PICK UP 5

MOMENT MAXIMUM

	L	CASE	M	Q	N
= = MEMBER 9 (3 - 7) C = =					
ITAN	0.000	(20)	36.911	-7.723	-101.172
JTAN	9.635	(21)	43.393	9.354	-103.112
= = MEMBER 10 (4 - 8) C = =					
ITAN	0.000	(20)	31.258	-6.409	-1.453
JTAN	9.635	(21)	37.518	8.231	-42.596

MOMENT MINIMUM

	L	CASE	M	Q	N
= = MEMBER 9 (3 - 7) C = =					
ITAN	0.000	(21)	-46.737	9.354	-87.696
JTAN	9.635	(20)	-37.499	-7.723	-116.588
= = MEMBER 10 (4 - 8) C = =					
ITAN	0.000	(21)	-41.790	8.231	-27.180
JTAN	9.635	(20)	-30.489	-6.409	-16.869

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PICK UP 5

SHEAR MAXIMUM

= = MEMBER 1 (1 - 2) G = =

	L	CASE	M	Q	N
ITAN	0.000	(20)	-27.458	19.428	-.773
1	.400	(20)	-19.846	18.592	-.773
2	.900	(20)	-10.875	17.232	-.773
3	1.100	(20)	-7.492	16.590	-.773
4	1.300	(20)	-4.243	15.892	-.773
5	3.250	(20)	18.115	6.152	-.773
6	3.650	(20)	20.082	3.720	-.773
7	3.650	(20)	20.082	1.720	-.773
8	5.200	(20)	11.544	-11.927	-.773
9	5.400	(20)	9.027	-13.215	-.773
10	5.600	(20)	6.258	-14.349	-.773
11	6.100	(20)	-1.487	-16.508	-.773
JTAN	6.500	(20)	-8.317	-17.541	-.773

= = MEMBER 2 (2 - 3) G = =

	L	CASE	M	Q	N
ITAN	0.000	(20)	-45.412	33.339	-1.429
1	.400	(20)	-32.262	32.306	-1.429
2	.900	(20)	-16.609	30.147	-1.429
3	1.100	(20)	-10.690	29.013	-1.429
4	1.300	(20)	-5.014	27.725	-1.429
5	3.250	(20)	32.384	8.434	-1.429
6	5.200	(20)	27.278	-11.633	-1.429
7	5.400	(20)	24.820	-12.922	-1.429
8	5.600	(20)	22.120	-14.057	-1.429
9	6.100	(20)	14.511	-16.217	-1.429
JTAN	6.500	(20)	7.797	-17.250	-1.429

= = MEMBER 3 (3 - 4) G = =

	L	CASE	M	Q	N
ITAN	0.000	(20)	-39.234	25.482	-1.371
1	.400	(20)	-28.855	25.365	-1.371
2	.900	(20)	-16.602	23.576	-1.371
3	1.100	(20)	-11.969	22.736	-1.371
4	1.300	(20)	-7.512	21.826	-1.371
5	1.650	(20)	-.164	20.276	-1.371
6	1.650	(20)	-.164	19.556	-1.371
7	2.250	(20)	10.975	17.534	-1.371
8	3.200	(20)	26.118	14.450	-1.371
9	3.400	(20)	28.950	13.881	-1.371
10	3.600	(20)	31.672	13.341	-1.371
11	4.000	(20)	38.028	12.113	-1.371
JTAN	4.500	(20)	42.698	11.257	-1.371

SHEAR MINIMUM

= = MEMBER 1 (1 - 2) G = =

	L	CASE	M	J	N
ITAN	0.000	(21)	42.821	1.813	-2.585
1	.400	(21)	43.386	.977	-2.585
2	.900	(21)	43.549	-.383	-2.585
3	1.100	(21)	43.409	-1.025	-2.585
4	1.300	(21)	43.135	-1.723	-2.585
5	3.250	(21)	31.143	-11.464	-2.585
6	3.650	(21)	26.064	-13.896	-2.585
7	3.650	(21)	26.064	-15.896	-2.585
8	5.200	(21)	-9.779	-29.542	-2.585
9	5.400	(21)	-15.819	-30.831	-2.585
10	5.600	(21)	-22.101	-31.965	-2.585
11	6.100	(21)	-38.664	-34.124	-2.585
JTAN	6.500	(21)	-52.540	-35.157	-2.585

= = MEMBER 2 (2 - 3) G = =

	L	CASE	M	J	N
ITAN	0.000	(21)	-6.706	21.088	-2.026
1	.400	(21)	1.543	20.055	-2.026
2	.900	(21)	11.071	17.896	-2.026
3	1.100	(21)	14.539	16.762	-2.026
4	1.300	(21)	17.765	15.474	-2.026
5	3.250	(21)	31.274	-3.817	-2.026
6	5.200	(21)	2.279	-23.884	-2.026
7	5.400	(21)	-2.629	-25.173	-2.026
8	5.600	(21)	-7.780	-26.308	-2.026
9	6.100	(21)	-21.514	-28.468	-2.026
JTAN	6.500	(21)	-33.128	-29.501	-2.026

= = MEMBER 3 (3 - 4) G = =

	L	CASE	M	J	N
ITAN	0.000	(21)	3.489	.755	-.451
1	.400	(21)	3.577	-.362	-.451
2	.900	(21)	2.967	-2.151	-.451
3	1.100	(21)	2.454	-2.991	-.451
4	1.300	(21)	1.766	-3.901	-.451
5	1.650	(21)	.109	-5.451	-.451
6	1.650	(21)	.109	-6.171	-.451
7	2.250	(21)	-4.187	-8.193	-.451
8	3.200	(21)	-13.486	-11.277	-.451
9	3.400	(21)	-15.798	-11.845	-.451
10	3.600	(21)	-18.222	-12.386	-.451
11	4.100	(21)	-24.729	-13.614	-.451
JTAN	4.500	(21)	-30.350	-14.470	-.451

PICK UP 5

SHEAR MAXIMUM

	L	CASE	M	Q	N
= = MEMBER 4 (5 - 6) G = =					
ITAN	0.000	(20)	-38.566	21.245	0.000
1	.400	(20)	-30.425	19.465	0.000
2	1.600	(20)	-10.271	14.125	0.000
3	3.250	(20)	6.977	6.782	0.000
4	4.900	(20)	12.110	-.560	0.000
5	6.100	(20)	8.233	-5.900	0.000
JTAN	6.500	(20)	5.517	-7.680	0.000
= = MEMBER 5 (6 - 7) G = =					
ITAN	0.000	(20)	-33.488	19.692	0.000
1	.400	(20)	-25.968	17.912	0.000
2	1.600	(20)	-7.677	12.572	0.000
3	3.250	(20)	7.009	5.229	0.000
4	4.900	(20)	9.580	-2.113	0.000
5	6.100	(20)	3.840	-7.453	0.000
JTAN	6.500	(20)	.503	-9.233	0.000
= = MEMBER 6 (7 - 8) G = =					
ITAN	0.000	(20)	-36.996	25.009	0.000
1	.400	(20)	-27.348	23.229	0.000
2	1.600	(20)	-2.677	17.889	0.000
3	2.250	(20)	8.011	14.997	0.000
4	2.900	(20)	16.818	12.104	0.000
5	4.100	(20)	28.139	6.764	0.000
JTAN	4.500	(20)	30.489	4.984	0.000
= = MEMBER 7 (1 - 5) C = =					
ITAN	0.000	(21)	-26.441	5.195	-29.173
JTAN	9.635	(21)	23.611	5.195	-44.589
= = MEMBER 8 (2 - 6) C = =					
ITAN	0.000	(21)	-40.654	8.339	-127.225
JTAN	9.635	(21)	39.697	8.339	-142.641
= = MEMBER 9 (3 - 7) C = =					
ITAN	0.000	(21)	-46.737	9.354	-87.696
JTAN	9.635	(21)	43.393	9.354	-103.112
= = MEMBER 10 (4 - 8) C = =					
ITAN	0.000	(21)	-41.790	8.231	-27.180
JTAN	9.635	(21)	37.518	8.231	-42.596

SHEAR MINIMUM

	L	CASE	M	Q	N
= = MEMBER 4 (5 - 6) G = =					
ITAN	0.000	(21)	23.611	4.322	0.000
1	.400	(21)	24.984	2.542	0.000
2	1.600	(21)	24.830	-2.798	0.000
3	3.250	(21)	14.156	-10.141	0.000
4	4.900	(21)	-8.633	-17.483	0.000
5	6.100	(21)	-32.817	-22.823	0.000
JTAN	6.500	(21)	-42.302	-24.603	0.000
= = MEMBER 5 (6 - 7) G = =					
ITAN	0.000	(21)	-2.605	10.586	0.000
1	.400	(21)	1.273	8.806	0.000
2	1.600	(21)	8.636	3.466	0.000
3	3.250	(21)	8.298	-3.877	0.000
4	4.900	(21)	-4.156	-11.219	0.000
5	6.100	(21)	-20.823	-16.559	0.000
JTAN	6.500	(21)	-27.803	-18.339	0.000
= = MEMBER 6 (7 - 8) G = =					
ITAN	0.000	(21)	15.590	-1.789	0.000
1	.400	(21)	14.518	-3.569	0.000
2	1.600	(21)	7.031	-8.909	0.000
3	2.250	(21)	.300	-11.802	0.000
4	2.900	(21)	-8.311	-14.694	0.000
5	4.100	(21)	-29.149	-20.034	0.000
JTAN	4.500	(21)	-37.518	-21.814	0.000
= = MEMBER 7 (1 - 5) C = =					
ITAN	0.000	(20)	43.838	-8.553	-46.788
JTAN	9.635	(20)	-38.566	-8.553	-62.204
= = MEMBER 8 (2 - 6) C = =					
ITAN	0.000	(20)	42.275	-8.436	-121.860
JTAN	9.635	(20)	-39.005	-8.436	-137.276
= = MEMBER 9 (3 - 7) C = =					
ITAN	0.000	(20)	36.911	-7.723	-101.172
JTAN	9.635	(20)	-37.499	-7.723	-116.588
= = MEMBER 10 (4 - 8) C = =					
ITAN	0.000	(20)	31.258	-6.409	-1.453
JTAN	9.635	(20)	-30.489	-6.409	-16.869

PICK UP 5

AXIAL MAXIMUM

	L	CASE	M	Q	N
= = MEMBER 4 (5 - 6) G = =					
ITAN	0.000	(20)	-38.566	21.245	0.000
1	.400	(20)	-30.425	19.465	0.000
2	1.600	(20)	-10.271	14.125	0.000
3	3.250	(20)	6.977	6.782	0.000
4	4.900	(20)	12.110	-.560	0.000
5	6.100	(20)	8.233	-5.900	0.000
JTAN	6.500	(20)	5.517	-7.680	0.000
= = MEMBER 5 (6 - 7) G = =					
ITAN	0.000	(20)	-33.488	19.692	0.000
1	.400	(20)	-25.968	17.912	0.000
2	1.600	(20)	-7.677	12.572	0.000
3	3.250	(20)	7.009	5.229	0.000
4	4.900	(20)	9.580	-2.113	0.000
5	6.100	(20)	3.840	-7.453	0.000
JTAN	6.500	(20)	.503	-9.233	0.000
= = MEMBER 6 (7 - 8) G = =					
ITAN	0.000	(20)	-36.996	25.009	0.000
1	.400	(20)	-27.348	23.229	0.000
2	1.600	(20)	-2.677	17.889	0.000
3	2.250	(20)	8.011	14.997	0.000
4	2.900	(20)	16.818	12.104	0.000
5	4.100	(20)	28.139	6.764	0.000
JTAN	4.500	(20)	30.489	4.984	0.000
= = MEMBER 7 (1 - 5) C = =					
ITAN	0.000	(21)	-26.441	5.195	-29.173
JTAN	9.635	(21)	23.611	5.195	-44.589
= = MEMBER 8 (2 - 6) C = =					
ITAN	0.000	(20)	42.275	-8.436	-121.860
JTAN	9.635	(20)	-39.005	-8.436	-137.276
= = MEMBER 9 (3 - 7) C = =					
ITAN	0.000	(21)	-46.737	9.354	-87.696
JTAN	9.635	(21)	43.393	9.354	-103.112
= = MEMBER 10 (4 - 8) C = =					
ITAN	0.000	(20)	31.258	-6.409	-1.453
JTAN	9.635	(20)	-30.489	-6.409	-16.869

AXIAL MINIMUM

	L	CASE	M	Q	N
= = MEMBER 4 (5 - 6) G = =					
ITAN	0.000	(20)	-38.566	21.245	0.000
1	.400	(20)	-30.425	19.465	0.000
2	1.600	(20)	-10.271	14.125	0.000
3	3.250	(20)	6.977	6.782	0.000
4	4.900	(20)	12.110	-.560	0.000
5	6.100	(20)	8.233	-5.900	0.000
JTAN	6.500	(20)	5.517	-7.680	0.000
= = MEMBER 5 (6 - 7) G = =					
ITAN	0.000	(20)	-33.488	19.692	0.000
1	.400	(20)	-25.968	17.912	0.000
2	1.600	(20)	-7.677	12.572	0.000
3	3.250	(20)	7.009	5.229	0.000
4	4.900	(20)	9.580	-2.113	0.000
5	6.100	(20)	3.840	-7.453	0.000
JTAN	6.500	(20)	.503	-9.233	0.000
= = MEMBER 6 (7 - 8) G = =					
ITAN	0.000	(20)	-36.996	25.009	0.000
1	.400	(20)	-27.348	23.229	0.000
2	1.600	(20)	-2.677	17.889	0.000
3	2.250	(20)	8.011	14.997	0.000
4	2.900	(20)	16.818	12.104	0.000
5	4.100	(20)	28.139	6.764	0.000
JTAN	4.500	(20)	30.489	4.984	0.000
= = MEMBER 7 (1 - 5) C = =					
ITAN	0.000	(20)	43.838	-8.553	-46.788
JTAN	9.635	(20)	-38.566	-8.553	-62.204
= = MEMBER 8 (2 - 6) C = =					
ITAN	0.000	(21)	-40.654	8.339	-127.225
JTAN	9.635	(21)	39.697	8.339	-142.641
= = MEMBER 9 (3 - 7) C = =					
ITAN	0.000	(20)	36.911	-7.723	-101.172
JTAN	9.635	(20)	-37.499	-7.723	-116.588
= = MEMBER 10 (4 - 8) C = =					
ITAN	0.000	(21)	-41.790	8.231	-27.180
JTAN	9.635	(21)	37.518	8.231	-42.596

PICK UP 5

AXIAL MAXIMUM

= = MEMBER 1 (1 - 2) G = =

	L	CASE	M	Q	N
ITAN	0.000	(20)	-27.458	19.428	-.773
1	.400	(20)	-19.846	18.592	-.773
2	.900	(20)	-10.875	17.232	-.773
3	1.100	(20)	-7.492	16.590	-.773
4	1.300	(20)	-4.243	15.892	-.773
5	3.250	(20)	18.115	6.152	-.773
6	3.650	(20)	20.082	3.720	-.773
7	3.650	(20)	20.082	1.720	-.773
8	5.200	(20)	11.544	-11.927	-.773
9	5.400	(20)	9.027	-13.215	-.773
10	5.600	(20)	6.258	-14.349	-.773
11	6.100	(20)	-1.487	-16.508	-.773
JTAN	6.500	(20)	-8.317	-17.541	-.773

= = MEMBER 2 (2 - 3) G = =

	L	CASE	M	Q	N
ITAN	0.000	(20)	-45.412	33.339	-1.429
1	.400	(20)	-32.262	32.306	-1.429
2	.900	(20)	-16.609	30.147	-1.429
3	1.100	(20)	-10.690	29.013	-1.429
4	1.300	(20)	-5.014	27.725	-1.429
5	3.250	(20)	32.384	8.434	-1.429
6	5.200	(20)	27.278	-11.633	-1.429
7	5.400	(20)	24.820	-12.922	-1.429
8	5.600	(20)	22.120	-14.057	-1.429
9	6.100	(20)	14.511	-16.217	-1.429
JTAN	6.500	(20)	7.797	-17.250	-1.429

= = MEMBER 3 (3 - 4) G = =

	L	CASE	M	Q	N
ITAN	0.000	(21)	3.489	.755	-.451
1	.400	(21)	3.577	-.362	-.451
2	.900	(21)	2.967	-2.151	-.451
3	1.100	(21)	2.454	-2.991	-.451
4	1.300	(21)	1.756	-3.901	-.451
5	1.650	(21)	.109	-5.451	-.451
6	1.650	(21)	.109	-6.171	-.451
7	2.250	(21)	-4.137	-8.193	-.451
8	3.200	(21)	-13.496	-11.277	-.451
9	3.400	(21)	-15.798	-11.845	-.451
10	3.600	(21)	-18.222	-12.386	-.451
11	4.100	(21)	-24.729	-13.614	-.451
JTAN	4.500	(21)	-30.350	-14.470	-.451

AXIAL MINIMUM

= = MEMBER 1 (1 - 2) G = =

	L	CASE	M	Q	N
ITAN	0.000	(21)	42.821	1.813	-2.585
1	.400	(21)	43.386	.977	-2.585
2	.900	(21)	43.549	-.383	-2.585
3	1.100	(21)	43.409	-1.025	-2.585
4	1.300	(21)	43.135	-1.723	-2.585
5	3.250	(21)	31.143	-11.464	-2.585
6	3.650	(21)	26.064	-13.896	-2.585
7	3.650	(21)	26.064	-15.896	-2.585
8	5.200	(21)	-9.779	-29.542	-2.585
9	5.400	(21)	-15.819	-30.831	-2.585
10	5.600	(21)	-22.101	-31.965	-2.585
11	6.100	(21)	-38.664	-34.124	-2.585
JTAN	6.500	(21)	-52.540	-35.157	-2.585

