
Bottom of girder	+ 926	-1,330	- 404 psi.

Ultimate R.M.: Mu = 2.44 (D.L. + L.L.) or D.L. + 6.12 L.L.

L.L. Deflection $= \frac{1}{2,240}$ span

Diaphragms:

Provide 3 Nos. 32" x 9" section with two cables each having an area of 0.40 sq. in at the total cost of $398.

b. Quantity for 9 prestressed beams

(i) Concrete: 9 x 80 x 465/144 = 2,320 cu. ft.

(ii) Cable: 9 x 80 x 8.62 = 6,200 lbs.

c. Cost of deck

(i) Concrete @ $3 per cu. ft. = 2,320 x 3 = $6,960

(ii) Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 6,200x0.6 = $3,720

(iii) Transportation up to 100 miles @ $4 per ton = 174x4 = $696

(iv) Erection @ $10 per ton = 174 x 10 = $1,740

(v) Slab 28' wide @ $28.56 per foot = 80 x 28.56 = $2,285

(vi) Diaphragms = $398

Hence, total cost of deck = $15,799

4. FOR 90' SPAN

a. Design

54" deep T section: 24" wide flange 5" thick. Web thickness = 8 in.

Gross Properties: Area = 521 sq. in., S.M. = 11,450 in^3.

Yt = 21.9", Yb = -32.1", e = -25.1", I = 141,946 in^4.

Girder Moment = 6,600,000 lbs. in.

Provide 101 wires .196" dia., area = 3.03 sq. in.

Slab Moment = 2,810,000 lbs. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 900	+ 1,020	-	+ 120 psi.
	+ 3,490	- 1,490	-	+ 2,000 psi.
Final	- 765	+ 1,020	+ 434	+ 689 psi.
	+ 2,970	- 1,490	- 635	+ 845 psi.

Composite Section Properties: Area, 764.21 sq. in.

S.M. = 14,998 in^3., Yt = 19.6", Yb = -38.4", I = 270,248 in^4.

Max. L.L. Moment with 23.2 percent impact = 15,420,000 lbs. in. per lane. Hence, design L.L. moment = 5,400,000 lbs. in.

Stress @	Before L.L.	L.L.	Combined
Top of slab	0	+ 392	+ 392 psi.
Top of girder	+ 689	+ 312	+ 1,001 psi.
Bottom of girder	+ 845	- 768	+ 77 psi.

Cracking Moment = 8,700,000 lbs. in.

Stress @	Before C.M.	C.M.	Combined
Top of slab	0	+ 630	+ 630 psi.
Top of girder	+ 689	+ 500	+ 1,189 psi.
Bottom of girder	+ 845	-1,230	- 385 psi.

Ultimate R.M.: Mu = 2.37 (D.L. + L.L.) or D.L. + 6.85 L.L.

L.L. Deflection $= \frac{1}{2,450}$ span

Diaphragms:

Provide 4 Nos. 36" x 9" section with two cables each having an area of 0.40 sq. in. at the total cost of $571.

b. Quantity for 9 prestressed beams

(i) Concrete: 9 x 90 x 521/144 = 2,930 cu. ft.

(ii) Cable: 9 x 90 x 10.43 = 8,450 lbs.

c. Cost of deck

(i)	Concrete @ $3 per cu. ft. = 2,930 x 3	= $8,790
(ii)	Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 8,450 x 0.6	= $5,070
(iii)	Transportation up to 100 miles @ $4 per ton = 220 x 4	= 880
(iv)	Erection @ $10 per ton = 220 x 10	= 2,200
(v)	Slab 28' wide @ $28.56 per foot = 90 x 28.56	= 2,570
(vi)	Diaphragms	= 571

Hence, total cost of deck = $20,081

Spiral shear connectors:

Using 5/8" Ø bar with 4-1/2" mean coil diameter:

Spiral	Pitch	Length	Lbs./ft.	Total wt. in lbs.
S_0	5"	10.75'	3.14	33.8
S_{10}	6"	10.75'	2.67	28.8
S_{20}	9"	10.75'	1.96	21.1
S_{30}	12"	10.75'	1.62	17.4
S_{40}	19"	10.75'	1.32	14.2
			Total	115.3 lbs.

Weight of spiral per foot of beam = 2.3 lbs.

Live load deflection = 1/956 of span.

Live load factor of safety = 3.51.

Provide 4 diaphragms each having 2 Ls 3"x3"x3/8" with 28"x3/8" plate.

b. Quantity

(i) 6 Nos. 36 WF. @ 245 for 100' = 245 x 6 x 100 = 147,000 lbs.

