DESIGN OF A BOX GIRDER BRIDGE

by

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NOMENCLATURE

a = depth of Whitney's stress block.

A_g = gross cross-sectional area.

A_s = area of steel.

C.L.L. = concentrated live load.

b = width of section.

B.M. = bending moment.

D.L. = dead load.

E = earthquake load.

f'_c = compressive strength of concrete.

f_y = yield strength of reinforcement.

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

L = length.

L.L. = live load.

M_B = bending moment at B.

M_C = bending moment at C.

M_D = bending moment at D.

M_u = ultimate bending moment.

S = spacing.

S.F. = shear force.

U = required ultimate load capacity of sections.

U.L.L. = uniform live load.

U_u = bond stress.

V_A = shear at A.

V_B = shear at B.

V_u = total ultimate shear.

V_u' = ultimate shear carried by web reinforcement.

W = wind load.

ϕ = capacity reduction factor.

Σo = perimeter of bars.

p = percentage of steel.

INTRODUCTION

Continuous concrete bridges with concrete piers are used for many stream crossings and grade separations. Continuous slab bridges are often built for spans under approximately 35 feet. For spans between 35 feet and 150 feet, T-girders offer an economical solution (1). In case of spans more than 150 feet, dead load will normally be too high for economical use of T-girder bridges. To minimize the dead load in case of long spans, hollow continuous concrete girder bridges are built for spans of 60 feet to 250 feet.

Continuous girder bridges are best proportioned when the interior spans are from 1.3 to 1.4 times the length of the end spans for loadings and stresses in common use (1).

Advantages of A Continuous Bridge Over A Bridge Built With Simple Spans

1. In the case of a continuous bridge, the piers can often be placed on the stream bank or outside the main channel of the stream crossings, or at the sides of the roadway for grade separations.

2. Single bearings only are required on interior supports of continuous bridges. The width of piers may thus be reduced as compared to simple spans.

3. The continuous bridges require fewer expansion joints. This reduces the first cost and cost of maintenance.

Length and Span Ratio of Bridges

The length and span depend entirely on the site conditions.

In every case it will be a matter for investigation on the part of the engineer to decide the overall length and the span ratios of a bridge. It may be taken as a general rule that if the foundation work is comparatively costly, it will be better to reduce the number of foundations as much as possible and use comparatively long spans. If, on the other hand, the foundations are likely to be inexpensive, it may be more economical to use short spans and a comparatively large number of foundations. A rule of thumb which has been suggested is that in multiple-span bridges the number of spans should be such that the cost of substructure and superstructure should be equal. This rule cannot be followed too rigidly. It is therefore a matter for the engineer in charge of design to decide in each case (3).

Spacing of Main Girders

The spacing of longitudinal main girders affects to a large extent the cost of the bridge; therefore, comparative estimates of several arrangements should be made before the final arrangement is adopted. The close spacing of girders means thinner slabs and a large number of main girders. Wide spacing of main girders means thicker slabs but a smaller number of girders.

In the United States, the cost of labor for form work is large in comparison to the cost of materials. Comparatively wide spacing of girders is therefore economical. Generally, a spacing of 8 to 10 feet is adopted for girders (4).

ULTIMATE STRENGTH DESIGN

Until very recently, most methods of structural design have been based on the assumption that the stress and strain are proportional, i.e., the material behaves in a purely elastic manner. This assumption is far from the truth, especially in the case of concrete (Refer to Plate I).

While steel behaves as an elastic material, i.e., there is a straight-line relationship between unit stress and unit strain almost to its yield point (40,000 psi minimum for intermediate grade steel), this is not true for concrete. For the standard rate of loading employed in cylinder tests, the graph is reasonably straight up to 50 per cent of f'_c. After that it becomes curved. When the stress is f'_c, the strain is from 0.0005 to 0.0015 inch per inch. The strain at fracture is from 0.004 to 0.008 inch per inch (5). This indicates that stresses and strains in concrete are proportional only at relatively low stresses, but at higher stresses the strain increases at a faster rate than the stress. This "inelastic" behavior of concrete at higher stresses is known as "plasticity".

History and Development

As early as 1905, Talbot recognized that, "even if the straight line relation be accepted as sufficient for use with ordinary working stress, the parabolic or other variable relation must be used in discussing experimental data when any considerable deformation is developed in the concrete." (6). Since

PLATE I

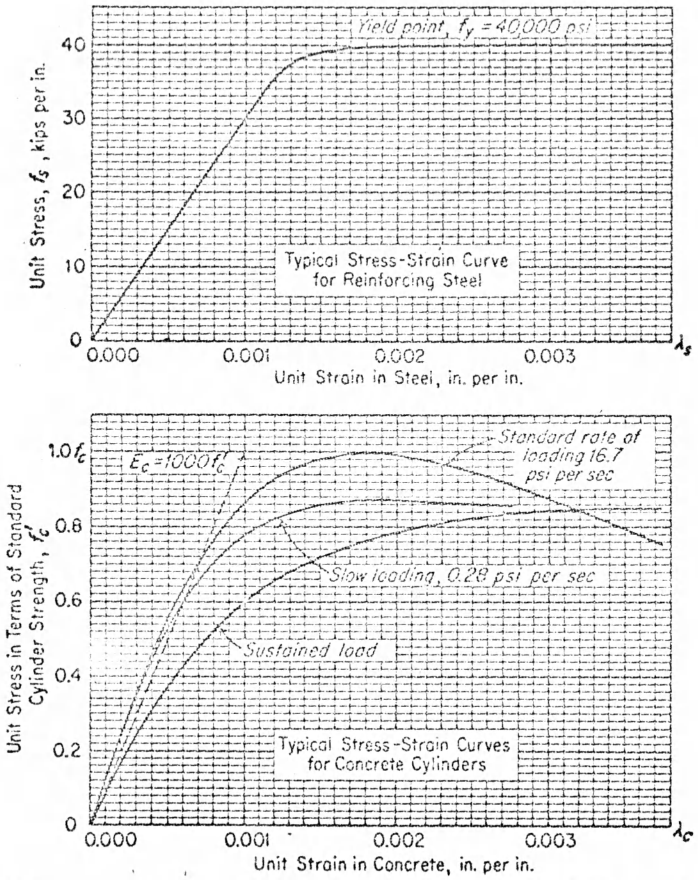


FIG. 4.2. The concrete curves of this figure are taken from C. S. Whitney, "Application of Plastic Theory of the Design of Concrete Structures," *J. Boston Soc. Civil Engrs.*, vol. 25, no. 1, 1948.

(a) Typical stress-strain curve for reinforcing steel.

(b) Typical stress-strain curves for concrete cylinders.

that time many proposals have been made for improving reinforced concrete design, but the straight-line relation has remained the basis of design, primarily due to its simplicity. In many cases this method led to uneconomical design, i.e., many reinforced concrete members proved to be much stronger than indicated by elastic theory. Smaller dimensions could often have been used if a more accurate design method had been available.

The American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE) formed a joint committee on reinforced concrete ultimate strength design in 1952. This committee published its report in 1955 (7). The report is based on evaluation and theoretical analysis of hundreds of tests. In this report, procedures are proposed for ultimate strength designs of tension-reinforced beams with and without compression reinforcement, and of concentrically and eccentrically loaded columns, rectangular as well as circular. These procedures permit the ultimate strength (failure load) of such members to be computed with considerable accuracy. The 1956 ACI Code states that "the ultimate strength design may be used for design of reinforced concrete members." That is, the Code permits the choice of the elastic method or ultimate strength method by the designer in charge.

Ultimate Strength Design Theory

The term "ultimate strength design" indicates a method of design based on the ultimate strength of a reinforced concrete cross-section in simple bending, combined bending and axial load, shear, or bond on the basis of inelastic action (7).

The advantages resulting from design by ultimate strength theory are:

1. The behavior of concrete is not elastic at higher stresses. Under some circumstances, the ultimate strength may be 50 per cent greater than that predicted by elastic design methods; therefore, it is sometimes uneconomical to use the elastic method (7).

2. Dead load remains unchanged throughout the life of a structure, but actual live loads are not predictable. They are beyond the designer's control. Therefore, it is unreasonable to apply the same load factors to dead and live loads. Ultimate strength design allows different factors, thus keeping in view the safety of the structure (7).

3. Conventional column design is a modified ultimate strength procedure, whereas the straight line theory is used for design for simple flexure. It is unavoidable, therefore, that various inconsistencies occur in design of sections subject to both axial load and bending. Designing all types of members on the basis of ultimate strength results in consistency in the design procedures (7).

4. For prestressed concrete it is necessary that design recommendations include investigation of ultimate strength to determine the factor of safety since, at high loads, stresses do not vary linearly. Straight line theory is therefore not applicable, and ultimate strength theory must be used (7).

The following equations for design loads for structures are given in the ACI Code (8):

$$U = 1.5D + 1.8L \qquad U = 0.9D + 1.1W$$

$$U = 1.25\ (D + L + W)$$

$$U = 1.25\ (D + L + E)$$

U = Ultimate load.

D.L.= Dead load.

L.L.= Live load plus impact (if any).

W = Effect of wind load.

E = Effect of earthquake.

The greatest load resulting from the above equations will be considered in the design.

The 1963 ACI Building Code gives the formulae for the design of reinforced concrete structures by the ultimate strength method. All these formulae are derived from first principles, but they are then multiplied by a reduction factor ϕ. For example, for a beam with tension reinforcement only,

$$M_u = \phi \left\{ A_s f_y\ (d - a/2) \right\}$$

M_u = Ultimate bending moment.

ϕ = 0.9 for flexure.

= 0.85 for diagonal tension, bend and anchorage.

= 0.75 for spirally reinforced compression members.

