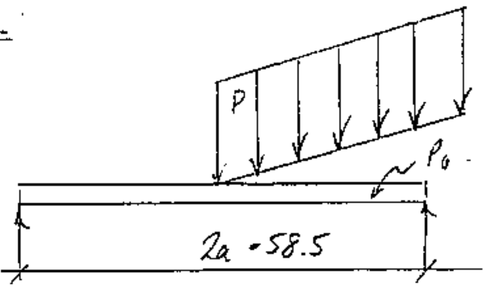
APPENDIX D

Structural Assessment calculations for Kwai Chung Road Flyover near Mei Foo Sun Chuen

LAND SPAN DECK.



BS153: PART 3A

$$P_0 \ (UDL) = 208 \ lb/ft^2$$
$$P \ (KEL) = 2,700 \ lb/ft.$$

BS5400 : PART 2

SDM1 TABLE 17
$$P_0 \ (UDL) = 320 \ lb/ft^2$$
$$P \ (KEL) = 2,700 \ lb/ft$$
(Assuming notional lane W = 3m)

$$\therefore \ M_y = M_{u} b \times \frac{4P_0}{\pi} + \frac{P}{a} M_x \times b.$$

$$= 0.244 \times \frac{80.9}{2} \times \left(\frac{4 \times 320}{3.14} + \frac{2700}{58.5/2} \right)$$

$$= 9.86 \ (408 + 92.2)$$

$= 4,932$ #'/ft wide (moment due to distribution analysis)
(1966 3,965 #'/ft wide)

Local Bending.

$a_{ac} = 12.5"$ $L = 76"$ $a_A = 12.5/76 = 0.164 = a_B$.

$r_{abc} = 7.5"$ $r_p = \frac{7.5"}{7} = 1.07 = r_B$

$b = 0.5$

Use FEM for UDL $= 0.097 \times wL^2$
FEM for H Load $= 0.15 \times PL$ } C.O. factor use 0.633.

Cantilever Moment at A

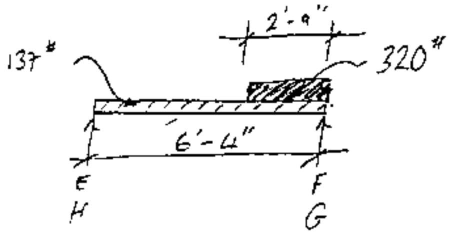
$$M = (20 \times 25 + 716) \times (1.25 + 1.17)$$

$$= 2220 \ \#'/ft.$$

Span AB, BC, CD, DE, HI, IJ, JK & KL (P.L only)

$$M_e = 534 \ \#'/ft.$$

Span EF & GH.



$$M_E = M_H = 0.095 \times 137 \times 6.33^2 + \frac{0.095}{0.0833} \times 0.042 \times (2.75 \times 320) \times 6.33 + 13$$

$$= 521 + 267 + 13$$

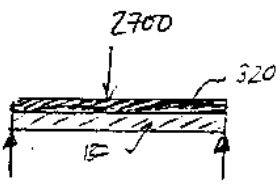
$$= 801 \ \#'/ft.$$

$$M_F = M_G = 0.095 \times 137 \times 6.33^2 + \frac{0.095}{0.0833} \times 0.112 \, (2.75 \times 320) \times 6.33 + 13$$

$$= 521 + 712 + 13$$

$$= 1246 \ \#'/ft.$$

Span FG



$$M_{\mathcal{C}} = 0.095 \times (137 + 320) \times 6.33^2 + 13 + 0.15 \times 2700 \times 6.33$$

$$= 1740 + 13 + 2560$$

$$= 4313 \ \#'/ft.$$

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0.633

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	#'	A		B		C		D		E		F
FEM	-2220	-534	-534	-534	-534	-534	-534	-534	-534	-801	-1246	-4313
D		-1686	0	0	0	0	0	0	-134	+134	-1534	+1534
C.O		0	+1065	0	0	0	0	+85	0	+971	-85	-971
D		0.	-533	+533	0	0	+43	-43	+486	-486	-443	+443
C.O.		+337	0	0	-337	-27	0	-308	+27	+280	+308	-280
D		-337	0	0	+155	-155	-104	+104	+127	-127	-294	+294
C.O		0	+213	-98	0	+66	+98	-80	-66	+186	+80	-186
D		0	-156	+156	+33	-33	-89	+89	+126	-126	-133	+133
C.O.		+99	0	-21	-99	+56	+21	-80	-56	+84	+80	-84
D		-99	-11	+11	+78	-78	-51	+51	+70	-70	-82	+82
C.O.		+7	+63	-49	-7	+32	+49	-44	-32	+52	+44	-52
D		-7	-56	+56	+20	-20	-47	+47	+42	-42	-48	+48
C.O		+35	+4	-13	-35	+30	+13	-27	-30	+30	+27	-30
D		-35	-9	+9	+33	-33	-20	+20	+30	-30	-29	+29
C.O		+6	+22	-21	-6	+13	+21	-19	-13	+18	+19	-18
D		-6	-22	+22	+10	-10	-20	+20	+16	-16	-19	+19
C.O		+14	+4	-6	-14	+13	-6	-10	-13	+12	+10	-12
D		-14	-5	+5	+14	-14	-8	+8	+3	-13	-11	+11

