DESIGN OF A THREE-SPAN
PLATE-GIRDER
BRIDGE

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PREFACE

The analysis and design of a three-span plate-girder bridge is presented in this report. A list of references, which includes only those publications that had a direct bearing on this report, is included. In addition to this list of references; class notes from lectures, in CIVEN 4D4 - Steel Structures, presented by Professor James W. Gillespie were referred to extensively.

The writer wishes to express his gratitude to the following persons:

To Professor Jan J. Tuma for his guidance through-out the writer's graduate works.

To Doctor James W. Gillespie, for his valuable instruction during his period of graduate study and for guiding the writer in this work.

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PART I

INTRODUCTION

1-1 General

The design of a continuous bridge structure consists of assuming a section and then investigating it for maximum stresses. Since the investigation is dependent upon the relative section variation, the final solution is not direct, but consists of a series of trials until the best design is obtained. Thus, it is desirable to begin with a reasonable section to reduce the number of required trials. The selection of the preliminary section is outlined in Part II.

1-2 Procedure

The design of a continuous plate-girder bridge consists of the following steps:

1. Assume the relative variation of cross-section.
2. Evaluate section properties.
3. Evaluate all necessary angular functions.
4. Evaluate the influence line ordinates for redundant bending moments.
5. Evaluate the influence line ordinates for intermediate bending moments and end shears and reactions.
6. Compute maximum live load plus impact bending moments.

7. Estimate dead load and obtain dead load bending moments.
8. Combine the live load plus impact and dead load bending moments, and check critical bending stresses.
9. Compute maximum shearing forces and check shearing stresses.
10. Design end and intermediate stiffeners.
11. Design end and intermediate diaphragms.
12. Design fixed and expansion bearings.
13. Design welds and/or rivets.
14. Design the roadway slab and curbs.
15. Design guard rail.
16. Compute dead load deflection diagram.
17. Prepare detailed engineering drawing.

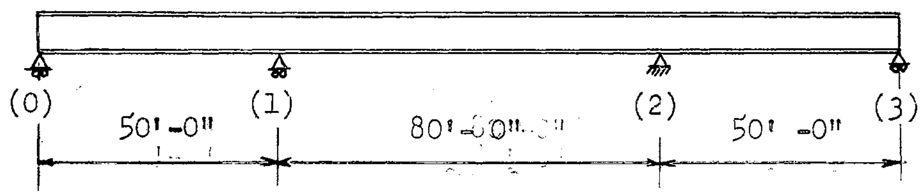
After steps 8,9, it may be necessary to revise the cross-sectional variation if there are sections that are over-stressed or sections that are considerably under-stressed.

1-3 Design Information

This three-span continuous bridge structure (Fig.1) is designed in accordance with the following specifications:

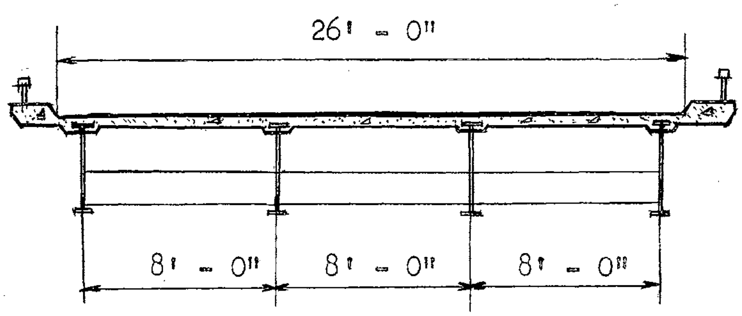
1. Standard Specifications for Highway Bridges, AASHO(1).
2. Standard Specifications for Welded Highway and Railway Bridges, AWS (2).

The roadway width is 26'-0" with safety curbs (Fig.1). The structure is designed for an H15-S12-44 loading.(1).



(a)

Elevation



(b)

Section

Fig. 1

Three-Span Bridge Structure

PART II

PRELIMINARY GIRDER SECTION

2-1 General

For economic design, the maximum stress should approach the allowable stress at as many sections as possible. With this criterion in mind, the following approach is used.

The depth of the girder is held approximately constant by using a constant depth web plate. Neglecting normal force, the general stress formula

$$f_x = \frac{F_x}{A_x} \mp \frac{M_x c}{I_x}$$

is simplified to

$$f_x = \frac{M_x c}{I_x} \;;$$

thus,

$$\frac{f_x}{c} = \frac{M_x}{I_x} = \text{Constant},$$

if the cross-sectional moment of inertia varies as the bending moment.

This type of variation is not possible if a continuous structure is desired, due to the requirement of hinges at points of zero moment.

2-2 Critical Sections

To achieve a continuous design, the ratio of moment of inertia to bending moment at critical sections is considered. The approximate locations of these critical sections are indicated as A, B, C (Fig.2).

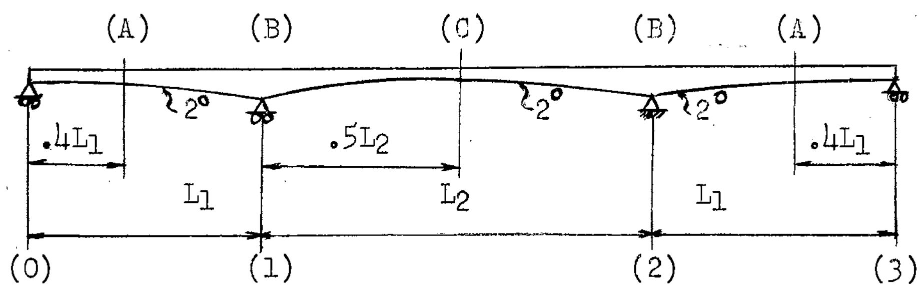


Fig. 2

Three Span Plate-Girder

To further simplify the preliminary calculations, the use of a table of beam constants is desired. In order to incorporate the use of tables, it is assumed that the variation in the moment of inertia of the girder is the same as that of a rectangular section with 2° parabolic haunches (Fig.2).

2-3 Influence Lines

The influence lines for the bending moments at Sections A, B, C are obtained (Fig.3) by assuming the relative variation in cross-section to be defined by

$$I_A = .965\, I_C, \quad I_B = 2.200\, I_C, \quad I_C = I_C .$$

From the influence lines, the maximum moments obtained are

$$M_A = .697\, M_C, \quad M_B = -1.27\, M_C, \quad M_C = M_C.$$

Further refinement of the preliminary section could be obtained by correcting the assumed cross-section variation to

$$I_A = .697\, I_C, \quad I_B = 1.27\, I_B, \quad I_C = I_C,$$

and the influence lines and bending moments recalculated. Since the general procedure is unchanged, the values obtained in the first trial are used for the final analysis.

2-4 Preliminary Section

The preliminary plate-girder section is shown (Fig.4) and the section properties are tabulated (Table 1).