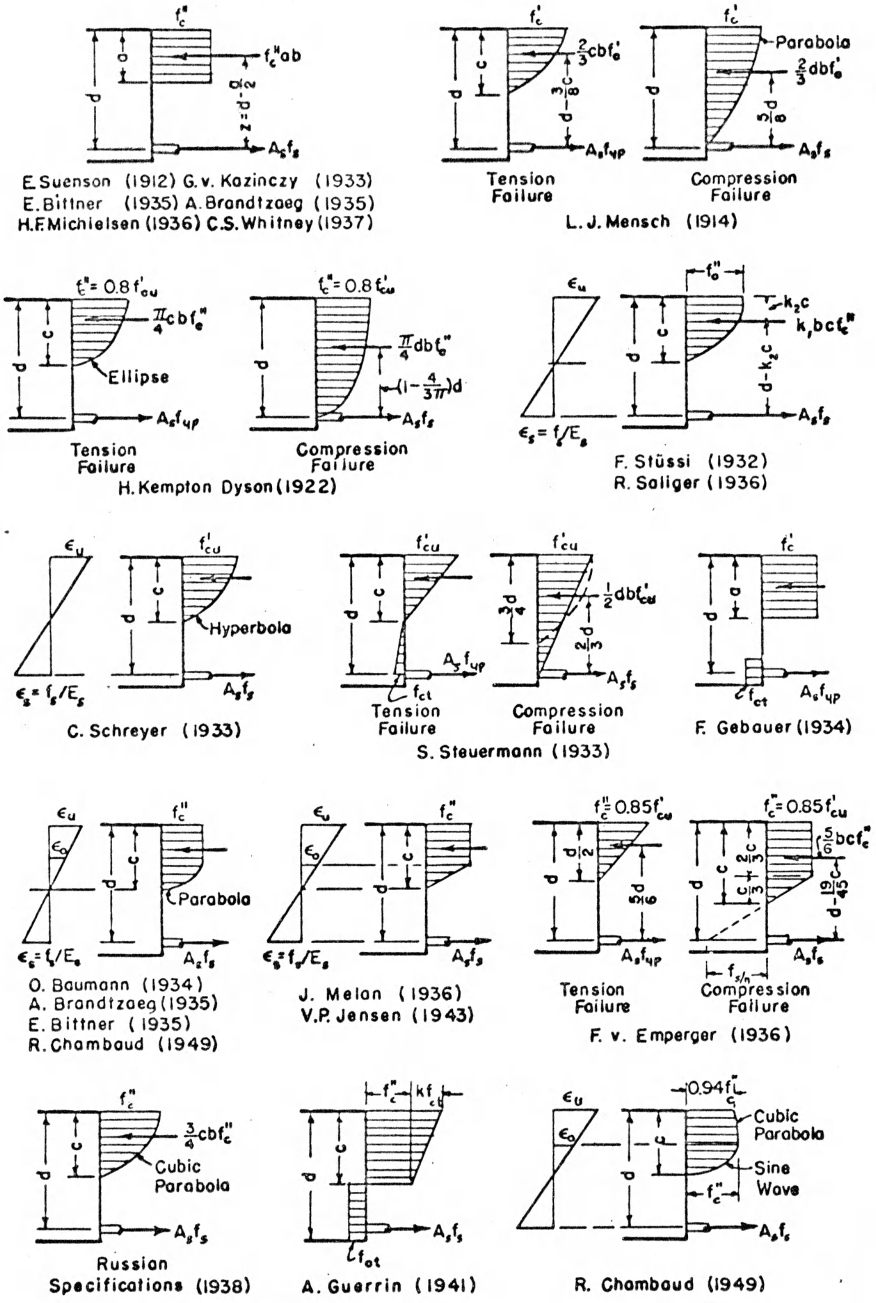
ϕ = 0.70 for tied compression members.

According to the 1963 ACI Building Code,

At ultimate strength, a concrete stress intensity of 0.85 f'_c shall be assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross-section, and a straight line located parallel to the neutral axis at a distance $A = k_1c$ from the fibre of maximum strain to the neutral axis is measured in a direction perpendicular to that axis. The fraction k_1 shall be taken as 0.85 for strengths 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.

Plate II shows a variety of stress diagrams which have been assumed during the period 1914 to 1949.

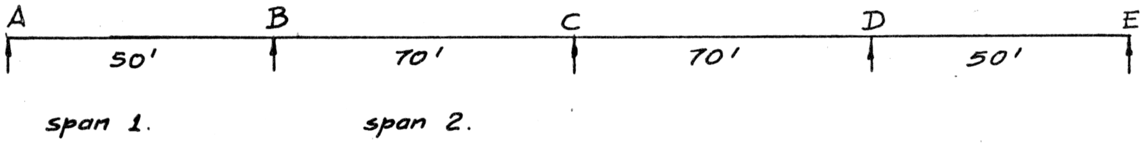
PLATE II



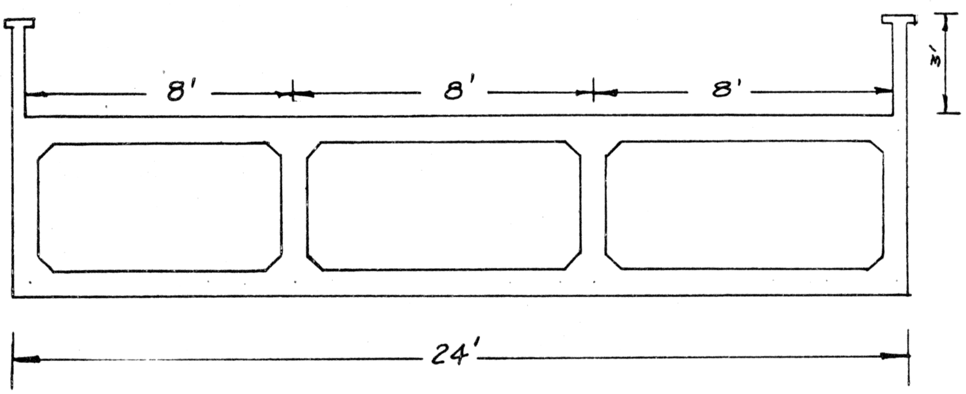
A variety of stress diagrams which have been assumed during the period 1914 to 1949.

DATA

As recommended by P.C.A. the span ratios of 1:1.4:1.4:1 are used. The various spans are as shown:



(a) Span layout of the bridge.



(b) A transverse cross-section of the two-lane highway bridge.

Fig. 1

LOADINGS AND STRESSES

A.A.S.H.O. Specifications, Equivalent of H-20, S-16,
44 Loading (9)

(18000 for Moment*
(
(26000 for Shear

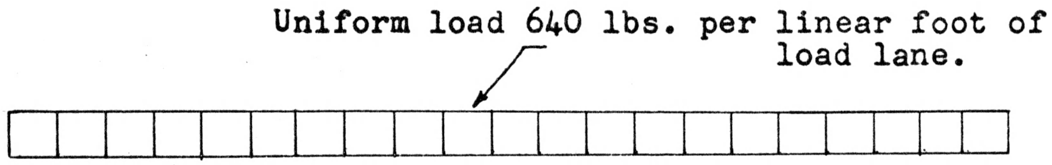


Fig. 2

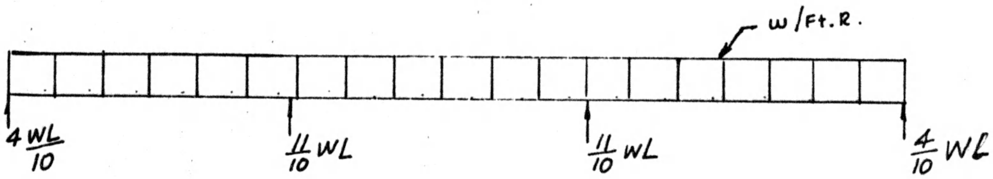
*For continuous spans another concentrated load of equal weight shall be placed in one other span in the series in such position as to produce maximum negative moments.

Stresses

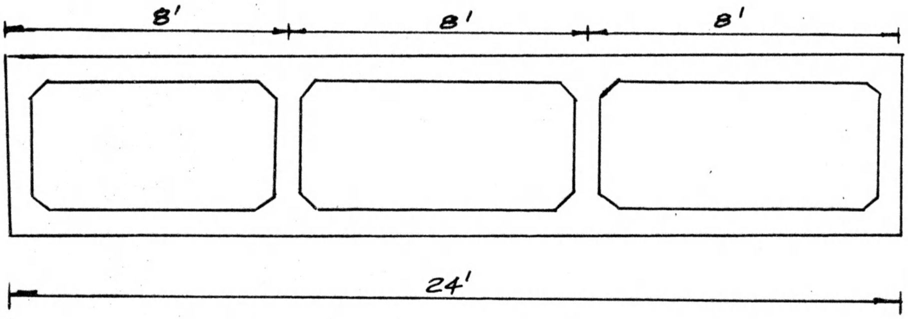
f'_c = 4000 psi. (28 day ultimate compressive strength of concrete)

f_y = 50,000 psi. (Yield stress for steel)

DESIGN OF SLAB



(a) Slab loaded with D.L.



(b) X - Section of roadway.

Fig. 3

Assuming 1' width of web

Span length for slab = 8' - (1' + 6") = 6' - 5"

Assuming 6" thick slab

Dead Load = 75 lbs./sq. ft. (w = 150 lbs./cu.ft. concrete)

<u>20 lbs</u>./sq. ft. (for coating, etc.)

Total D.L.= 95 lbs./sq. ft.

<u>Maximum ultimate dead load moment</u>

$$= \frac{95 \times 6.5 \times 6.5}{10} \times 1.5 \quad = 600 \text{ lb. ft.}$$

<u>Live load moment</u>

$$\text{B.M.} = \frac{S + 2}{32} \; P_{20} \text{ (Impact not included).}$$

$$= \frac{6.5 + 2}{32} \times 16000 \times 0.8$$

$$= \frac{8.5}{32} \times 16000 \times 0.8 = 3400 \text{ lb. ft.}$$

(For slabs continuous over three or more supports, a continuity factor of 0.8 will be applied for both positive and negative moment.) A.A.S.H.O. 1961.

Maximum ultimate live load moment

= 3400 x 1.8

= 6120 lb. ft.

Impact.

Live load stresses produced by H.S. loading will be increased due to dynamic, vibrating and impact effects.

The increase is computed by

$$I = \frac{50}{125+L^*} = \frac{50}{125+6.5} = \frac{50}{131.5} = 0.38 \text{ or } 38\%$$

But maximum allowable = 30%.

