PART ONE: SECONDARY MOMENTS IN BUILDING BENTS DUE TO COLUMN SHORTENING

PART TWO: PRIMARY AND SECONDARY MOMENTS IN A BUILDING BENT DUE TO FOUNDATION SETTLEMENT

MERIT P. WHITE

Submitted as a Requirement for the Degree

of

Master of Science in Civil Engineering

California Institute of Technology

Pasadena, California

June 1, 1932

PRELIMINARY:

The Hardy Cross method of distributing fixed end moments was employed. The directional convention of moment sign is used, that is, a moment which rotates a joint clockwise is positive. On the summary sheets, all lengths are in inches (unless otherwise specified) and all moments are in inch-kips.

The first part of this paper contains the conclusions and the summarizing sheets for both parts of this thesis as well as a copy of a discussion on 'Wind-bracing in Buildings' submitted to the 'Proceedings of the American Society of Civil Engineers' by V. A. Vanoni and the writer. The second part is made up of computation sheets.

PART ONE

SECONDARY MOMENTS IN BUILDING BENTS

DUE TO COLUMN SHORTENING UNDER HORIZONTAL LOADS

The primary purpose of this investigation was to determine for a particular building bent the importance of secondary moments due to column shortening. It was also desired to obtain criteria for predicting the probable importance of such secondaries.

The twenty-story, three-bay bent analyzed in Bulletin 80 of the University of Illinois was selected for investigation.

In this investigation the foundations of the bent were assumed firm, no settlement was considered. Any such movement would tend to increase the secondary moments. The second part of this discussion deals with the effect of settlement of alternate columns of a bent.

Starting with no displacement at the foundations the cumulative column shortening in each column at each floor was determined. The fixed end moments (bending moments which would result from differential vertical movement of the ends without rotation) of the girders were computed and distributed by the Hardy Cross Method.

Summary sheet #1 outlines the procedure and gives all data necessary to obtain fixed end girder moments. All values are obtained directly or indirectly from Bulletin 80 (University of Illinois). "P" is the column load in pounds, tension on one side of the center-line, compression on the other. Under "h" are given column lengths between floors, in inches. "A" is the column area in $inches^2$. I is the girder moment of inertia in $inches^4$. L^2 is the girder length squared in $inches^2$, while M_f is the fixed end girder moment in inch-pounds. Referring to the diagram on summary sheet #2, "d" is the cumulative differential column shortening at any story. "d_a" is the difference between the cumulative shortening (or lengthening) of columns "A" and "B". "d_b" is twice the cumulative shortening of column "B". The computations give expressions for "M_{fa}" and "M_{fb}" in terms of I, L, P, h, and A.

The fixed end moments thus obtained were applied to the girders and distributed by the Hardy Cross Method, taking account of sidesway. Because of the proportions of the bent sidesway was considerable and seven cycles were required before satisfactory approach to equilibrium obtained.

The final moments obtained were surprising to say the least, and at first very disappointing. In general their magnitudes were of the order of the probable accuracy of the computations. In other words the moments obtained were probably accurate within several hundred percent. The relatively largest final girder moment was only eleven percent of the original fixed end moment. Summary sheets #2, #3, #4, give the primary wind moments (not the fixed end moments) and the resulting secondary moments. It must be remembered that these values are not exact and do no more than give the order of magnitude of the secondary moments.

The value of this investigation is not immediately obvious. In fact it seems a waste of time to spend a term discovering that the secondary moments in a building bent are negligeable. V.A.Vanoni, making the same investigation on a different bent, found very large secondary moments, sometimes even greater than the original wind moments, and generally about half as great. Apparently secondary moments are not always negligeable.

The question immediately arises: Can we know beforehand whether or not secondary moments require consideration? The present investigation furnishes a clue to the answer.

In the bent under discussion the column stresses on one side of the center-line of the bent are of the same sign, and the column stress and therefore the column shortening is roughly proportional to the distance from the center-line. Obviously when equilibrium is reached the bent will have moved a considerable distance sidewise but the girders will be nearly straight.

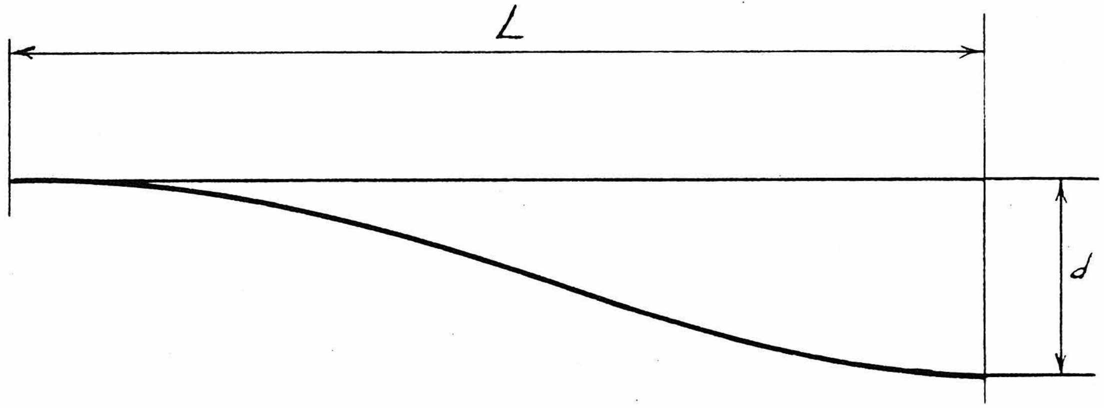
In Mr. Vanoni's bent, on the other hand, the columns are alternately in tension and compression, so that the girders are subjected to considerable bending. Sidesway is small but column and girder moments are large.

There is included a discussion submitted to the "Proceedings of the American Society of Civil Engineers" by V.A. Vanoni and the writer.

This discussion includes a diagram summarizing the results obtained by Mr. Vanoni. The reader is also referred to Mr. Vanoni's Thesis.