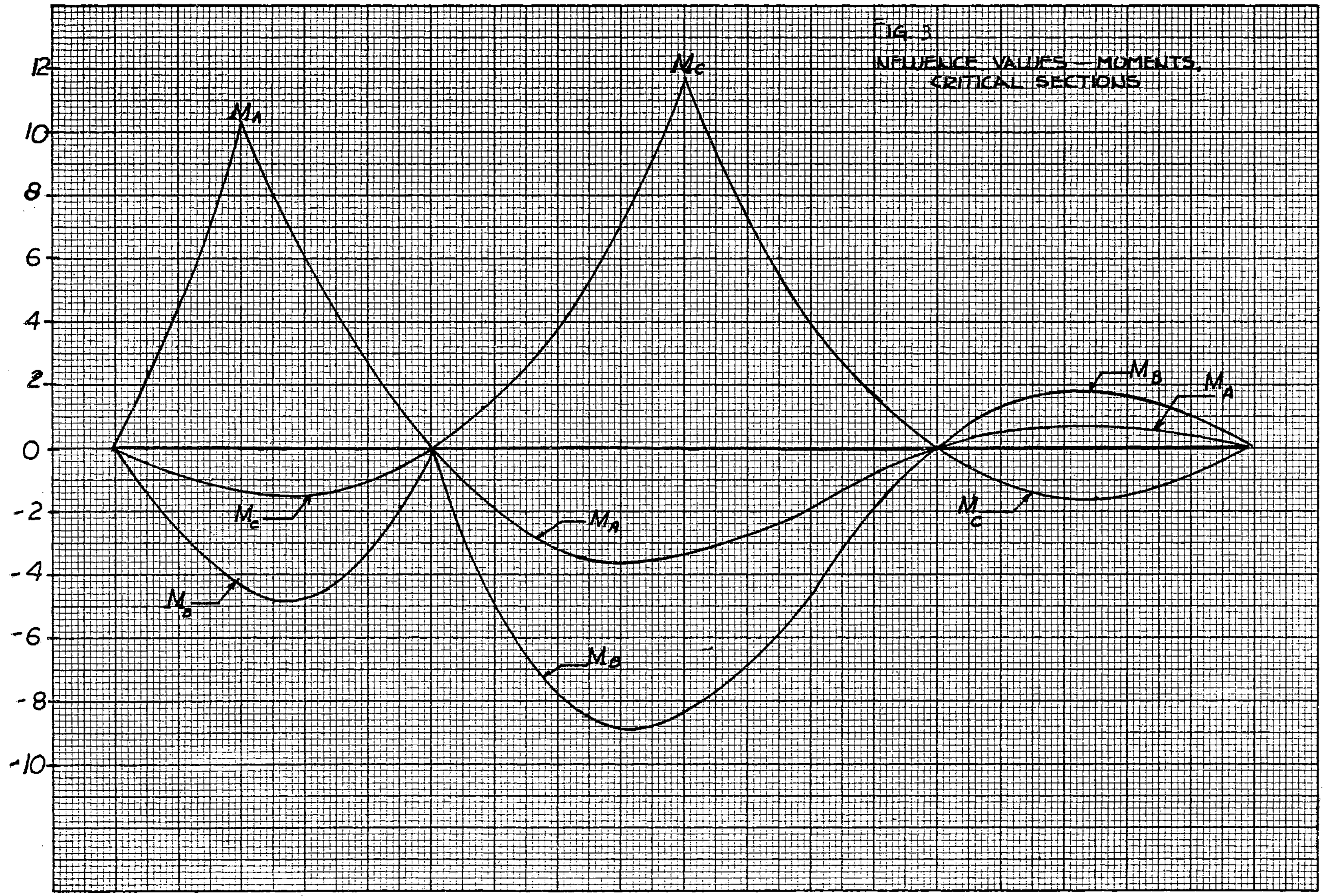


FIG. 3
INFLUENCE VALUES — MOMENTS, CRITICAL SECTIONS

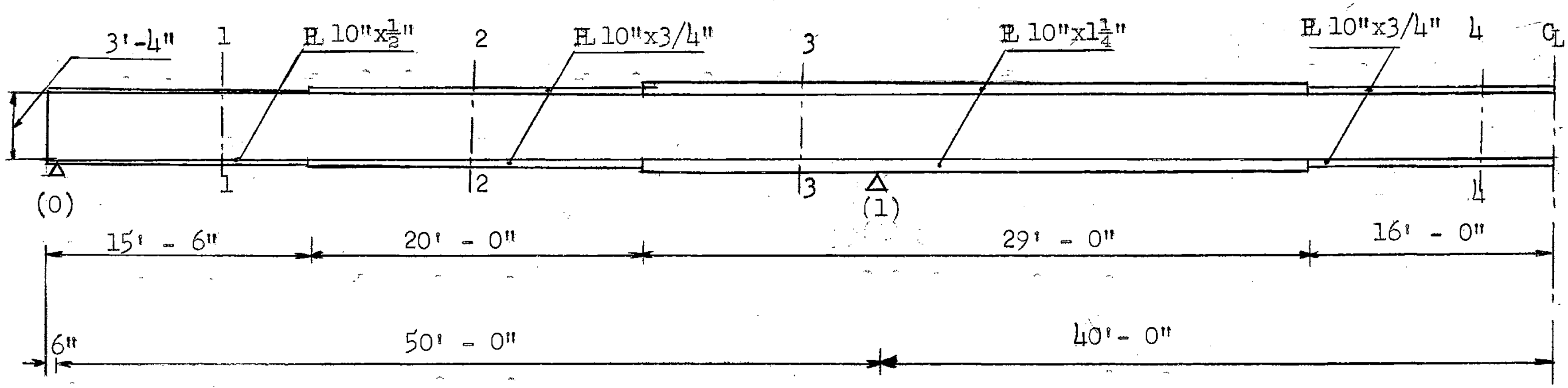


Fig. 4

Preliminary Plate-Girder Section

TABLE 1 SECTION PROPERTIES

Sect.	I(in.^{4})	$Z = \frac{I}{c}$(in.^{3})	w (# /ft.)
1-1	6100	297	85
2-2	8230	396	102
3-3	12600	593	136
4-4	8230	396	102

PART III

FINAL ANALYSIS AND DESIGN

3-1 Influence Lines

The influence line ordinates for the redundant bending moments are computed by the carry-over moment procedure (3, 4). The required angular functions are computed by the conjugate beam method based on a small strips approximation. Each span is resolved into ten equal length segments, and an equivalent elastic load is applied at the center of each strip (Tables 2, 3).

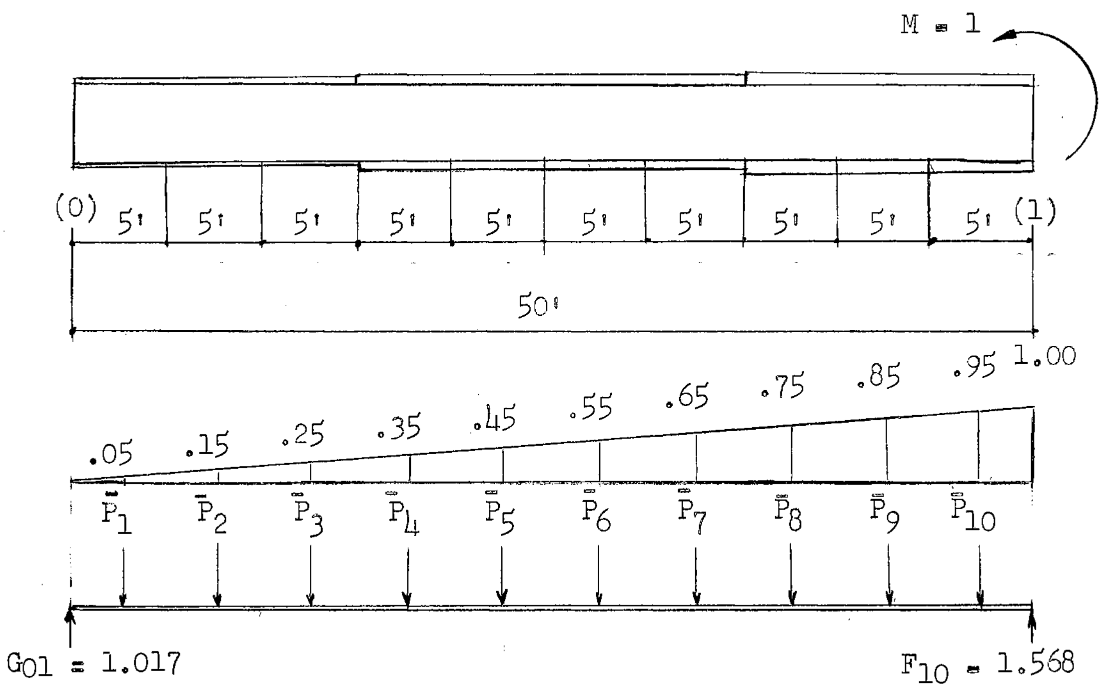
The influence line for the end slope of a simple beam is equal to the deflection curve of the beam due to a unit moment applied at that end. (Maxwell's Theorem). The ordinates of the corresponding deflection curves are tabulated (Tables 2, 3).

From Tables 2, 3, the carry-over factors are

$$r_{12} = r_{21} = -\frac{1.379}{4.020} = -.343.$$

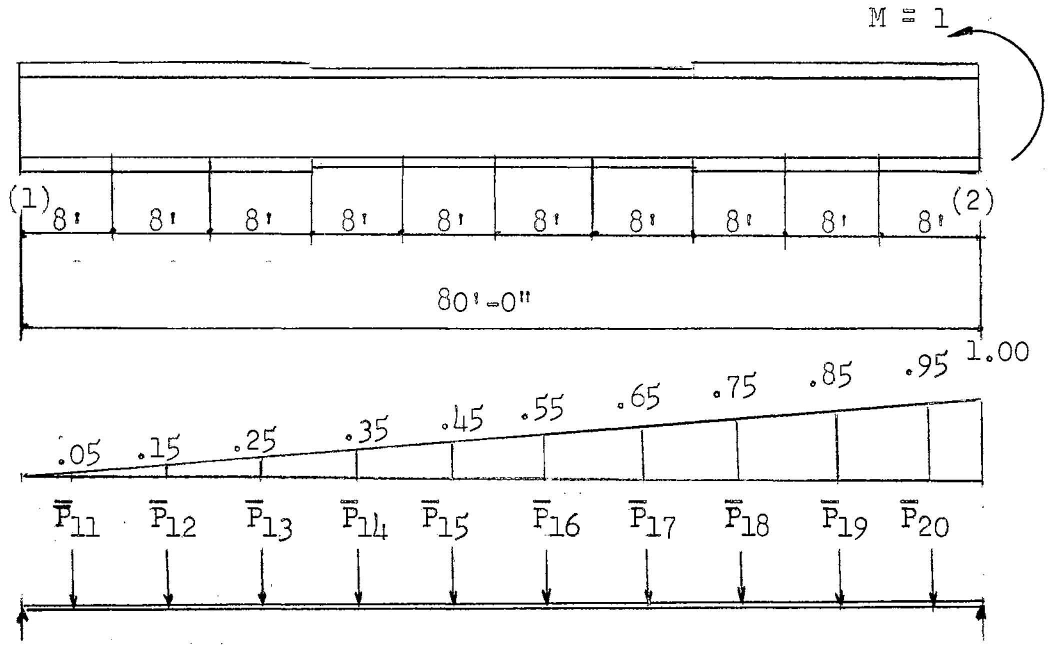
The influence of an initial starting moment at joint (1) is obtained by preforming a carry-over procedure for a unit starting moment at joint (1) (Table 4).

TABLE 2 ANGULAR FUNCTIONS SPAN $\overline{01}$



m	Δs	$\frac{1}{I'}$	$\frac{\Delta s}{I'}$	$\frac{x}{L}$	$\bar{\bar{P}}_x = \frac{\Delta s}{I'} \cdot \frac{x}{L}$	$\bar{\bar{P}}_x \frac{x}{L}$	$\bar{\bar{V}}_x$	(LL) $\bar{\bar{M}}_x = \tau_{10}$
0				.00				0.00
							1.017	
1	5	.164	.820	.05	.041	.002		2.54
							.976	
2	5	.164	.820	.15	.123	.018		7.42
							.853	
3	5	.164	.820	.25	.205	.051		11.69
							.648	
4	5	.121	.605	.35	.212	.074		14.93
							.436	
5	5	.121	.605	.45	.272	.122		17.11
							.164	
6	5	.121	.605	.55	.333	.183		17.93
							-.169	
7	5	.121	.605	.65	.393	.255		17.09
							-.562	
8	5	.079	.395	.75	.296	.222		14.29
							-.858	
9	5	.079	.395	.85	.335	.285		9.99
							-1.193	
10	5	.079	.395	.95	.375	.356		4.01
							1.568	
					2.585	1.568		

TABLE 3 ANGULAR FUNCTIONS SPAN $\overline{12}$



$G_{12} = 1.379.$ $F_{21} = 2.452$

m	Δs	$\frac{1}{I'}$	$\frac{\Delta s}{I'}$	$\frac{x}{L}$	$\overline{P}_x = \frac{\Delta s}{I'} \cdot \frac{x}{L}$	$\overline{P}_x \frac{x}{L}$	$\overline{V}_x$	$\overline{M}_x = {}^{(LL)}_{21}$
10				.00			1.379	0.00
11	8	.079	.632	.05	.032	.002	1.347	5.52
12	8	.079	.632	.15	.095	.014	1.252	16.30
13	8	.079	.632	.25	.158	.040	1.094	26.32
14	8	.121	.968	.35	.339	.119	.755	35.07
15	8	.121	.968	.45	.435	.195	.320	41.10
16	8	.121	.968	.55	.532	.292	-.212	43.56
17	8	.121	.968	.65	.630	.409	-.842	41.87
18	8	.079	.632	.75	.474	.356	-1.316	35.11
19	8	.079	.632	.85	.536	.455	1.852	24.59
20	8	.079	.632	.95	.600	.570	2.452	9.79
					3.831	2.452		

TABLE 4
CARRY-OVER PROCEDURE

	(1)	(2)
r's	- .343	- .343
m's	+ 1.000	
		- .343
	+ .118	
		- .043
	+ .014	
		- .005
	+ .002	
M's	+ 1.134	- .391

From Table 4, the following influence line equations for the redundant moments at supports (1), (2) are written

$$\begin{aligned} M_1 &= +1.134\, m_1 - .391\, m_2 \\ M_2 &= -.391\, m_1 + 1.134\, m_2 \end{aligned} \qquad (1)$$

or

$$\begin{aligned} M_1 &= -.283\,\Sigma\,\tau_1 + .097\,\Sigma\,\tau_2 \\ M_2 &= +.097\,\Sigma\,\tau_1 - .283\,\Sigma\,\tau_2 \quad , \end{aligned} \qquad (1a)$$

where m_1 = starting moment at joint (1)

m_2 = starting moment at joint (2)

$\Sigma\tau_1$ = sum of end slopes at joint (1)

$\Sigma\tau_2$ = sum of end slopes at joint (2).

