MATERIAL LIST
FOR
EIAN BASHI
OVER
YOSHII-GAWA

OKAYAMA PREFECTURE

表紙共二十二枚

表

提出原図

Material list for Yeian-Bashi Okayama-Ken

Mark	Description	No.	Section in Mm.	Length M.	Length Mm.	Wt of One M. in kgs	Wt of Main Section	Wt of Details.
			Main girder	MG 1 $^{R}_{L}$		2	Req'd.	
	Web Pls	2	1,200x9		9165	8478	1,554.0	
	Flg Ls	8	150x150x11		9170	2495	1,830.3	
	Cov Pls	4	340x13		5590	34697	775.8	
	End Stif Ls	8	125x90x10		1188	1609		152.9
	Fills	8	90x11		905	7772		56.3
	Stif Ls	26	125x75x10		1210	1491		469.1
	" "	2	"		1188	"		35.4
	Fills	2	75x11		905	6476		11.7
	Stif L	1	100x75x10		1158	1295		15.0
	" "	1	125x75x10		1158	1491		17.3
	Fills	2	75x15		880	8831		15.5
	Spl. pls	4	165x11		930	14248		53.0
	" "	2	480x11		566	41448		46.9
	" "	2	340x13		1,080	34697		74.9
	Spl. Ls	4	150x140(cut 150x150)x15		930	3355		124.8
	pls	2	225x13		325	22961		14.9
	Sole pls	2	370x19		530	55186		58.5
	Ls	2	90x60x9		145	996		2.9
	"	4	"		295	"		11.8
	"	2	"		220	"		4.4
	"	2	90x75x9		230	1102		5.1
	Bed pls	2	400x35		530	1099		116.5
	Ls	2	75x75x9		350	996		7.0
	Anchor bolts	6	32φ		700	@ 52		31.2
	washers	6	150x9		150	10598		9.5
							4160.1	+ 1334.6
								= 5494.7
								x 2
								10,989.4
			Floor beam	FB1		4	Req'd.	
	Web. Pl.	1	670x8		4830	42076	203.2	
	Flg. Ls	2	75x75x9		4830	996	96.2	
	" "	2	"		4570	"	91.0	
	End stif Ls	2	130x90(cut 150x90)x9		520	1632		17.0
	Stif Ls	8	75x65x8		680	828		45.0
	pls	2	320x9		340	22608		15.4

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Material list for Yeian-Bashi, Okayama-Ken.

Mark	Description	No.	Section in ins.	Length ft.	Length ins.	Wt. of One ft. in lbs.	Wt. of One Piece in lbs.	Total wt. in lbs.
	Weight of Rivet Heads for Girder spans (1 Span)							
	3.860		Shop rivet head		22φ	@0.0964	372.1	
	990		Field " "		"	@ "	95.4	
	1.430		Shop " "		19φ	@0.0646	92.4	
	780		Field " "		"	@ "	50.4	
							610.3	

照査者	設計者	
		材料計算

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Material list for Yeian-Bashi, Okayama-Ken

Mark	Description	No.	Section in Mm.	Length		Wt. of One M. in Kigs	Wt. of Main Section	Wt of Details
				M.	Mm.			
							390.4 +	77.4
								= 467.8
								x 4
								1,871.2
				FB 2.		1 Reqd.		
	Web pl.	1	670x8	4	800	42076	202.0	
	Flg Ls.	2	75x75x9	4	800	996	95.6	
	"	2	"	4	540	"	90.4	
	End stif Ls	2	140x90(cut 150x90)x9		510	1632		16.6
	Stif Ls	8	75x65x8		680	828		45.0
	Pls	2	320x9		340	22608		15.4
							388.0 +	77.0
								= 465.0
								x 1
								465.0
				Stringer		1 Reqd.		
S1.	Is	2	300x150@4834	4	775		461.6	
S2	"	2	"	4	755		459.7	
S3	"	4	"	4	390		848.9	
	Connection Pls	12	230x9		320	1625		62.4
							1770.2 +	62.4
								= 1832.6
								x 1
								1832.6
			Lateral bracing & gusset pls.				1 Req'd.	
LB1	Ls	4	125x75x10	5	550	14.91	331.0	
LB2	"	8	"	3	290	"	392.4	
	Pls	4	270x9		510	19.076		38.9
P1&P2	"	4	345x9		555	24374		54.1
P3&P4	"	4	345x9		615	"		60.0
P5&P6	"	2	330x9		630	23315		29.4
							723.4 +	182.4
								= 905.8
								x 1
								905.8

②

Material list for Yeian-Bashi. Okayama-Ken

Summary of Wt. for 1 girder span

Main girders	10,989.4
Floor beams	2,336.2
Stringers	1,832.6
Lateral bracings & Gussets	905.8
	16,064.0
Rivet heads	610.3
	16,674.3

Summary for 7 spans

16,674.3 × 7 = 116,720.1

Brackets for Lamppost 6 Req'd.

Mark	Description	No.	Section in [illegible]	Length	Wt. of One [illegible]	Wt. [illegible]	[illegible]
	L	1	75×75×9	460	9 96		4.6
	〃	1	〃	370	〃		3.7
	pl.	1	380 × 8	460	23 864		11.0
							19.3
							× 6
							115.8

Grand Summary of Wt. for 7 girder spans (Rivet head ヲ含ム)

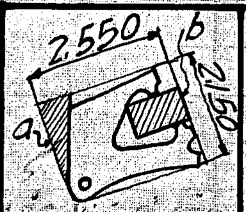
116,720.1
115.8 kg. tons
116,835.9 or 116.8359

③

照査者 設計者 材料計算

1.

Material list for Yeian-Bashi, Okayama-Ken

Mark	Description	No.	Section in Mm.	Length		Wt. of One M. in kgs.	Wt. of Main Section	Wt. of Details
				M.	Mm			
			TOP CHORD TC1				4 Required.	
							kgs	kgs.
	Cov pl.	1	560 · 8	6	650	35.168	233.9	
	Ls	2	150×100×9	6	750	17.020	229.8	
	p/s	2	250 × 9	4	900	17.663	173.1	
	Ls	2	150×100×12	6	250	22.410	280.1	
	p/s	2	250 × 9	3	980	17.663	140.6	
	Ls	2	90×75×9	1	740	11.020		38.3
	"	2	"	1	810	" "		39.9
	pl	1	315 × 9	1	740	22.255		38.7
	Ls	2	75×75×9		304	9.960		6.1
	Fills	2	75 × 9		165	5.299		1.7
	Ls	2	90×90×10	1	185	13.340		31.6
	Fills	2	90 × 4		210	2.826		1.2
	Ls	2	150×100×12	1	580	22.410		70.8
	Tie pl	1	315 × 9		380	22.255		8.5
	Ls	2	150×100×12	1	800	22.410		80.7
	Tie pl.	1	315 × 9		380	22.255		8.5
	p/s	2	2.150 × 9	2	550	151.898	 a=43.2 b=44.5 87.7 2 175.4 774.7−175.4 =559.3	559.3
	"	2	300 × 9	2	060	21.195		87.3
	"	2	740 × 13	1	440	75.517		217.5
	pin p/s	4	380 × 13		545	38.779		84.5
	Tie pl	1	455 × 9		540	32.140		17.4
	~~Ls~~	~~2~~	~~150×100×9~~		~~410~~	~~17.020~~		
	Fills	2	90 × 13		250	9.185		4.6
	p/s	4	250 × 9		580	17.663		41.0
	"	4	100 × 9		380	7.065		10.7
	Ls	4	75×75×9	1	120	9.900		44.6
	Tie p/s	2	210 × 9		315	14.837		9.3
	Lac bars	3	60 × 9		320	4.239		4.1
	Ls	4	75×75×9	1	860	9.960		74.1
	"	4	"	1	410	" "		56.2
	"	4	"	1	780	" "		70.9
	"	4	"	1	370	" "		54.6
	"	4	"	1	710	" "		68.1
	"	4	"	1	330	" "		53.0
	"	4	"	1	290	" "		51.4
	Fills	2	150 × 9		350	10.598		7.4
	Tie p/s	2	160 × 8		275	10.048		5.5

照査者 設計者 材料計算

Material list for Yuian Bashi, Okayama Ken

Mark	Description	No.	Section in Mm.	Length		Wt. of One M. in kgs.	Wt. of Main Section	Wt. of Details.
				M.	Mm.			
	Tie p/s	4	160 × 8		290	10.048		11.7
	Tie pl.	1	530 × 9		540	37.445		20.2
	Tie p/s	3	305 × 9		"	21.548		34.9
	Lac. bars	4	70 × 11		690	6.045		16.7
	" "	2	"		730	"		8.8
	Washers	4	70φ × 11			@ 0.332		1.3
	pl	1	560 × 11		640	48.456		31.0
	Fill	1	320 × 3		560	7.530		4.2
	Ls	2	150×100×12		720	22.410		32.3
	p/s	2	250 × 9		480	17.663		17.0
	"	2	100 × 9		320	7.065		4.5
	Fills	2	130 × 3		360	3.063		2.2
	"	2	80 × 3		"	1.884		1.4
	Ls	2	150×100×15		720	27.670		39.8
	p/s	2	250 × 9		480	17.663		17.0
	"	2	100 × 9		320	7.065		4.5
	pl	1	320 × 9		540	22.608		12.2
	Fills	2	130 × 3		360	3.063		2.2
	"	2	80 × 3		"	1.884		1.4
	L	1	100×75×10		"	12.950		4.7
							1057.5	+2115.5
								=3173.0
								× 4
								12,692.0
				TCZ		4	Required.	
	Cov pl.	1	560 × 11	5.	500	48.456	266.5	
	Ls	2	150×100×12	5.	500	22.410	240.5	
	p/s	2	250 × 9	5.	480	17.663	193.6	
	Ls	2	150×100×15	5	600	27.670	309.9	
	p/s	2	250 × 9	5	600	17.663	197.8	
	Ls	4	75×75 × 9	1	580	9.960		62.9
	"	4	"	1	260	" "		50.2
	"	4	"	1	530	" "		61.0
	"	4	"	1	230	" "		49.0
	"	4	"	1	470	" "		58.6
	"	4	"	1	200	" "		47.8
	"	4	"	1	410	" "		56.2

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照査者	設計者	
		材料計算

Material list for Yeian Bashi - Okayama-Ku

Mark	Description	No.	Section in Mm.	Length M.	Length Mm.	Wt. of One M. in kgs.	Wt. of Main Section	Wt. of Details.
	∠s	4	75×75×9	1	160	9 960		46.2
	"	4	"	1	140	" "		45.4
	Tie p/s	8	160 × 8		290	10.048		23.3
	" "	4	230 × 9		540	16.250		35.1
	Tie pl.	1	305 × 9		"	21.548		11.6
	Lac. bars	12	70 × 11		690	6.045		50.1
	" "	2	"		730	" "		8.8
	Washers.	8	70φ × 11			@ 0.332		2.7
	pl	1	560 × 11		640	48 450		31.0
	∠s	2	150×100×15		720	27 670		39.8
	p/s	2	250 × 9		480	17.663		17.0
	"	2	100 × 9		320	7.065		4.5
	∠s	2	150×100×15		720	27 670		39.8
	p/s	2	250 × 9		480	17.663		17.0
	"	2	100 × 9		320	7.065		4.5
	pl	1	320 × 9		540	22.608		12.2
							1,214.3	+ 774.7
								=1,989.0
								× 4
								7,956.0
					TC3	4	Required	
	Cov pl	1	560 × 11	6	300	48 456	305.3	
	∠s	2	150×100×12	6	300	22 410	282 4	
	p/s	2	250 × 9	6	250	17.663	220 8	
	∠s	2	150×100×15	6	350	27 670	351.4	
	p/s	2	250 × 9	6	350	17 663	224.3	
	∠s	4	75×75×9	1	300	9 960		51.8
	"	4	"	1	120	" "		44.6
	"	4	"	1	260	" "		50.2
	"	4	"	1	110	" "		44.2
	"	4	"	1	220	" "		48.6
	"	4	"	1	100	" "		43.8
	"	4	"	1	180	" "		47.0
	"	4	"	1	090	" "		43.4
	"	4	"	1	080	" "		43.0
	"	4	"	1	130	" "		45.0
	"	4	"	1	070	" "		42.6

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照査者 設計者 材料計算

Material list for Yeian-Bashi Okayama-Ken

Mark	Description	No.	Section in Mm.	Length		Wt. of One M. in kgs.	Wt. of Main Section	Wt. of Details.
				M.	Mm.			
	Tie pls	10	160 × 8		290	10.048		29.1
	" "	6	230 × 9		540	16.250		52.7
	Tie pl	1	290 × 9		540	20.489		11.1
	Lac bars	12	70 × 11		660	6.045		47.9
	" "	4	"		730	" "		17.7
	Washers	10	70φ × 11			@ 0.332		3.3
	pl.	1	560 × 11		640	48.450		31.0
	∠s	2	150×100×15		720	27.070		39.8
	pls	2	250 × 9		480	17.663		17.0
	"	2	100 × 9		320	7.065		4.5
	∠s	2	150×100×15		720	27.670		39.8
	pls	2	250 × 9		480	17.663		17.0
	"	2	100 × 9		320	7.065		4.5
	pl	1	320 × 9		540	22.608		12.2
							1.384.2	+ 837.8
								=2216.0
								× 4
								8,864.0
					TC4	4	Required	
	Cov pl	1	560 × 11	6	150	48.450	298.0	
	∠s	2	150×100×12	6	150	22.410	275.6	
	pls	2	250 × 9	6	100	17.663	215.5	
	∠s	2	150×100×15	6	150	27.670	340.3	
	pls	2	250 × 9	6	100	17.663	215.5	
	∠s	4	75×75 × 9	1	100	9.960		43.8
	"	16	"	1	070	" "		170.5
	"	4	"	1	050	" "		41.8
	"	8	"	1	060	" "		84.5
	"	4	"	1	010	" "		40.2
	"	4	"	1	000	" "		39.8
	"	4	"	1	080	" "		43.0
	Tie pls	10	160 × 8		290	10.048		29.1
	" "	6	230 × 9		540	16.250		52.7
	Lac bars	8	70 × 11		660	6.045		31.9
	" "	6	"		730	" "		26.5
	Washers	10	70φ × 11			@ 0.332		3.3
	Tie pl	1	290 × 9		540	20.489		11.1

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照査者 設計者
材料 計算

Material list for Yeiah-Bashi Okayama-Ken

Mark	Description	No.	Section in Mm.	Length		Wt. of One M. in kgs.	Wt. of Main Section.	Wt. of Details
				M.	Mm.			
	pl	1	560 × 11		640	48.450		31.0
	Ls	2	150×100×15		720	27.670		39.8
	p/s	2	250 × 9		480	17.663		17.0
	"	2	100 × 9		320	7.065		4.5
	Ls	2	150×100×15		720	27.670		39.8
	p/s	2	250 × 9		480	17.663		17.0
	"	2	100 × 9		320	7.065		4.5
	pl	1	320 × 9		540	22.608		12.2
							1,344.9	+ 784.0
								=2,128.9
								4
								8,515.6
					TC5	2	Required.	
	Cov. pl.	1	560 × 17		4950	48.450	239.9	
	Ls	2	150×100×12		4950	22.410	221.9	
	p/s	2	250 × 9		4.900	17.663	173.1	
	Ls	2	150×100×15		5.200	27.670	287.8	
	p/s	2	250 × 9		" "	17.663	183.7	
	Ls	8	75×75×9		990	9.960		78.9
	"	8	"		980	" "		78.1
	"	16	"		1080	" "		172.1
	Tie p/s	8	160 × 8		290	10.048		23.3
	" "	6	230 × 9		540	16.250		52.7
	Lac bars	8	70 × 11		730	6.045		35.3
	Washers	8	70φ × 11			@ 0.332		2.7
							1,100.4	+ 443.1
								=1,549.5
								× 2
								3,099.0
			Summary for Top chord				41,126.6	

照査者	設計者	
		材料
		計算

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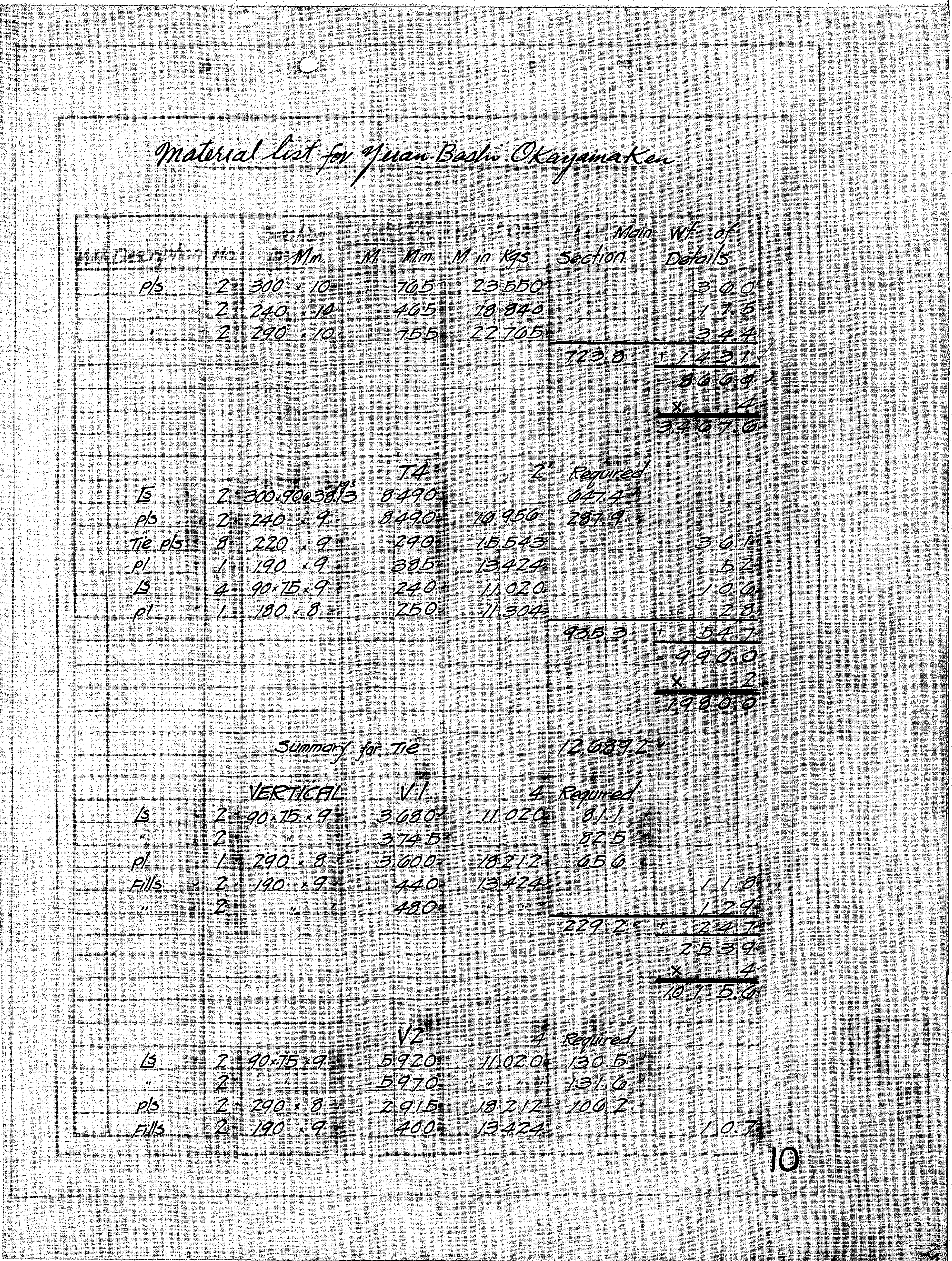
Material list for Yeion-Bashi Okayama-Ken

Mark	Description	No.	Section in Mm.	Length M.	Length Mm.	Wt. of One M. in kgs.	Wt. of Main Section	Wt. of Details
			TIE	T1		4	Required.	
	⌐s	2	300×90@38.13 kgs.	4	870		371.4	
	p/s	2	240 × 9	4	200	16.950	142.4	
	Tie p/s	2	290 × 9		370	20.489		15.2
	" "	3	220 × 9		290	15.543		13.5
	p/	1	190 × 9		385	13.424		5.2
	p/s	2	300 × 10		765	23.550		36.0
	"	2	240 × 10		465	18.840		17.5
	"	2	290 × 10		755	22.765		34.4
	∟s	4	90×75×9		240	11.020		10.6
	p/	1	180 × 8		250	11.304		2.8
							513.8	+ 135.2
								= 649.0
								× 4
								2,596.0
				T2		4	Required.	
	⌐s	2	300×90@38.13 kgs.	8	340		730.0	
	p/s	2	240 × 9	"	"	16.950	282.8	
	Tie p/s	8	220 × 9		290	15.543		36.1
	p/	1	190 × 9		385	13.424		5.2
	∟s	4	90×75×9		240	11.020		10.6
	p/	1	180 × 8		250	11.304		2.8
	p/s	2	300 × 10		765	23.550		36.0
	"	2	240 × 10		465	18.840		17.5
	"	2	290 × 10		755	22.765		34.4
							1,018.8	+ 142.0
								= 1,161.4
								× 4
								4,645.0
				T3		4	Required.	
	⌐s	2	300×90@38.13 kgs.	6	570		501.0	
	p/s	2	240 × 9	6	570	16.950	222.8	
	Tie p/s	4	220 × 9		290	15.543		18.0
	p/s	2	190 × 9		385	13.424		10.3
	∟s	8	90×75×9		240	11.020		21.2
	p/s	2	180 × 8		250	11.304		5.7

照査者
設計者

増田淳氏関係資料（独立行政法人 土木研究所蔵）



Material list for Yeian-Bashi Okayamaken

Mark	Description	No.	Section in Mm.	Length M	Length Mm.	Wt. of One M in kgs.	Wt. of Main Section	Wt of Details
	p/s	2	300 × 10		765	23.550		36.0
	"	2	240 × 10		465	18.840		17.5
	"	2	290 × 10		755	22.765		34.4
							723.8	+ 143.1
								= 866.9
								× 4
								3.467.6
					T4	2	Required	
	Ls	2	300×90@38.13 kgs		8490		647.4	
	p/s	2	240 × 9		8490	10.956	287.9	
	Tie p/s	8	220 × 9		290	15.543		36.1
	pl	1	190 × 9		385	13.424		5.2
	Ls	4	90×75×9		240	11.020		10.6
	pl	1	180 × 8		250	11.304		2.8
							935.3	+ 54.7
								= 990.0
								× 2
								1,980.0
			Summary for Tie				12,689.2	
			VERTICAL		V1.	4	Required	
	Ls	2	90×75×9		3680	11.020	81.1	
	"	2	"		3745	" "	82.5	
	pl	1	290 × 8		3600	18.212	65.6	
	Fills	2	190 × 9		440	13.424		11.8
	"	2	"		480	" "		12.9
							229.2	+ 24.7
								= 253.9
								× 4
								1,015.6
					V2	4	Required	
	Ls	2	90×75×9		5920	11.020	130.5	
	"	2	"		5970	" "	131.6	
	pls	2	290 × 8		2915	18.212	106.2	
	Fills	2	190 × 9		400	13.424		10.7

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増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Material list for Yeian Bashi, Okayama-Ken

Mark	Description	No.	Section in M.m.	Length M	Length M.m	Wt. of One M in kgs.	Wt. of Main Section	Wt of Details.
	Fills	2	190 × 9		450	13424		12.1
	p/s	2	145 × 8		285	9.106		5.2
							368.3	+ 28.0
								= 396.3
								× 4
								1585.2
				V	3	4	Required	
	Ls	2	90×75×9	7	520	11.020	165.7	
	"	2	"	7	555	" "	166.5	
	p/s	2	290 × 8	3	690	18.212	134.4	
	Fills	2	190 × 9		370	13.424		9.9
	"	2	"		400	" "		10.7
	p/s	2	145 × 8		285	9.106		5.2
							466.6	+ 25.8
								= 492.4
								× 4
								1969.6
				V	4	4	Required	
	Ls	2	90×75×9	8	490	11.020	187.1	
	"	2	"	8	510	" "	187.6	
	p/s	2	290 × 8	4	215	18.212	153.5	
	Fills	4	190 × 9		370	13.424		19.9
	p/s	2	145 × 8		285	9.106		5.2
							528.2	+ 25.1
								= 553.3
								× 4
								2213.2
				V	5	2	Required	
	Ls	4	90×75×9	8	800	11.020	387.9	
	p/s	2	280 × 8	4	370	17.584	153.7	
	Fills	4	190 × 9		330	13.424		17.7
	p/s	2	145 × 8		285	9.106		5.2
							541.6	+ 22.9
								= 564.5
								× 2

照査者	設計者	
		材料計算書

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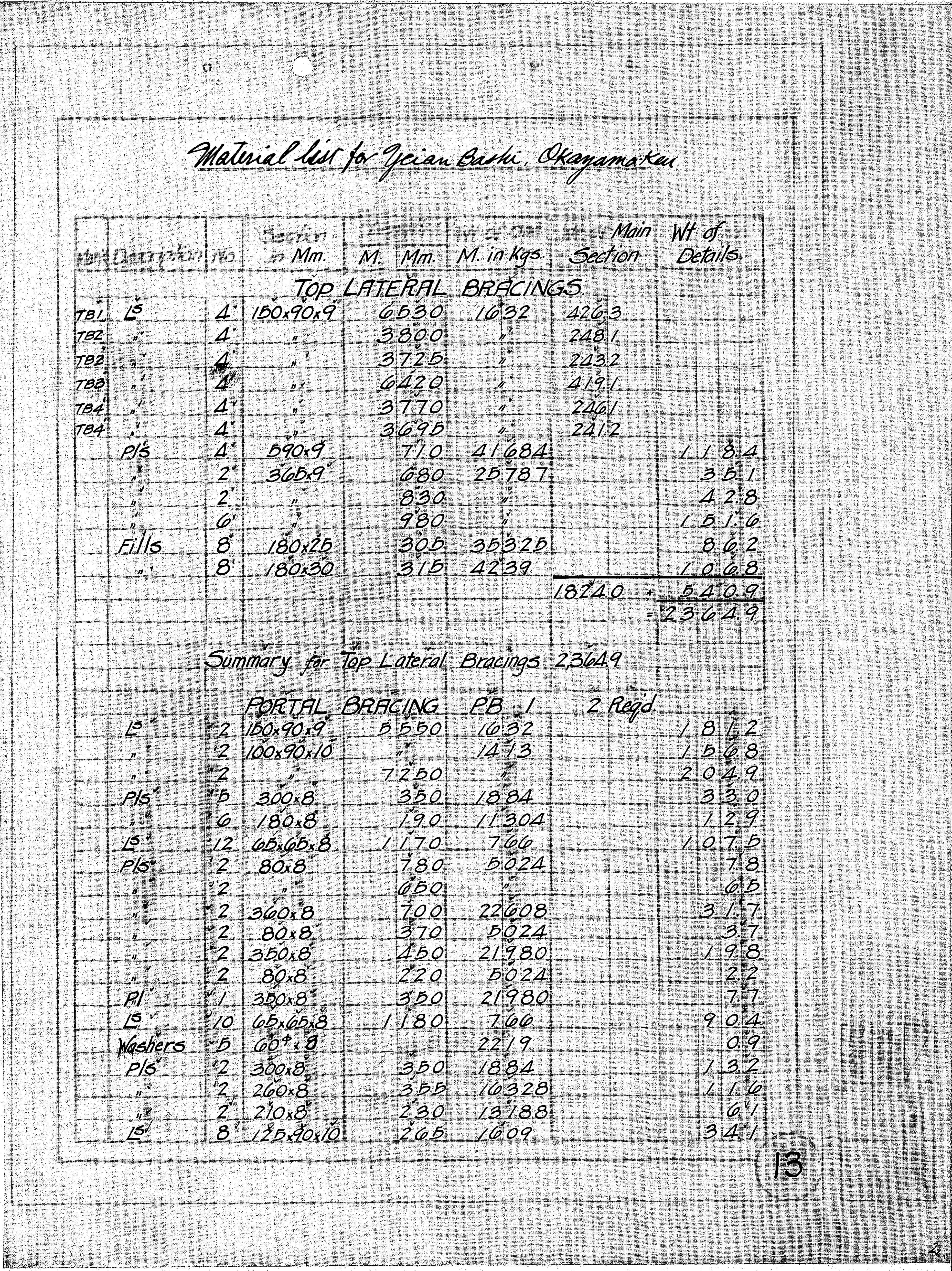
Material list for Yeian Bashi; Okayamaken

Mark	Description	No.	Section in Mm.	Length M	Length Mm	Wt. of One M. in Kgs.	Wt. of Main Section	Wt. of details.
								1,129.0
			Summary for Verticals				7,912.6	
			BOTTOM LATERAL BRACINGS					
BL1	Ls	2	150×90×9	6	510	16 320	212.5	
"	"	2	"	6	580	" "	214.8	
"	p/s	2	510 × 9		830	36 032		59.8
BL2	Ls	4	150×90×9	3	685	16 320	240.6	
BL3	"	4	"	3	870	" "	252.6	
BL4	"	8	"	6	640	" "	866.9	
"	p/s	4	510 × 9		690	36 032		99.4
BL5	Ls	16	150×90×9	3	870	16 320	1,010.5	
BL6	"	4	150×100×9	6	710	17 020	456.8	
"	p/s	4	380 × 9		580	26 847		62.3
BL7	Ls	8	150×100×9	3	960	17 020	539.2	
							3,793.8	+ 221.5
								= 4015.3
			BOTTOM LATERAL PLATES					
	p/s	2	480 × 9		840	33 912		57.0
	"	2	600 × 9		680	42 390		57.7
	"	2	640 × 9		800	45 216		72.3
	"	2	690 × 9		"	48 749		78.0
	"	4	640 × 9		780	45 216		141.1
	"	2	"		710	" "		64.2
	"	2	650 × 9		680	45 923		62.5
	"	6	570 × 9		650	40 271		157.1
								689.9
			Summary for Bottom Lateral Bracings & Plates				4,705.2	

照査者 設計者
材料計算



Material list for Yeian Bashi, Okayamaken

Mark	Description	No.	Section in Mm.	Length M.	Length Mm.	Wt. of One M. in Kgs.	Wt. of Main Section	Wt of Details.
			TOP LATERAL BRACINGS.					
TB1	Ls	4	150x90x9	6	530	1632	426.3	
TB2	"	4	"	3	800	"	248.1	
TB2	"	4	"	3	725	"	243.2	
TB3	"	4	"	6	420	"	419.1	
TB4	"	4	"	3	770	"	246.1	
TB4	"	4	"	3	695	"	241.2	
	Pls	4	590x9		710	41684		118.4
	"	2	365x9		680	25787		35.1
	"	2	"		830	"		42.8
	"	6	"		980	"		151.6
	Fills	8	180x25		305	35325		86.2
	"	8	180x30		315	4239		106.8
							1824.0 +	540.9
							=	2364.9
			Summary for Top Lateral Bracings 2,364.9					
			PORTAL BRACING			PB 1	2 Reqd.	
	Ls	2	150x90x9	5	550	1632		181.2
	"	2	100x90x10		"	1413		156.8
	"	2	"	7	250	"		204.9
	Pls	5	300x8		350	1884		33.0
	"	6	180x8		190	11304		12.9
	Ls	12	65x65x8	1	170	766		107.5
	Pls	2	80x8		780	5024		7.8
	"	2	"		650	"		6.5
	"	2	360x8		700	22608		31.7
	"	2	80x8		370	5024		3.7
	"	2	350x8		450	21980		19.8
	"	2	80x8		220	5024		2.2
	Pl	1	350x8		350	21980		7.7
	Ls	10	65x65x8	1	180	766		90.4
	Washers	5	60φx8			2219		0.9
	Pls	2	300x8		350	1884		13.2
	"	2	260x8		355	16328		11.6
	"	2	210x8		230	13188		6.1
	Ls	8	125x90x10		265	1609		34.1

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Material list for Yeian Bashi, Okayamaken

Mark	Description	No	Section in Mm.	Length M	Length Mm.	Wt. of One M. Kgs.	Wt. of Main Section	Wt of Details
	Fills	2	100x12		230	9.42		4.3
	„	2	185x8		860	11.618		20.0
	„	2	100x15		230	11.775		5.4
								961.7
								x 2
								1,923.4
			SWAY BRACING			SB 1	2 Reqd.	
	L^s	4	150x90x9		5550	16.32		362.3
	Fills	10	145x8		180	9.106		16.4
	L^s	12	65x65x8		1370	7.66		125.9
	Fills	6	60^{ϕ}x26		[illegible]	22.19		3.5
	L^s	8	125x90x10		150	16.09		19.3
	P/s	2	155x8		370	9.734		7.2
	„	2	145x8		320	9.106		5.8
	„	2	210x8		230	13.188		6.1
	L^s	4	100x90x10		2150	14.13		121.5
								668.0
								x 2
								1,336.0
			SB 2				1 Reqd.	
	L^s	4	150x90x9		5550	16.32		362.3
	Fills	10	145x8		200	9.106		18.2
	L^s	12	65x65x8		1350	7.66		124.1
	Fills	6	60^{ϕ}x26		[illegible]	22.19		3.5
	L^s	8	125x90x10		135	16.09		17.4
	P/s	2	155x8		370	9.734		7.2
	„	2	145x8		320	9.106		5.8
	„	2	210x8		230	13.188		6.1
	L^s	4	100x90x10		2150	14.13		121.5
								666.1
								x 1
								666.1
	Summary for Portal Bracings & Sway Bracings =							3,925.5

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照査者 設計者 材料

Material list for Yeian Bashi, Okayama Ken

Mark	Description	No.	Section in Mm.	Length		Wt. of One M. in Kgs.	Wt. of Main Section	Wt of Details.
				M	Mm.			
			FLOOR BEAM FB 1			2 Req'd.		
	Web Pl.	1	670x8		5065	42076	213.1	
	Flg Ls.	4	90x90x10		5560	1334	296.7	
	Ls	4	125x90x10		770	1609		49.6
	"	4	90x90x10		380	1334		20.3
	Pls	2	380x8		1155	23864		55.1
	Ls	4	75x75x9		660	9.96		26.3
	"	8	"		680	"		54.2
	Pls	4	160x10		490	1256		24.6
	"	4	150x9		255	10598		10.8
							509.8	+ 240.9
								= 750.7
								x 2
								1,501.4
			FB 2 & FB 3			9 Req'd.		
	Web Pl.	1	670x8		5065	42076	213.1	
	Flg Ls	2	90x90x10		5560	1334	148.3	
	" "	2	"		5885	"	157.0	
	Ls	4	150x90x9		790	1632		51.6
	Pls	2	410x8		975	25748		50.2
	Ls	4	75x75x9		660	996		26.3
	"	8	"		680	"		54.2
	Pls	4	160x10		490	1256		24.6
	"	4	150x9		255	10598		10.8
							518.4	+ 217.7
								= 736.1
								x 9
								6,624.9
	Summary for Floor Beams.					8,126.3		
			STRINGERS					
S1	Is	2	300x150@48.34 kgs.		5385		520.6	
S2	"	2	"		5350		517.2	
S3	"	2	"		5320		514.3	
S4	"	2	"		5285		511.0	
S5	"	32	"		4990		7,718.9	

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Material list for Yeian Bashi, Okayamaken

			Section	Length		Wt. of One	Wt. of Main	Wt of
Mark	Description	No.	in Mm.	M.	Mm.	M in Kgs	Section	Details.
	Pls	72	230×9		320	16.25		374.4
							9,782.0 +	374.4
								= 10,156.4
	Summary for Stringer						10.156.4	
	SHOES							
RS1	cast steel shoe.	2				@ 186.0		372.0
	cast steel dust guard.	4				@ 18.5		74.0
	cast steel bed pls.	2				@ 168.0		336.0
	Dust guard pls	4	135 × 8		680	8.478		23.1
	Tapped bolts	20	7φ		025	@ 0.014		0.3
	Bolts	8	22φ		085	@ 0.539		4.3
	Rollers	8	100φ		610	61.650		300.9
	pls	4	70 × 13		400	7.144		11.4
	pins	8	25φ		065	@ 0.20		1.6
		8	"		045	@ 0.18		1.4
FS1	Cast steel shoes	2				@ 342.0		684.0
PN1	Pins	4	120φ		580	88.78		206.0
"	Nuts	8				@ 2.09		16.7
AB1	Anchor bolts	16	32φ		800	@ 5.75		92.0
	pls	16	150 × 9		150	10.598		25.4
								2,149.1
						(Cast steel only		1,466.0)
	Summary of Wt. for 1-Truss Span							
	Top chord						41,126.6	
	Tie						12,689.2	
	Verticals						7,912.6	
	Bottom lateral bracing & plates						4,705.2	
	Top lateral bracings						2,364.9	
	Potal bracings and Sway bracings						3,925.5	
	Floor beams						8,126.3	
	Stringers						10,156.4	
							91,006.7	
	Shoes						2,149.1	96,542.0
	Rivet heads See Page no. 20.						3,386.2	− 1,466.0
							96,542.0	95,076.0

Material list for Yeian-Bashi, Okayama-ken

Materials for through the Bridge

Mark	Description	No.	Section in Mm.	Length M.	Length Mm.	Wt. of One M. in kgs.	Wt. of Main Section	Wt. of Details.
			EXPANSION	JOINT		EJ1	1 Req'd.	
	Checkered pls	1	251x9	5	545	19 978		110.8
	L	1	75x65x8	5	535	8 28	A. 114.1	45.8
	"	1	"	5	520	"		45.7
	pls	2	175x8		370	10 99		8.1
	"	2	170x8		200	10 676		4.3
	"	2	170x8		210	"		4.5
	pl	1	170x8		215	"		2.3
	Bolts	8	19ϕ		55	@ 0.3131		2.5
	Beveled washers	4	60x8		60	3 768		0.9
								224.9
								x 1
								224.9
				EJ	2		1 Req'd.	
	L	1	125x75x10	5	545	14 91	B. 170.5	82.7
	L	1	75x65x8	5	520	8 28		45.7
	bar	1	50x11	5	510	4 318		23.8
	pls	2	165x8		370	10 362		7.7
	pls	2	170x8		190	10 676		4.1
	"	2	170x8		200	"		4.3
	pl	1	170x8		205	"		2.2
	Anchor bolts	7	19ϕ		300	@ 0 84		5.9
								176.4
								x 1
								176.4
				EJ	3		6 Req'd.	
	Checkered pl	1	227x9	5	540	18 068		100.1
	A							114.1
								214.2
								x 6
								1,285.2

照査者 設計者

材料計算

2

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Material list for Yeian-Bashi, Okayama-ken

Mark	Description	No.	Section in Mm.	Length M.	Length Mm.	Wt. of One M. in kgs.	Wt. of Main Section	Wt. of Details
				E J	4.	6	Reqd.	
	B.							170.5
	Bolts.	8	19ϕ		55	@ 3131		2.5
	Beveledwasher	4	60x8		60	3768		0.9
								173.9
								x 6
								1,043.4
				E J	5	3	Reqd.	
	Checkered Pl.	1	236x9	5	540	18784		104.1
	L	1	75x65x8	5	535	828	C.1150	45.8
	L	1	〃	5	520	〃		45.7
	Pls	2	175x8		370	1099		8.1
	〃	2	170x8		200	10676		4.3
	〃	2	170x8		210	〃		4.5
	Pl	1	170x8		215	〃		2.3
	Bolts	8	19ϕ		55	@ 3131		2.5
	Beveledwashers	8	60x8		60	3768		1.8
								219.1
								x 3
								657.3
				E J	6	3	Req'd.	
	L	1	150x90x9	5	545	1632		90.5
	L	1	75x65x8	5	520	828	D.1772	45.7
	Bar	1	50x11	5	510	4318		23.8
	Pls	2	155x8		370	9734		7.2
	〃	2	170x8		180	10676		3.8
	〃	2	170x8		190	〃		4.1
	Pl	1	170x8		195	〃		2.1
	Bolts	8	19ϕ		55	@ 3131		2.5
	Beveledwashers	8	60x8		60	3768		1.8
								181.5
								x 3
								544.5

設計者

材料計算

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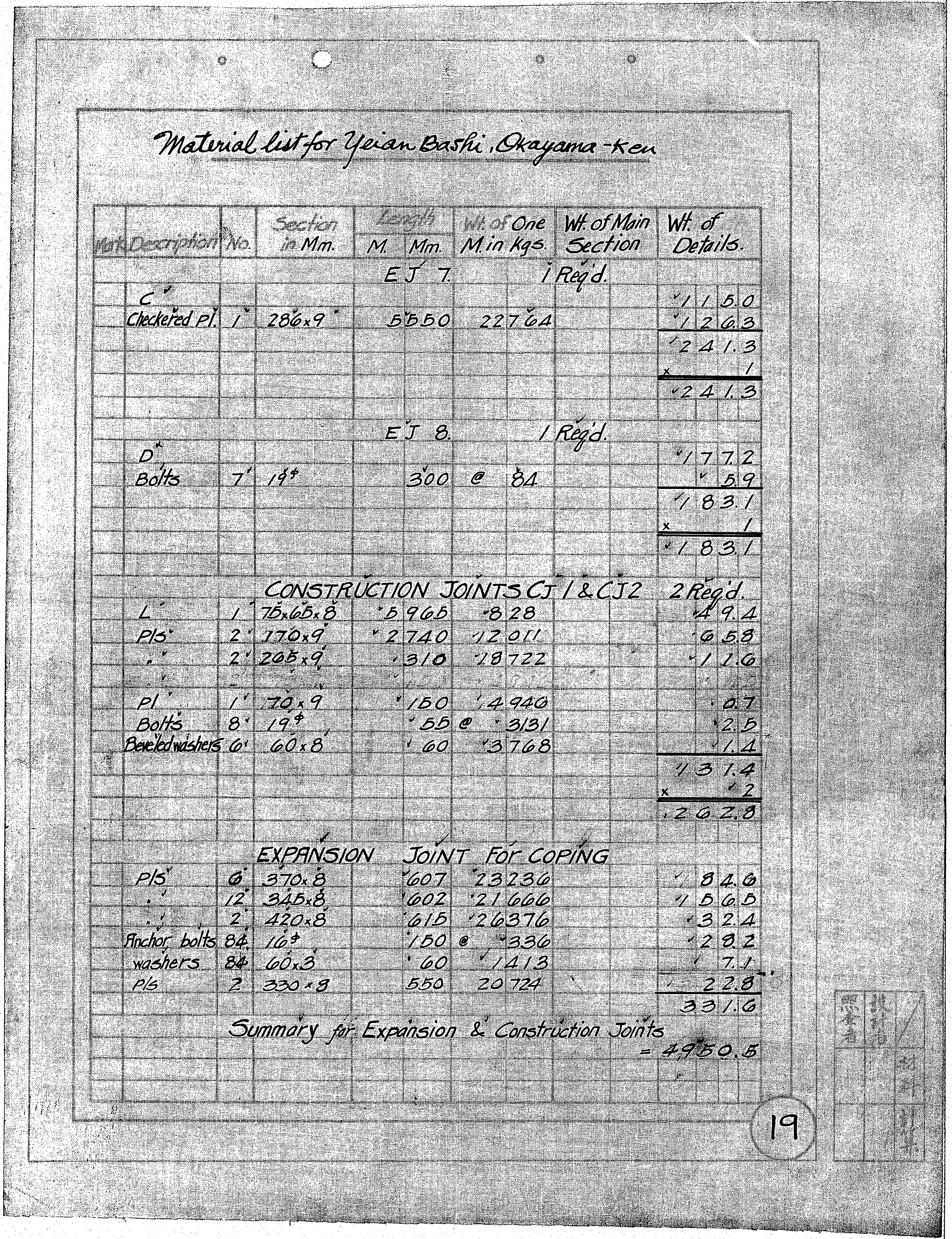

Material list for Yeian Bashi, Okayama-ken

Mark	Description	No.	Section in Mm.	Length		Wt. of One M. in Kgs.	Wt. of Main Section	Wt. of Details.
				M.	Mm.			
				EJ	7.		1 Req'd.	
	C							115.0
	Checkered Pl.	1	286×9		5550	227.64		126.3
								241.3
								× 1
								241.3
				EJ	8.		1 Req'd.	
	D							177.2
	Bolts	7	19ϕ		300	@ 84		5.9
								183.1
								× 1
								183.1
			CONSTRUCTION	JOINTS	CJ 1	& CJ2		2 Req'd.
	L	1	75×65×8		5965	8.28		49.4
	Pls	2	170×9		2740	12.011		65.8
	"	2	265×9		310	18.722		11.6
	Pl	1	70×9		150	4.946		0.7
	Bolts	8	19ϕ		55	@ 3.131		2.5
	Beveled washers	6	60×8		60	3.768		1.4
								131.4
								× 2
								262.8
			EXPANSION	JOINT	For	COPING		
	Pls	6	370×8		607	23.236		84.6
	"	12	345×8		602	21.666		156.5
	"	2	420×8		615	26.376		32.4
	Anchor bolts	84	16ϕ		150	@ .336		28.2
	washers	84	60×3		60	1.413		7.1
	Pls	2	330×8		550	20.724		22.8
								331.6

Summary for Expansion & Construction Joints = 4,950.5

照査者 設計者

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増田淳氏関係資料（独立行政法人 土木研究所蔵）

Material list for Yeian Bashi, Okayama ken.