Story	Column A					Column B					Girder a				Girder b			
	P	h	A	$\frac{Ph}{A}$	$\Sigma \frac{Ph}{A}$	P	h	A	$\frac{Ph}{A}$	$\Sigma \frac{Ph}{A}$	Diff. $\Sigma \frac{Ph}{A}$	I	L^2	M_f	$2\Sigma_B \frac{Ph}{A}$	I	L^2	M_f
1	14,464	264	112.8	33852	33852	4587	264	112.8	10735	10735	23117	8058	69696	16036	21470	8058	46656	22249
2	12,444	192	"	21181	55033	4317	192	"	7348	18083	36950	5641	"	17943	36166	6303	"	29315
3	10,977	168	103.8	17766	72799	3689	168	103.8	5971	24054	48745	"	"	23672	48108	5641	"	34899
4	9617	"	"	15565	88364	3294	"	"	5331	29385	58979	5161	"	26204	58770	5161	"	39006
5	8389	"	92.55	15228	103592	2813	"	94.8	4985	34370	69222	"	"	30755	68740	"	"	45623
6	7179	"	"	13031	116623	2489	"	"	4411	38781	77842	3717	"	24909	77562	3717	"	37075
7	6193	144	83.55	10674	127297	2163	144	85.8	3630	42411	84886	3387	"	24751	84822	3387	"	36946
8	5273	"	"	9088	136385	1855	"	"	3113	45524	90861	2025	"	15840	91048	2025	"	23710
9	4561	"	72.42	9069	145454	1571	"	79.05	2862	48386	97068	"	"	16919	96773	"	"	25201
10	3856	"	"	7667	153121	1288	"	"	2346	50732	102389	"	"	17849	101464	"	"	26423
11	3195	"	64.24	7162	160283	1037	"	67.80	2202	52934	107349	"	"	18714	105868	"	"	27570
12	2595	"	"	5817	166100	827	"	"	1756	54690	111410	"	"	19422	109380	"	"	28484
13	2057	"	53.79	5507	171607	640	"	57.92	1591	56281	115326	"	"	20105	112562	"	"	29313
14	1579	"	"	4227	175834	478	"	"	1188	57469	118365	"	"	20635	114938	"	"	29932
15	1163	"	42.26	3963	179797	342	"	44.39	1109	58578	121219	"	"	21132	117156	"	"	30509
16	809	"	"	2757	182554	237	"	"	769	59347	123207	"	"	21479	118694	"	"	30910
17	521	"	38.86	1931	184485	150	"	41.00	527	59874	124611	"	"	21723	119748	"	"	31184
18	300	"	"	1112	185597	83	"	"	292	60166	125431	"	"	21866	120332	"	"	31337
19	140	"	"	519	186116	37	"	"	130	60296	125820	"	"	21934	120592	"	"	31404
20	43	"	"	159	186275	8	"	"	28	60234	125951	"	"	21957	120668	"	"	31419

FIXED END MOMENTS



$$M_f = 2EK\left(3\frac{d}{L}\right)$$ ~ SLOPE DEFLECTION EQUATION

$$= \frac{6EK}{L}d = \frac{6EI}{L^2}d$$

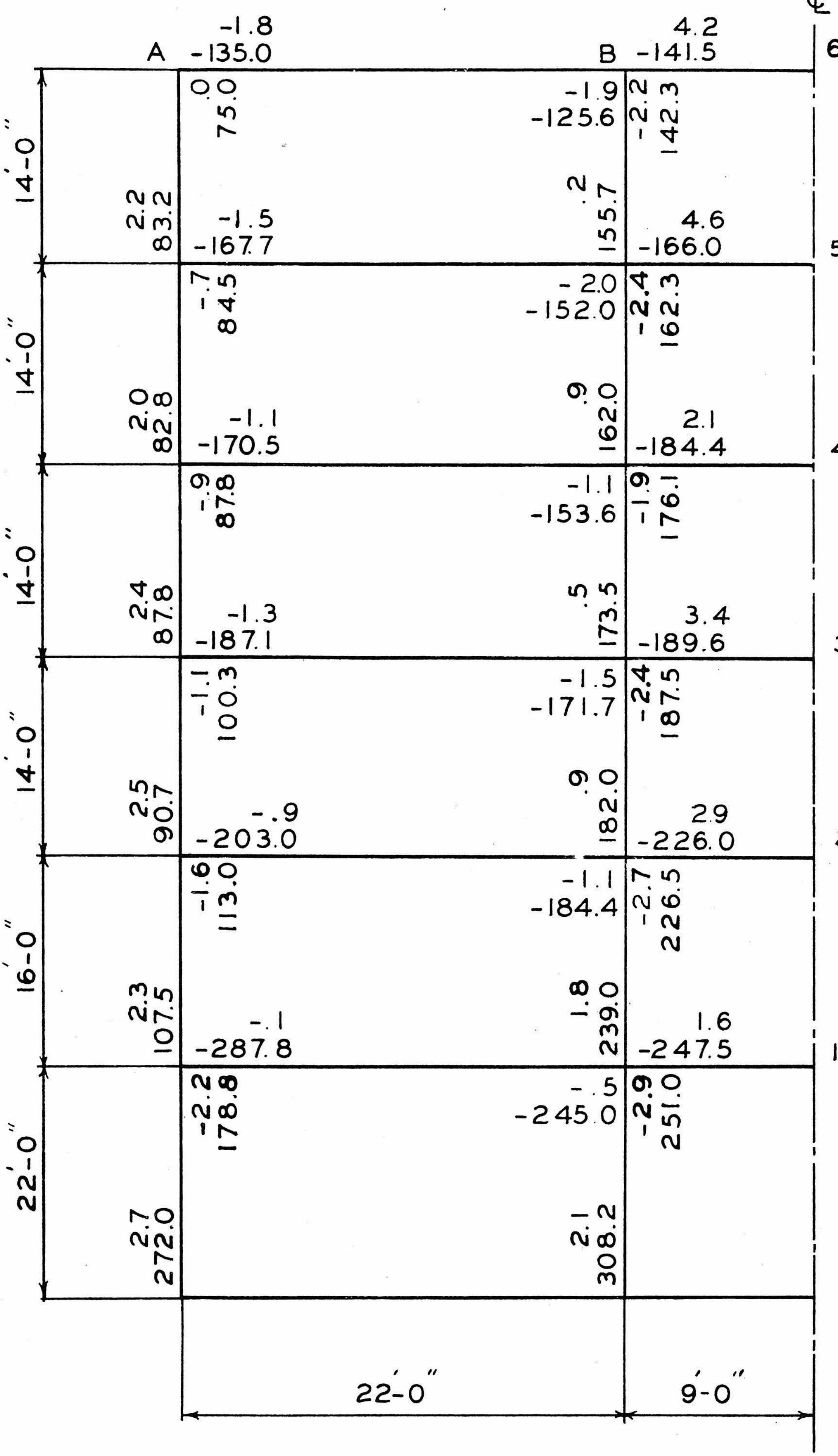
$$d_a = \Sigma\delta_A - \Sigma\delta_B \qquad \delta_A = \frac{P_A h_A}{A_A E} \qquad \delta_B = \frac{P_B h_B}{A_B E}$$