-3353 #'

Free span M for span FG

$$M = \frac{1}{8}(320 + 137) \times 6.33^2 - \frac{1}{4} \times 2700 \times 6.33$$

$$= 6562 \text{ #'/ft}$$

$\therefore$ Local sagging $M = 6562 - 3353 = 3209$ #'/ft

$\therefore$ Max sagging $M = 4932 \times 0.95 + 3209$
$= 7894$ #'/ft
$= 94,728$ #"/ft. (1966 → 78,500 #"/ft)

Hogging Moment.

$M_y = 0.1292 \times 40.45 \left(\frac{4 \times 370}{3.14} + \frac{2700}{29.25} \right)$

$= 5.23 (500)$

$= 2615$ #'/ft. due to Morice Analysis (1966 ⇒ 1,950#'/ft)

Local bending

By Morice analysis & Local bending.

Cantilever Moment at 'A' & 'L'

$M = 716 \times 2.42 = 1720$ #'

Span BC, & IJ

$M_c = 0.095 (320 + 137) \times 6.33^2 + 13.$

$= 1740$#' $+ 13$#'

$= 1753$#'/ft.

Span CD, DE, HI & JK

$M_c = 4313$#'/ft.

Span AB, EF, FG, GH, & KL

$M_c = 521 + 13$

$= 534$#'/ft

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fixed

	A	A	B	B	C	C	D	D	E	E	F	F	G	G	H	H	I	I	J	J	K	K	L	L
EM.	-1720	-534	-534	-1753	-1753	-4313	-4313	-4313	-4313	-534	-534	-534	-534	-534	-534	-4313	-4313	-1753	-1753	-4313	-4313	-534	-534	1720
D		0	-610	+610	-1280	+1280	0	0	+1890	-1890	0	0	0	0	-1890	+1890	+1280	-1280	-1280	+1280	+1890	-1890	0	
C.O.		+386	0	+810	-386	0	-810	-1196	0	0	+1196	0	0	+1196	0	-810	-1196	+810	+810	-1196	-810	0	+1196	
D		0	+405	-405	+193	-193	-193	+193	0	0	-598	+598	+598	-598	-405	+405	+1003	-1003	-1003	+1003	+405	-405	0	
O		256	0	-122	+256	+122	+122	0	-122	+378	0	-378	-378	+256	+378	-635	-256	+635	+635	-256	-635	0	+256	
D		0	-61	+61	-67	+67	-61	+61	+250	-250	-189	+189	+317	-317	-507	+507	+446	-446	-446	+446	+318	-318	0	
O		+39	0	+42	-39	+39	-42	-158	-39	+120	+158	-201	-170	+221	+201	-282	-321	+282	+282	-201	-282	0	+201	
		0	+21	-21	+39	-39	-58	+58	+41	-41	-180	+180	+221	-221	-242	+242	+302	-302	-242	+242	+141	-141	0	
		-13	0	-25	+13	+37	+25	-26	-37	+114	+26	-140	-114	+153	+140	-191	-153	+153	+191	-89	-153	0	+153	
		0	-13	+13	+12	-12	-26	+26	+76	-76	-83	+83	+134	-134	-166	+166	+153	-153	-140	+140	+77	-77	0	
		+8	0	-8	-8	+16	+8	-48	-16	+53	+48	-85	-53	+105	+85	-97	-105	+89	+97	-49	-89	0	+49	
		0	-4	+4	+12	-12	-28	+28	+35	-35	-67	+67	+79	-79	-91	+91	+97	-97	-73	+73	+45	-45	0	
					-3008	-3008	-5376	-5375	-2235	-2161														
					-3008#'		-5376#'		-2198#'															