Mark	Description	No.	Section in ins	Length ft	Length ins	Wt. of One ft in lbs.	Wt. of One Piece in lbs.	Total wt. in lbs.

Weight of Rivet Heads for Truss Spans. 4 Req'd

2,240	Shop rivet head	22φ	@ 0.0964	215.9
9,410	Field " "	"	@ "	907.1
29,840	Shop " "	19φ	@ 0.0646	1,927.7
3,110	Field " "	"	@ "	200.9
3,450	Shop " "	16φ	@ 0.0390	134.6
				3,386.2 kgs
				× 4
				13,544.8 kgs.

Weight of Rivet Heads for Expansion Joints 1 Req'd

2,180	Shop rivet head	19φ	@ 0.0646	140.8
820	" " "	16φ	@ 0.0390	32.0
				172.8 kgs

Summary 13,717.6 kgs.
or 13.7176 kg tons.

Summary for Expansion joints & Construction joint

Expansion & Construction joint	4,950.5
Rivet heads	172.8
	5,123.3 or 5.1233 kg tons

Grand Summary of Weights (Rivet heads ヲ含ム)

	Kg. Tons	
Girders for 7 Spans	116.8359	
Truss Span 95.0760 × 4	380.3040	
Cast Steel 1.4660 × 4	5.8640	For Girder Spans 3.1600
Expansion & Const. Joints	5.1233	" Truss Spans 1.9633
	508.1272	

増田淳氏関係資料（独立行政法人 土木研究所蔵）

岡山縣吉井川架橋

永安橋設計々算書

及材料調書

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

MADE BY J. Masuda DATE 4-4-9 FILE NO ______
CHECKED BY M. Jinda DATE 4-5-21 PAGE NO 1

CALCULATIONS FOR

Design of Eian-Bashi for Okayama-Ken

General layout of the bridge

4 spans @ 50.0 meters c to c of End bearings on right bank
7 spans @ 18.35 out to out of girders

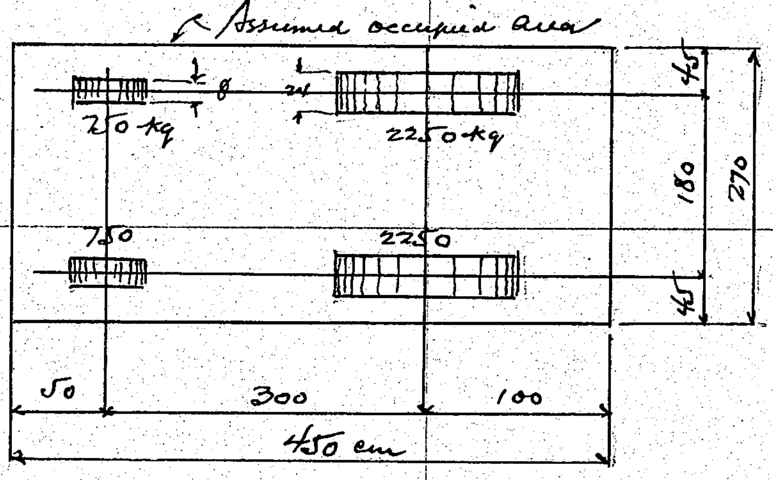
Total length of bridge between parapet walls of abutments 331.30 meters
Clear Roadway 5.4 meters pavement 3.8 cm asphaltic block on reinforced concrete slab.
Pier and abutment will be 10° skew

Assumed loadings

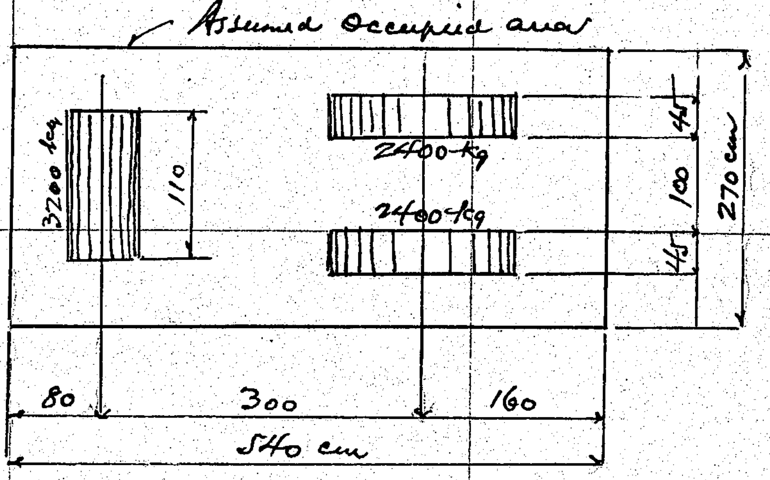
Uniform load on Roadway $w = \frac{100.000}{170+l} \leqq 500$ kg/m²

where w = Uniform load in kg per sq meters
l = span length in meter

6 ton motor truck loading



8 ton road roller



Two rows of motor truck traffic on roadway with occupied width of 270 cm; Unoccupied space around the motor truck shall be filled with uniform load specified above
One road roller on one span
Impact for motor truck loading Coef $= \frac{20}{60+l}$ where l = loaded length in meter
max impact 30%
No impact for roadroller and Uniform live load.

Allowable working strength

Structural steel or Reinforcing Bars	~~1200~~	
Tension net	1200	kg/cm²
Extreme fibre stress net	1200	"
Shear of web Gross section	900	"
Compression member $1500(1-0.0055\ l/r)$ not over	1000	"

where l = length of member in cm
r = least radius of gyration in cm

Compression flange of girder $1200(1-0.012\ l/b)$ not over	1100	"
Shear on shop driven rivets (machine)	850	
" " field " " and turned bolts	750	
Shear on pin	900	
Bearing on shop driven rivets (machine driven).	1700	
" " field " "	1500	
" " pin.	1800	

Rollers 45d kg where d = diameter of roller in cm

Concrete 1:2:4

Direct Compression	35	kg/cm²
Fibre stress due to bending	45	"
Combined stress direct and bending	35	"

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

MADE BY C. Masuda DATE 4-4-10 FILE NO

CHECKED BY M. Jinda DATE 4-5-21 PAGE NO 2

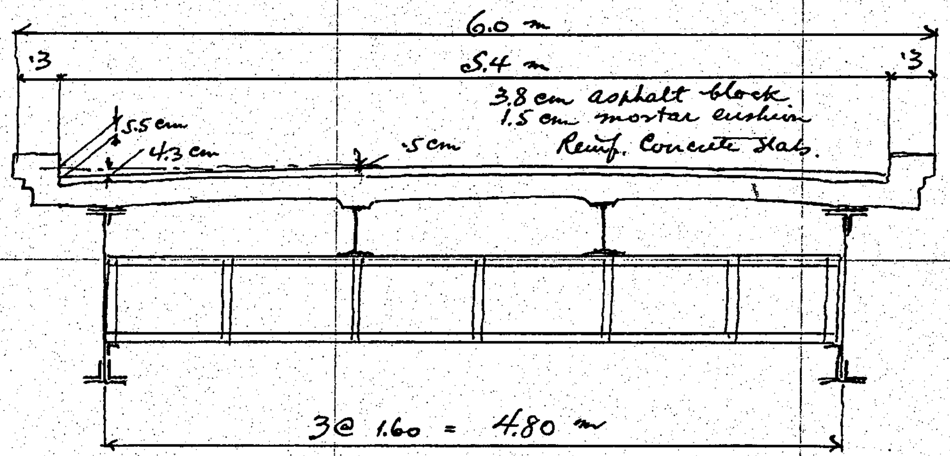
CALCULATIONS FOR

Design of Eian-Bashi for Okayama-Ken

Concrete 1:2:4 mixture
Punching shear of Concrete --------- 9 kg/cm²
Shear of plain Concrete --------- 4
Bearing --------- 45
Bond stress for plain bars --------- 6
" " " deformed bars --------- 9

For combined wind or temperature stress with dead live and impact stress, the allowable working strength shall be increased 25%. In case of earthquake increase unit stress 80%. Seismic accelleration assumed 1000 mm/sec² or k = 0.10

Design of 18.35 meter girder span.
Cross section of girder span as shown on sketch below.



Design of floor slabs. span length 1.60 meters
Dead Load

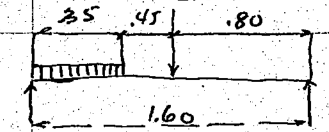
asphalt block pavement	3.8 cm	@	21 kg	=	80
mortar cushion	2.0 cm	@	22 "	=	44
Concrete slabs	13.0 cm	@	24 "	=	312
					436
	misc concrete say				14
					450 kg/m²

Dead Load moment = 1/10 · 450 · 1.60² = 115. kgm
~~Live~~ Dead Load ~~moment~~ shear = 1/2 · 450 · 1.60 = 360 kg.

Live load motortruck loading Rear wheel Concentration 2250
Impact 30% 675
2925 kg
Front wheel Concentration 2925 ÷ 3 = 975 kg.

Distribution of wheel Concentration
Contact between wheel and pavement 20.0
distribution 2 · 5.8 = 11.6
Longitudinal distribution a = 31.6
Transverse distribution b = 24 + 11.6 = 35.6

Effective width E = 2/3 l + a where l = span length in meter
= 1.39 meters
Load per meter strip = 2925 ÷ 1.39 = 2100 kg.
Uniform live load 500 kg per square meter



Uniform load $\frac{500 \cdot .35^2}{2 \cdot 1.60}$ = 19.1 kg

moment at center
Due to uniform load 19.1 · 0.8 = 15
Due to motor truck $\frac{2100}{2}$ · 0.8 = 840
855 kg meter

For continuity of slab 0.8 · 855 = 684 kgm

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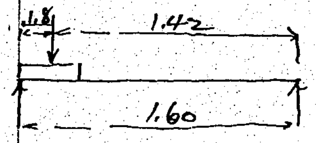
CALCULATIONS FOR

Design of Eian-Bashi for Okayama-Ken

Endshear



$2100 \times \frac{1.42}{1.60} = 1860$ kg.

Summary for moments and shears

	moments	shear
Dead Load	115	360
Live Load	684	1860
	799 kgm	2220 kg.

Effective depth required for $f_s = 1200$ kg/cm² and $f_c = 45$ kg/cm²

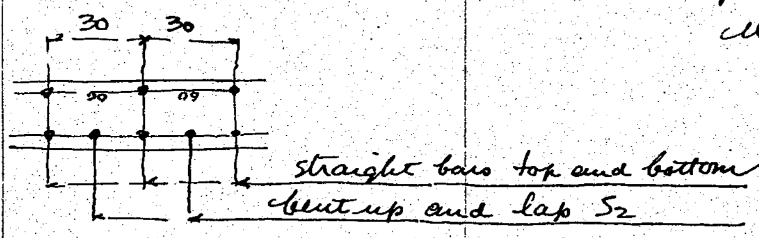
$R = \frac{M}{bd^2}$ $d = \sqrt{\frac{M}{bR}}$ $R = 7.18$ $d = \sqrt{\frac{799 \times 100}{100 \times 7.18}} = 10.55$ cm

Use 13 cm slabs with insulation at bottom of 2.45 cm

Steel area reqd $= \frac{799 \times 100}{7/8 \times 10.55 \times 1200} = 7.20$ cm² per meter strips

Spacing of 13mm bars $\frac{1.33 \times 100}{7.20} = 18.5$ cm

Use 15 cm spacing.



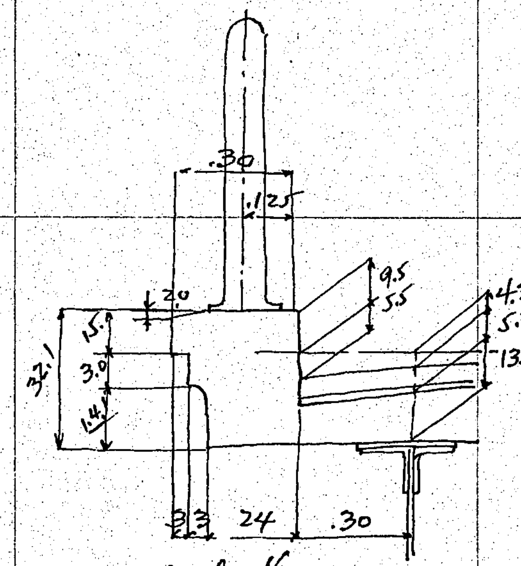
	dia	Circumf.		
S1	13mm	4.08	× 3.33	= 13.6
S2	"	"	× 6.67	= 27.2
				40.8

With lap. Unit bond $= \frac{2220}{7/8 \times 10.55 \times 40.8} = 5.90$ kg/cm²

Without lap Unit bond $= \frac{2220}{7/8 \times 10.55 \times 27.2} = 8.85$ "

Overhanging slab beyond outside stringer

approximate weight of Handrail. 70 kg per lin. meter
" " " Coping 192 " " " "



scale 1/20

Dead Load		load		
Handrail		70	× 0.425	= 29.8
Coping		192	× say .44	= 84.5
Slab + pavement	450 × .30 =	135	× .15	= 20.2
		397	.34	134.5

Live Load motor truck rear wheel at curb line assumed

Distribution or effective width on ℄ stringer assumed thus.

$2 \times .30 + .20 = .80$

Transverse distribution $24 + 11.6 = 35.6$ say 36 cm

Load on 30 cm $2925 \times \frac{30}{36} = 2440$

For one meter strip $2440 \div 0.8 = 3050$ kg.

Live load moment $= 3050 \times .15 = 458.0$ kgm

" " shear $= 3050$ kg

Summary for moments and shears

	moment	shear
Dead Load	135	397
Live Load	458	3050
	593 kgm	3447 kg

Effective depth reqd $d = \sqrt{\frac{593 \times 100}{100 \times 7.18}} = 9.10$ cm

Use 13 cm slab effective depth say 10.50 cm

Unit shear $= \frac{3447}{7/8 \times 100 \times 10.50} = 3.75$ kg/cm²

Use	13mm bars	15 cm spacing	1.33	× 6.67	=	8.85
	9mm bars	" "	.64	× "	=	4.26
						13.11 cm² per meter strips

Circumference	13mm bars	4.08	× 6.67	= 27.20
	9 "	2.83	× "	= 18.90
				46.10 cm per lin meter strip

Unit bond $= \frac{3447}{7/8 \times 10.5 \times 46.10} = 8.10$ kg/cm²

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CALCULATIONS FOR

Design of Eian Bashi for Okayama-Ken.

Design of I Beam Stringer span length 4.40

Dead Load

floor slab and pavement $450 \times 1.60 = 720$

stringer assumed 50

770 kg.

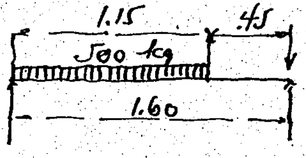
Dead Load moment $= 1/8 \times 770 \times 4.4^2 = 1870$ kgm

Dead Load shear $= 1/2 \times 770 \times 4.4 = 1694$ kg

Live Load

motor truck rear wheel with impact = 2925 kg

Uniform live load 500 kg per square meter.

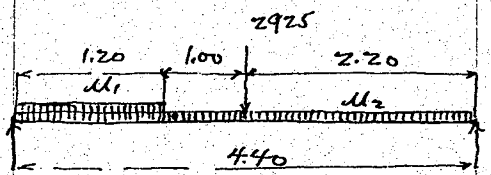


load on stringer $= \frac{500 \times 1.15^2}{2 \times 1.60} = 207.$ kg. -- U_2

full uniform load $= 500 \times 1.60 = 800$

less $- 207$

U_1 593 kg



Moment

Due to motor truck $\frac{2925}{2} \times 2.2 = 3220$

Unif. load U_1 $\frac{593 \times 1.20^2}{2 \times 4.4} \times 2.2 = 214$

Unif. load U_2 $1/8 \times 207 \times 4.4^2 = 501$

3935 kgm

End shear

Uniform load U_1 $\frac{593 \times 3.4^2}{2 \times 4.4} = 780$

Uniform load U_2 $207 \times \frac{4.4}{2} = 455$

motor truck loading 2925

4160 kg.

Summary for moments end shears

	moment	shear
Dead Load	1870	1694
Live Load	3935	4160
	5805	5854

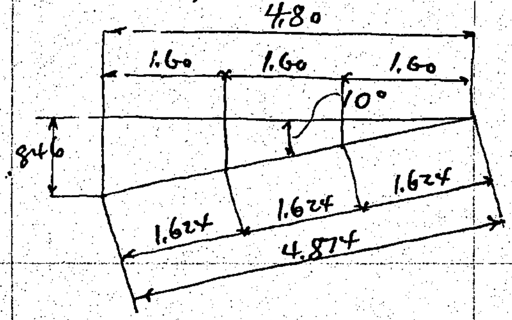
section modulus req'd $= \frac{580500}{1100} = 528.0$

Use 300 × 150 I @ 48.34 kg $S_m = 633.2$

Unit stress $= \frac{580500}{633.2} = 918$ kg/cm²

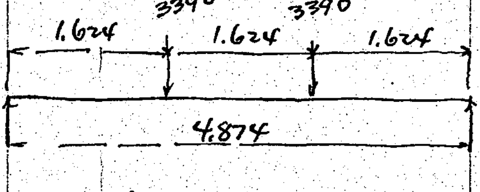
Design of Intermediate Floor Beam span length 4.8 meters as square

actual length of floor beam along skew = 4.874 meters



Load on floor beam figured as square beam and the bending moment of floor beam figured for actual span length along slope of floor beam

Dead Load Concentration on stringer $770 \times 4.40 = 3390$ kg.



moment $= 3390 \times 1.624 = 5510$ kgm

$m = 1/8 \times 100 \times 4.874^2 = 297$

5807 kgm

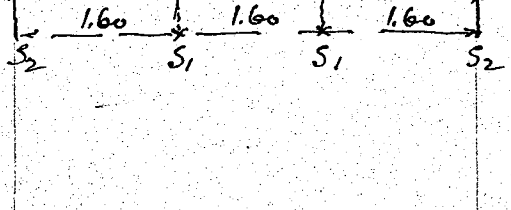
Dead Load beam assumed 100 kg. End shear from concentration 3390

beam $\frac{1}{2} \times 100 \times 4.874 = 244$

3634 kg.

Live Load motor truck loading rear wheel concentration with impact = 2925 kg

front wheel " " " 975 "



Load on S_1 $\frac{.15}{1.60} = 0.093$

1.000

$1.093 \times 2925 = 3200$ kg

$1.093 \times 975 = 1065$ "

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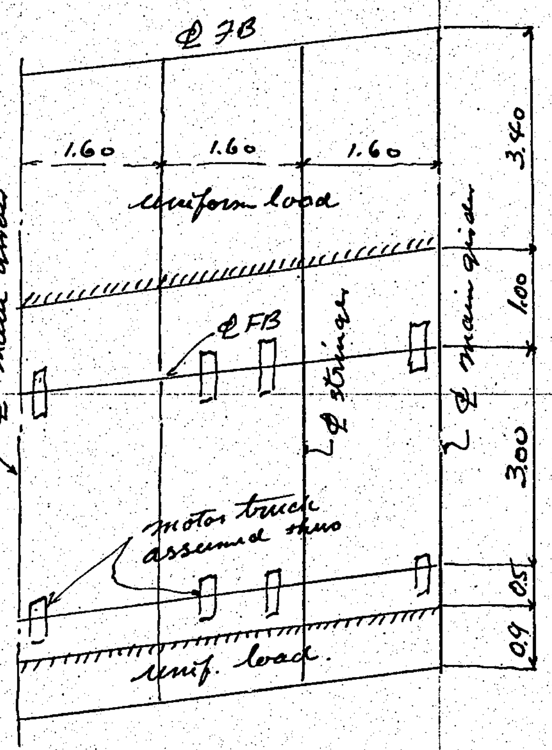
CALCULATIONS FOR

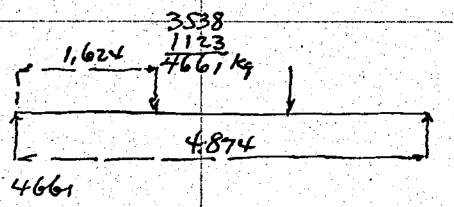
Design of Eiaw-Bashi for Okayama-Ken



Uniform live load $500 \times 1.60 = 800$ kg on stringer

Concentration on floor beam
motortruck loading rear wheel 3200
front wheel $1065 \times \frac{1.4}{4.4} = \underline{338}$
3538 kg.

Uniform load $\frac{800 \times 3.4^2}{2 \times 4.4} =$ 1050
$\frac{800 \times 0.9^2}{2 \times 4.4} =$ $\underline{73}$
1123 kg



moment = $4661. \times 1.624 =$ 7580 kgm
shear = 4661 kg.

Summary for moments and shears

	moment	shear
Dead Load	5807	3634
Live Load	7580	4661
	13387 kgm	8295 kg

section modulus reqd $= \frac{1338700}{1100} = 1215$
use 470×175 I @ 91.66 kg $S_m = 1743.$
Unit stress $= \frac{1338700}{1743} = 767$ kg/cm²
Use built section.

~~web assumed~~ $50. \times 0.8 = 40.0$ cm² $\frac{1}{8}$ web $= 5.0$ cm Back to back of L^s 51.0 cm
Effective depth = 46.7

flange stress $= \frac{1338700}{46.7} = 28600$ kg.
$SR = 28600 \div 1200 =$ 2380
$\underline{500}$
18.80 cm² net

$2L^s$ $75 \times 75 \times 9 = 25.38 - 3.96 = 21.42$ □ cm net

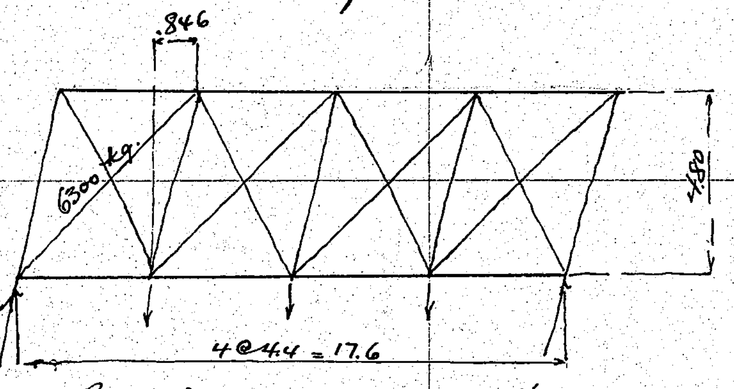
End Floor Beam use same as for intermediate floor beam

Approximate weight of floor beam

flanges	$4L^s$ $75 \times 75 \times 9$	@ 9.96	×	4.85	=	194.
web	1Pl. 500×8	@ 31.40	×	4.85	=	152.
End conn	$2L^s$ $125 \times 90 \times 10$	@ 16.09	×	0.50	=	16
fills	2Pls. 125×9	@ 8.85	×	0.35	=	6
stiffs	$4L^s$ $75 \times 65 \times 8$	@ 8.28	×	0.50	=	17
Base for stringer	2Pls. 240×9	@ 16.95	×	0.35	=	12
Rivet heads and variation						13
						410 kg.

$410 \div 4.85 = 84.5$ kg per lin meter

Lower Lateral Bracing



Reaction $2840 \times 1.5 = 4260$

Wind load assumed 600 kg per lin. meter
panel concentration $600 \times 4.4 = 2840$ kg.

Diagonal length. $\sqrt{4.8^2 + 5.246^2} = 7.10$
$\sqrt{4.8^2 + 3.554^2} = 5.97$

Approximate stress
$4260 \times \frac{7.10}{4.80} = 6300$ kg
$4260 \times \frac{5.97}{4.80} = 5300$ kg.

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CALCULATIONS FOR

Design of Eian-Bashi for Okayama-Ken

Diagonal at End. SR = 6300 ÷ 1200 = 5.25 □ cm net   try 1L 75×75×9 = 12.69 □ cm

Approximate weight of lower lateral bracing

4Ls 75×75×9	@ 9.96	× 7.10	=	283
4Ls "		× 5.95	=	237
Center connection say	4 @ 10		=	40
Connection pls + Ls	10 @ 20		=	200
				760 kq.

760 ÷ 18.0 = 42.0 kq per lin. meter.

Design of main girder span length 18.35 meters out to out
17.98 meters c to c of End bearings.

Dead Load

flooring.

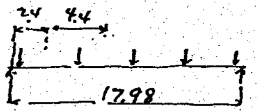
floor and pavement	450 × 5.40	=	2430
Handrails and copings	262 × 2	=	524
			2954.0 kq.

Structural steel

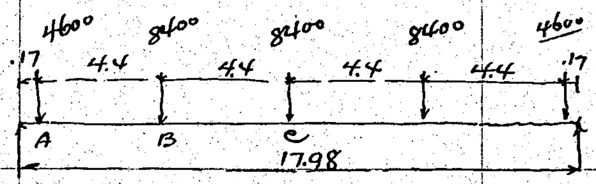
Stringers.	2@50	=	100
Floor beam	410 ÷ 4.4	= say	95
Laterals	$\frac{760 \div 4}{4.4}$	=	44
main girders			640
			879.0
			3833.0

For one girder 3833 ÷ 2 = 1917 kq per lin. meter.

Let us assume the load concentrated at panel point.



Intermediate Panel concentration = 1917 × 4.4 = 8440 kq.
End " " = 1917 × 2.4 above = 4600 "



17200 kq.

Moment at A   17200 × .17 = 2900 kgm

Moment at B   17200 × 4.57 = 78600
4600 × 4.4 = −20200
58400 kgm

Moment at C   17200 × 8.99 = 154500
8400 × 4.4 = − 37000
4600 × 8.8 = − 40500
77000 kgm

max End shear = 172.00 kq.
Dead Load on shoe or bearing = 1917 × $\frac{18.35}{2}$ = 17600
add for shoe = 400
18000 kq on masonry

Live Load   Uniform live load 500 kq per sq meter
impact for motor truck loading. = $\frac{20}{60+18.0}$ = 25.6 %

motor truck rear wheel concentration 2250
impact 25.6 % 577
2827 kq.

Front wheel with impact 2827 ÷ 3 = 942 kq.

Uniform live load 500 × 2.70 = 1350 kq per lin meter

motor truck concentration 2 wheels @ 2827 = 5654 kq.
2 " " 942 = 1884 "

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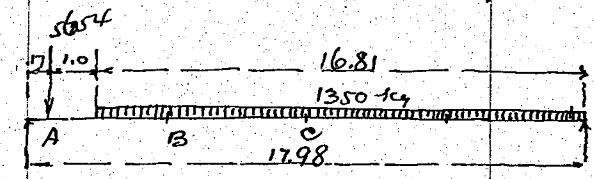
CALCULATIONS FOR

Design of Eian-Bashi for Okayama-Ken



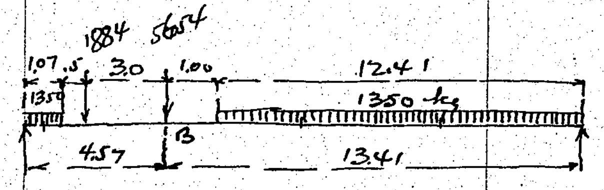
moment at A.

Reaction. $5654 \times \frac{17.81}{17.98} = 5600$

$\frac{1350 \times 16.81^2}{2 \times 17.98} = 10620$

$16\overset{2}{8}20$ kg.

moment $= 16\overset{2}{8}20 \times .17 = 27\overset{6}{8}0$ kgm



1.07 × 1350 = 1444 kg.

moment at B.

Reaction $5654 \times \frac{13.41}{17.98} = \overset{220}{4300}$

$1884 \times \frac{16.41}{17.98} = 1717$

Unif. $\frac{1350 \times 12.41^2}{2 \times 17.98} = 5770$

Unif. $1444 \times \frac{17.45}{17.98} = 1400$

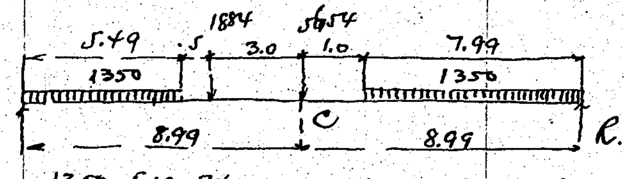
$131\overset{0}{8}7$ kg.

moment $131\overset{0}{8}7 \times 4.57 = 60000$

$1884 \times 3.00 = -5650$

$1444 \times 4.03 = -5830$

$48\underset{5}{6}20$ kgm.



1350 × 5.49 = 7400 1350 × 7.99 = 10800

moment at C

Reaction $5654 \div 2 = 2827$

$1884 \times \frac{5.99}{17.98} = 628$

Unif. $7400 \times \frac{2.75}{17.98} = 1135$

" $10800 \times \frac{13.98}{17.98} = 8400$

12990 kg.

moment $12990 \times 8.99 = 116900$

$10800 \times 5.0 = 54000$

62900 kgm.

Max End shear say rear wheel at ℄ of bearing.

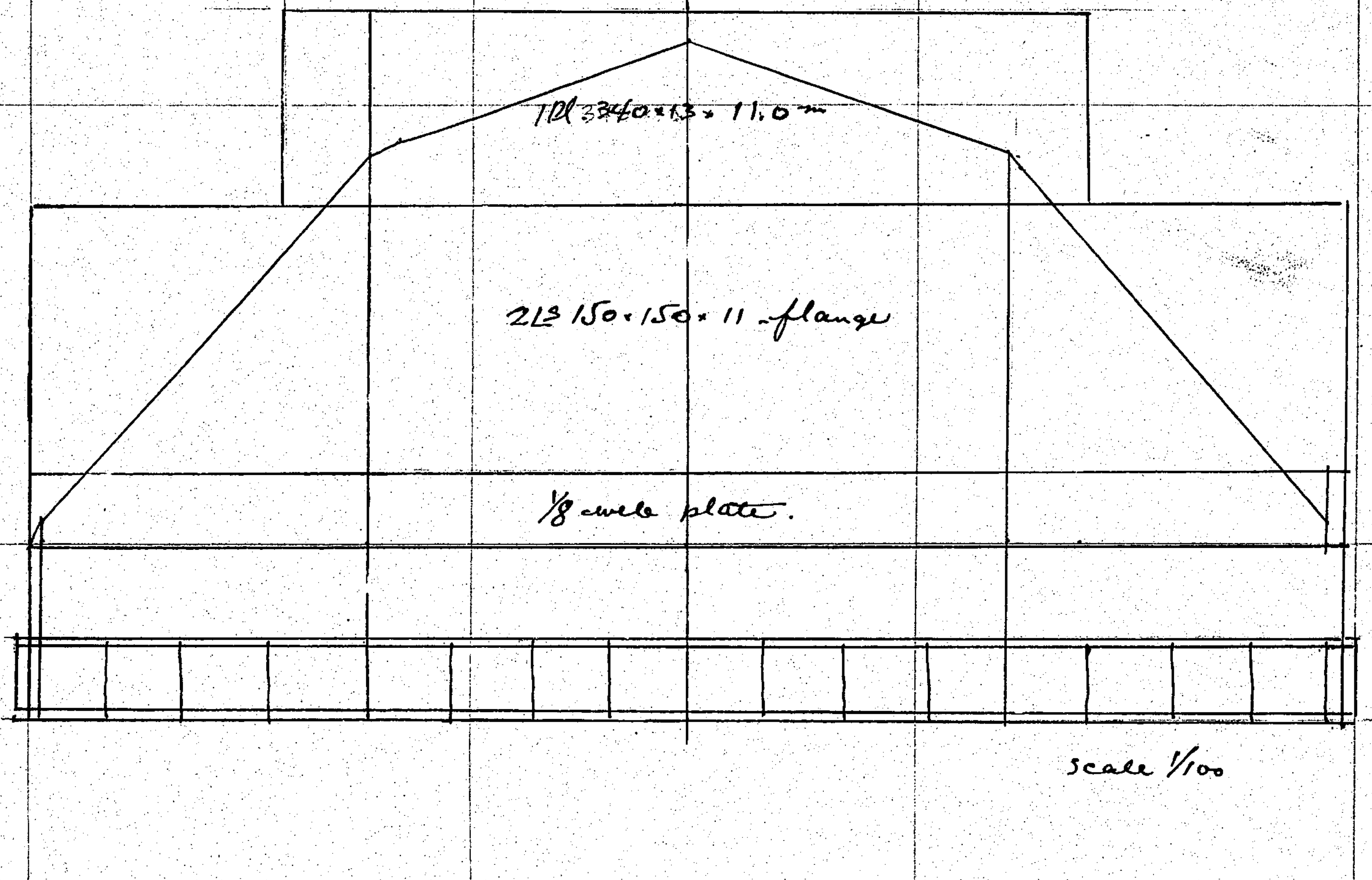
Unif. load $1350 \times \frac{16.98^2}{2 \times 17.98} = 10800$

motor truck 5654

16454 kg.

Summary for moment and shear

	moment A	moment B	moment C	End shear
Dead Load	2900	58400	77000	17200
Live load	2760	48520	62900	16454
	5660	106920	139900 kgm	33654 kg



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CALCULATIONS FOR

Design of Eian - Bashi for Okayama-Ken

Section of main girder.

depth of web pl. 120. webarea = 120 × 0.9 = 108.0 □cm 1/8 web = 13.5 □ cm

121 cm back to back of Ls Effective depth 116.8 cm

flange stress = $\frac{139900}{1.168}$ = 120.000 kg.

Section reqd = 120.000 ÷ 1200 = 100.0

13.5

86.5 □cm net.

4 holes 22 Rivets

try	2Ls	150 × 150 × 11	@ 31.79	=	63.58	- 11.0	=	52.58
	1Pl.	340 × 13	@		44.20	- 6.5	=	37.70
								90.28 □ net.

2 holes 22 Rvs.

Approximate weight of main girder

web	1Pl.	1200 × 9	@ 84.5	×	18.35	=	1550
flanges	4Ls	150×150×11	@ 24.95	×	18.35	=	1830
"	2Pls.	340 × 13	@ 34.70	×	11.00	=	762
End stiffs.	8Ls	125 × 90 × 10	@ 16.09	×	1.20	=	154
fills	8Pls.	125 × 11	@ 10.78	×	.90	=	78
Int stiffs	30Ls	125 × 75 × 10	@ 14.91	×	1.20	=	537
fills	2Pls.	75 × 11	@ 6.46	×	.90	=	12
splice.	4Ls	150 × 150 × 15	@ 33.55	×	.86	=	116
"	2Pls.	340 × 13	@ 34.70	×	.90	=	63
"	4Pls.	100 × 11	@ 8.63	×	.90	=	31
"	2Pls.	300 × 11	@ 25.90	×	.70	=	36
Sole plate	2Pls.	380 × 19	@ 56.60	×	.50	=	57
Shoe	2Pls	380 × 35	@ 104.30	×	.50	=	104
"	4Ls	75 × 75 × 9	@ 9.96	×	.21	=	9
Rivet heads + variation							201
							5540 ÷ 18.35 = 302 kg/m

Approximate weight of steel in one span

stringers.	100 × 18.35	=	1835
floor beams	5 @ 410	=	2050
Lateral Bracing complete			760
Girders + shoes	2 @ 5540	=	11080
misc steel say			500
			16,225 kg.

Load on shoe

Dead Load	18000
Live load say	16500
	34500 kg.

Area of base plate 38 × 50 = 1900

Unit bearing = $\frac{34500}{1900}$ = 18.2 kg/cm²

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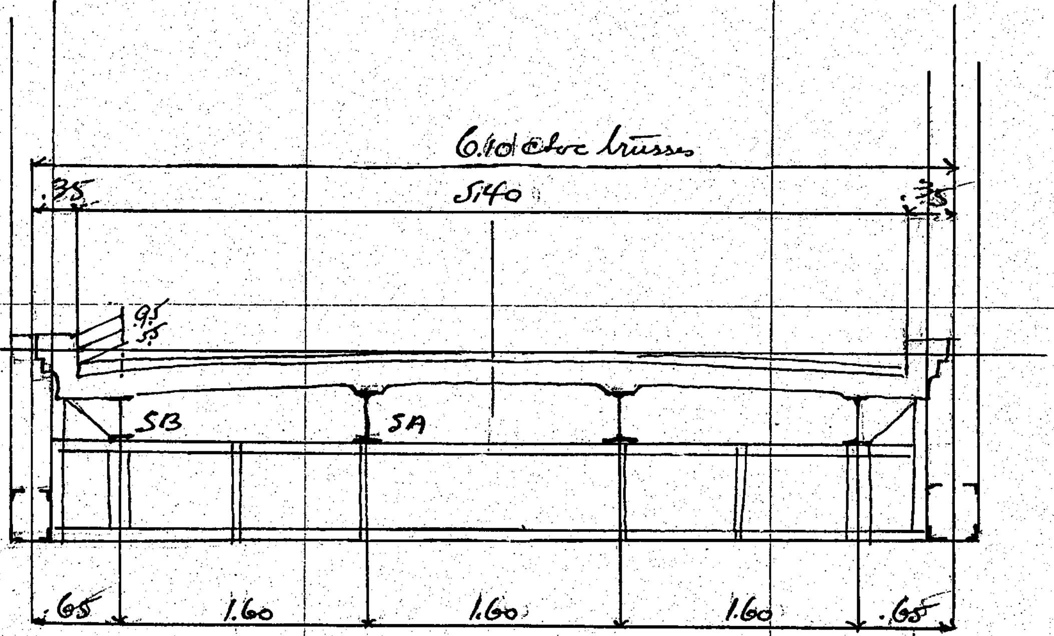
CALCULATIONS FOR

Design of Eian-Bashi for Okayama-Ken

50 meter tied arch span Roadway 5.4 meters clear
Cross section of truss span



Design of floor slab same as for girder span see pages 2-3

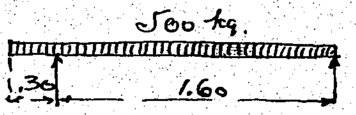
Design of I Beam Stringer span length 5.0 meters.
Dead Load Outside stringer. SB

Load on stringer	overhanging arm direct	397
	extra 134.5 ÷ 1.60 =	84
	between stringer $450 \cdot \frac{1.60}{2}$ =	360
		841
	Beam assumed	50
		891. kg per lin meter.

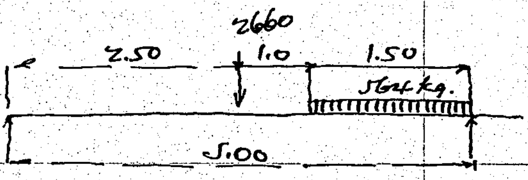
Dead Load moment $= \frac{1}{8} \cdot 891 \cdot 5.0^2 = 2790$ kgm
Dead Load shear $= \frac{1}{2} \cdot 891 \cdot 5.0 = 2230$ kg

Live Load motor truck loading rear wheel concentration with impact = 2925 kg.

Uniform live load $500 \cdot \frac{1.90^2}{2 \cdot 1.60} = 564$ kg per lin. meter.



motor truck $2925 \cdot \frac{1.45}{1.60} = 2660$ kg.



Reaction due to unif. load $= \frac{564 \cdot 1.50^2}{2 \cdot 5.00} = 127.0.0$

Bending moment $\frac{2660}{2} \cdot 2.50 = 3320$
$127.0 \cdot 2.50 = 318.0$
3638. kgm

max End shear
Uniform load $\frac{564 \cdot 4.0^2}{2 \cdot 5.00} = 902$
motor truck concentration 2660
3562 kg.

Summary for moments and shears

	moment	shear
Dead Load	2790	2230
Live Load	3638	3562
	6428 kgm	5792 kg

Use 300 · 150 I @ 48.34 kg $S_m = 633.2$

Unit stress $= \frac{642800}{633.2} = 1018$ kg/cm²

Inside Stringer SA.

Dead Load floor slab and pavement $450 \cdot 1.60 = 720$
stringer assumed 50
770
negative reaction from overhanging arm -84
686 kg per lin. meter

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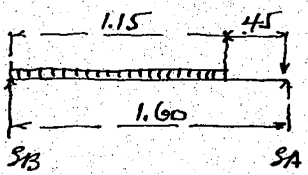
CALCULATIONS FOR

Design of Eian-Bashi for Okayama-Ken

Dead Load moment = $\frac{1}{8} \cdot 686 \cdot 5.0^2 = 2140$ kgm
Dead Load shear = $\frac{1}{2} \cdot 686 \cdot 5.0 = 1720$ kg.

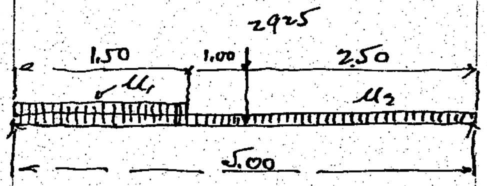
Live Load motor truck rear wheel concentration with impact 2925 kg.



Unif. live load 500 kg per square meter

Unif. load U_2 on stringer SA $\frac{500 \cdot 1.15^2}{2 \cdot 1.60} = 207$ kg.