= = MEMBER 2 (2 - 3) G = =

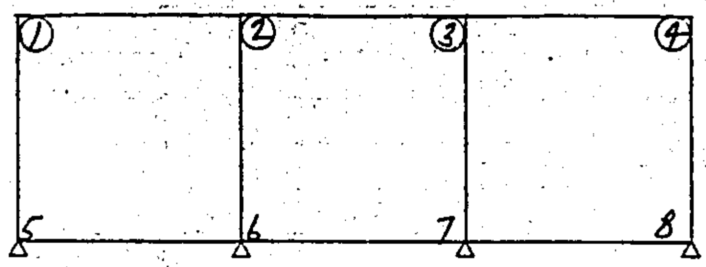
	L	CASE	M	Q	N
ITAN	0.000	(21)	-6.706	21.088	-2.026
1	.400	(21)	1.543	20.055	-2.026
2	.900	(21)	11.071	17.896	-2.026
3	1.100	(21)	14.539	16.762	-2.026
4	1.300	(21)	17.765	15.474	-2.026
5	3.250	(21)	31.274	-3.817	-2.026
6	5.200	(21)	2.279	-23.884	-2.026
7	5.400	(21)	-2.629	-25.173	-2.026
8	5.600	(21)	-7.780	-26.308	-2.026
9	6.100	(21)	-21.514	-28.468	-2.026
JTAN	6.500	(21)	-33.128	-29.501	-2.026

= = MEMBER 3 (3 - 4) G = =

	L	CASE	M	Q	N
ITAN	0.000	(20)	-39.234	26.482	-1.371
1	.400	(20)	-28.855	25.365	-1.371
2	.900	(20)	-16.602	23.576	-1.371
3	1.100	(20)	-11.959	22.736	-1.371
4	1.300	(20)	-7.512	21.826	-1.371
5	1.650	(20)	-.164	20.276	-1.371
6	1.650	(20)	-.164	19.556	-1.371
7	2.250	(20)	10.975	17.534	-1.371
8	3.200	(20)	26.118	14.450	-1.371
9	3.400	(20)	28.950	13.881	-1.371
10	3.600	(20)	31.672	13.341	-1.371
11	4.100	(20)	38.028	12.113	-1.371
JTAN	4.500	(20)	42.698	11.257	-1.371

[7] Calculation of upper beam

(1) Stress calculation of upper beam



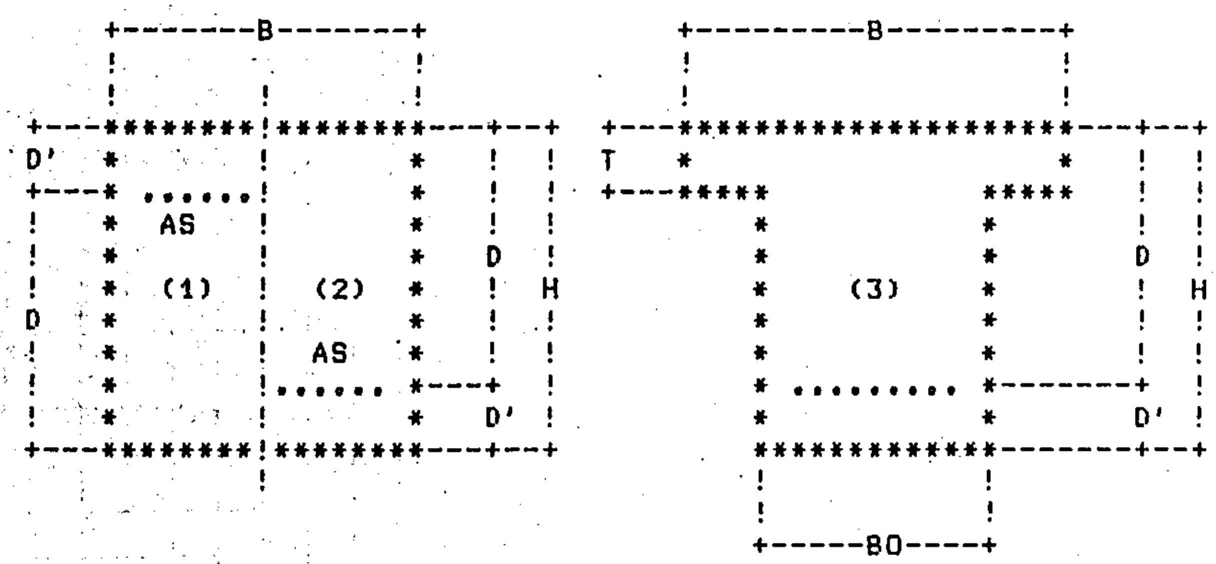
a) Bending moment

		—	①~②						②~③					
		—	1 point		1~2 span		2 point		2 point		2~3 span		3 point	
pick up 1	Top	—	—	—	—		-30.43	1	-26.06	1	—		-12.67	1
pick up 2	Top	—	-18.31	14	—		-37.92	9	-41.07	9	—		-22.09	15
	Bottom	—	28.93	15	29.05	15	—	—	1.03	15	82.76	9	30.60	9

		③~④					
		3 point		3~4 span		4 point	
pick up 1	Top	-17.87	1	—		—	—
pick up 2	Top	-63.56	9	—		-20.23	15
	Bottom	2.39	15	—	—	28.47	14

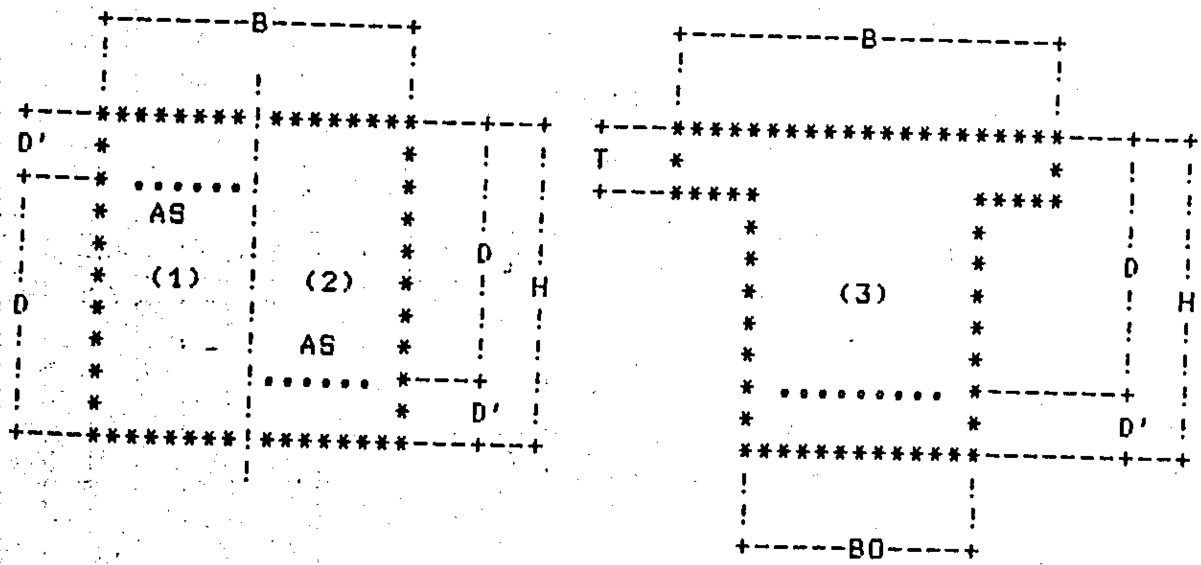
(d) stress Calculation

NO.		1~2 span (Bottom) 1	2 point (TOP) 2	2~3 span (Bottom) 3	3 point (TOP) 4
		case 15	case 9	case 9	case 9
M	[T*M]	29.05	41.07	82.76	63.56
S	[T]				
B	[CM]	215.00	80.00	215.00	80.00
H	[CM]	110.00	140.00	110.00	140.00
D'	[CM]	5.80	9.10	9.00	9.10
D	[CM]	104.20	130.90	101.00	130.90
T	[CM]	28.00		28.00	
BO	[CM]	80.00		80.00	
		7.00-D29	7.00-D29	9.00-D29	7.00-D29
AS	[CM**2]	44.97	44.97	57.82	44.97
P		0.002007	0.004294	0.002662	0.004294
K		0.217	0.300	0.245	0.300
J		0.928	0.900	0.918	0.900
1/LC		9.93	7.40	8.87	7.40
1/LS		537.07	258.78	409.06	258.78
SIG-C	[KG/CM**2]	12.36	22.18	33.48	34.32
SIG-S	[KG/CM**2]	668.35	775.31	1543.58	1199.88
TAU	[KG/CM**2]				
SIG-CA	[KG/CM**2]	90.00	90.00	90.00	90.00
SIG-SA	[KG/CM**2]	1800.00	1800.00	1800.00	1800.00
TAU-A	[KG/CM**2]				
		(2)	(2)	(2)	(2)



crock

NO.	2point 5 (TOP)
	case 1
M [T*M]	30.43
S [T]	
B [CM]	80.00
H [CM]	140.00
D' [CM]	9.10
D [CM]	130.90
T [CM]	
BO [CM]	
	7.00-D29
AS [CM**2]	44.97
P	0.004294
K	0.300
J	0.900
1/LC	7.40
1/LS	258.78
SIG-C [KG/CM**2]	16.43
SIG-S [KG/CM**2]	574.45
TAU [KG/CM**2]	
SIG-CA[KG/CM**2]	90.00
SIG-SA[KG/CM**2]	1000.00
TAU-A [KG/CM**2]	
	(2)



(e) Required minimum cross section of re-bars

$$As = \frac{15 \cdot b \cdot d}{\sigma sy}$$

$$As = \frac{15 \times 80 \times 101.0}{3000} = 40.4^{cm^2}$$

(i) At the top of support point

$$As = \frac{M}{\sigma sa \cdot j \cdot d} \times \frac{4}{3} = \frac{63.56 \times 10^5}{1800 \times 0.875 \times 130.9} \times \frac{4}{3}$$

$$= 41.10^{cm^2}$$

Hence $D29 - 7 = 44.97^{cm^2} > 41.10^{cm^2}$

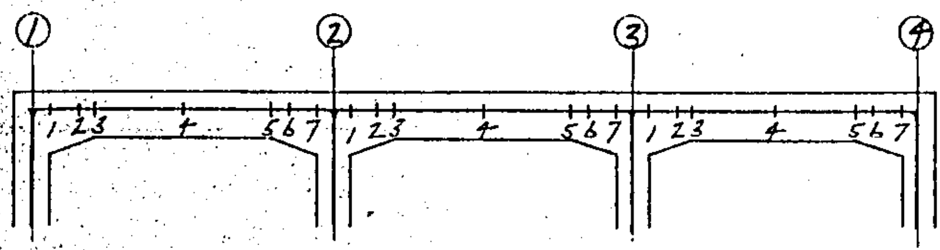
(ii) At the span center point

$$As = \frac{82.76 \times 10^5}{1800 \times 0.875 \times 101.0} \times \frac{4}{3} = 69.37^{cm^2}$$

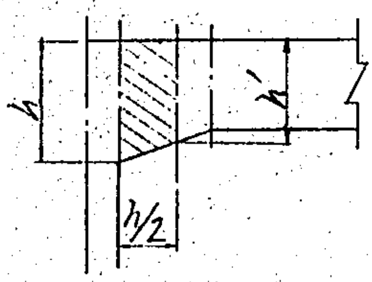
Hence $D29 - 9 = 57.82^{cm^2} < 69.37^{cm^2}$

$> 40.4^{cm^2}$

(2) Shearing stress of upper beam



(i) Summary of shearing stress



For the disign of section of beam end, the value of shearing stress at h/2 point is applied

$$h' = 1.40 - \frac{0.70}{0.90} \times 0.30 = 1.167^{m}$$

correction for shearing stress

$$S = S_0 \pm \frac{M}{d} \cdot (\tan\alpha)$$

where

S_0 : Shearing stress caused by bending (t)

M : Bending moment (t·m)

d : Effective height (m)

α : An angle of elevation of the member

(1) Shearing stress

$$\tau_b = \frac{S}{b \cdot d}$$

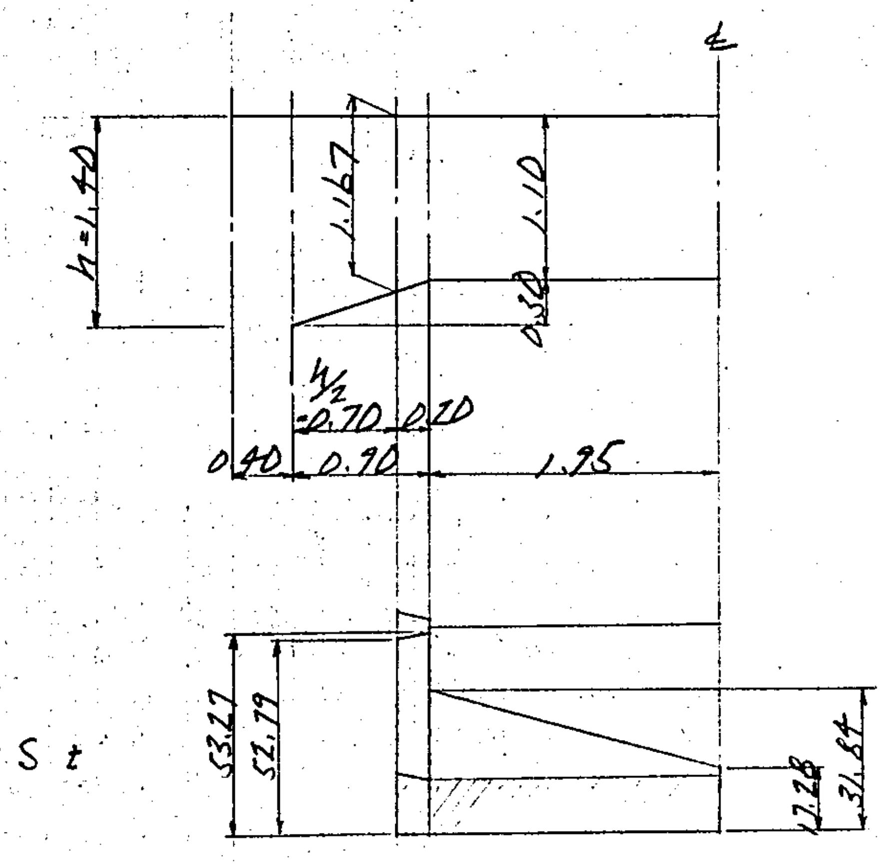
τ_b: shearing stress (kg/cm^2)
b : width of member (cm)
d : Effective height of member (cm)
S : shearing force (t)

member	case	No.	S_0 (t)	M (t·m)	tan α	S (t)	b (cm)	d (cm)	τ_b (kg/cm^2)
①~②		2		—	—				
		2'							
		3							
		4							
①~②	9	1	12.48	-3.98	0.30/0.90	11.47	80	130.9	1.09
		2	10.48	4.10	〃	9.21	〃	107.6	1.07
		3	9.78	6.13	〃	7.75	〃	100.9	0.96
		3'	9.78	—	—	9.78	〃	〃	1.21
		4	0.04	—	—	0.04	〃	〃	0
	7		0.04	—	—	0.04	〃	〃	0
		5	18.98	—	—	18.98	〃	〃	2.35
		5'	18.98	-1.97	0.30/0.90	18.33	〃	〃	2.27
		6	21.91	-6.06	〃	20.03	〃	107.6	2.33
		7	30.93	-24.67	〃	24.65	〃	130.9	2.35

member	case	No.	So (t)	M (t·m)	tanα	S (t)	b (cm)	d (cm)	τb (kg/cm²)
②~③	9	1	49.46	-20.41	0.30/0.90	44.26	80	130.9	4.23
		2	40.43	11.17	〃	36.97	〃	107.6	4.29
		3	37.51	18.96	〃	31.25	〃	100.9	3.87
		3'	37.51	—	—	37.51	〃	〃	4.65
		4	17.28	—	—	17.28	〃	〃	2.14
	9		17.28	—	—	17.28	〃	〃	2.14
		5	31.84	—	—	31.84	〃	〃	3.94
		5'	31.84	64.87	0.30/0.90	53.27	〃	〃	6.60
		6	34.76	58.21	〃	52.79	〃	107.6	6.13
		7	43.79	30.60	〃	51.58	〃	130.9	4.93
③~④	9	1	26.21	-52.65	0.30/0.90	12.80	〃	130.9	1.49
		2	23.58	-35.17	〃	12.68	〃	107.6	1.47
		3	22.67	-30.55	〃	12.58	〃	100.9	1.56
		3'	22.67	—	—	22.67	〃	〃	2.81
		4	18.38	—	—	18.38	〃	〃	2.28
	10		17.92	—	—	17.92	〃	〃	2.22
		5	15.24	—	—	15.24	〃	〃	1.89
		5'	15.24	7.06	0.30/0.90	12.91	〃	〃	1.60
		6	14.75	10.06	〃	11.63	〃	107.6	1.35
		7	13.21	19.82	〃	8.16	〃	130.9	0.78

(2) Calculation of diagonal tension re-bars

(i) Shearing stress caused by bending



(4) Calculation of diagonal tension re- bers

(a) Calculation of shearing

$$\Sigma SR = Sc + Sv + Sb$$

where

Sc : Shearing force beared by concrete (t)

Sv : Shearing force beared by sterrup (t)