Moments due to impact = 0.3 x 6120 = 1836 lb. ft.

$$\text{Total moment (negative)} = \overset{\text{D.L.}}{600} + \overset{\text{L.L.}}{6120} + \overset{\text{Impact}}{1836} = 8556 \text{ lb. ft.}$$

Positive B.M. due to dead load

$$\frac{wx\,(0.8L)^2}{8} = 95 \times 6.5 \times 6.5 \times \frac{0.64}{8} = 322 \text{ lb. ft.}$$

Ultimate D.L. B.M. (positive)

= 322 x 1.5 = 483 lb. ft.

Total ultimate (positive) B.M. = 483 + 6120 + 1836 = 8349 lb. ft.

Since the difference in B.M. (between positive and negative) is small, the same amount of steel will be provided at top and bottom.

---

*L = length of span.

Design of Reinforcement (Ultimate Design)

f_y = 50,000 psi. f'_c = 4,000 psi.

Design.

$$\frac{M_u}{bd^2} = \frac{8556 \times 12}{12 \times 4.75 \times 4.75} = 380$$

From Whitney's Graph corresponding to $\frac{M_u}{bd^2} = 380$,

f_y = 50,000 psi. and f'_c = 4000 psi.

$p = 0.009$

$A_s = pbd$

$A_s = 0.009 \times 12 \times 4.75$ effective depth = 6 - 1.25 = 4.75"

$= 0.514$ sq. in./ft.

Spacing $= 12 \times \frac{0.196}{0.514}$ (½" Ø Area = 0.196)

$= 4.58"$

Main Reinforcement. Provide ½" Ø at 4½" center to center at top and bottom.

Distribution Reinforcement (9).

$$\text{Percentage} = \frac{220}{\sqrt{S}} = \frac{220}{\sqrt{6.5}} = \frac{220}{2.25}$$

$$= 98 > 67 \quad \text{(67\% maximum allowable)}$$

$$A_s = \frac{67}{100} \times 0.514 = 0.344 \text{ sq. in.}$$

$$\text{Spacing} = 12 \times \frac{0.196}{0.344} = 6.84"$$

Provide ½" Ø at 6½" center to center.

Sketch showing steel (see page 44).

UNIT CONCENTRATED LOAD MOMENTS AT SUPPORTS

Span #1 (50')				Span #2 (70')			
	B	C	D		B	C	D
A	0	0	0	B	0	0	0
.1	-1.11	0.301	-0.088	.1	-3.20	-1.00	0.29
.2	-2.11	0.571	-0.167	.2	-4.74	-2.23	0.65
.3	-3.08	0.831	-0.242	.3	-6.28	-3.45	1.01
.4	-3.65	0.986	-0.288	.4	-6.18	-4.45	1.30
.5	-4.23	1.14	-0.333	.5	-6.07	-5.44	1.58
.6	-4.13	1.10	-0.325	.6	-5.00	-5.66	1.62
.7	-4.03	1.08	-0.317	.7	-3.92	-5.68	1.65
.8	-2.98	0.803	-0.234	.8	-2.54	-4.29	1.25
.9	-1.93	0.52	-0.152	.9	-1.17	-2.91	0.85
B	0	0	0	C	0	0	0

The coefficients under B, C, and D represent the bending moment at supports B, C, and D.

UNIT CONCENTRATED LOAD MOMENTS AT TENTH POINTS

Span #1 (50')

	V_A	.1	.2	.3	.4	.5	.6	.7	.8	.9	B
.1	.889	4.40	3.78	3.17	2.56	2.00	1.33	1.17	0.09	-0.5	-1.11
.2	.758	3.79	7.58	6.37	5.16	3.95	2.74	2.37	0.22	-0.9	-2.11
.3	.639	3.20	6.384	9.6	7.78	6.00	4.15	3.58	0.50	-1.27	-3.08
.4	.527	2.635	5.27	7.9	10.54	8.18	5.8	4.905	1.08	-1.28	-3.65
.5	.4153	2.08	4.154	6.231	8.31	10.4	7.46	6.23	1.62	-1.30	-4.23
.6	.3174	1.567	3.174	4.76	6.35	7.94	9.52	7.76	2.70	-0.72	-4.13
.7	.2194	1.10	2.20	3.3	4.39	5.5	6.6	9.30	3.78	-0.13	-4.03
.8	.1404	0.70	1.40	2.1	2.81	3.51	4.21	6.10	5.62	+1.32	-2.98
.9	.0604	0.31	0.614	0.921	1.33	1.54	1.84	2.92	2.45	+2.76	-1.93

UNIT CONCENTRATED LOAD MOMENTS AT TENTH POINTS

Span #2 (70')

	V_B	B	.1	.2	.3	.4	.5	.6	.7	.8	.9	C
.1	.931	-3.2	3.32	2.84	2.36	1.88	1.40	0.92	0.44	-0.04	-0.52	-1.00
.2	.836	-4.74	1.07	6.97	5.80	4.66	3.52	2.37	1.22	-0.07	-1.08	-2.23
.3	.7404	-6.28	-1.1	4.09	9.27	7.45	5.63	3.82	2.00	0.184	-1.63	-3.45
.4	.625	-6.18	-1.81	2.57	6.94	11.31	8.69	6.06	3.43	0.80	-1.82	-4.45
.5	.509	-6.07	-2.5	1.06	4.62	8.18	11.74	8.31	4.87	1.43	-2.00	-5.44
.6	.32	-5.00	-2.256	0.49	3.24	5.98	8.72	11.47	7.20	2.95	-1.30	-5.66
.7	.275	-3.92	-1.97	-0.07	1.852	3.78	5.72	7.62	9.56	5.74	-0.60	-5.68
.8	.165	-2.54	-1.30	-0.09	1.14	2.36	3.59	4.8	6.05	8.52	+1.49	-4.29
.9	.075	-1.17	-0.60	-0.12	0.408	0.935	1.46	2.00	3.53	3.30	+3.57	-2.90

UNIFORM DEAD LOAD MOMENTS AT TENTH POINTS

All Calculations Per Foot Width of Slab

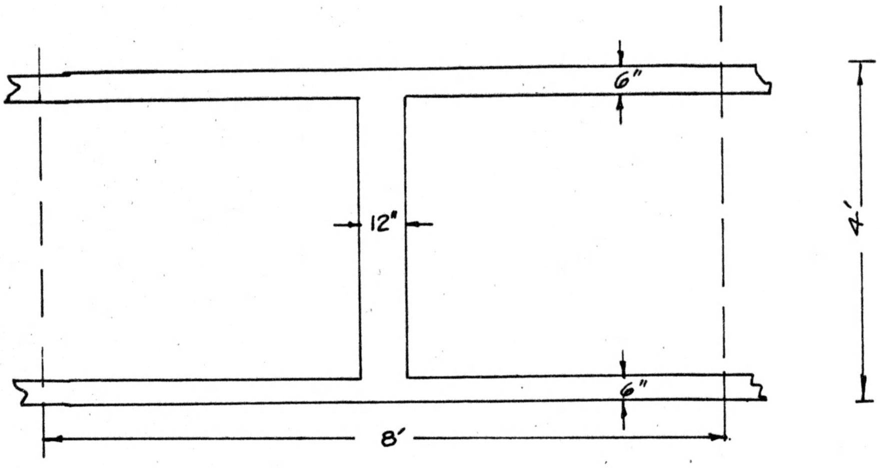


Fig. 4. Typical 8' section.

D.L./ft. of Web.

$$12 \times 36 \times \frac{150}{144} = 450 \; (8')$$

$450/8 = 57$ lb.

D.L./ft.

Slab	150 lb.
Web	57 lb.
	207 lb.

Uniform Dead Load Moments.

$$M_B = M_D = 0.207\ (420.68) - 0.207\ (71.4)$$

$$= 0.207\ (349.28) = -72.2 \text{ K-ft.}$$

$$M_C = 0.207\ (490.14) - 0.207\ (73.3)$$

$$= -86.00 \text{ K-ft.}$$

Span #1 (50')

	A	.1	.2	.3	.4	.5	.6	.7	.8	.9	B
V	3.71	2.68	1.64	0.6	-0.44	-1.46	-2.50	-3.54	-4.58	-5.62	-6.54
M	0	17.3	29.8	36.6	38.8	32.2	27.0	18.7	-8.35	-27.6	-72.2

Span #2 (70')

	B	.1	.2	.3	.4	.5	.6	.7	.8	.9	C
V	7.16	5.71	4.26	2.81	1.36	0.09	-1.54	-3.00	-4.45	-5.9	-7.4
M	-72.2	-46.2	-2.34	23.4	37.2	45	46.0	35.9	-5.86	-74.5	-86.0

L.L. Calculations Per Width of Slab

Live Load Moments

Load in spans #1 and 3 | Load in 2 and 4

$M_B = \frac{0.64}{10}(-136.25 + 71.4) = -4.15\ K'$ $= M_D$

$M_C = \frac{0.64}{10}(+36.6 - 245.07) = -13.3\ K'$ $= M_C$

$M_D = \frac{0.64}{10}(-10.73 - 273.7) = -18.2\ K'$ $= M_B$

Load in spans #1, 2, and 4

$M_B = \frac{0.64}{10}(-136.25 - 273 - 10.73) = -27.0\ K'$

$M_C = \frac{0.64}{10}(36.66 - 245.07 + 36.66) = -11.0\ K'$

$M_D = \frac{0.64}{10}(-10.73 + 71.4 - 136.25) = -4.8\ K'$

Load in spans #2 and 3

$M_B = M_D = \frac{0.64}{10}(-273.7 + 71.4) = -12.9\ K'$

$M_C = \frac{0.64}{10}(-245.07 - 245.07) = -31.4\ K'$

UNIFORM LIVE LOAD MOMENTS AT TENTH POINTS

Span #1 (50')