Full Unif. load $500 \cdot 1.60 = 800$
-207
Uniform load U_1 - - - - - - - 593 kg



moment due to motor truck $\frac{2925}{2} \times 2.5$ = 3660
Unif load U_1 $\frac{593 \cdot 1.50^2}{2 \cdot 5.00} \cdot 2.5$ = 334
Unif. load U_2 = $\frac{1}{8} \cdot 207 \cdot 5.0^2$ = 647
4641 kgm

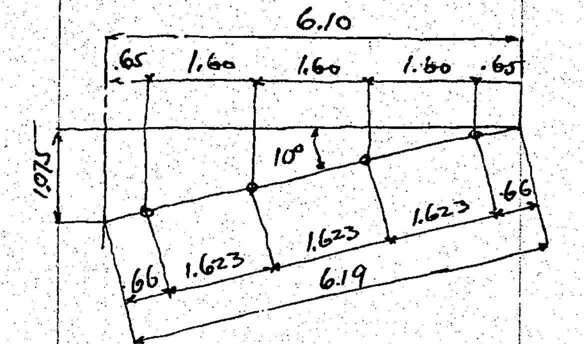
End shear
Uniform load U_1 $\frac{593 \cdot 4.00^2}{2 \cdot 5.00}$ = 948
Uniform load U_2 $207 \cdot \frac{5.00}{2}$ = 517
motor truck loading. 2925
4390 kg.

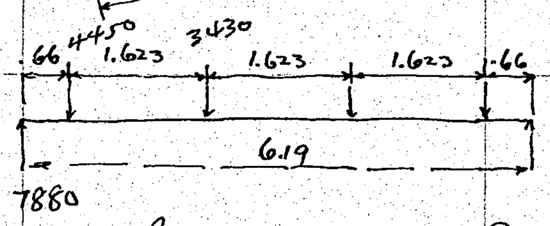
Summary for moments and shears

	moment	shear
Dead Load	2140	1720
Live Load	4641	4390
	6781 kgm	6110 kg

Section modulus reqd = $\frac{678100}{1100} = 617.0$
Use 300 · 150 I @ 48.34 kg Sm = 633.2
Unit Stress = $\frac{678100}{633.2} = 1070$ kg/cm²

Design of intermediate Floor Beam span length 6.10 meters measured in square
Bridge 10° skew. along skew 6.19 meters





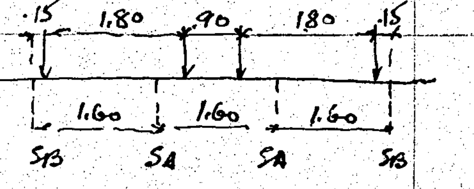
Load on stringer SA. ~~without~~ cantilever effect $686 \cdot 5.0 = 3430$ kg
" " " SB " " " $891 \times 5.0 = 4450$ kg.

Dead Load of beam assumed 160 kg per lin. meter.

moment $7880 \cdot 2.283$ = 18000
$4450 \cdot 1.623$ = -7220
10780 kgm
beam $\frac{1}{8} \cdot 160 \cdot 6.19^2$ = 766
Total DL. moment 11546 kgm

Dead Load shear Due to Concentration 7880
beam $\frac{1}{2} \cdot 160 \cdot 6.19$ = 495
8375 kg.

Live Load motor truck loading rear wheel Concentration with impact say 2925 kg
front wheel " " " 975 "



Load on SA. $\frac{.15}{1.60}$ = 0.093
1.000
1.093 × 2925 = 3200 kg Rear
" × 975 = 1065 " front

Load on SB. .907 · 2925 = 2660 " Rear
" · 975 = 885 " front

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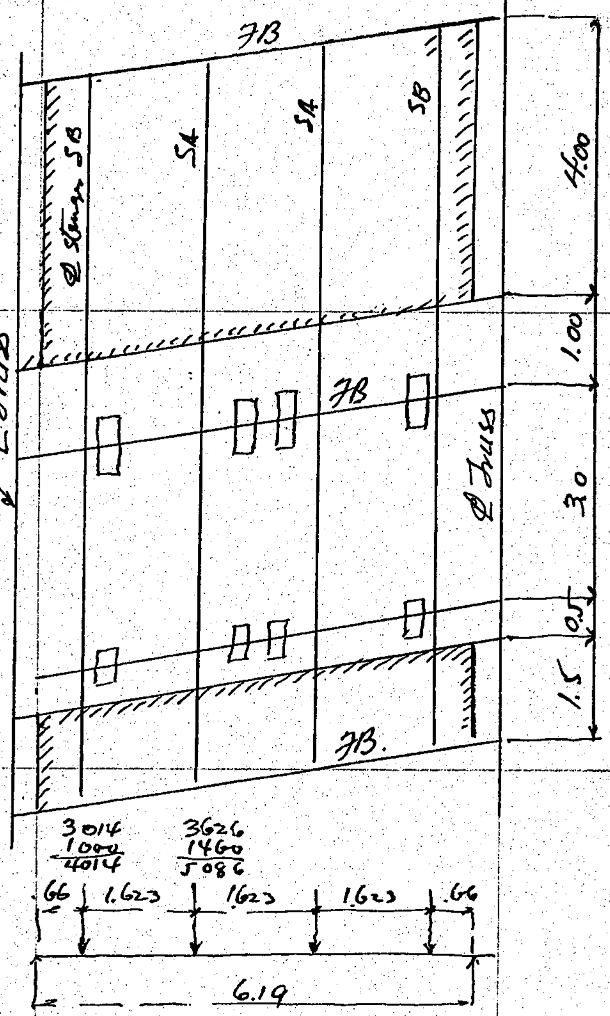
CALCULATIONS FOR

Design of Eiau-Bashi for Okayama-Ken

Uniform live load SA $500 \cdot 1.60 = 800$ kg per lin meter of stringer
SB $500 \cdot 1.10 = 550$ " " " " " "



motor truck loading
Concentration on floor beam
stringer SA $1065 \cdot \frac{2.0}{5.0} = 426$
3200
3626 kg

Stringer SB $885 \cdot \frac{2.0}{5.0} = 354$
2660
3014 kg.

Uniform live load.
stringer SA. $800 \cdot \frac{4.0^2}{2 \cdot 5.0} = 1280$
$800 \cdot \frac{1.5^2}{2 \cdot 5.0} = 180$
1460 kg.

Stringer SB. $1460 \cdot \frac{550}{800} = 1000$ kg.

5086
4014
9100 kg.

moment $9100 \cdot 2.283 = 20800$
$4014 \cdot 1.623 = -6520$
14280 kgm

End shear = 9100 kg.

Summary for moments and shears

	moment	shear
Dead Load	11546	8375
Live Load	14280	9100
	25826 kgm	17475 kg.

web assumed $67 \cdot 0.8 = 53.60$ □cm 1/8 web = 6.70
Back to back of Ls 68.0 cm Effective depth = 62.90
flange stress $= \frac{2582600}{62.9} = 41100$ kg
Section reqd $= \frac{41100}{1200} = 34.20$
$- 6.70$
27.50 □ cm net

Use 2Ls $90 \cdot 90 \cdot 10 = 34.00 - 4.4 = 29.60$ □ cm net

Allawable stress in compressive flange $1200 \left(1 - 0.012 \cdot \frac{167.3}{18.8}\right) = 1150$ kg/cm²

Approximate weight of Intermediate Floor Beam

flanges	4Ls 90·90·10	@	13.34 kg	5.89	=	314.0
web.	1Pl. 670·8	@	42.10	5.89	=	248.0
End Connection	4Ls 125·75·10	@	14.95	0.97	=	58.
fills	4Pls. 125·10	@	9.80	0.88	=	34.
Stiffs.	8Ls 75·75·9	@	9.96	0.67	=	54.
fills -	4Pls. 150·10	@	11.80	0.49	=	23
seat.	4Pls. 260·9	@	18.25	0.23	=	17
	misc detail say					50.
	Rivet heads &c					22
						820 kg.

$820 \div 5.89 = 140$ kg per lin meter

End Floor Beam use same as for intermediate floor beam

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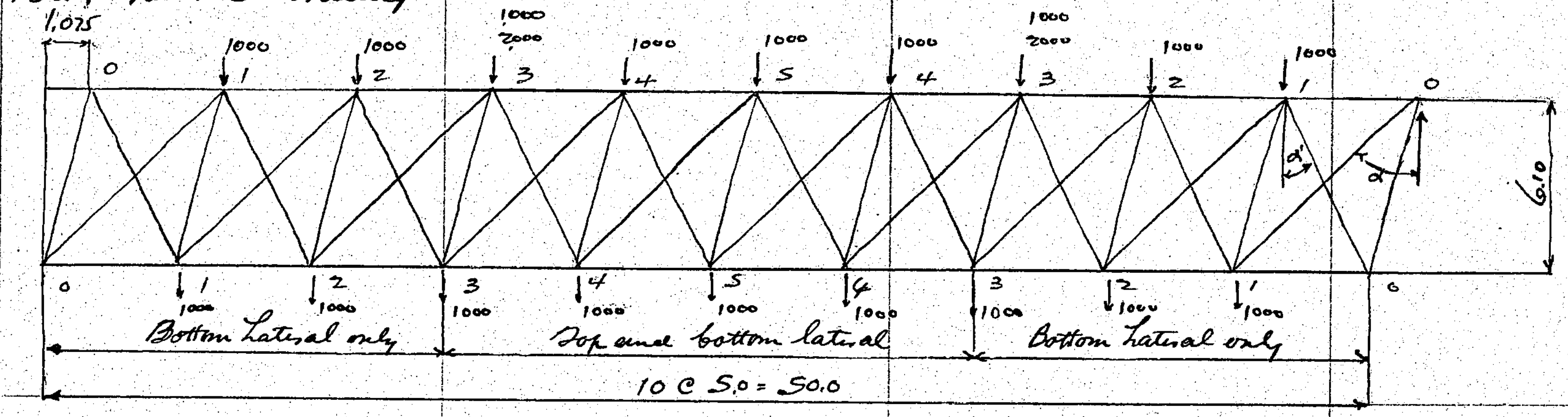
CALCULATIONS FOR

Design of Eian-Bashi for Okayama-Ken

Lower Lateral Bracing



Diagonal Length $\sqrt{6.1^2+6.075^2} = 8.61^m$ Sec α = 8.61 ÷ 6.1 = 1.41

$\sqrt{6.1^2+3.925^2} = 7.25^m$ Sec α' = 7.25 ÷ 6.1 = 1.~~00~~19

Panel Load Upper lateral bracing 100 × 5 = 500 kg

Reaction at end 500 × 2 = 1,000 kg on wind-ward and leeward side each

Diagonal stress 0-1 1000 × 2 × 1.41 = 2820 kg for longer diagonal

0-1 1000 × 2 × 1.19 = 2,380 kg " Shorter "

S.R. = 2820 ÷ 1200 = 2.37 ^{ocm} min r reqd = 810 ÷ 200 = 4.05 cm

2L5 150 × 90 × 9 = 41.58 cm^2 22φ rivet no reqd = 2820 ÷ 2851 = 1 use 6–19φ rivets ~~3 rivets~~

Use the same section for all diagonal members of upper lateral bracings.

Panel load for Lower Lateral bracings 200 × 5 = 1000 kg on windward and leeward panel pts. each.

Load transmitted from upper lateral = 2000 kg on windward panel pt. 3 only

Reaction for full load 9 @ 1000 = 9000 kg

2000

11,000 kg

Reaction for max. stress on 0-1 = 11,000 kg

do 1-2 = 11000 − 2000 × $\frac{9}{10}$ = 9,200 kg

do 2-3 = 11000 − 2000 × $\frac{9+8}{10}$ = 7600 "

do 3-4 = 7600 − 4000 × $\frac{7}{10}$ = 4,800 "

do 4-5 = 4800 − 2000 × 6/10 = 3,600 "

By the graphical solutions, we obtained the max. diagonal stresses, moving load, as follows:
Note: all stresses are for longer diagonal members.

members	max. stresses	S.R.	Section used	22φ rivet no reqd
0 − 1	17,550 kg T	14.65 cm^2 net	2L5 150 × 90 × 9 = 41.58 cm^2 gr − 9.0 = 32.58 cm^2 net	6.2
1 − 2	13,000 T	10.85	do	4.6
2 − 3	10,730 T	8.95	do	3.8
3 − 4	6 840 T	5.70	1L 150 × 100 × 9 = 21.69 − 4.5 = 17.19 "	2.4
4 − 5	5,100 T	4.25	do	1.8

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CALCULATIONS FOR

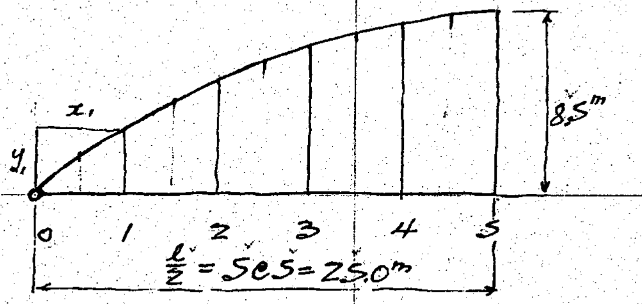
Eian Bashi for Okayama Ken

Design of Tied Arch spans. Span length = 10 @ 5.0 = 50.00 meters, Rise 8.5 meters

For neutral axis, extrados and intrados, all parabolic curves are used.

Nominal Dimensions of the Arch Rib.

Curve for Neutral Axis.



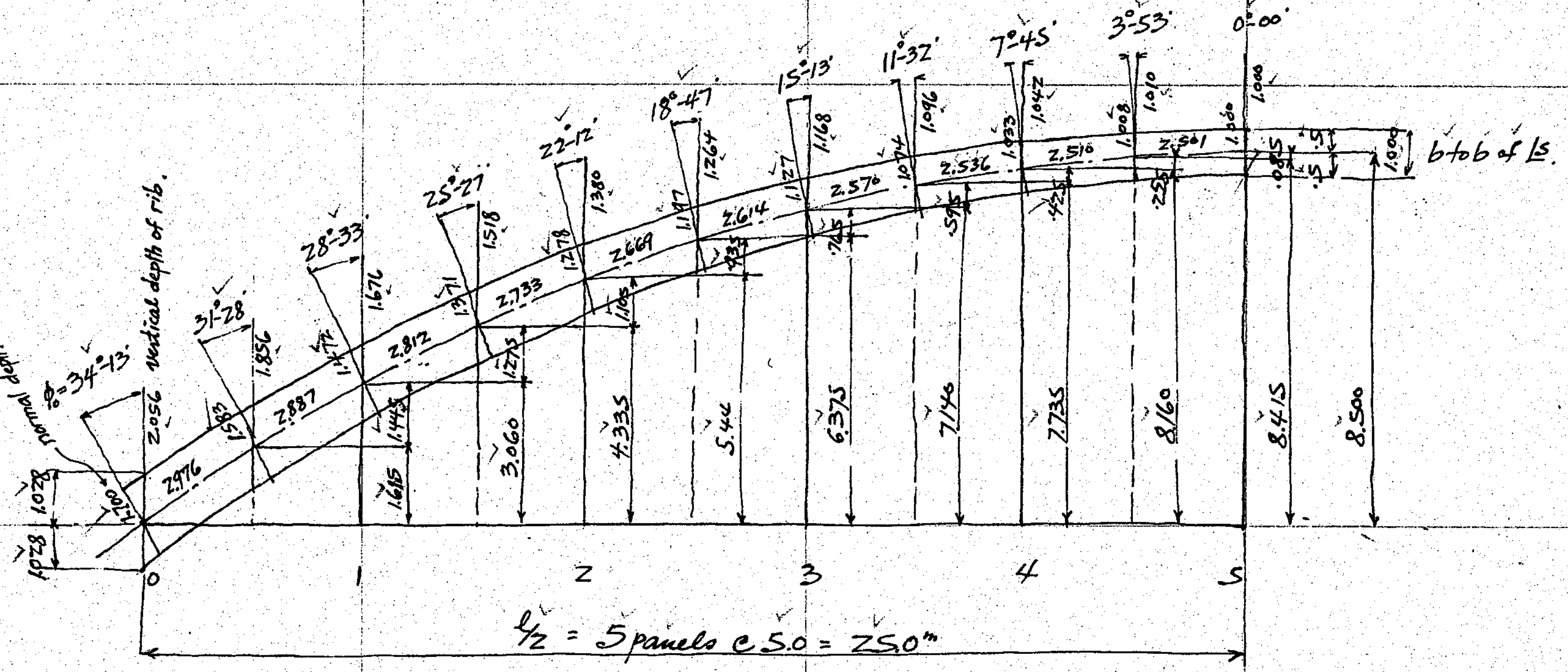
Equation of parabola whose origin is at o.

$$y = 4h\left(\frac{x}{l} - \frac{x^2}{l^2}\right)$$

Equation of slope.

$$\frac{dy}{dx} = 4h\left(\frac{1}{l} - \frac{2x}{l^2}\right) = \frac{8h}{l}\left(\frac{1}{2} - \frac{x}{l}\right)$$

Panel pt.	x	x/l	$\frac{1}{2}-\frac{x}{l}$		$\frac{8h}{l}$		$\tan\phi$	ϕ	$\sin\phi$	$\cos\phi$	$\sec\phi$	y	curve length	dc
0	0	0	.50	×	1.36	=	.680	34°-13'	.5623	.8269	1.2093	0.000	2.976	1.488
0-1	2.5	.05	.45	×	〃	=	.612	31°-28'	.5220	.8529	1.1724	1.615	2.887	2.932
1	5.0	.10	.40	×	〃	=	.544	28-33	.4779	.8784	1.1384	3.060	2.812	2.849
1-2	7.5	.15	.35	×	〃	=	.476	25-27	.4297	.9030	1.1075	4.335	2.733	2.773
2	10.0	.20	.30	×	〃	=	.408	22-12	.3778	.9259	1.0801	5.440	2.669	2.701
2-3	12.5	.25	.25	×	〃	=	.340	18-47	.3220	.9467	1.0563	6.375	2.614	2.641
3	15.0	.30	.20	×	〃	=	.272	15-13	.2625	.9649	1.0363	7.140	2.570	2.592
3-4	17.5	.35	.15	×	〃	=	.204	11-32	.1999	.9798	1.0206	7.735	2.536	2.553
4	20.0	.40	.10	×	〃	=	.136	7-45	.1349	.9909	1.0092	8.160	2.510	2.523
4-5	22.5	.45	.05	×	〃	=	.068	3-53	.0677	.9977	1.0023	8.415	2.501	2.506
5	25.0	.50	.00	×	〃	=	.000	0-00	.0000	1.0000	1.0000	8.500		1.250
												$\frac{l'}{2}$ =	26.808ᵐ	26.808



Extrados and Intrados curve. parabola, span length 50.0ᵐ rise of extrados = 7.972ᵐ rise of intrados = 9.028ᵐ

From the above equation of parabola, we obtained the ordinates for both curves. (ordinate measured from panel pt 0)

Panel pts.	x	y for extrados	y for intrados.	vert. depth of rib		$\cos\phi$		normal depth of rib.
0	0	1.028	−1.028	2.056ᵐ	×	.8269	=	1.700ᵐ
0-1	2.5	2.543	.687	1.856	×	.8529	=	1.583
1	5.0	3.898	2.222	1.676	×	.8784	=	1.472
1-2	7.5	5.094	3.576	1.518	×	.9030	=	1.371
2	10.0	6.130	4.750	1.380	×	.9259	=	1.278
2-3	12.5	7.007	5.743	1.264	×	.9467	=	1.197
3	15.0	7.724	6.556	1.168	×	.9649	=	1.127
3-4	17.5	8.283	7.187	1.096	×	.9798	=	1.074
4	20.0	8.681	7.639	1.042	×	.9909	=	1.033
4-5	22.5	8.920	7.910	1.010	×	.9977	=	1.008
5	25.0	9.000	8.000	1.000	×	1.0000	=	1.000

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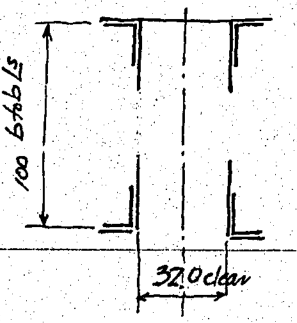
CALCULATIONS FOR

Eiian Bashi for Okayama Ken.

Calculation of Horizontal Thrust H.
Assumed section of the arch rib.
Crown section a panel pt. 5.



Top flange:
ZLs 150×100×12 = 57.12
ZPls 250×9 = 45.00
1 cov. pl. 560×9 = 50.40
152.52 cm² gr

Bottom flange.
ZLs 150×100×15 = 70.50
ZPls 250×9 = 45.00
Z Cov. pls 110×17 = 37.40
152.90 cm² gr
total = 305.42

Moment of inertia
640.2×2 + 57.12×45.12² = 117730
1171.2×2 + 45.0×37.2² = 164640
50.4×50.45² = 128,200
310,570

7849×2 + 70.5 × 45.01² = 144570
1171.2×2 + 45.0×37.2² = 164640
37.4×50.85² = 96700
305910
Crown. I_5 = 616,480 cm⁴

Moment of inertia at panel pt. 4 I_4. 103.3 b to b Ls.
top. ZLs 640×2 + 57.12×46.77² = 126,080
2Pls. 1171×2 + 45.0×38.85² = 70,260
1 cov pl. 50.4×52.10² = 136,760
333,100

bottom ZLs 785×2 + 70.5×46.77² = 155,750
ZPls 1171×2 + 45.0×38.85² = 70,260
2 cov pls. 37.4×52.50² = 103,090
329,100
I_4 = 662,200 cm⁴

I_3 at panel pt 3, 112.7 b to b Ls.
640×2 + 57.12×51.47² = 152,580
1171×2 + 45.0×43.55² = 87,640
50.4×56.80² = 162680
402,900

785×2 + 70.5×51.36² = 187,670
1171×2 + 45.0×43.55² = 87,640
37.4×57.20² = 122490
397,800
I_3 = 800,700 cm⁴

I_2 at panel pt. 2. 127.8 b to b Ls
1280 + 57.12×59.02² = 200,300
2340 + 45.0×51.1² = 119,900
50.4×64.35² = 208,500
528,700

1570 + 70.5×58.91² = 246,300
2340 + 45.0×51.1² = 119900
37.4×64.75² = 156800
523,000
I_2 = 1,051,700

I_1 at panel pt. 1 147.2 b to b Ls
1280 + 57.12×68.72² = 271,400
2340 + 45.0×60.8² = 168,800
50.4×74.05² = 276,100
716,300

1570 + 70.5×68.61² = 333,600
2340 + 45.0×60.8² = 168800
37.4×74.45² = 207,300
709,700
I_1 = 1426,000

I_0 at panel pt. 0. 170.0 b to b Ls.
1280 + 57.12×80.12² = 368,000
2340 + 45.0×72.2² = 236,700
50.4×85.45² = 368,300
973,000

1570 + 70.5×80.01² = 457,800
2340 + 45.0×72.2² = 236,700
37.4×85.85² = 275,500
965,000
I_0 = 1938,000

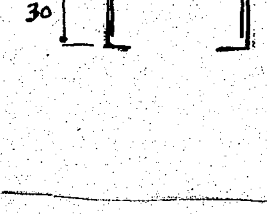
Average moment of inertia of Rib.

I	dc.	
I_0 = 1938,000	× 2.976 =	5750,000
I_1 = 1426,000	× 5.699 =	8120,000
I_2 = 1,051,700	× 5.402 =	5680,000
I_3 = 800,700	× 5.184 =	4150,000
I_4 = 662,200	× 5.046 =	3,340,000
I_5 = 616,500	× 2.501 =	1,540,000
	26.808 m =	28,580,000

Average moment of inertia

$$I = \frac{28,580,000}{26.808} = \underline{\underline{1065000}} \text{ cm}^4$$

Section of Tie assumed.



ZLs 300×90 @ 38.13 = 97.15 − 19.37 = 77.78
ZPls 220×12 = 52.80 − 10.57 = 42.23
149.95 cm² gross. 120.01 cm² net.

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CALCULATIONS FOR

Eian-Bashi for Okayama ken.

Horizontal Thrust due to unity at several panel points.

Approximate formula for H. See Hirois, Statically indeterminate stresses on page 107.

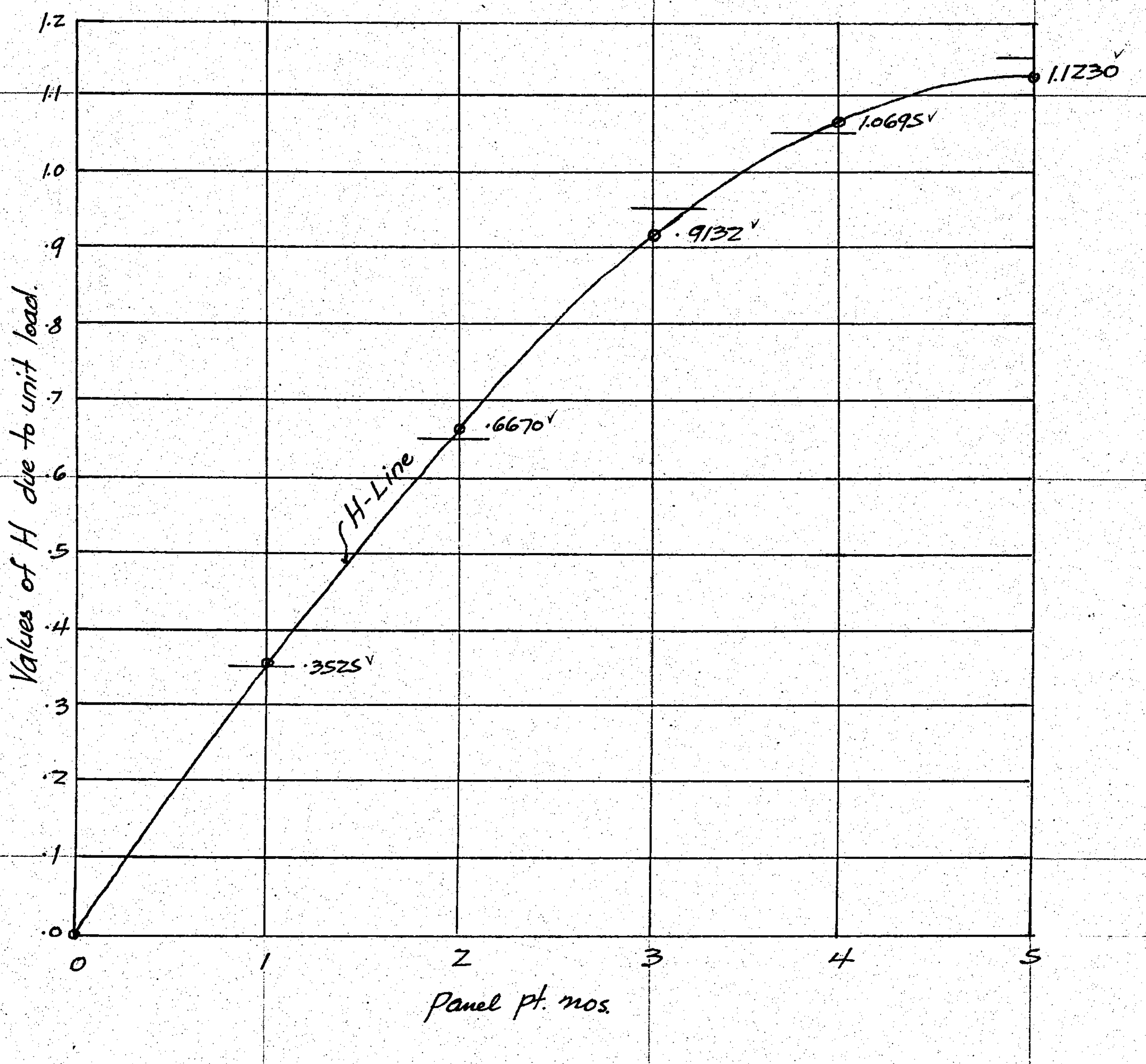
$$H = \frac{5a(l-a)(l^2+al-a^2)}{l^3\left(8h + \frac{15I}{A_t h}\right)} W.$$

where
a = distance of a load from left hinge.
l = Span length = 50.0 meters.
h = rise = 8.5 "
I = moment of inertia of the normal section of the rib   average $I = 1065000^{cm^4}$ or $.01065^{m^4}$
A_t = Net cross sectional area of the tie $= 120^{cm^2} = .012^{m^2}$
W = $1.^{kg}$

note: This formula for H is based on the following assumptions
Flat parabolic arch $dx = dc$,
Cross-section of the arch rib to be uniform throughout,
Effect of axial thrust neglected.

$$H = \frac{5a(50-a)(2500+50a-a^2)}{125000\left(8\times8.5+\frac{15\times.01065}{.012\times8.5}\right)} \times 1 = \frac{5a(50-a)(2500+50a-a^2)}{8695775}$$

Load at panel pt.	a	5a		(50-a)		(2500+50a-a²)		numerator		denominator		Hor. thrust H	
0	0	0	×	50	×	2,500	=	0	÷	8695775	=	0.0000	= H_0
1	5	25	×	45	×	2,725	=	3,065,625	÷	〃	=	0.3525	= H_1
2	10	50	×	40	×	2,900	=	5,800,000	÷	〃	=	0.6670	= H_2
3	15	75	×	35	×	3,025	=	7,940,625	÷	〃	=	0.9132	= H_3
4	20	100	×	30	×	3,100	=	9,300,000	÷	〃	=	1.0695	= H_4
5	25	125	×	25	×	3,125	=	9,765,625	÷	〃	=	1.1230	= H_5



H for whole span = 7.1274

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CALCULATIONS FOR

Eeian-Bashi for Okayama Ken.

Vertical Shear due to unity

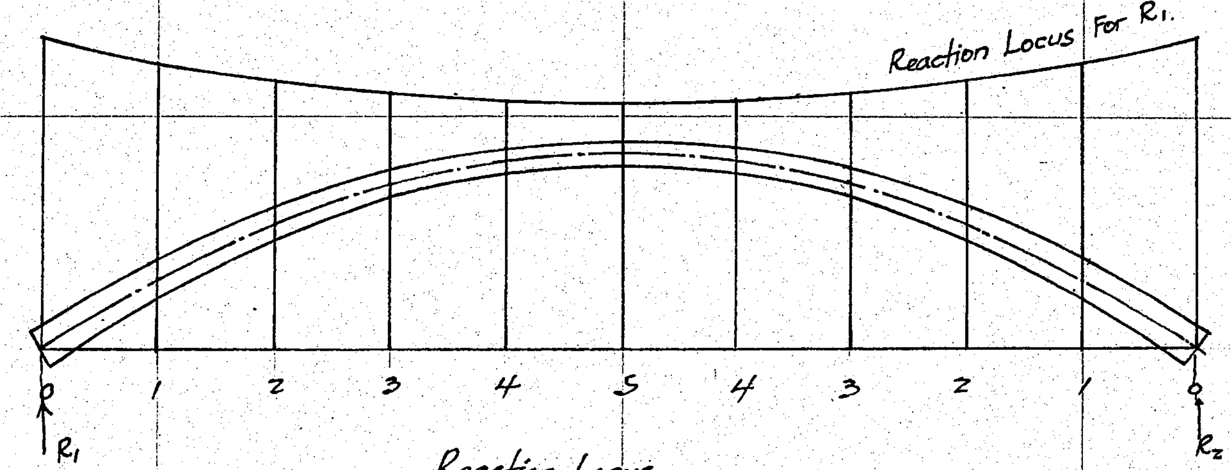
Load at panel pt	Vert. Shear V1.
0	$1 \times 10/10 = 1.0000$
1	$1 \times 9/10 = .9000$
2	$1 \times 8/10 = .8000$
3	$1 \times 7/10 = .7000$
4	$1 \times 6/10 = .6000$
5	$1 \times 5/10 = .5000$

Approximate Reaction Locus. See Merriman and Jacoby's Roofs and Brides part 4 Page 239

equation of Reaction Locus.

$$q = \frac{1.6h}{1+\frac{a}{l}-\left(\frac{a}{l}\right)^2}$$

Panel pt.	a/l	q	for h=8.5
0	0.00	$1.6000h =$	13.600^m
1	0.10	$1.4679h =$	12.477
2	0.20	$1.3793h =$	11.724
3	0.30	$1.3223h =$	11.240
4	0.40	$1.2903h =$	10.968
5	0.50	$1.2800h =$	10.880



Reaction Locus.

No 6

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CALCULATIONS FOR

Eian Bashi for Okayama Ken

Dead Load Stresses.

Panel Load.

			Panel load for one truss.	
Floor	Slab, pavement &c	450 × 5 × 2.7	= 6070	
	Handrail	70 × 5	= 350	
	Coping say	194 × 5	= 970	
	Stringers	2 × 50 × 5	= 500	
	Floor Beam	820 ÷ 2	= 410	
				8300 kg
Truss assumed	700 kg/lin meter	5 × 700		3500
Lateral Bracing		5 × 45		225
Sway & portal bracing &c		say		475
		Total Dead Panel Load	=	12500 kg

Horizontal Thrust H.

Summary of ordinates for H = (.0000 + .3525 + .6670 + .9132 + 1.0695) 2 + 1.123 = 7.1274

Dead Load H = 12,500 × 7.1274 = 89,090 kg

Vertical Shear

Reaction V_1 = 12,500 × 4.5 = 56,250 kg

Panel pt.	V_1	Panel load.		Shear (vert.) V.
0–1	56,250	− 0	=	56,250 kg
1–2	〃	− 12500 × 1	=	43,750
2–3	〃	− 12500 × 2	=	31,250
3–4	〃	− 12500 × 3	=	18,750
4–5	〃	− 12500 × 4	=	6,250
5	〃	− 12500 × 4.5	=	0

Normal thrust $N = -(V_1 \sin\phi + H\cos\phi)$ and Normal Shear $V = +(V_1 \cos\phi - H\sin\phi)$

Panel pt.	V_1	H	Sin φ	Cos φ	N.	V.
0	56,250	89,090	.5623	.8269	− 105,100 kg	− 3,600 kg
1	56,250	〃	.4779	.8784	− 105,100	+ 6,830
2	43,750	〃	.3778	.9259	− 99,000	+ 6,850
3	31,250	〃	.2626	.9649	− 94,200	+ 6,700
4	18,750	〃	.1349	.9909	− 90,700	+ 6600
5	6,250	〃	.0000	1.0000	− 89,100	+ 6,250

Dead Load moment at several panel pts. $m = M_0 - Hy$.

panel pt.	$V_1\,na$			Σpa.			Hy			m
	V_1	na		p	a		H	y		$V_1na - \Sigma pa - Hy$.
0	56,250 ×	0 =	0	12500 ×	0 =	0	89,090 ×	0 =	0	0 kgm
1	〃	× 5 =	281,250	〃	× 0 =	0	〃	× 3.06 =	272,500	+ 8,750
2	〃	× 10 =	562,500	〃	× 1 × 5 =	62,500	〃	× 5.44 =	484,500	+ 15,500
3	〃	× 15 =	843,750	〃	× 3 × 5 =	187,500	〃	× 7.14 =	636,000	+ 20,250
4	〃	× 20 =	1125,000	〃	× 6 × 5 =	375,000	〃	× 8.16 =	727,000	+ 23,000
5	〃	× 25 =	1,406,250	〃	× 10 × 5 =	625,000	〃	× 8.50 =	757,000	+ 24,250

Dead Load Stresses

panel pt.	Thrust N.	Shear V	moment m.
0	− 105,100 kg	− 3,600 kg	0 kgm.
1	− 105,100	+ 6,830	+ 8,750
2	− 99,000	+ 6,850	+ 15,500
3	− 94,200	+ 6,700	+ 20,250
4	− 90,700	+ 6,600	+ 23,000
5	− 89,100	+ 6,250	+ 24,250

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Live Load Stresses:

Panel Load.

Uniform load on Roadway $w = \frac{100,000}{170+l} = \frac{100,000}{170+50} = 455 \text{ kg/m}^2$

Load for one truss
$455 \times 2.7 = 1,230 \text{ kg/lin meter}$

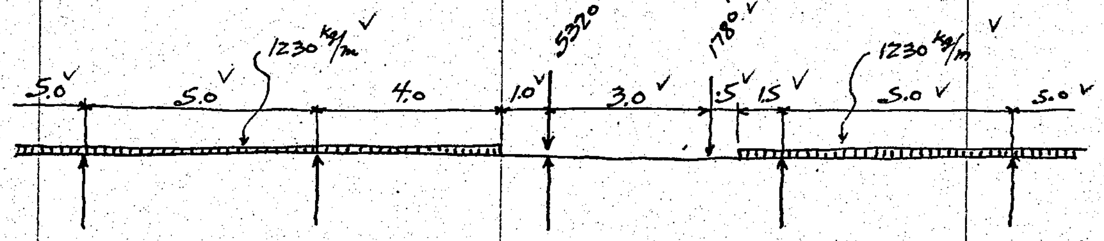
Motor truck loading Impact coeff. $= \frac{20}{60+l} = \frac{20}{60+50} = 18.2\%$

Rear wheel concentration 2250
impact $2250 \times 18.2 = 410$
2660 kg

$2660 \times 2 = 5320$ kg
$890 \times 2 = 1780$

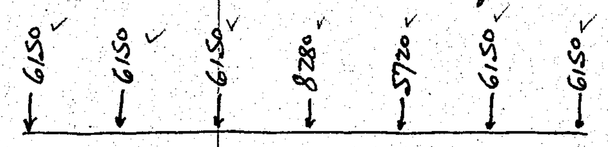
Front wheel concentration with impact $= 2660 \div 3 = 890$ kg

Floor Beam concentrations.



3075
3075 ---- 3075
6150 2950 ---- 1970 3075 3075
6025 5320 ---- 0 3075
710 ---- 1070 6150
280 ---- 1570
8280 5715

Live panel loads assumed as follows.



Stresses at crown or Panel pt. 5.
Positive moment at panel pt. 5

By the aid of reaction locus diagram, max. pos. m will occur when the panel pts. 4, 5 & 4 is loaded.

panel pt.	panel load.	H unit	H		V_1
0					
1	0	.3525	—		
2	0	.6670	—		
3	0	.9132	—		
4	6150	1.0695	6,580	.6	3690
5	8280	1.1230	9,300	.5	4140
4	5720	1.0695	6,120	.4	2290
3	0	.9132	—		
2	0	.6670	—		
1	0	.3525	—		
0			H = 22,000		V_1 = 10,120

Normal thrust $N = H = 22000$ kg
Tangential Shear $V = 10120 - 6150 = +3970$ kg

max. pos. m.

$V_1 \frac{l}{2}$	$10,120 \times 25 =$	+	253000
Pa	$6150 \times 5 =$	−	30750
Hy	$22000 \times 8.5 =$	−	187000
		+	35250 kgm

Max Shear at crown.

panel pt.		coeff.	V_1
5	8280	.5	4,140
4	5720	.4	2290
3	6150	.3	1,850
2	"	.2	1230
1	"	.1	620
0			—

V = +10,130 kg

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CALCULATIONS FOR

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Max. negative moment at crown.

panel pt.	panel load.	Hunit	H.	V1.	
0					
1	6150	.3525	2170	.9	5530
2	〃	.6670	4100	.8	4920
3	〃	.9132	5610	.7	4300
4	—	1.0695	—	.6	—
5	—	1.1230	—	.5	—
4	—	1.0695	—	.4	—
3	8280	.9132	7560	.3	2480
2	5720	.6670	3810	.2	1,140
1	6150	.3525	2170	.1	620
0					
			H= 25420 kg		V1 = 18990

max. neg. moment

$V_1 \frac{l}{2}$ 18,990 × 25 = + 474,500

pa. 6,150 × 9 × 5 = − 276,500

Hy 25420 × 8.5 = − 216,000

− 18,000 kgm

~~max~~ normal thrust N = −25420 kg = H

Max. Live Load thrust Full load. max load at center.

panel pt.	panel load	Hunit	H	V1.	
0					
1	6150	.3525	2170	.9	5,530
2	〃	.6670	4,100	.8	4,920
3	〃	.9132	5610	.7	4,300
4	〃	1.0695	6,580	.6	3,690
5	8280	1.1230	9,300	.5	4,140
6	5720	1.0695	6,120	.4	2,290
3	6150	.9132	5,610	.3	1,850
2	〃	.6670	4,100	.2	1,230
1	〃	.3525	2170	.1	620
0					
			H = 45,760 kg		V1 = 28,570 kg

moment for full load.

$V_1 \frac{l}{2}$ 28,570 × 25 = + 714,000

Pa. 6150 × 10 × 5 = − 307,500

Hy 45760 × 8.5 = − 389,000

+ 17,500 kgm

normal thrust N = H = − 45,760 kg.

Max. stresses for panel pt. 3.

max. pos. moment. Load on 1 ~ 4

panel pt.	panel load.	H unit	H	V1		$\sin\phi_3$	$\cos\phi_3$
0							
1	6,150	.3525	2170	.9	5,530	.2625	.9649
2	6,150	.6670	4,100	.8	4,920		
3	5,720	.9132	5,220	.7	4,000		
4	8,280	1.0695	8,850	.6	4,970		
5	—	—					
		~~H=~~	H = 20,340 kg		V1 = 19420 kg − 2@6150 = 7120		

$N = -(V_1 \sin\phi + H \cos\phi) = -(7{,}120 \times .2625 + 20340 \times .9649) = -21{,}490$ kg

$V = -(V_1 \cos\phi - H \sin\phi) = -(7{,}120 \times .9649 - 20340 \times .2625) = -11{,}530$ kg

pos moment.

$V_1 x$ 19420 × 15 = + 291,300

pa. 6150 × 3 × 5 = − 92200

Hy 20340 × 7.14 = − 145,100

+ 54,000 kgm.

max Shear load on 3 − 0 right side

Panel Pt.	load	H unit	H	V1	
3	8280	.9132	7,560	.7	5800
4	5720	1.0695	6120	.6	3430
5	6150	1.1230	6900	.5	3080
4	〃	1.0695	6570	.4	2460
3	〃	.9132	5610	.3	1850
2	〃	.6670	4100	.2	1230
1	〃	.3525	2170	.1	620
			H = 39030		V1 = 18470

N = −42,550

V = − 7,600

$V_1 x$ = 18470 × 15 = + 277,000

pa = 0

Hy = 39030 × 7.14 = −279,000

m = −2000

Max. negative moment Load on 5 ~ 1 right side

panel pt.	panel load	H unit	H	V1	
5	8280	1.1230	9300	.5	4140
4	5720	1.0695	6120	.4	2290
3	6150	.9132	5610	.3	1850
2	6150	.6670	4,100	.2	1230
1	6150	.3525	2,170	.1	620
			H = 27,300 kg		V1 = 10,130 kg

$N = -(10130 \times .2625 + 27300 \times .9649) = -29{,}000$ kg

$V = -(10130 \times .9649 - 27300 \times .2625) = -2620$ 〃

moment

$V_1 x$ 10130 × 15 = + 152,000

~~pa 6150 × 3 × 5~~

Hy 27300 × 7.14 = − 195,000

− 43,000 kgm.