Sb : Shear beared by turned up bar (t)

$$Sv \geqq Sb$$

(i) Shearing force beared by concrete

$$Sc = 1/2 \cdot \tau c \cdot b \cdot d \qquad (d' = 9.1^{cm})$$

$$Sc_1 = 1/2 \times 3.9 \times 80 \times (116.7 - 9.1) \times 10^{-3}$$
$$= 16.79^{t}$$

$$Sc_2 = 1/2 \times 3.9 \times 80 \times (110.0 - 9.1) \times 10^{-3}$$
$$= 15.74^{t}$$

(ii) Shearing force beared by sterrup

$$S_v = \frac{A_v \cdot \sigma_{sa} \cdot d}{1.15 \cdot s}$$

Arrange stirrups D13-2 sets in 20.0 cm ctc

$A_v = 1.267 \times 4 = 5.07\ cm^2$

$$S_{v1} = \frac{5.07 \times 1800 \times 107.6}{1.15 \times 20} \times 10^{-3}$$

$= 42.69^t$

$$S_{v2} = \frac{5.07 \times 1800 \times 100.9}{1.15 \times 20} \times 10^{-3}$$

$= 40.04^t$

(iii) Resultant resisting shear

$$\Sigma SR = Sc + Sv + Sb$$

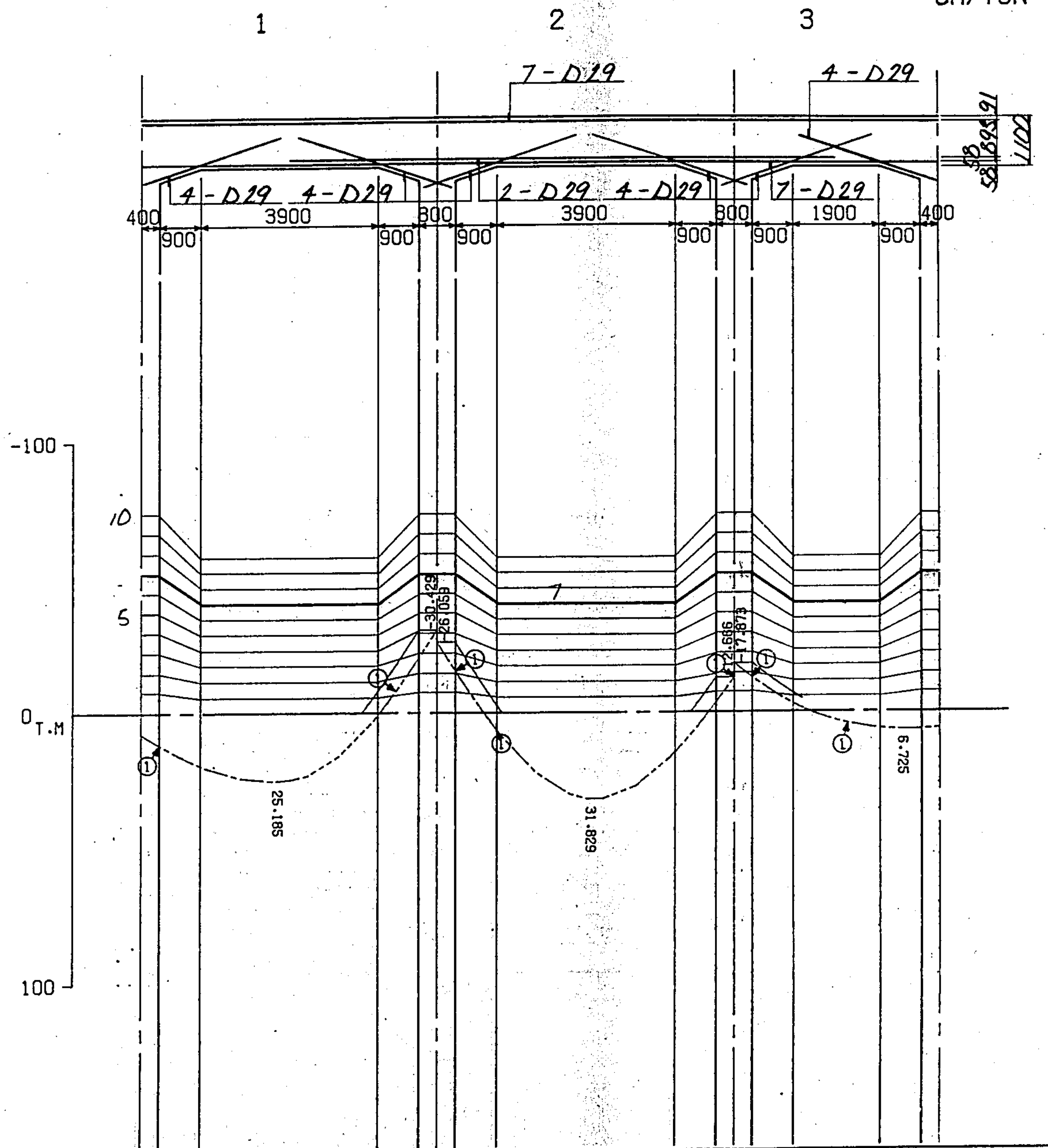
$SR_1 = 16.79 + 42.69 = 59.48^t > 53.27^t$

$SR_2 = 15.74 + 40.04 = 55.78^t > 52.79^t$

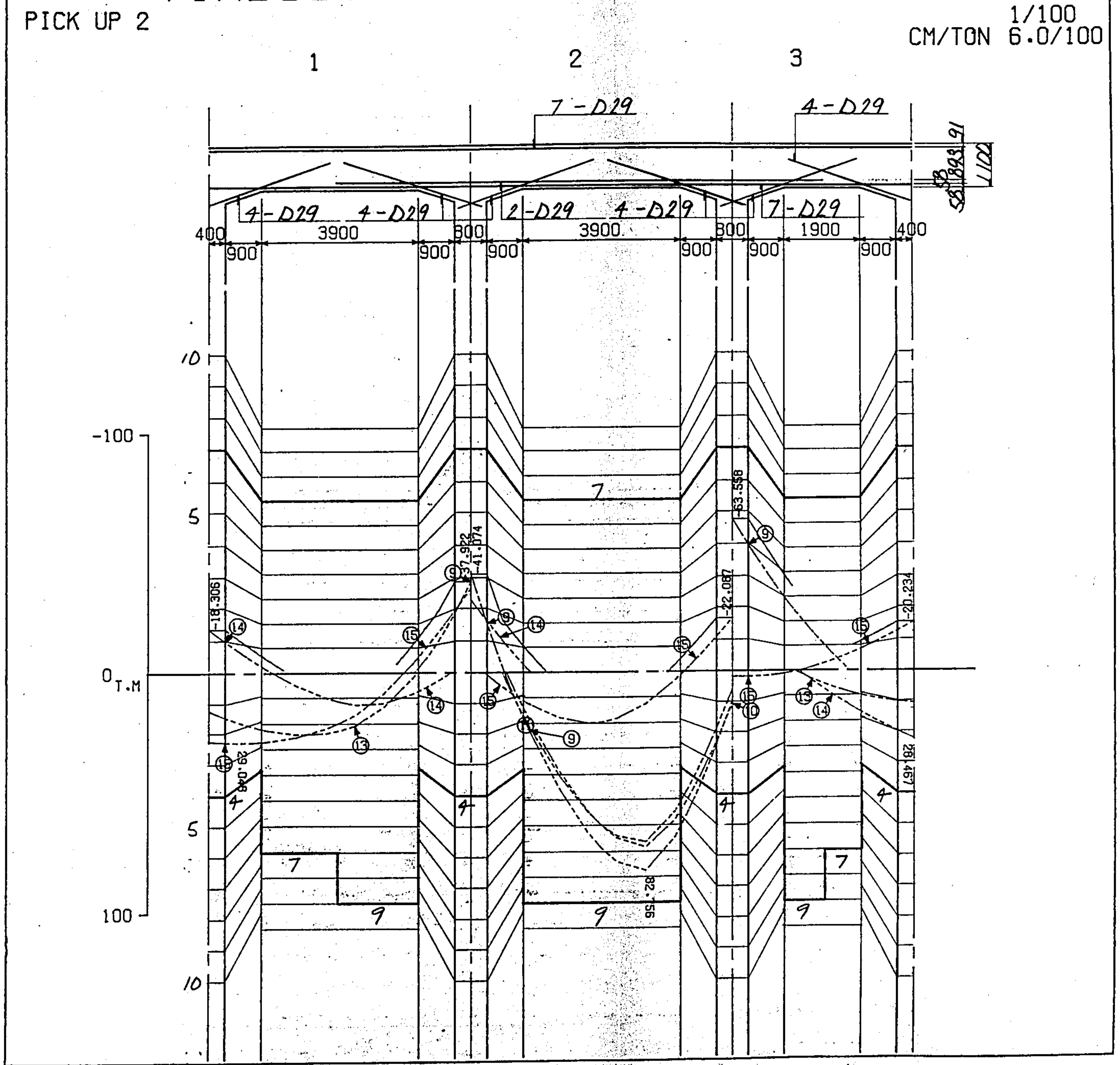
VIADUCT OF DOUBLE TRACK C-2

PICK UP 1

1/100
CM/TON 6.0/100

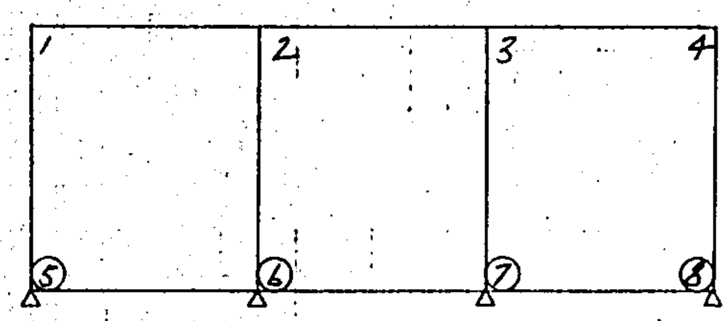


VIADUCT OF DOUBLE TRACK C-2



(8) Calculation of buried beam

(1) Summary of stresses



a) Bending moment and shearing force

		⑤~⑥ member				⑥~⑦ member	
		⑤ point		⑥ point		⑥ point	
Dead load (pick up 1)	TOP	-7.48	1	-18.39	1	-18.05	1
	Bottom	—	—	10.88	1	—	—
(pick up 2)	TOP	-25.71	14	-28.20	15	-26.18	9
	Bottom	17.05	15	—	—	—	—
Shearing (pick up 2)		10.30	9	11.66	15	10.19	9

		⑥~⑦ member		⑦~⑧ member			
		⑦ point		⑦ point		⑧ point	
Dead load (pick up 1)	TOP	-13.65	1	-10.70	1	-3.52	1
	Bottom	7.65	1	—	—	4.42	1
(pick up 2)	TOP	-18.54	15	-24.67	14	-25.01	15
	Bottom	7.30	9	10.39	15	20.33	14
Shearing (pick up 2)		7.48	15	11.93	14	9.80	15

(b) Stress Calculation

NO.	6 point (Top) 1	7 point (Top) 1	7~8 span (Bottom) 2	5~6 span (Bottom) 2
CASE	(15)	(1)	(14)	(1)
M [T*M]	28.20	18.39	20.33	10.88
S [T]				
B [CM]	70.00	70.00	70.00	70.00
H [CM]	90.00	90.00	90.00	90.00
D [CM]	79.30	79.30	72.10	72.10
D' [CM]	10.70	10.70	17.90	17.90
AS (DIA-HON)	D29- 6.0	D29- 6.0	D29- 6.0	D29- 6.0
[CM**2]	38.54	38.54	38.54	38.54
AS' (DIA-HON)				
[CM**2]				
P	0.006944	0.006944	0.007637	0.007637
P'				
K	0.3640	0.3640	0.3776	0.3776
LC	0.1599	0.1599	0.1650	0.1650
SIG-C [KG/CM**2]	40.06	26.12	33.85	18.12
SIG-S [KG/CM**2]	1050.01	684.74	836.89	447.88
TAU [KG/CM**2]				
SIG-CA [KG/CM**2]	90.00	90.00	90.00	90.00
SIG-SA [KG/CM**2]	1800.00	1400.00	1800.00	1400.00
TAU-A [KG/CM**2]				

```
          +-------B-------+
          !               !
          !               !
+---+---***************
!  D'   *               *
!  +---*  ...........   *
!       *       AS'     *
!       *               *
D       *               *
!       *               *
!       *               *
!       *       AS      *
+------*  ...........   *
        *               *
        ***************
```

b.) Allowable stress for upper beam, safe against cracking

(i) At the support point

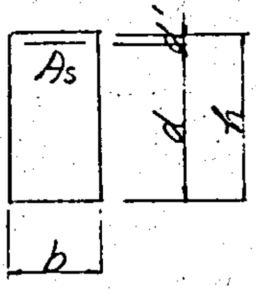
Dead load	M_d =	17.87 t·m	(case 1)
Train load + Impact	M_L =	35.33 "	(case 2)
	ΣM =	53.20 t·m	

$$\alpha = \frac{35.33}{53.20} = 0.66 > 0.25$$

Hence $\sigma_{sa} = 1{,}000\ kg/cm^2$

(c) Cross section used for stress calculation

(i) Cross section at the support point



$b = 80^{cm}$

$h = 140$ "

$d' = 2.5 + 1.9 + 3.2 + 1.5 = 9.1^{cm}$

$d = 140 - 9.1 = 130.9^{cm}$

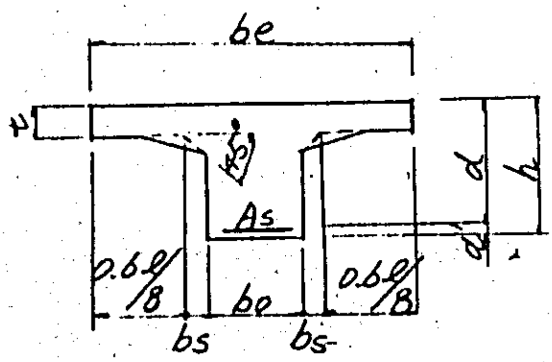
Cross section at the span center point

(ii) Effective width of T-beam at compression fibre

$$be = b_0 + 2\left(bs + \frac{0.6}{8}\cdot \ell\right)$$

$$be_1 = 0.80 + 2 \times \left(0.15 + \frac{0.60}{8} \times 7.00\right)$$

$$= 2.15^{m}$$



$b_0 = 70^{cm}$

$h = 110$ "

$d' = 3.0 + 1.3 + 1.5 + 1.6 = 5.8^{cm}$
$(3.0 + 1.3 + 1.5 + 1.6 + 3.2 = 9.0^{cm})$

$d = 110 - 5.8 = 104.2^{cm}$
$(110 - 9.0) = 101.0^{cm}$

$t = 28^{cm}$

(C) Stress calculation

(i) Top (C-2)

$$As = \frac{28.20 \times 10^5}{1800 \times 0.875 \times 79.3} \times \frac{4}{3} = 22.58 \times \frac{4}{3} = 30.10^{cm^2}$$

$$As = 15 \times 70 \times 79.3 \times 1/3000 = 27.76^{cm^2}$$

Hence D29 - 5 = $32.12^{cm^2} > 27.76^{cm^2}$

(ii) Bottom (C-2)

$$As = \frac{20.33 \times 10^5}{1800 \times 0.875 \times 72.1} \times \frac{4}{3} = 17.90 \times \frac{4}{3} = 23.87^{cm^2}$$

$$As = 15 \times 70 \times 72.1 \times 1/3000 = 25.24^{cm^2}$$

Hence D29 - 4 = $25.70^{cm^2} > 23.87^{cm^2}$

(d) Inside of 1.5H

$A_s = 0.0020 \times 70 \times 15$

$= 2.10^{cm^2} < D13-2 = 2.53^{cm^2}$

$1/4\,h = 1/4 \times 90 = 22.5^{cm}$

$8 \times 25 = 20.0^{cm}$

$13 \times 24 = 31.2$ 〃

(i) Shearing stress beared by concrete

$S_c = 1/2 \cdot \tau_c \cdot b \cdot d$

$= 1/2 \times 3.9 \times 70 \times 72.1 \times 10^{-3} = 9.84^t$

(ii) Shearing force beared by stirrup

$$S = \frac{1800 \times 72.1 \times 2 \times 1.267}{1.15 \times 15 \times 10^3} \times \frac{1}{1.2}$$