Substituting values of the end slopes from Tables 2, 3 into Eq's. (1a), the influence line ordinates for the redundant bending moment at support (1) are tabulated (Table 5). From statics, the influence line ordinates for the end shears and reactions are evaluated (Table 5).

The influence line ordinates for intermediate bending moments are evaluated from statics, and the results tabulated (Table 6). The influence lines for end shears and reactions and intermediate shears are plotted from the values tabulated for end shears and reactions in Table 5 (Fig. 5). The influence lines for bending moments are obtained from the values tabulated in Table 6 (Fig's. 6, 7).

3-2 End Shears and Reactions

Areas under the influence lines for end shears and reactions are evaluated (Table 7), and the dead load reactions are computed assuming the dead load uniformly distributed with intensity

$$w_{DL} = .72^{k}/ft.$$

The end shears and reactions due to live load for an assumed truck loading and for a lane loading are evaluated in Table 8. The influence of impact and the total maximum end shears and reactions including dead load are tabulated (Table 8).

3-3 Bending Moments

The dead load bending moments are obtained by assuming the dead load as equivalent concentrated loads acting at the center of each strip. These moments are also computed as an independent check, assuming the dead load uniformly distributed of intensity .72 ^{k}/ft. These values are tabulated in Tables 9, 10.

The maximum live load bending moments, for a truck loading and a lane loading, are tabulated (Table 11). The influence of impact and the addition of dead load is included in Table 12.

3-4 Stresses

The final maximum live load plus dead load stresses are tabulated in Table 12. It is noted that the girder is slightly over-stressed (approximately 3.6%) at the center of span $\overline{12}$.

A more satisfactory variation of maximum stresses could be obtained by assuming a new section, based on these results, and repeating the foregoing procedure. Since the main purpose of this report is to outline the general approach, this section is accepted as satisfactory.

TABLE 5 INFLUENCE VALUES

REDUNDANT MOMENTS, END SHEARS AND REACTIONS

Sta.	τ_{10}	τ_{12}	$-.283\Sigma\tau_1$	$.097\Sigma\tau_2$	M_1	$V_{01} = R_0$	V_{10}	V_{12}	R_1
(0)	0.00	0.00	0.00	0.00	0.00	1.000	.000	.000	.000
1	2.54		- .72		- .72	.935	.065	.012	.077
2	7.42		- 2.10		-2.10	.808	.192	.035	.227
3	11.69		- 3.31		-3.31	.684	.316	.055	.371
4	14.93		- 4.23		-4.23	.565	.435	.071	.506
5	17.11		- 4.85		-4.85	.453	.547	.081	.628
6	17.93		- 5.08		-5.08	.348	.652	.085	.737
7	17.09		- 4.84		-4.84	.253	.747	.081	.828
8	14.29		- 4.05		-4.05	.170	.830	.068	.898
9	9.99		- 2.83		-2.83	.093	.907	.047	.954
10	4.01		- 1.13		-1.13	.027	.973	.019	.992
(1)	0.00	0.00	0.00	0.00	0.00	0.000	1.000	.000	1.000
11		9.79	- 2.77	.53	-2.24	- .045	.045	.972	1.017
12		24.59	- 6.97	1.58	-5.39	- .108	.108	.891	.999
13		35.11	- 9.95	2.55	-7.40	- .148	.148	.792	.940
14		41.87	-11.86	2.92	-8.94	- .179	.179	.688	.867
15		43.56	-12.31	3.99	-8.32	- .166	.166	.561	.727
16		41.10	-11.61	4.22	-7.39	- .148	.148	.439	.587
17		35.07	- 9.93	4.06	-5.87	- .117	.117	.312	.429
18		26.32	- 7.45	3.40	-4.05	- .081	.081	.208	.289
19		16.30	- 4.62	2.49	-2.13	- .043	.043	.109	.152
20		5.52	- 1.56	.95	- .61	- .012	.012	.028	.040
(2)		0.00	0.00	0.00	0.00	.000	.000	.000	.000
21				.39	.39	.008	- .008	-.019	- .027
22				.97	.97	.019	- .019	-.047	- .066
23				1.38	1.38	.028	- .028	-.068	- .096
24				1.66	1.66	.033	- .033	-.081	- .114
25				1.74	1.74	.035	- .035	-.085	- .120
26				1.66	1.66	.033	- .033	-.081	- .114
27				1.45	1.45	.029	- .029	-.071	- .100
28				1.13	1.13	.023	- .023	-.055	- .078
29				.72	.72	.014	- .014	-.035	- .049
30				.25	.25	.005	- .005	-.012	- .017
(3)	0.00	0.00	0.00	0.00	0.00	.000	.000	.000	- .000
					M_2				

TABLE 6 INFLUENCE VALUES-MOMENTS SPANS $\overline{01}$ AND $\overline{12}$

Sta.	$M_{(1)}$	$M_{(2)}$	BM_1	M_1	BM_2	M_2	BM_3	M_3	BM_4	M_4	BM_5	M_5	M_6	M_7	M_8	M_9	M_{10}	BM_{11}	M_{11}	BM_{12}	M_{12}	BM_{13}	M_{13}	BM_{14}	M_{14}	BM_{15}	M_{15}	BM_c	M_c
(0)	.00	.00	.00	.000	.00	.000	.00	.000	.00	.000	.00	.000	.000	.000	.000	.000	.000	.00	.000	.00	.000	.00	.000	.00	.000	.00	.000	.0	.000
1	- .72	.25	2.37	2.334	2.12	2.012	1.87	1.690	1.62	1.368	1.37	1.046	.724	.403	.080	- .242	- .560		- .672		- .575		- .443		- .331		- .284		- .240
2	- 2.10	.72	2.12	2.015	6.37	6.055	5.62	5.095	4.87	4.135	4.12	3.185	2.230	1.255	.300	- .665	- 1.625		- 1.964		- 1.677		- 1.333		- 1.026		- .831		- .690
3	- 3.31	1.13	1.87	1.705	5.62	5.120	9.37	8.844	8.12	6.962	6.87	5.380	3.810	2.220	.640	- .940	- 2.515		- 3.084		- 2.635		- 2.135		- 1.668		- 1.312		-1.090
4	- 4.23	1.45	1.62	1.408	4.87	4.235	8.12	7.060	11.37	9.890	9.62	7.720	5.550	3.370	1.200	- .980	- 3.140		- 3.938		- 3.383		- 2.755		- 2.170		- 1.680		-1.390
5	- 4.85	1.66	1.37	1.130	4.12	3.390	6.87	5.660	9.62	7.920	12.37	10.140	7.450	4.720	1.980	- .750	- 3.480		- 4.517		- 3.871		- 3.205		- 2.541		- 1.925		-1.600
6	- 5.08	1.74	1.12	.870	3.37	2.610	5.62	4.350	7.87	6.090	10.12	7.830	9.570	6.310	3.050	- .200	- 3.460		- 4.743		- 4.060		- 3.395		- 2.720		- 2.118		-1.670
7	- 4.64	1.66	.87	.630	2.62	1.890	4.37	3.160	6.12	4.420	7.88	5.700	6.950	8.220	4.480	.750	- 2.980		- 4.517		- 3.903		- 3.278		- 2.643		- 1.925		-1.590
8	- 4.05	1.38	.62	.423	1.87	1.262	3.12	2.108	4.37	2.955	5.63	3.810	4.640	5.490	6.330	2.180	- 1.980		- 3.771		- 3.213		- 2.758		- 2.234		- 1.610		-1.340
9	- 2.83	.97	.37	.234	1.12	.695	1.87	1.164	2.62	1.630	3.38	2.110	2.570	3.030	3.500	3.970	- .570		- 2.642		- 2.255		- 1.940		- 1.588		- 1.120		- .930
10	- 1.13	.39	.12	.064	.37	.200	.62	.340	.87	.474	1.12	.612	.748	.886	1.022	1.160	1.300		- 1.051		- .902		- .788		- .643		- .445		- .320
(1)	.00	.00	.00	.000	.00	.000	.00	.000	.00	.000	.00	.000	.000	.000	.000	.000	.000	.00	.000	.00	.000	.00	.000	.00	.000	.00	.000	.0	.000
11	- 2.24	- .61		- .112		- .336		- .560		- .785		- 1.010	- 1.230	- 1.450	- 1.680	- 1.905	- 2.120	3.80	1.649	3.40	1.404	3.00	1.168	2.60	.936	2.20	.696	2.0	.580
12	- 5.39	- 2.13		- .269		- .808		- 1.350		- 1.890		- 2.430	- 2.970	- 3.510	- 4.040	- 4.580	- 5.110	3.40	- 1.816	10.20	5.300	9.00	4.428	7.80	3.544	6.60	2.672	6.0	2.240
13	- 7.40	- 4.05		- .370		- 1.111		- 1.852		- 2.583		- 3.324	- 4.065	- 4.816	- 5.537	- 6.288	- 7.020	3.00	- 4.222	9.00	2.104	15.00	8.451	13.00	6.769	11.00	5.115	10.0	4.280
14	- 8.94	- 5.87		- .447		- 1.340		- 2.240		- 3.150		- 4.020	- 4.920	- 5.810	- 6.710	- 7.600	- 8.500	2.60	- 6.194	7.80	- .680	13.00	4.820	18.20	10.330	15.40	7.840	14.0	6.600
15	- 8.32	- 7.39		- .416		- 1.250		- 2.080		- 2.910		- 3.740	- 4.570	- 5.400	- 6.220	- 7.070	- 7.900	2.20	- 6.070	6.60	- 1.580	11.00	2.930	15.40	7.420	19.80	11.910	18.0	10.140
16	- 7.39	- 8.32		- .370		- 1.110		- 1.850		- 2.580		- 3.320	- 4.060	- 4.800	- 5.530	- 6.280	- 7.010	1.80	- 5.626	5.40	- 2.130	9.00	1.370	12.60	4.890	16.20	8.400	20.0	12.140
17	- 5.87	- 8.94		- .294		- .880		- 1.470		- 2.060		- 2.640	- 3.230	- 3.820	- 4.400	- 5.000	- 5.570	1.40	- 4.617	4.20	- 2.140	7.00	.360	9.80	2.830	12.60	5.350		
18	- 4.05	- 7.40		- .202		- .608		- 1.012		- 1.415		- 1.820	- 2.230	- 2.630	- 3.040	- 3.440	- 3.850	1.00	- 3.220	3.00	- 1.550	5.00	.108	7.00	1.787	9.00	3.440		
19	- 2.13	- 5.39		- .106		- .320		- .532		- .746		- .958	- 1.171	- 1.385	- 1.600	- 1.810	- 2.020	.60	- 1.689	1.80	- .808	3.00	.050	4.20	.925	5.40	1.800		
20	- .61	- 2.24		- .031		- .091		- .152		- .214		- .274	- .336	- .396	- .457	- .518	- .580	.20	- .492	.60	- .254	1.00	- .017	1.40	.219	1.80	.454		
(2)	.00	.00		.000		.000		.000		.000	.00	.000	.000	.000	.000	.000	.000	.00	.000	.00	.000	.00	.000	.00	.000	.00	.000		
21	.39	- 1.13		.019		.058		.097		.137		.175	.214	.254	.293	.332	.370		.314		.162		.010		- .141		- .294		
22	.97	- 2.83		.048		.145		.242		.339		.436	.533	.630	.727	.824	.920		.780		.400		.020		- .330		- .737		
23	1.38	- 4.05		.079		.207		.345		.482		.621	.760	.897	1.032	1.170	1.310		1.108		.563		.031		- .523		-1.060		
24	1.66	- 4.84		.083		.249		.415		.581		.747	.912	1.080	1.245	1.410	1.575		1.333		.684		.035		- .610		-1.268		
25	1.74	- 5.08		.087		.261		.435		.609		.782	.956	1.130	1.305	1.480	1.650		1.396		.718		.035		- .650		-1.334		
26	1.66	- 4.85		.083		.249		.415		.581		.747	.912	1.080	1.245	1.410	1.575		1.333		.684		.035		- .610		-1.268		
27	1.45	- 4.23		.072		.217		.362		.507		.562	.798	.942	1.087	1.230	1.375		1.163		.595		.027		- .548		-1.102		
28	1.13	- 3.31		.056		.170		.282		.396		.508	.622	.734	.848	.960	1.070		.905		.463		.022		- .426		- .868		
29	.72	- 2.10		.036		.108		.180		.252		.324	.396	.467	.540	.612	.683		.578		.297		.015		- .268		- .549		
30	.25	- .72		.012		.037		.062		.087		.112	.137	.162	.187	.212	.237		.201		.104		.007		- .090		- .187		
(3)	.00	.00	.00	.000	.00	.000	.00	.000	.00	.000	.00	.000	.000	.000	.000	.000	.000	.00	.000	.00	.000	.00	.000	.00	.000	.00	.000		
			BM_{10}		BM_9		BM_8		BM_7		BM_6																		