6 Nos. 18-1/2" x 1" plate 60' long = 62.9x6x60 = 2,264.4

8 Ls 3" x 3" x 3/8" - 30' long = 7.2 x 8 x 30 = 1,728.0

4 Nos. 28"x3/8" plate 30' long = 35.7 x 4 x 30 = 4,284.0

Total 155,276.4 lbs.

(ii) Spiral shear developers for 6 beams 100' long = 2.3 x 6 x 100 = 1,380 lbs.

c. Cost

(i) Structural steel (fabrication, erection, painting) @ \$0.135 per lb. = 155,276.4 x .135 = 20,962.26

(ii) Spiral shear developers @ 50¢ per lb. = 1380 x 0.50 = 690.00

(iii) 7" slab 28' x 100' @ \$1.378 per sq. ft. = 3,858.40

Hence, total cost of deck, \$25,510.66.

APPENDIX B

PRESTRESSED CONCRETE CONSTRUCTION FOR 50' - 70' - 80' - 90'- 100' SPAN

1. FOR 50' SPAN

a. Design

30" deep T section: 24" wide flange 4-1/2" thick. Web thickness = 8 in.

Gross properties: Area = 321 sq. in. S.M. = 3,811.5 in^3.

Yt = 11.9", Yb = -18.1", e = -13.6", I = 27,361 in^4.

Girder Moment = 1,250,000 lbs. in.

Provide 52 wires 0.196" dia., area = 1.56 sq. in.

Slab Moment = 866,000 lbs. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 635	- 545	-	- 90 psi.
	- 2830	- 830	-	- 2000 psi.
Final	- 540	- 545	- 376	- 381 psi.
	- 2410	- 830	- 574	- 1006 psi.

Composite Section Properties: Area = 553.92 sq. in.

S.M. = 6,045 in^3., Yt = 10.9", Yb = -23.1", I = 54,283 in^4.

Max. L.L. Moment with 28.6 percent impact = 6,890,000 lbs. in.

per lane. Hence, design L.L. Moment = 2,410,000 lbs. in.

Stress @	Before L.L.	L.L.	Combined
Top of slab	0	- 485	- 485 psi.
Top of girder	- 381	- 306	- 687 psi.
Bottom of girder	-1,006	-1002	- 4 psi.

Cracking Moment = 3,623,000 lbs. in.

Stress @	Before C.M.	C.M.	Combined
Top of slab	0	+ 726	+ 726 psi.
Top of girder	+ 381	+ 502	+ 883 psi.
Bottom of girder	+1,006	-1,540	- 536 psi.

Ultimate R.M.: Mu = 2.46 (DL + LL) or D.L. + 4.96 L.L.

L.L. Deflection = $\frac{1}{2,000}$ span.

Diaphragms:

Provide 2 Nos. 20" x 9" section with two cables each having an area of 0.40 sq. in. at the total cost of $205.

b. Quantity for 9 prestressed beams

(i) Concrete: 9x50x321/144 = 1,000 cu. ft.

(ii) Cable: 9x50x5.38 = 2,420 lbs.

c. Cost of deck

(i) Concrete @ $3 per cu. ft. = 1,000x3	=	$3,000
(ii) Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 2,420x 0.6	=	1,452
(iii) Transportation up to 100 miles @ $4 per ton = 75 x 4	=	300
(iv) Erection @ $10 per ton = 75 x 10	=	750
(v) Slab 28' wide @ $28.56 per ft. = 50 x 28.56	=	1,428
(vi) Diaphragms	=	205

Hence, total cost of deck = $7,135

2. FOR 70' SPAN

a. Design

42" deep T section: 24" wide flange 4-1/2 thick. Web thickness = 8 in.

Gross Properties: Area 417 sq. in., S. M. = 7,261.5 in^3.

Yt = 17.4", Yb = -24.6", e = -20.1", I = 73,492 in^4.

Girder Moment = 3,200,000 lbs. in.

Provide 74 wires @ 1.96" dia., area = 2.22 sq. in.

Slab Moment = 1,700,000 lbs. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 790	+ 756	-	- 34
	+ 3030	-1,030	-	+2,000
Final	- 671	+ 756	+ 402	+ 487
	+ 2580	-1,030	- 570	+ 980

Composite Section Properties: Area = 656.76 sq. in.

S.M. = 10,298 in^3., Yt = -15.7", Yb = -29.3", I = 135,185 in^4.

Max. L.L. Moment with 25.65 percent impact = 10,774,000 lbs. in. per lane. Hence, design L.L. Moment = 3,780,000 lbs. in.

Stress @	Before L.L.	L.L.	Combined
Top of slab	0	+ 422	+ 422 psi.
Top of girder	+ 487	+ 328	+ 815 psi.
Bottom of girder	+ 980	- 850	+ 130 psi.