$= 19.06 \times 1/1.2 = 15.88^t$

(iii) Total shear

$S = 9.84 + 15.88 = 25.72^t$

(e) Inside of 1.5H

$As = 0.0015 \times 70 \times 20$

$= 2.1^{cm^2} < 2 \times 1.267 = 2.53^{cm^2}$

$1/2\, h = \frac{72.1-15}{2} = 28.6^{cm}$

$15 \times 2.5 = 37.5^{cm}$

$48 \times 1.3 = 62.4$ 〃

(i) Shearing stress beared by concrete

$S = 1/2 \times 3.9 \times 70 \times 72.1 \times 10^{-3} = 9.84^{t}$

(ii) Shearing force beared by stirrup

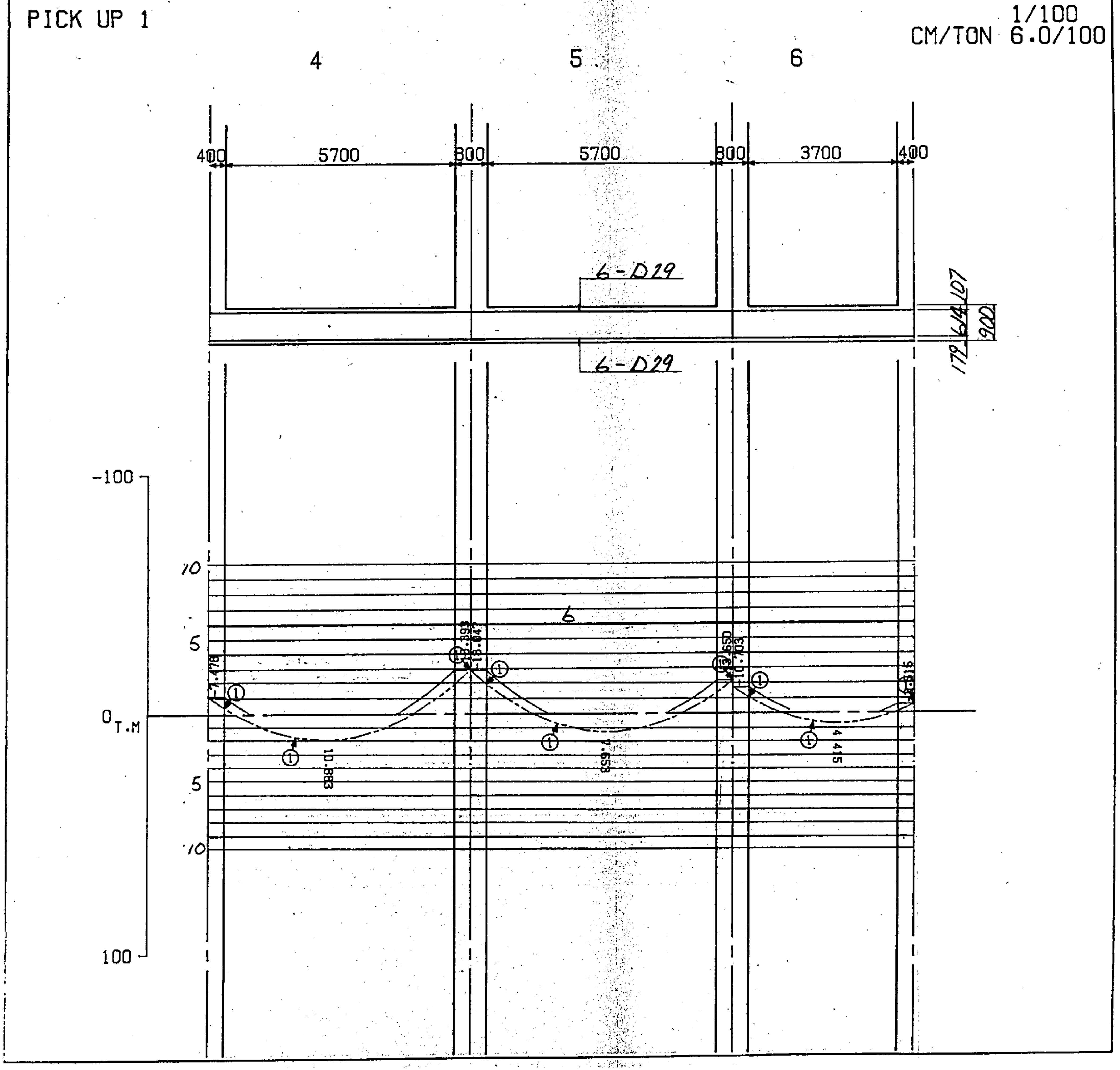
$$S = \frac{1800 \times 72.1 \times 2 \times 1.267}{1.15 \times 20 \times 10^{3}} \times \frac{1}{1.2}$$

$= 14.30 \times 1/1.2 = 11.92^{t}$

(iii) Total shear

$S = 9.84 + 11.92 = 21.76^{t}$

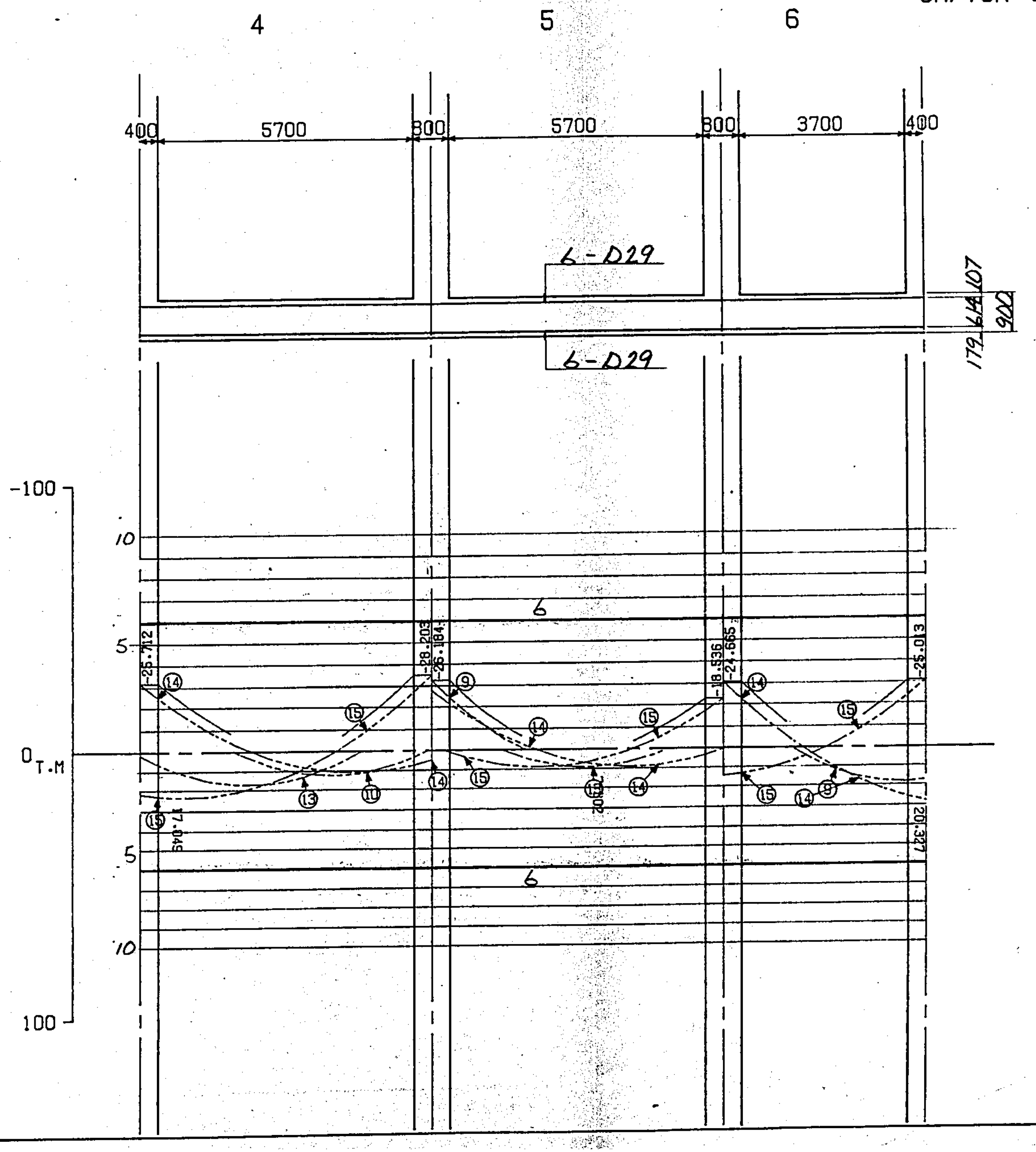
VIADUCT OF DOUBLE TRACK C-2



VIADUCT OF DOUBLE TRACK C-2

PICK UP 2

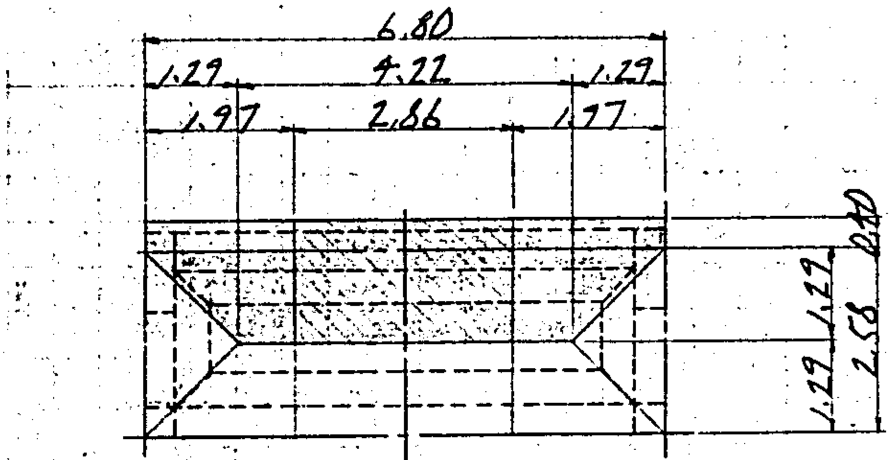
1/100
CM/TON 6.0/100



§7. Calculation of beam

1. Calculation of loads

(1) Dead load



(a) Distributed load (A)

Ballast	1.9×0.481	$= 0.91$	t/m²
Grading concrete	2.35×0.07	$= 0.16$	"
slab	2.5×0.28	$= 0.70$	"
		$wd = 0.86$	t/m²

$\therefore wd_1 = 0.86 \times 1.29 = 1.11$ t/m

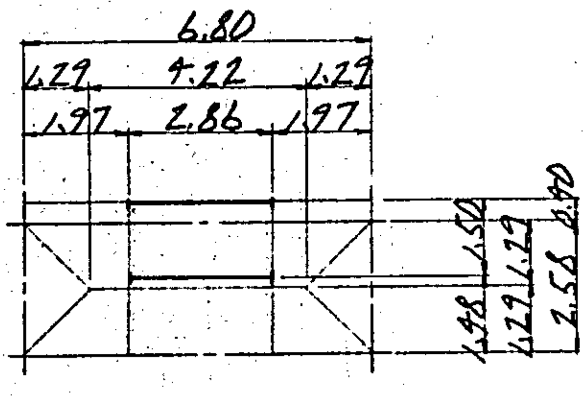
Track weight

$$w_{d2} = 0.45 \times 1/2.86 \times (1.29 + 0.40) = 0.27\ t/m$$

(b) Distributed load (B)

Distributed load	0.86×0.40	=	$0.34\ t/m$
beam	$2.5 \times 0.50 \times 0.72$	=	0.90 "
slab	$2.5 \times 0.15 \times 0.15$	=	0.06 "
slab haunch	$2.5 \times 1/2 \times 0.45 \times 0.15$	=	0.08 "
	w_{d3}	=	$1.38\ t/m$

(2) Train load + Impact



$$l = 6.80^m \longrightarrow i = 0.462$$

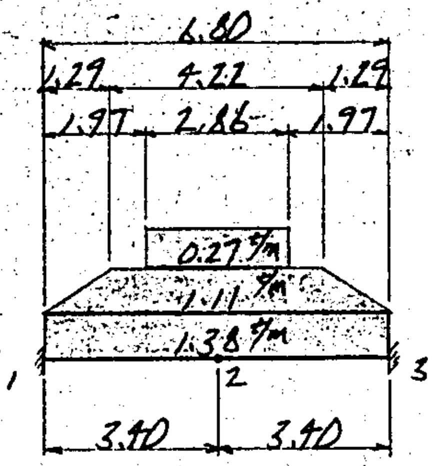
$$w_{l+i} = 16 \times 1/2.86 \times (1 + 0.462) \times 2$$

$$= 16.36\ t/m$$

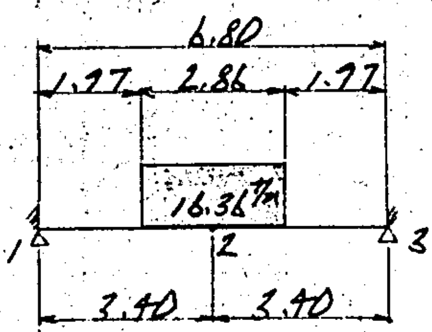
2. Calculation of stress

(1) Loading load

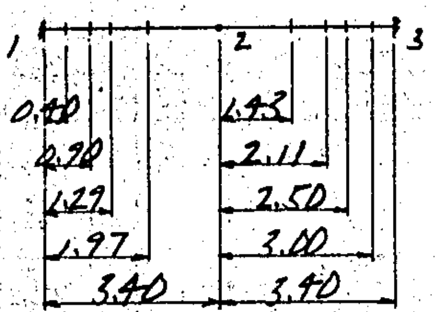
case 1 Dead load



case 2 Train load + Impact



(2) point of computing stresses



Input Data

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=== INPUT DATA ===                                                        CRC-FANSY | V6.3   PAGE -   1
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................................................................................

TITLE     FIX                                                                          1
JOINT           1     0.0       0.0                                                    2
                2     3.40      0.0                                                    3
                3     6.80      0.0                                                    4
MEMBER          1  1 1    1    2                                                       5
                2  1 1    2    3                                                       6
SUPPORT         1  000                                                                 7
                3  000                                                                 8
SECTOIN         1     1.000     1.000                                                  9
PROPERTY        1         2.70E6                                                      10
POINT     10.40       0.90      1.29      1.97                                        11
          21.43       2.11      2.50      3.00                                        12
LOAD            1                                                                     13
LL              1  2 2-0.27    -0.27      1.97      1.97                              14
                1  2 2-1.38    -1.38      0.0       0.0                               15
DL              1    2-1.11     1.29      0.0                                         16
                2    2-1.11     0.0       1.29                                        17
END                                                                                   18
LOAD            2                                                                     19
LL              1  2 2-16.36   -16.36     1.97      1.97                              20
END                                                                                   21
FINISH                                                                                22
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CONTROL DATA

METHOD	STRUCTURE	J.RENUMBER	M.RENUMBER	S.F. DIS.	UNI.SPRING	STAN.STIF.	BARA	SKEW MEM.
DIS	*RAHMEN*	*OFF*	*OFF*	*OFF*	*OFF*	*OFF*	*OFF*	*OFF*

LOAD TITLE

LOAD 1 LOAD 2

JOINT DATA

JOINT NUMBER 3

ZAHYO(M) JOINT	X	Y
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2	3.4000	0.0000
3	6.8000	0.0000

MEMBER DATA

MEMBER NUMBER 2

MEMBER	ITAN	JTAN	CONNECT. ITAN	JTAN	LENGTH	A	I	AES	KD(BANE)	PRO.NUM
1	1	2	FIX	FIX	3.4000	1.00000	1.0000000			1
2	2	3	FIX	FIX	3.4000	1.00000	1.0000000			1

PROPERTY DATA

PROPERTY NUMBER 1

PROPERTY	E	G	EPS
1	2.700E+06	0.	1.000E-05

SUPPORT DATA

SUPPORT NUMBER 2

SUPPORT	SUPPORT-JYOKEN X	Y	THET Z	X(BANE)	Y(BANE)	THET Z(BANE)
1	FIX	FIX	FIX	0.0	0.0	0.0
3	FIX	FIX	FIX	0.0	0.0	0.0

STRUCTURAL FIGURE

```
+-----------------------------+-----------------------------+
# 1                           2                             # 3
```

MOVE DATA

	GOIKI		POINT													
MEMBER	ITAN	JTAN	1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	......	......	.400	.900	1.290	1.970										
2	......	......	1.430	2.110	2.500	3.000										

LOAD DATA

	M/J	NAME	D	W1	W2	L1	L2
LOAD - 1							
MEMBER	1	LINEAR	Y	-.270	-.270	1.970	0.000
		LINEAR	Y	-1.380	-1.380	0.000	0.000
		LINEAR	Y	0.000	-1.110	0.000	2.110
		LINEAR	Y	-1.110	-1.110	1.290	0.000
	2	LINEAR	Y	-.270	-.270	0.000	1.970
		LINEAR	Y	-1.380	-1.380	0.000	0.000
		LINEAR	Y	-1.110	-1.110	0.000	1.290
		LINEAR	Y	-1.110	0.000	2.110	0.000
LOAD - 2							
MEMBER	1	LINEAR	Y	-16.360	-16.360	1.970	0.000
	2	LINEAR	Y	-16.360	-16.360	0.000	1.970

REACTION

LOAD	SUPPORT	X (TON)	Y (TON)	Z (TON.M)
LOAD - 1	1	0.000	8.136	-9.934
	3	0.000	8.136	9.934
LOAD - 2	1	0.000	23.395	-37.426
	3	0.000	23.395	37.426

D E F L E C T I O N

LOAD - 1

JOINT	X (MM)	Y (MM)	Z (MMRAD)
1	0.000	0.000	0.000
2	0.000	-.005	.000
3	0.000	0.000	0.000

LOAD - 2

JOINT	X (MM)	Y (MM)	Z (MMRAD)
1	0.000	0.000	0.000
2	0.000	-.024	.000
3	0.000	0.000	0.000

LOAD 1

= = MEMBER 1 (1 - 2) G = =

	L	M	(M)	Q	N
ITAN	0.000	-9.934		8.136	0.000
1	.400	-6.799		7.515	0.000
2	.900	-3.275		6.546	0.000
3	1.290	-.894		5.640	0.000
4	1.970	2.365		3.947	0.000
JTAN	3.400	5.187		-.000	0.000

= = MEMBER 2 (2 - 3) G = =

	L	M	(M)	Q	N
ITAN	0.000	5.187		.000	0.000
1	1.430	2.365		-3.947	0.000
2	2.110	-.894		-5.640	0.000
3	2.500	-3.275		-6.546	0.000
4	3.000	-6.799		-7.515	0.000
JTAN	3.400	-9.934		-8.136	0.000

LOAD 2

= = MEMBER 1 (1 - 2) G = =

	L	M	(M)	Q	N
ITAN	0.000	-37.426		23.395	0.000
1	.400	-28.068		23.395	0.000
2	.900	-16.371		23.395	0.000
3	1.290	-7.247		23.395	0.000
4	1.970	8.662		23.395	0.000
JTAN	3.400	25.389		-.000	0.000