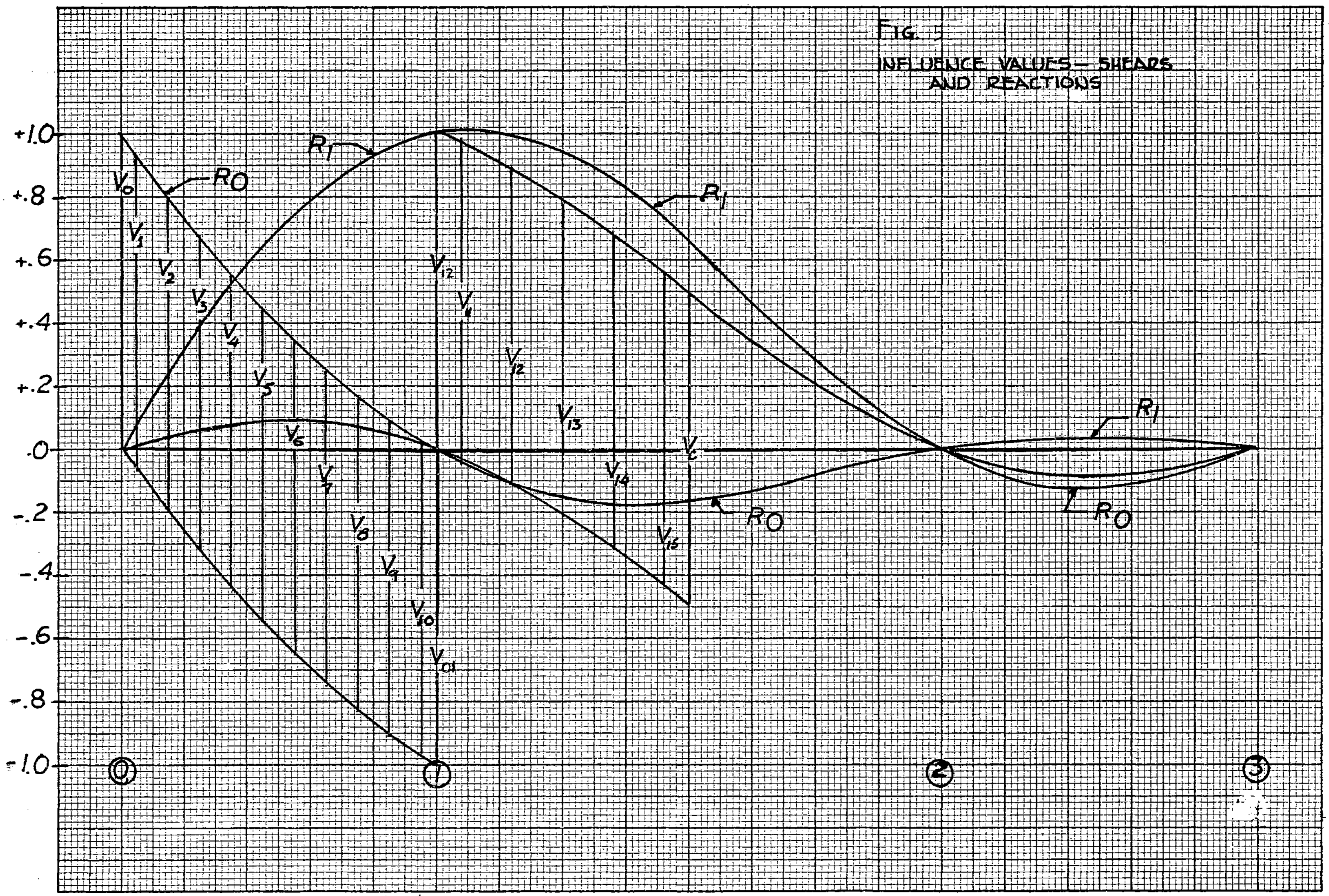


FIG. 5
INFLUENCE VALUES — SHEARS AND REACTIONS

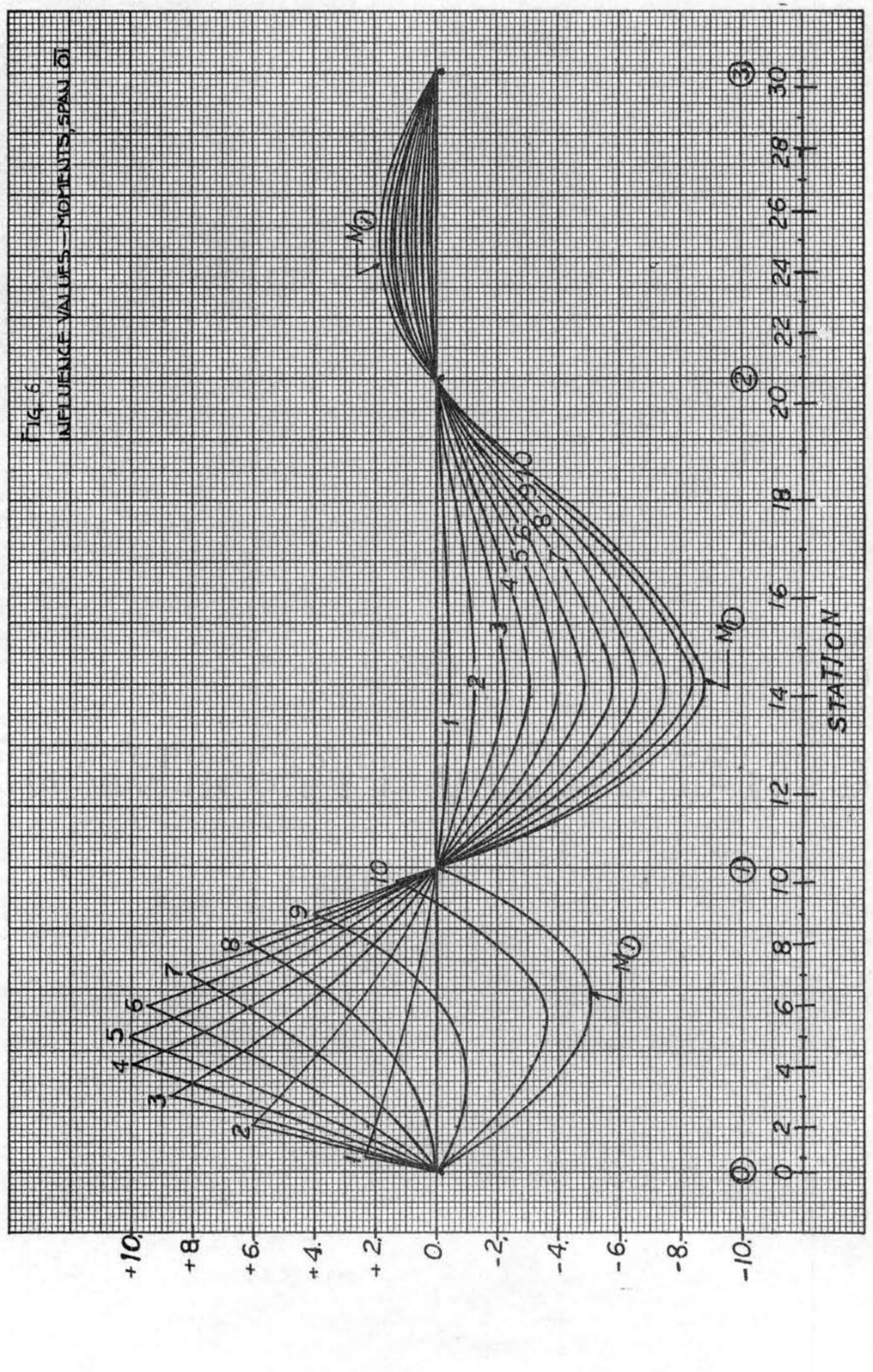


Fig. 6
INFLUENCE VALUES—MOMENTS, SPAN 01
STATION
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30
0 1 2 3
+10 +8 +6 +4 +2 0 -2 -4 -6 -8 -10
1 2 3 4 5 6 7 8 9 10
M1
M2
M3