$$d_b = 2\Sigma\delta_B$$

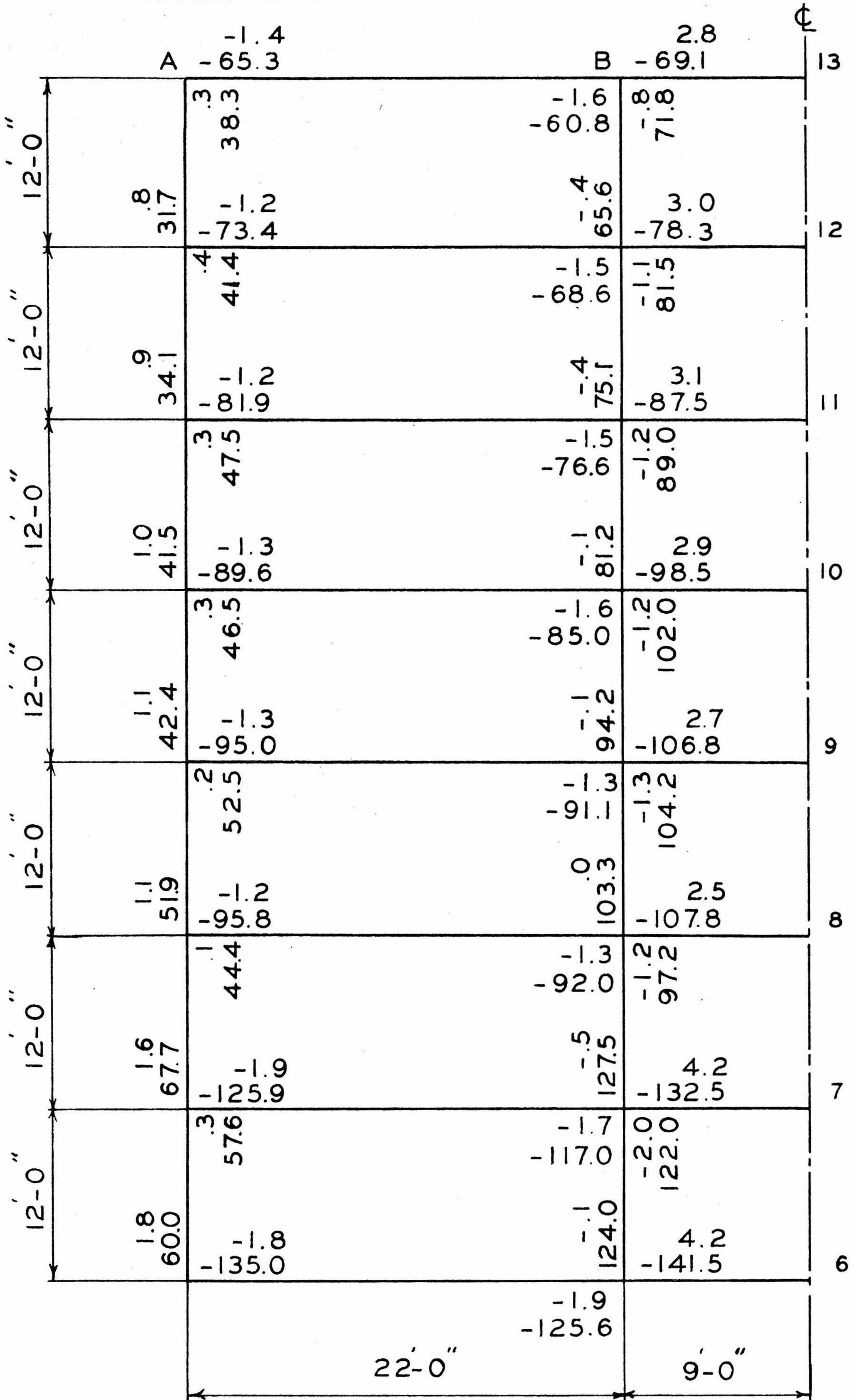
$$M_{f_a} = \frac{6I_a}{L_a^2}\left(\Sigma\frac{P_A h_A}{A_A} - \Sigma\frac{P_B h_B}{A_B}\right)$$