= = MEMBER 2 (2 - 3) G = =

	L	M	(M)	Q	N
ITAN	0.000	25.389		.000	0.000
1	1.430	8.662		-23.395	0.000
2	2.110	-7.247		-23.395	0.000
3	2.500	-16.371		-23.395	0.000
4	3.000	-28.068		-23.395	0.000
JTAN	3.400	-37.426		-23.395	0.000

D E F L E C T I O N

LOAD - 1

JOINT	X (MM)	Y (MM)	Z (MMRAD)
1	0.000	0.000	-.047
2	0.000	-.105	0.000
3	0.000	0.000	.047

LOAD 1

	----L----	----M----	---(M)---	----Q----	----N----
= = MEMBER 1 (1 - 2)G = =					
ITAN	0.000	.000		23.395	0.000
1	.400	9.358		23.395	0.000
2	.900	21.055		23.395	0.000
3	1.290	30.179		23.395	0.000
4	1.970	46.088		23.395	0.000
JTAN	3.400	62.815		-.000	0.000
= = MEMBER 2 (2 - 3)G = =					
ITAN	0.000	62.815		.000	0.000
1	1.430	46.088		-23.395	0.000
2	2.110	30.179		-23.395	0.000
3	2.500	21.055		-23.395	0.000
4	3.000	9.358		-23.395	0.000
JTAN	3.400	.000		-23.395	0.000

			point 1	0.40	0.90	1.29	1.97	point 2
Bending moment	Dead load	Fixed	-9.93	-6.80	-3.28	-0.89	2.37	5.19
	Train load + Impact	Fixed	-37.43	-28.07	-16.37	-7.25	8.66	25.39
		simple	0	9.36	21.06	30.18	46.09	62.82
		Sub Fixed	-18.72	-9.36	2.35	11.47	27.38	44.11
M^{tm}	composite		-47.36	-34.87	-19.65	-8.14	11.03	30.58
			-28.65	-16.36	-0.93	10.58	29.75	49.30
Shearing	Dead load		8.14	7.52	6.55	5.64	3.95	0
	Train load + Impact		23.40	23.40	23.40	23.40	23.40	0
S^t	composite		31.54	30.92	29.95	29.04	27.35	0

allowable stress of clack

$$\frac{Tr + I}{D + Tr + I} = \frac{37.43}{47.36} = 0.79 > 0.25$$

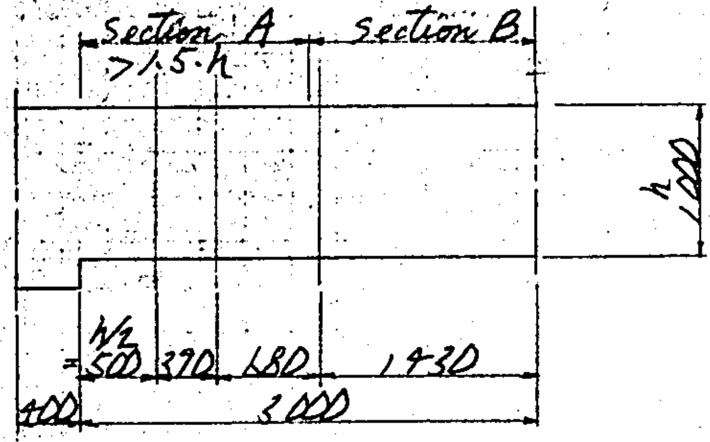
$$\therefore \sigma_{sa} = 800 \ kg/cm^2$$

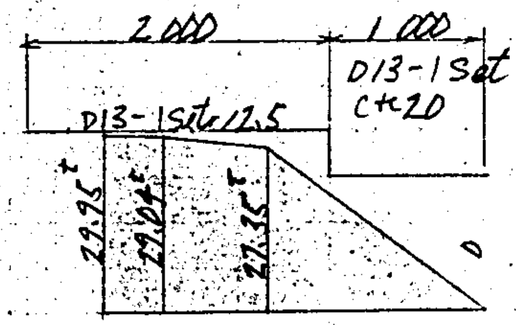
3. Calculation of stress

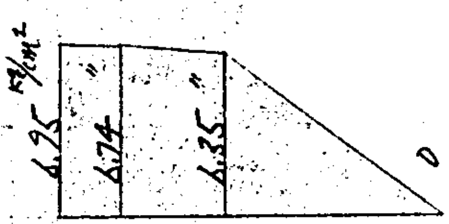
	Top of point		Bottom of span	
	D	D+Tr+I	D+Tr+I	
M (tm)	-9.93	-47.36	49.30	
N (t)				
S (t)		39.95		
b (cm)	50		50	
h (cm)	100		100	
d (cm)	86.1		89.4	
d' (cm)	13.9		10.6	
As (cm²)	D25 − 5/2 = 35.47		D25 − 2/5 = 35.47	
p	0.00824		0.00794	
As' (cm²)				
p'				
e=M/N (cm)				
e=M/N+u (cm)				
e=M/N−u (cm)				
e/h				
d/e				
d'/h				
d'/d				
Ne/bd² (kg/cm²)	2.68	12.78	12.34	
k				
c				
j				
1/Lc	5.91		5.98	
1/Ls	140		144	
β=σs/σc				
σc (kg/cm²)	15.8	75.5	73.8	
σs (kg/cm²)	375	1789	1777	
τ (kg/cm²)		6.96		
σsa (kg/cm²)	90	90	90	
σca (kg/cm²)	800	1800	1800	
τa (kg/cm²)		3.9		
Nomo No.	M₁	M₁	M₁	

4. Calculation of diagonal Tension re-bars

(a) Shearing stress caused by bending







$$\tau = \frac{S}{bd} = \frac{S \times 10^3}{50 \times 86.1} = 0.232 \cdot S \quad \text{kg/cm}^2$$

(b) Calculation of diagonal tension re-bars

(1) section A

(i) Shearing force beared by concrete

$S_c = 1/2 \cdot \tau_c \cdot b \cdot d$

τ_c : 3.9 kg/cm² ($\sigma_{ck} = 240$ kg/cm²)

$\therefore S_c = 1/2 \times 3.9 \times 50 \times 86.1 \times 10^{-3}$

$= 8.39^t$

(ii) Shearing force beared by stirrup

$S_v = \frac{A_v \cdot \sigma_{sa} \cdot d}{1.15 \cdot s}$

Arrange stirrups D13 - 1 set in 12.5 cm ctc

$A_v = 1.267 \times 2 = 2.53$ cm²

$\sigma_{sa} = 1800$ kg/cm² $s = 12.5^{cm} < 8 \times 2.5 = 20^{cm}$

$\therefore S_v = \frac{2.53 \times 1800 \times 86.1}{1.15 \times 15.0} \times 10^{-3}$

$= 22.73^t$

$\Sigma S_R = S_c + S_v$

$= 8.39 + 22.73$

$= 31.12^t > S = 29.95^t$

(iii) More than 0.2% of concrete

$$As = \frac{100 \cdot b \cdot 0.002}{n}$$

$$= \frac{100 \times 50 \times 0.002}{6.67}$$

$$= 1.50^{cm^2}$$

$$\therefore Av = 1.267 \times 2 = 2.53^{cm^2} > As = 1.50^{cm}$$

(2) Section B

(i) Shearing force beared by concrete

$$\therefore Sc = 8.39^{t}$$

(ii) Shearing force beared by stirrup

Arrange stirrup D13 - 1 set. 20^{cm} c+c

$$Av = 1.267 \times 2 = 2.53^{cm^2}$$

$$s = 20^{cm} < 1.5 \times 25 = 37.5^{cm}$$

$$\therefore Sv = \frac{2.53 \times 1800 \times 86.1}{1.15 \times 25} \times 10^{-3}$$

$$= 13.64^{t}$$

$$\Sigma SR = 8.39 + 13.64$$

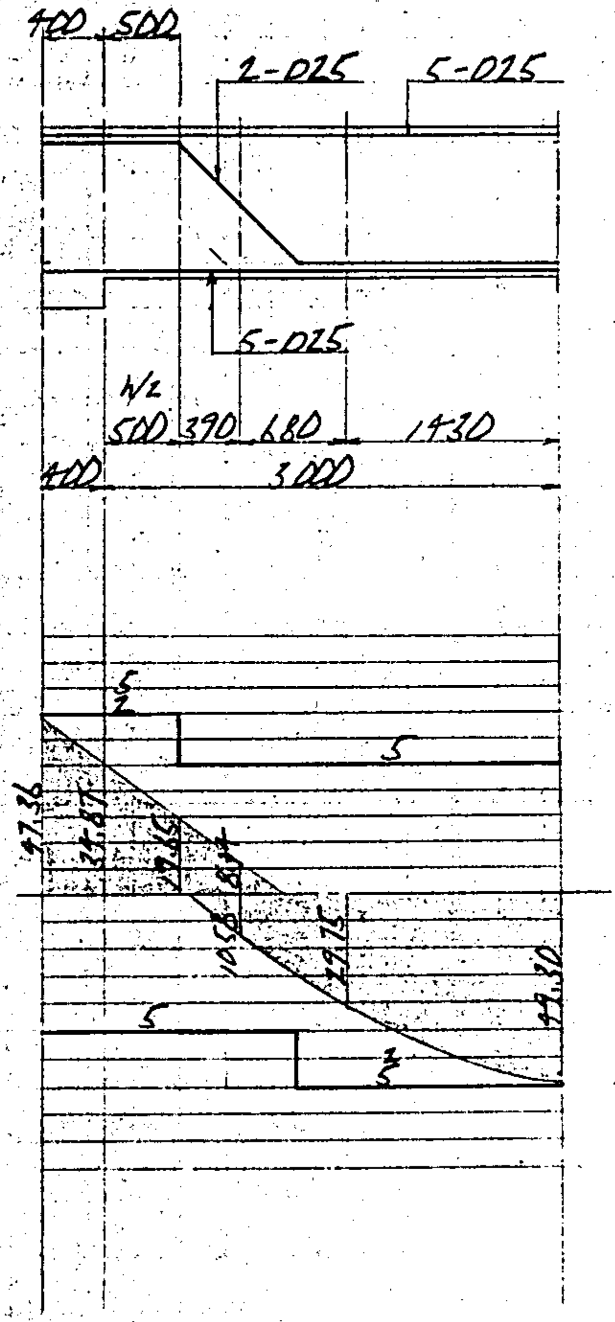
$$= 22.03^{t}$$

(iii) More than 0.15% of concrete

$$As = \frac{100 \times 50 \times 0.0015}{5}$$

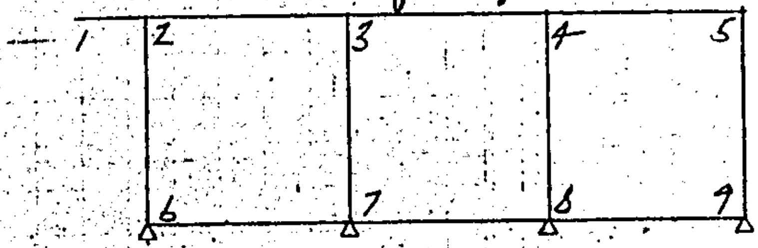
$$= 1.50^{cm^2} < Av = 1.267 \times 2 = 2.53^{cm^2}$$

5. Resisting bending moment diagram



§8. Calculation of column.

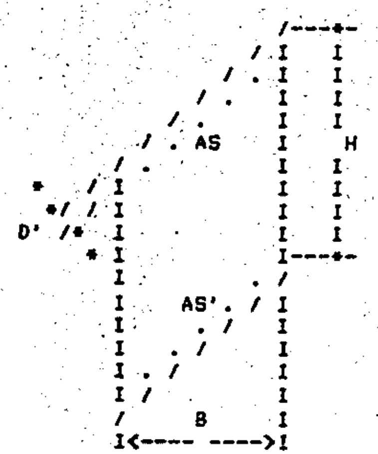
1) Rahmen (Riged frome) calculation in railway profile.



a) Critical case

		upper			under		
		M (tm)	N (t)	case	M (tm)	N (t)	case
② ~ ⑥	Mmax	36.15	84.00	29	23.43	79.85	28
	Mmin	32.64	69.62	28	35.04	94.22	29
	Nmax	15.16	206.87	27	21.08	220.21	27
	Nmin	—	—	—	—	—	—
③ ~ ⑦	Mmax	40.55	79.61	29	29.73	95.22	28
	Mmin	41.14	85.00	28	36.89	89.83	29
	Nmax	1.65	243.70	17	2.72	259.04	17
	Nmin	—	—	—	—	—	—
④ ~ ⑧	Mmax	39.59	71.30	29	38.11	75.55	28
	Mmin	42.10	65.33	28	38.81	81.53	29
	Nmax	1.08	219.05	20	1.03	234.39	20
	Nmin	—	—	—	—	—	—
⑤ ~ ⑨	Mmax	33.95	71.80	29	34.27	76.98	28
	Mmin	34.42	86.76	28	14.85	82.02	29
	Nmax	3.09	258.40	23	8.18	273.74	23
	Nmin	—	—	—	—	—	—

b) Calculation of stress (L-1)

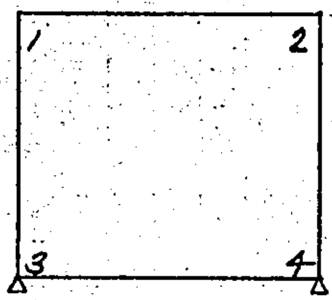



```
B     =  80.00                H =  80.00               D'   = 6.400

THETA      = 73    0'  0"                              D'/H=    .080

AS   = AS'   = D 25 -   6 =  30.402                    P    =   .00475


CASE 1

( , - )  M =  36.15     SHIGUMA C =   64.43            E    =  43.0        Y =  52.968
         N =  84.00     SHIGUMA S =  782.              E/H  =     .538

CASE 2

(  -  )  M =  32.64     SHIGUMA C =   58.60            E    =  46.9        Y =  50.853
         N =  69.62     SHIGUMA S =  777.              E/H  =     .586

CASE 3

(  -  )  M =  15.16     SHIGUMA C =   44.82            E    =   7.3        Y = 141.604
         N = 206.87     SHIGUMA S =     0.             E/H  =     .092

CASE 4

(  -  )  M =  23.43     SHIGUMA C =   40.73            E    =  29.3        Y =  65.749
         N =  79.85     SHIGUMA S =  279.              E/H  =     .367

CASE 5

(  -  )  M =  35.04     SHIGUMA C =   61.69            E    =  37.2        Y =  57.182
         N =  94.22     SHIGUMA S =  625.              E/H  =     .465

CASE 6

(  -  )  M =  21.08     SHIGUMA C =   53.11            E    =   9.6        Y = 120.649
         N = 220.21     SHIGUMA S =     0.             E/H  =     .120

CASE 7

(  -  )  M =  40.55     SHIGUMA C =   73.30            E    =  50.9        Y =  49.030
         N =  79.61     SHIGUMA S = 1049.              E/H  =     .637

CASE 8

(  -  )  M =  41.14     SHIGUMA C =   74.06            E    =  48.4        Y =  50.128
         N =  85.00     SHIGUMA S = 1012.              E/H  =     .605
```

```
CASE 9
( - )  M =   1.65    SHIGUMA C =   35.13     E   =   .7     Y =1019.590
       N = 243.70    SHIGUMA S =    0.       E/H =  .008

CASE10
( - )  M =  39.73    SHIGUMA C =   70.62     E   =  41.7    Y =  53.795
       N =  95.22    SHIGUMA S =  827.       E/H =  .522

CASE11
( - )  M =  36.89    SHIGUMA C =   65.49     E   =  41.1    Y =  54.233
       N =  89.83    SHIGUMA S =  753.       E/H =  .513

CASE12
( - )  M =   2.72    SHIGUMA C =   38.39     E   =   1.1    Y = 675.985
       N = 259.04    SHIGUMA S =    0.       E/H =  .013

CASE13
( - )  M =  39.59    SHIGUMA C =   72.03     E   =  55.5    Y =  47.345
       N =  71.30    SHIGUMA S = 1106.       E/H =  .694

CASE14
( - )  M =  42.10    SHIGUMA C =   77.34  ✓  E   =  64.4    Y =  44.872
       N =  65.33    SHIGUMA S = 1317.       E/H =  .806

CASE15
( - )  M =   1.08    SHIGUMA C =   31.14     E   =    .5    Y =1380.652
       N = 219.05    SHIGUMA S =    0.       E/H =  .006

CASE16
( - )  M =  38.11    SHIGUMA C =   68.84     E   =  50.4    Y =  49.233
       N =  75.55    SHIGUMA S =  977.       E/H =  .631

CASE17
( - )  M =  38.81    SHIGUMA C =   69.77     E   =  47.6    Y =  50.502
       N =  81.53    SHIGUMA S =  939.       E/H =  .595

CASE18
( - )  M =   1.03    SHIGUMA C =   33.18     E   =    .4    Y =1542.684
       N = 234.39    SHIGUMA S =    0.       E/H =  .005

CASE19
( - )  M =  33.95    SHIGUMA C =   61.00     E   =  47.3    Y =  50.656
       N =  71.80    SHIGUMA S =  816.       E/H =  .591

CASE20
( - )  M =  34.42    SHIGUMA C =   60.92     E   =  39.7    Y =  55.217
       N =  86.76    SHIGUMA S =  672.       E/H =  .496
```

```
CASE21

( - )  M =   3.09    SHIGUMA C =   38.71      E   =   1.2     Y = 599.940
       N = 258.40    SHIGUMA S =    0.        E/H =    .015

CASE22

( - )  M =  34.27    SHIGUMA C =   60.09      E   =  35.3     Y =  58.850
       N =  96.98    SHIGUMA S =  566.        E/H =    .442

CASE23

( - )  M =  24.85    SHIGUMA C =   43.23      E   =  30.3     Y =  64.473
       N =  82.02    SHIGUMA S =  315.        E/H =    .379

CASE24

( - )  M =   8.18    SHIGUMA C =   46.36      E   =   3.0     Y = 271.410
       N = 273.74    SHIGUMA S =    0.        E/H =    .037
```