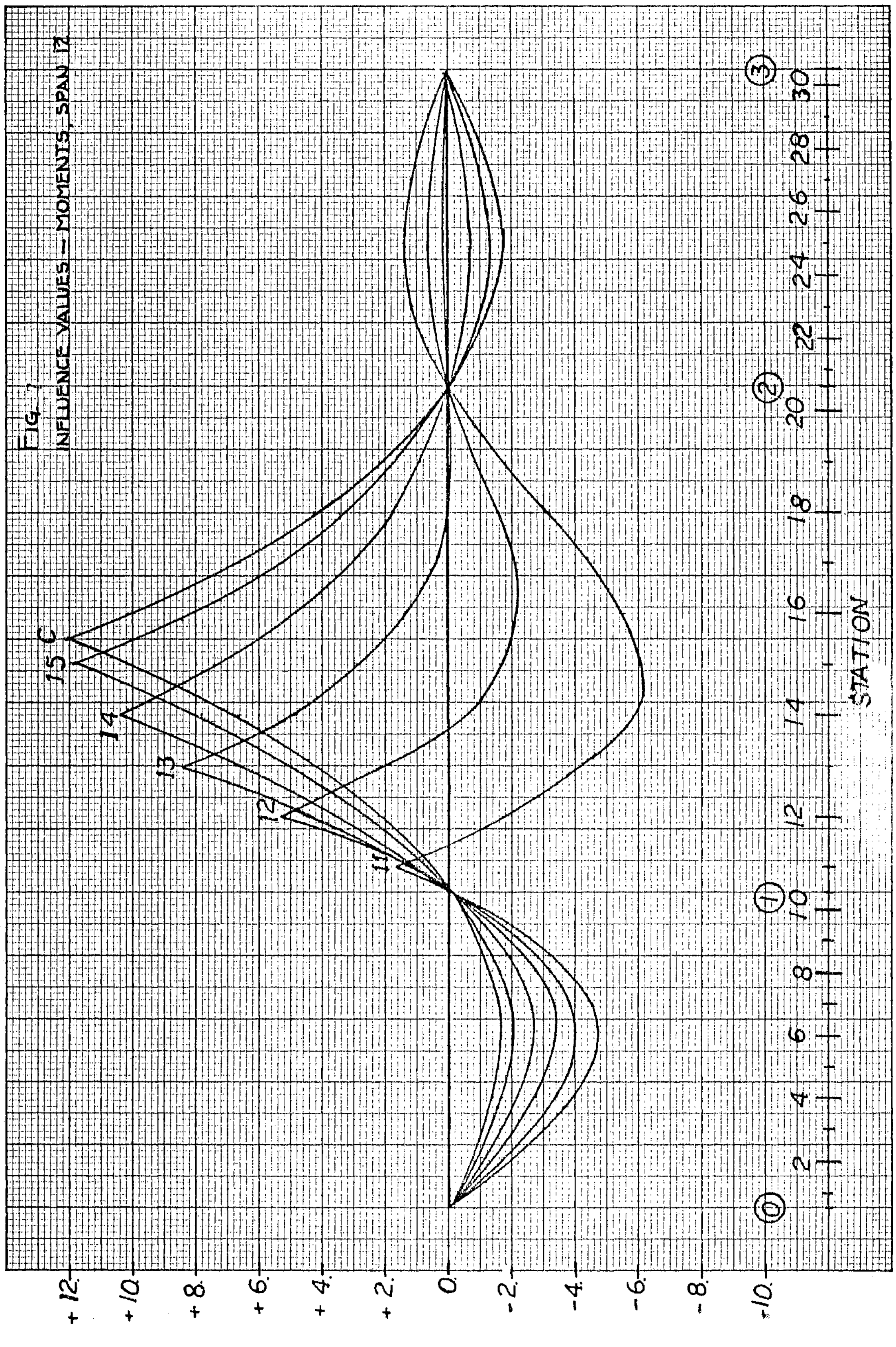


FIG. 7
INFLUENCE VALUES — MOMENTS, SPAN 12

TABLE 7 INFLUENCE LINE AREAS END SHEARS AND REACTIONS				
	$R_0 = V_{01}$	V_{10}	V_{12}	R_1
+	22.82	36.55	42.74	79.29
-	8.25	1.12	2.74	3.86
Σ	+14.57	+35.43	+40.00	+75.43
$w_{DL} = .72$	10.50	25.50	28.80	54.20

TABLE 8 MAXIMUM END SHEARS AND REACTIONS										
	TRUCK LOAD			Σ	LANE LOAD		Σ	I	(LL+I)	(T)
	24	24	6		.48	19.5			$.725V_{MAX.}$	V_{MAX}
R_0	1.000	.648	.340	41.6	22.82	1.000	30.4	.286	38.8	49.3
V_{10}	1.000	.805	.536	46.5	36.55	1.000	37.0	.286	43.4	68.9
V_{12}	1.000	.916	.688	50.1	42.74	1.000	40.0	.245	45.2	74.0
R_1	1.017	.909	.881	51.5	79.29	1.017	57.9	.263	53.0	107.2
R_0*	-.179	-.153	-.118	- 8.7	-8.25	-.179	- 7.5	.245	- 7.9	2.6

*Minimum Exterior Reaction (Indicates That There Is No Uplift)

TABLE 9			BENDING MOMENTS				DEAD LOAD*
Sect.	$M_{(1)}$	P_x	$M_{(1)}^{(P_x)}$	$P_x(\frac{x'}{L})$	V_x	BM_x	M_x
(0)	.000	.000	.00	.00	17.390	0.0	0.0
1	- .720	3.425	- 2.46	3.25	13.965	43.5	24.8
2	-2.100	3.425	- 7.20	2.91	10.540	113.2	56.7
3	-3.310	3.425	-11.34	2.56	7.115	165.9	71.7
4	-4.230	3.510	-14.85	2.28	3.605	203.4	71.4
5	-4.850	3.510	-17.05	1.93	0.095	221.4	52.0
6	-5.080	3.510	-17.85	1.58	- 3.405	220.9	14.0
7	-4.840	3.510	-17.00	1.23	- 6.915	203.9	- 41.0
8	-4.050	3.680	-14.90	.92	-10.595	169.3	-113.0
9	-2.830	3.680	-10.40	.55	14.275	116.3	-203.7
10	-1.130	3.680	- 4.17	.18	17.955	44.9	-313.0
(1)	.000				28.870	0.0	-377.3
11	-2.240	5.880	-13.20		22.990	115.0	-262.3
12	-5.390	5.880	-31.80		17.110	299.0	- 78.3
13	-7.400	5.880	-43.70		11.230	436.0	58.7
14	-8.940	5.615	-50.20		5.615	525.5	148.2
15	-8.320	5.615	-46.70		0.000	570.5	193.2
16	-7.390	5.615	-41.50				
17	-5.870	5.615	-33.00				
18	-4.050	5.880	-23.90				
19	-2.130	5.880	-12.60				
20	- .610	5.880	- 3.60				
(2)	.000	.000	.00				
21	.390	3.680	1.43				
22	.970	3.680	3.57				
23	1.380	3.680	5.10				
24	1.660	3.510	5.83				
25	1.740	3.510	6.11				
26	1.660	3.510	5.83				
27	1.450	3.510	5.10				
28	1.130	3.425	3.87				
29	.720	3.425	2.46				
30	.250	3.425	.86				
(3)	.000	.000	.00				
			377.26	17.39			

*Assuming Concentrated At Various Stations

TABLE 10	BENDING MOMENTS		DEAD LOAD*	
Sect.	+	−	Σ	M
(0)	.0	.0	.0	.0
1	54.0	20.6	33.4	24.0
2	143.0	61.8	81.2	58.4
3	209.0	103.0	106.0	76.3
4	247.0	145.0	102.0	73.5
5	256.0	186.0	70.0	50.3
6	250.0	227.0	23.0	16.5
7	214.0	268.0	− 54.0	− 38.8
8	154.5	310.0	−155.5	−111.7
9	86.0	368.0	−282.0	−203.0
10	59.5	492.0	−432.5	−311.0
(1)	56.0	576.0	−520.0	−374.2
11	51.4	418.0	−366.6	−264.0
12	89.2	201.2	−112.0	− 80.6
13	188.0	109.0	79.0	56.8
14	314.0	107.0	207.0	149.0
15	380.0	108.0	272.0	196.0
C	387.0	107.2	279.8	201.2

*Assuming $w_{DL} \doteq .72\ {}^{k}/ft.$

TABLE 11 MAXIMUM BENDING MOMENTS LIVE LOAD

Sect.	TRUCK LOAD				LANE LOAD		
	24	24	6	$M^{(LL)}$	.48	13.5	$M^{(LL)}$
(0)	0.00	0.00	0.00	0.	0	0.00	0.
1	+ 2.33	+1.47	+ .72	+ 96.	+ 57	+ 2.33	+ 59.
2	+ 6.05	+3.56	+1.51	+240.	+143	+ 6.05	+151.
3	+ 8.84	+4.61	+1.57	+333.	+209	+ 8.84	+220.
4	+ 9.89	+5.09	+1.41	+368.	+247	+ 9.89	+252.
5	+10.14	+4.19	+3.62	+366.	+256	+10.14	+260.
6	+ 9.57	+4.16	+2.98	+348.	+250	+ 9.57	+249.
7	- 5.81	-4.95	-3.83	-281.	-268	- 5.81	-207.
8	- 6.71	-5.7	-4.42	-325.	-310	- 6.71	-239.
9	- 7.60	-6.48	-5.00	-368.	-368	- 7.60	-280.
10	- 8.50	-7.21	-5.59	-411.	-492	- 8.50	-351.
(1)	- 8.94	-7.62	-5.90	-433.	-576	- 8.94	-398.
11	- 6.20	-5.75	-2.42	-301.	-418	- 6.20	-285.
12	- 4.06	-2.78	-2.45	-179.	-201	- 4.06	-151.
13	+ 8.45	+3.42	+1.98	+297.	+188	+ 8.45	+204.
14	+10.33	+5.52	+4.35	+407.	+314	+10.33	+290.
15	+11.91	+6.11	+5.79	+467.	+380	+11.91	+343.
C	+12.14	+5.91	+5.91	+468.	+387	+12.14	+350.