$$M_{f_b} = \frac{12 I_b}{L_b^2}\Sigma\frac{P_B h_B}{A_B}$$

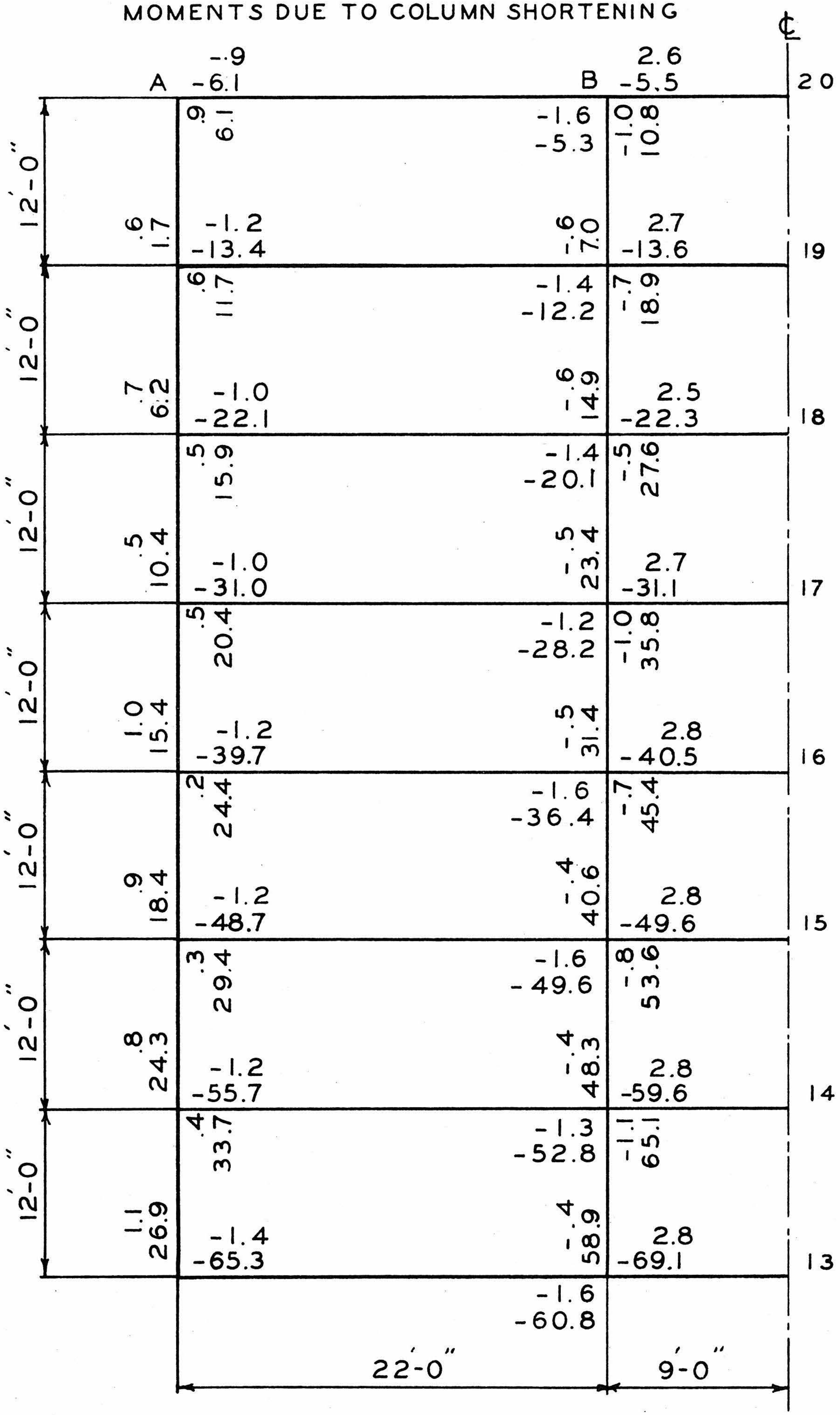
PRIMARY WIND MOMENTS AND SECONDARY MOMENTS DUE TO COLUMN SHORTENING



PRIMARY WIND MOMENTS AND SECONDARY
MOMENTS DUE TO COLUMN SHORTENING



PRIMARY WIND MOMENTS AND SECONDARY MOMENTS DUE TO COLUMN SHORTENING



WIND-BRACING IN STEEL BUILDINGS

DISCUSSION

A rather sketchy investigation disclosed only one reference to the subject of Secondary Stresses in building bents due to column shortening under wind-load. Bulletin 80 of the University of Illinois covers the subject in a paragraph. The shortening of the first story columns of the twenty-story bent is computed and the fixed-end moments in the first story girders determined. These moments are small compared with the original wind moments. However, it must be remembered that the effect of column shortening is cumulative from bottom to top, while the girder moments due to wind become smaller so that the effect of column shortening should be much greater at the top than near the ground.

In its discussion the Committee on Wind-Bracing states that in the case of a high, narrow building secondary moments require investigation. In the relatively high and narrow Wilson-Maney bent secondary moments are negligible. Apparantly there is another criterion. The importance of secondary moments depends upon the relative size of bays and upon the relative stiffness of columns and girders. In the Wilson-Maney bent both columns on one side of the center line have the same kind of stress under wind load and this stress and therefore the shortening is roughly proportional to the distance from the center line. Since all the fixed-end moments in the girders due to column shortening act in the same direction, they will all be reduced by the resulting sidesway and the girders in their final position (after equilibrium is reached) will be nearly straight. When we have alternate tension and compression in the columns, the girders are constrained and may therefore have large bending moments after equilibrium is reached. In such a case sidesway will increase certain girder moments and decrease others.

The committee does not consider the possibility that the secondary moments as first obtained may require correction. For example, in general (not always) the secondary moments in the girders will be opposite to the primary moments. If the secondaries are large, say 50% of the primaries, the resultant moments will be one-half of the original. But the secondaries will also produce column shortening which will cause more secondary moments, ordinarily of the same sign as the primary moments. If the first secondaries are 50% of the primary moments then the second secondaries will be about 25% of the primary moments which will make the resulting moments 75% of the original moments instead of 50%.

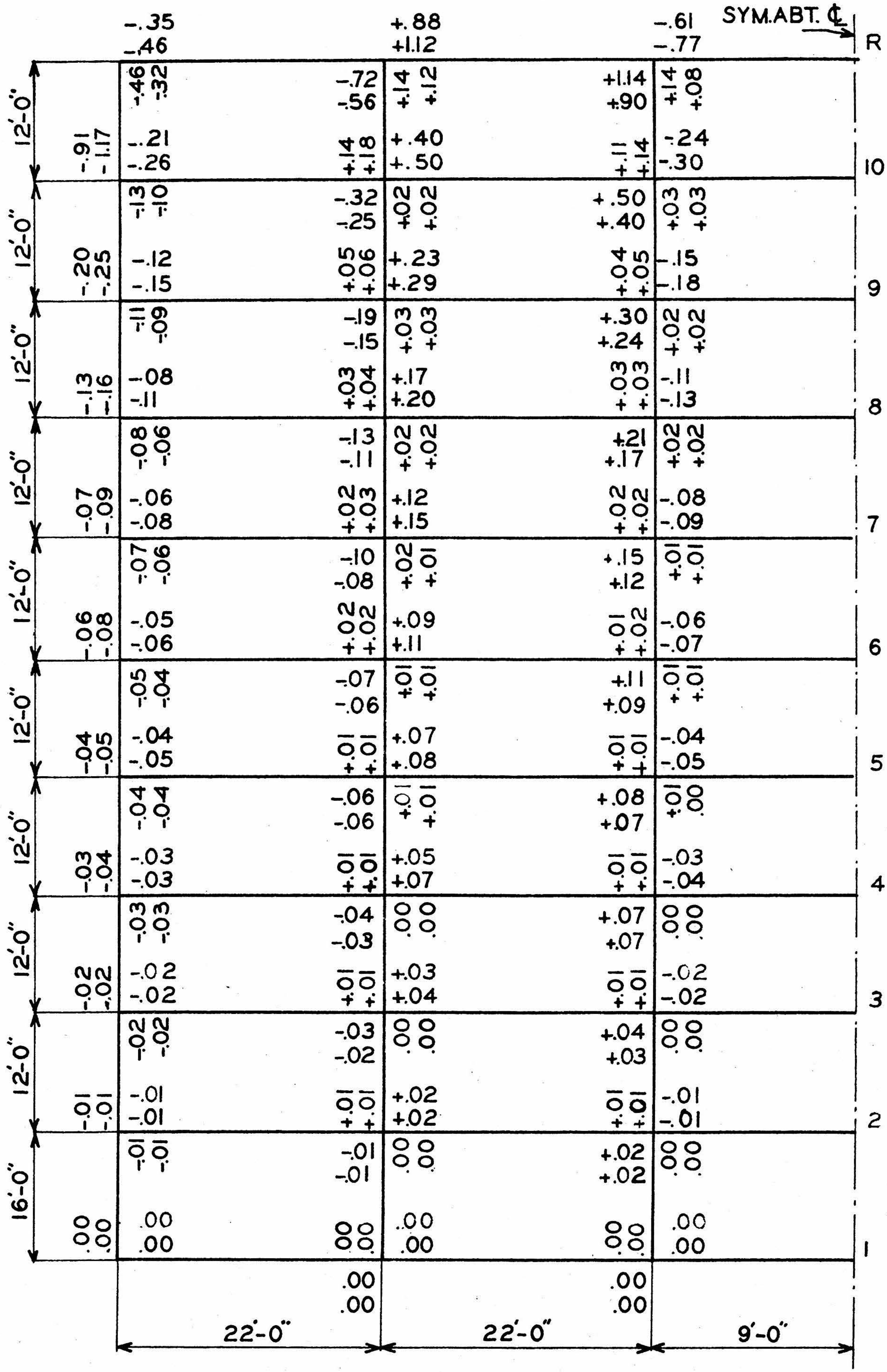
In general the first secondary moments will give results which, for girders, are on the unsafe side.