		A	.1	.2	.3	.4	.5	.6	.7	.8	.9	B
Load in 1 & 3	V	1.518	1.2	0.876	0.556	0.236	-0.1	-0.42	-0.724	-1.04	-1.36	-1.68
	M	0	+6.68	+11.6	+15.0	+17.3	+16.6	+16.0	+14.9	+7.82	2.60	-4.15
Load in 1, 2 & 4	V	1.163	0.84	0.52	0.20	-0.11	-0.43	-0.75	-1.07	-1.39	-1.71	-2.03
	M	0	5.0	8.6	10.1	10.3	8.35	6.29	4.09	-4.65	-14.4	-27

Span #2 (70')

		B	.1	.2	.3	.4	.5	.6	.7	.8	.9	C
Load in 1, 2 & 4	V	2.085	+1.64	1.2	0.76	0.32	-.19	-0.63	-1.07	-1.51	-1.95	-2.325
	M	-2.7	-14.3	-11.0	8.72	14.9	19.5	20.2	20.6	10.2	1.11	-11
Load in 2 & 4	V	2.24	1.79	1.34	0.89	0.44	0	-0.44	-0.89	-1.34	-1.79	-2.24
	M	-16.8	-2.78	8.0	16.1	21.0	23.8	22.8	21.8	12.9	-.282	-13.3
Load in 2 & 3	V	1.81	1.37	0.93	0.49	0.05	-.37	-.81	-1.25	-1.69	-2.13	-2.57
	M	-12.9	-2.8	-1.15	14.4	17.6	17.1	15.3	7.45	-1.38	1.11	-31.4

SUMMARY OF MAXIMUM NEGATIVE MOMENTS AND STEEL FOR VARIOUS SECTIONS
(All Calculations per Foot Width of Slab)

	Point	D.L.	U.L.L.	C.L.L.	C.L.L. in other span	Impact	Total L.L.	Ulti-mate	A_s sq. in.
	A	0	0	0	0	-	-	-	-*
Imp. = 0.204	0.1	17.3	-1.33	0	-1.13	-	-	-	-*
	0.2	29.8	-2.34	0	-2.26	-	-	-	-*
	0.3	36.6	-3.9	0	-3.4	-	-	-	-*
	0.4	38.8	-5.29	0	-4.54	-	-	-	-*
	0.5	32.2	-6.68	0	-5.65	-	-	-	-*
	0.6	27.0	-7.7	0	-6.76	-	-	-	-*
	0.7	18.7	-9.1	0	-7.9	-3.46	-20.4	-8.7	0.0516
	0.8	-8.35	-10.4	0	-9.0	-3.96	-23.36	-54.5	0.323
.178	0.9	-27.6	-14.3	-2.34	-10.9	-5.15	-32.70	-100.4	0.593
0.204	B	-72.2	-27	-7.6	-11.3	-8.65	-54.5	-206.5	1.22
.182	0.1	-46.2	-16.2	-4.5	-6.65	-4.6	-27.35	-118.2	0.700
0.200	0.2	-2.34	-8.95	-.216	-5.65	-3.02	-17.83	-67.1	0.398
Imp. = 0.204	0.3	23.4	-8.9	-	-4.65	-2.76	-16.3	-	-
	0.4	37.2	-9.4	-	-3.8	-2.7	-15.9	-	-
	0.5	45	-9.8	-	-3.6	-2.7	-16.1	-	-
	0.6	46.0	-8.65	-	-3.1	-2.4	-14.15	-	-
	0.7	35.9	-10.7	-	-6.22	-3.45	-20.37	-	-
.191	0.8	-5.86	-15.5	-.126	-7.5	-4.45	-27.57	-137.5	.813
.168	0.9	-74.5	-19.1	-3.6	-8.9	-5.3	-36.9	-178.4	1.06
0.188	C	-86.0	-31.4	-10.2	-10.2	-9.74	-61.54	-240	1.42

*We do not have freely supported beam in practice. At the last support we will have some moment. So, some steel is provided, as shown in Fig. 19.

SUMMARY OF MAXIMUM POSITIVE MOMENTS AND STEEL FOR VARIOUS SECTIONS
(All Calculations per Foot Width of Slab)

	Point	D.L.	U.L.L.	C.L.L.	Impact	Total L.L.	Ultimate moment	A_s sq. in.
	A	0	0	0	0	0	0	-
Impact = 0.204	.1	17.3	6.68	7.92	2.9	17.50	56.7	0.336
	.2	29.8	11.6	14.25	5.26	31.11	101.7	0.600
	.3	36.6	15.0	17.3	6.6	38.9	124.8	0.74
	.4	38.8	17.3	19.0	7.35	43.65	136.7	0.81
	.5	32.2	16.6	18.9	7.24	42.74	125	0.74
	.6	27.0	16.0	17.15	6.75	39.9	112	0.665
	.7	18.7	14.9	16.7	6.45	38.05	96.4	0.57
	.8	-8.35	7.82	10.1	3.66	21.58	26.3	0.156
0.238	.9	-27.6	4.7	4.96	-	-	-	-*
	B	-72.2	-	-	-	-	-	-*
	.1	-46.2	-	-	-	-	-	-*
0.208	.2	-2.34	8.0	12.5	4.18	24.68	40.49	0.24
0.204	.3	23.4	16.1	16.7	6.7	39.5	106.6	0.63
	.4	37.2	21.0	20.4	8.45	49.85	145.8	0.865
	.5	45	23.8	21.1	9.15	54.05	164.8	0.976
	.6	46.0	22.8	20.6	8.85	52.25	163.00	0.965
	.7	35.9	21.8	17.2	7.95	46.95	138.30	0.85
0.22	.8	-5.86	12.9	17.2	6.6	36.7	56.4	0.336
	.9	-74.5	5.12	6.42	-	-	-	-*
	C	-86.0	-	-	-	-	-	-*

*One-third of maximum steel in the span is provided at supports, as shown in Fig. 19.

SUMMARY OF SHEARS AND STEEL FOR SHEAR

	Calculations per foot width of slab			L.L. Shear			V'	#4 Stirrups
	D.L.	U.L.L. + C.L.L.	Impact	8' Length	Ultimate shear in kips	By conc.	By web reinfor.	S
A	3.71	+4.117	+0.84	39.6	115.68	62	53.68	14"
.1	2.68	+3.502	+0.728	33.6	92.5	62	30.5	20"
.2	1.64	+2.928	+0.62	28.4	70.6	62	8.6	24"
.3	0.6	+2.382	+0.5	23.2	48.9	62	-	24"
.4	-0.44	+1.912	+0.425	+18.5	+27.92	62	-	24"
.5	-1.46	-2.38	-0.44	-22.5	-58	62	-	24"
.6	-2.50	-2.91	-0.53	-27.6	-79.5	62	17.5	24"
.7	-3.54	-3.12	-0.556	-29.5	-95.3	62	33.5	20"
.8	-4.58	-3.77	-0.66	-35.4	-118.6	62	56.6	14"
.9	-5.62	-4.27	-0.735	-40.2	-139.9	62	77.9	9.5"
B	-6.54)	-4.74)	-0.78)	-44.2)	-158)	62	-96)	7.5"
	7.16)	+5.38)	+0.87)	+50)	+175)		+113)	6.5"
0.1	5.71	+4.455	+0.77	+41.9	+143.5	62	81.5	9"
0.2	4.26	+3.816	0.678	+36	+115.8	62	53.8	14"
0.3	2.81	+3.212	0.585	+30.4	+88.4	62	26.4	24"
0.4	1.36	+2.604	0.486	+24.7	+60.8	62	-	24"
0.5	0.09	+2.048	+0.394	+19.5	+36.08	62	-	24"
0.6	-1.54	-2.852	-0.600	-27.6	-68.1	62	6.1	24"
0.7	-3.00	-3.517	-0.703	-33.8	-96.8	62	-34.8	20"
0.8	-4.45	-4.138	-0.82	-39.6	-124.6	62	-62.6	12"
0.9	-5.9	-4.837	-0.937	-46.2	-153.8	62	-91.8	8"
C	∓7.4	∓5.104	∓1.00	∓48.8	∓177	62	∓115	6.5"

Shear taken by conc. $= 2\phi \sqrt{f'_c}$

$= 2 \times 0.85 \times \sqrt{4000}$

$= 107.5$ psi.

$107.5 \times 45 \times 12 = 62^K$

BOND STRESSES

Bond stress $U_u = \dfrac{V_u}{\phi \Sigma ojd}$

$\phi = 0.85$

Bond for tension bars $= 4.2 \sqrt{f'_c}$

$= 4.2 \sqrt{4000}$

$= 266$ psi.

Bond for compression bars $= 13 \sqrt{f'_c}$ or 800 psi.

NEGATIVE B. M. CHECK FOR BOND

Point	A_s in sq.in. (per foot)	Number of bars per foot ($\frac{1}{2}$" Ø)	Number of bars for 8'	Perimeter in inches	Ultimate shear in kips	Bond stress in psi.
A	-	-	-	-	-	-
0.1	-	-	-	-	-	-
0.2	-	-	-	-	-	-
0.3	-	-	-	-	-	-
0.4	-	-	-	-	-	-
0.5	-	-	-	-	-	-
0.6	-	-	-	-	-	-
0.7	0.0516	2	16	25.2	95.3	89.7
0.8	0.323	2	16	25.2	118.6	112
0.9	0.593	3	24	37.8	139.9	87.8
B	1.22	7	56	88	-158	42.7
					+175	47.4
0.1	0.700	4	32	50.4	143.5	67.7
0.2	0.398	2	16	12.6	115.8	118
0.3	-	-	-	-	-	-
0.4	-	-	-	-	-	-
0.5	-	-	-	-	-	-
0.6	-	-	-	-	-	-
0.7	-	-	-	-	-	-
0.8	0.813	5	40	62	124.6	47.8
0.9	1.06	6	48	74.4	153.8	49.3
C	1.42	8	64	99.2	177	42.5