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CALCULATIONS FOR

Eian Bashi for Okayama Ken

max. Stresses for panel point 2.

max. positive moment. Load same as for 3.

$H = 20340\ ^{kg}$ $V_1 = 19420\ ^{kg}$ $\sin\phi_4 = .3778$ $\cos\phi_4 = .9259$

$N = -(V_1 \sin\phi + H\cos\phi) = -(13270 \times .3778 + 20,340 \times .9259) = -\ 23870\ ^{kg}$

$V = -(V_1\cos\phi - H\sin\phi) = -(13270 \times .9259 - 20340 \times .3778) = -\ 4600$

pos. moment

$V_1 x$ $19420 \times 10 = +194200$

Pa $6150 \times 5 = -\ 30800$

Hy $20340 \times 5.44 = -\ 110700$

$+\ 52,700\ ^{kgm.}$

max. negative moment.

$H = 27300\ ^{kg}$, $V_1 = 10130\ ^{kg}$

$N = -(10,130 \times .3778 + 27300 \times .9259) = -\ 29070\ ^{kg}$

$V = -(10130 \times .9259 - 27,300 \times .3778) = -\ 910$

neg. moment.

$V_1 x$ $10130 \times 10 = +101,300$

Pa — — — 0

Hy $27300 \times 5.44 = -148500$

$-\ 47,200\ ^{kgm.}$

Max shear at panel pt. 0. loading same as for max thrust of crown section. or full load.

$H = 45,760\ ^{kg}$, $V_1 = 28,570\ ^{kg}$ $\sin\phi_0 = .5623$, $\cos\phi_0 = .8269$

$N = -(28,570 \times .5623 + 45760 \times .8269) = -\ 53,900\ ^{kg}$

$V = -(28,570 \times .8269 * 45,760 \times .5623) = +$ ~~HE~~ $21,200\ ^{kg}$

Summary of Live and Dead Load Stresses in Arch Rib.

	panel pt. 0.			panel pt. 2			panel pt. 3			panel pt. 5 (crown)		
	N	m	V	N	m	V	N	m	V	N	m	V
Dead Load	-105100	0		-99000	+15500		-94200	+20250		-89100	+24250	
Pos. Live Load		0		-23780	+52700		-21490	+54000		-22000	+35250	
Total.	-105100	0		-122780	+68200		-115690	+74250		-111,100	+59,500	
Dead Load	-105100			-99000	+15,500		-94,200	+20250		-89,100	+24250	
neg. Live Load	-53900	0		-29,070	-47200		-29000	-43000		-25420	-18000	
Total.	-159000			-128,070	-31,700		-123200	-22,750		-114,520	+6250	
Dead Load	-105,100		-3600			+6850			+6700	-89100	+24250	+6250
Live Load max N	-53,900	0	+21,200						-7600	-45760	+17,500	+10130
Total	-159,000		+17600						-900	-134,860	+41750	+16380

Stress at crown section sectional area gross section $= 305.42\ ^{cm^2}$

For pos. max. moment $m = +59,500\ ^{kgm}$, $N = 111,100\ ^{kg}$

Direct compression $= \dfrac{-111.100}{305.42} = -364\ ^{kg}/_{cm^2}$

Bending stress $= \dfrac{59500 \times 100}{616480} \times 51 = \mp 492$

Extreme fibre stress $= -856\ ^{kg}/_{cm^2}$ comp. on upper flg. and $128\ ^{kg}/_{cm^2}$ Tension on bottom flange.

For max thrust (or full load) $m = +41,750\ ^{kgm}$, $N = -134860\ ^{kg}$

Direct comp $= \dfrac{-134860}{305.42} = -442\ ^{kg}/_{cm^2}$

Bending stress $= \dfrac{41750 \times 100}{616,480} \times 51 = \mp 346$

Extreme fibre stress $= -788\ ^{kg}/_{cm^2}$ comp. on upper flange

or $-\ 96$ " " " lower "

ok

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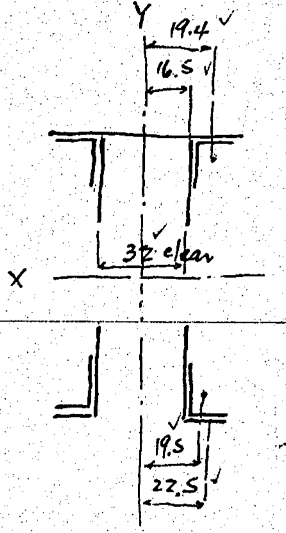
CALCULATIONS FOR

Eian Bashi for Okayama Ken.

Radius of gyration of the arch rib at crown.

$r_x = \sqrt{\frac{I}{A}} = \sqrt{\frac{616480}{305.42}} = 45^{cm}$.



moment of inertia about Y-axis.

Top flange.	2Ls. 150×100×12	= 57.12	$458 + 57.12 \times 19.4^2$	= 21,958
	2Pls 250×9	= 45.00	45.0×16.5^2	= 12,250
	1cov.pl. 560×9	= 50.40		= 13172
		152.52^{cm^2} gr.		$47,380^{cm^4}$
Bottom flange.	2Ls 150×100×15	= 70.50	$557 + 70.5 \times 19.5^2$	= 27,360
	2Pls 250×9	= 45.00	45.0×16.5^2	= 12,250
	2cov.pls. 110×17	= 37.40	$38 + 37.4 \times 22.5^2$	= 18,990
		152.90^{cm^2} gr		$58,600^{cm^4}$
	Total	= 305.42^{cm^2} gr		$105,980^{cm^4}$

radius of gyration $r_x = \sqrt{\frac{105,980}{305.42}} = \sqrt{347} = 18.65^{cm}$.

unsupported length of rib at crown = 5.11^m (between 5-4)

$l/r = \frac{511}{18.65} = 27.4$

allowable unit comp. = $1500(1-0.0055 \times 27.4) = 1275$ use $1,000$ kg/cm²

Fibre Stresses at panel pt. 3. $N = -115690^{kg}$, $m = +74,250^{kgm}$

Direct compression $= \frac{-115690}{305.42} = -379$ kg/cm²

Bending stress $= \frac{74,250 \times 100}{800700} \times 57.3 = \mp 531$

Extreme fibre stress $= -\underline{910}$ c kg/cm² on top. -152 c kg/cm² bottom. ok.

$l/r = \frac{528.3}{18.65} = 28.4$ allowable unit comp. $= 1500(1-.0055 \times 28.4) = 1265$ use 1000 kg/cm² ok.

For min moment. $N = -123200^{kg}$, $m = -22750^{kgm}$

Direct comp. $= \frac{-123200}{305.42} = -404$ kg/cm²

Bending stress $= \frac{-22750 \times 100}{800700} \times 57.3 = \pm 163$ "

Extreme fibre stress $= -241$ kg/cm² c on bottom flg. or -567 kg/cm² c on top. ok.

Fibre Stresses at panel pt 2. $N = -122780^{kg}$, $m = +68200^{kg}$

Direct comp. $= \frac{-122780}{305.42} = -402$ kg/cm²

Bending Stress $= \frac{+68200 \times 100}{1051700} \times 64.8 = \mp 420$

Extreme fibre stress $= -822$ kg/cm² C on top flg. or $+18$ kg/cm² T on bott. flg. ok.

Fibre stress at panel pt. 0. $N = -159000^{kg}$, $m = 0$

Direct comp. $= \frac{-159000}{305.42} = -521$ kg/cm² C ok.

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CALCULATIONS FOR

Eian Bashi for Okayamaken.

Section of web members.

At panel pt. O. — Shear $V = 17600^{kg}$ ✓

vertical member — approx. stress = 17600 × 1.4 = 24600^{kg} comp. — SR = 24.6^{cm^2} gr.

4Ls 75×75×9 ✓ = 50.9 cm^2 gr — 22φ rivet — 7.5 φ ✓

or 4Ls 75×65×8 ✓ = 42.2 ✓ — 8.2

diagonal member — approx stress 18000 T. — SR = 15 cm^2

4Ls 75×75×9 = 50.9 cm^2 gr. — 22φ rivet — 5.5 φ

or 4Ls 75×65×8 = 42.2 ✓ " — " 6 "

At crown. — Shear $V = 16380^{kg}$ ✓

diagonal stress — 16380 × 1.41 = 23200^{kg} T — S.R. = 19.3^{cm^2} net

4Ls 75×75×9 = 50.9 − 9 = 41.9 cm^2 net. — Rivet 22φ — 7.2 φ ✓

or 4Ls 75×65×8 = 42.2 − 8 = 34.2 ✓ " — " " " 7.8 φ ✓

Vert. stress — 16380^{kg} C. — SR = 16.38 ✓

use 4Ls 75×65×8 = 42.2^{cm^2} gr — Rivet 5.5 — 22φ.

Hangers.

Dead Load. — — — — 12500^{kg}.

Live Load.

$$\begin{array}{r} 1970 \\ 280 \\ \hline 2250 \end{array} \times \frac{500}{455} = 2480$$

$$\begin{array}{r} 5320 \\ 710 \\ \hline 6030 \end{array} \times \frac{1.3}{1.182} = \underline{6630}\quad 9100$$

Stress on hanger = 21600^{kg}. T. — S R = 18.0^{cm^2} net

4Ls 90×90×10 = 68.0 − 8×2.5 = 48.0

1 Pl. 300×9 = 27.0 − 2×2.25 = 22.5

95.0 cm^2 gr — 70.5 cm^2 net. — $l/r = \frac{780}{4.1} = 190$ alt.

Δ

JIUN MASUDA
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JIJI BLDG, TOKYO

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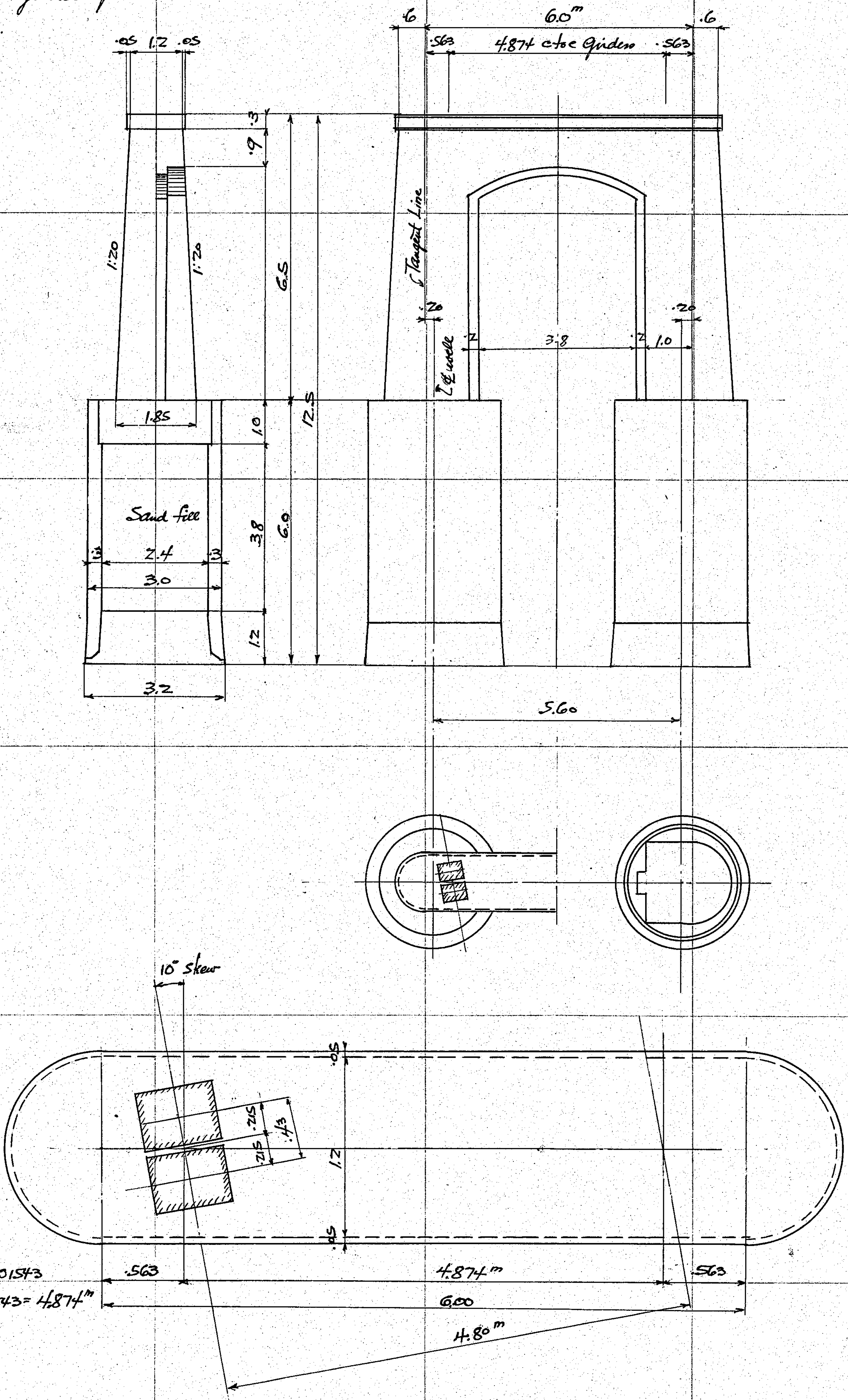
CALCULATIONS FOR

Eian Bashi for Okayama Ken.

Design of Piers
Piers for Girder spans.



Sec 10° = 1.01543
4.8 × 1.01543 = 4.874m

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CALCULATIONS FOR

Eian Bashi for Okayama Ken.

Volume of Concrete for Pier.

Coping $1.3\phi \times 0.3 = 0.40$
" $1.3 \times 6.0 \times 0.3 = \underline{2.34}$
2.74

Shaft. $1.53 \times 1.0 \times 6.2 \times 2 = 18.97$
" $1.53\phi \times 6.2 = 11.40$
" $0.5 \times 0.2 \times 4.7 \times 2 = \underline{0.94}$
31.31

Top Strut $1.5 \times 1.28 \times 4.0 - = 7.68$
$0.5 \times 0.2 \times 3.8 = \underline{0.38}$
8.06

42.11 m^3 for one pier

Well Shell $(3.0\phi - 2.4\phi) \times 5.0 = 12.72$
$(3.1\phi - 2.5\phi) \times 1.0 = \underline{2.64}$
15.36

Top fill $2.4\phi \times 1.0 =$ 4.52
Bottom fill $2.5\phi \times 1.2$ say $=$ $\underline{5.89}$
25.77 m^3 for one well.

Sand fill $2.4\phi \times 3.8 =$ 17.19 m^3 " "

Summary of Concrete for one pier
Shaft 42.11 m^3
Well 2 @ 25.77 = $\underline{51.54}$
Total = 93.65 cub. meters.
Sand fill 2 @ 17.19 = 34.38 " "

Weight and Center of gravity of Pier.
Shaft.

lever arm

Coping 2.74 @ 2400 = 6,570 × 6.35 = 41,700
Shaft 31.31 @ " = 75,200 × 2.80 = 210,500
Top Strut. 8.06 @ " = $\underline{19,330}$ × 5.70 = $\underline{110,200}$
101,100 kg × $\underline{\underline{3.58}}$ = 362,400

For one half of Shaft $S = 101,100 \div 2 = 50,600$ kg
Seismic force $S' = 50600 \times 0.1 = 5,100$ "

Well.

Shell 12.72 @ 2400 = 30,500 × 3.5 = 106,700
" 2.64 @ " = 6,340 × 0.5 = 3,200
Top fill 4.52 @ 2200 = 9,950 × 5.5 = 54,700
Bottom fill 5.89 @ " = 12,960 × 0.6 = 7,800
Sand fill 17.19 @ 1700 = $\underline{29,250}$ × 3.1 = $\underline{90,700}$
wt. of one well $W =$ 89,000 kg × $\underline{\underline{2.96}}$ m = 263,100
Seismic force $W' =$ 89000 × 0.1 = 8,900 kg.

Superimposed Load on Pier.
Load for one-half of pier.
Dead Load 2 Shoes @ 18,000 = 36,000 kg.
Live Load say 2 " @ 16,500 = $\underline{33,000}$
69,000 kg

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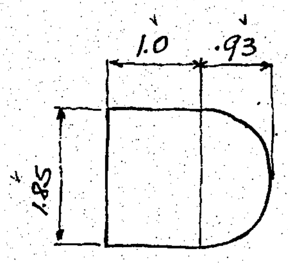
CALCULATIONS FOR

Eian Bashi for Okayama Ken.

Stability of Pier Shaft



Case 1. Stability at normal state.

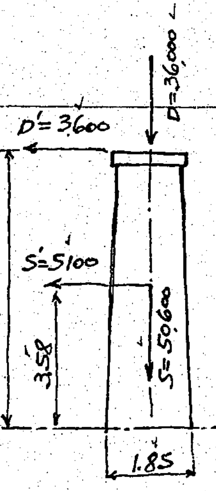
Bottom of shaft.

Sectional area. $1.85 \times 1.0 = 1.85$

$1.85^{\phi} \div 2 = 1.34$

$3.19^{\square m} = 31{,}900\ cm^2$

Unit compression $= \frac{69000}{31900} = 2.16\ kg/cm^2$ ok.



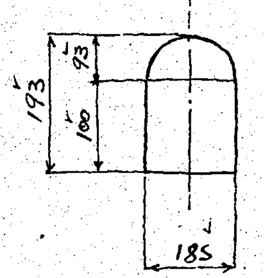
Case 2. Stability during Earthquake. $K = 0.1$ assumed.

Taking moment about center of bottom.

Loads	Horizontal forces	Vertical forces		Lever arm	Moment
D		36,000	×	0 =	0
D'	3600		×	6.5 =	23,400
S		50,600	×	0 =	0
S'	5100		×	3.58 =	18,300
	ΣH = 8,700 kg	ΣV = 86,600 kg	×	0.48 m	ΣM = 41,700 kgm.

Equivalent Rectangle having the same moment of inertia.

Moment of inertia of Bottom area.



$\frac{1.0 \times 1.85^3}{12} = .526$

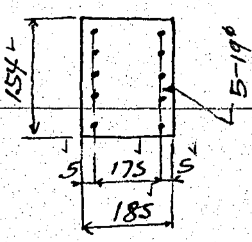
$\frac{0.0491 \times 1.85^4}{2} = .287$

$.813\ m^4$

Moment of inertia of equivalent rectangle.

$\frac{b \times 1.85^3}{12} = .813$

$b = \frac{.813 \times 12}{1.85^3} = 1.54^m.$



Try $5-19^{mm\phi}$ bars on each side $= 5 \times .284 = 14.2^{\square cm}$

Steel ratio $p_0 = 2p = \frac{14.2 \times 2}{154 \times 185} = 0.001$

$e/h = .48/1.85 = .26$ $d'/h = 5/185 = .027$

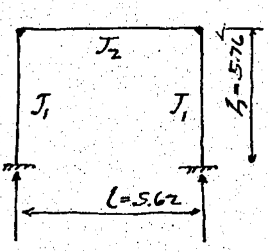
From the prepared diagrams of combined stresses, we obtain:

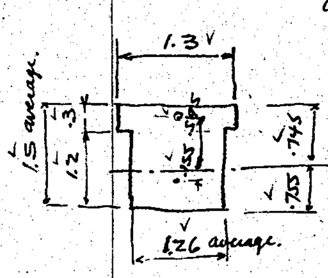
$K = .74,\ L = .098$

$f_c = \frac{M}{Lbh^2} = \frac{41{,}700 \times 100}{.098 \times 154 \times 185^2} = 8.06\ kg/cm^2$ ok

$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 8.06\left(\frac{180}{.74 \times 185} - 1\right) = 39\ kg/cm^2$ ok.

Transverse Moment.





Moment of inertia of Top strut J_2.

area. $1.2 \times 1.26 = 1.51 \times 0.6 = 0.906$

$0.3 \times 1.30 = 0.39 \times 1.35 = 0.527$

$1.90 \times 0.755 = 1.433$

$\frac{1.26 \times 1.2^3}{12} + 1.51 \times .155^2 = .182 + .036 = .218$

$\frac{1.3 \times .3^3}{12} + .039 \times .595^2 = .003 + .138 = .141$

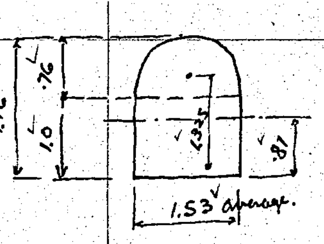
$J_2 = .359\ m^4$

Moment of inertia of shaft J_1

area. $1.53 \times 1.0 = 1.53 \times 0.5 = .765$

$1.53^{\phi} \div 2 = .92 \times 1.325 = 1.220$

$2.45 \times .81 = 1.985$



$\frac{1.53 \times 1.0^3}{12} + 1.53 \times .31^2 = .128 + .147 = .275$

$.0069 \times 1.53^4 + .92 \times .515^2 = .039 + .244 = .283$

$J_1 = .558\ m^4$

$h = 6.50 - .745 = 5.76^m$ $l = 4.0 + .81 \times 2 = 5.62$

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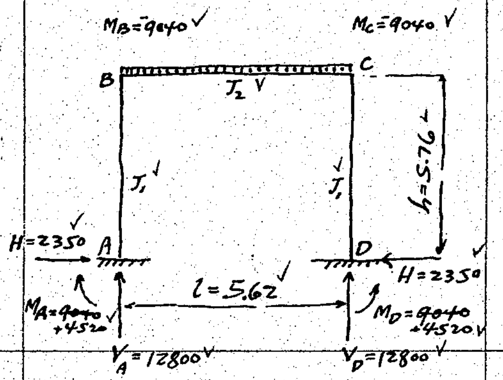
CALCULATIONS FOR

Eian-Bashi for Okayama Ken.

Dead Load moment



Weight of top strut. $1.9 \times 2400 = 4560$ kg/lin meter $= q$

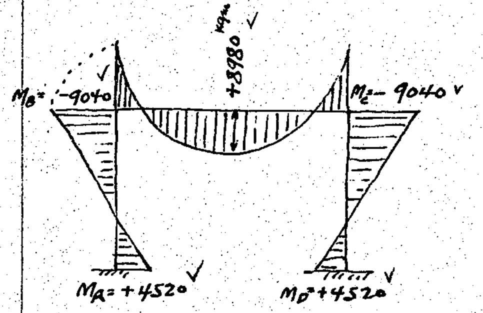
See Kleinlogel's Rahmenformeln Page 89.

$$K = \frac{J_2}{J_1} \cdot \frac{h}{l} = \frac{.359}{.558} \times \frac{5.76}{5.62} = .66$$

$$V = \frac{ql}{2} = \frac{4560 \times 5.62}{2} = 12{,}800 \text{ kg}$$

$$H = \frac{ql^2}{4h(K+2)} = \frac{4560 \times 5.62^2}{4 \times 5.76 \times 2.66} = 2350 \text{ kg}$$

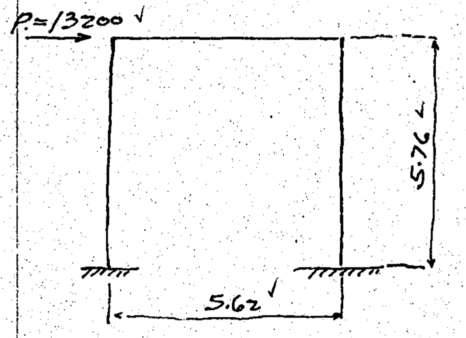
$$M_B = M_C = -\frac{ql^2}{6(K+2)} = -\frac{4560 \times 5.62^2}{6 \times 2.66} = -9{,}040 \text{ kgm}$$



$$M_A = M_D = +\frac{ql^2}{12(K+2)} = +4{,}520 \text{ kgm.}$$

$$M_{max} = +\frac{ql^2}{24} \cdot \frac{2+3K}{K+2} = \frac{4560 \times 5.62^2}{24} \times \frac{3.98}{2.66} = +8980 \text{ kgm.}$$

Seismic moment.



Seismic force due to

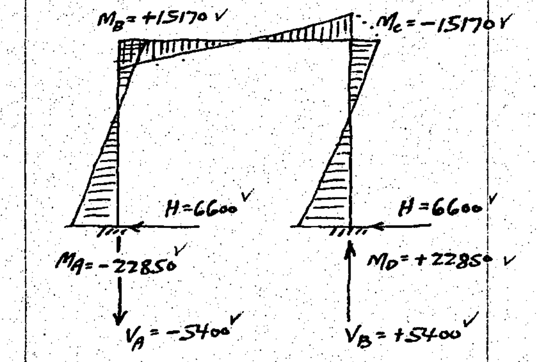
		Top	bottom
Top strut	$= 4560 \times 4.0 \times 0.1 =$	1,830 kg	—
Shaft. say	$80{,}000 \times 0.1 \times \frac{3.0}{5.76} =$	4160	3840
D.L. on shoes	$\frac{36000 \times 2 \times .1}{170{,}300} =$	7200	—
		13,190	3,840

Vert. load on one shaft = 85,200 kg   Call this 13,200 kg = P.

$$K = 0.66, \quad 3K = 1.98, \quad 6K+1 = 4.96, \quad 3K+1 = 2.98$$

$$V = \frac{3Phk}{l(6K+1)} = \frac{3 \times 13200 \times 5.76 \times .66}{5.62 \times 4.96} = 5{,}400 \text{ kg}$$

$$H = P \div 2 = 13200 \div 2 = 6{,}600 \text{ kg}$$



$$M_A = -\frac{Ph}{2} \cdot \frac{3K+1}{6K+1} = -\frac{13200 \times 5.76}{2} \cdot \frac{2.98}{4.96} = -22{,}850 \text{ kgm.}$$

$$M_D = + \quad " \quad " \qquad = +22850$$

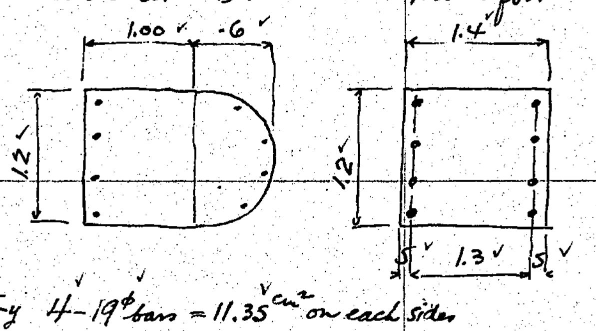
$$M_B = +\frac{Ph}{2} \cdot \frac{3K}{6K+1} = +\frac{13200 \times 5.76}{2} \times \frac{1.98}{4.96} = +15{,}170$$

$$M_C = - \quad " \quad " \qquad = -15{,}170$$

Summary of moments, Reactions + Hor. thrush.

	M_A	M_B	M_C	M_D	M_{center}	V	H.
Dead Load.	+4520	-9,040	-9040	+4520	+8980	+12,800	± 2350
Seismic force.	-22850	+15170	-15,170	+22850	0	± 5400	+ 6600
	-18,330 kgm	+ 6,130 kgm	-24210 kgm	+27370 kgm	+8980 kgm	+18,200 kg	+ 8,950 kg
						- 7400	+ 4250

Section at C.



Transformed rectangle of equal moment of inertia assumed. 1.2 × 1.4 m

Vert. load. = D.L. on shoes = 36000 kg

Top str. = 9200

45,200 kg.

moment $M_C = -24210$ kgm.

Eccentricity $\varepsilon = \frac{24210}{45200} = 0.535$ m

$\varepsilon/h = .535/1.4 = .382$, $d'/h = .05/1.4 = .036$

$K = .49$, $L = .091$

$$f_c = \frac{24210 \times 100}{.091 \times 120 \times 140^2} = 11.3 \text{ kg/cm}^2 \text{ ok}$$

$$f_s = 15 \times 11.3 \left(\frac{135}{.49 \times 140} - 1\right) = 164 \text{ kg/cm}^2 \text{ ok.}$$

Try 4-19φ bars = 11.35 cm² on each sides

$$p_0 = 2p = \frac{11.35 \times 2}{140 \times 120} = .00135$$

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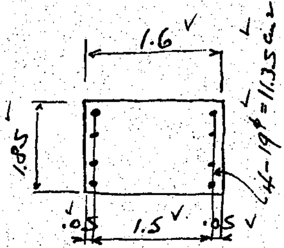
CALCULATIONS FOR

Eian-Bashi for Okayama Ken.

Section at Bottom of Shaft. Transformed rectangle of equal moment of inertia say 1.6×1.85



Moment $M_D = 27370$ kgm

Vert load. Dead Load on shoes = 36000 kg
Top strut + Shaft = 50600
86600
V = 18200
N = 104800 kg

Eccentricity $\varepsilon =$ ~~1048~~ $\frac{27370}{104800} = .261$

$\varepsilon/h = .261/1.6 = .163$, $d'/h = .05/1.6 = .031$, $p_0 = 2p = \frac{11.35 \times 2}{185 \times 160} = 0.0008$

$K = 1.95$

$f_c = \frac{NK}{bh} = \frac{104800 \times 1.95}{185 \times 160} = 6.9$ kg/cm² ok

$f_s = 6.9 \times 15 = 104$ kg/cm² ok.

Stability of pier as a whole.

Case 1. Stability at normal state.

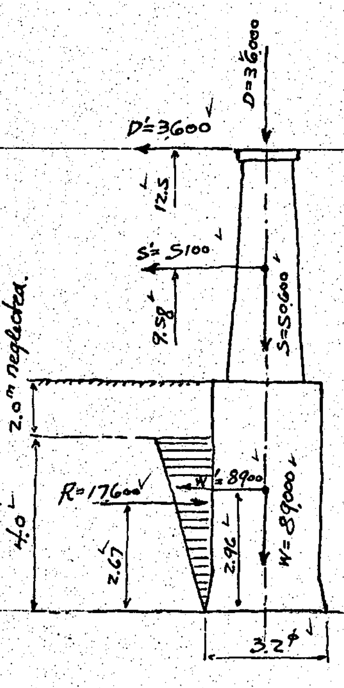
Super-imposed Loads on pier for one well.
D.L. + L.L. = 69000 kg
Wt. of Shaft = 50600
" " well = 89000
208600 kg

If the frictional resistance be taken into account,
friction = 3.0φ × 4.0 × 1000 = 37700 kg
load on bottom = 208600 − 37700 = 170900 kg
unit bearing pres. = $\frac{170900}{8.04}$ = 21300 kg/m² or (1.94 tons/□')

Bottom area of well = 3.2 mφ = 8.04 m²

Unit bearing pressure = $\frac{208600}{8.04}$ = 25950 kg/m² or (2.37 tons/□') ok.

Case 2. Stability during Earthquake. K = 0.1 assumed.



Loads	Hor. forces	Vert. forces	Leverarm	Moment
D		36000	× 0	= 0
D'	3600		× 12.5	= 45000
S		50600	× 0	= 0
S'	5100		× 9.58	= 48800
W		89000	× 0	= 0
W'	8900		× 2.96	= 26400
	ΣH = 17600 kg	ΣV = 175600 kg		ΣM = 120200 kgm

Frictional resistance on well 1000 kg/m², effective penetration of well assumed 4.0 m.
average width say 3.0 × 0.7 = 2.10
frictional couple = 3.0 × 1000 × 2.10 × 4 = 25200 kgm
Earth reaction, moment = 17600 × 2.67 = 47000
72200 kgm.

Resultant moment 120200 − 72200 = 48000 kgm

Eccentricity $\varepsilon = \frac{48000}{175600} = 0.273$ m

~~m~~ Section modulus of bottom area = .0982 × 3.2³ = 3.21 m³. Area 3.2φ = 8.04 m²

max. toe pressure = $\frac{175600}{8.04} \pm \frac{48000}{3.21}$ = 21850 ± 14950 = 36800 kg/m² = (3.36 tons/□')
or 6900 = (.63 tons/□')

max. lateral pressure on earth 2.0 m from ground surface

$\frac{17600 \times 2}{3.0 \times 4.0} = 2935$ kg/m²

Safe lateral bearing power of earth 2.0 below ground surface.

$= wh \frac{1+\sin\phi}{1-\sin\phi} = 1600 \times 2.0 \times 3 = 9600$ kg/m² > 2935 ok.

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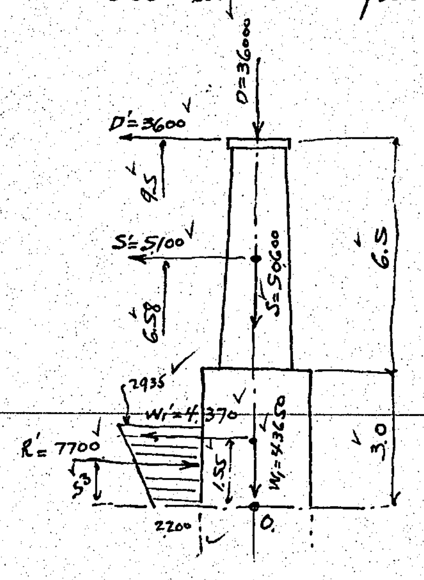
CALCULATIONS FOR

Eian-Bashi for Okayamaken.

Moment at 3.0m from top of well.



Weight of well.

Shell	30500 × $\frac{3}{5}$ =	18,300 × 1.5 =	27400
Top fill		9,950 × 2.5 =	24900
Sand fill	29250 ~~17.19~~ × $\frac{2.0}{3.8}$ =	15,400 × 1.0 =	15400
		W1 = 43,650 Kg × 1.55 =	67,700

$R' = \frac{2935+2200}{2} \times 1 \times \overset{3}{\cancel{4}} = 7700$ kg

Taking moment about O.

Loads	Hor. forces	Vert. forces	Lev. arm	Moment.
D		36000 ×	0 =	0
D'	3600	×	9.5 =	34200
S		50600 ×	0 =	0
S'	5100	×	6.58 =	$3\overset{3}{\cancel{4}}550$
W1		43650 ×	0 =	0
W1'	4370	×	1.55 =	6770
R'	− 7700	×	0.53 =	−4080
	5370	130,250 kg		70,440 Kgm

less frictional couple 25200 ÷ 4 = 6300

64,140 Kgm.

Eccentricity $\varepsilon = \frac{64140}{130250} = 0.49^m$

Moment of inertia of well $= 0.0491 (3.0^4 - 2.4^4) = 2.35 \, m^4$ (47.8) area $= 3.0^{\phi} - 2.4^{\phi} = 2.55 \, ^{\square m}$

Extreme fibre stress $= \frac{130250}{2.55} \pm \frac{64140 \times 1.5}{2.35} = 51100 \pm 40900 = 92000 \, Kg/m^2 C = 9.2 \, Kg/cm^2$ OK

or 10,200 = 1.0

No vertical reinforcement required theoreatically, but, for practical porpose use 13mmφ bars at 50cm c to c on both sides.

Horizontal reinforcement same as for vertical bars.

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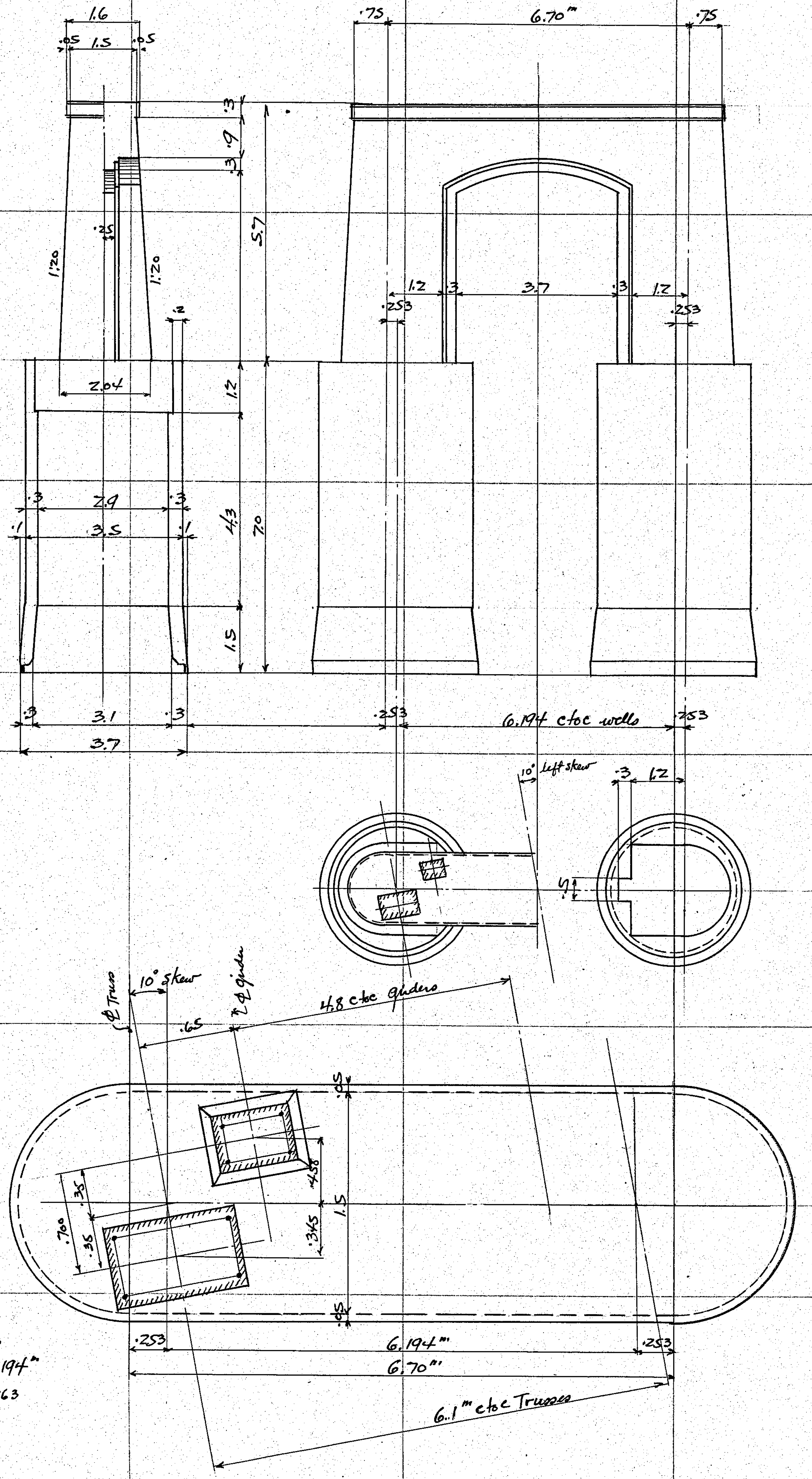
CALCULATIONS FOR

Eian-Bashi for Okayama Ken.
Piers Between Girder and Arch spans.



Sec 10° = 1.01543
6.1 × 1.01543 = 6.194m
~~$~~ tan 10° = 0.1763

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CALCULATIONS FOR

Eian-Bashi for Okayama Ken

Volume of Concrete for Pier

Coping 1.6ϕ√ × 0.3√ = 0.60√
" 1.6√ × 6.7√ × 0.3√ = 3.21√
3.81√

Shaft. 1.77√ × 1.2√ × 5.4√ × 2√ = 22.90√
" 1.77ϕ√ × 5.4√ × ~~2~~ = 13.30√
" 0.5√ × 0.3√ × 3.9√ × 2√ = 1.17√
37.37√

Top strut 1.5√ × 1.58√ × 4.3√ = 10.20√
" 0.5√ × 0.3√ × 3.7√ = .56√
10.76√
51.94 m³√ for one pier shaft.

Well. Shell (3.5ϕ√ − 2.9ϕ√) 5.5√ = 16.58√
(3.6ϕ√ − 3.0ϕ√) 1.5√ = 4.67√
21.25√

Top fill 2.9ϕ√ × 1.2√ = 7.93√
Bottom fill 3.0ϕ√ × 1.5√ = 10.60√
39.78 m³√ for one well.

Sand fill 2.9ϕ√ × 4.3√ = 28.40 m³√ for one well.

Summary of Concrete for one pier.
Shaft 51.94√
well 2 @ 39.78√ = 79.56√
131.50√ cub.m.

Sand fill 2 @ 28.4√ = 56.80√

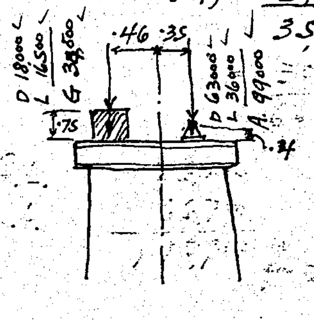
Weight and center of gravity of Pier.
Shaft.
Coping 3.81√ @ 2400√ = 9150√ × 5.55√ = 50,800√
Shaft√ 37.37√ @ " √ = 89600√ × 2.50√ = 224,000√
Top strut 10.76√ @ " √ = 25850√ × 4.95√ = 128,000√
124,600√ × 3.23 m√ = 402,800√

For one-half of Shaft S = 124600√ ÷ 2√ = 62300√ kg
Seismic force S′ = 62300√ × .1√ = 6200√

Well
Shell. 16.58√ @ 2400√ = 39,800√ × 4.25√ = 169,000√
" 4.67√ @ " √ = 11,200√ × .75√ = 8,400√
Top fill 7.93√ @ 2200√ = 17,500√ × 6.40√ = 112,000√
Bottom fill 10.60√ @ " √ = 23300√ × .75√ = 17,500√
Sand fill 28.40√ @ 1700√ = 48,300√ × 3.65√ = 176,500√
Weight of one well W = 140100√ kg × 3.45 m√ = 483,400√
Seismic force W′ = 14000√ kg

Superimposed Loads on pier. due to arch span due to Girder span
Dead Load. 5 @ 12500 = 62,500√ kg 18,000√
Shoe say = 500√
63,000√ + 18000√ = 81,000√ kg for one-half of pier

Live Load.
8280 × 1.0√ = 8,280√ 16500√
6150 × (.1 + .2 + .3 + .4 + .5 + .6 + .7 + .8 + .9) = 27680√
35,960√ 16,500√ say = 52,500√ kg for one-half of pier
Total = 133,500√ kg " "

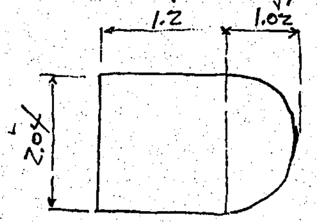


Eian-Bashi for Okayama Ken.

P9

Stability of Pier Shaft.

Case 1. Stability at normal state.



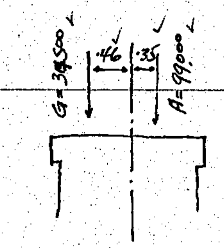
Bottom of Shaft.