Calculations were made on two bents; one, the twenty story Wilson-Maney bent, gave negligible secondary moments. The other bent, which was obtained by adding twenty-two foot bays to the upper ten stories of the Wilson-Maney bent, gave very interesting results. Under wind loads the columns of this bent were alternately in tension and compression. The accompanying diagram gives the ratio of the first secondary to the original wind moments (lower figure), and the ratio of the secondary moment after two corrections (algebraic sum of secondary plus first correction plus second correction) to the original moment.

V.A. Vanoni,
Junior Member, Am.Soc.C.E.
Graduate Student, Calif. Inst. Tech.

M.P. White,
Assistant in Civil Engineering,
California Institute of Technology

RATIOS OF FIRST SECONDARY MOMENTS AND CORRECTED SECONDARY MOMENTS TO WIND MOMENTS



PART TWO

PRIMARY AND SECONDARY MOMENTS IN A BUILDING BENT

DUE TO FOUNDATION SETTLEMENT

In connection with the first part of this investigation it was decided to find the effect of column settlement. Because of the short time availible a very simple bent was selected, the upper six stories of an interior bay of the Insurance Exchange Building in Los Angeles. This building is of reinforced concrete. All essential dimensions are given on the summarizing diagram.

Fixed end moments resulting from one-quarter inch settlement of alternate footings were calculated and distributed by the Hardy Cross Method. The assumption of one-quarter inch differential settlement is certainly pessimistic, but is not an impossibility. In any case the effect would be proportional to the displacement.

The final moments thus obtained are much greater in the center than in the exterior bays. Dead load moments and wind load moments (assuming thirty pounds per square foot) had previously been determined for this bent. In the exterior panels the moments due to footing settlement were about equal to dead load moments and to wind load moments, but in the interior panel the moments due to settlement were from two to three times the dead load or wind load moments.

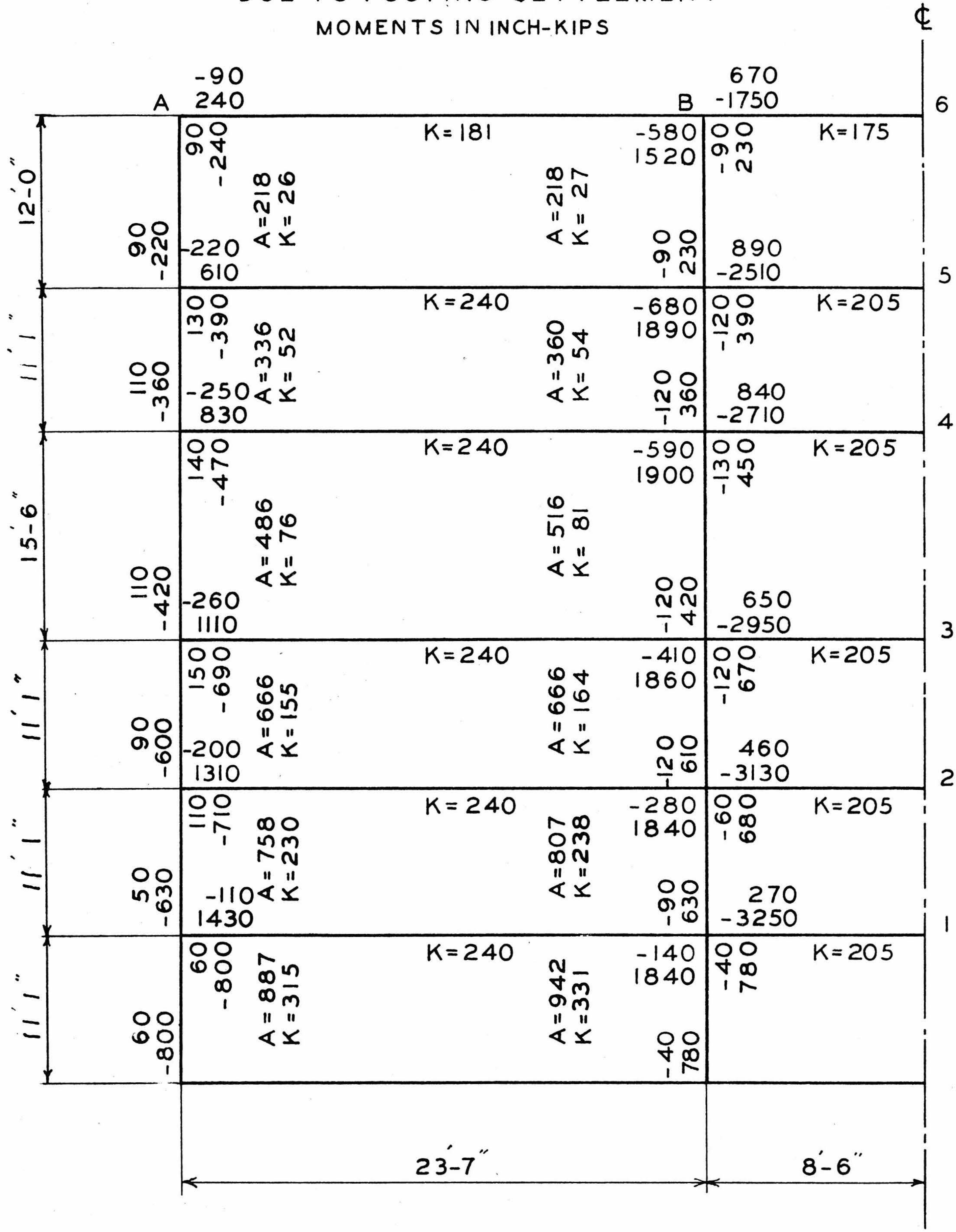
The next step was the calculation and distribution of fixed end moments due to column shortening. The final moments obtained by this process of course are correction moments for the original moments. They are of opposite sign and therefore, if neglected, constitute a factor of safety. They vary from about eight percent at the bottom to about thirty-five percent at the top of the bent.

Unlike secondary moments due to wind loads, moments due to footing settlement and the resulting secondary moments do not depend on the characteristics of the bent for their importance (except that they increase directly with stiffness of members). They are primarily a linear function of the settlement.