POSITIVE B. M. CHECK FOR BOND

Point	A_s in sq.in. (per foot)	Number of bars per foot ($\frac{1}{2}$" Ø)	Number of bars for 8'	Perimeter in inches	Ultimate shear in kips	Bond stress in psi.
A	-	-	-	-	-	-
0.1	0.336	2	16	25.2	92.5	88
0.2	0.600	4	32	50.4	70.6	33.4
0.3	0.74	4	32	50.4	38.9	18.4
0.4	0.81	5	40	62	27.22	10.7
0.5	0.74	5	40	62	58	22.2
0.6	0.665	4	32	50.4	72.5	37.6
0.7	0.57	4	32	50.4	95.3	45
0.8	0.156	2	16	25.2	118.6	112
0.9	-	-	-	-	-	-
B	-	-	-	-	-	-
0.1	-	-	-	-	-	-
0.2	0.24	2	16	25.2	115.8	110
0.3	0.63	3	24	37.8	88.4	55.7
0.4	0.865	5	40	62	60.8	23.4
0.5	0.976	6	48	74.4	36.08	11.5
0.6	0.965	6	48	74.4	68.1	20
0.7	0.85	5	40	62	96.8	37.2
0.8	0.336	2	16	25.2	124.6	118
0.9	-	-	-	-	-	-
C	-	-	-	-	-	-

DEFLECTIONS

Calculations for half-road way (12')

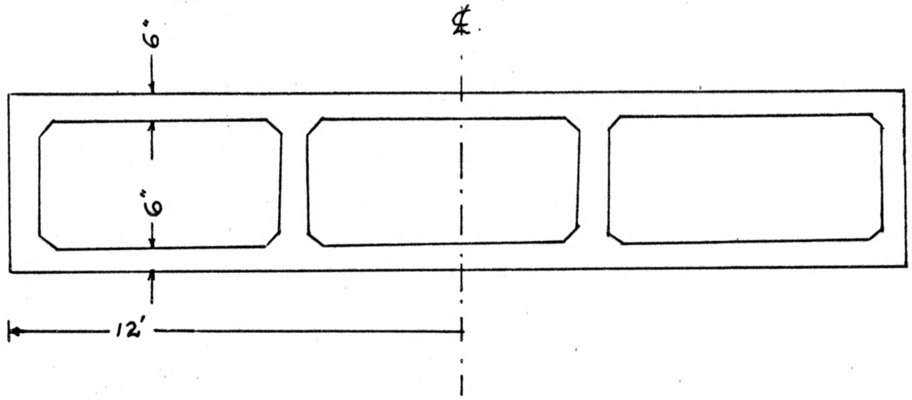


Fig. 5. X - Section of roadway.

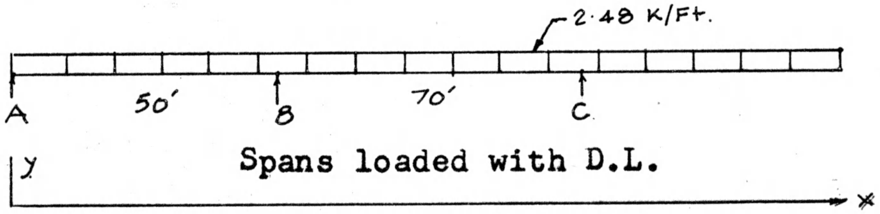
Calculations for moment of inertia.

$$\text{Flanges} \quad 2\left(\frac{1}{12}(12)\left(\tfrac{1}{2}\right)^3 + \left(12 \times \tfrac{1}{2}\right)(1.75)^2\right) = 48.25$$

$$\text{Web} \quad \frac{1}{12}\left(\tfrac{1}{2}\right)(3)^3 + \frac{1}{12}(1)(3)^3 = \underline{3.375}$$

$$51.625$$

Span #1



Spans loaded with D.L.

Fig. 6

$$EIy'' = 45.72x - 2.48\frac{x^2}{2}$$

$$EIy' = 45.72\frac{x^2}{2} - 2.48\frac{x^3}{6} + C_1$$

$$EIy = 45.72\frac{x^3}{6} - 2.48\frac{x^4}{24} + C_1x + C_2 \qquad \begin{pmatrix} x = 0 & x = 50 \\ y = 0 & y = 0 \end{pmatrix}$$

Where C_1 and C_2 are constants of integration

$$C_2 = 0 \qquad C_1 = -6800.$$

$$EIy = 7.62x^3 - 1.02\frac{x^4}{10} - 6800\,x.$$

Span #2.

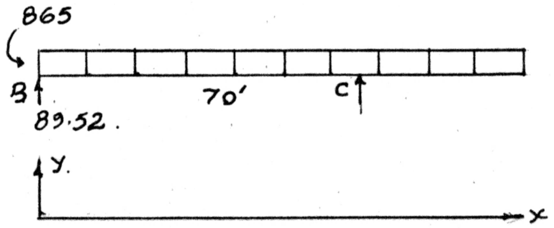


Fig. 7. Spans loaded with D.L.

$$EIy'' = 89.52x - 865 - 2.48\,\frac{x^2}{2}$$

$$EIy' = 89.52\,\frac{x^2}{2} - 2.48\,\frac{x^3}{3} - 865x + C_1$$

$$EIy = 89.52\,\frac{x^3}{6} - 2.48\,\frac{x^4}{24} - 865\,\frac{x^2}{2} + C_1x + C_2$$

Where C_1 and C_2 are constants of integration

$$\begin{pmatrix} x = 0 & & x = 70 \\ y = 0 & & y = 0 \end{pmatrix}$$

$$C_2 = 0$$

$$C_1 = -8500$$

$$EIy = 14.9\,x^3 - 1.02\,\frac{x^4}{10} - 432\,x^2 - 8500\,x$$

$$= x\left(- \frac{x^3}{10} \times 1.02 + 14.9\,x^2 - 432\,x - 8500 \right)$$

$E = 1{,}000{,}000$ psi. $I = 51.625$ ft.^{4}

Span #1		Span #2	
Point	Deflection in ft.	Point	Deflection in ft.
A	0	B	0
0.1	0.00453	0.1	0.0102
0.2	0.00824	0.2	0.0223
0.3	0.0109	0.3	0.0364
0.4	0.0122	0.4	0.0382
0.5	0.0121	0.5	0.0416
0.6	0.0107	0.6	0.0430
0.7	0.00806	0.7	0.0376
0.8	0.00506	0.8	0.0262
0.9	0.00322	0.9	0.0011
B	0	C	0

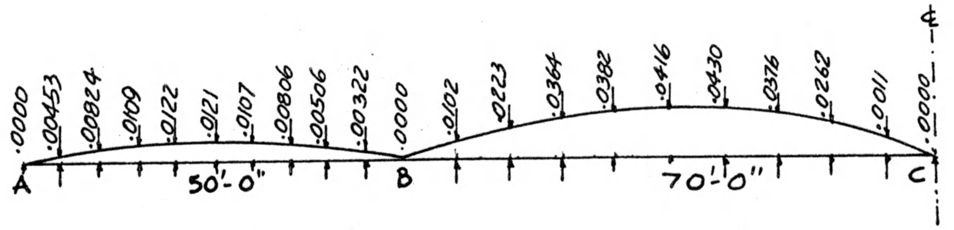
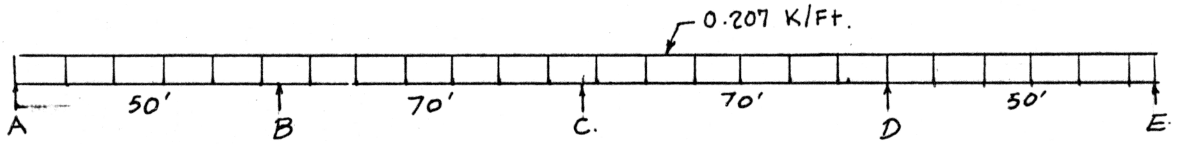
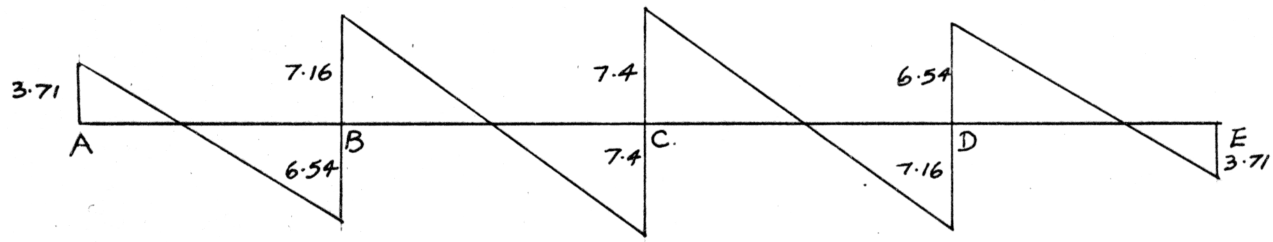


Fig. 8. D.L. camber diagram.



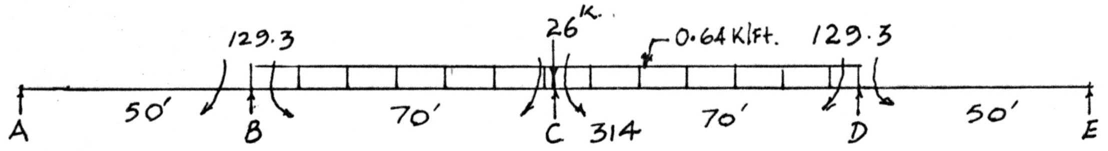
(a) D.L/ft. of slab on 12" width.



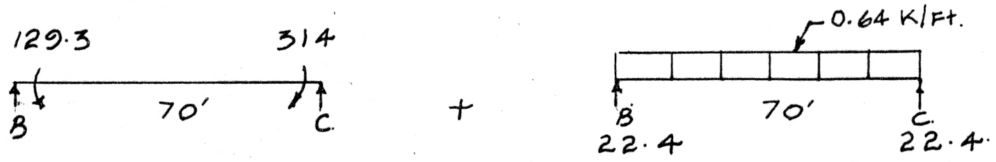
(b) Shear force diagram for dead load.

Fig. 9.

Calculations for Maximum Reaction at C



(a) Loading for maximum reaction at C.



(b) BC taken as free body.