TABLE 12 MAXIMUM STRESSES

Sect.	$M^{(LL)}_{Max}$	I	$.725M^{(LL+I)}_{Max}$	$M^{(DL)}$	$M^{(T)}_{Max}$	Z	Stress.
(0)	0.	-	-	0.0	0.0	-	-
1	+ 96.	.286	+ 89.	+ 24.8	+113.8	297	4.60
2	+240.	"	+224.	+ 56.7	+280.7	297	11.35
3	+333.	"	+310.	+ 71.7	+381.7	297	15.42
4	+368.	"	+343.	+ 71.4	+414.4	396	12.57
5	+366.	"	+341.	+ 52.0	+393.0	396	11.90
6	+348.	"	+324.	+ 14.0	+338.0	396	10.25
7	-281.	.263	-258.	- 41.0	-299.0	396	9.10
8	-325.	"	-298.	-113.0	-411.0	593	8.31
9	-368.	"	-337.	-203.7	-540.7	593	10.94
10	-411.	"	-377.	-313.0	-690.0	593	13.96
(1)	-433.	"	-397.	-377.3	-774.3	593	15.65
11	-301.	"	-276.	-262.3	-538.3	593	10.90
12	-179.	"	-164.	- 78.3	-242.3	593	4.87
13	+297.	.245	+268.	+ 58.7	+326.7	593	6.62
14	+407.	"	+368.	+148.2	+516.2	396	15.62
15	+467.	"	+421.	+193.2	+614.2	396	18.60
C	+468.	"	+422.	+193.2	+615.2	396	18.65

PART IV

SECONDARY DESIGN

4-1 End Stiffeners

The allowable stress for the end bearing stiffeners is (1)

$$f = 18 \text{ ksi}$$

Thus, at supports (0), (3), the required area is

$$A = \frac{49.3}{18} = 2.74 \text{ in.}^2$$

and the required area at supports (1), (2) is

$$A = \frac{107.2}{18} = 5.97 \text{ in.}^2$$

From these results, the following stiffener plates were selected:

supports (0), (3)

$$2 \text{ PL's } 5'' \times \frac{5''}{16} = 3.12 \text{ in.}^2$$

supports (1), (2)

$$2 \text{ PL's } 5'' \times \frac{3''}{4} = 7.50 \text{ in.}^2$$

4-2 Intermediate Stiffeners

The clear distance between intermediate stiffeners shall not be greater than (1)

(a) 6'

(b) clear unsupported depth of the web (3'-4")

(c) distance given by the formula

$$d = \frac{9000\ t}{\sqrt{s}}$$

The maximum average shearing stress in the web is

$$s = \frac{74000}{\frac{3}{8}(40)} = 4930 \text{ psi};$$

thus,

$$d = \frac{9000\ (\frac{3}{8})}{\sqrt{4930}} = 48.2'' \doteq 4'\text{-}0''.$$

From this, criterion (b) governs and a spacing

$$d = 3' - 4''$$

is used. The exact spacing is indicated on the engineering drawings, Part V.

The width of the stiffener plate (1)

(a) must not be more than 16 times its thickness

$$b = 16\ (\frac{5}{16}) = 5.0''$$

(b) must not be less than 2" plus $\frac{1}{30}$ times the depth of the girder

$$b = 2 + \frac{1}{30}(41) = 3.4" \doteq 4.0"$$

The thickness of the stiffener plate (1)

(a) must not be less than $\frac{1}{16}$ times its width

$$t = \frac{1}{16}(4.0) = \frac{1}{4}"$$

(b) must not be less than $\frac{5}{16}"$

$$t = \frac{5}{16}"\ .$$

From the preceding criteria, the following intermediate stiffener plate is selected:

$$PL\ 4" \times \frac{5}{16}"\ .$$

The stiffener plate must also meet the following minimum moment of inertia requirement

$$I = \frac{dt^3J}{11.0}\ ,$$

where

$$J = 3.75\left(\frac{40}{40}\right)^4 = 3.75.$$

thus,

$$I = \frac{40\left(\frac{3}{8}\right)^3(3.75)}{11.0} = .715\ \text{in.}^4$$

Since the intermediate stiffeners occur as single units, the moment of inertia furnished is

$$I = \frac{\frac{5}{16}(4)^3}{3} = 6.67 \text{ in.}^4 > .715.$$

Thus, the selected stiffener plate is satisfactory.

4-3 Diaphragms

The maximum spacing of the diaphragms should not exceed 25', and they must be designed to resist 300 #/f applied laterally to the bridge structure.(1). It is found that this requirement is minor in comparison to the stability requirements. Thus, the requirement

$$\frac{L}{r} \leqslant 200$$

is the controlling design factor.(3).

The girder spacing is 8'; thus, the length of the diaphragms is

$$L = 8' = 96''.$$

the required minimum radius of gyration is

$$r = \frac{96}{200} = .48''.$$

The following section is selected for the diaphragms:

15 ⊔ 40.

The allowable axial compressive load on this section is

$$P = (11.34)(11.70) = 133 \text{ k} > 25 (.3) = 7.5\text{k}.$$

4-4 Bearings

A fixed bearing is designed for support (2). The remaining bearings must be designed to allow for a temperature expansion of $1\frac{1}{4}$" in 100'; thus,

$$\Delta_0 = 1.30 \quad (1.25) = 1.625''$$

$$\Delta_1 = .80 \quad (1.25) = 1.000''$$

$$\Delta_2 = .50 \quad (1.25) = .625'',$$

where

$$\Delta = \text{allowance for expansion}$$

The girder is supported on 4" Ø half-rounds. The allowable bearing capacity of these half-rounds on steel 15 (3)

$$P = 128 \ ^{k}/\text{in.}$$

The required thicknesses of the supporting plates are supports (0), (3)

$$t = \frac{49.3}{128} = .385 \doteq \frac{1''}{2}$$

supports (1), (2)

$$t = \frac{107.2}{128} = .840 \doteq 1''.$$

Two plates of these thicknesses are used for each of the respective bearings (Part V).

The allowable bearing stresses for the rocker plates are computed from (1)

$$f_s = \frac{p - 13000}{20000} \, 600 \, d \, .$$

for supports (0), (3), use

$$d = 12'', \qquad f_s = 7200 \text{ psi}$$

and for supports (1),(2), use

$$d = 18'', \qquad f_s = 10800 \text{ psi.}$$

From these allowable stresses, the lengths of the rocker plates are:

supports (0),(3):

$$L = \frac{49.3}{7200} + 2(1) = 8.85'', \text{ use } 11''$$

supports (1), (2):

$$L = \frac{107.2}{10800} + 2(1) = 11.94, \text{ use } 15'',$$

where the diameter of the anchor bolts is assumed as

$$\phi = 1''.$$

The anchor bolt connection in the rocker plate must be tapered to permit rotation due to expansion (Fig. 8).

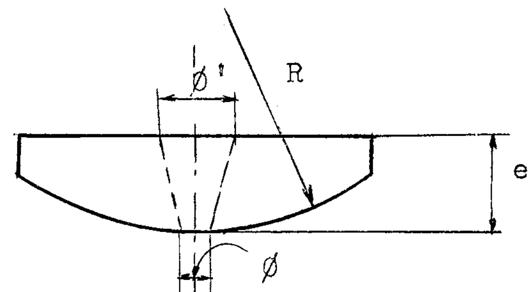


Fig. 8

Rocker Plate

From Fig. 8

$$\frac{e}{R} = \frac{\frac{\phi' - \phi}{2}}{\Delta} ,$$

where

$\Delta =$ Expansion Allowance.

thus,

$$\phi' = \phi + \frac{e\Delta}{2R} = \phi + \frac{e\Delta}{d} .$$

For support (0),

$$\phi'_o = 1" + \frac{1.5}{12} (1.625) = 1.20"$$

for support (1),

$$\phi'_1 = 1" + \frac{2}{18} (1.000) = 1.11"$$

for support (2),

$$\phi_2' = 1'' + \frac{2}{18}(0) = 1.00'' \text{ (Fixed)}$$

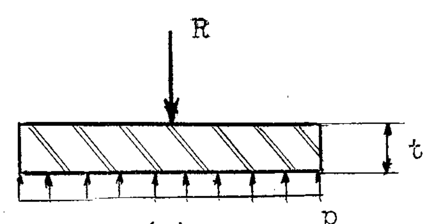
and for support (3)

$$\phi_3' = 1'' + \frac{1.5}{12}(.625) = 1.08''.$$

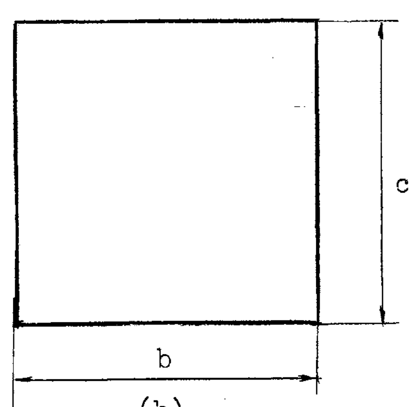
from this, a constant value for ϕ' is selected

$$\phi' = 2''.$$

The design of the bearing plate is controlled by bending and the allowable bearing stress on the piers and abutments. A plate loaded by a concentrated load and supported by a uniform pressure is considered (Fig. 9).



(a)
Loading



(b)
Bearing Area

Fig. 9
Bearing Plate

The design bending moment is

$$M = \frac{Rb}{8}.$$

The stress is

$$f_s = \frac{M}{Z} = \frac{6M}{ct^2}$$

or the required thickness is

$$t = \sqrt{\frac{6M}{cf_s}}.$$

For supports (0), (3),

$$M = \frac{(49.3)(6)}{8} = 37.0 \text{ k-in.}$$

$$t = \sqrt{\frac{6(37.0)}{11(18)}} = 1.06'', \text{ use } t = 1\tfrac{1}{2}'',$$

For supports (1), (2),

$$M = \frac{(107.2)(8.5)}{8} = 114.0 \text{ k-in.}$$

$$t = \sqrt{\frac{6(114.0)}{15(18)}} = 1.59'', \text{ use } t = 2''.$$

The allowable bearing stress on the concrete is (1)

$$f_c = 1000 \text{ psi.}$$

For supports (0), (3),

$$\frac{49.3}{(11)(6)} = .75 < 1.00, \text{ OK}$$

For supports (1) (2),

$$\frac{107.2}{(15)(8.5)} = .84 < 1.00, \text{ OK.}$$

4-5 Weld Design

The design of fillet welds connecting the flange plates to the web plate and the stiffeners to the web is determined by the strength and minimum size requirements. The minimum size welds for connecting plates of different thicknesses is specified by AWS.(2). This minimum size requirement is the governing factor for many cases.