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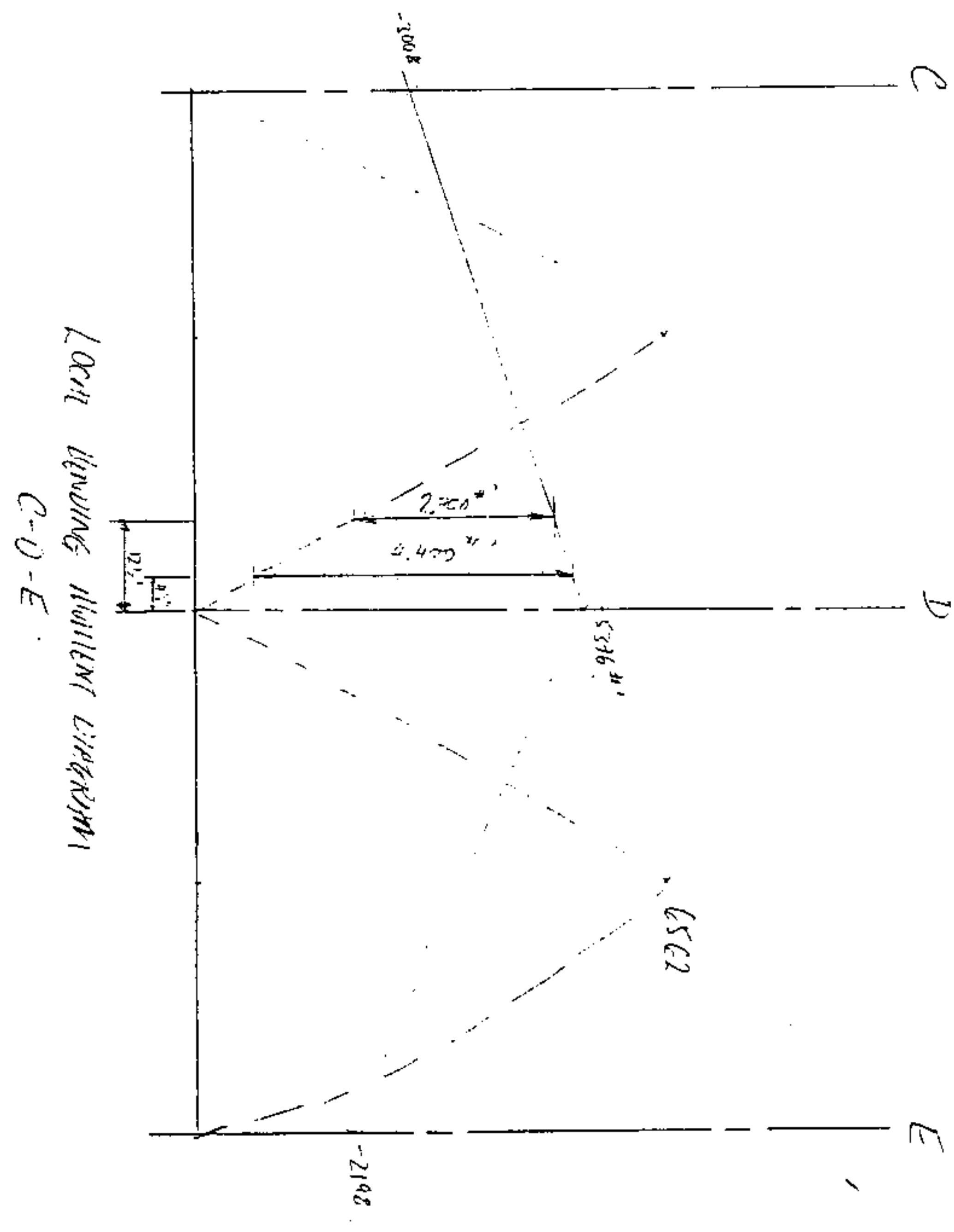

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Max hogging moment at 4½" from ℄ of Beam

$= 4,400 + 2615 = 7015\ \#'/ft = 84,180\ \#''/ft$

Max hogging moment at 12½" from ℄ of Beam.

$= 2,700 + 2615 = 5315\ \#'/ft = 63,780\ \#''/ft.$

$\therefore$ Design M $= 84,180\ \#''/ft$ (1966 → 70,400 #"/ft) $h = 14½''$

$= 63,780\ \#''/ft$ (1966 → 49,700 #"/ft) $h = 7''$

$Q = \dfrac{63780}{12 \times 5.25^2} = 193$

(1966 Q=1

try ½ H.T. @ 6"

$f_s = \dfrac{63,780}{0.393 \times 0.85 \times 5.25} = 36,368$ psi

$T = \dfrac{12/6 \times 5.75 \times 30 \times 10^6 \sqrt{0.5}}{12 \times 7^3/12 / 3.5 \times 36,368} = \dfrac{224 \times 10^6}{3.56 \times 10^6} = 63 \not> 70$

(Table 30.3) $\therefore$ unacceptable

Check at 4½" from ℄ of Beam.

$f_s = \dfrac{84,180}{0.393 \times 0.85 \times 12.75} = 19,765$ psi $\not< 18,000$ psi $\therefore$ unacceptable

(1966 16,600 psi)