Sectional area

$$2.04 \times 1.2 = 2.45$$
$$2.04^{2} \div 2 = \underline{1.64}$$
$$4.08^{\ m} = 40{,}900\ \text{cm}^2$$
9

Moment of inertia of the bottom area.

$$\frac{1.2 \times 2.04^3}{12} = .848$$
$$\frac{0.0491 \times 2.04^4}{2} = \underline{.425}$$
$$1.273^{\ m^4} = 127300000\ \text{cm}^4$$



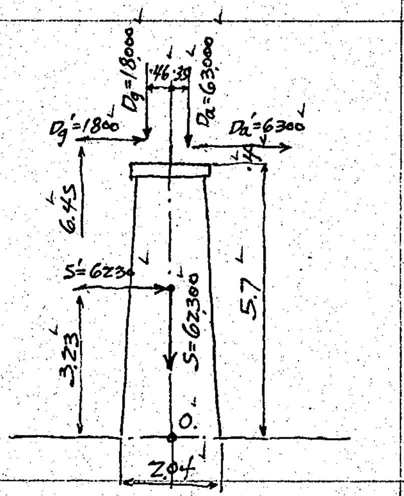
Moment on shaft.

$$A \quad 99000 \times .35 = 34650$$
$$G \quad \underline{34500} \times .46 = -\underline{15850}$$
$$133{,}500 \qquad 18{,}800\ \text{kgm}$$

$$\text{Unit bearing pressure} = \frac{133500}{40900} \pm \frac{18800 \times 100 \times 102}{127{,}300{,}000}$$

$$= 3.26 \pm 1.51 = 4.77\ \text{kg/cm}^2\ C \text{ or } 1.75\ \text{kg/cm}^2 \quad \text{ok.}$$

Case 2 Stability during Earthquake. K. = 0.1 assumed.



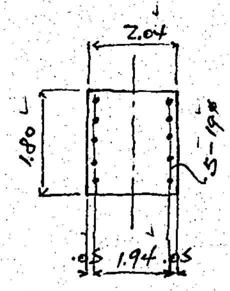
Taking moment about O.

Loads	Horizontal forces	Vertical forces		Lever arms		Moments
Dg		18,000	×	−.46	=	−8300
Dg'	1,800		×	+6.45	=	+11,600
Da		63,000	×	+.35	=	+22,000
Da'	6,300		×	+6.10	=	+38,400
S		62,300	×	0	=	0
S'	6,230		×	+3.23	=	+20,100
	ΣH= 14,330 Kg	ΣV= 143300 kg	×	.58	ΣM=	83,800 kgm

Equivalent rectangle of same moment of inertia

$$\frac{b \times 2.04^3}{12} = 1.273$$

$$\text{or } b = \frac{1.273 \times 12}{2.04^3} = 1.80^{m}$$



$$e/h = .58/2.04 = .284, \quad d'/h = .05/2.04 = .025$$

$$p_0 = 2p = \frac{14.2 \times 2}{204 \times 180} = .0007$$

From the prepared diagrams of combined stresses, we have:

$$K = .67 \qquad L = .096$$

$$f_c = \frac{83800 \times 100}{.096 \times 180 \times 204^2} = 11.7\ \text{kg/cm}^2 \quad \text{ok.}$$

$$f_s = 15 \times 11.7 \left(\frac{199}{.67 \times 204} - 1\right) = 80.0\ \text{kg/cm}^2 \quad \text{ok.}$$

$$\text{Unit Shear} = \frac{14330}{180 \times \frac{7}{8} \times 199} = .45\ \text{kg/cm}^2 \quad \text{ok}$$

$$\text{Unit bond} = \frac{14330}{5 \times 5.97 \times \frac{7}{8} \times 199} = 2.8\ \text{kg/cm}^2 \quad \text{ok.}$$

For transverse moment use the same reinforcements as for arch pier see page 35

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CALCULATIONS FOR

Eian-Bashi for Okayama Ken.

Stability of pier as a whole.

Case 1. Stability at normal state.

Superimposed Dead and Live Load = 133,500 kg

Eccentric loading. moment = 18,800 kgm. see the last page.

Loads on pier	D.L. + L.L.	= 133,500
weight of shaft		62,300
wt. of well.		140,100
		335,900 kg

Frictional resistance on well 1200 kg/m² assumed for this case. Effective penetration say 6.0m.

Average width say 0.7 × 3.7 = 2.59m

Frictional couple = 3.7 × 6 × 1200 × 2.59 = 69,000 kgm

Frictional couple overcome the moment due to eccentric loading. Bottom area = 3.7ϕ = 10.752 ☐m

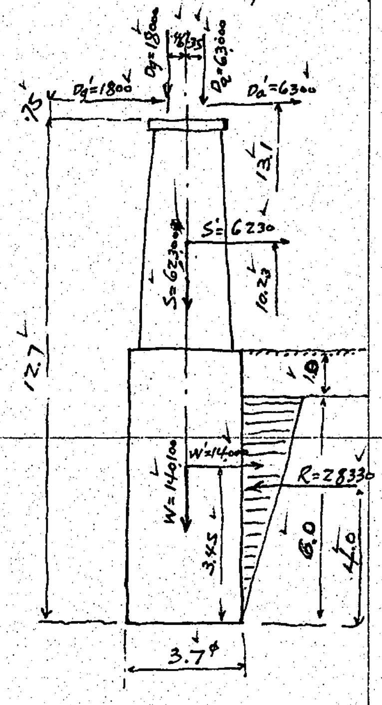
Frictional resistance on well surface = 3.5ϕ × 6.0 × 1200 = 79,400 kg

Counting frictional resistance, Load on bottom of wall

335900
79,400
256,500 kg

Unit Bearing pressure = $\frac{256,500}{10.752}$ = 23,400 kg/m² or (2.18 tons/☐') ok.

Case 2. Stability during Earthquake. k assumed 0.1



Taking moment about center of bottom area.

Loads	Hor. forces,	Vert. forces	Lev. arms	Moments.
Dg		18,000 ×	−.46 =	− 8,300
Dg'	1,800	×	13.45 =	24,200
Da		63000 ×	.35 =	22,000
Da'	6,300	×	13.10 =	82,500
S		62,300 ×	0 =	0
S'	6230	×	10.23 =	6$\overset{3}{\cancel{4}}$,800
W		140,100 ×	0 =	0
W'	14000	×	3.45 =	48,300
R.	−28330	×	+ 4.00 =	−115,400
	0	ΣV = 283,400 kg ×	=	117,100 kgm

Frictional couple. See case 1. − 69,000
48,100 kgm

Eccentricity $\varepsilon = \frac{48100}{283400} = .17^m < \frac{d}{8} = \frac{3.7}{8} = .463$

Resultant force within core area.

moment of inertia of bottom area = .0491 × 3.7⁴ = 9.22 m⁴

Unit bearing pressure = $\pm\frac{48,100 \times 1.85}{9.22} + \frac{283400}{10.752} = \pm 9650 + 26,300$

= 35950 kg/m² (3.29 tons/☐') Ok.

or = 16650 " (1.52 ") ok.

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MADE BY K. Inaba DATE 4-5-9 FILE NO

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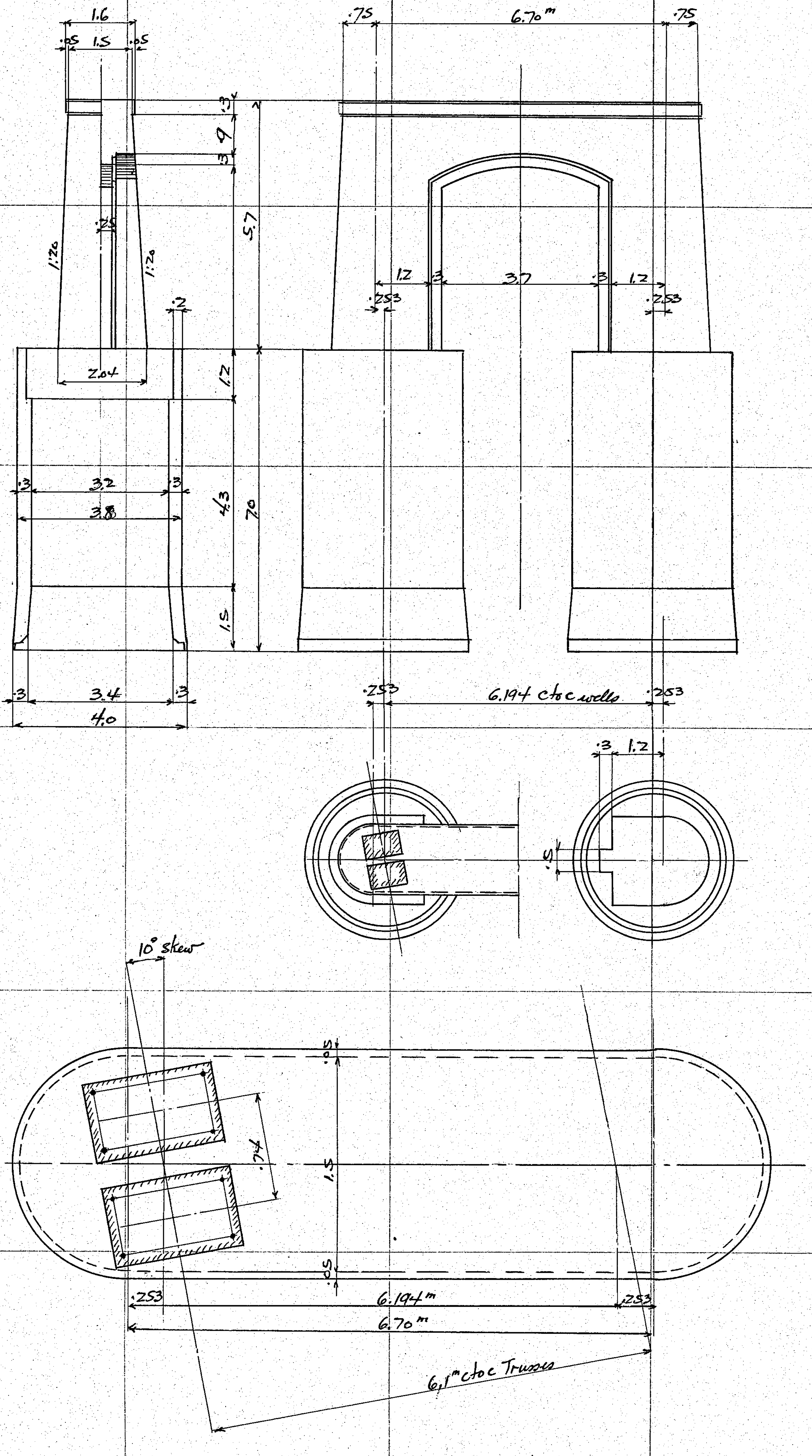
CALCULATIONS FOR

Eiau-Bashi for Okayama Ken.
Pier for Arch Span.



MADE BY K. Inaba DATE 4-5-9 **FILE NO**

CHECKED BY M.J. DATE 4-5-25 **PAGE NO** 34

CALCULATIONS FOR

Eian-Bashi for Okayamaken.

Volume of Concrete, weight and center of gravity of Shaft, same as for pier between girder + arch spans.

Superimposed Loads on pier:

Dead Load. panel load. $5 @ 12{,}500 = 62500^{kg}$

Shoe say 500

$63000^{kg} \times 2 = 126000^{kg}$ for 2 Shoes or $\frac{1}{2}$ of pier

Live Load.

1st span. $8280 \times 1.0 = 8280$

$6{,}150 (.1+.2+.3 \cdots +.9) = 27680$

35960

2nd span $6150 \times 5 = 30750$

66710 call this $66{,}700$

$192{,}700^{kg}$ for one-half of pier.

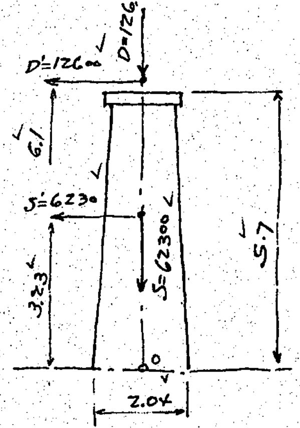
Eccentricity is very small and neglected.

Stability of Shaft.

Case 1. Stability at normal state.

$$\text{Unit bearing pressure} = \frac{192{,}700}{40{,}900} = 4.72^{kg/cm^2} \quad ok.$$

Case 2 Stability during Earthquake.

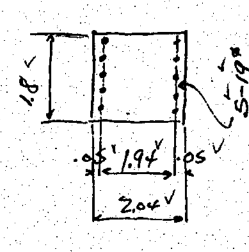


Loads	Hor. forces	Vert. forces	Lev. arm	Moment.
D		126,000 ×	0 =	0
D'	12600	×	6.1 =	76800
S		62300 ×	0 =	0
S'	6230	×	3.23 =	20100
	$\Sigma H = 18830^{kg}$	$\Sigma V = 188{,}300^{kg}$ ×	.515 =	$\Sigma M = 96{,}900^{kgm}$

Equivalent rectangle having same moment of inertia: 2.04×1.80

$e/h = .515/2.04 = .253$, $d'/h = .025$, $p_0 = .0007$.

$K = .76$, $L = .097$



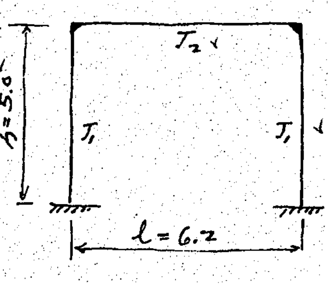
$$f_c = \frac{96900 \times 100}{.097 \times 180 \times 204^2} = 13.3^{kg/cm^2} \quad ok$$

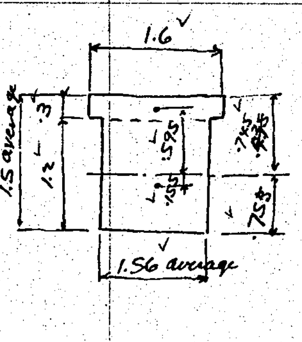
$$f_s = 15 \times 13.3 \left(\frac{199}{.76 \times 204} - 1\right) = 57^{kg/cm^2} \quad ok$$

$$\text{Unit Shear} = \frac{18830}{180 \times \frac{7}{8} \times \cancel{20}\,199} = .6 \quad ok$$

$$\text{unit bond} = \frac{18830}{5 \times 5.97 \times \frac{7}{8} \times 199} = 3.62 \quad ok.$$

Transverse Moment





moment of inertia of Top strut J_2.

Area. $1.2 \times 1.56 = 1.87 \times .6 = 1.122$

$0.3 \times 1.6 = .48 \times 1.35 = .648$

$2.35 \times \overset{.755}{\cancel{.53}} = 1.770$

$\frac{1.56 \times 1.2^3}{12} + 1.87 \times .155^2 = .225 + .0449 = .270$

$\frac{1.6 \times .3^3}{12} + .48 \times .595^2 = .0036 + .170 = .174$

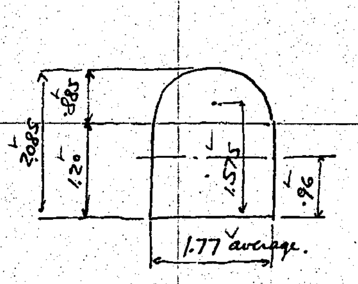
$J_2 = .444^{m^4}$

Moment of inertia of Shaft J_1.

Area $1.77 \times 1.2 = 2.13 \times .6 = 1.28$

$1.77^{\phi} \div 2 = 1.23 \times 1.575 = 1.94$

$3.36 \times .96 = 3.22$



$\frac{1.77 \times 1.2^3}{12} + 2.13 \times .36^2 = .255 + .276 = .531$

$.0069 \times 1.77^4 + 1.23 \times .615^2 = .068 + .465 = .533$

$J_1 = 1.064^{m^4}$

$h = 5.7 - .745 = 4.96$ call this 5.0^m

$l = 4.3 + .96 \times 2 = 6.22 \quad \therefore \quad 6.2^m$

$$k = \frac{J_2}{J_1} \cdot \frac{h}{l} = \frac{.444}{1.064} \times \frac{5.0}{6.2} = .336$$

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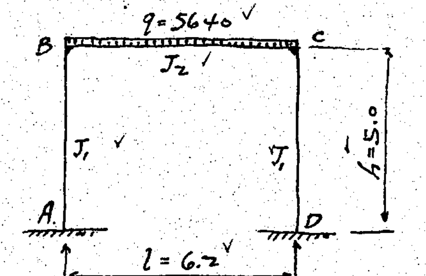
CALCULATIONS FOR

Eian-Bashi for Okaya-Ken.

Dead Load moment.



Weight of Top strut. $2.35 \times 2400 = 5640^{kg}$ / lin meter of span. = q.

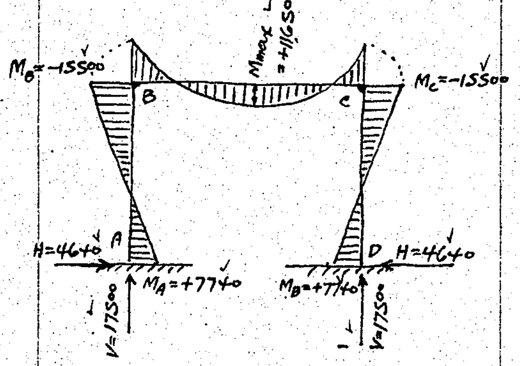
$K = .336$

$V = \frac{ql}{2} = \frac{5640 \times 6.2}{2} = 17500^{kg}$

$H = \frac{ql^2}{4h(K+2)} = \frac{5640 \times 6.2^2}{4 \times 5.0 \times 2.336} = 4640$ kg

$M_B = M_C = -\frac{ql^2}{6(K+2)} = -\frac{5640 \times 6.2^2}{6 \times 2.336} = -15500$ kgm.

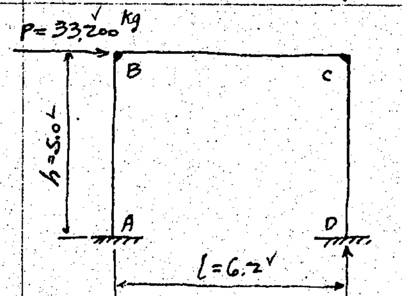
$M_A = M_D = +\frac{ql^2}{12(K+2)} = +\frac{5640 \times 6.2^2}{12 \times 2.336} = +7740$ kgm.



$M_{max} = +\frac{ql^2}{24} \cdot \frac{2+3K}{K+2} = +\frac{5640 \times 6.2^2}{24} \times \frac{3.008}{2.336} = +11650$ kgm

Transverse Seismic moment. Seismic forces due to.

		Top	Bottom
Top strut.	$5640 \times 4.3 \times 0.1 =$	2430	—
Shaft	say $99000 \times 0.1 \times \frac{2.8}{5.0} =$	5540	4360
D.L. on shoes	$\frac{126000 \times 2 \times .1}{\Sigma V = 375300} =$	25200	—
		33170	4360

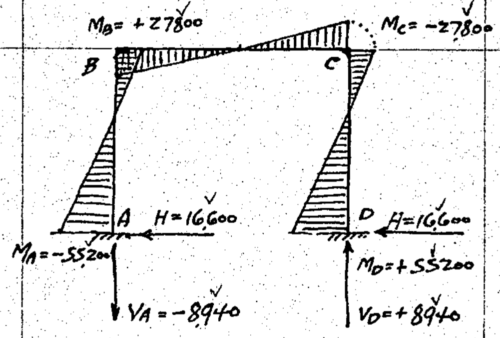


Vert. load on one shaft = 187700 kg. Call this 33200 kg = P.

$K = .336$, $3K = 1.01$, $6K+1 = 3.02$, $3K+1 = 2.01$.

$V = \frac{3PKh}{l(6K+1)} = \frac{3 \times 33200 \times .336 \times 5.0}{6.2 \times 3.02} = \pm 8940$ kg

$H = \frac{P}{2} = \frac{33200}{2} = 16600$ kg



$M_A = -\frac{Ph}{2} \cdot \frac{3K+1}{6K+1} = -\frac{33200 \times 5.0}{2} \cdot \frac{2.01}{3.02} = -55200$ kgm.

$M_D = +$ 〃 $= +55200$ kgm

$M_B = +\frac{Ph}{2} \cdot \frac{3K}{6K+1} = +\frac{33200 \times 5.0}{2} \times \frac{1.01}{3.02} = +27800$ kgm

$M_C = -$ 〃 $= -27800$ kgm.

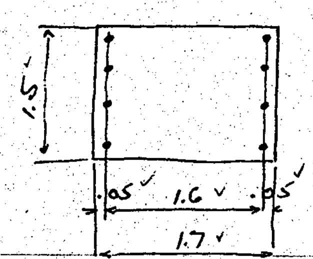
Summary of moments, Reactions and Horizontal thrusts.

	M_A	M_B	M_C	M_D	M_{center}	V	H.
Dead Load.	+ 7740	− 15500	− 15500	+ 7740	+ 11650	+ 17500	± 4640
Seismic force	− 55200	+ 27800	− 27800	+ 55200	0	± 8940	+ 16600
	− 47460	+ 12300	− 43300	+ 62940	+ 11650	+ 26440	+ 21240
						+ 8560	+ 11960

Section at C

Transformed rectangle of equal moment of inertia assumed $1.5 \times$ ~~2.2~~ 1.7

vert. load
D.L. on shoes. 126000
D.L. V. 17500
Seismic V. 8940
152440 call this 152500 kg = N.



$M_C = -43300$ kgm Eccentricity $\varepsilon = \frac{43300}{152500} = 0.284^m$

$\varepsilon/h = .284/1.7 = .167$, $d'/h = .05/1.7 = .03$

$K = 1.96$,

Try $4 - 16^\phi$ bars $= 8.04^{cm}$

$p_0 = \frac{2 \times 8.04}{150 \times 170} = .00063$

$f_c = \frac{NK}{bh} = \frac{152500 \times 1.96}{150 \times 170} = 11.7$ kg/cm² C ok

Compression over whole section.

$f_s = 11.7 \times 15 = 176$ kg/cm² C. ok

8

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MADE BY K. Inaba DATE 4-5-9 **FILE NO**

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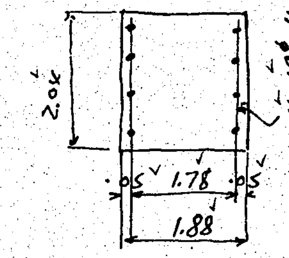
CALCULATIONS FOR

Eian-Bashi for Okayama Ken

Section at Bottom of Shaft. Transformed rectangle of equal moment of inertia say 2.04 × 1.88



moment M_D = + 62,940 kgm H = 21240 kg

vert. load D.L. on Shoes 126,000
D.L. V 17,500
Seismic V 8940
Wt. Shaft. 62300 − 17500 = 44800
197,240 call this 197,200 kg

Eccentricity $\varepsilon = \frac{62,940}{197,200} = 0.32^{m}$, $\varepsilon/h = 0.32/1.88 = 0.17$

$d'/h = .05/1.88 = .0266$, $p_o = \frac{11.35 \times 2}{204 \times 188} = .0006$

$K = .995$, $L = .086$

$f_c = \frac{62940 \times 100}{.086 \times 204 \times 188^2} = 10.2$ kg/cm² ok

$f_s = 15 \times 10.2 \left(\frac{183}{.995 \times 188} - 1\right)$ = negligible.

Unit shear $= \frac{21240}{204 \times \frac{7}{8} \times 183} = .65$ kg/cm² ok

Unit bond $= \frac{21240}{4 \times 5.97 \times \frac{7}{8} \times 183} = 5.55$ kg/cm² ok.

Stability of Pier as a whole.
Case 1. Stability at normal state.

Superimposed Loads on pier for one well.
D.L. + L.L. = 192,700 kg
weight of Shaft = 62,300
" " well = 164,200 See below
419,200 kg

Frictional resistance $6.0 \times 3.8^{\phi} \times 1200 = 81700$ kg
Resulting load = 419,200 − 81700 = 337,500 kg
~~Unit bearing pressure = $\frac{337,500}{4.0^{\phi}}$ = 26800 kg/m² or (2.45 tons/□')~~

Weight, volume of concrete and center of gravity of well.

Shell	$(3.8^{\phi} - 3.2^{\phi}) \times 5.5$ =	18.15	@ 2400 =	43600	× 4.25 =	185,300
"	$(3.9^{\phi} - 3.3^{\phi}) \times 1.5$ =	5.09	@ 2400 =	12,200	× .75 =	9,200
Top fill	$3.2^{\phi} \times 1.2$ =	9.66	@ 2200 =	21,300	× 6.4 =	136,000
bottom fill	$3.3^{\phi} \times 1.5$ =	12.83	@ 2200 =	28,300	× .75 =	21,200
		45.73 m³		105,400	× ~~2.27~~ =	~~239300~~
Sand fill	$3.2^{\phi} \times 4.3$ =	34.60	@ 1700 =	58,800	× 3.65 =	215,000
				164,200	× 3.45 m =	~~454300~~ 566700

Total volume of concrete for one pier
Shaft. 51.94
2 wells @ 45.73 = 91.46
143.40 m³

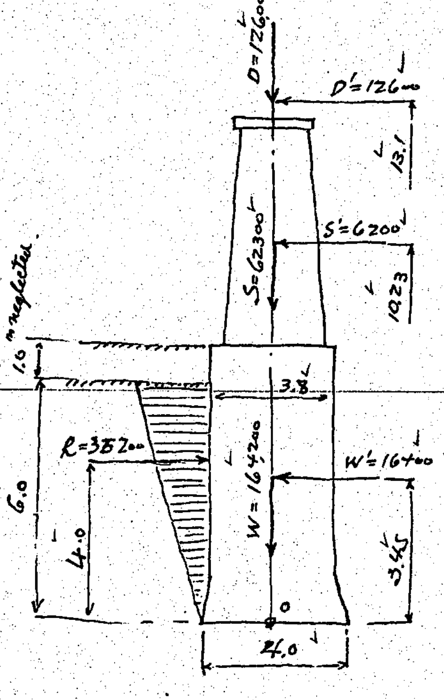
Sand fill " = 34.60 m³

wt. for ½ of pier Shaft S = 62,300 kg S' = 6,200 kg
well W = 164,200 W' = 16,400

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Eian-Bashi for Okayama Ken

Case 2. Stability during Earthquake.



Taking moment about center of base O.

Loads	Hor. forces	Vert. forces	Lever arm	Moments.
D		126000	0	0
D'	12600		13.1	165000
S		62300	0	0
S'	6200		10.23	63500
W		164200	0	0
W'	16400		3.45	56500
R	−35200		4.00	−141000
	0	ΣV= 352500		ΣM= 144000

Frictional couple. lev arm say $3.8 \times .7 = 2.6^m$

$3.8 \times 1200 \times 6.0 \times 2.6 = 71200$ kgm.

Resulting moment $= 144000 - 71200 = 72800$ kgm.

Eccentricity $E = \frac{72800}{352500} = 0.21^m < \frac{4.0}{8} = 0.5$ within the kern linea.

moment of inertia of bottom area $= 0.0491 \times 4.0^4 = 12.56\ \mathrm{m^4}$, area $4.0^\phi = 12.57\ \mathrm{m^2}$

max. toe pressure $= \frac{352500}{12.57} \pm \frac{72800 \times 2.0}{12.56} = 28000 \pm 11600$

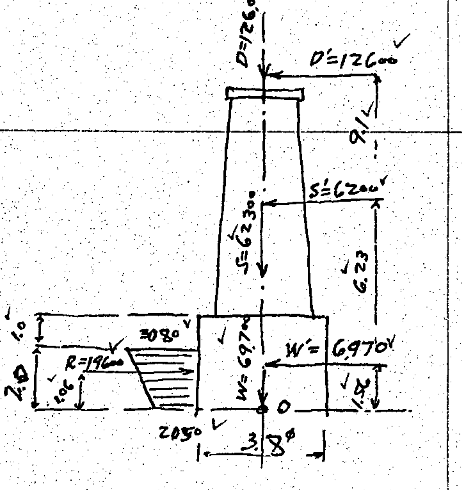
$= 39600$ kg/m² c (3.62 tons/□') ok

or $= 16400$ " c " (1.50 " ")

Max. lateral bearing pressure on earth $= \frac{35200 \times 2}{3.8 \times 6.0} = 3080$ kg/□m

allowable " " " " $= 1600 \times 1 \times 3 = 4800$ " > 3080 ok.

Section at 4.0m above bottom.



$R = \frac{3080 + 2050}{2} \times 2.0 \times 3.8 = 19600$

wt. W.

Shell. $43600 \times \frac{3.0}{5.5} = 23800 \times 1.5 = 35700$

topfill $= 21300 \times 2.4 = 51100$

Sand fill $58800 \times \frac{1.8}{4.3} = 24600 \times .9 = 22100$

$W = 69700$ kg $\times 1.56^m = 108900$

frictional couple $71200 \times \frac{2}{6} = 23700$ kgm.

Taking moment at O.

Loads	Hor. forces	vert. forces,	Lev. arm	moment.
D		126000	0	0
D'	12600		9.1	114500
S		62300	0	0
S'	6200		6.23	38600
W		69700	0	0
W'	6970		1.56	10900
R	−19600		1.06	−20800
	6170 kg	258000 kg		= 143200 kgm
		frictional couple, less		−23700
				119500 kgm

Eccentricity $E = \frac{119500}{258000} = .463^m$

Moment of inertia of the section

$0.0491(3.8^4 - 3.2^4) = 5.08\ \mathrm{m^4}$ area $= 3.8^\phi - 3.2^\phi = 3.30\ \mathrm{m^2}$

extreme fibre stress $= \frac{258000}{3.30} \pm \frac{119500 \times 1.9}{5.08} = 78200 \pm 44700$

$= 122900$ kg/m² or 12.3 kg/cm² c ok

or 33500 " or 3.4 " c

No vertical reinforcement required theoretically.
Use 13mmφ bars at 50 cm c to c about in practice.
For horizontal reinforcement use the same as above.

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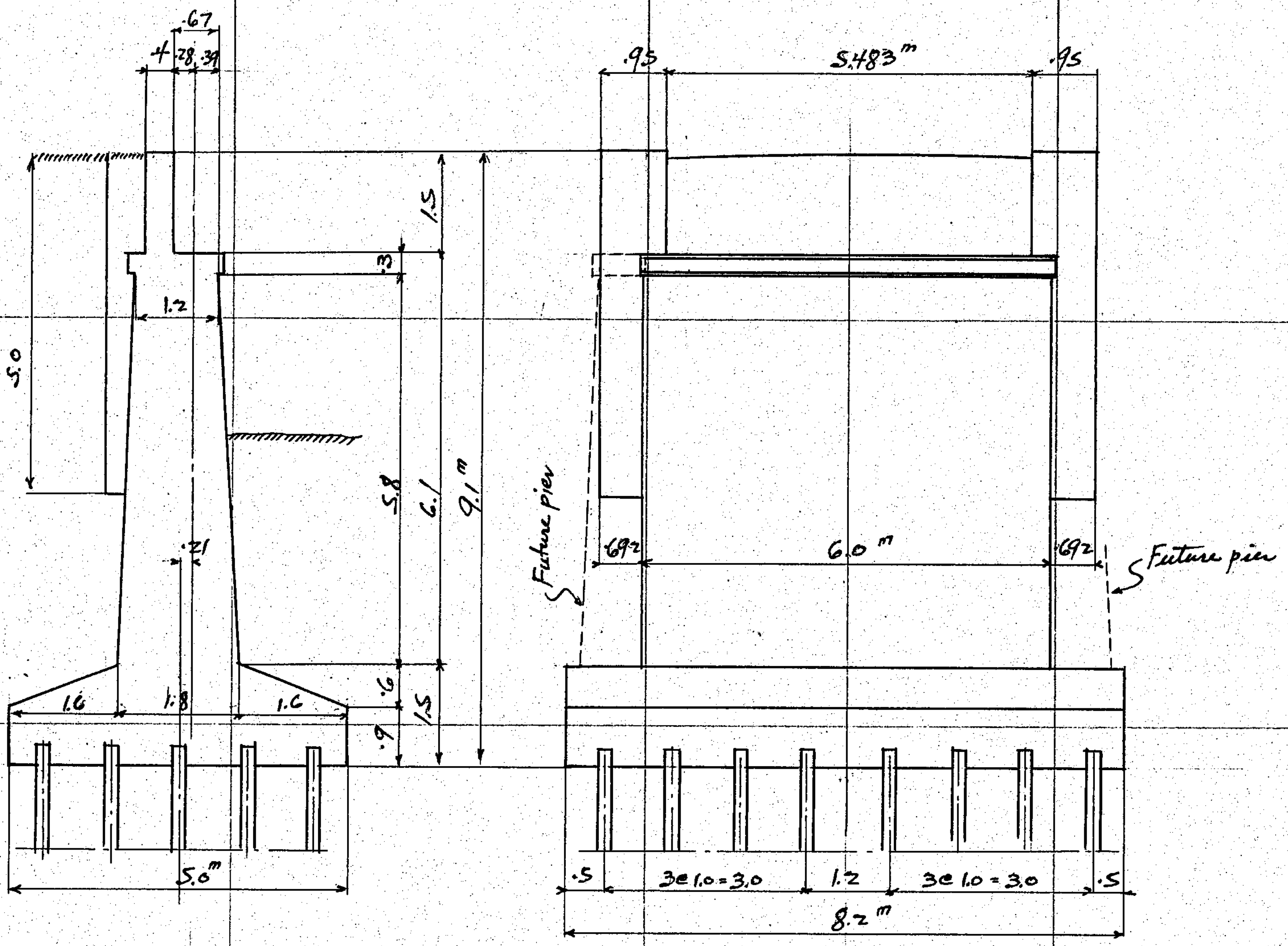
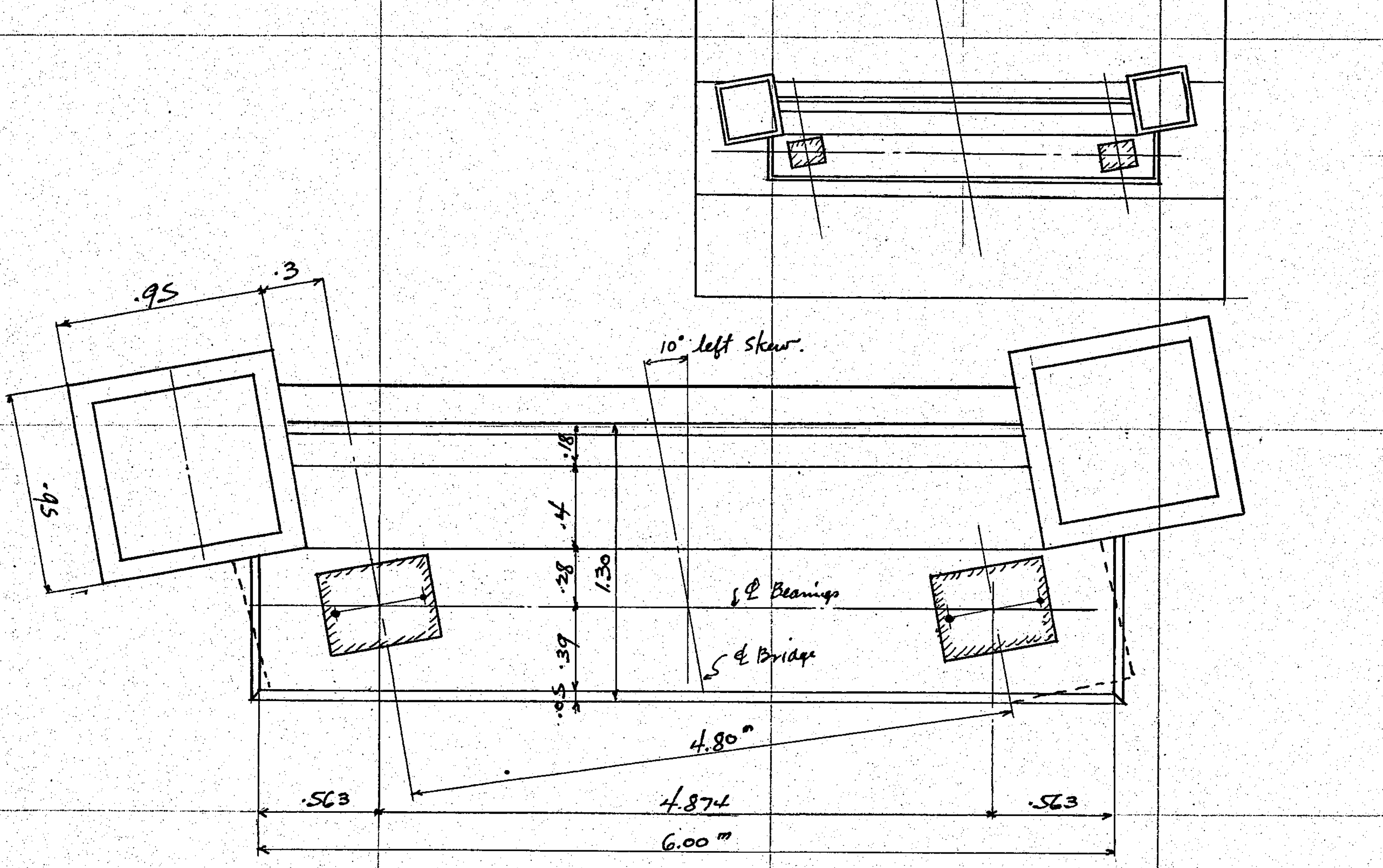
CALCULATIONS FOR

Eian-Bashi for Okayama-Ken.
Abutment for Girder span.



Sketch for Abutment
Scale 1:100



Scale 1:30.

備考 本橋臺ハ將來ニ於ケル河川改修工事ノ結果橋梁ノ延長ヲ要スルコトアルベキヲ豫想シ此場合必要ニ應ジテ
橋脚ニ改造シ得ル樣設計セルモノナリ。

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CHECKED BY M.J. DATE 4-5-26 PAGE NO 39

CALCULATIONS FOR

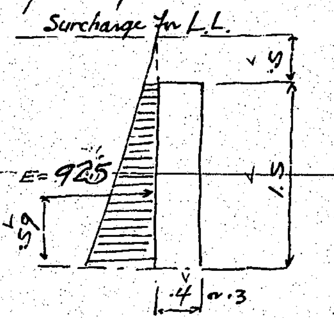
Eian-Bashi for Okayama Ken.

Volume of concrete, weight, and center of gravity of abutment.

Volume of concrete Shaft.

					vert lev. arm.		hor lev. arm. ℄ shoe	
parapet wall.	.4×1.5×5.483 =	3.29	@ 2400 =	7900 ×	8.35 =	66000	.48	3800
coping	.3×1.3×6.1 =	2.38	@ " =	5710 ×	7.45 =	42600	.21	1,200
post under pedestals	.95×.95×5.0×2 =	9.02	@ " =	21,650 ×	6.60 =	142,900	.75	16,300
less "	.24×.7×3.5 = −	.59	@ " = −	1420 ×	5.90 = −	8,400	.5	− 700
" "	.40×.7×3.5 = −	.98	@ " = −	2350 ×	5.90 = −	13,800	.65	− 1,500
Shaft	1.5×6.0×5.8 =	52.20	@ " =	125,300 ×	2.73 =	342,000	.21	26300
Total concrete for Shaft =		65.32 cub.m.		156,800 kg ×	3.65 m (y) =	571,300	.29 (x)	45400
Base concrete.	3.4×.6×8.2 =	16.73	@ 2400 =	40,150 ×	1.15 =	46,200		
"	5.0×.9×8.2 =	36.90	@ " =	88,550 ×	.45 =	39,800		
		53.63 cub.m.		128,700 kg ×	.67 m =	86,000		
Total concrete =		118.95						

Parapet wall.



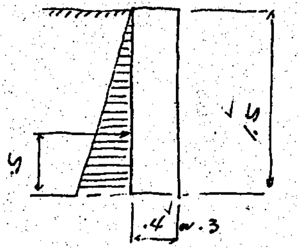
Earth pressure at normal state

$0.5 \times 1600 \div 3 = 267$

$2.0 \times 1600 \div 3 = 1067$

$1234 \div 2 = 617 \times 1.5 = 925$ kg per lin meter of wall.

moment on wall = $925 \times .59 = 546$ kgm per lin meter.



Earth pressure during Earthquake $k = 0.1$ assumed.

$$E = 0.37\frac{wh^2}{2} = 0.37 \times \frac{1600 \times 1.5^2}{2} = 667 \text{ kg per lin meter of wall.} < 925$$

Normal state governs.

Effective depth reqd $= \sqrt{\frac{546 \times 100}{100 \times 7.18}} = 8.4$ cm

Use 37 cm effective depth with 3 cm insulation.

Steel area reqd. $= \frac{546 \times 100}{1200 \times \frac{7}{8} \times 37} = 1.41$ □cm per lin meter

Use 13 mmϕ bars at 40 cm c to c $= 3.32$ cm²

Unit shear $= \frac{925}{100 \times \frac{7}{8} \times 37} = .29$ kg/cm² ok

Unit bond $= \frac{925}{2.5 \times 4.08 \times \frac{7}{8} \times 37} = 2.80$ kg/cm² ok.

If the thickness of wall be 30 cm, effective depth 27 cm.

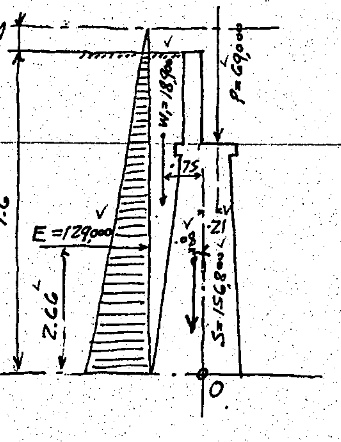
steel area reqd $= \frac{546 \times 100}{1200 \times \frac{7}{8} \times 27} = 1.93$ cm²

Use 13 mmϕ bars at 30 cm c to c $= 4.42$ cm²

unit bond $= \frac{925}{3.33 \times 4.08 \times \frac{7}{8} \times 27} = 2.88$ kg/cm² ok.

Shaft.

Stability of shaft at normal state. Case 1.



L.L. + D.L. = 69,000 kg for the whole abutment = P.

wt. of shaft = 156,800 " " = S

Earth fill on rear = $.42 \times 7.6 \times 7.4 \div 2 \times 1600 = 18900$ = W_1

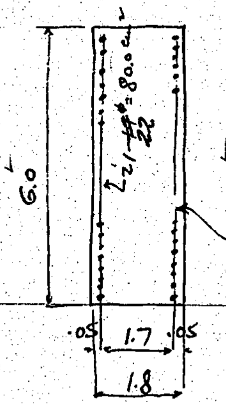
Earth pressure $\frac{1600}{3} \times .5 = 267$

$\frac{1600}{3} \times 8.1 = 4320$

$4587 \div 2 = 2294 \times 7.4 \times 7.6 = 129,000$ kg

備考　橋台前面土砂ハ将来行ハルベキ改修ノ結果低水位附近マデ切取ラルベキヲ豫想シ其土圧ヲ無視セリ.

Eian-Bashi for Okayama Ken.



Taking moment about point O. center of bottom.