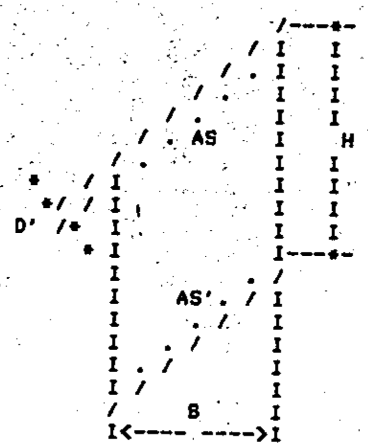
2) ①-① Rahmen (Rigid frame) Calculation in the direction of railway cross section.



a) Critical cases

		upper			under		
		M (tm)	N (t)	case	M (tm)	N (t)	case
① ~ ③	Mmax	62.28	225.68	10	35.60	85.13	15
	Mmin	43.70	75.27	15	55.79	238.53	10
	Nmax	51.17	253.50	9	46.37	268.28	9
	Nmin	—	—	—	—	—	—
② ~ ④	Mmax	46.85	213.07	10	47.94	120.64	15
	Mmin	52.05	110.79	15	38.42	225.92	10
	Nmax	33.43	251.04	9	26.39	265.82	9
	Nmin	43.85	82.59	14	35.46	92.44	14

(b) Calculation of stress (C-1)

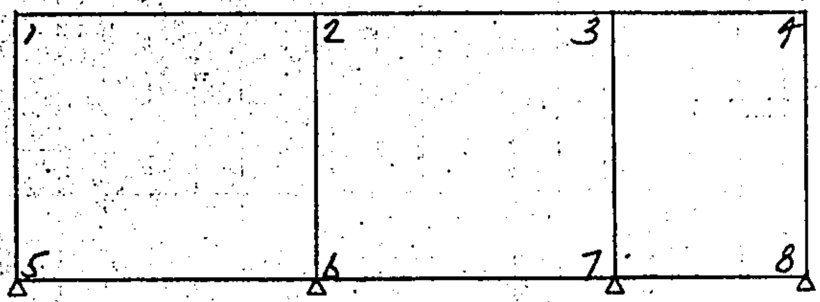


B = 80.00		H = 80.00		D' = 6.400	
THETA = 73 0' 0"				D'/H= .080	
AS = AS' = D 25 - 12 = 60.804				P = .00950	

CASE 1				
(-)	M = 62.28 N = 225.68	SHIGUMA C = 87.26 SHIGUMA S = 402.	E = 27.6 E/H = .345	Y = 73.291
CASE 2				
(-)	M = 43.70 N = 75.27	SHIGUMA C = 57.64 SHIGUMA S = 704.	E = 58.1 E/H = .726	Y = 52.821
CASE 3				
(-)	M = 51.17 N = 253.50	SHIGUMA C = 76.97 SHIGUMA S = 113.	E = 20.2 E/H = .252	Y = 87.244
CASE 4				
(-)	M = 35.60 N = 85.13	SHIGUMA C = 47.75 SHIGUMA S = 430.	E = 41.8 E/H = .523	Y = 59.877
CASE 5				
(-)	M = 55.79 N = 238.53	SHIGUMA C = 80.78 SHIGUMA S = 237.	E = 23.4 E/H = .292	Y = 80.126
CASE 6				
(-)	M = 46.37 N = 268.28	SHIGUMA C = 71.16 SHIGUMA S = 0.	E = 17.3 E/H = .216	Y = 96.376
CASE 7				
(-)	M = 46.85 N = 213.07	SHIGUMA C = 68.86 SHIGUMA S = 160.	E = 22.0 E/H = .275	Y = 82.975
CASE 8				
(-)	M = 52.05 N = 110.79	SHIGUMA C = 69.34 SHIGUMA S = 707.	E = 47.0 E/H = .587	Y = 57.052

```
CASE 9

( - )  M =  33.43    SHIGUMA C =  60.46     E   =  13.3     Y = 105.480
       N = 251.04    SHIGUMA S =   0.       E/H =   .166

CASE10

( - )  M =  43.85    SHIGUMA C =  58.07     E   =  53.1     Y =  54.476
       N =  82.59    SHIGUMA S = 661.       E/H =   .664

CASE11

( - )  M =  47.94    SHIGUMA C =  64.51     E   =  39.7     Y =  61.241
       N = 120.64    SHIGUMA S = 546.       E/H =   .497

CASE12

( - )  M =  38.42    SHIGUMA C =  59.14     E   =  17.0     Y =  97.493
       N = 225.92    SHIGUMA S =   0.       E/H =   .213

CASE13

( - )  M =  26.39    SHIGUMA C =  55.96     E   =   9.9     Y = 123.657
       N = 265.82    SHIGUMA S =   0.       E/H =   .124

CASE14

( - )  M =  35.46    SHIGUMA C =  47.84     E   =  38.4     Y =  62.230
       N =  92.44    SHIGUMA S = 387.       E/H =   .480
```

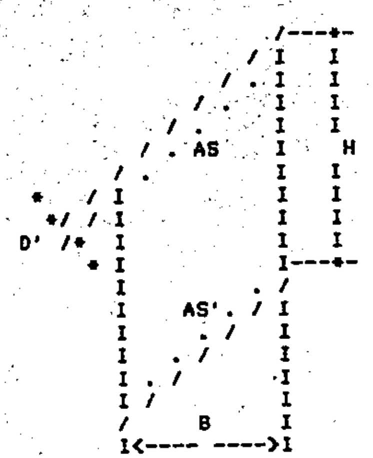
3) ②-② Ralmen (Rigid frame) Calculation in the direction of railway cross section.



a) Critical cases

		Upper			Under		
		M (t·m)	N (t)	case	M (t·m)	N (t)	case
① ~ ⑤	Mmax	30.42	36.70	10	15.74	29.73	15
	Mmin	17.63	19.45	15	25.72	41.47	14
	Nmax	28.88	40.68	9	24.15	56.09	9
	Nmin	—	—	—	—	—	—
② ~ ⑥	Mmax	29.21	196.06	10	26.47	95.10	15
	Mmin	27.10	84.82	15	26.01	91.52	14
	Nmax	26.39	225.92	9	21.28	241.34	9
	Nmin	—	—	—	—	—	—
③ ~ ⑦	Mmax	24.61	67.45	14	28.93	68.75	15
	Mmin	31.16	58.47	15	25.00	77.73	14
	Nmax	7.60	184.07	9	13.70	199.49	9
	Nmin	—	—	—	—	—	—
④ ~ ⑧	Mmax	20.84	0.97	14	25.01	28.40	15
	Mmin	27.86	18.12	15	10.33	11.25	14
	Nmax	—	—	—	—	—	—
	Nmin	14.99	-1.41	10	—	—	—

(b) Calculation of stress (C-2)




```
B      =  80.00            H =  80.00          D'   = 6.400

THETA      = 73   0'  0"                       D'/H=   .080

AS   = AS'   = D 25 -  6 =  30.402             P    =   .00475


CASE 1
( - )  M =  30.42     SHIGUMA C =   56.54     E   =  82.9      Y =  41.675
       N =  36.70     SHIGUMA S = 1102.       E/H =   1.036

CASE 2
( - )  M =  17.63     SHIGUMA C =   32.87     E   =  90.6      Y =  40.770
       N =  19.45     SHIGUMA S =  666.       E/H =   1.133

CASE 3
( - )  M =  28.88     SHIGUMA C =   53.33     E   =  71.0      Y =  43.514
       N =  40.68     SHIGUMA S =  961.       E/H =    .887

CASE 4
( - )  M =  15.74     SHIGUMA C =   28.54     E   =  52.9      Y =  48.250
       N =  29.73     SHIGUMA S =  422.       E/H =    .662

CASE 5
( - )  M =  25.72     SHIGUMA C =   47.14     E   =  62.0      Y =  45.461
       N =  41.47     SHIGUMA S =  783.       E/H =    .775

CASE 6
( - )  M =  24.15     SHIGUMA C =   43.05     E   =  43.1      Y =  52.955
       N =  56.09     SHIGUMA S =  523.       E/H =    .538

CASE 7
( - )  M =  29.21     SHIGUMA C =   57.03     E   =  14.9      Y =  98.543
       N = 196.06     SHIGUMA S =    0.       E/H =    .186

CASE 8
( - )  M =  27.10     SHIGUMA C =   47.23     E   =  32.0      Y =  62.433
       N =  84.82     SHIGUMA S =  379.       E/H =    .399

CASE 9
( - )  M =  26.39     SHIGUMA C =   59.68     E   =  11.7      Y = 108.299
       N = 225.92     SHIGUMA S =    0.       E/H =    .146
```

```
CASE10

( - )  M =  26.47    SHIGUMA C =   46.02     E   =  27.8     Y =  67.921
       N =  95.10    SHIGUMA S =  283.       E/H =    .348

CASE11

( - )  M =  26.01    SHIGUMA C =   45.21     E   =  28.4     Y =  67.054
       N =  91.52    SHIGUMA S =  291.       E/H =    .355

CASE12

( - )  M =  21.28    SHIGUMA C =   56.21     E   =   8.8     Y = 126.510
       N = 241.34    SHIGUMA S =    0.       E/H =    .110

CASE13

( - )  M =  24.61    SHIGUMA C =   43.26     E   =  36.5     Y =  57.793
       N =  67.45    SHIGUMA S =  427.       E/H =    .456

CASE14

( - )  M =  31.16    SHIGUMA C =   56.52     E   =  53.3     Y =  48.122
       N =  58.47    SHIGUMA S =  840.       E/H =    .666

CASE15

( - )  M =   7.60    SHIGUMA C =   33.46     E   =   4.1     Y = 210.860
       N = 184.07    SHIGUMA S =    0.       E/H =    .052

CASE16

( - )  M =  28.93    SHIGUMA C =   51.46     E   =  42.1     Y =  53.564
       N =  68.75    SHIGUMA S =  609.       E/H =    .526

CASE17

( - )  M =  25.00    SHIGUMA C =   43.59     E   =  32.2     Y =  62.186
       N =  77.73    SHIGUMA S =  353.       E/H =    .402

CASE18

( - )  M =  13.70    SHIGUMA C =   42.22     E   =   6.9     Y = 147.600
       N = 199.49    SHIGUMA S =    0.       E/H =    .086

CASE19

( - )  M =  20.84    SHIGUMA C =   39.23     E   =2148.5     Y =  32.646
       N =    .97    SHIGUMA S = 1138.       E/H =  26.856

CASE20

( - )  M =  27.86    SHIGUMA C =   52.45     E   = 153.8     Y =  37.053
       N =  18.12    SHIGUMA S = 1247.       E/H =   1.922

CASE21

( - )  KEISAN DEKINAI
```

```
CASE22
(  -  )  M =  25.01     SHIGUMA C =   46.58     E   =  88.1     Y =  41.051
         N =  28.40     SHIGUMA S =  932.       E/H =   1.101
CASE23
(  -  )  M =  20.33     SHIGUMA C =   38.32     E   = 180.7     Y =  36.306
         N =  11.25     SHIGUMA S =  942.       E/H =   2.259
```

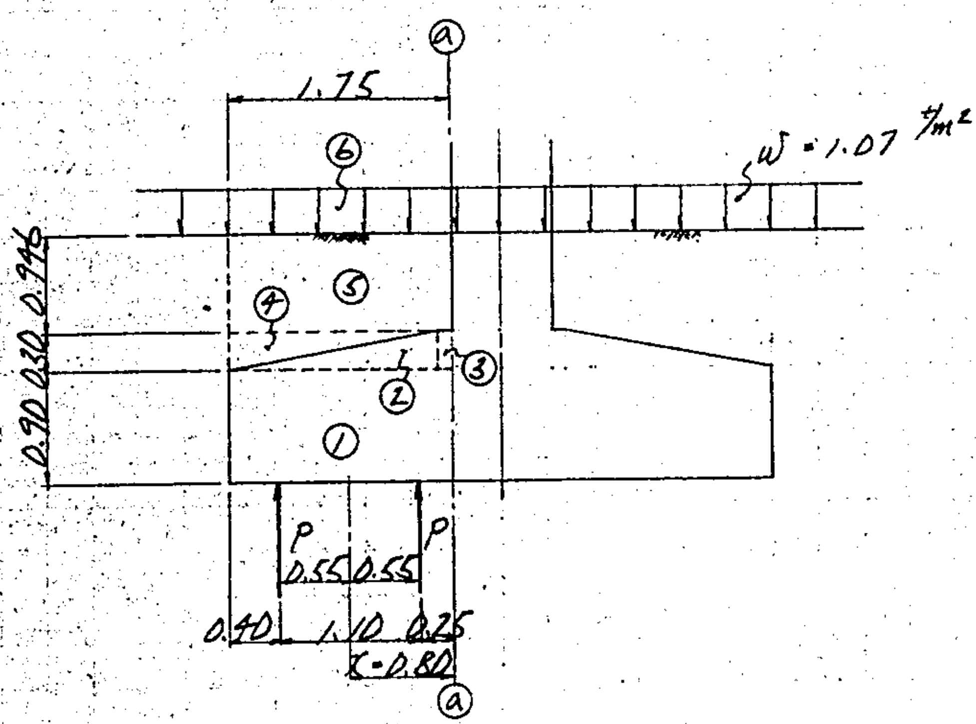
§9. Calculation of foundation

1. Right frame anglyis on longitudinal direction of elevated structure (L-1)

lond	Member NO	Vertical force ΣN t	Horyzontal force ΣH t	Pile n pile	Reaction of one Pile N t/pile	H t/pile	allowable bearing Power Ra t/pile
Dead load	end ⑥	225.86	---	12	18.82	—	
	middle ⑦	247.30	—	〃	20.61	—	
α = 1.00	〃 ⑧	225.26	---	〃	18.77	—	24
case 1	end ⑨	221.30	---	〃	18.44	—	
Dead load + Train load and Impact	end ⑥	c16 295.43		〃	24.61	—	
	middle ⑦	c17 366.71		〃	30.56	—	36
α = 1.00	〃 ⑧	c20 340.97	—	〃	28.42	—	
	end ⑨	c23 360.85	—	〃	30.07	—	
Dead load + Seismic load	end ⑥	205.37	14.00	〃	17.11	1.17	
	middle ⑦	255.18	17.88	〃	21.27	1.49	48
α = 1.50	〃 ⑧	216.30	17.78	〃	18.03	1.48	
case 34	end ⑨	242.87	15.98	〃	20.24	1.33	
Dead load + Seismic load	end ⑥	246.36	-16.37	〃	20.53	1.36	
	middle ⑦	239.41	-17.35	〃	19.95	1.45	48
α = 1.50	〃 ⑧	234.23	-17.50	〃	19.52	1.46	
case 35	end ⑨	199.73	-14.43	〃	16.64	1.20	

(1) Calculation of Bending moment

(1) Bending moment of footing and eath weight



$N_1 = 2.5 \times 1.75 \times 0.90 \times 3.20 = 12.60\ ^t$

$N_2 = 2.5 \times 1/2 \times 1.65 \times 0.30 \times 3.20 = 1.98$ "

$N_3 = 2.5 \times 0.10 \times 0.30 \times 3.20 = 0.24$ "

$N_4 = 1.8 \times 1/2 \times 1.65 \times 0.30 \times 3.20 = 1.43$ "

$N_5 = 1.8 \times 1.75 \times 0.946 \times 3.20 = 9.54$ "

$N_6 = 1.07 \times 1.75 \times 3.20 = 5.99$ "

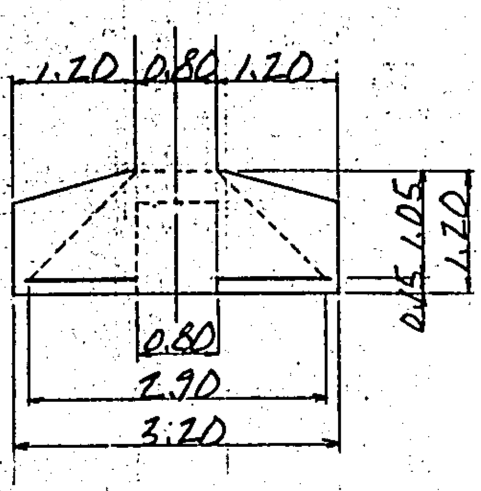
$$Ma = (11.61 + 8.79 + 5.52) \times 0.60 + 1.77 \times 0.467 + 0.32 \times 0.05 + 1.28 \times 0.833$$
$$= 17.46^{t \cdot m}$$

$$\Sigma Ma = P \cdot n \cdot x - Ma$$

		α	Reaction of pile P (t/pil)	pile n	arm x (m)	P.n.x (t.m)	-Ma (t.m)	ΣMa (t.m)
end ⑥	ordinary case	1.00	18.82	3	0.80	90.34	-27.63	62.71
	ordinary case + temporary	1.00	24.61	〃	〃	118.13	〃	90.50
	Earthquake case	1.50	20.53	〃	〃	98.54	〃	70.91
middle ⑦	ordinary case	1.00	20.61	〃	〃	98.93	〃	71.30
	ordinary case + temporary	1.00	30.56	〃	〃	146.69	〃	119.06
	Earthquake case	1.50	21.27	〃	〃	102.10	〃	74.47
middle ⑧	ordinary case	1.00	18.77	〃	〃	90.10	〃	62.47
	ordinary case + temporary	1.00	28.42	〃	〃	130.42	〃	108.79
	Earthquake case	1.50	19.52	〃	〃	93.70	〃	66.07
end ⑨	ordinary case	1.00	18.44	〃	〃	88.51	〃	60.88
	ordinary case + temporary	1.00	30.07	〃	〃	144.34	〃	116.71
	Earthquake case	1.50	20.24	〃	〃	97.15	〃	69.52