Fig. 10

$$\frac{2(22.4 + 2.64) + 26}{10} \times 1.189 = 9.08$$

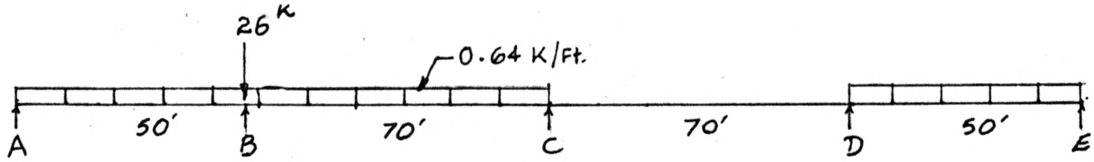
Lane width = 10'

$$\text{Impact} = \frac{50}{125+140} = .189$$

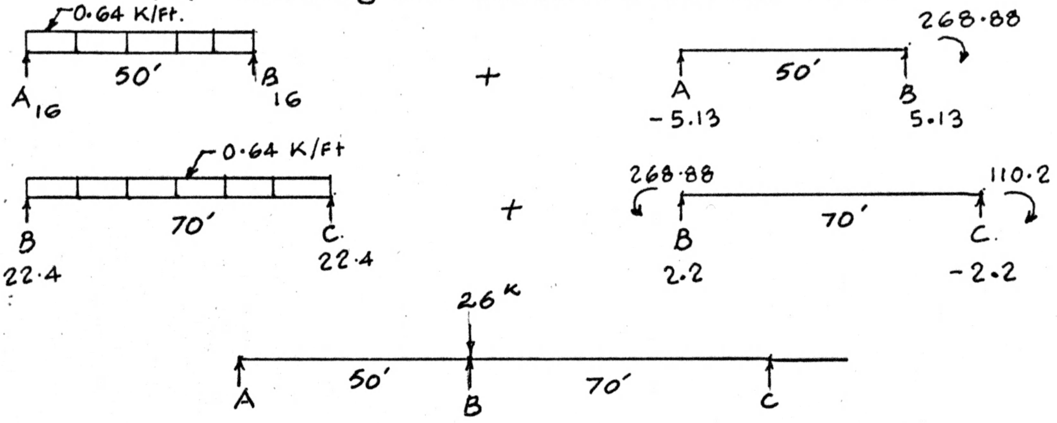
(Per foot of slab) $R_c = 1.5(15.8) + 1.8\ (9.08) = 38.5$

For 8' $R_c = 38.5 \times 8 = 308$

Calculations for Maximum Reaction at B



(a) Loading for maximum reaction at B.



(b) AB and BC as free body.

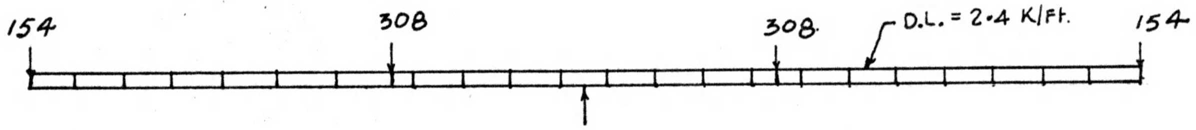
Fig. 11

$$\frac{(16 + 5.13 + 22.4 + 2.2 + 26)}{10} \times 1.17 = 8.4$$

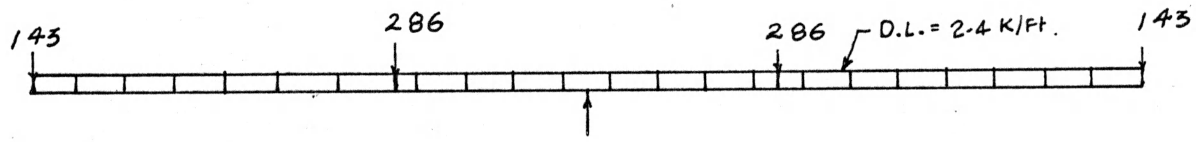
$$1.5(13.7) + 1.8(8.4) = 20.6 + 15.1 = 35.7$$

For 8' width $35.7 \times 8 = 286^K$

DESIGN OF TRANSVERSE BEAMS 1 AND 2



(a) Beam 2 loaded with D.L. and L.L.



(b) Beam 1 loaded with D.L. and L.L.

Fig. 12

Since the difference in loadings between 1 and 2 is 5%, therefore, the same design will be adequate for beams 1 and 2.

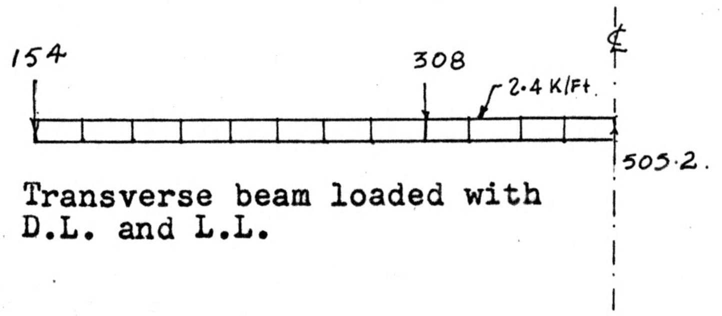
$$\text{B.M. at support} = 154 \times 12 + 308 \times 4 + 2.4 \times 12 \times 6(1.5)$$
$$= 1850 + 1232 + 258$$
$$= 3340$$

$$\frac{M_u}{bd^2} = \frac{3340 \times 1000 \times 12}{48 \times 45 \times 45} = 412$$

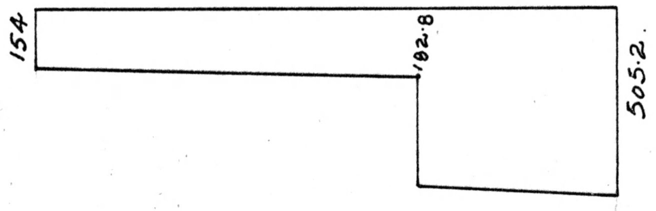
The relation between $\frac{M_u}{bd^2}$ and percentage of steel is given in Whitney's graph (7).

$$A_s = 0.009 \times 48 \times 45 = 19.4 \ (20 - 1\ 1/8'' \ \phi)$$

Shear for Beams 1 and 2



(a) Transverse beam loaded with D.L. and L.L.



(b) S.F. Diagram.

Fig. 13

505.2

<u>248.0</u> by concrete $\left(\frac{107.5 \times 48 \times 45}{1000} \right)$

257.2 to be carried by web reinforcement

$$S = \frac{2 \times 0.306 \times 50{,}000 \times 45}{257.2 \times 1000} = 5.35''$$

$$v_u = \frac{V}{bd} \quad \frac{505.2}{48 \times 45} = 234 \text{ psi.} < 538$$

Stress Restriction (8)

The shear stress, v_u, shall not exceed $10\ \phi \sqrt{f_c'}$ in sections with web reinforcement

$= 10 \times 0.85 \sqrt{4000}$

$= 538$ psi.

DESIGN OF COLUMN

Same Design for Columns 1 and 2

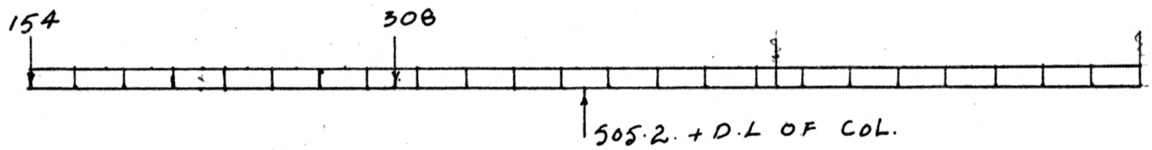


Fig. 14

First condition:

$$B.\ M. = 154 \times 12 + 308 \times 4 = 3082$$

$$e = \frac{3082 \times 12}{505.2} = 74''$$

$$D.L./ft. = \frac{\pi}{4} \times 48^2 \times \frac{150}{144} = 1.73^K$$

Assuming 25' ht. $A_g = 1810$ sq. in.

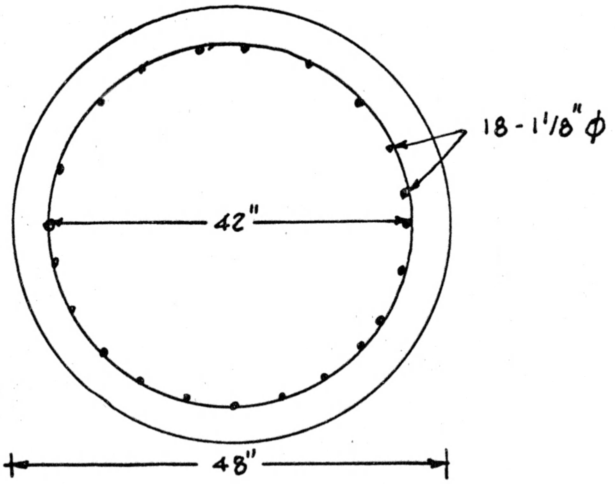


Fig. 15

Design P = 505.2 + 25 x 1.73 x 1.5 (Load factor = 1.5)

= 505.2 + 65 = 570.2

$$P_u = \phi \left(\frac{\dfrac{A_s \times f_y}{\dfrac{3e}{D_s} + 1}}{} \quad \frac{A_g\, f'_c}{\dfrac{9.6\ D\ e}{(0.8D + 0.67\ D_s)^2} + 1.18} \right)$$

Providing 1% steel

$$P_u = 0.85 \left(\frac{\dfrac{\pi}{4} \times \dfrac{48 \times 48}{100} \times 50}{\dfrac{3 \times 74}{42} + 1} + \frac{\dfrac{\pi}{4} \times 48 \times 48 \times 4}{\dfrac{9.6 \times 48 \times 74}{(0.8 \times 48 + 0.67 \times 42)^2} + 1.18} \right)$$

$$= 0.85\ (144 + 818) = 720^K$$

Second condition (Check the section provided above):

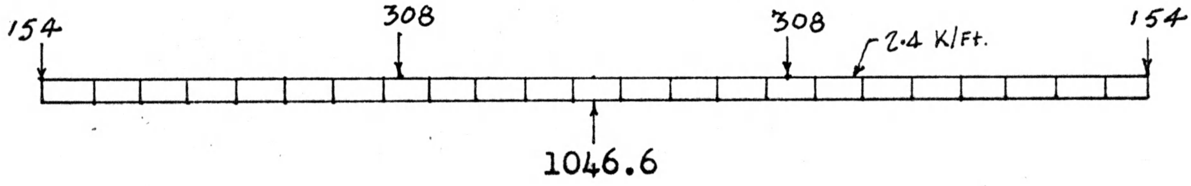


Fig. 16. Transverse beam loaded with D.L. and L.L.