The fillet welds connecting the flange plates to the web must develop the following shear per unit length

$$q = \frac{V\,Q}{I} .$$

The critical shear is

$$V = 77^k .$$

At this section,

$$Q = 258 \text{ in}^3.$$

and

$$I = 12600 \text{ in}^4.$$

From this,

$$q = \frac{(77)(258)}{12600} = 1.51^k/\text{in.}$$

For two fillet welds (both sides);
thus,

$$q = .76^k \text{ /in.per weld.}$$

The minimum size requirement controls (Part V).

For the bearing stiffeners,

at supports (0), (3),

$$q = \frac{49.3}{4(40)} = .31^{k}/\text{ in.}$$

at supports (1), (2),

$$q = \frac{107.2}{4(40)} = .67^{k}/\text{in.}$$

The minimum size requirement controls (Part V).

Since the intermediate stiffeners do not transfer calculated stress, use minimum size welds (Part V).

4-6 Slab Design

The clear span of the roadway slab is (1)

$$S = 96 - \frac{10}{2} = 91'' = 7.58' .$$

The design bending moment is (1)

$$M = \mp .2 \frac{P_1}{E} S$$

where

$$P_1 = 24^{k}$$

and

$$E = .4(7.58) + 3.75 = 6.79.$$

thus,

$$M = \pm .2 \frac{24}{6.79}(7.58) = \pm 5.37 \text{ k-ft.}$$

For 1" clear cover,

$$d = 6" - 1" - \frac{.75}{2} = 4.625",$$

assuming the slab thickness

$$t = 6"$$

and the maximum size reinforcing bar

$$\phi = \frac{3}{4}" \quad (\#6)\ .$$

For balanced design, the concrete resisting moment is

$$M_c \doteq 4.45 \text{ k-ft.}$$

for the allowable stresses

$$f_s = 18000 \text{ psi}$$

$$f_c = .4f_c{}' = 1200 \text{ psi.}$$

From this, there must be compressive steel to resist

$$M_s = 5.37 - 4.45 = .92 \text{ k-ft.}$$

The total tensile reinforcement required is

$$A_s = \frac{4.45}{(1.29)(4.625)} + \frac{.92(12)}{(18)(4.625-1.375)}$$

$$= .745 + .189 \doteq .93 \text{ in.}^2,$$

use bars #6 @ $5\frac{1}{2}$" (A_s = .96 in.^{2}) top and bottom.

The required compressive reinforcement is

$$A_s = \frac{.92(12)}{(5.97)(4.625-1.375)} = .57 \text{ in}^2.$$

This is less than the .96 in.^{2} provided; thus this requirement is satisfied.

Distribution reinforcement perpendicular to the main reinforcement should be provided as follows (1)

$$\frac{1}{\sqrt{s}} A_s ,$$

but not greater than

$$\frac{1}{2} A_s .$$

From this

$$\frac{1}{\sqrt{7.58}} (.93) \doteq .34 \text{ in.}^2$$

Use bars #4 @ 7" ($A_s = .34$ in.^{2}).

Bond and shear shall not be considered critical if the preceding design approach is taken. (1).

4-7 Guardrail Design

A standard guardrail detail is selected from AISC (6). This detail is shown on the engineering drawing (Part V).

4-8 Dead Load Deflection

The dead load deflection curve is calculated using the conjugate beam method (Table 13). The general shapes of the dead load moment diagram, the conjugate beam, and the deflection curve are shown (Fig. 10).

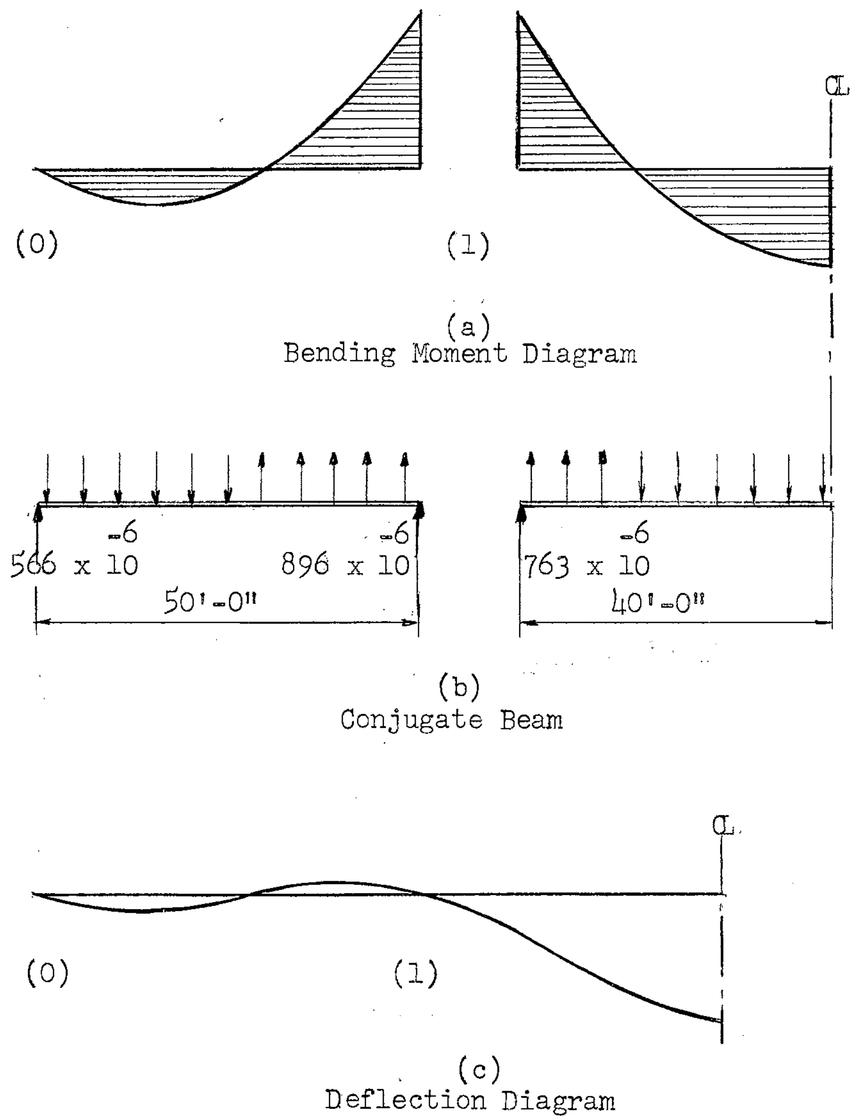


Fig. 10

Dead Load Deflection

TABLE 13 DEAD LOAD DEFLECTION

Sect.	$\frac{1}{I_i}(10)^6$	$M^{(DL)}$	$\bar{P}_i$ *	$\bar{P}_i \frac{x'}{L}(10)^6$	$\bar{V}(10)^6$	$(\bar{M}_i=\Delta_i)(10)^2$
0	-	0	-	-		0.00
					+ 566	
1	164.0	298	101	96.0		+ 1.70
					+ 465	
2	164.0	680	231	196.0		+ 4.50
					+ 234	
3	164.0	860	292	219.0		+ 5.90
					- 58	
4	121.5	856	215	140.0		+ 5.57
					- 273	
5	121.5	624	157	86.0		+ 3.93
					- 430	
6	121.5	168	42	19.0		+ 1.35
					- 472	
7	121.5	- 492	-123	- 43.0		- 1.48
					- 349	
8	79.5	-1356	-223	- 56.0		- 3.57
					- 126	
9	79.5	-2440	-402	- 60.0		- 4.32
					+ 276	
10	79.5	-3756	-620	- 31.0		- 2.67
					+ 896	
(1)	-	-4524	-	V_{01} = 566.0		0.00
					+ 763	
11	79.5	-3150	-826			+ 3.66
					+1589	
12	79.5	- 940	-247			+18.90
					+1839	
13	79.5	705	185			+36.50
					+1651	
14	121.5	1780	716			+52.40
					+ 935	
15	121.5	2320	935			+61.40
					000	