(2) Stress calculation

middle ⑦ (8 + 8)



$M = 119.06$ t·m

$B = 80 + 2 \times 105$

$= 290$ cm

$b_e = 290 - 80$

$= 210$ cm

$A_s = D25 - 15^{cm} ctc = 5.067 \times \frac{210}{15} = 70.94$ cm²

$p = \frac{70.94}{210 \times 105} = 0.00322$

From the Nomogram (M-1)

$1/L_c = 8.23 \qquad 1/L_s = 340$

$\therefore \sigma_c = \frac{119.06 \times 10^5}{210 \times 105^2} \times 8.23 = 42.3$ kg/cm² $< \sigma_{ca} = 90$ kg/cm²

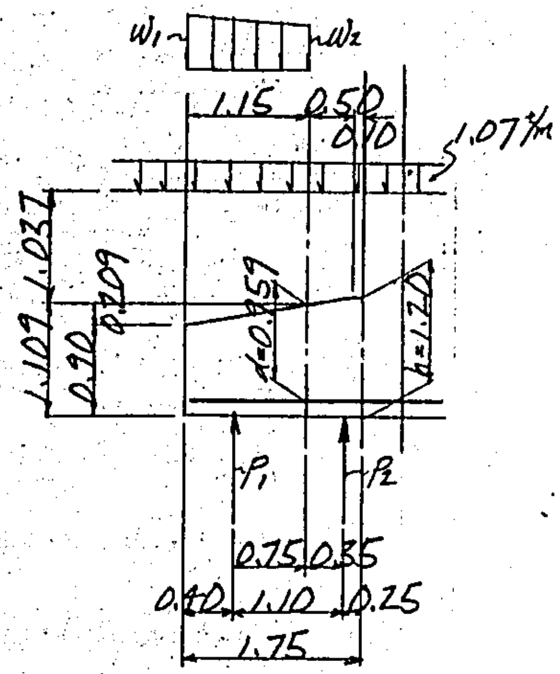
$\therefore \sigma_s =$ 〃 $\times 340 = 1748$ 〃 $< \sigma_{sa} = 1800$ 〃

Check, safe against cracking

$M = 71.30$ t·m

$\therefore \sigma_s = \frac{71.30 \times 10^5}{210 \times 105^2} \times 340 = 1047$ kg/cm² $< \sigma_{sa} = 1400$ kg/cm²

(3) Calculation of shearing fore caused by beading



$a = 1.35^{m}$

$d = 0.959^{m}$

$\therefore \gamma_1 = \dfrac{2}{a/d} = \dfrac{2 \times 0.959}{1.35}$

$= 1.421 < 4$

(i) Load

$W_1 = (2.5 \times 0.90 + 1.8 \times 1.246 + 1.07) \times \dfrac{3.20}{1.421}$

$= 12.53$ t/m

$W_2 = (2.5 \times 1.109 + 1.8 \times 1.037 + 1.07) \times \dfrac{3.20}{1.421}$

$= 12.86$ t/m

(ii) Reaction of pile

$P_1 = 30.56^{t}$

$P'_1 = 30.56 \times 3 \times \dfrac{1}{1.421} = 64.52^{t}$

(iii) Shearing

$\Sigma S = 1/2 \times (12.53 + 12.86) \times 1.15 - 64.52 = -49.92^{t}$

(iv) Shearing stress

$S = 49.92^{t}$

$\therefore \tau = \frac{49.92 \times 10^3}{320 \times 95.9} = 1.63^{kg/cm^2} < \tau a = 3.9^{kg/cm^2}$

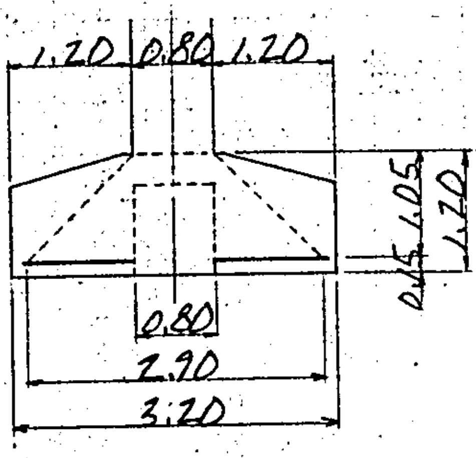
1. distributing bar
more than 1/6 of main reinforcement.
main reinforcement D25 - 6.67(ctc 15) × 1/6 = 5.63 cm^2
distributing bar D22 - 6.67 (ctc 15) = 25.82 "

2. TOP side
more than 1/6 of Baton reinforcement.
D25 - 6.67 (ctc 15) × 1/6 = 5.63 cm^2
Top reinforcement D16 - 3.33 (ctc 30) = 6.61 "

3. Ratio of tensile reinforcement.

0.3 % over
$P = 0.00322 = 0.322\% > 0.3\%$

Calculation of stress
end ⑥ (3+8)



$M = 90.50^{t \cdot m}$

$Be = 80 + 2 \times 105$
$= 290^{cm}$

$be = 290 - 80$
$= 210^{cm}$

$As = D25 - 15^{m} ctc = 3.871 \times \frac{210}{15} = 54.19^{cm^2}$

$P = \frac{54.19}{210 \times 105} = 0.00246$

From the Nomogram (M-1)

$1/Lc = 9.15 \quad 1/Ls = 442$

$\therefore \sigma_c = \frac{90.50 \times 10^5}{210 \times 105^2} \times 9.15 = 35.8 \ kg/cm^2 < \sigma_{ca} = 90 \ kg/cm^2$

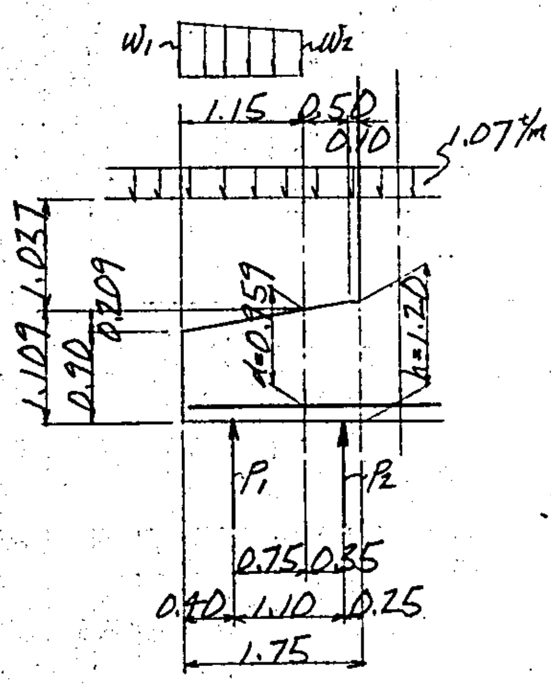
$\therefore \sigma_s =$ 〃 $\times 442 = 1728$ 〃 $< \sigma_{sa} = 1800$ 〃

cheak. safe against crocking

$M = 62.71^{t \cdot m}$

$\therefore \sigma_s = \frac{62.71 \times 10^5}{210 \times 105^2} \times 442 = 1197 \ kg/cm^2 < \sigma_{sa} = 1400 \ kg/cm^2$

Calculation of Bending Shearing stress.



$a = 1.35^{m}$

$d = 0.959^{m}$

$\therefore \gamma_1 = \frac{2}{a/d} = \frac{2 \times 0.959}{1.35}$

$= 1.421 < 4$

(i) Load

$w_1 = (2.5 \times 0.90 + 1.8 \times 1.246 + 1.07) \times \frac{3.20}{1.421}$

$= 12.53\ ^{t/m}$

$w_2 = (2.5 \times 1.109 + 1.8 \times 1.037 + 1.07) \times \frac{3.20}{1.421}$

$= 12.86\ ^{t/m}$

(ii) Reaction of pile

$P_1 = 24.61^{t}$

$P'_1 = 24.61 \times 3 \times \frac{1}{1.421} = 51.96^{t}$

Shearing

$\Sigma S = \frac{1}{2} \times (12.53 + 12.86) \times 1.15 - 51.96 = -37.36^{t}$

(iV) Shearing stress

$S = 37.36^{t}$

$\therefore \tau = \dfrac{37.36 \times 10^3}{320 \times 95.9} = 1.22^{kg/cm^2} < \tau a = 3.9^{kg/cm^2}$

1. distributing bar
more than 1/6 of main reinforcement.
main reinforcement D22-6.67 (ctc15) × 1/6 = 4.30^{cm^2}
distributing bar D22-6.67 (ctc15) = 25.02 "

2. ToP side
more than 1/6 of Batom reinforcement.
D22- 6.67 (ctc15) × 1/6 = 4.31^{cm^2}
Top reimforcement D16- 6.67 (ctc 30) = 6.61 "

3. Ratio of tensile reinforcement

0.3 % over
$P = 0.00246 - 0.246\% \not> 0.3\%$
$S = 37.36 \times 1.421 = 53.09^{t}$ $\tau = \dfrac{53.09 \times 10^3}{320 \times 95.9} = 1.73$

$\tau ax = 3.9 \times \sqrt{\dfrac{0.246}{0.3}} = 3.53^{kg/cm^2} > 1.73^{kg/cm^2}$

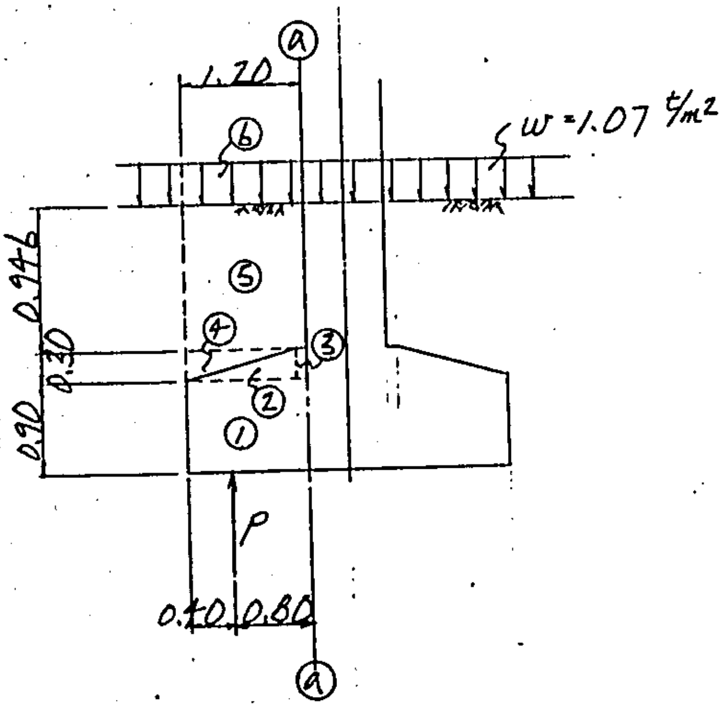
2. Rigid frame analysis on transversel.

c.-1

load	Member NO	Vertical force ΣN t	Horizontal force ΣH t	Pile n pile	Reaction of one Pile N t/pile	Reaction of one Pile H t/pile	allowable bearing Power t/pile
Dead load α = 1.00 case 1	end ③	238.98	—	12	19.92	—	24
	middle ⑦		—	〃		—	
	〃 ⑧		—	〃		—	
	end ④	250.02	—	〃	20.84	—	
Dead load + Train load and Impact α = 1.00 case 9	end ③	369.16	—	〃	30.76	—	36
	middle ⑦		—	〃		—	
	〃 ⑧		—	〃		—	
	end ④	345.30	—	〃	28.78	—	
Dead load + seismic load α = 1.50 case 20	end ③	278.53	21.08	〃	23.21	1.76	48
	middle ⑦			〃			
	〃 ⑧			〃			
	end ④	210.47	17.72	〃	17.54	1.48	
Dead load + seismic load α = 1.50 case 21	end ③	199.43	17.72	〃	16.62	1.48	48
	middle ⑦			〃	—	—	
	〃 ⑧			〃			
	end ④	289.56	21.08	〃	23.21	1.76	

(1) Calculation of Bending moment

(i) Bending moment of footing and eath weight.



$N_1 = 2.5 \times 1.20 \times 0.90 \times 4.30 = 11.61^{t}$

$N_2 = 2.5 \times 1/2 \times 1.10 \times 0.30 \times 4.30 = 1.77$ "

$N_3 = 2.5 \times 0.10 \times 0.30 \times 4.30 = 0.32$ "

$N_4 = 1.8 \times 1/2 \times 1.10 \times 0.30 \times 4.30 = 1.28$ "

$N_5 = 1.8 \times 1.20 \times 0.946 \times 4.30 = 8.79$ "

$N_6 = 1.07 \times 1.20 \times 4.30 = 5.52$ "

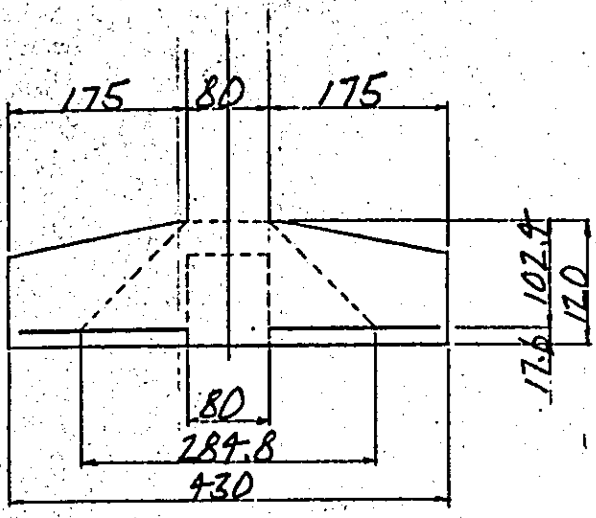
$$Ma = (11.61 + 8.79 + 5.52) \times 0.60 + 1.77 \times 0.467 + 0.32 \times 0.05 + 1.28 \times 0.833 = 17.46^{t\cdot m}$$

$$\Sigma Ma = P \cdot n \cdot x - Ma$$

		α	Reaction of pile P (t/pile)	pile n	arm x (m)	P.n.x (t.m)	-Ma (t.m)	ΣMa (t.m)
end ③	ordinary case	1.00	19.92	4	0.80	63.74	-17.46	46.28
	ordinary case + temporary	1.00	30.76	"	"	98.43	'	80.97
	Earthquake case	1.50	23.21	"	"	74.27	"	56.82
middle ◯	ordinary case							
	ordinary case + temporary							
	Earthquake case							
middle ◯	ordinary case							
	ordinary case + temporary							
	Earthquake case							
end ④	ordinary case	1.00	20.84	4	0.80	66.69	-17.46	49.23
	ordinary case + temporary	1.00	28.78	"	"	92.10	"	74.64
	Earthquake case	1.50	23.21	"	"	74.27	"	56.81

(2) Stress calculation

middle ③



$M = 80.97^{t \cdot m}$

$B_e = 80 + 2 \times 102.4$

$= 284.8^{cm}$

$b_e = 284.8 - 80$

$= 204.8^{cm}$

From the Nomogram (M-1)

$A_s = D22 - 15^{cm} ctc = 3.871 \times \frac{204.8}{15} = 52.85^{cm^2}$

$P = \frac{52.85}{204.8 \times 102.4} = 0.00252$

ノモグラム M-1より

$1/L_c = 9.07 \quad 1/L_s = 432$

$\therefore \sigma_c = \frac{80.97 \times 10^5}{204.8 \times 102.4^2} \times 9.07 = 34.2\ kg/cm^2 < \sigma_{ca} = 90\ kg/cm^2$

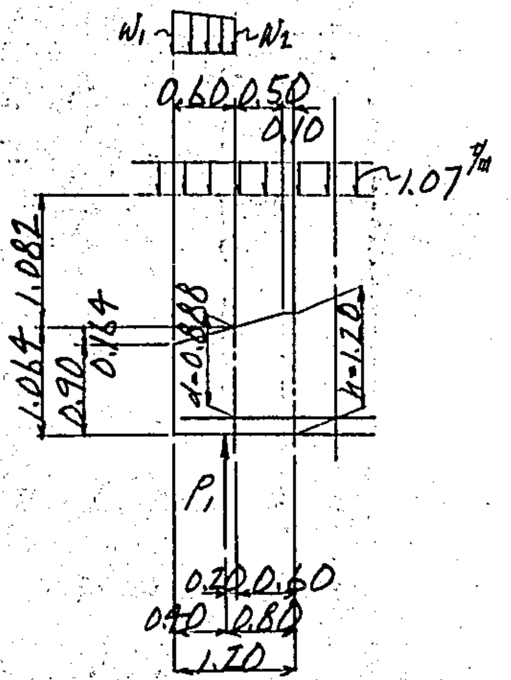
$\therefore \sigma_s = \quad " \quad \times 432 = 1629\ " < \sigma_{sa} = 1800\ "$

check, safe against cracking

$M = 49.23^{t \cdot m}$

$\therefore \sigma_s = \frac{49.23 \times 10^5}{204.8 \times 102.4^2} \times 432 = 990\ kg/cm^2 < \sigma_{sa} = 1400\ kg/cm^2$

(3) Calculation of shearing fore caused by beading.