Minimum ecc. by ACI Code = 0.05D

$$= 0.05\ (48) = 2.4$$

$$P_u = 0.85 \left(\frac{\dfrac{\pi}{4} \dfrac{(48)(48)}{100} \times 50}{\dfrac{3 \times 2.4}{42} + 1} + \frac{\dfrac{\pi}{4}(48) = 1810}{\dfrac{9.6 \times 48 \times 2.4}{0.8 \times 48 + 0.67 \times 42} + 1.18} \right)$$

$$= 765 + 613$$

$$= 1378^K$$

Section is safe.

First condition - Section can take 720^K as against 570.2^K required.

Second condition - Section can take 1378^K as against 1046.6^K required.

This seems a conservative design, but a vehicle might hit one of the columns on the lower level. The column might have to resist more load than the designed load; therefore, a somewhat conservative section is provided.

ACKNOWLEDGMENT

The author wishes to express his thanks and appreciation to his Major Instructor, Dr. Robert R. Snell, for his aid and criticism given him during the preparation of this report. An expression of gratitude is extended to Professor Vernon H. Rosebraugh for use of his collection of American Concrete Institute publications.

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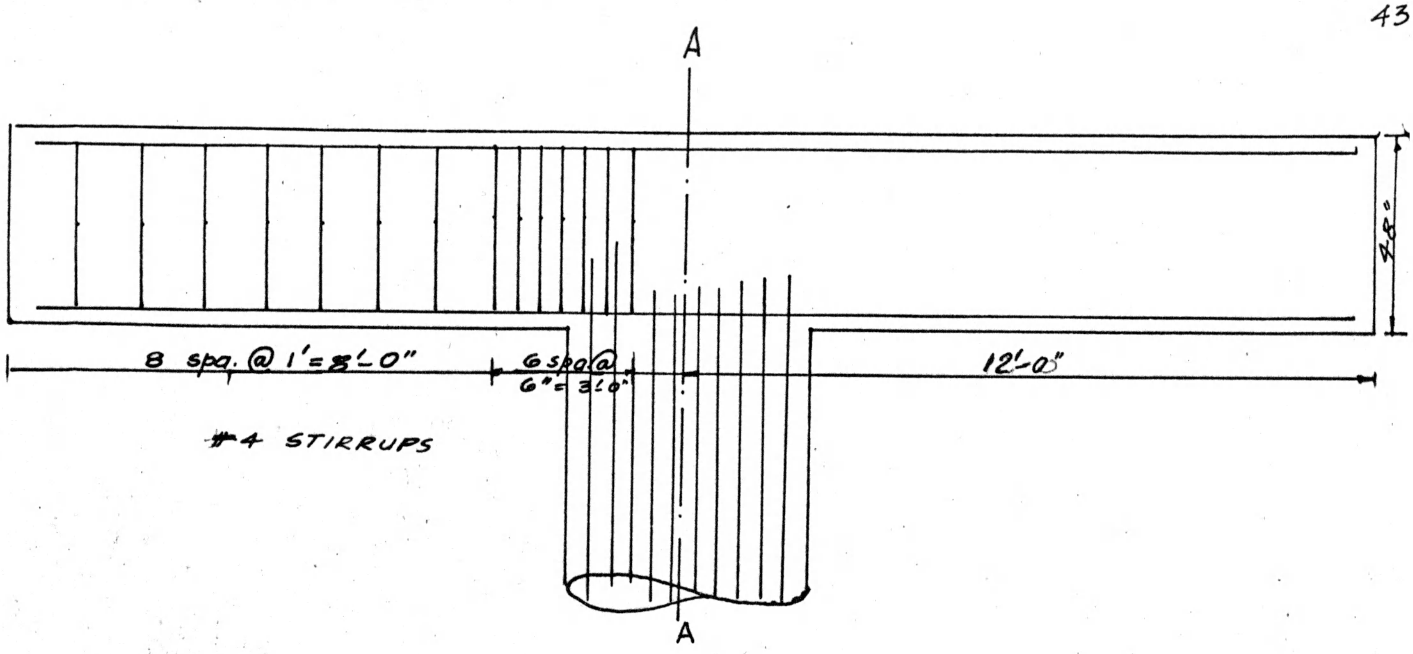
DETAIL DRAWINGS

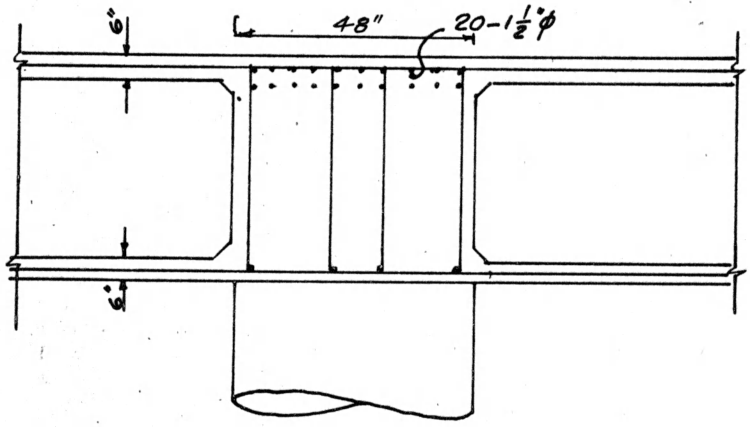
TABLE 1

PARTIAL BILL OF REINFORCING STEEL
FOR THE BOX GIRDER

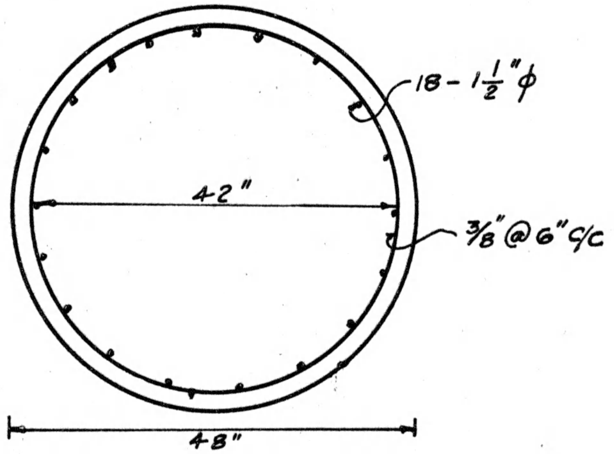
Mark	Size	Length	Center to center distance
G_1	#6	100' - 0"	Spaced alternatively at 16" and 8"
G_2	#6	21' - 0"	Spaced alternatively at 16" and 8"
G_3	#6	26' - 0"	24"
G_4	#6	14' - 0"	24"
H_1	#8	44' - 0"	Spaced alternatively at 6" and 12"
H_2	#8	16' - 0"	12"
I_1	#6	240' - 0"	24"
I_2	#6	37' - 0"	12"
I_3	#6	71' - 0"	24"
I_1'	#6	100' - 0"	24"
J_2	#6	31' - 0"	Spaced alternatively at 18" and 6"
J_3	#6	22' - 0"	Spaced alternatively at 18" and 36"
S_1	#6	24' - 0"	4½"
S_2	#6	240' - 0"	6½"
S_3	#3	24' - 0"	12"