Cracking Moment = 5,920,000 lbs. in.

Stress @	Before C.M.	C.M.	Combined
Top of slab	0	+ 690	+ 690 psi.
Top of girder	+ 487	+ 515	+1,002 psi.
Bottom of girder	+ 980	-1,330	- 350 psi.

Ultimate R.M.: Mu. = 2.46 (D.L. + L.L.) or D.L. + 6.12 L.L.

L.L. Deflection = $\frac{1}{2,260}$ span.

Diaphragms:

Provide 3 Nos. 28" x 9" section with two cables each having an area of 0.40 sq. in. at the total cost of $367.

b. Quantity

(i) Concrete: 9x70x417/144 = 1,825 cu. ft.

(ii) Cable: 9x70x7.68 = 4,840 lbs.

c. Cost of deck

(i) Concrete @ $3 per cu. ft. = 1,825 x 3 = $5,475

(ii) Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 4,840 x 0.6 = $2,904

(iii) Transportation up to 100 miles @ $4 per ton = 137 x 4 = $548

(iv) Erection @ $10 per ton = 137 x 10 = $1,370

(v) Slab 28' wide @ $28.56 per foot = 70x28.56 = $1,999

(vi) Diaphragms = $ 367

Hence, total cost of deck = $12,663

3. FOR 80' SPAN

a. Design

48" deep T section: 24" wide flange 4-1/2" thick. Web thickness = 8 in.

Gross properties: Area = 465 sq. in., S. M. = 9,411.5 in^3.

Yt = 20.2", Yb = -27.8", e = -23.3", I = 104,162 in^4.

Girder Moment = 2,220,000 lbs. in.

Provide 86 wires 0.196" dia., Area = 2.58 sq. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 917	+ 900	-	- 17 psi.
	+3,240	-1,240	-	+ 2,000 psi.
Final	- 780	+ 900	+431	+ 581 psi.
	+2,760	-1,240	-594	+ 926 psi.

Composite Section Properties: Area = 705.06 sq. in.

S.M. = 12,758 in^3., Yt = 18.1", Yb = -33.9", I = 190,000 in^4.

Max. L.L. Moment with 24.4 percent impact = 13,330,000 lbs. in. per lane. Hence, design L.L. Moment = 4,650,000 lbs. in.

Stress ℄	Before L.L.	L.L.	Combined
Top of slab	0	+ 444	+ 444 psi.
Top of girder	+ 581	+ 346	+ 927 psi.
Bottom of girder	+ 926	- 830	+ 96 psi.

Cracking Moment = 7,360,000 lbs. in.

Stress ℄	Before C.M.	C.M.	Combined
Top of slab	0	+ 702	+ 702 psi.
Top of girder	+ 581	+ 507	+1,088 psi.
Bottom of girder	+ 926	-1,330	- 404 psi.

Ultimate R.M.: Mu = 2.44 (D.L. + L.L.) or D.L. + 6.12 L.L.

L.L. Deflection $= \frac{1}{2,240}$ span

Diaphragms:

Provide 3 Nos. 32" x 9" section with two cables each having an area of 0.40 sq. in at the total cost of $398.

b. Quantity for 9 prestressed beams

(i) Concrete: 9 x 80 x 465/144 = 2,320 cu. ft.

(ii) Cable: 9 x 80 x 8.62 = 6,200 lbs.

c. Cost of deck

(i) Concrete @ $3 per cu. ft. = 2,320 x 3 = $6,960

(ii) Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 6,200x0.6 = $3,720

(iii) Transportation up to 100 miles @ $4 per ton = 174x4 = $696

(iv) Erection @ $10 per ton = 174 x 10 = $1,740

(v) Slab 28' wide @ $28.56 per foot = 80 x 28.56 = $2,285

(vi) Diaphragms = $398

Hence, total cost of deck = $15,799

4. FOR 90' SPAN

a. Design

54" deep T section: 24" wide flange 5" thick. Web thickness = 8 in.

Gross Properties: Area = 521 sq. in., S.M. = 11,450 in^3.

Yt = 21.9", Yb = -32.1", e = -25.1", I = 141,946 in^4.

Girder Moment = 6,600,000 lbs. in.

Provide 101 wires .196" dia., area = 3.03 sq. in.

Slab Moment = 2,810,000 lbs. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 900	+ 1,020	-	+ 120 psi.
	+ 3,490	- 1,490	-	+ 2,000 psi.
Final	- 765	+ 1,020	+ 434	+ 689 psi.
	+ 2,970	- 1,490	- 635	+ 845 psi.

Composite Section Properties: Area, 764.21 sq. in.