$$* \ \bar{P}_i = \frac{M_i^{(DL)} \Delta s_i}{EI_i}(10)^6$$

PART V

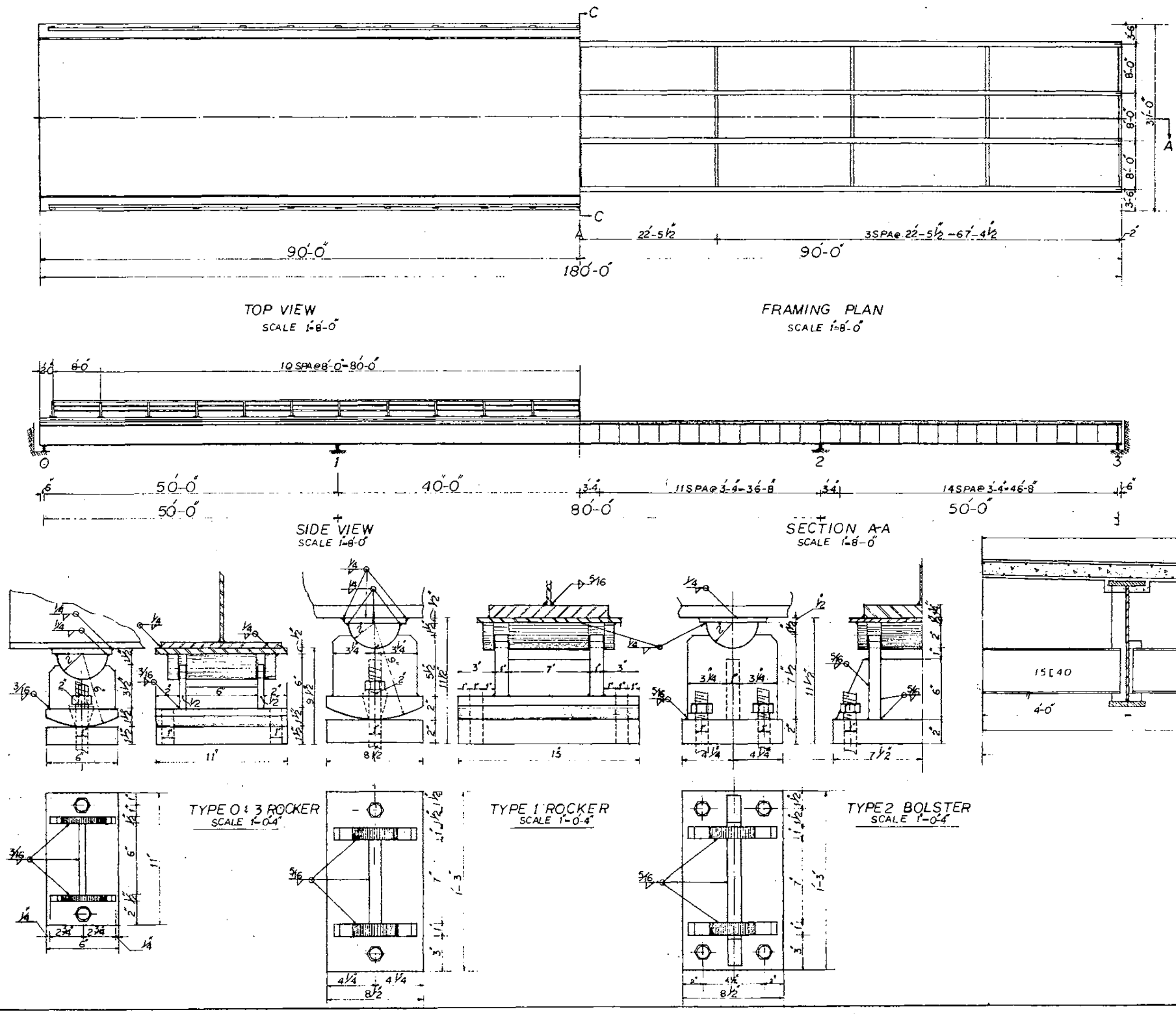
SUMMARY AND DETAILED DRAWINGS

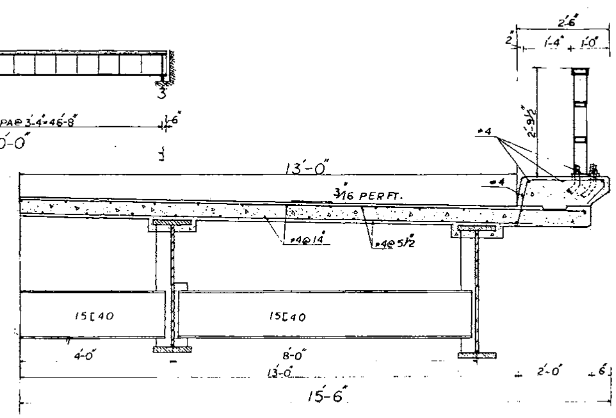
5-1 Summary

A three-span plate-girder bridge has been analyzed and designed in this report. The AASHO Standard Specifications for Highway Bridges was used as the guide in this design. The computation of the redundant influence line ordinates was accomplished by the carry-over moment method. Deformations were evaluated by means of the conjugate beam principle.

5-2 Detailed Drawings

The final Engineering drawings are shown as Plates 1, 2.





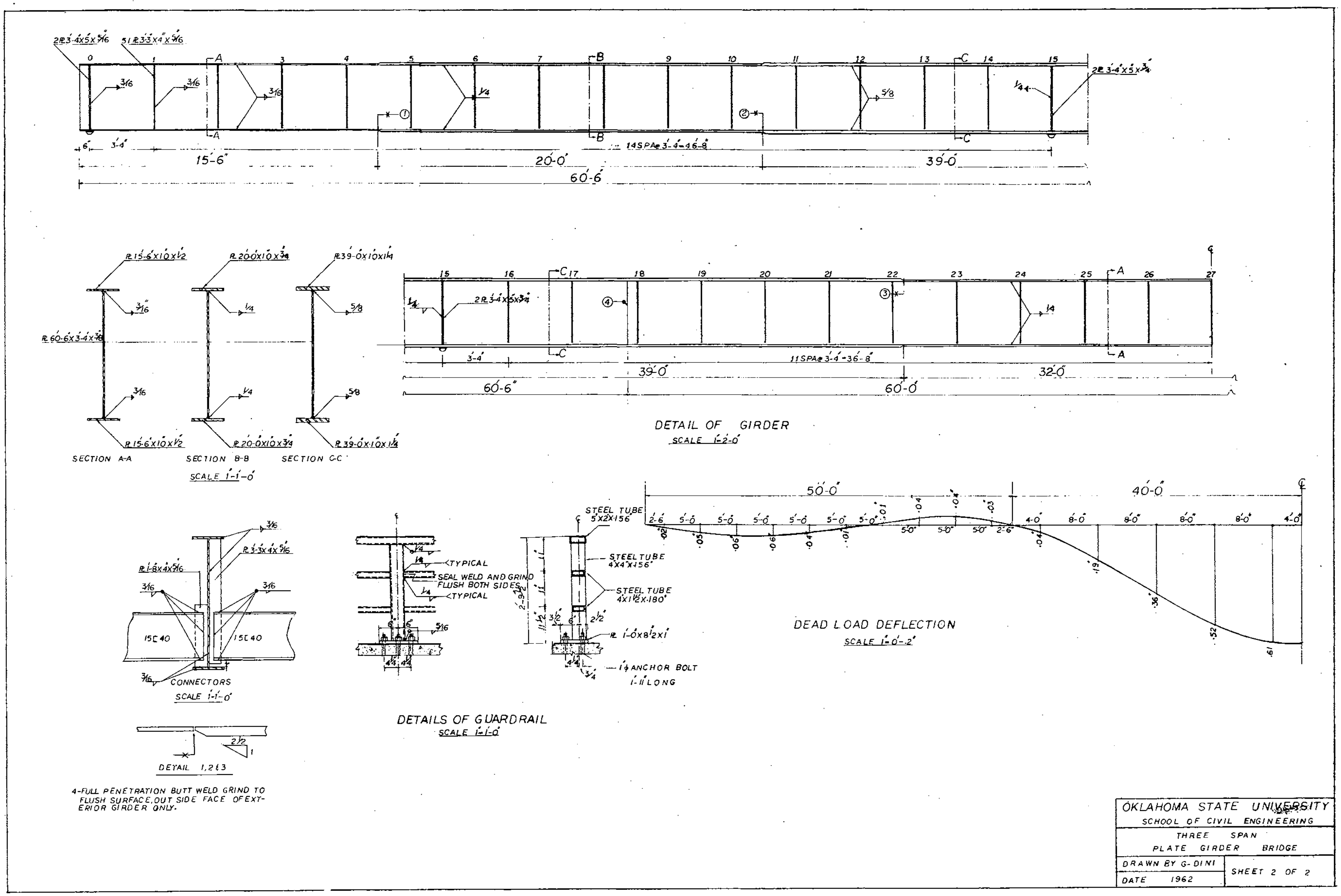
STEEL TABLE FOR FRAMING

DESCRIPTION	SIZE	NO	WEIGHT
TOW FLANGS	15'X10"X1/2"=900	16	4100
" "	20'-0"X10"X3/4"=1800	16	8,200
" "	39'-0"X10"X1 1/4"=5850	16	26,600
" "	32'-0"X10"X3/4"=2880	8	6,500
WEB PL	60'-0"X40"X3/8"=10800	12	37,000
STIFFENERS	3'-3"X4"X5/16=48.8	204	2,840
B "	3'-4"X5"X5/16=62.5	16	.280
" "	3'-4"X5"X3/4=150	16	.680
CONNECTOR PLS	2'-0"X4"X5/16=30	27	.230
DIAPHRAGMS	7'-10" U (15"-40)	27	8,450
		Σ	94,880

OKLAHOMA STATE UNIVERSITY
SCHOOL OF CIVIL ENGINEERING

THREE SPAN PLATE GIRDER BRIDGE

DRAWN BY G-DINI
DATE 1962
SHEET 1 OF 2



DETAIL OF GIRDER
SCALE 1'-2'-0"
SECTION A-A
SECTION B-B
SECTION C-C
SCALE 1'-1'-0"
DEAD LOAD DEFLECTION
SCALE 1"-0'-2"
CONNECTORS
SCALE 1'-1'-0"
DETAILS OF GUARDRAIL
SCALE 1'-1'-0"
DETAIL 1,2 & 3
4-FULL PENETRATION BUTT WELD GRIND TO FLUSH SURFACE, OUT SIDE FACE OF EXTERIOR GIRDER ONLY.
STEEL TUBE 5X2X156
STEEL TUBE 4X4X156"
STEEL TUBE 4X1½X.180"
1¼ ANCHOR BOLT 1'-11" LONG
TYPICAL
SEAL WELD AND GRIND FLUSH BOTH SIDES
OKLAHOMA STATE UNIVERSITY
SCHOOL OF CIVIL ENGINEERING
THREE SPAN
PLATE GIRDER BRIDGE
DRAWN BY G- DINI
DATE 1962
SHEET 2 OF 2

REFERENCES

1. American Association of State Highway Officials, Standard Specifications for Highway Bridges, seventh edition, Washington, 1957.

2. American Welding Society, Standard Specifications for Welded Highway and Railway Bridges, fifth edition, New York, 1956.

3. Jan J. Tuma, Samuel E. French, Jr. and Therman I. Lassley, "Analysis of Continuous Beam Bridges, Volume I - Carry-Over Procedure", School of Civil Engineering Research Publication, No. 3, Oklahoma State University, Stillwater, 1959.

4. Kerry S. Havner and Jan J. Tuma, "Influence Lines for Continuous Beams", Oklahoma Engineering Experiment Station Publication, No.106, Oklahoma State University, Stillwater, 1959.

5. American Institute of Steel Construction, Steel Construction Manual, fifth edition, New York, 1956.

6. American Institute of Steel Construction, Highway Bridges of Steel, New York.

7. N.L. Ashton, "A Modern Steel Deck Girder Highway Bridge", Studies in Structural Arc Welding, James F. Lincoln Arc Welding Foundation, Bulletin 838, Cleveland, 1952.

8. James G. Clark (Editor), Welded Interstate Highway Bridges, James F. Lincoln Arc Welding Foundation, Cleveland, 1960.

9. James W. Gillespie and Donald L. Wickens, "Design of a Continuous Rigid-Frame Concrete Bridge", Office of Engineering Research, No.121 Oklahoma State University, Stillwater, 1961.

10. J. Sterling Kinney, Indeterminate Structural Analysis, Addison-Wesley, complete edition, reading, 1957.

11. Edwin H. Gaylord, Jr. and Charles N. Gaylord, Design of Steel Structures, McGraw-Hill, New York, 1957.

12. Boris Bresler and T.Y. Lin, Design of Steel Structures, John Wiley & Sons, New York, 1960.

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