Loads		~~Ho~~ Lever arm		moment
P	69,000	× +.21	=	14,500
S	156,800	× −.08	=	− 12,550
W_1	18,900	× −.75	=	−14,150
E	129,000	× +2.66	=	343,000
	ΣV = 244700	× ~~1.40~~ 1.35 m	=	~~343350~~ 330,800 Kgm
	ΣH = 129,000			

Try reinforcement 21-~~19~~ 22ϕ bars on bothsides = $80 \times 2 = 160.0$ cm²

Steel ratio $p_0 = 2p = \frac{160}{600 \times 180} = .0015$

$z/h = 135/180 = .75$, $d'/h = 5/180 = .028$

$K = .23$, $L = .072$

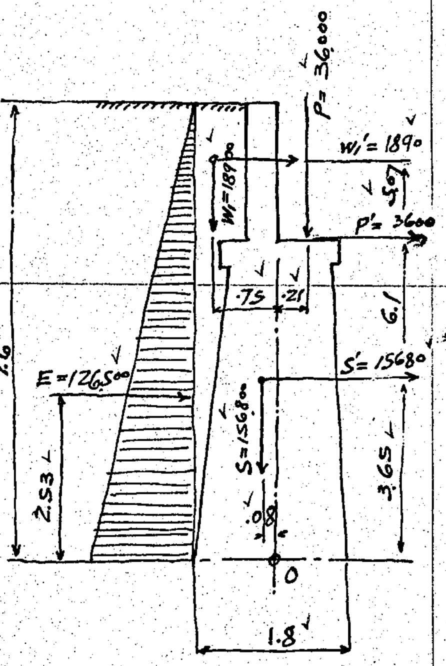
$f_c = \frac{330800 \times 100}{.072 \times 600 \times 180^2} = 23.6$ kg/cm² ok

$f_s = 15 \times 23.6 \times \left(\frac{175}{.23 \times 180} - 1\right) = 1144$ kg/cm² ok

Unit Shear $= \frac{129000}{600 \times \frac{7}{8} \times 175} = 1.4$ kg/cm² ok

Unit bond $= \frac{129000}{691 \times 21 \times \frac{7}{8} \times 175} = 5.8$ kg/cm² ok

Case 2 Stability of Shaft during Earthquake $k = 0.1$ assumed.



Earth pressure during Earthquake.

$E = 0.37 \times \frac{1600 \times 7.6^2}{2} \times 7.4 = 126,500$ kg

Taking moment about pt. O.

Loads	Hor. forces	Vert. forces	Lev. arm		moments.
P		36,000	× + .21	=	+ 7,560
P'	3,600		× + 6.10	=	+ 21,950
S	~~15,680~~	156,800	× − .08	=	− 12,550
S'	15,680		× + 3.65	=	+ 57,250
W_1		18,900	× − .75	=	− 14,180
W_1'	1,890		× + 5.07	=	+ 9,580
E	126,500		× + 2.53	=	+ 320,000
	ΣH = 147,670	ΣV = 211,700	× 1.84	=	+ 389,610

$z/h = 1.84/1.8 = 1.02$, $d'/h = .028$, $p_0 = .0015$

$K = .205$, $L = .069$

$f_c = \frac{389610 \times 100}{.069 \times 600 \times 180^2} = 29$ kg/cm² ok

$f_s = 15 \times 29 \left(\frac{175}{.205 \times 180} - 1\right) = 1,630$ kg/cm² $< 1200 \times 1.8$ ok.

Unit Shear $= \frac{147670}{600 \times \frac{7}{8} \times 175} = 1.6$ kg/cm² ok

Unit bond $= \frac{147670}{6.91 \times 21 \times \frac{7}{8} \times 175} = 6.65$ kg/cm² $< 6.0 \times 1.8$ ok.

Assumption for the bottom section ok.

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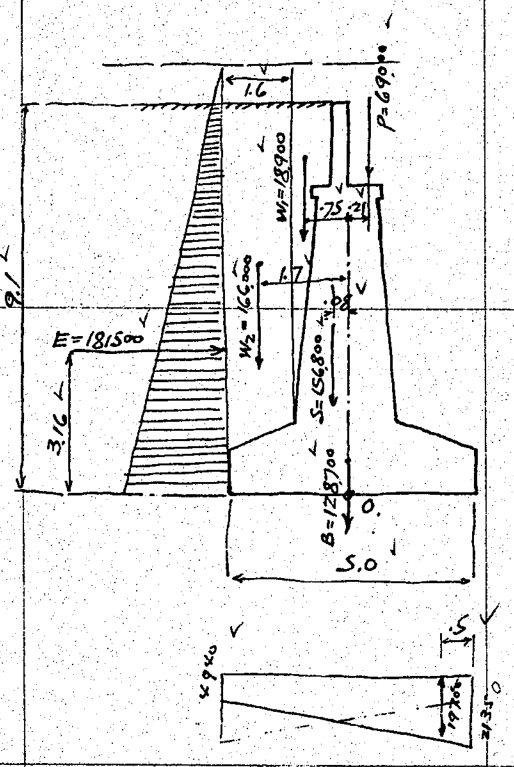
CALCULATIONS FOR

Eiau-Bashi for Okayama Ken.

Stability of Abutment as a whole.

Case 1. Stability at normal state.



Earth pressure = $\frac{1600 \times .5}{3} = 267$

$\frac{1600 \times 9.6}{3} = 5120$

$8387 \div 2 = 2694 \times 9.1 \times 7.4 = 181,500$ kg $= E$

Earth fill on rear $1.6 \times 7.9 \times 8.2 \times 1600 = 166,000$ kg $= W_2$.

moment about center of base O.

Loads	Hor. forces	Vert. forces		Lev. arms		Moments
P		69,000	×	.21	=	14,500
S		156,800	×	-.08	=	-12,550
B		128,700	×	0	=	~~14,150~~ 0
W₁		18,900	×	-.75	=	-14,150
W₂		166,000	×	-1.70	=	-282,000
E	181500		×	3.16	=	574,000
	181,500 kg	539,400 kg	×	.52 m	=	279,800 kgm

Resultant force within middle third.

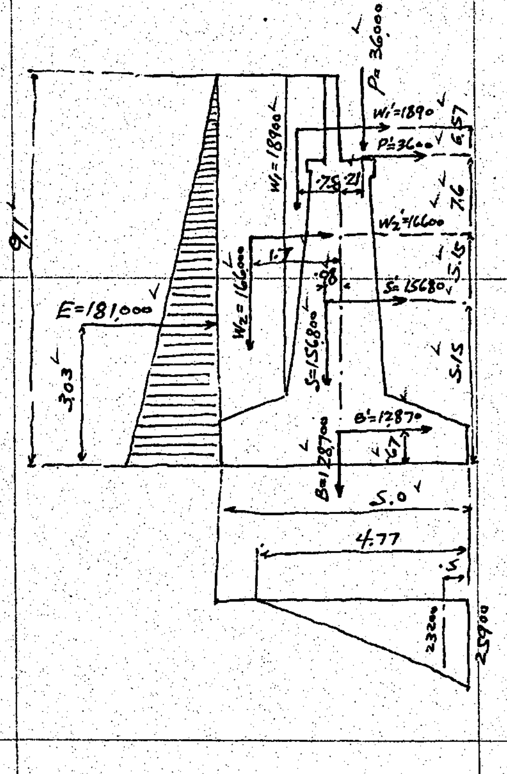
max. toe pressure = $\frac{539,400}{5.0 \times 8.2}\left(1 \pm \frac{6 \times .52}{5.0}\right) = 21,350$ kg/m² or (1.95 tons/□') ok

or 4,940 " or (.45 ") "

Load on one pile 19,700 × 1 × 1 = 19700 or 19.7 kg tons. on one pile.

Case 2. Stability during Earthquake.

Earth pressure = $\frac{0.37 \times 1600 \times 9.1^2}{2} \times \frac{7.4}{8.2} =$ ~~245,000~~ 181,000 kg



Loads	Hor. forces	Vert. forces		Lev. arms		moments
P		36,000	×	+ .21	=	+ 7,560
P'	3,600		×	+ 7.60	=	+ 27,350
S		156,800	×	- .08	=	- 12,550
S'	15,680		×	+ 5.15	=	+ 80,800
B		128,700	×	0	=	0
B'	12,870		×	+ .67	=	+ 8,620
W₁		18,900	×	- .75	=	- 14,180
W₁'	1,890		×	+ 6.57	=	+ 12,400
W₂		166,000	×	- 1.70	=	-282,000
W₂'	16,600		×	+ 5.15	=	+ 85,500
E	181,000		×	+ 3.03	=	+ 548,000
	ΣH= 231,640 kg	ΣV= 506,400 kg	×	.91 m	=	461,530 kgm

Resultant force out of middle third. pressure area = $3(2.5-.91) = 4.77^{m} \times 8.2 = 39.1$ □m

max. toe pressure = $\frac{506,400 \times 2}{39.1} = 25900$ kg/m² or (2.37 tons/□') ok

Load on pile = 23200 × 1 × 1 = 23.2 kg tons on one pile

Case 3. Stability during earthquake. Seismic force backward.

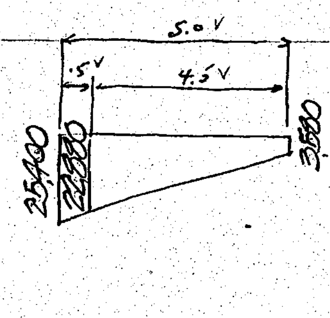
Earth pres. front side

$\frac{1600 \times 5^2}{6} \times 7.4 = 49,300$ kg

Earth on base

$1.6 \times 3.8 \times 8.2 \times 1600 = 79,600$ kg.

	Hor. forces	vert. forces		Lev. arm.		moment.
	32,150 kg	506,400 kg	×	.825		417,910
Earth press. front.	49,300		×	1.67	=	82,400
Earth on base "		79,600	×	-1.70	=	-135,500
	81,450 kg	586,000 kg	×	.62	=	364,810 kgm



max. toe pressure = $\frac{586,000}{5.0 \times 8.2}\left(1 \pm \frac{6 \times .62}{5.0}\right) = 25,400$ kg/m² or (2.28 tons/□') ok. 78

or 3,580 " (.33 ")

If Earth on front side be neglected

max. toe pressure = $\frac{506,400}{5 \times 8.2}\left(1 \pm \frac{6 \times .825}{5.0}\right) = 24,600$ kg/m² or (2.25 tons/□') ok.

or 120 " or (.01 ")

Load on one pile = 22880 × 1 × 1 = 22.9 kg tons on one pile.

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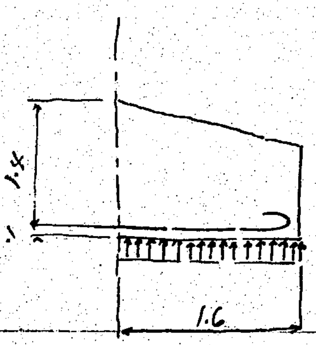
CALCULATIONS FOR

Eian-Bashi for Okayama Ken.

Cantilever footing at toe.



Upward pressure, normal state 21,350 kg/m²
downward - 1.2×2400 = − 2,880
" earth. say 1.0×1600 = − 1,600
16,870 kg/m²

moment $= \frac{16870 \times 1.6^2}{2} = 21,600$ kgm per meter strip.

Shear $= 16870 \times 1.6 = 27000$ kg

Steel area req'd $= \frac{21600 \times 100}{1200 \times \frac{7}{8} \times 140} = 14.7$ cm²/m

Use ~~22~~ 19φ bars at ~~30~~ 15 cm ctoc = ~~16.38~~ 18.9 cm²

Unit Shear $= \frac{27000}{100 \times .941 \times 140} = 2.05$ kg/cm² ok

Unit bond $= \frac{27000}{5.97 \times 6.7 \times .941 \times 140} = 5.12$ kg/cm² ok

$p = \frac{18.9}{100 \times 140} = .00135$

$j = .941$

For footing at heel use same detail as above.

D

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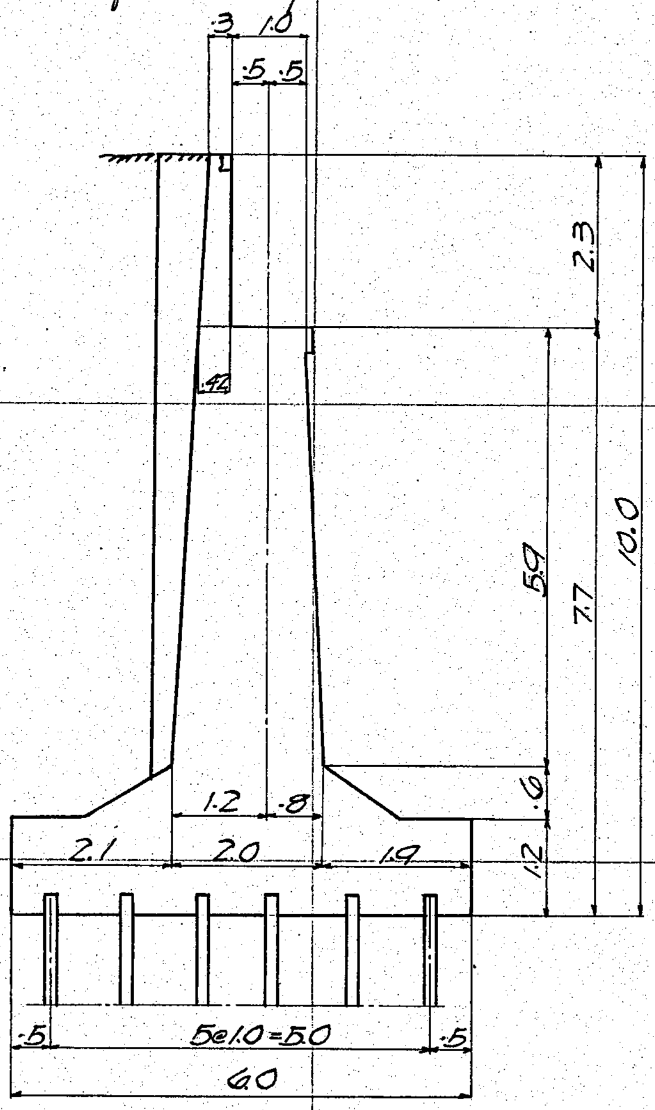
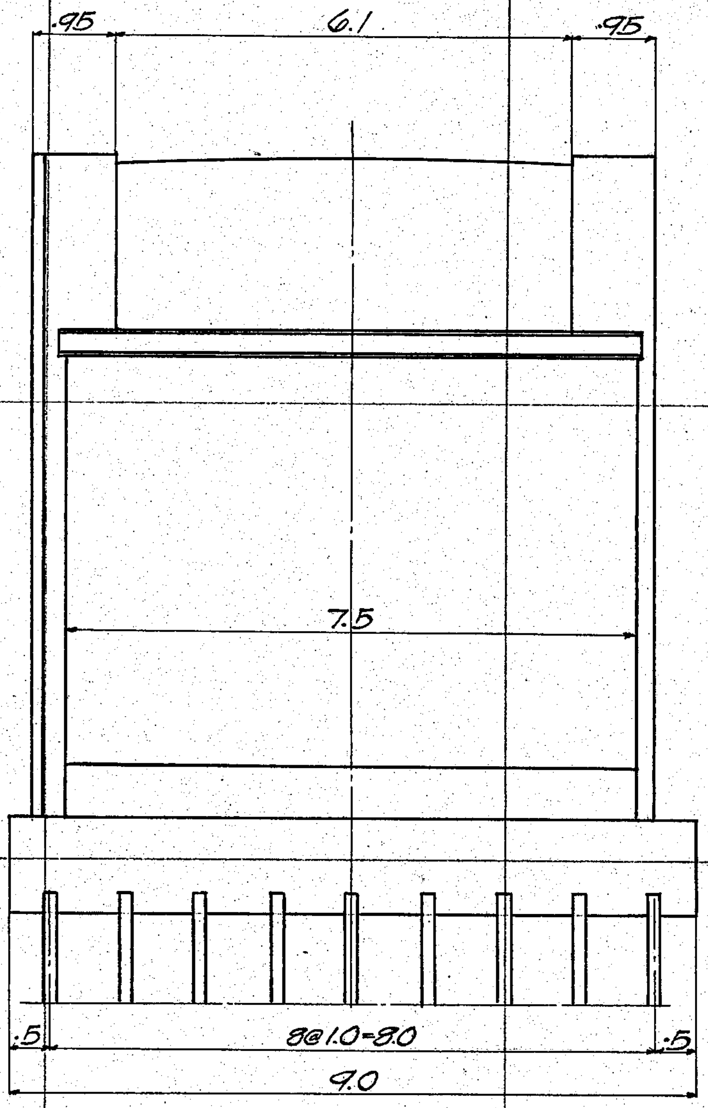
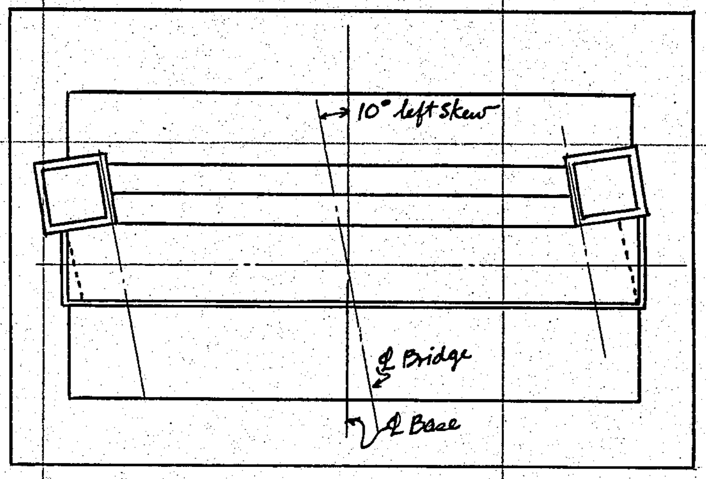
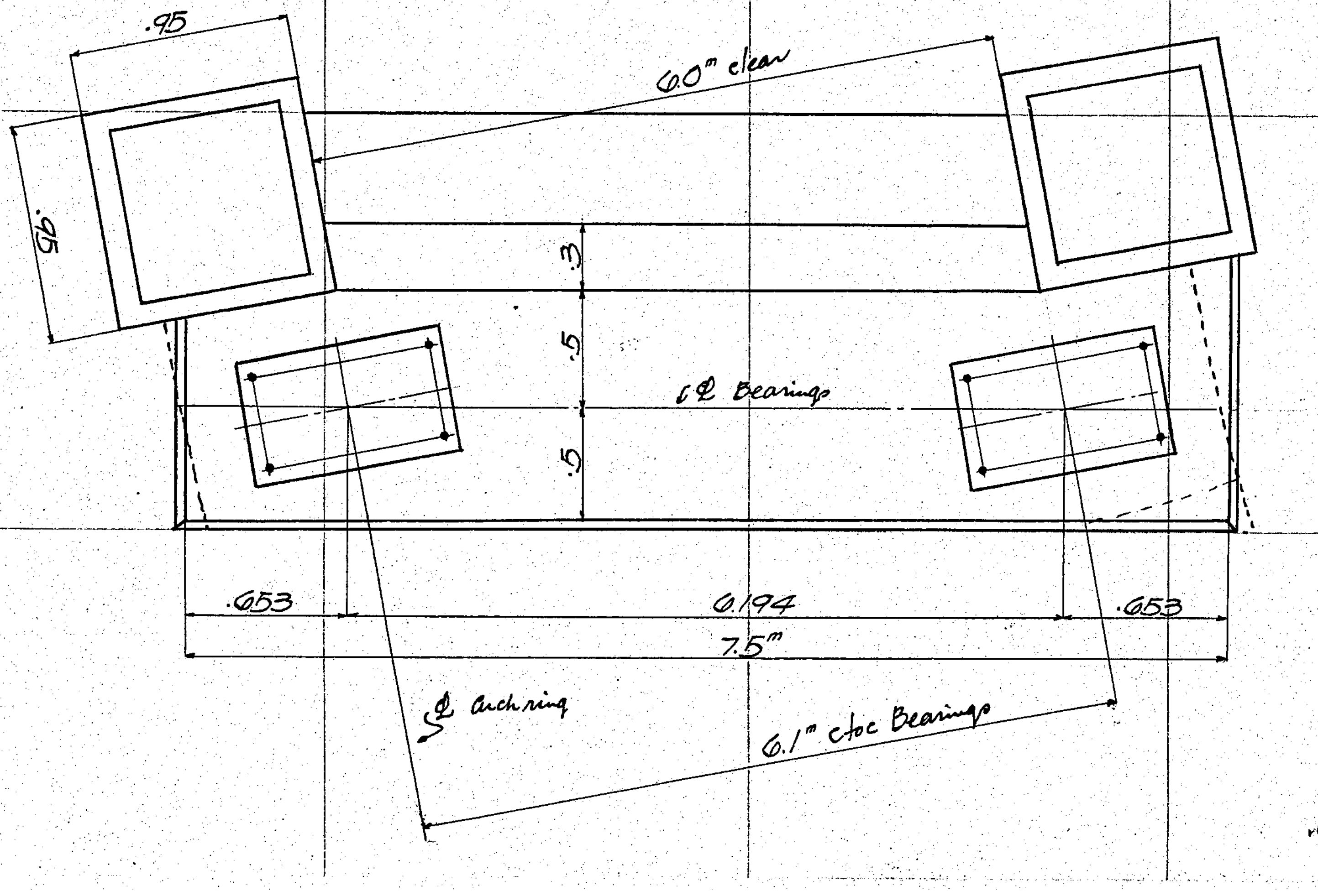
CALCULATIONS FOR

Eian Bashi for Okayama ken
Abutment for Arch span. A2.









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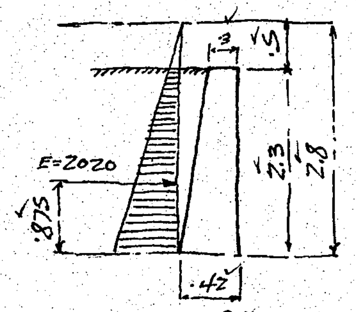
CALCULATIONS FOR

Eian Bashi for Okayama Ken

Parapet wall.



Earth pressure at normal state.

$.5 \times 1600 \times \frac{1}{3} = 267$

$2.8 \times 1600 \times \frac{1}{3} = \underline{1493}$

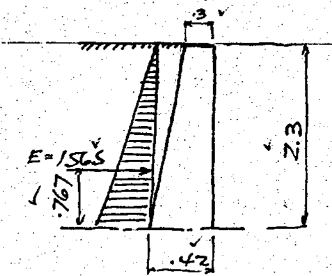
$1760 \div 2 = 880 \times 2.3 = 2020$ kg/m of wall.

moment $= 2020 \times .875 = 1770$ kgm per meter strip of wall.

Earth pressure during Earthquake. $K = 0.1$

$\frac{0.37wh^2}{2} = \frac{0.37 \times 1600 \times 2.3^2}{2} = 1565$ kg/meter of wall. < 2020

Normal state governs.



Effective depth required $= \sqrt{\frac{1770 \times 100}{100 \times 7.18}} = 15.7$ cm

Use 42 cm total depth with 3 cm insulation, effective depth = 39 cm.

Steel area required $= \frac{1770 \times 100}{1200 \times \frac{7}{8} \times 39} = 4.32$ cm² per meter strip

Use 13φ bars at 30 cm c to c = 4.42 cm²

Unit Shear $= \frac{2020}{100 \times \frac{7}{8} \times 39} = 0.6$ kg/cm² ok

Unit bond $= \frac{2020}{4.08 \times 3.33 \times \frac{7}{8} \times 39} = 4.36$ kg/cm² ok.

Shaft.

Volume of Concrete, weight and center of gravity of abutment.

				Vert. lev. arm		Hor. lev. arm, toe of Shaft.	
Concrete for Shaft.							
parapet wall.	.36 × 2.3 × 6.1 = 5.05	@ 2400 =	12,100	7.00	84,700	1.43	17,300
coping	.05 × .3 × 9.6 = .14	@ " =	300	5.75	1,700	.35	100
Shaft.	1.71 × 5.9 × 7.5 = 75.60	@ " =	181,500	2.78	504,500	1.00	181,500
post under pedestals	.95 × .95 × 2.35 × 2 = 4.24	@ " =	10,200	7.07	72,100	1.80	18,400
"	.95 × .55 × 6 × 2 = 6.27	@ " =	15,100	3.16	47,700	2.00	30,200
Total concrete for Shaft =	91.30 m³	@	219,200 kg	÷ 3.24 m =	710,700	. 1.13 m =	247,500
						from toe of base	
Concrete for Base.	3.0 × .6 × 9.0 = 16.20	@ 2400 =	38,900	1.47	57,200	2.9	112,800
	1.2 × 6.0 × 9.0 = 64.80	@ " =	155,500	.60	93,300	3.0	466,500
Total concrete for Base =	81.00 m³		194,400 kg	· .77 =	150,500	2.98	579,300
Total concrete for abutment =	172.3 m³					Call this 3.0	

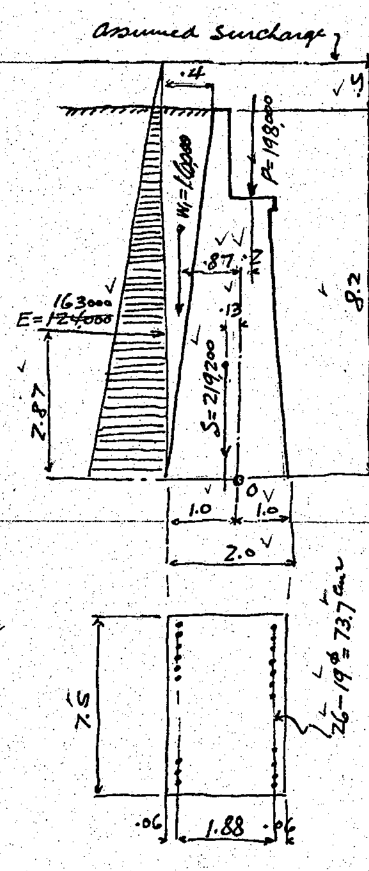
Stability of Shaft

Case 1. Stability at Normal State.

Superimposed Loads on abutment. D.L. = 126,000 kg

L.L. = 72,000

P = 198,000 kg for one abutment.



Weight of earthfill on rear. $\frac{.4}{2} \times 8.2 \times 6.1$ @ 1600 = 16,000 kg = W₁

Earth pressure $1600 \times 8.7 \times \frac{1}{3} = 4695$

$1600 \times 0.5 \times \frac{1}{3} = \underline{267}$

$4962 \div 2 = 2481 \times \overset{8.0}{\cancel{6.1}} \times 8.2 = \overset{163000}{\cancel{124,000}}$ kg = E

Taking moment about center of bottom of shaft.

Loads	Hor. forces	Vert. forces		Lever arms		moments
P		198,000	×	0.20	=	39,600
S		219,200	×	− 0.13	=	− 28,500
W₁		16,000	×	− 0.87	=	− 13,900
E	163,000 ~~124,000~~		×	2.87	=	467,800
	163,000 kg	433,200 kg	÷	1.07 m	=	465,000 kgm

$e/h = 1.07/2.0 = 0.535$ $d'/h = .06/2.0 = .03$

Try 26-19φ bars on both sides $= 73.7 \times 2 = 147.4$ cm²

$p_0 = \frac{147.4}{200 \times 750} = .001$

$K = .29$, $L = .070$

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CALCULATIONS FOR

Eian Bashi for Okayama Ken.

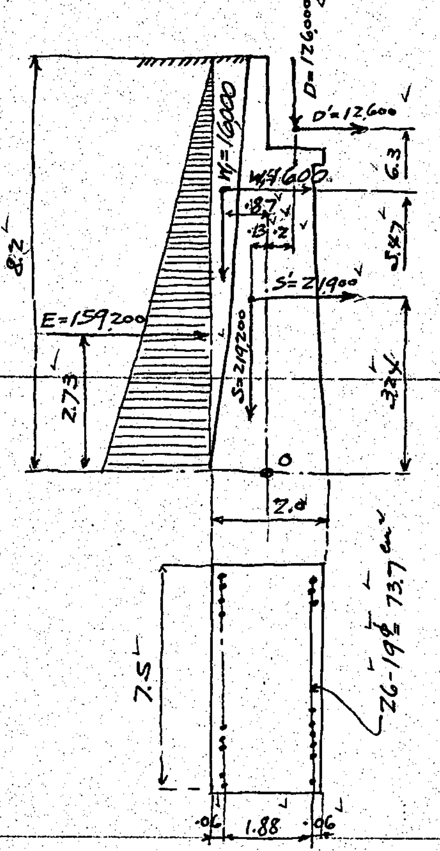
$$f_c = \frac{465000 \times 100}{0.070 \times 750 \times 200^2} = 222 \text{ kg/cm}^2 \quad ok$$

$$f_s = 15 \times 222 \left(\frac{194}{.29 \times 200} - 1\right) = 780 \quad ok$$

$$\text{Unit Shear} = \frac{163000}{750 \times .94 \times 194} = 1.2 \quad ok \quad , \quad j = .94$$

$$\text{Unit bond} = \frac{163000}{597 \times 26 \times .94 \times 194} = 5.8 \quad ok$$

Case 2 Stability during Earthquake, $k = 0.1$ assumed.



Earth pressure during earthquake. $= \frac{0.37 \times 1600 \times 8.2^2}{2} = 19900 \times 8.0 = 159{,}200$ kg

Taking moments about center of bottom area.

Loads	Hor. forces	Vert. forces	Lev. arm	Moments
D		126,000	.20	25200
D'	12600	?	6.30	79400
S		219200	− .13	− 28500
S'	21900		3.24	71,000
W_1		16,000	− .87	− 13900
W_1'	1600		5.47	8800
E	159200		2.73	435000
	195300 kg	361200 kg	1.60 m	577000 kgm

$e/h = 1.60/2.0 = 0.80$, $d'/h = .03$, $p_0 = .0010$

$k = .20$ $L = .061$

$$f_c = \frac{577000 \times 100}{.061 \times 750 \times 200^2} = 315 \text{ kg/cm}^2 \quad ok$$

$$f_s = 15 \times 315 \left(\frac{194}{20 \times 200} - 1\right) = 1820 \text{ kg/cm}^2 < 1200 \times 1.8 \quad ok$$

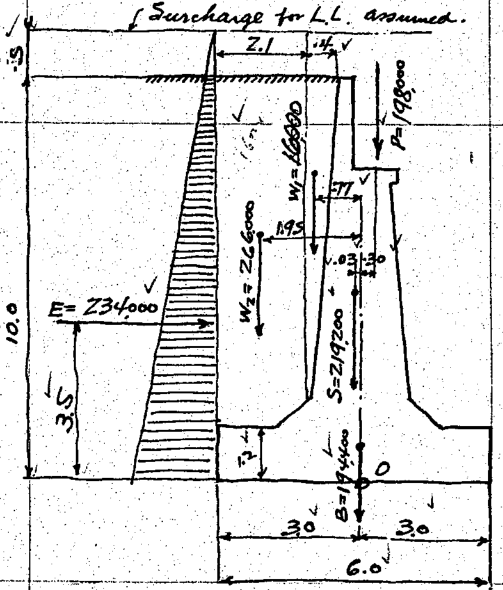
$$\text{Unit shear} = \frac{195300}{750 \times .94 \times 194} = 1.44 \quad ok$$

$$\text{Unit bond} = \frac{195300}{597 \times 26 \times .94 \times 194} = 6.96 \quad ok < 6.0 \times 1.8$$

Assumed Section is ample.

Stability of Abutment as a whole.

Case 1. Stability at Normal State.



weight of earth fill on rear $2.1 \times 8.8 \times 9.0 @ 1600 = 266{,}000$ kg $= W_2$.

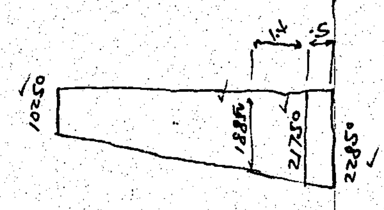
Earth pressure $= \frac{1600 \times 0.333 \times 10.5^2}{2} = 29{,}350$

$\frac{1600 \times 0.333 \times 0.5^2}{2} = -70$

$29{,}280 \times 8.0 = 234{,}000$ kg

Taking moment about center of base O.

Loads	Hor. forces	Vert. forces	Lev. arms	moments
P		198,000	.30	59300
S		219200	− .03	− 6600
B		194400	0	0
W1		16000	− .77	− 12,300
W2		266000	− 1.95	− 518,500
E	234000		3.50	819,000
	234000 kg	893,600 kg	0.38	340,900 kgm



Resultant force within middle third.

$$\text{max. toe pressure} = \frac{893600}{9.0 \times 6.0}\left(1 \pm \frac{6 \times .38}{6.0}\right) = 22850 \text{ kg/m}^2 \text{ or } (2.09 \text{ tons/□}) \text{ ok}$$

or 10,250 " or (0.94 ") "

Max. Load on one pile $= 21750 \text{ kg} \times 1.0 \times 1.0 = 21.75$ kg tons.

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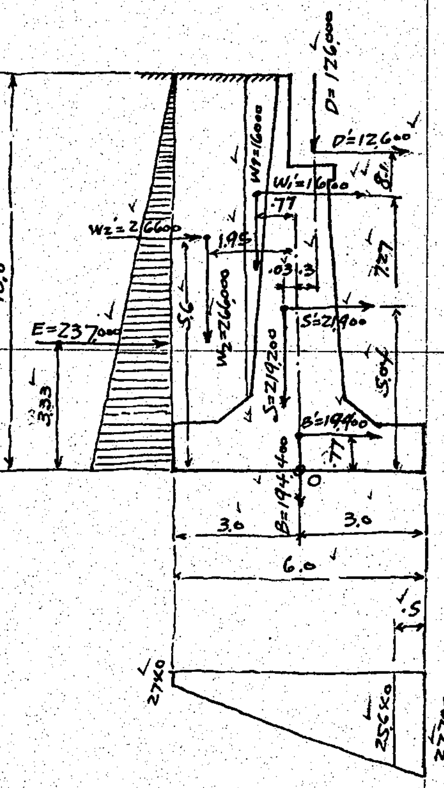
CALCULATIONS FOR

Eian-Bashi for Okayama Ken.

Case 2. Stability during Earthquake. k=0.1 assumed.

Earth pressure $= \frac{0.37 \times 1600 \times 10^2}{2} = 29600 \times 8.0 = 237000$ kg



Taking moment about center of base O.

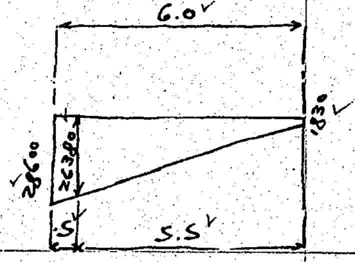
Loads	Hor. Forces	Vert. forces		Lev. arm		Moments
D		126,000	×	·30	=	37,800
D'	12,600		×	8.10	=	102,000
S		219,200	×	− ·03	=	− 6600
S'	21,900		×	5.04	=	110,400
B		194,400	×	0	=	0
B'	19,400		×	·77	=	14,900
W1		16,000	×	− ·77	=	− 12,300
W1'	1,600		×	7.27	=	11,600
W2		266,000	×	− 1.95	=	− 518,500
W2'	26,600		×	5.60	=	149,000
E	237,000		×	3.33	=	789,000
	319,100 kg	821,600 kg	×	·82 m	=	677,300 kgm.

Resultant force within middle third.

max. toe pressure $= \frac{821,600}{9.0 \times 6.0} \times \left(1 \pm \frac{6 \times .82}{6.0}\right) = 27,700$ kg/m² or (2.53 tons/□') ok
or 2740 " " (·25 ")

Max. Load on one pile = 25,640 kg × 1.0 × 1.0 = 25.64 kg tons

Case 3. Stability during Earthquake, Seismic forces backward.



Modifying above table of moment.

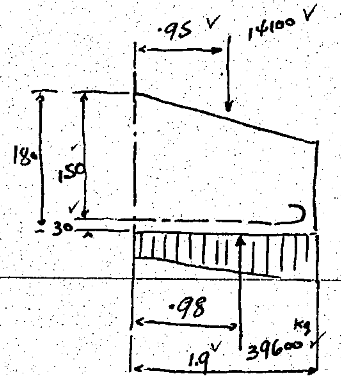
Hor. forces	Vert. forces		Lev. arm		moment
53,900 kg	821,600 kg	×	·88 m	=	726,900

Resultant force within middle third.

max. toe pressure $= \frac{821,600}{9.0 \times 6.0}\left(1 \pm \frac{6 \times .88}{6.0}\right) = 28,600$ kg/m² or (2.62 tons/□') ok
or 1830 " " (·17 ") .

Max. Load on one pile = 26,380 kg × 1.0 × 1.0 = 26.38 kg tons

Cantilever footing at toe.



Upward pressure at toe, normal state 22,850 kg/m² at toe.
18850 " at face of shaft.
41700 ÷ 2 = 20850 average.

total upward force = 20850 kg / meter strip × 1.9 = 39600
wt. of footing say 1.3 × 2400 × 1.9 = 5900 " } 14100 kg
" " earth on base 2.7 × 1600 × 1.9 = 8,200 "

Moment · 39600 × .98 = 38,800
14100 × .95 = −13,400
25,400 kgm per lin meter of base

Shear = 39600 − 14100 = 25500 kg

Effective depth req'd $= \sqrt{\frac{25400 \times 100}{100 \times 7.18}} = 59.5$ cm

Use 150 cm effective depth with 30 cm pile space or total 180 cm.

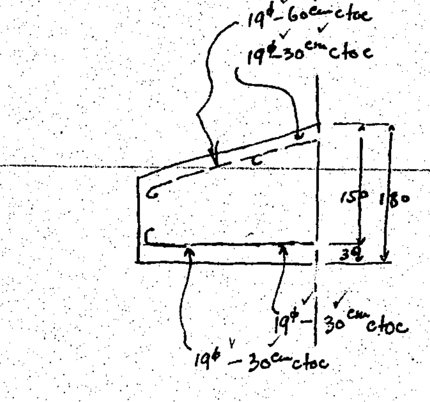
Steel area req'd. for moment $= \frac{25400 \times 100}{1200 \times \frac{7}{8} \times 150} = 16.1$ cm² per meter strip

Use 19φ bars at 15 cm c to c = 18.9 cm²

Unit Shear $= \frac{25500}{100 \times \frac{7}{8} \times 150} = 1.95$ kg/cm² ok

Unit bond $= \frac{25500}{5.97 \times 6.7 \times \frac{7}{8} \times 150} = 4.9$ kg/cm² ok

Cantilever footing at heel.



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Final Design of Gian Bashi for Okayama ken

Panel Dead Load: -

Floor asphalt block pavement 3.8 cm @ 21 kg = 79.8
mortar cushion 1.5 cm @ 22 " = 33.0
Concrete slab 13 " @ 24 " = 312.0
424.8 × 2.7 × 5.0 = ~~5460~~ 5730 kg

Coping .3 × .15 = .045
.03 × .27 = .008
.141 × .24 = .034
.087 × 5.0 = .435 m³ @ 2400 = 1,045 kg

Fillet .038 × .2 × 5.0 = .038 @ 2400 = 90

6865 kg

Handrail 5.0 m @ 60.0 kg = 300 "

Stringers 10,160 ÷ 20 = 508 "
Floor beam, intermediate 736 ÷ 2 = 368 "

Total panel load for floor = 8041 kg

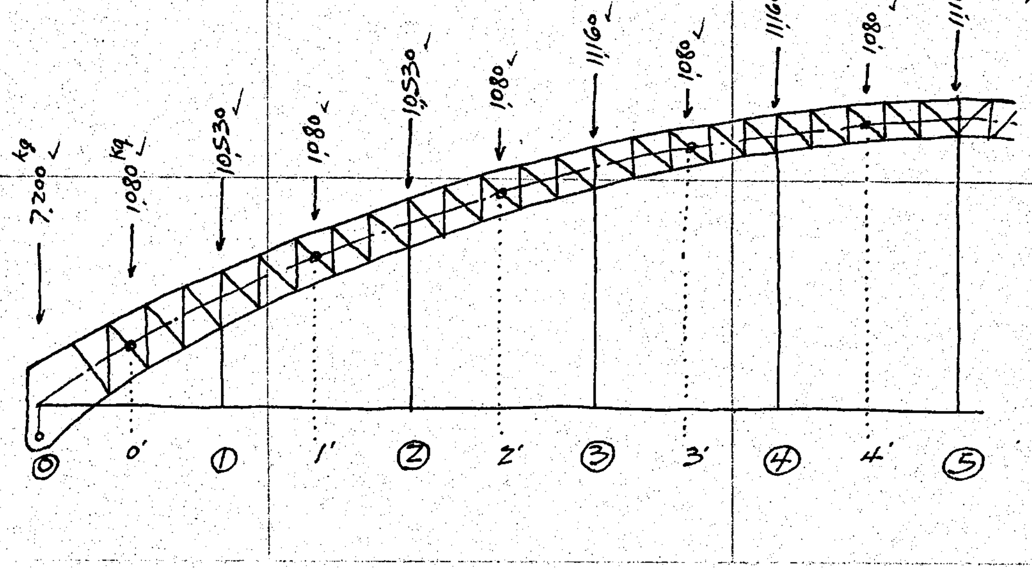
Lateral Bracings, Portal + Sway Bracings.
Bottom Lateral bracings 4,705 ÷ 20 = 235 kg for all intermediate panels.
Top Lateral bracings 2,365 ÷ 8 = 296 kg for panel pt. 4 + 5 only
" 2,365 ÷ 16 = 148 " " " " 3 "
Portal Bracing 962 ÷ 2 = 481 " " " " 3 "
Sway Bracings 668 ÷ 2 = 334 " " " " 4 + 5 "

Main Trusses.
Arch Rib 41,126 ÷ 40 = 1028 for panel pt. 1, 1', 2, 2' ...... 4, 4' + 5
Ties 13,064
Hangers 7,913 } = 20977 ÷ 20 = 1049 " " 1, 2, 3, 4 + 5
62,103 ÷ 20 = 3105 kg for all panels.

Rivet heads. 4,553 ÷ 20 = 228 " " " " Say 176 for 1, 2, 3, 4 + 5 + 52 for 0', 1', 2', 3' + 4'

Summary for Panel Dead Loads for several panel points.
Panel Loads due to.

Panel pts.	Floor	Bottom Lateral Br.	Top Lateral Br.	Portal + Sway Br.	Arch Ring	Tie + Hanger	Rivet heads.	Total	Call this
0	4,700	117			1250 shoe 537	347	250	7,201	7,200 kg
0'					1,028		52	1,080	1,080
1	8,041	235			1,028	1,049	176	10,529	10,530
1'					1,028		52	1,080	1,080
2	8,041	235			1,028	1,049	176	10,529	10,530
2'					1,028		52	1,080	1,080
3	8,041	235	148	481	1,028	1,049	176	11,158	11160
3'					1,028		52	1,080	1,080
4	8,041	235	296	334	1,028	1,049	176	11,159	11160
4'					1,028		52	1,080	1,080
5	8,041	235	296	334	1,028	1,049	176	11,159	11,160



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

MADE BY K. Inaba DATE 4-6-10 **FILE NO**

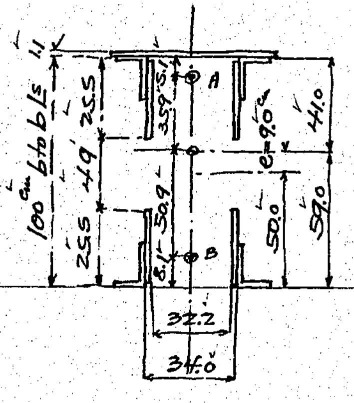
CHECKED BY M. Jinda DATE 4-6-15 **PAGE NO** 48

CALCULATIONS FOR

Final Design of Eian-Bashi for Okayama Prefecture.