S.M. = 14,998 in^3., Yt = 19.6", Yb = -38.4", I = 270,248 in^4.

Max. L.L. Moment with 23.2 percent impact = 15,420,000 lbs. in. per lane. Hence, design L.L. moment = 5,400,000 lbs. in.

Stress @	Before L.L.	L.L.	Combined
Top of slab	0	+ 392	+ 392 psi.
Top of girder	+ 689	+ 312	+ 1,001 psi.
Bottom of girder	+ 845	- 768	+ 77 psi.

Cracking Moment = 8,700,000 lbs. in.

Stress @	Before C.M.	C.M.	Combined
Top of slab	0	+ 630	+ 630 psi.
Top of girder	+ 689	+ 500	+ 1,189 psi.
Bottom of girder	+ 845	-1,230	- 385 psi.

Ultimate R.M.: Mu = 2.37 (D.L. + L.L.) or D.L. + 6.85 L.L.

L.L. Deflection = $\frac{1}{2,450}$ span

Diaphragms:

Provide 4 Nos. 36" x 9" section with two cables each having an area of 0.40 sq. in. at the total cost of $571.

b. Quantity for 9 prestressed beams

(i) Concrete: 9 x 90 x 521/144 = 2,930 cu. ft.

(ii) Cable: 9 x 90 x 10.43 = 8,450 lbs.

c. Cost of deck

(i)	Concrete @ $3 per cu. ft. = 2,930 x 3	= $8,790
(ii)	Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 8,450 x 0.6	= $5,070
(iii)	Transportation up to 100 miles @ $4 per ton = 220 x 4	= 880
(iv)	Erection @ $10 per ton = 220 x 10	= 2,200
(v)	Slab 28' wide @ $28.56 per foot = 90 x 28.56	= 2,570
(vi)	Diaphragms	= 571

Hence, total cost of deck = $20,081

5. FOR 100' SPAN

a. Design

60" deep T section: 24" wide flange 5" thick. Web thickness = 8 in.

Gross Properties: Area = 569 sq. in., S.M. = 14,574 in^3.

Yt = 25.7", Yb = -34.3", e = -29.3", I = 199,846 in^4.

Girder Moment = 8,860,000 lbs. in.

Provide 116 wires 0.196" dia., area = 3.48 sq. in.

Slab Moment = 3,460,000 lbs. in.

Stress Analysis:

	Prestress	Girder	Slab	Combined
Initial	- 1,050	+ 1,140	-	+ 90 psi.
	+ 3,540	- 1,540	-	+2,000 psi.
Final	- 895	+ 1,140	+ 445	+ 700 psi.
	+ 3,020	- 1,540	- 594	+ 886 psi.

Composite section properties: Area = 815.5 sq. in.

S.M. = 19,078 in^3., Yt = 23.4", Yb = -40.6", I = 346,462 in^4.

Max. L.L. Moment with 22.2 percent impact = 18,330,000 lbs. in. per lane. Hence, Design L.L. moment = 6,420,000 lbs. in.

Stress @	Before L.L.	L.L.	Combined
Top of slab	0	+ 447	+ 447 psi.
Top of girder	+ 700	+ 370	+ 1,070 psi.
Bottom of girder	+ 886	- 774	+ 112 psi.

Cracking Moment = 10,413,000 lbs. in.

Stress @	Before C. M.	C.M.	Combined
Top of slab	0	+ 704	+ 704 psi.
Top of girder	+ 700	+ 583	+ 1,283 psi.
Bottom of girder	+ 886	-1,220	- 334 psi.

Ultimate R.M.: Mu = 2.48 (D.L. + L.L.) or D.L. + 5.1 L.L.

L.L. Deflection $= \frac{1}{2,350}$ span

Diaphragms:

Provide 4 Nos. 40" x 9" section with two cables each having an area of 0.40 sq. in. at the total cost of $612.

b. Quantity for 9 prestressed beams

(i) Concrete: 9 x 100 x 569/144 = 3,550 cu. ft.

(ii) Cable: 9 x 100 x 12 = 10,800 lbs.

c. Cost of deck

(i)	Concrete @ $3 per cu. ft. = 3,550 x 3	=	$10,650
(ii)	Cable (includes fitting, prestressing, anchoring) @ 60 cents per lb. = 10,800 x 0.60	=	$ 6,480
(iii)	Transportation up to 100 miles @ $4 per ton = 266.2 x 4	=	1,065
(iv)	Erection = 266.2 x 10	=	2,662
(v)	Slab 28' wide @ $28.56 per foot = 100 x 28.56	=	2,856
(vi)	Diaphragms	=	612

Hence, total cost of deck = $24,325

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