a) CROSS SECTION OF TRANSVERSE BEAM

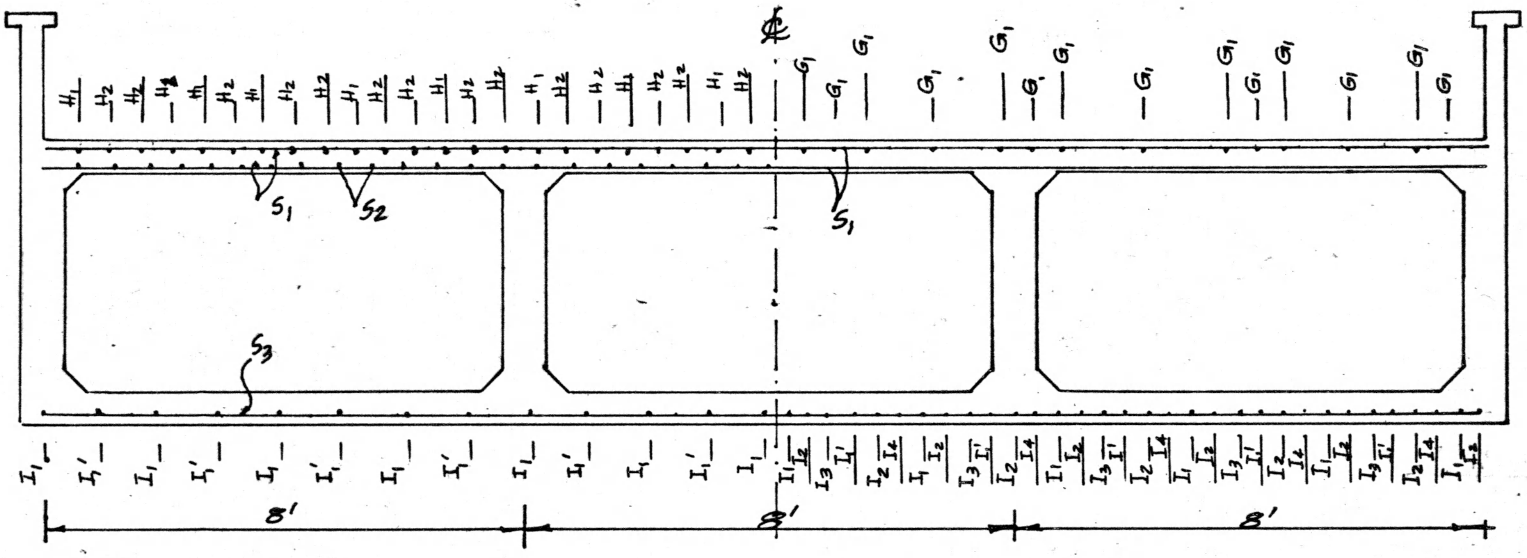


b) SECTION A – A



c) CROSS SECTION OF COLUMN

FIG. 18



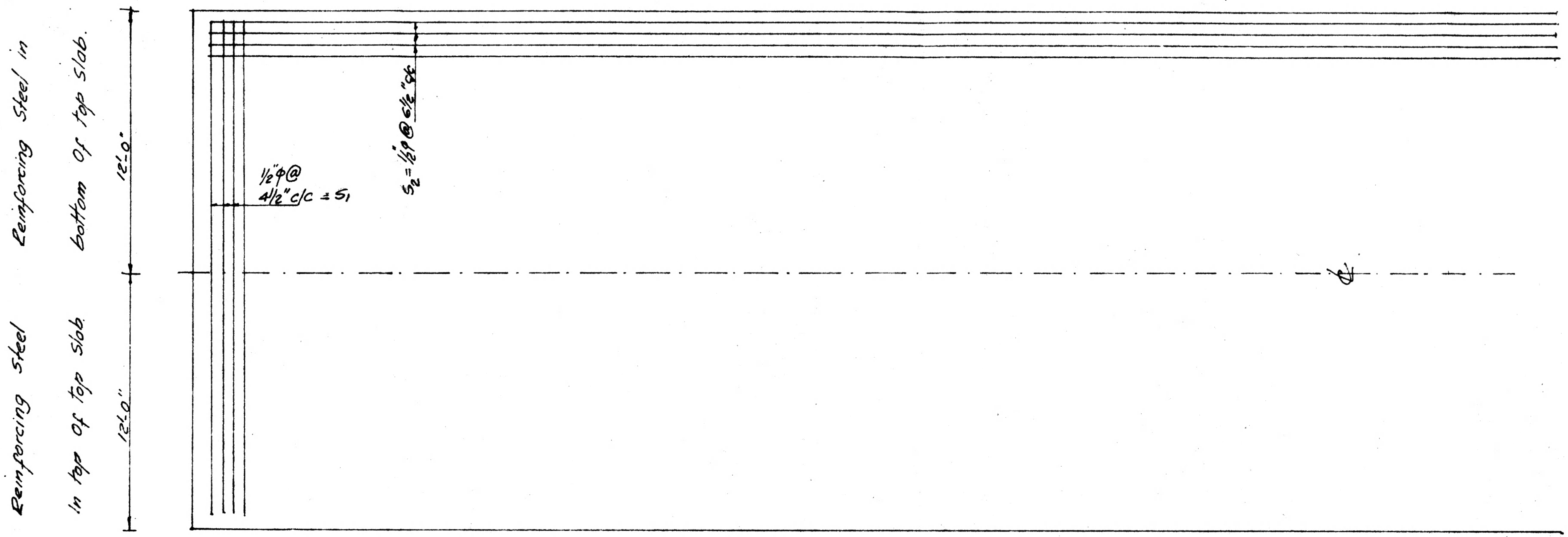
Half Section Near Pier-C

Half Section Near Midspan

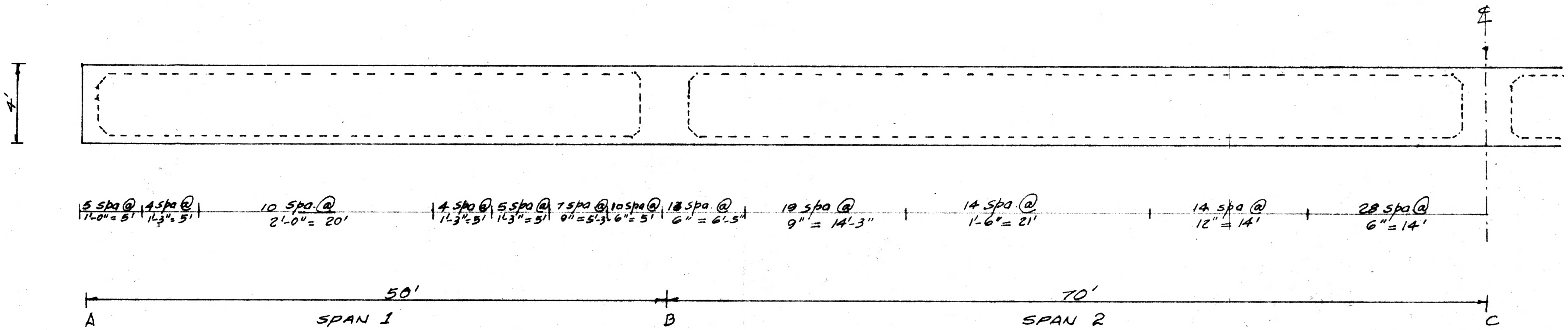
SPAN BC

For Notations Of Bars See Table 1. Page (42)

FIG 19

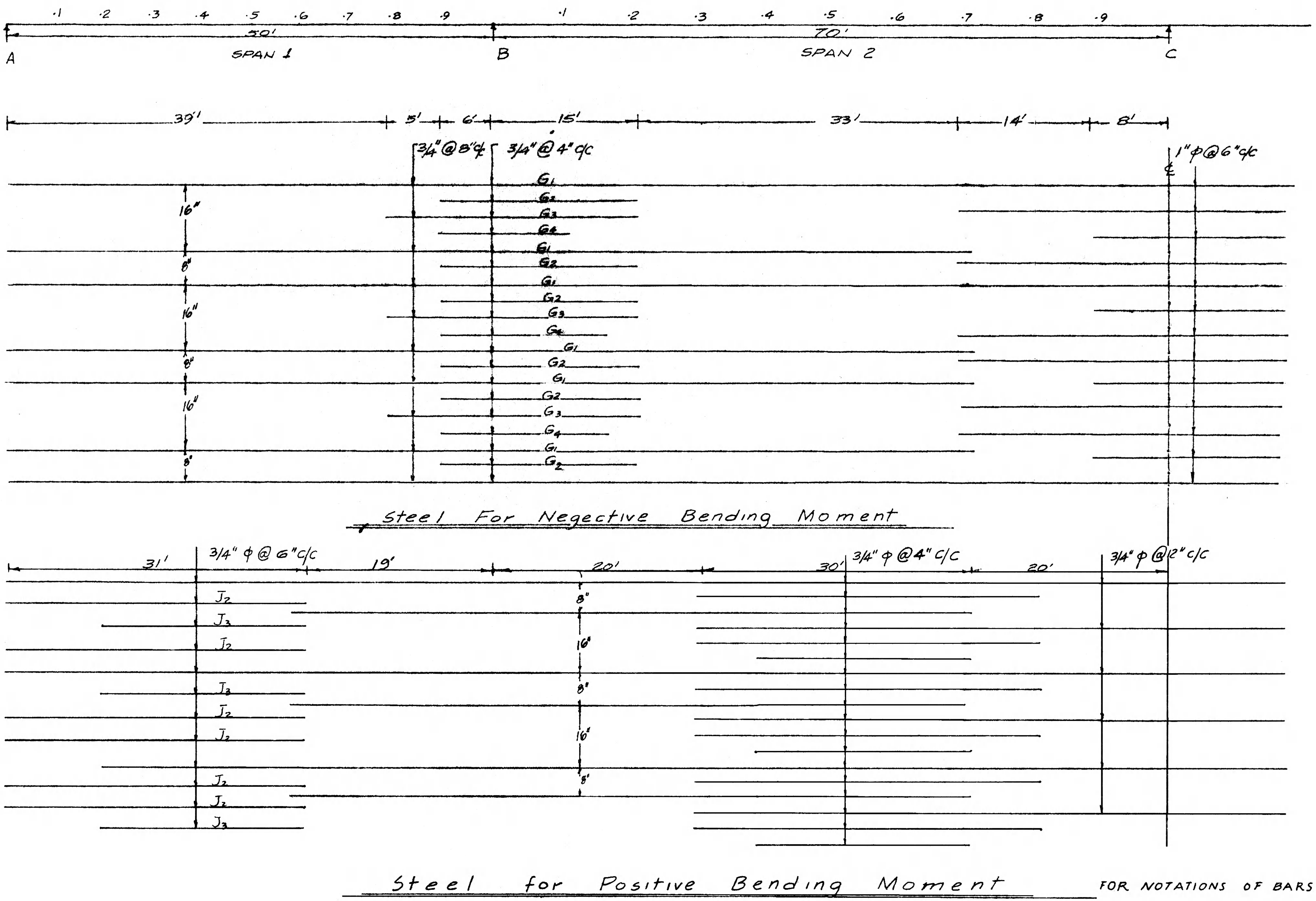


Reinforcement Of Slab



Cross Section Showing The Spacing Of The Web Reinforcement

FIG 20



FOR NOTATIONS OF BARS SEE TABLE I PAGE 42

FIG 21

DESIGN OF A BOX GIRDER BRIDGE

by

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AN ABSTRACT OF A MASTER'S REPORT

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1965

ABSTRACT

A box girder bridge was designed by Ultimate Strength Design procedures. The width of the bridge roadway was 24 feet, and the total length of the bridge was 240 feet. It was divided into 4 spans, the end spans being 50 feet each and the middle spans 70 feet each. The required depth of the box girder was found to be 4' 0". The clear distance for underpasses was 20' 0". The bridge was symmetrical about the center line. The bridge was designed by The American Association of State Highway Officials specifications for an H20 - S16 - 44 loading.

The influence coefficients were calculated for shear and moment at the 1/10 points in all spans, the maximum shear and maximum positive and negative moments were found for each section, and the required section properties were calculated. The various sections were checked for bond and deflection. Detail drawings were prepared showing: the section properties at critical sections; the reinforcement for the slab; reinforcement for beams for positive and negative Bending Moment and for columns.