$a = 0.60^m$

$d = 0.888^m$

$$\therefore \gamma_1 = \frac{2}{a/d} = \frac{2 \times 0.888}{0.60} = 2.960 < 4$$

(i) Load

$$W_1 = (2.5 \times 0.90 + 1.8 \times 1.246 + 1.07) \times \frac{4.30}{2.960} = 8.08 \ ^t/_m$$

$$W_2 = (2.5 \times 1.064 + 1.8 \times 1.082 + 1.07) \times \frac{4.30}{2.960} = 8.25 \ ^t/_m$$

(ii) Reaction of pile

$$P_1 = 30.76^t$$

$$P'_1 = 30.76 \times 4 \times \frac{1}{2.960} = 41.57^t$$

(iii) Shearing

$$\Sigma S = \frac{1}{2} \times (8.08 + 8.25) \times 0.60 - 41.57 = -36.67^t$$

(iv) Shearing stress

$S = 36.67^{t}$

$\therefore \tau = \dfrac{36.67 \times 10^3}{430 \times 88.8} = 0.96 \, kg/cm^2 < \tau a = 3.9 \, kg/cm^2$

(4)

1. Distri buting bar

more than 1/6 of main reinforcement.

main reinforcement D22-6.67(ctc15) × 1/6 = 4.30 cm^2

distributing bar D25-6.67 (ctc 15) (D22) = 33.80 " (25.82) "

2. Top side

more than 1/6 of Bottom reinforcement.

D22-6.67(ctc15) × 1/6 = 4.30 cm^2

Top reinfor cement D16-6.67(ctc30) = 6.61 "

3. Ration of tensile reinforcement.

0.3 % over

$p = 0.00252 = 0.252\% \ngtr 0.3\%$

$S = 36.67 \times 2.96 = 108.54^{t}$ $\tau = \dfrac{108.54 \times 10^3}{430 \times 88.8} = 2.84 \, kg/cm^2$

$\tau ad = 3.9 \times \sqrt{\dfrac{0.252}{0.3}} = 3.57 \, kg/cm^2 > 2.84 \, kg/cm^2$

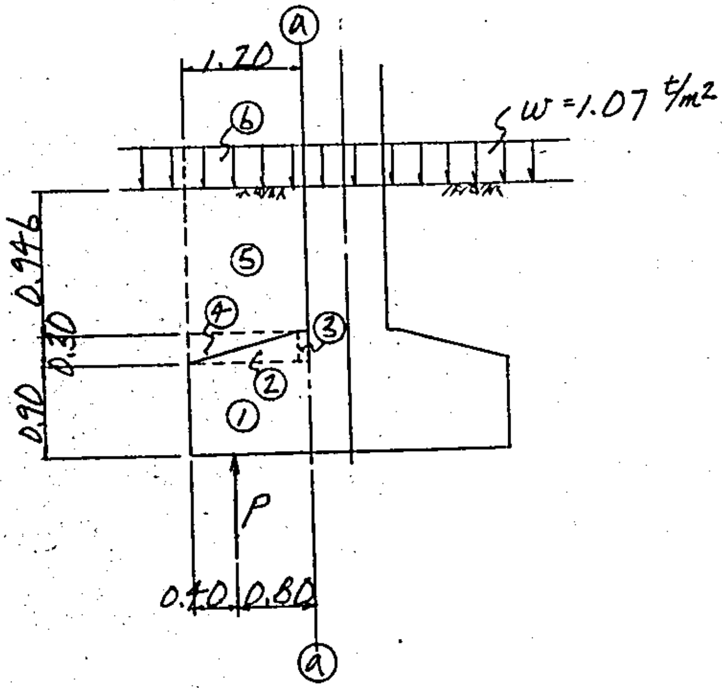
3. Rigid frame analysis on transversal.

C-2

load	Member NO	Vertical force ΣN t	Horyzontal Porce ΣH t	Pile n pile	Reaction of one Pile N t/pile	H t/pile	allowable bearing Power Ra t/pile
Dead load α = 1.00 case 1	end ⑤	150.36	—	12	12.53	—	24
	middle ⑥	248.30	—	"	20.69	—	
	" ⑦	212.31	—	"	17.69	—	
	end ⑧	122.33	—	"	10.19	—	
Dead load + Train load and Impact α = 1.00 case 9	end ⑤	157.70	—	"	13.14	—	36
	middle ⑥	347.21	—	"	28.93	—	
	" ⑦	305.69	—	"	25.47	—	
	end ⑧	102.71	—	"	8.56	—	
Dead load + seismic load α = 1.50 case 20	end ⑤	167.63	13.78	"	13.97	1.15	48
	middle ⑥	241.71	13.67	"	20.14	1.60	
	" ⑦	227.89	12.95	"	18.99	1.08	
	end ⑧	96.07	11.64	"	8.01	0.97	
Dead load + seismic load α = 1.50 case 21	end ⑤	133.09	10.43	"	11.09	0.87	48
	middle ⑥	254.89	13.57	"	21.24	1.13	
	" ⑦	196.72	14.58	"	16.39	1.22	
	end ⑧	148.59	13.46	"	12.38	1.12	

(1) Calculation of Bending moment

(i) Bending moment of footing and eath weight.



$N_1 = 2.5 \times 1.20 \times 0.90 \times 4.30 = 11.61^t$

$N_2 = 2.5 \times 1/2 \times 1.10 \times 0.30 \times 4.30 = 1.77$ "

$N_3 = 2.5 \times 0.10 \times 0.30 \times 4.30 = 0.32$ "

$N_4 = 1.8 \times 1/2 \times 1.10 \times 0.30 \times 4.30 = 1.28$ "

$N_5 = 1.8 \times 1.20 \times 0.946 \times 4.30 = 8.79$ "

$N_6 = 1.07 \times 1.20 \times 4.30 = 5.52$ "

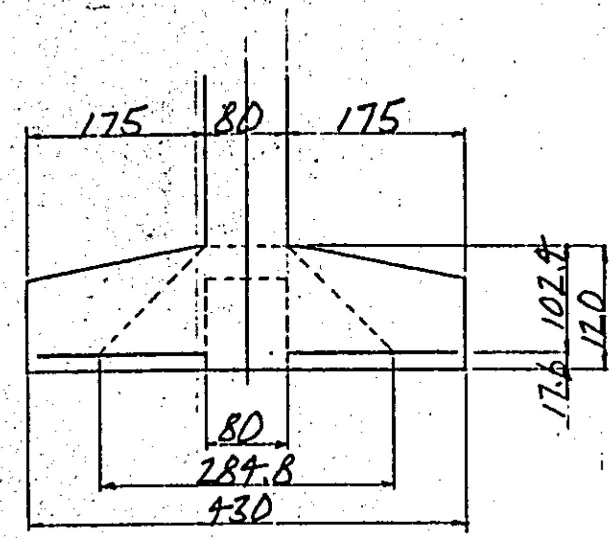
$$Ma = (11.61 + 8.79 + 5.52) \times 0.60$$
$$+ 1.77 \times 0.467 + 0.32 \times 0.05 + 1.28 \times 0.833$$
$$= 17.46^{t \cdot m}$$

$$\Sigma Ma = P \cdot n \cdot x - Ma$$

		α	Reaction of pile P (t/pile)	pile n	arm x (m)	P·n·x (t·m)	-Ma (t·m)	ΣMa (t·m)
end ⑤	ordinary case	1.00	12.53	4	0.80	40.10	-17.46	22.64
	ordinary case + temporary	1.00	13.14	〃	〃	42.05	〃	24.59
	Earthquake case	1.50	13.97	〃	〃	44.70	〃	27.24
middle ⑥	ordinary case	1.00	20.69	〃	〃	66.21	〃	48.75
	ordinary case + temporary	1.00	28.93	〃	〃	92.58	〃	75.12
	Earthquake case	1.50	21.24	〃	〃	67.97	〃	50.51
middle ⑦	ordinary case	1.00	17.69	〃	〃	56.61	〃	39.15
	ordinary case + temporary	1.00	25.47	〃	〃	81.50	〃	64.04
	Earthquake case	1.50	18.99	〃	〃	60.77	〃	43.31
end ⑧	ordinary case	1.00	10.19	〃	〃	32.61	〃	15.15
	ordinary case + temporary	1.00	8.56	〃	〃	27.39	〃	9.93
	Earthquake case	1.50	12.38	〃	〃	39.62	〃	22.16

(2) Stress calculation

middle ⑦



$M = 75.12$ t·m

$Be = 80 + 2 \times 102.4$

$= 284.8$ cm

$be = 284.8 - 80$

$= 204.8$ cm

$As = D22-15^{cm}\,ctc = 3.871 \times \frac{204.8}{15} = 52.85\ cm^2$

$P = \frac{52.85}{204.8 \times 102.4} = 0.00252$

From the Nomogram

$1/Lc = 9.07 \qquad 1/Ls = 432$

$\therefore \sigma_c = \frac{75.12 \times 10^5}{204.8 \times 102.4^2} \times 9.07 = 31.7\ kg/cm^2 < \sigma_{ca} = 90\ kg/cm^2$

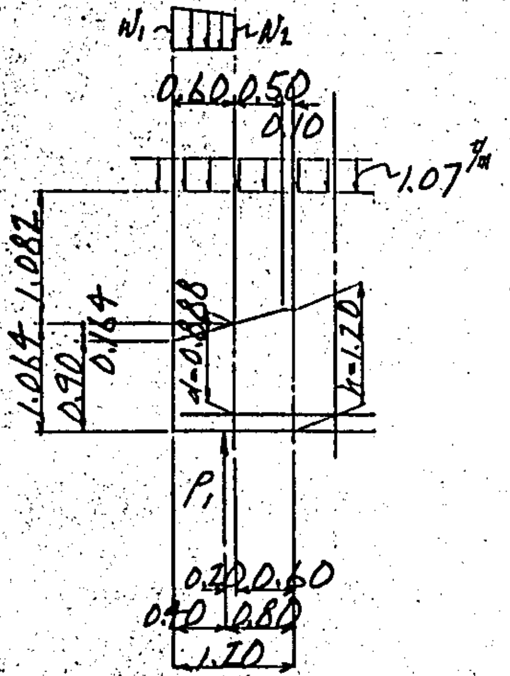
$\therefore \sigma_s =$ 〃 $\times 432 = 1511$ 〃 $< \sigma_{sa} = 1800$ 〃

check, safe against crocking

$M = 48.75$ t·m

$\therefore \sigma_s = \frac{48.75 \times 10^5}{204.8 \times 102.4^2} \times 432 = 981\ kg/cm^2 < \sigma_{sa} = 1400\ kg/cm^2$

(3.) Calculation of shearing fore caused by beading



$a = 0.60^m$

$d = 0.888^m$

$\therefore \gamma_1 = \frac{2}{a/d} = \frac{2 \times 0.888}{0.60}$

$= 2.960 < 4$

(i) Load

$W_1 = (2.5 \times 0.90 + 1.8 \times 1.246 + 1.07) \times \frac{4.30}{2.960}$

$= 8.08\ ^{t/m}$

$W_2 = (2.5 \times 1.064 + 1.8 \times 1.082 + 1.07) \times \frac{4.30}{2.960}$

$= 8.25\ ^{t/m}$

(ii) Reaction of pile

$P_1 = 28.93^t$

$P'_1 = 28.93 \times 4 \times \frac{1}{2.960} = 39.09^t$

(iii) Shearing

$\Sigma S = 1/2 \times (8.08 + 8.25) \times 0.60 - 39.09 = -34.20^t$

(iv) Shearing stress

$S = 34.20^{t}$

$\therefore \tau = \frac{34.20 \times 10^3}{430 \times 88.8} = 0.90^{kg/cm^2} < \tau a = 3.9^{kg/cm^2}$

(4)

1. Distri buting bar

mare than 1/6 of main reinforcement.

main reinforcement D22-6.67 (ctc15) × 1/6 = 4.30^{cm^2}

distributing bar D25-6.67 (ctc 15) (D22) = 33.80 " (25.82 ")

2. ToP side

more than 1/6 of Botton reinforcement.

D22-6.67(ctc15) × 1/6 = 4.30^{cm^2}

ToP reinfor cement D16-3.33 (ctc30) = 6.61 "

3. Ration of tensile reinforcement.

0.3 % over

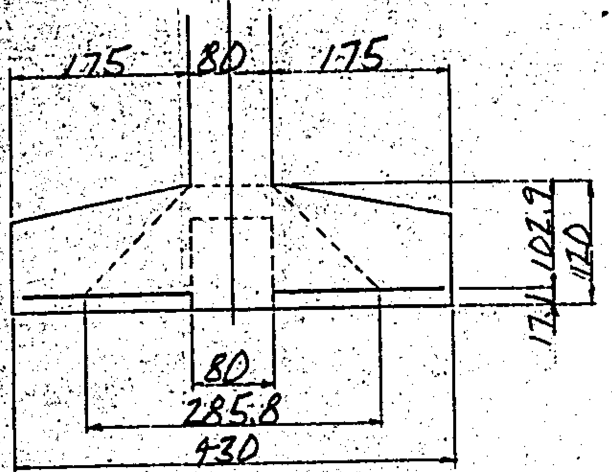
$p = 0.00252 = 0.252\% \ngtr 0.3\%$

$S = 34.20 \times 2.96 = 101.23^{t}$ $\tau = \frac{101.23 \times 10^3}{430 \times 88.8} = 2.65^{kg/cm^2}$

$\tau a_d = 3.9 \times \sqrt{\frac{0.252}{0.3}} = 3.57^{kg/cm^2} > 2.65^{kg/cm^2}$

(5) Stress Calculation

end ⑤



$M = 24.59^{\,t \cdot m}$

$Be = 80 + 2 \times 102.9$

$= 285.8^{\,cm}$

$be = 285.8 - 80$

$= 205.8^{\,cm}$

$As = D16 - 15^{cm} ctc = 1.986 \times \frac{205.8}{15} = 27.25^{\,cm^2}$

$p = \frac{27.25}{205.8 \times 102.9} = 0.00129$

From the Nomogram

$1/L_c = 11.9 \qquad 1/L_s = 825$

$\therefore \sigma_c = \frac{24.59 \times 10^5}{205.8 \times 102.9^2} \times 11.9 = 13.4 \ kg/cm^2 < \sigma_{ca} = 90 \ kg/cm^2$

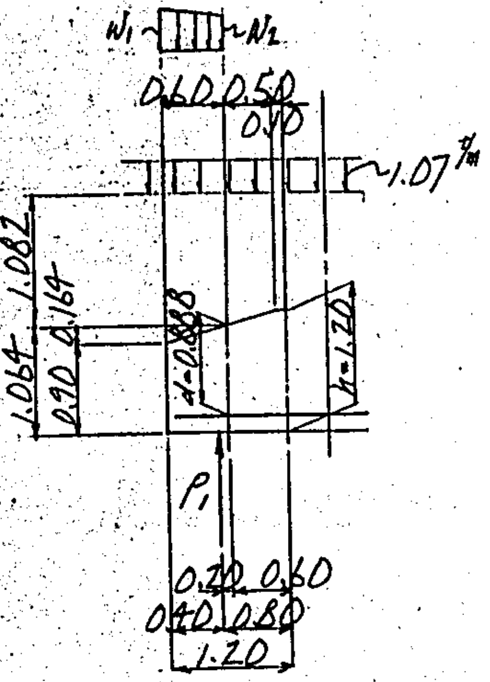
$\therefore \sigma_s = \quad " \quad \times 825 = 931 \ " < \sigma_{sa} = 1800 \ "$

Check, safe against crocking

$M = 22.64^{\,t \cdot m}$

$\therefore \sigma_s = \frac{22.64 \times 10^5}{205.8 \times 102.9^2} \times 825 = 857 \ kg/cm^2 < \sigma_{sa} = 1400 \ kg/cm^2$

(b) Calculation of shearing fore caused by beading



$a = 0.60^m$

$d = 0.888^m$

$\therefore \gamma_1 = \dfrac{2}{a/d} = \dfrac{2 \times 0.888}{0.60}$

$= 2.960 < 4$

(i) Load

$W_1 = (2.5 \times 0.90 + 1.8 \times 1.246 + 1.07) \times \dfrac{3.20}{2.960}$

$= 8.08$ t/m

$W_2 = (2.5 \times 1.064 + 1.8 \times 1.082 + 1.07) \times \dfrac{3.20}{2.960}$

$= 8.25$ t/m

(ii) Reaction of pile

$P_1 = 13.14^t$

$P'_1 = 13.14 \times 4 \times \dfrac{1}{2.960} = 17.76^t$

(iii) Shearing

$\Sigma S = \frac{1}{2} \times (8.08 + 8.25) \times 0.60 - 17.76 = -12.86^t$

(iV) Shearing stress

$S = 12.86^{t}$

$\therefore \tau = \frac{12.86 \times 10^3}{430 \times 88.8} = 0.34^{kg/cm^2} < \tau a = 3.9^{kg/cm^2}$

(4)

1. Distri buting bar

mare than 1/6 of main reinforcement.

main reinforcement D16 - 6.67 (ctc15) × 1/6 = 2.21 $^{cm^2}$

distributing bar D22 - 6.67 (ctc 15) = 25.82 "
(D25) (33.80) "

2. ToP side

more than 1/6 of Bottom reimforcement.

D16 - 6.67(ctc15) × 1/6 = 2.21 $^{cm^2}$

ToP reimfor cement D16 - 3.33 (ctc30) = 6.61 "

3. Ration of tensile reinforcement.

0.3 % over

$p = 0.00129 = 0.129\% \ngtr 0.3\%$

$S = 12.86 \times 2.960 = 38.07^{t} - \tau = \frac{38.07 \times 10^3}{430 \times 88.8} = 1.00^{kg/cm^2}$

$\tau_{ad} = 3.9 \times \sqrt{\frac{0.129}{0.3}} = 2.56^{kg/cm^2} > 1.00^{kg/cm^2}$