Cross section and moment of inertia of Arch Ring at Several Panel Points.

Panel point 5 or Crown.



Top flange 1 Cov. Pl. 560×11 = 61.60 × -.65 = - 33.9
2Ls 150×100×12 = 57.12 × 4.88 = 278.7
2Pls 250×9 = 45.00 × 13.00 = 585.0
163.72cm² × 5.1cm = 829.8

Bottom flange 2Ls 150×100×15 = 70.50 × 95.01 = 6695.0
2Pls. 250×9 = 45.00 × 87.00 = 3915.0
115.50cm² × 91.9cm = 10,610.0
279.22cm² × 41.0cm = 11,439.8

Moment of inertia I_5.

moment of inertia of Top flange about its center of gravity Ⓐ

Top flange 1 cov. pl. $6 + 61.6 \times 5.7^2$ = 2010
" 2 Ls $640 \times 2 + 57.12 \times 0.22^2$ = 1280
" 2 Pl^s $1172 \times 2 + 45.0 \times 7.9^2$ = 5150
8440 —— ◎

163.72×35.9^2 = 211,060
219,500

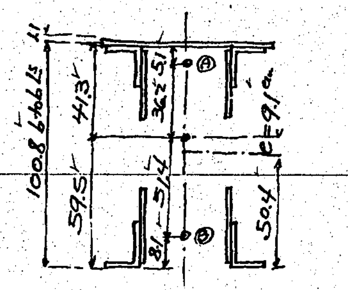
moment of inertia of Bottom flange about its center of gravity

Bottom flange 2Ls $785 \times 2 + 70.5 \times 3.11^2$ = 2,250
2Pls $1172 \times 2 + 45.0 \times 4.9^2$ = 3420
5,670 —— ◎

115.5×50.9^2 = 298,030
303700

I_5 = 523,200 cm⁴

Panel Point 4'



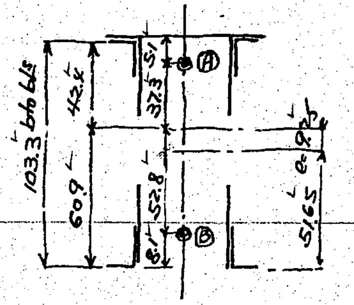
Sectional area same as for Panel Pt. 5. Depth 100.8 b to b Ls.

neutral axis by proportion

distance Ⓐ-Ⓑ at 5 $35.9 + 50.9 = 86.8$ cm $35.9 \times \frac{87.6}{86.8} = 36.2$
" " at 4' $100.8 - 13.2 = 87.6$ $50.9 \times$ " = 51.4

I of Top flange $8440 + 163.72 \times 36.2^2$ = 223,000
" " Bottom " $5670 + 115.5 \times 51.4^2$ = 310700
I_4' = 533,700 cm⁴

Panel point 4.

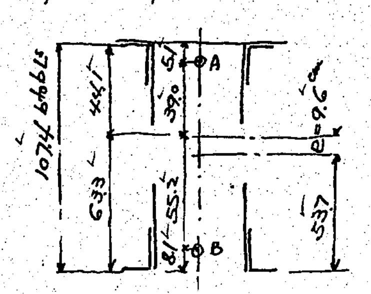


Sectional area same as for Panel pt. 5. Depth = 103.3 cm b to b Ls.

distance ⒶⒷ at 5 86.8cm $35.9 \times \frac{90.1}{86.8} = 37.3$ cm
" " at 4 $103.3 - 13.2 = 90.1$ $50.9 \times$ " = 52.8

I for Top flange $8440 + 163.72 \times 37.3^2$ = 236,300
" " Bottom " $5670 + 115.50 \times 52.8^2$ = 327,600
I_4 = 563,900 cm⁴

Panel Point 3'

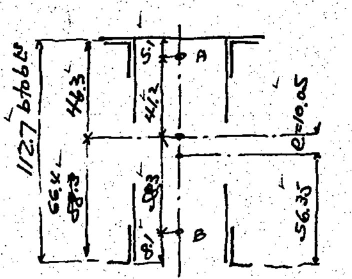


Sectional area same as for Panel point 5. Depth = 107.4 cm b to b Ls.

Distance Ⓐ-Ⓑ at 5 86.8cm $35.9 \times \frac{94.2}{86.8} = \overset{39.0}{\cancel{38.95}}$
" " " 3' $107.4 - 13.2 = 94.2$ $50.9 \times$ " = 55.2

I for Top flange. $8440 + 163.72 \times 39.0^2$ = 257,400
" " Bottom " $5670 + 115.5 \times 55.2^2$ = 357,500
I_3' = 614,900 cm⁴

Panel Point 3.



Sectional area same as for crown. Depth 112.7cm b to b Ls.

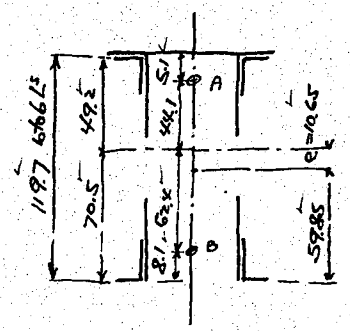
Distance A-B. at 5 86.8cm $35.9 \times \frac{99.5}{86.8} = 41.2$
" " at 3 $112.7 - 13.2 = 99.5$ $50.9 \times$ " = 58.3

I for Top flange $8440 + 163.72 \times 41.2^2$ = 286,200
" " Bottom " $5670 + 115.5 \times 58.3^2$ = 398,200
I_3 = 684,400 cm⁴

CALCULATIONS FOR

Trial Design of Eian Bashi for Okayama prefecture.

Panel point Z'



Sectional area same as for Crown. Depth 119.7 cm b to b of Ls.

Distance A-B at Z' $119.7 - 13.2 = 106.5$ $35.9 \times \frac{106.5}{86.8} = 44.1$ cm

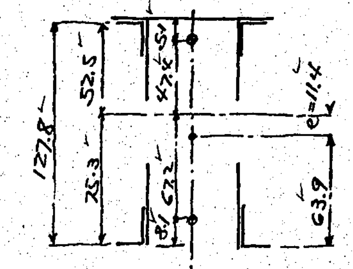
" " " S $= 86.8$ $50.9 \times$ " $= 62.4$

I for top flange $8440 + 163.72 \times 44.1^2 = 326,700$

I " bottom " $5670 + 115.5 \times 62.4^2 = 455,200$

$I_{Z'} = 781,900$ cm⁴

Panel point Z



Sectional area same as for crown. Depth 127.8 cm b to b of Ls.

Distance A-B = $127.8 - 13.2 = 114.6$ $35.9 \times \frac{114.6}{86.8} = 47.4$

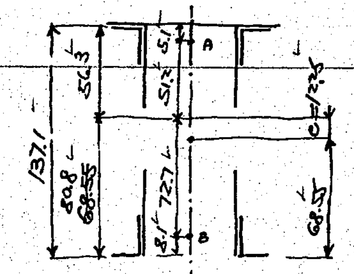
$50.9 \times$ " $= 67.2$

I for top flange $8440 + 163.72 \times 47.4^2 = 376,100$

I " bottom " $5670 + 115.5 \times 67.2^2 = 527,000$

$I_Z = 903,100$ cm⁴

Panel point 1'



Sectional area same as for crown. Depth 137.1 cm b to b Ls.

Distance A-B = $137.1 - 13.2 = 123.9$ $35.9 \times \frac{123.9}{86.8} = 51.2$

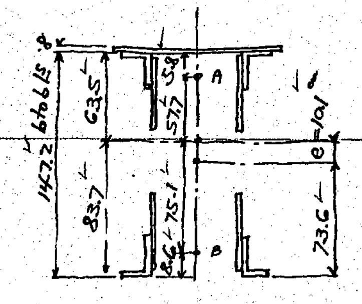
$50.9 \times$ " $= 72.7$

I for top flange $8440 + 163.72 \times 51.2^2 = 437,500$

I " bottom " $5670 + 115.5 \times 72.7^2 = 616,000$

$I_{1'} = 1,053,500$ cm⁴

Panel Point 1



A-B = 57.7 + 75.1 = 132.8 cm

Section Depth 147.2 cm b to b of Ls.

Top flg.	1 cov. pl.	$560 \times 8 =$	$44.80 \times$	$-.4 =$	$-18.$
	2 Ls.	$150 \times 100 \times 9 =$	$43.38 \times$	$4.77 =$	207
	2 Pls.	$250 \times 9 =$	$45.00 \times$	$13.00 =$	585
			$133.18 \times$	$5.8 =$	774
Bott. flg.	2 Ls	$150 \times 100 \times 12 =$	$57.12 \times$	$142.32 =$	8135
	2 Pls	$250 \times 9 =$	$45.00 \times$	$134.2 =$	6035
			$102.12 \times$	$13\overset{8.6}{8.8} =$	14170
			235.30 cm $\times$	63.5 cm $=$	14944

moment of inertia of Top flange about its center of gravity A.

1 cov. pl. $2 + 44.8 \times 6.2^2 = 1725$

2 Ls $486 \times 2 + 43.38 \times 1.03^2 = 1020$

2 Pls. $1172 \times 2 + 45.0 \times 7.2^2 = 4675$

7420

$133.18 \times 57.7^2 = 443280$

$450,700$ cm⁴

moment of inertia of bottom flange about its center of gravity B.

2 Ls $640 \times 2 + 57.12 \times 3.72^2 = 2070$

2 Pls. $1172 \times 2 + 45.0 \times 4.4^2 = 3220$

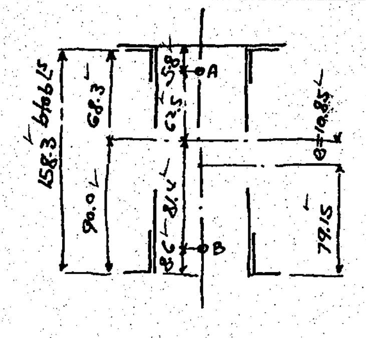
5290

$102.12 \times 75.1^2 = 576010$

581300

$I_1 = 1,032,000$ cm⁴

Panel Point 0'



Sectional area same as for Panel point 1. Depth = 158.3 cm b to b Ls.

Distance A-B panel pt. 1 132.8 cm $57.7 \times \frac{143.9}{132.8} = 62.5$

" " " 0' $158.3 - 14.4 = 143.9$ $75.1 \times$ " $= 81.4$

I for top flange $7420 + 133.18 \times 62.5^2 = 527400$

I " bottom flange. $5290 + 102.12 \times 81.4^2 = 681,000$

$I_{0'} = 1208400$ cm⁴

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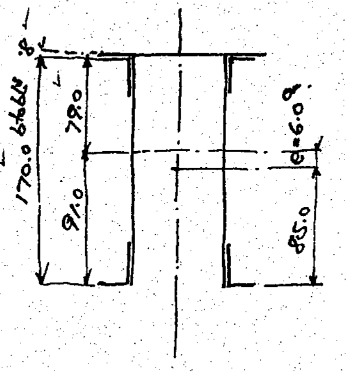
CALCULATIONS FOR

Final Design of Gion Bashi for Okayama prefecture.

Panel Point 0.



Section Depth 170 cm b to b of Ls.

1 cov. pl.	560 × 8 =	44.8 ×	-.4 =	-18
2 Ls	150×100×9 =	43.38 ×	4.77 =	207
2 Ls	150×100×12 =	57.12 ×	165.12 =	9430
2 Pls	169 × 9	= 304.2 ×	85.0 =	25850
		449.5 ×	79.0 =	35469

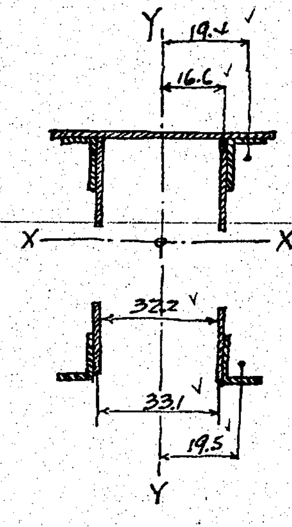
moment of inertia

1 cov. pl.	2. + 44.8 × 79.4²	=	282,400
2 Ls	486×2 + 43.38 × 74.23²	=	240,100
2 Ls	640×2 + 57.12 × 86.12²	=	424,800
2 Pls.	362,000×2 + 304.2 × 6.0²	=	735,000

$I_0 = 1,682,300 \text{ cm}^4$

Radius of Gyrations at several sections.

Crown section. or panel pt. 5



x-x axis. $r_x = \sqrt{\frac{523200}{279.22}} = 43.3 \text{ cm}$

y-y axis.

1 cov. pl.	560 × 11	16,100	=	16,100
2 Ls	150×100×12	229×2 + 57.12 × 19.4²	=	21,960
2 Ls	150×100×9 15	279×2 + 70.5 × 19.5²	=	27,360
4 Pls.		90.0 × 16.6²	=	24,800
				90,220 cm⁴

$r_y = \sqrt{\frac{90220}{279.22}} = 18.0 \text{ cm}$

Same values of r_y for panel pts. 5 ~ 1'.

Panel points 1 & 0' y-y axis.

1 cov. pl.	560 × 8		~~16,100~~ 11,700
2 Ls	150×100×9	175×2 + 43.38 × 19.3² =	16,500
2 Ls	150×100×12	229×2 + 57.12 × 19.4² =	21,960
4 Pls	250 × 9	90.0 × 16.6² =	24,800
			~~79,360~~ cm⁴
			74,960

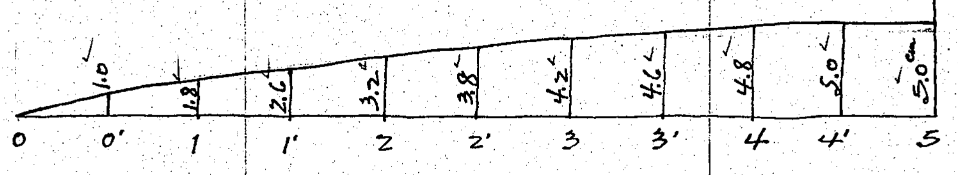
$r_y = \sqrt{\frac{74960}{235.3}} = 17.9 \text{ cm}$

Panel pt.	Sectional area A.	Moment of inertia I_x	Dist. of extreme fibre. Top.	Bottom	Eccentricity E.	do vertical.	Radius of gyration r_x	r_y	l for r_y.	l/r	Allowable comp.
5	279.22 cm²	523,200 cm⁴	42.1 cm	59.0 cm	9.0 cm	9.0 cm	43.3 cm	18.0 cm	5.0 m	27.8	1000 kg/cm²
4'	〃	533,700	42.4	59.5	9.1	9.1	43.7	〃			
4	〃	563,900	43.5	60.9	9.26	9.3	44.9	〃	5.01	27.8	〃
3'	〃	614,900	45.2	63.3	9.6	9.8	47.0	〃	5.106	28.4	〃
3	〃	684,400	47.4	66.4	10.05	10.4	49.5	〃			
2'	〃	781,900	50.3	70.5	10.65	11.2	52.9	〃	assume 3–1 10.828	60.2	〃
2	〃	903,100	53.6	75.3	11.4	12.3	56.9	〃			
1'	〃	1,053,500	57.4	80.8	12.26	13.6	61.4	〃			
1	235.30	1,032,000	64.3	83.7	10.1	11.5	63.8	17.9			
0'	〃	1,208,400	69.1	90.0	10.85	12.7	71.7	〃			
0	449.50	1,682,300	79.8	91.0	6.0	7.2	61.2	17.3			

Camber of floor.

Floor will be cambered to the amount nearly equal to the deflection of the arch ring, by shortening the hangers.

Chamber assumed. 1/1000 of span length = 50. ÷ 1000 = .05 m or 5 cm. Parabolic curbe.



Final Design of Eian Bashi for Okayama prefecture

Calculation of x, y, and dc. for neutral axis of the arch ring.

Panel point	x	y for ℄ Rib.	Eccentricity	Camber	y for N.A. of Rib.	dc.	φ.	Sin φ.	cos φ.	sec φ.
0 ✓	0.0 m ✓	0.000 m ✓	+.072 ✓	−0.000 ✓	0.072 ✓	0.000	34°-20' ✓	.5640 ✓	.8258 ✓	1.2110 ✓
0' ✓	2.5 ✓	1.615 ✓	+.127 ✓	−.010 ✓	1.732 ✓	3.001	31°-40' ✓	.5250 ✓	.8511 ✓	1.1749 ✓
1 ✓	5.0 ✓	3.060 ✓	+.115 ✓	−.018 ✓	3.157 ✓	2.878 ✓	28°-29' ✓	.4769 ✓	.8790 ✓	1.1377 ✓
1' ✓	7.5 ✓	4.335 ✓	+.136 ✓	−.026 ✓	4.445 ✓	2.812 ✓	25°-24' ✓	.4289 ✓	.9033 ✓	1.1070 ✓
2 ✓	10.0 ✓	5.440 ✓	+.123 ✓	−.032 ✓	5.531 ✓	2.726 ✓	21°-50' ✓	.3719 ✓	.9283 ✓	1.0773 ✓
2' ✓	12.5 ✓	6.375 ✓	+.112 ✓	−.038 ✓	6.449 ✓	2.663 ✓	18°-29' ✓	.3170 ✓	.9484 ✓	1.0544 ✓
3 ✓	15.0 ✓	7.140 ✓	+.104 ✓	−.042 ✓	7.202 ✓	2.611 ✓	14°-43' ✓	.2540 ✓	.9672 ✓	1.0339 ✓
3' ✓	17.5 ✓	7.735 ✓	+.098 ✓	−.046 ✓	7.787 ✓	2.568 ✓	11°-21' ✓	.1968 ✓	.9804 ✓	1.0199 ✓
4 ✓	20.0 ✓	8.160 ✓	+.093 ✓	−.048 ✓	8.205 ✓	2.535 ✓	7°-37' ✓	.1325 ✓	.9912 ✓	1.0089 ✓
4' ✓	22.5 ✓	8.415 ✓	+.091 ✓	−.050 ✓	8.456 ✓	2.513 ✓	3°-50' ✓	.0669 ✓	.9978 ✓	1.0022 ✓
5 ✓	25.0 ✓	8.500 ✓	+.090 ✓	−.050 ✓	8.540 ✓	2.501	0°-00' ✓	.0000 ✓	1.0000 ✓	1.0000 ✓

Horizontal Thrust H. See Hirois' Statically Indeterminate Stresses on page 107. Equation (131).

$$H=\frac{1}{2}\cdot\frac{\int_0^{a'}\frac{xydc}{I}+a\int_{a'}^{\frac{l'}{2}}\frac{ydc}{I}-\int_0^{a}\frac{\sin\phi dx}{A}}{\int_0^{\frac{l'}{2}}\frac{y^2dc}{I}+\int_0^{\frac{l}{2}}\frac{\cos\phi dx}{A}+\frac{l}{2A_t}}W.$$

panel pt. no	x (m)	y (m)	dc (m)	I (cm⁴)	$\frac{xydc}{I}$	a (m)	$\frac{ydc}{I}$	Sin φ	dx (m)	A □cm	$\frac{\sin\phi dx}{A}$	
0 ✓	0.0 ✓	0.072 ✓	0.000 ✓	1,682,300 ✓	.00000 ✓	0.0 ✓	.0000 ✓	.5640 ✓	0.0 ✓	449.50 ✓	.0000 ✓	
0' ✓	2.5 ✓	1.732 ✓	3.001 ✓	1,208,400 ✓	10.7533 ✓	2.5 ✓	.0430 ✓	.5250 ✓	2.5 ✓	235.30 ✓	.5578 ✓	
1 ✓	5.0 ✓	3.157 ✓	2.878 ✓	1,032,000 ✓	44.0206 ✓	5.0 ✓	.0880 ✓	.4769 ✓	〃 ✓	〃 ✓	.5067 ✓	
1' ✓	7.5 ✓	4.445 ✓	2.812 ✓	1,053,500 ✓	88.9844 ✓	7.5 ✓	.1186 ✓	.4289 ✓	〃 ✓	279.22 ✓	.3840 ✓	
2 ✓	10.0 ✓	5.531 ✓	2.726 ✓	903,100 ✓	166.9528 ✓	10.0 ✓	.1670 ✓	.3719 ✓	〃 ✓	〃 ✓	.3330 ✓	
2' ✓	12.5 ✓	6.449 ✓	2.663 ✓	781,900 ✓	274.5506 ✓	12.5 ✓	.2196 ✓	.3170 ✓	〃 ✓	〃 ✓	.2838 ✓	
3 ✓	15.0 ✓	7.202 ✓	2.611 ✓	684,400 ✓	412.1367 ✓	15.0 ✓	.2748 ✓	.2540 ✓	〃 ✓	〃 ✓	.2274 ✓	
3' ✓	17.5 ✓	7.787 ✓	2.568 ✓	614,900 ✓	569.1133 ✓	17.5 ✓	.3252 ✓	.1968 ✓	〃 ✓	〃 ✓	.1762 ✓	
4 ✓	20.0 ✓	8.205 ✓	2.535 ✓	563,900 ✓	737.7079 ✓	20.0 ✓	.3689 ✓	.1325 ✓	〃 ✓	〃 ✓	.1186 ✓	
4' ✓	22.5 ✓	8.456 ✓	2.513 ✓	533,700 ✓	895.8654 ✓	22.5 ✓	.3982 ✓	.0669 ✓	〃 ✓	〃 ✓	.0599 ✓	
5 ✓	25.0 ✓	8.540 ✓	2.501 ✓	523,200 ✓	1,020.5724 ✓	25.0 ✓	.4082 ✓	.0000 ✓	〃 ✓	〃 ✓	.0000 ✓	
Summary			$\frac{l'}{2}$ = 26.808 m ✓		$\Sigma_0^{a'}\frac{xydc}{I}$		$a\Sigma_{a'}^{\frac{l'}{2}}\frac{ydc}{I}$				$\Sigma_0^{a}\frac{\sin\phi dx}{A}$	Numerators of H
0 ✓					0.0000 ✓		0.000 ✓				.0000 ✓	0.0000 ✓
0' ✓					10.7533 ✓		592.125 ✓				.5578 ✓	602.3205 ✓
1 ✓					54.7739 ✓		1,140.250 ✓				1.0645 ✓	1,193.9594 ✓
1' ✓					143.7583 ✓		1,621.425 ✓				1.4485 ✓	1,763.7348 ✓
2 ✓					310.7111 ✓		1,994.900 ✓				1.7815 ✓	2,303.8296 ✓
2' ✓					585.2617 ✓		2,219.125 ✓				2.0653 ✓	2,802.3214 ✓
3 ✓					997.3984 ✓		2,250.750 ✓				2.2927 ✓	3,245.8557 ✓
3' ✓					1,566.5117 ✓		2,056.775 ✓				2.4689 ✓	3,620.8178 ✓
4 ✓					2,304.2196 ✓		1,612.800 ✓				2.5875 ✓	3,914.4321 ✓
4' ✓					3,200.0850 ✓		918.450 ✓				2.6474 ✓	4,115.8876 ✓
5 ✓					4,220.6574 ✓		0.000 ✓				2.6474 ✓	4,218.0100 ✓

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CHECKED BY M. Jinda DATE 4-6-15 **PAGE NO** 52

CALCULATIONS FOR

Final Design of Eian Bashi for Okayama prefecture.

Section of Tie

2Is 300×90×38.13 = 97.15
2Pls 240×9 = 43.20
140.35 cm² = At

Span length $l = 5000$ cm

$$\frac{l}{2A_t} = \frac{5000}{2\times140.35} = 17.8126$$

Panel Pt. no.	y (m)	dc (m)	I (cm⁴)	$\frac{y^2 dc}{I}$	cos φ	dx (m)	A (cm²)	$\frac{\cos\phi\, dx}{A}$	$\frac{l}{2At.}$
0	.072	0.000	1,682,300	.0000	.8258	0.0	449.50	.0000	17.8126
0'	1.732	.3001	1,208,400	7.4476	.8511	2.5	235.30	.9043	〃
1	3.157	2.878	1,032,000	27.7816	.8790	〃	〃	.9339	〃
1'	4.445	2.812	1,053,500	52.7177	.9033	〃	279.22	.8088	〃
2	5.531	2.726	903,100	92.3677	.9283	〃	〃	.8312	〃
2'	6.449	2.663	781,900	141.6200	.9484	〃	〃	.8492	〃
3	7.202	2.611	684,400	197.9110	.9672	〃	〃	.8660	〃
3'	7.787	2.568	614,900	253.2332	.9804	〃	〃	.8778	〃
4	8.205	2.535	563,900	302.6828	.9912	〃	〃	.8875	〃
4'	8.456	2.513	533,700	336.7179	.9978	〃	〃	.8934	〃
5	8.540	2.501	523,200	348.6028	1.0000	〃	〃	.8954	〃
Summary			$\Sigma_0^{\frac{l}{2}} \frac{y^2 dc}{I}$	= 1761.082			$\Sigma_0^{\frac{l}{2}} \frac{\cos\phi dx}{A}$ =	8.7475	$\frac{l}{2At}$ = 17.8126

Denominator = 1761.082 + 8.7475 + 17.8126 = 1787.6421

H-Surface

Panel Point	Numerator		2× denominator		H for unit load		
0	0.0000	÷	3575.2842	= H_0 =	0.0000		0.0000
0'	602.3205	÷	〃	= H_0' =	0.1685		
1	1,193.9594	÷	〃	= H_1 =	0.3339		0.3339
1'	1,763.7348	÷	〃	= H_1' =	0.4933		
2	2,303.8296	÷	〃	= H_2 =	0.6444		0.6444
2'	2,802.3214	÷	〃	= H_2' =	0.7838		
3	3,245.8557	÷	〃	= H_3 =	0.9079		0.9079
3'	3,620.8178	÷	〃	= H_3' =	1.0127		
4	3,914.4321	÷	〃	= H_4 =	1.0949		1.0949
4'	4,115.8876	÷	〃	= H_4' =	1.1512		
5	4,218.0100	÷	〃	= H_5 =	1.1798	×½ =	0.5899

for total span = 3.5710 × 2 = 7.1420

Comparison of H for preliminary to Final values.

$$\frac{\text{Final } H}{\text{Preliminary } H} = \frac{7.1420}{7.1274} = 1.00205$$

Error of preliminary H = -0.21 % of Final H.

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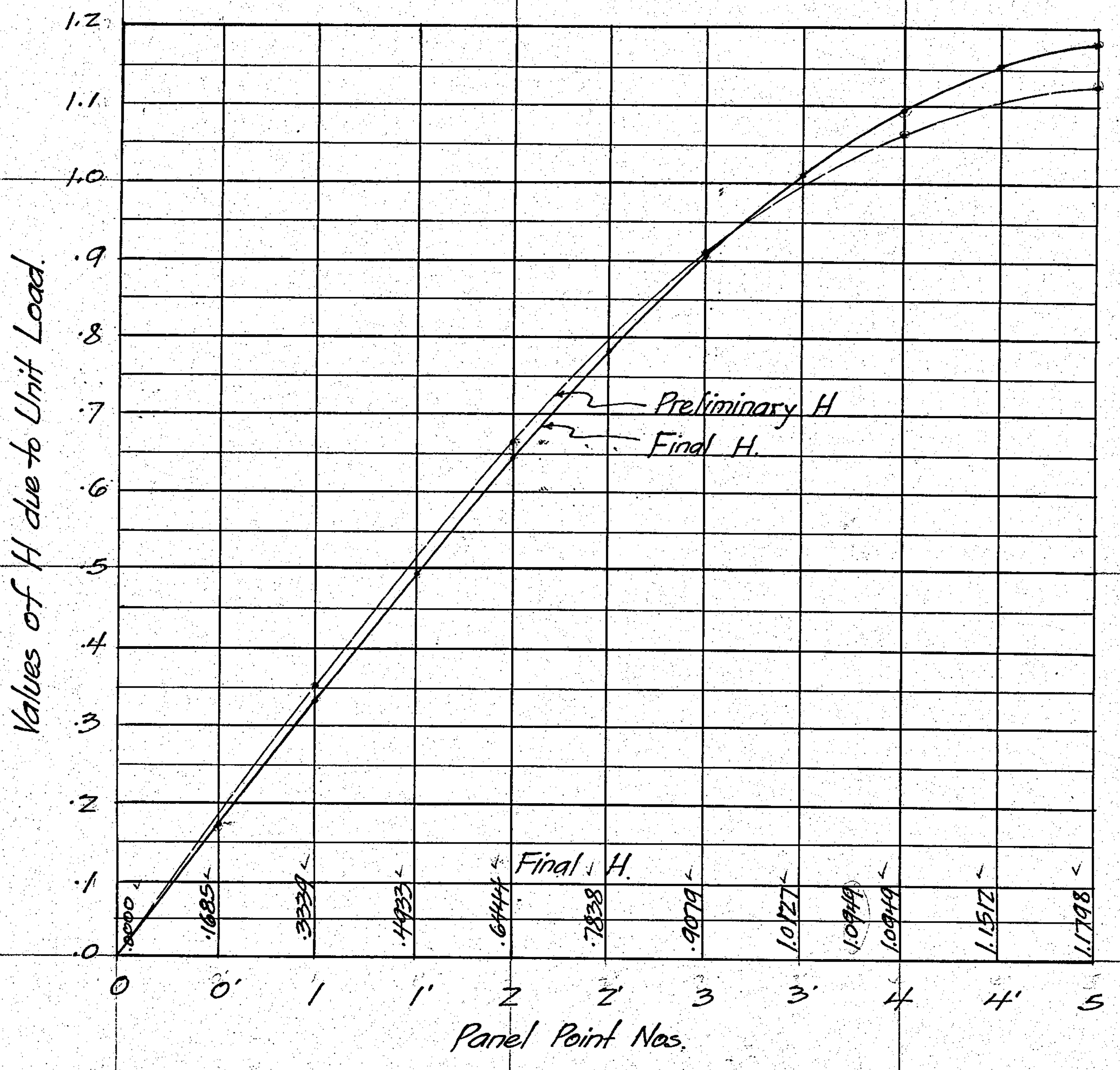
CALCULATIONS FOR

Final Design of Gian Boshi for Okayama prefecture.

H - Influence Lines.
preliminary and final H.



Vertical Shear due to unity

Load at panel pt.		Left Reaction V1.
0 ✓	$1 \times 20/20$ ✓ =	1.000 ✓
0' ✓	$1 \times 19/20$ ✓ =	.95 ✓
1 ✓	$1 \times 18/20$ ✓ =	.90 ✓
1' ✓		.85 ✓
2 ✓		.80 ✓
2' ✓		.75 ✓
3 ✓		.70 ✓
3' ✓		.65 ✓
4 ✓		.60 ✓
4' ✓		.65 ✓
5 ✓		.50 ✓
4' ✓		.45 ✓
4 ✓		.40 ✓
3' ✓		.35 ✓
3 ✓		.30 ✓
2' ✓		.25 ✓
2 ✓		.20 ✓
1' ✓		.15 ✓
1 ✓		.10 ✓
0' ✓		.05 ✓
0 ✓		.00 ✓

Reaction Locus. neutral axis assumed to be a parabola.

$q = \frac{1.6h}{1+k-k^2}$ ✓ $k = a/l$, $h = 8.54^m$ ✓

Panel point	$k = a/l$	k^2	$1+k-k^2$	$1.6h$	q
0 ✓	.00 ✓	.0000 ✓	1.0000 ✓	13.664 ✓	13.664 ✓
0' ✓	.05 ✓	.0025 ✓	1.0475 ✓	〃 ✓	13.044 ✓
1 ✓	.10 ✓	.0100 ✓	1.0900 ✓	〃 ✓	12.536 ✓
1' ✓	.15 ✓	.0225 ✓	1.1275 ✓	〃 ✓	12.119 ✓
2 ✓	.20 ✓	.0400 ✓	1.1600 ✓	〃 ✓	11.779 ✓
2' ✓	.25 ✓	.0625 ✓	1.1875 ✓	〃 ✓	11.507 ✓
3 ✓	.30 ✓	.0900 ✓	1.2100 ✓	〃 ✓	11.293 ✓
3' ✓	.35 ✓	.1225 ✓	1.2275 ✓	〃 ✓	11.132 ✓
4 ✓	.40 ✓	.1600 ✓	1.2400 ✓	〃 ✓	11.019 ✓
4' ✓	.45 ✓	.2025 ✓	1.2475 ✓	〃 ✓	10.953 ✓
5 ✓	.50 ✓	.2500 ✓	1.2500 ✓	〃 ✓	10.931 ✓

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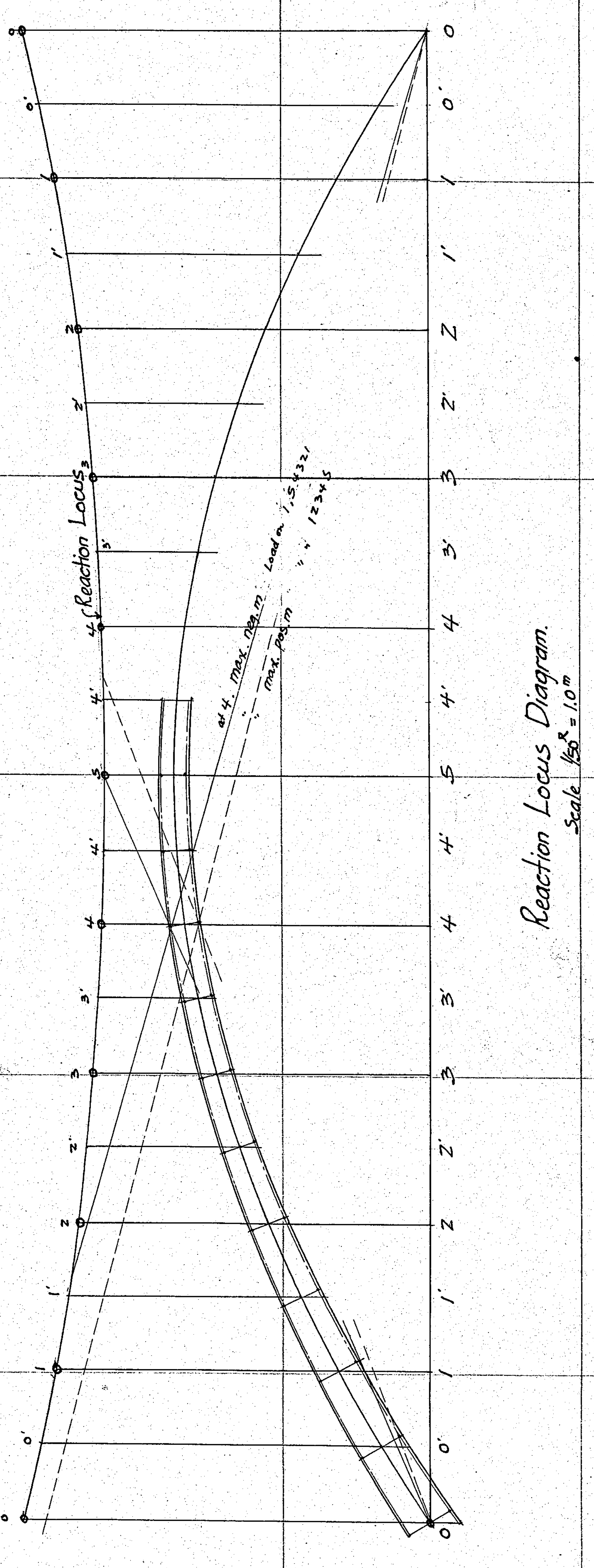
CALCULATIONS FOR

Final Design of Eian-Bashi for Okayama Ken.
Reaction Locus Diagram.



Final Design of Eian Bashi for Okayama Prefecture

Dead Load Stresses. For Panel Loads, see page 47.
Horizontal Thrust H and Vertical Shear V due to Dead Load.

Panel Point	Panel Loads	H unity	H	Vert. Shear V.		V.
0'	1,080 kg	0.1685	182 kg	54,360	=	54,360 kg
1	10,530	0.3339	3,517	54,360 − 1,080	=	53,280
1'	1,080	0.4933	532	53,280 − 10,530	=	42,750
2	10,530	0.6444	6,792	42,750 − 1,080	=	41,670
2'	1,080	0.7838	846	41,670 − 10,530	=	31,140
3	11,160	0.9079	10,130	31,140 − 1,080	=	30,060
3'	1,080	1.0127	1,094	30,060 − 11,160	=	18,900
4	11,160	1.0949	12,215	18,900 − 1,080	=	17,820
4'	1,080	1.1512	1,244	17,820 − 11,160	=	6,660
5	5,580	1.1798	6,580	6,660 − 1080	=	5,580
	V_1 = 54,360 kg		43,132			
	Shoe + extra steel = 7,200		× 2			
	Load on Shoe = 61,560 kg		86,264 kg			
		Call this	86,260 kg = H			

Normal Thrust $N = -(V \sin\phi + H \cos\phi)$ and Tangential Shear $T = +(V \cos\phi - H \sin\phi)$

Panel Point	V	H.	Sin φ	Cos φ	N		T.
0'	54,360 kg	86,260 kg	.5250	.8511	−101,890 kg	+	1,000
1	53,280	〃	.4769	.8790	−101,100	+	5,700
1'	42,750	〃	.4289	.9033	−96,230	+	1,650
2	41,670	〃	.3719	.9283	−95,490	+	6,730
2'	31,140	〃	.3170	.9484	−91,670	+	2,190
3	30,060	〃	.2540	.9672	−91,040	+	7,180
3'	18,900	〃	.1968	.9804	−88,220	+	1,560
4	17,820	〃	.1325	.9912	−87,760	+	6,240
4'	6,660	〃	.0669	.9978	−86,450	+	870
5	5,580	〃	.0000	1.0000	−86,260	+	5,580

Dead Load Moments at Several Sections. $m = M_0 - Hy = V_1 na - \Sigma pa - Hy$.

Panel points.	$V_1 na$			Σpa.	Hy.	m
	V_1	na			H y.	$V_1 na - \Sigma pa - Hy$.
0'	54,360 ×	2.5 =	135,900	0	86,260 × 1.732 = 149,200	− 13,300 Kgm
1	〃 ×	5.0 =	271,800	1080 × 2.5 = 2,700	〃 × 3.157 = 272,200	− 3,100
1'	〃 ×	7.5 =	407,500	1080 × 5.0 = 5400 10530 × 2.5 = 26350 } 31,750	〃 × 4.445 = 383,000	− 7,250
2	〃 ×	10.0 =	543,600	1080 × 2.5 × 4 = 10,800 10530 × 5.0 = 52,700 } 63,500	〃 × 5.531 = 477,000	+ 3,100
2'	〃 ×	12.5 =	679,500	1080 × 2.5 × 6 = 16,200 10530 × 2.5 × 4 = 105,300 } 121,500	〃 × 6.449 = 556,000	+ 2,000
3	〃 ×	15.0 =	815,000	1080 × 2.5 × 9 = 24,300 10530 × 5 × 3 = 158,000 } 182,300	〃 × 7.202 = 621,000	+ 11,700
3'	〃 ×	17.5 =	951,000	1080 × 2.5 × 12 = 32,400 10530 × 2.5 × 8 = 210,600 11160 × 2.5 × 1 = 27,900 } 270,900	〃 × 7.787 = 671,800	+ 8,300

Final Design of Eian Bashi for Okayama Prefecture.

Panel points	V_1	na	V_1.na	P a	Σpa		H y	Hy	m = V_1na − Σpa − Hy
4	54,360 ×	20.0 =	1,087,200	1080 × 2.5 × 16 =	43,200	362,500	86,260 × 8.205 =	707,500	+ 17,200 kgm
				10350 × 5.0 × 5 =	263,500				
				11160 × 5.0 =	55,800				
4'	"	× 22.5 =	1,223,000	1080 × 2.5 × 20 =	54,000	481,600	" × 8.456 =	729,000	+ 12,400
				10530 × 2.5 × 12 =	316,000				
				11160 × 2.5 × 4 =	111,600				
5	"	25.0 =	1,359,500	1080 × 2.5 × 25 =	67,500	603,500	" × 8.540 =	736,000	+ 20,000
				10530 × 2.5 × 14 =	368,500				
				11160 × 2.5 × 6 =	167,500				

Summary for Dead Load Stresses

Panel Point	Normal Thrust N	Tangential Shear T	Moment m		Horizontal Thrust. H	Max. load on shoe. D
0'	− 101,890 kg	+ 1,000 kg	−13,300 kgm	13.1	86,260 kg	61,560 kg on panel pt 0.
1	− 101,100	+ 5,700	− 3,100	− 3.0	"	
1'	− 96,230	+ 1,650	− 7,250	− 7.5	"	
2	− 95,490	+ 6,730	+3,100	3.2	"	
2'	− 91,670	+ 2,190	+ 2,000	3.2	"	
3	− 91,040	+ 7,180	+11,700	13.0	"	
3'	− 88,220	+ 1,560	+ 8,300	9.4	"	
4	− 87,760	+ 6,240	+17,200	19.6	"	
4'	− 86,450	+ 870	+12,400	14.4	"	
5	− 86,260	+ 5,580	+20,000	23.2	"	

Live Load Stresses.

Panel loads
See page 18.

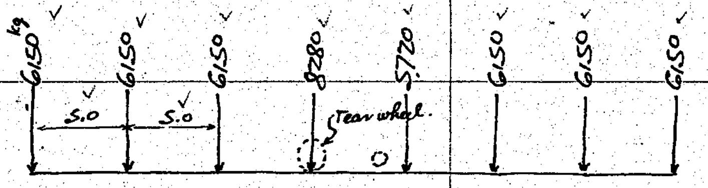


Table of V_1 and H due to single load at several panel points.

Panel Points		V_1				H				
		1 kg	6,150 kg	8,280 kg	5,720 kg	6,150 kg	8,280 kg	5,720 kg	1. kg	
Left of ℄	1	.9	5,535	7,450	5,150	2,055	2,765	1,910	.3339	1
	2	.8	4,920	6,625	4,575	3,965	5,335	3,685	.6444	2
	3	.7	4,305	5,795	4,005	5,585	7,515	5,195	.9079	3
	4	.6	3,690	4,970	3,430	6,735	9,065	6,265	1.0949	4
Right of ℄	5	.5	3,075	4,140	2,860	7,255	9,770	6,750	1.1798	5
	4	.4	2,460	3,310	2,290	6,735	9,065	6,265	1.0949	4
	3	.3	1,845	2,485	1,715	5,585	7,515	5,195	.9079	3
	2	.2	1,230	1,655	1,145	3,965	5,335	3,685	.6444	2
	1	.1	615	830	570	2,055	2,765	1,910	.3339	1

$N = -(V \sin\phi + H \cos\phi)$ $V = V_1 - \Sigma_0^{na} P$

$T = (V \cos\phi - H \sin\phi)$

$m = V_1 na - \Sigma pa - Hy$

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CALCULATIONS FOR

Final Design of Eiam-Bashi for Okayama Prefecture.

Stresses at Crown = panel point 5.