PRIMARY AND SECONDARY MOMENTS

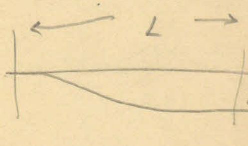
DUE TO FOOTING SETTLEMENT

MOMENTS IN INCH-KIPS



Story	Column A					Column B					Girder a				Girder b			
	P_{lbs}	h_{in}	A_{in^2}	$\frac{Ph}{A}$	$\Sigma\frac{Ph}{A}$	P	h	A	$\frac{Ph}{A}$	$\Sigma\frac{Ph}{A}$	Diff $\Sigma\frac{PL}{A}$	I_{in^4}	$L^2_{ft^2}$	$M_{in\,lbs}$	$(2\Sigma_b\frac{PL}{A})$	I	L^2	$M_{in\,lbs}$
1	14464	264	112.8	33852	33852	4587	264	112.8	10735	10735	23117	8058	64696 ~~67700~~	16,036	128820	8058	46656	22249
2	12444	192	112.8	21181	55033	4317	192	112.8	7348 (70)	18083	36950	5641	11616 (76)	17943	216996	6303	"	29315
3	10977	168	103.8	17766	72799	3689	168	103.8	5971	24054	48745	5641	"	23672	288648	5641	"	34899
4	9617	168	103.8	15565	88364	3294	168	103.8	5331	29385	58979	5161	"	26204	352620	5161	"	39006
5	8389	168	92.55	15228	103592	2813	168	94.8	4985	34370	69222	5161	"	30755	412440	5161	"	45623
6	7179	168	92.55	13031	116623	2459	168	94.8	4411	38781	77842	3717	"	24909	465372	3717	"	37075
7	6193	144	83.55	10674	127297	2163	144	85.8	3630	42411	84886	3387	"	24751	508932	3387	"	36946
8	5273	144	83.55	9088	136385	1855	"	85.8	3113	45524	90861	2025	"	15840	546288	2025	"	23710
9	4561	144	72.42	9069	145454	1571	"	79.05	2862	48386	97068	$6\frac{I}{L^2}$ = .17433	"	16919	580632	$\frac{I}{L^2}$ = .043403	"	25201
10	3856	144	72.42	7667	153121	1288	"	79.05	2346	50732	102389	"	"	17849	608784	"	"	26423
11	3195	144	64.24	7162	160283	1037	"	67.80	2202	52934	107349	"	"	18714	635208	"	"	27570
12	2595	144	64.24	5817 ~~5725~~	166100	827	"	67.80	1756	54690	111410	"	"	19422	656280	"	"	28484
13	2057	144	53.79	5507	171607	640	"	57.92	1568 (91)	56281	115326	"	"	20105	675372	"	"	29313
14	1579	144	53.79	4227	175834	478	"	57.92	1188	57469	118365	"	"	20635	689628	"	"	29932
15	1163	144	42.26	3963	179797	382	"	48.39	1109	58578	121219	"	"	21132	702936	"	"	30509
16	809	144	42.26	2757	182554	237	"	48.39	769	59347	123207	"	"	21479	712164	"	"	30910
17	521	144	38.86	1931	184485	150	"	41.0	527	59874	124611	"	"	21723	718488	"	"	31184
18	300	144	38.86	1112	185597	83	"	41.0	292	60166	125431	"	"	21866	721992	"	"	31337
19	140	144	38.86	519	186116	37	"	41.0	130	60296	125820	"	"	21934	723552	"	"	31404
20	43	144	38.86	159	186275	8	"	41.0	28	60324	125951	"	"	21957	723888	"	"	31419

Note: Values of P, h, A, I obtained from tables in bulletin 80 (U. of I)



$$\delta = \frac{ML^2}{6EI}; \quad M = \frac{6EI}{L^2}\delta$$

$$\delta_a = \Sigma\delta_A - \Sigma\delta_B$$

$$\delta_A = \frac{P_A h_A}{A_A E}$$

$$M_a = \frac{6 E I_a}{L_a^2 E}\left(\Sigma\frac{P_A h_A}{A_A} - \Sigma\right.$$

$$M_b = \frac{6 I_b}{L_b^2}\left(2\Sigma\frac{P_B h_B}{A_B}\right)$$

$$= \frac{12 I_b}{L_b^2}\left(\Sigma\frac{P_B h_B}{A_B}\right)$$

Moments in a Building Bent due to $\frac{1}{4}''$ settlement of alternate columns.

Fixed-end Girder Moments: due to $\frac{1}{4}''$ settlement.

$$M_f = 2EK\left(-\frac{3d}{L}\right) = \frac{K}{L} \times 6 \times 2 \times 10^6 \cdot \frac{1}{4} = \frac{3K}{L} \quad \text{in lbs} (10^6)$$

$K = \frac{I}{L}$

All moments exp. in 10^6 in lbs.

Floor	AB K	AB L	AB M_f	BC K	BC L	BC M_f
6	181	283	1.92	175	204	2.57
5	240	283	2.55	205	204	3.02
4	240	283	2.55	205	204	3.02
3	240	283	2.55	205	204	3.02
2	240	283	2.55	205	204	3.02
1	240	283	2.55	205	204	3.02

See sheets 2.3.4 for determination of final moments.

Column Stresses.

Floor	Col. A $\Sigma M_{gird.(AB)}$	Col. A P (lbs)	Col. B $\Sigma M_{gir(BC)}$	Col. B P (lbs)
6	1.76	− 6220	−3.50	+ 17140
5	4.26	− 15050	− 8.52	+ 41750
4	7.41	− 26200	−14.78	+ 72500
3	10.68	− 37700	−21.28	+ 104300
2	13.41	− 47400	− 26.70	+ 131000
1	16.38	− 57800	−32.60	+ 160,000

Joint	6 A			6 B			
Mem.	6A5		A6B	B6A	B6C	B6C	6B5
K/ΣK St.	.245						.255
K/ΣK joint	.125		.875	.473	.456		.071
Mf			+1.92	+1.92	−2.57		+.04
Bal. j.	−.24		−1.68	+.31	+.30		+.04
C.O.	−.10		+.15	−.84	+.15		+.01
S.S	+.14						+.14
B.J.	−.02		−.17	+.26	+.24		+.04
CO	−.01		+.13	−.08	+.12		+.02
SS	−.01						−.02
B.J.	−.01		−.10	−.02	−.02		0
CO	0		−.01	−.05	−.01		−.01
SS	+.01						+.01
B.J.	0		0	+.03	+.03		0
Co	0		+.01	0	+.01		0
SS	0						0
B.J.	0		−.01	−.01	0		0
CO							0
ok	−.24		+.24	+1.52	−1.75		+.23

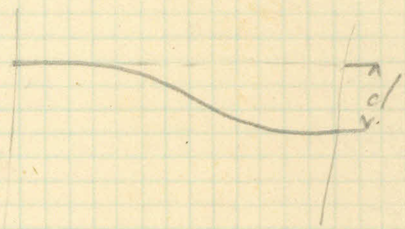
Joint	5 A			5 B			
Mem.	5A6	5A4	A5B	B5A	B5C	5B4	5B6
K/ΣK Story	.245	.245				.255	.255
K/ΣK joint	.080	.165	.755	.455	.390	.100	.055
Mf	[illegible]		+2.55	+2.55	−3.02		
BJ	−.20	−.42	−1.93	+.21	+.18	+.05	+.03
CO	−.12	−.18	+.10	−.96	+.09	+.02	+.02
SS	+.14	+.25				+.27	+.14
B.J.	−.02	−.03	−.14	+.28	+.24	+.06	+.04
CO	−.01	−.01	+.14	−.07	+.12	+.00	+.02
SS	−.01	0				−.01	−.02
BJ	−.01	−.02	−.08	−.02	−.02	0	0
CO	0	0	−.01	−.04	−.01	0	0
SS	+.01	+.01				+.01	+.01
B.J.	0	0	−.01	+.01	+.01	+.01	0
Co	0	0	0	0	0	0	0
SS	0	0				0	0
BJ	0	0	0	0	0	0	0
BJ,				−.09	−.08	−.02	−.01
CO			−.04	0	−.04	0	0
BJ.		+.01	+.03	+.02	+.02		
ok	−.22	−.39	+.61	+1.89	−2.51	+.39	+.23

Floor	h	A Col. 37 Eq. Area gross	B Col. 27	$\frac{P_A h}{A_A}$	$\frac{P_B h}{A_B}$	$\Sigma\left(\frac{P_A}{A_A}+\frac{P_B}{A_B}\right)h$	M_{fAB} in lbs	$\Sigma \frac{P_B}{A_B}h$	M_{fBC}
6									
	144"	218	218	4110	11330	144,710	-555,000	111,160	+1,145,000
5									
	133"	336	360	5470	14130	129,270	-796 000	99830	+1,203,000
4									
	186"	486	516	10,000	26100	109,670	-674 000	85,700	+1,033,000
3									
	133	624	666	7390	19100	82,570	-508 000	59,600	+719 000
2									
	133	758	807	7630	19800	56,050	-345 000	40,500	+488 000
1									
	133	887	942	7950	20700	28,650	-176 000	20,700	+249 000
0									

8 18 Girder

Fixed end Moments due to column shortening.

$$M_f = \frac{12K}{L}(10^6)\, d \quad (\text{in lbs})$$



d = cumulative col. shortening

$$M_{f_{AB}} = \frac{12K}{L}(10^6)\,\Sigma\left(\frac{P_A h}{A_A E} + \frac{P_B h}{A_B E}\right)$$

$$= \frac{12Kh10^6}{L\cdot 2\cdot 10^6}\,\Sigma\left(\frac{P_A}{A_A} + \frac{P_B}{A_B}\right) = \frac{6K}{L}\,\Sigma\left(\frac{P_A}{A_A} + \frac{P_B}{A_B}\right)h$$

$$M_{f\,BC} = \frac{12K}{L}\left(\Sigma \frac{P_B}{A_B}h\right)$$

Joint	4A		4A	4B		4B	
Memb	4A3	4A5	A4B	B4A	B4C	4B5	4B3
$\frac{K}{\Sigma K}$ st.	.242	.245				.255	.258
$\frac{K}{\Sigma K}$ jo.	.206	.141	.653	.415	.355	.090	.140
Mf			+2.55	+2.55	-3.02		
BJ	-.52	-.36	-1.67	+.20	+.17	+.04	+.06
CO	-.20	-.21	+.10	-.83	+.08	+.02	+.03
SS	+.29	+.25				+.27	+.31
BJ.	-.05	-.03	-.15	+.05	+.04	+.01	+.02
CO	-.02	-.01	+.02	-.07	+.02	+.03	-.01
SS	+.03	0				-.01	+.04
BJ	0	0	-.02	0	0	0	0
CO	0	-.01	0	-.01	0	0	0
SS	0	+.01				+.01	0
BJ.	0	0	0	0	0	0	0
CO	0	0	0	0	0	0	0
SS	0	0				0	0
BJ.	0	0	0	0	0	0	0
CO						-.01	0
BJ.				+.01			
OK	-.47	-.36	+.83	+1.96	-2.71	+.36	+.45

Joint	3A		3A	3B		3B	
Memb.	3A4	3A2	A3B	B3A	B3C	3B2	3B4
$\frac{K}{\Sigma K}$ sto.	.242	.243				.257	.258
$\frac{K}{\Sigma K}$ joint	.161	.339	.510	.348	.297	.238	.117
Mf			+2.55	+2.55	-3.02		
BJ.	-.41	-.84	-1.30	+.16	+.14	+.11	+.06
CO	-.26	-.31	+.08	-.65	+.07	+.04	+.03
SS	+.29	+.46				+.49	+.31
BJ.	-.04	-.09	-.13	-.10	-.09	-.07	-.03
CO	-.02	-.03	-.05	-.06	-.04	-.07	+.01
SS	+.03	+.13				+.14	+.04
BJ	-.01	-.02	-.03	-.01	-.01	0	0
CO	0	-.02	0	-.01	0	-.01	0
SS	0	+.03				+.03	0
BJ	0	0	-.01	-.01	0	0	0
CO	0	0	0	0	0	0	0
SS	0	0				.01	0
BJ.	0	0	0	-.01	0	0	0
OK	-.42	-.69	+1.11	+1.86	-2.95	+.67	+.42

Joint	2A			2B			
Mem	2A1	2A3	A2B	B2A	B2C	2B3	2B1
$\frac{K}{\Sigma K}$ st.	.246	.243				.257	.254
$\frac{K}{\Sigma K}$ j	.368	.248	.384	.283	.242	.194	.281
Mf			-.345	-.34	+.49		
BJ	+.13	+.08	+.13	-.04	-.04	-.03	-.04
CO	+.02	+.08	-.02	+.06	-.02	-.02	-.01
SS	-.04	-.06				-.07	-.04
BJ.	+.01	0	+.01	+.03	+.02	+.02	+.03
CO	0	0	+.01	0	+.01	0	+.02
SS	-.02	-.01				-.02	-.02
BJ	+.01	0	+.01	+.01	0	0	0
	+.11	+.09	-.20	-.28	+.46	-.12	-.06

Joint	1A			1B			
Mem	1A2	1A0	A1B	B1A	B1C	1BD	1B2
$K/\Sigma K$ st.	.246	.246				.254	.254
$K/\Sigma K$ j	.293	.401	.306	.237	.202	.326	.235
Mf			-.18	-.18	+.25		
BJ	+.05	+.07	+.06	-.02	-.01	-.02	-.02
CO	+.06	+.03	-.01	+.03	0	-.01	-.02
SS	-.04	-.03				-.04	-.05
B.J.	0	-.01	0	+.02	+.02	+.03	+.02
CO	0	0	+.01	0	+.01	+.01	+.01
SS	-.02	-.01				-.02	-.03
B.J.	0	+.01	+.01	+.01	0	+.01	0
	+.05	+.06	-.11	-.14	+.27	-.04	-.09

Joint	4A			4B			
Mem	4A3	4A5	A4B	B4A	B4C	4B5	4B3
$\frac{K}{\Sigma K}$ st.	.242	.245				.255	.258
$\frac{K}{\Sigma K}$ j	.206	.141	.653	.415	.355	.090	.140
Mf			-.67	-.67	+1.03		
Bj	+.14	+.09	+.44	-.15	-.13	-.03	-.05
CO	+.04	+.06	-.08	+.22	-.06	-.02	-.01
SS	-.05	-.05				-.06	-.06
B.J.	+.02	+.01	+.05	-.01	0	0	0
CO	0	0	0	+.03	+.01	0	0
S.S.	-.01	0				-.01	-.01
B.J.	0	0	+.01	-.01	-.01	0	0
	+.14	+.11	-.25	-.59	+.84	-.12	-.13

Joint	3A			3B			
Mem	3A4	3A2	A3B	BA3	B3C	3B2	3B4
$\frac{K}{\Sigma K}$ st.	.242	.243				.257	.258
$\frac{K}{\Sigma K}$ j	.161	.329	.510	.348	.297	.238	.117
Mf			-.51	-.51	+.72		
BJ	+.08	+.17	+.26	-.07	-.06	-.05	-.03
CO	+.07	+.04	-.04	+.13	-.03	-.01	-.02
SS	-.05	-.06				-.07	-.06
B.J.	+.01	+.01	+.02	+.02	+.02	+.01	+.01
CO	+.01	0	+.01	+.01	0	+.01	0
	-.01	-.01				-.01	-.02
B.J.	0	0	0	+.01	0	0	0
	+.11	+.15	-.26	-.41	+.65	-.12	-.12

Joint	2A			2B			
Memb	2A1	2A3	A2B	B2A	B2C	2B3	2B1
$\frac{K}{\Sigma K}$ st.	.246	.243				.257	.254
$\frac{K}{\Sigma K}$ jo.	.368	.248	.384	.283	.242	.194	.281
Mf			+2.55	+2.55	−3.02		
B'J	−.94	−.63	−.98	+.13	+.12	+.09	+.13
C.O.	−.37	−.42	+.06	−.49	+.06	+.05	+.05
SS	+.54	+.46				+.49	+.55
B.J.	−.10	−.07	−.10	−.20	−.17	−.14	−.20
C.O.	−.03	−.04	−.10	−.05	−.08	−.03	−.12
SS	+.22	+.13				+.14	+.23
BJ.	−.07	−.04	−.07	−.03	−.02	−.02	−.02
CO	−.04	−.01	−.01	−.03	−.01	0	−.02
SS	+.08	+.03				+.03	+.08
B.J.	−.02	−.01	−.02	−.02	−.01	−.01	−.01
CO	−.01	0	−.01	−.01	0	0	−.01
SS	+.03	0				+.01	+.03
B.J.	0	0	−.01	−.01	0	0	−.01
CO							
SS ok							
	−.71	−.60	+1.31	+1.84	−3.13	+.61	+.68 ?

Joint	1A			1B			
Memb.	1A2	1A0	A1B	B1A	B1C	1B6	1B2
$\frac{K}{\Sigma K}$ Sto.	.246	.246				.254	.254
$\frac{K}{\Sigma K}$ Joint	.293	.401	.306	.237	.202	.326	.235
Mf			+2.55	+2.55	−3.02		
BJ.	−.75	−1.02	−.78	+.11	+.10	+.15	+.11
C.O.	−.47	−.51	+.05	−.39	+.05	+.07	+.06
SS	+.54	+.64				+.67	+.55
B.J.	−.07	−.10	−.08	−.24	−.20	−.33	−.24
C.O.	−.05	−.05	−.12	−.04	−.10	−.16	−.10
S.S.	+.22	+.31				+.33	+.23
BJ.	−.09	−.12	−.10	−.04	−.03	−.05	−.04
CO	−.03	−.06	−.02	−.05	−.01	−.02	−.01
SS.	+.08	+.12				+.13	+.08
B.J.	−.02	−.04	−.03	−.03	−.02	−.04	−.03
Co	−.01	−.02	−.01	−.01	−.01	−.02	0
SS	+.03	+.06				+.06	+.03
B.J	−.01	−.02	−.02	−.01	−.01	−.02	−.01
CO	0	−.01	0	−.01	0	−.04	0
SS	+.01	+.03				+.03	+.01
B.J	−.01	−.01	−.01		0	−.01	−.01
ok	−.63	−.80	+1.43	+1.84	−3.25	+.78	+.63

Joint	6A			6B			
Mem	6A5		A6B	B6A	B6C		6B5
$\frac{K}{\Sigma K}$ St.	.245						.255
$\frac{K}{\Sigma K}$ j	.125		.875	.473	.456		.071
Mf			-.55	-.55	+1.15		
BJ	+.07		+.48	-.28	-.28		-.04
CO	+.03		-.14	+.24	-.14		-.01
SS	-.02						-.03
B.J.	+.02		+.11	-.03	-.03		0
CO	0		-.02	+.06	-.01		0
S.S.	-.01						-.01
B.J.	0		+.03	-.02	-.02		0
	+.09		-.09	-.58	+.67		-.09

Joint	5A			5B			
Mem	5A6	5A4	A50	B5A	B5C	5B4	5B6
$\frac{K}{\Sigma K}$ St.	.245	.245				.255	.255
$\frac{K}{\Sigma K}$ j	.080	.165	.755	.455	.390	.100	.055
Mf			-.80	-.80	+1.20		
BJ	+.06	+.13	+.61	-.18	-.16	-.04	-.02
CO	+.03	+.04	-.09	+.32	-.08	-.01	-.02
SS	-.02	-.05				-.06	-.03
B.J.	+.01	+.01	+.07	-.05	-.05	-.01	-.01
CO	+.01	0	-.03	+.04	-.02	0	0
SS	0	0				0	-.01
BJ	0	0	+.02	-.01			
	+.09	+.13	-.22	-.68	+.89	-.12	-.09