By the aid of Reaction Locus diagram, on page 54, we have loading conditions for max moments and shear.

positive max. moment at crown. Loads on 3, 4, ⑤, 4 + 3 note: O-mark shoes position of rear wheel of motor truck.

panel points	panel loads	H	V_1
3	6,150	5,585	4,305
4	〃	6,735	3,690
5	8,280	9,770	4,140
4	5,720	6,265	2,290
3	6,150	5,585	1,845
		H=33,940 kg	V_1=16,270 kg

Normal Thrust N = H = − 33,940 kg

Tangential shear V = 16,270 − 6150×2 = 3,970 kg

max. pos. m

V.x = 16270 × 25.0 = 406,500

Σpa = 6150 × 5.0 × 3 = − 92,300

Hy = 33,940 × 8.54 = − 289,800

+ 24,400 kgm

max. shear at crown. (positive)

panel pt.	panel loads	V_1
5	8,280	4,140
4	5,720	2,290
3	6,150	1,845
2	〃	1,230
1	〃	615
		10,120 kg

max positive shear at crown V = V_1 = +10,120 kg

Negative max. moment at crown. Loads on 1, ②, 3, 3, 2 + 1.

panel pts.	panel loads	H	V_1
1	6,150	2,055	5,535
2	8,280	5,335	6,625
3	5,720	5,195	4,005
3	6,150	5,585	1,845
2	〃	3,965	1,230
1	〃	2,055	615
		H = 24,190	V_1 = 19,855 kg

max. neg. moment.

V.x = 19855 × 25.0 = +496,400

Σpa = 6150 × 20 = 123,000; 8280 × 15 = 124,200; 5720 × 10 = 57,200 } − 304,400

Hy = 24,190 × 8.54 = − 206,500

− 14,500 kgm.

Normal thrust N = H = − 24,190 kg

Max. Live Load Thrust at Crown Full load.

1	6150	2,055	5,535
2	〃	3,965	4,920
3	〃	5,585	4,305
4	〃	6,735	3,690
5	8280	9,770	4,140
4	5,720	6,265	2,290
3	6,150	5,585	1,845
2	〃	3,965	1,230
1	〃	2,055	615
		H=45,980 kg	V_1=28,570 kg

moment.

V.x = 28,570 × 25.0 = +714,500

Σpa = 6150 × 10 × 5.0 = − 307,500

Hy = ~~8280~~ × 45,980 × 8.54 = − 392,500

+ 14,500 kgm

Normal thrust N = H = − 45,980 kg

Panel Point 4'

max. positive moment. Loads on 2, 3, ④, 5 + 4

2	6,150	3,960	4,920
3	6150	5,585	4,305
4	8280	9,065	4,970
5	5,720	6,750	2,860
4	6,150	6,735	2,460
		H= 32,095 kg,	V_1= 19,515 kg
			− 20,580
			V= −1,065 kg

moment.

V.x = 19,515 × 22.5 = + 439,000

Σpa = 6150 × 2.5 × 8 = 123,000; 8280 × 2.5 = 20,700 } = − 414,900

Hy = 32,095 × 8.456 = 271,200

+ 34,100 kgm

$N = -(V\sin\phi + H\cos\phi)$

$= -(-1065 \times .0669 + 32095 \times .9978) = -31,970$ kg

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CALCULATIONS FOR

Final Design of Eian Bashi for Okayama prefecture.

Max. negative moment Loads on 1, 2, + 4, 3, ②, 1

Panel points	Panel Loads	H	V_1
1	6150	2055	5535
2	〃	3965	4920
4	〃	6735	2460
3	〃	5585	1845
2	8280	5335	1655
1	5720	1910	570
		H = 25585 kg	V_1 = 16985
			12300
			V = 4685

moment

$V_1 x = 16{,}985 \times 22.5 = + 382{,}000$

$\Sigma pa = 6150 \times 2.\overset{5}{\cancel{75}} \times 12 = - 184{,}500$

$Hy = 25{,}585 \times 8.456 = - 216{,}200$

$- 18{,}700$ kgm

$N = -(4685 \times .0669 + 25585 \times .9978) = - 25850$ kg

Panel Point 4.

max. positive moment. Loads on 1, 2, 3, ④ + 5 left side.

1	6150	2055	5535
2	〃	3965	4920
3	〃	5585	4305
4	8280	9065	4970
5	5720	6750	2860
		H = 27420	V_1 = 22,590
			18,450
			V = 4,140

moment.

$V_1 x = 22{,}590 \times 20.0 = + 451{,}800$

$\Sigma pa = 6150 \times 2.5 \times 12 = - 184{,}500$

$Hy = 27{,}420 \times 8.205 = - 225{,}000$

$+ 42{,}300$ kgm

$N = -(4140 \times .1325 + 27420 \times .9912) = - 27750$ kg

negative max. moment. Loads on 1 + 4, ③, 2, 1

1	6150	2055	5535
4	〃	6735	2460
3	8280	7515	2485
2	5720	3685	1145
1	6150	2055	615
		H = 22,045	V_1 = 12240
			6150
			V = 6,090

moment.

$V_1 x = 12{,}240 \times 20.0 = + 244{,}800$

$\Sigma pa = 6150 \times 15.0 = - 92{,}300$

$Hy = 22045 \times 8.205 = - 180{,}800$

$- 28{,}300$ kgm

$N = -(6090 \times .1325 + 22045 \times .9912) = - 22660$ kg

max. positive Shear Loads on ④, 5, 4, 3, 2 + 1

4	8280	9065	4970
5	5720	6750	2860
4	6150	6735	2460
3	〃	5585	1845
2	〃	3965	1230
1	〃	2055	615
		H = 34,155	V_1 = 13980 = V

$T = (13980 \times .9912 - 34155 \times .1325) = + \overset{9{,}340}{\cancel{21010}}$ kg

Panel Point 3'

max. positive moment. Loads on 1 2 3 ④ + 5 left.

Loading same as for 4.

H = 27420

V_1 = 22,590

V = 4140

moment.

$V_1 x = 22590 \times 17.5 = + 395200$

$\Sigma pa = 6150 \times 2.5 \times 9 = - 138{,}300$

$Hy = 27420 \times 7.787 = - 213{,}500$

$+ 43{,}400$ kgm

$N = -(4140 \times .1968 + 27420 \times .9804) = - 27720$ kg

negative max. moment. Loads on 5, 4, ③, 2 + 1 right

5	6150	7255	3075
4	〃	6735	2460
3	8280	7515	2485
2	5720	3685	1145
1	6150	2055	615
		H = 27,245	V_1 = 9780 = V

moment.

$V_1 x = 9780 \times 17.5 = + 171{,}300$

$Hy = 27245 \times 7.787 = - 212{,}200$

$- 40900$ kgm

$N = -(9780 \times .1968 + 27245 \times .9804) = - 28630$ kg

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CALCULATIONS FOR

Final Design of Eian Bashi for Okayama Prefecture.

Panel Point 3.

max. positive moment. Loads on 1.2.③.4.5 left.

panel point	Panel Loads	H	V1
1	6,150	2,055	5,535
2	〃	3,965	4,920
3	8,280	7,515	5,795
4	5,720	6,265	3,430
5	6,150	7,255	3,075
		H = 27,055	V1 = 22,755
			12,300
			V = 10,455

moment
V1x = 22,755 · 15.0 = +341,200
Σpa = 6,150 · 5.0 · 3 = − 92,250
Hy = 27,055 · 7.202 = − 194,600
+ 54,350 kgm

N = −(10,455 · .254 + 27,055 · .9672) = −28,800 kg

negative max. moment. Loads on 5.4.③.2.1 right
Loading same as for 3'
H = 27,245 V1 = 9,780 = V

moment
V1x = 9,780 · 15.0 = + 146,700
Hy = 27,245 · 7.202 = − 196,100
− 49,400 kgm

N = −(9780 · .254 + 27,245 · .9672) = −28,840 kg

max. positive shear at 3. Loads on ③ 4.5.4.3.2.1.

3	8,280	7,515	5,795
4	5,720	6,265	3,430
5	6,150	7,255	3,075
4	〃	6,735	2,460
3	〃	5,585	1,845
2	〃	3,965	1,230
1	〃	2,055	615
		H = 39,375	V1 = 18,450 = V.

T = (18450 · .9672 − 39,375 · .254) = + 7,850 kg

Panel Point 2'

max. positive moment. Loads on 1.2.③+4 left.

1	6150	2055	5,535
2	〃	3965	4,920
3	8,280	7515	5,795
4	5720	6265	3,430
		H = 19,800	V1 = 19,680
			12,300
			V = 7,380

moment
V1x = 19,680 · 12.5 = + 246,000
Σpa = 6,150 · 2.5 · 4 = − 61,500
Hy = 19,800 · 6.449 = − 127,600
+ 56,900

N = −(7,380 · .3170 + 19800 · .9484) = − 21,120 kg

max. negative moment. Loads on 4.5.4.③.2+1 right

4	6,150	6,735	3,690
5	〃	7,255	3,075
4	〃	6,735	2,460
3	8280	7,515	2,485
2	5,720	3685	1,145
1	6,150	2055	615
		H = 33,980	V1 = 13,470 = V

moment
V1x = 13,470 · 12.5 = + 168,300
Hy = 33,980 · 6.449 = − 219,200
− 50,900 kgm

N = −(13470 · .3170 + 33980 · .9484) = − 36,520 kg

Panel Point 2

max. positive moment. Loads on 1②3+4 left.

1	6150	2,055	5,535
2	8,280	5,395	6,625
3	5,720	5,195	4,005
4	6,150	6,735	3,690
		H = 19,320	V1 = 19,855
			6,150
			V = 13,705

moment
V1x = 19,855 · 10.0 = + 198,550
Σpa = 6,150 · 5.0 = − 30,750
Hy = 19,320 · 5.531 = − 106,800
+ 61,000 kgm.

N = −(13,705 · .3719 + 19,320 · .9283) = − 23,050 kg

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Negative max. moment. Loads on 4.5.4③2+1
Loading same as for 2'
H = 33,980 V₁ = 13470 = V

moment
V.x = 13470 × 10.0 = + 134,700
Hy = 33980 × 5.531 = − 188,000
− 53300 kgm

N = −(13470 × .3719 + 33980 × .9283) = − 36,560 kg

Max. positive shear Loads on ②3 4 5

panel points	panel loads	H	V₁
2	8280	5335	6625
3	5720	5195	4005
4	6150	6735	3690
5	〃	7255	3075
		H = 24,520	V₁ = 17395 = V

T = (17395 × .9283 − 24520 × .3719) = + 7030 kg

Panel Point 1'

max. positive moment. Loads on 1②3 4 left.
Loading same as for 2.
H = 19320, V₁ = 19855
V = 13705

moment
V.x = 19855 × 7.5 = + 149000
Σpa = 6150 × 2.5 = − 15370
Hy = 19320 × 4.445 = − 85830
+ 47800 kgm

N = −(13705 × .4289 + 19320 × .9033) = − 23,830 kg

Negative max. moment. Loads on 4 5 4③2 1
Loading same as for 2.
H = 33,980 V₁ = 13470 = V

moment
V.x = 13470 × 7.5 = + 101,000
Hy = 33980 × 4.445 = − 151,100
− 50,100 kgm

N = −(13470 × .4289 + 33980 × .9033) = − 36480 kg

Panel Point 1.

max. positive moment. Loads on ①2,3,4 left.

1	8280	2765	7450
2	6150	3965	4920
3	6150	5585	4305
4	6150	6735	3690
		H = 19,050	V₁ = 20,365 = V.

moment
V.x = 20365 × 5.0 = + 101,900
Hy = 19,050 × 3.157 = − 60,100
+ 41800 kgm

N = −(20365 × .4769 + 19050 × .8790) = − 26450 kg

Max. negative moment. Loads on 3.4.5.④3.2.+1

3	6150	5585	4305
4	〃	6735	3690
5	〃	7255	3075
4	8280	9065	3310
3	5780	5195	1,715
2	6150	3965	1,230
1	〃	2055	615
		H = 39,855	V₁ = 17,940 = V.

moment
V.x = 17940 × 5.0 = + 89,700
Hy = 39855 × 3.157 = − 125,800
− 36,100 kgm

N = −(17940 × .4769 + 39855 × .8790) = − 43,600 kg

max positive shear. Loads on ①2,3,4
same as for pos. m.
H = 19050, V₁ = 20365 = V

T = (20365 × .8790 − 19050 × .4769) = + 8820 kg

Panel point 0'

max. positive moment Loads on ①2.3.4.5.4

1	8280	2765	7450
2	6150	3965	4920
3	〃	5585	4305
4	〃	6735	3690
5	〃	7255	3075
4	〃	6735	2460
		H = 33,040	V₁ = 25,900 = V.

moment
V.x = 25900 × 2.5 = + 64,800
Hy = 33,040 × 1.732 = − 57,200
+ 7,600 kgm

N = −(25900 × .5250 + 33,040 × .8511) = − 41,700 kg

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max. negative moment. Loads on 3,4,5④3,2+1.

panel pt.	panel loads	H	V₁
3	6,150	5585	4305
4	〃	6735	3690
5	〃	7255	3075
4	8280	9065	3310
3	5720	5195	1715
2	6150	3965	1230
1	〃	2055	615
		H= 39,855	V₁= 17940 = V.

moment.

$V.x = 17940 \cdot 2.5 = + 44850$

$Hy = 39855 \cdot 1.732 = - 69000$

$- 24150$ kgm

$N = -(17940 \cdot .5760 + 39855 \cdot .8511) = - 43350$ kg

Summary for Dead and Live Load Stresses.

	Panel Point 0'		Panel Point 1		Panel Point 1'		panel point 2		Panel Point 2'	
	N	m	N	m	N	m	N	m	N	m
Dead Load	-101,890	- 13,300	-101,100	- 3,100	-96,230	-7,250	-95,490	+ 3,100	-91,670	+ 2,000
+ Live Load	- 41,700	+ 7,600	-26,450	+ 41,800	-23,330	+47,800	-23,050	+61,000	-21,120	+56,900
Total.	-143,590 kg	- 5,700 kgm	-127,550	+ 38,700	-119,560	+40,550	-118,540	+64,000	-112,790	+58,900

18.3 15.8 20.5

	Panel Point 3		panel point 3'		panel point 4		Panel point 4'		Panel point 5 (crown)	
	N	m	N	m	N	m	N	m	N	m
Dead Load	- 91,040	+ 11,700	- 88,220	+ 8,300	-87,760	+ 17,200	- 86,450	+12,400	- 86,260	+ 20,000
+ Live Load	- 28,800	+ 54,350	- 27,720	+ 43,400	-27,750	+ 42,300	- 31,970	+ 34,100	-33,940	+ 24,400
Total.	-119,840	+ 66,050	-115,940	+51,700	- 115,510	+ 59,500	- 118,420	+ 46,500	-120,200	+44,400

	Panel Point 0'		panel point 1		panel point 1'		panel point 2		panel point 2'	
	N	m	N	m	N	m	N	m	N	m
Dead Load	- 101,890	-13,300	-101,100	- 3,100	- 96,230	- 7,250	- 95,490	+3,100	-91,670	+ 2,000
- Live Load	- 43,350	- 24,150	- 43,600	-36,100	-36,480	-50,100	-36,560	-53,300	-36,520	-50,900
Total	-145,240 kg	-37,450 kgm	-144,700	- 39,200	-132,710	-57,350	-132,050	-50,200	-128,190	-48,900

	panel point 3		Panel point 3'		Panel point 4		Panel point 4'		panel point 5 (crown)	
	N	m	N	m	N	m	N	m	N	m
Dead Load	- 91,040	+ 11,700	-88,220	+ 8,300	-87,760	+ 17,200	-86,450	+ 12,400	-86,260	+ 20,000
- Live Load	-28,840	- 49,400	-28,630	-40,900	-22,660	- 28,300	-25,850	- 18,700	-24,190	-14,500
Total.	-119,880	-37,700	-116,850	-32,600	-110,420	- 11,100	-112,300	- 6,300	-110,450	+ 5,500

Panel points

Tangential Shear T.	0'	1	1'	2	2'	3	3'	4	4'	5
Dead Load T	+1,000	+5,700	+ 1650	+ 6,730	+ 2190	+ 7180	+1,560	+ 6240	+ 870	+ 5580
Live Load T.		+8820		+7030		+7850		+9340		+10120
		+14,520 kg		+ 13,760 kg		+15,030 kg		+15,580 kg		+15,700 kg

max. Load on shoe

Live Load $6150 \cdot (.9+.8+.7+ \cdots +.2+.1) = 27,680$

8280

35960

Dead Load. 61,560

97520 kg max. load on one shoe.

Shoe say 480

98000 kg

Max. Stress on tie

Dead Load H = 86,260

Live 〃 H = 45980

total = 132240 kg max. tension on tie

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Fibre Stresses at Several Sections of Arch Ring.

Crown Section or panel point 5.

Positive moment = $+44,400^{\text{kgm}\checkmark}$, $N = -120,200^{\checkmark\text{kg}}$

moment stress on top flange $= \dfrac{44,400^{\checkmark} \times 100^{\checkmark}}{523,200^{\checkmark}} \times 42.1^{\checkmark} =$ $-357^{\checkmark}$

Direct compression 〃 $= \dfrac{-120,200^{\checkmark}}{279.22^{\checkmark}} =$ $\underline{-431^{\checkmark}}$

$-788^{\checkmark}$ comp. kg/cm² ok.

Negative moment = $+5,500^{\checkmark\text{kgm}}$ $N = -110,450^{\checkmark\text{kg}}$

moment stress on bottom flange $= \dfrac{+5,500^{\checkmark} \times 100^{\checkmark}}{523,200^{\checkmark}} \times 59.0^{\checkmark} =$ $+62^{\checkmark}$

Direct compression 〃 〃 $= \dfrac{-110,450}{279.22^{\checkmark}} = \; =$ $\underline{-395^{\checkmark}}$

~~-457~~ $333^{\checkmark}$ comp. kg/cm² ok.

Panel point 4'

positive moment $= +46,500^{\text{kgm}\checkmark}$, $N = -118,420^{\checkmark\text{kg}}$

moment stress on top flange $= \dfrac{46,500^{\checkmark} \times 100^{\checkmark}}{533,700^{\checkmark}} \times 42.4^{\checkmark} =$ $-370^{\checkmark}$

Direct compression 〃 〃 $= \dfrac{118,420^{\checkmark}}{279.22^{\checkmark}}$ $= \underline{-424^{\checkmark}}$

$-794^{\checkmark}$c kg/cm² ok

Negative moment $= -6,300^{\text{kgm}\checkmark}$, $N = -112,300^{\text{kg}\checkmark}$

Moment stress on ~~top~~ bottom flange $= \dfrac{6300 \times 100^{\checkmark}}{533,700^{\checkmark}} \times 59.5^{\checkmark} = -70^{\checkmark}$

Direct comp. 〃 〃 $= \dfrac{112,300^{\checkmark}}{279.22^{\checkmark}}$ $= \underline{-403^{\checkmark}}$

$-473^{\checkmark}$c 〃 ok

Panel point 4.

max positive moment $= +59,500^{\text{kgm}\checkmark}$, $N = -115,510^{\text{kg}\checkmark}$

moment stress on top flange $= \dfrac{59,500 \times 100^{\checkmark}}{563,900^{\checkmark}} \times 43.5^{\checkmark} = -459^{\checkmark}$

Direct comp. 〃 〃 $= \dfrac{115,510^{\checkmark}}{279.22^{\checkmark}}$ $= \underline{-414^{\checkmark}}$

$-873^{\checkmark}$c 〃 ok

negative moment $= -11,100^{\text{kgm}\checkmark}$, $N = -110,420^{\text{kg}\checkmark}$

moment stress on bottom flange $= \dfrac{11100 \times 100^{\checkmark}}{563,900^{\checkmark}} \times 60.9^{\checkmark} = -120^{\checkmark}$

Direct comp. 〃 〃 $= \dfrac{110,420^{\checkmark}}{279.22^{\checkmark}}$ $= \underline{-396^{\checkmark}}$

$-516^{\checkmark}$C 〃 ok

Panel point 3'

positive moment $= +51,700^{\checkmark\text{kgm}}$, $N = -115,940^{\checkmark\text{kg}}$

moment stress on top flange $= \dfrac{51,700 \times 100^{\checkmark}}{614,900^{\checkmark}} \times 45.2^{\checkmark} = -380^{\checkmark}$

Direct comp. on 〃 〃 $= \dfrac{115,940^{\checkmark}}{279.22^{\checkmark}}$ $= \underline{-415^{\checkmark}}$

$-795^{\checkmark}$c 〃 ok

negative moment $= -32,600^{\checkmark\text{kgm}}$, $N = -116,850^{\text{kg}\checkmark}$

moment stress on bottom flange $= \dfrac{32600 \times 100^{\checkmark}}{614,900^{\checkmark}} \times 63.3^{\checkmark} = -336^{\checkmark}$

Direct comp. 〃 〃 $= \dfrac{116,850^{\checkmark}}{279.22^{\checkmark}}$ $= \underline{-419^{\checkmark}}$

$-755^{\checkmark}$c 〃 ok

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CALCULATIONS FOR

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Panel point 3.

Positive moment = +66,050 kgm, N = -119,840 kg

moment stress on top flange = $\frac{66050 \times 100}{684400} \times 47.4 = -458$

Direct compression = $\frac{119840}{279.22} = -430$

} 888 kg/cm² c ok

Negative moment = -37700 kgm, N = -119,880 kg

moment stress on bottom flange = $\frac{37700 \times 100}{684400} \times 66.4 = -366$

Direct comp. = $\frac{119880}{279.22} = -430$

} 796 kg/cm² c ok

Panel point 2'

Positive moment = +58900 kgm, N = -112,790 kg

moment stress on top flange = $\frac{58900 \times 100}{781900} \times 50.3 = -379$

Direct comp. = $\frac{112790}{279.22} = -406$

} 785 kg/cm² c ok

Negative moment = -48900 kgm, N = 128,190 kg

moment stress on bottom flange = $\frac{48900 \times 100}{781900} \times 70.5 = 441$

Direct comp. = $\frac{128190}{279.22} = -459$

} 900 kg/cm² c ok

Panel point 2.

Positive moment = +64,100 kgm, N = 118540 kg

Moment stress on top flange = $\frac{64100 \times 100}{903100} \times 53.6 = -381$

Direct comp. = $\frac{118540}{279.22} = -425$

} 806 kg/cm² c ok

Negative moment = -50,200 kgm, N = -132,050 kg

moment stress on bottom flange = $\frac{50,200 \times 100}{903100} \times 75.3 = -419$

Direct comp. = $\frac{132050}{279.22} = -473$

} 892 kg/cm² c ok

Panel point 1'

positive moment = +40550 kgm, N = 119560 kg

Moment stress on top flange = $\frac{40550 \times 100}{1053500} \times 57.4 = -222$

Direct comp. = $\frac{119560}{279.22} = -429$

} 651 kg/cm² c ok

Negative moment = -57350 kgm, N = -132,710 kg

moment stress on bottom flange = $\frac{57350 \times 100}{1053500} \times 80.8 = -440$

Direct comp. = $\frac{132710}{279.22} = -475$

} 915 kg/cm² c ok.

Panel point 1.

Positive moment = +38,700 kgm, N = -127,550 kg

Moment stress on top flange = $\frac{38700 \times 100}{1032000} \times 64.3 = -242$

Direct comp. = $\frac{127550}{235.30} = -543$

} 785 kg/cm² c ok.

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negative moment = $-39{,}200^{kgm}$, $N = -144{,}700^{kg}$

moment stress on bottom flange = $\frac{39{,}200 \times 100}{1032000} \times 83.7 = -31.8$ } $934^{kg/cm^2}$ ok

Direct comp. " " = $\frac{144700}{235.30} = -616$

Panel Point O'

positive moment = ~~+~~ $-5{,}700^{kgm}$, $N = -143{,}590^{kg}$

moment stress on top flange = $\frac{5700 \times 100}{1208400} \times 69.1 = -33$ } $644^{kg/cm^2}$ ok

Direct comp. " = $\frac{143590}{235.30} = -611$

negative moment = $-37{,}450^{kgm}$, $N = -145{,}240^{kg}$

moment stress on bottom flange = $\frac{37450 \times 100}{1208400} \times 90.0 = -279$ } $897^{kg/cm^2}$ ok.

Direct comp. on " = $\frac{145240}{235.30} = -618$

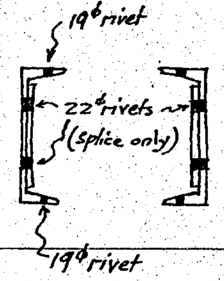
Max. Tension on Tie.

Dead Load. $H = 86{,}260$

Live Load $H = 45980$

$132{,}240^{kg}$ net area required = $\frac{132240}{1200} = \underline{\underline{110.3}}^{cm^2}$ net.

Final Section of Tie.



2[s 300×90@38.18 = 97.15
2Pls. 240×9 = 43.20

140.35^{cm^2} gr. − 28.56 = $\underline{\underline{111.79}}^{cm^2}$ net. ok.

Reduction area of rivet holes.

2[s flange $4 \times 1.2 \times 2.2 = 10.56$
" web $4 \times .9 \times 2.5 = 9.00$
2Pls. $4 \times .9 \times 2.5 = 9.00$

$28.56^{□cm}$.

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CALCULATIONS FOR

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Stresses for web members of Arch Truss. (Coefficient being found by graphical method).

Panel point no.	Tangential Shear, T	Diagonal coeff.	Vertical coeff.	Diagonal stress Tension	vertical stress Compression	Section Reqd. cm²	Rivet no required Diagonal	Rivet no required vertical
1	+ 14,520	1.005	1.125	14,590 ^{kg}T	16,340 ^{kg}c		6.9–19ϕ	7.7–19ϕ
2	+ 13,760	1.045	1.075	14,370 〃	14,800 〃		6.8–19ϕ	7.0–〃
3	+ 15,030	1.130	1.030	16,980 〃	15,480 〃		8 –19ϕ	7.3–〃
4	+ 15,580	1.250	1.010	19,480 〃	15,750 〃	max.	6.8–22ϕ	7.4–〃
crown 5	+ 15,700	1.350	1.000	$\boxed{21,200}$ 〃	15,700 〃	$\boxed{17.7\ ^{cm^2}_{net}}$	7.4–22ϕ	7.4–〃

Final Section of Diagonal members max. stress = 21,200 ^{kg} tension.

net sectional area req'd = 17.7 $^{cm^2}$ net.

Use 4Ls 75×75×9 = 50.76 $^{cm^2}$ − 8×2.25 = 32.76 $^{cm^2}$ ok.

Stresses on Hanger

Dead Load
Slab, pavement, handrail, stringers, floor beam &c. = 8,041
Bottom lateral bracing = 235
Tie and hanger = 1,049
Rivet head &c = 176
9501 call this 9,500 ^{kg}

Live Load.
motor truck rear wheel concentration = 2,250
Impact coeff. = $\frac{\cancel{6}\cdot 20}{60+10}$ = 28.6% = 650
2900 ^{kg} × 2 = 5800 ^{kg}
motor truck front wheel concentration with impact ≑ 5800 ÷ 3 ≑ 1930 ^{kg}

Panel Load.
2950 ---- 1970
5800 ---- 0
770 ---- 1160
280 ---- 1570
8820

max Load on hanger
Dead Load = 9,500
Live Load 8820
18,320 ^{kg}

net sectional area required for hanger = $\frac{18320}{1200}$ = 15.3 $^{cm^2}$ net. 19ϕ rivet no. = $\frac{18320}{2126}$ = 8.6

Final section of hanger 4Ls 90×75×9 = 56.16 − ~~1~~8×1.98 = 40.30
1Pl 290×8 = 23.20 − 2×1.76 = 19.68
59.98 $^{cm^2}$ net. ok.

Least radius of gyration = 4.1 cm abt.
unsupported length of the longest hanger = 8.5 − .5 − .9 = 7.1 ^{m}
Slenderness ratio l/r = 710/4.1 = 173 < 200 ok.

Vertical members. Use same section as for diagonal members.

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CALCULATIONS FOR

Material List of Eian Bashi for Okayama prefecture.

Materials for Floor.

Concrete : 1:2:4 mixture.

Total net length of Slab.

Span nos. 1 to 6 inclusive 6@ 18.30 = 109.80
Construction jts. 6@ .01 = .06
109.74 m net

Span no 7. 18.335
Const. joint .01
18.325 net.

} 128.07

Span no 8 50.79
Const. joints 4@.01 = .04
50.75 net

Span nos. 9 to 11 inclusive 3@ 50.64 = 151.92
Const. joints 12@.01 = .12
151.80 net

} 202.55

Total = 330.62 meters net.

Cross Sectional area of Slab.

Slab between curb lines 5.4 × 0.13 = 0.702 □m
Fillets on Stringers 2@.188 × .038 = 0.014
Coping 2@.30 × .18 = 0.108
" 2@.24 × .141 = 0.068
0.892 × 330.62 = 295.00 cub meters.

Fillet on ~~Floor Beams~~. Expansion metal = 22@.07 × .05 × 5.4 = 0.42
fillet instead of cover pl. at end of girders. = 28@.36 × .35 × .013 = 0.46
Less drain holes. 90@.005 = − 0.45
Less knotches of Coping for truss ends. 16@.02 = − 0.32
Total concrete for floor slab. = 295.11 cub meters.

Forms for Slab.

bottom of slab 5.40 = 5.40
coping outside + bott. 0.62 × 2 = 1.24
curb line 0.203 × 2 = 0.41
7.05 × 330.62 = 2330.8 □m

Stringer. top (girder) .15 × 2 × 128.07 = 38.4
" truss .15 × 4 × 202.55 = 121.5
Girder. top .34 × 2 × 128.07 = 87.1
Exp. metal. flange 22 × .07 × 5.4 = 8.3
− 255.3 □m
2075.5 sq. meters.

Construction Joints

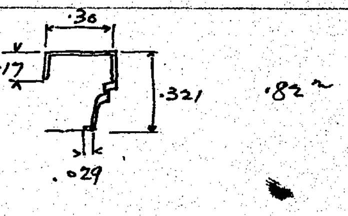
24 joints. Use Carey Elastite filling.

Pavement. Asphalt block. 3.8 cm thick with 1.5 cm cement mortar cushion.

Total net length of slab = 330.6 m.
width of exp. metals. − 2.6 .08 × 10 + .225 × 8 = 2.60
add const. joints .23 + .03 = + .26
328.26 meter net.

area of pavement 328.26 × 5.4 = ~~1232.60~~ 1772.6 □m
less drain area .22 × .315 × 90 = − 6.20
1766.4 sq. meters.

Artificial Granite finish Coping. 2 × 0.82 × 330.62 = 542.0
Handrail posts area. 404 @.2 × .17 = − 13.7
528.3 □m



Reinforcements. plain Bars. 38.7350 kg tons.

Cast iron drains. 90 drains reqd. @ 17.9 kg. = 1.611 kg tons.
90 Steel tubes 2 mm thick

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CALCULATIONS FOR

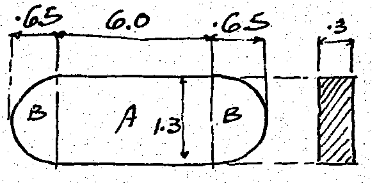
Material List of Eian Bashi for Okayama Prefecture.

Materials for Piers P1, P2, P3, P4, P5, + P6.

Shaft.

Concrete 1:2:4 mixture.

Coping.	Section	Length	Reqd. no.	Volume	Remarks.
Coping rectangle	1.3×.3	6.0	1	2.34	A
" Circular ends	1.3^{ϕ}	·3	1	·40	B.
				2.74 cub. meter.	



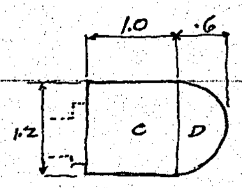
Shaft

Top section (for 2 columns.)

Rectangle C 1.0×1.2 ×2 = 2.400

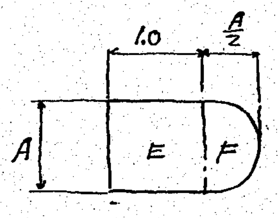
Circular ends. D. 1.2^{ϕ} = 1.131

3.531 □m



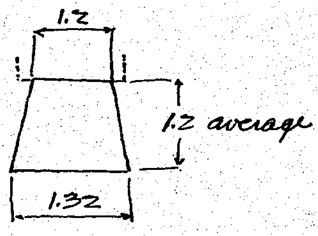
Bottom section (for 2 columns)

Piers	Dimension A		area E		Circular ends F		Bottom area
P1	1.789	×1.0×2 =	3.578	+	2.514	=	6.092 □m
P2	1.798	"	3.596	+	2.539	=	6.135
P3	1.805	"	3.610	+	2.559	=	6.169
P4	1.812	"	3.624	+	2.579	=	6.203
P5	1.816	"	3.632	+	2.590	=	6.222
P6	1.820	"	3.640	+	2.602	=	6.242



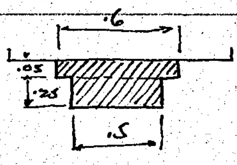
Top Beam.

$$\frac{1.2+1.32}{2} \times 1.2 \times 4.0 = 6.048 \text{ cub. meters}$$



Ribs of Shaft + Beam.

Pier	Sectional area	Length				Total length	Volume
P1	·155 □m	4.291 m ×2	+	3.92 m	=	12.502 m	1.938 m³
P2	"	4.378 "		"		12.676	1.964
P3	"	4.453 "		"		12.826	1.988
P4	"	4.515 "		"		12.950	2.007
P5	"	4.564 "		"		13.048	2.022
P6	"	4.601 "		"		13.122	2.034



Sectional area
.05×.6 = .03
.25×.5 = .125
·155 □m

Total volume of shafts for several piers.

Piers	Top area		Bottom area.		Average		Height		Shaft.		Coping		Top. Beam		Ribs		Total volume
P1	3.531	+	6.092	÷2=	4.812 □m	×	5.891	=	28.347 m³	+	2.740	+	6.048	+	1.938	=	39.074 ³
P2	"		6.135		4.833		5.978		28.892		"		"		1.964		39.644
P3	"		6.169		4.850		6.053		29.357		"		"		1.988		40.133
P4	"		6.203		4.867		6.115		29.762		"		"		2.007		40.557
P5	"		6.222		4.877		6.164		30.062		"		"		2.022		40.872
P6	"		6.242		4.887		6.201		30.304		"		"		2.034		41.126
																	241.405

average = 40.234

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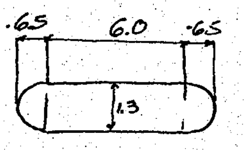
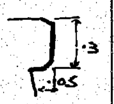
CALCULATIONS FOR

Material List of Eian Bashi for Okayama prefecture.

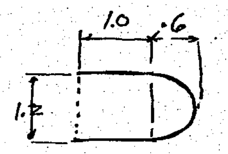
Forms for Shaft.

Coping Straight portion 6.0 × 2 = 12.00
Circular ends. 1.3ϕ = 4.08
16.08 × .35 = 5.63 °m for all piers



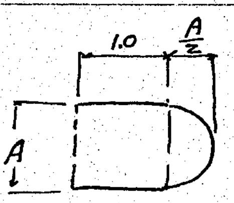


Shaft. total perimeter at top. (for 2 columns)
1.0 × 4 = 4.00
1.2ϕ = 3.77
7.77 m

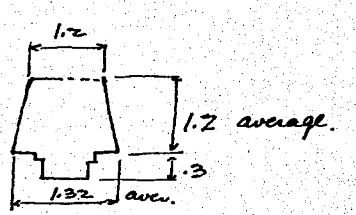


Perimeter at bottom. (for 2 columns).

Piers	Dimension A	Sides	Circular ends	perimeter at bottom.
P1	1.789	1.0 × 4 = 4.0 m	5.62 m	9.62 m
P2	1.798	〃	5.65	9.65
P3	1.805	〃	5.67	9.67
P4	1.812	〃	5.69	9.69
P5	1.816	〃	5.71	9.71
P6	1.820	〃	5.72	9.72



Top Beam Sides bottom total area
1.2 × 4.0 × 2 + 1.92 × 3.92 = 17.12 °m



Inside face of Shaft. for 2 cols.

Pier	top width	bottom width	average	prodjection	total	height	Area	total area
P1	1.36 m	1.789	1.575	+ .60 =	2.175 ×	4.291 =	9.33 × 2 =	18.66 °m
P2	〃	1.798	1.579	〃	2.179	4.378	9.54 〃 =	19.08
P3	〃	1.805	1.583	〃	2.183	4.453	9.72 〃 =	19.44
P4	〃	1.812	1.586	〃	2.186	4.515	9.87 〃 =	19.74
P5	〃	1.816	1.588	〃	2.188	4.564	9.99 〃 =	19.98
P6	〃	1.820	1.590	〃	2.190	4.601	10.08 〃 =	20.16

Total form for Shaft.

Pier	top perim.	Bott. perim.	average	Height	Shaft total	Coping	Top Beam.	Inside face	Total area
P1	7.77 m	9.62	8.70 m ×	5.891 =	51.25 °m +	5.63 °m +	17.12 °m +	18.66 °m =	92.66 °m
P2	〃	9.65	8.71	5.978	52.07	〃	〃	19.08	93.90
P3	〃	9.67	8.72	6.053	52.78	〃	〃	19.44	94.97
P4	〃	9.69	8.73	6.115	53.38	〃	〃	19.74	95.87
P5	〃	9.71	8.74	6.164	53.87	〃	〃	19.98	96.60
P6	〃	9.72	8.75	6.201	54.26	〃	〃	20.16	97.17

Total = 571.17 °m
average = 95.19 °m

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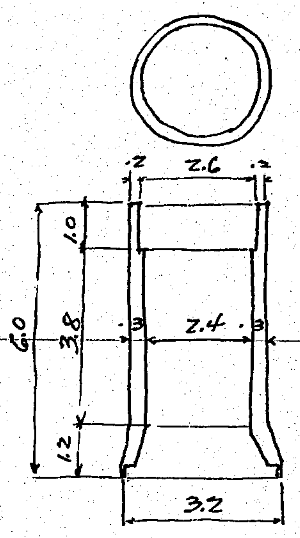
CALCULATIONS FOR

Material List of Eian-Bashi for Okayama Prefecture.



Wells:

Wells for Piers P1, P2, P3, P4, P5 + P6 $3.0^{m\phi} \times 6.0$ well.

Concrete 1:2:4 mixture

Shell top 1.0^m

Sectional area 3.0^ϕ = 7.069

2.6^ϕ = -5.309

$1.760^{\square m} \times 1.0^m$ = 1.760 cub. meters.

Shell Intermediate 3.8^m

Sectional area 3.0^ϕ = 7.069

2.4^ϕ = -4.524

$2.545 \times 3.8 =$ 9.671 cub. meters

Shell Bottom 1.2^m

Sectional area average 3.1^ϕ = 7.548

" 2.5^ϕ = -4.909

mean

$2.639 \times \times 1.04 = 2.745$ cub. meters.

Total vol. of shell concrete = 14.176 cub. meters.

Top filling 1:2:4 mix.

$2.6^\phi = 5.309 \times 1.0 = 5.309^{m^3}$

Bottom filling "

$2.5^\phi = 4.909 \times .9 = 4.415$

$2.742^\phi = 5.90 \times .21 = 1.239$

$3.2^\phi = 8.042 \times .09 = .724$

$6.\overset{37}{\cancel{73}}8^{m^3}$

Sand filling $2.4^\phi = 4.524 \times 3.8 = 17.190^{m^3}$

Summary for concrete (one well)

Concrete for	for one well		for 2 wells.	
Shell	14.176	× 2	= 28.352	
top filling	5.309	× 2	= 10.618	23.374
bottom filling	6.378	× 2	= 12.756	
	25.863^{m^3}		51.726^{m^3}	
Sand filling	17.190^{m^3}	× 2	= 34.380 "	

Forms.

Top 1.0^m of shell

Total length of perimeter outside 3.0^ϕ = 9.42

inside 2.6^ϕ = 8.17

$17.59 \times 1.0 = 17.59^{\square m}$

Intermediate 3.8^m

outside 3.0^ϕ = 9.42

inside 2.4^ϕ = 7.54

$16.96 \times 3.8 = 64.40^{\square m}$

Bottom 1.2^m outside $3.1^\phi = 9.74 \times .9 = 8.76$

inside $2.5^\phi = 7.85 \times .9 = 7.07$

" $2.742^\phi = 8.61 \times .21 = 1.81$

Bottom area of top fill 2.4^ϕ = 4.52

$104.15^{\square m}$ for one well.

Material List of Eian-Bashi for Okayama Prefecture.

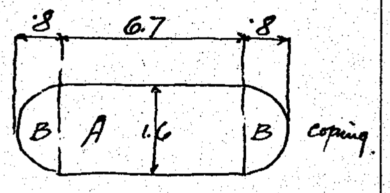
Reinforcements plain Bars

Shaft 1.0267 kgtons.
well 2 @ .60225 = 1.2045
2.2312 kgtons for one pier

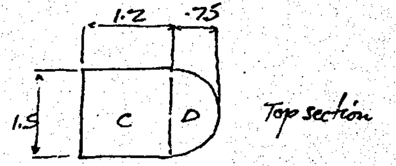
Reinforcements same as for Piers P1, P2, P3, P4, P5, + P6.

Structural steel for Curb Shoes.
0.4267 kgtons for one shoe.

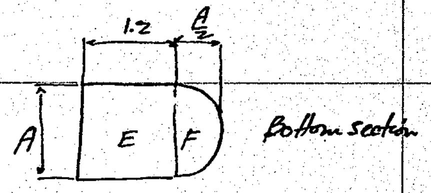
Materials for Piers P7, P8, P9 + P10.
Concrete 1:2:4 mixture.
Shaft.



Coping rectangle A $1.6 \times .3 \times 6.7 = 3.22$
circular ends B $1.6^{\phi} \times .3 = .60$
3.82^{m^3}

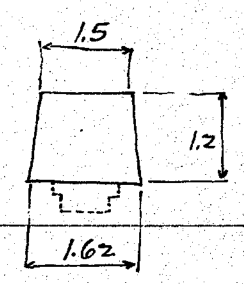


Shaft. Top section.
rectangle C $1.2 \times 1.5 \times 2 = 3.600$
Circular ends D. $1.5^{\phi} = 1.767$
5.367^{om}



Bottom section

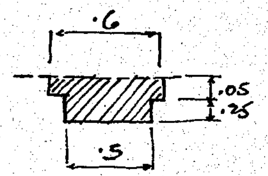
Piers	width A	2×area E	2×area F	Bottom area.
P7+P8	2.048 ×1.2×2 =	4.915	+ 3.300 =	8.215
P9	2.038 " =	4.891	3.268	8.159
P10.	2.019 " =	4.846	3.205	8.051



Top Beam. $\frac{1.5+1.62}{2} = 1.56 \times 1.2 = 1.872^{om} \times 4.3 = 8.050$ cub meters.

Rib inside of columns + bottom of top beam.

Piers.	Sectional area	Length.	volume
P7+P8	.155 ×	2×3.895 + 4.26 = 12.056	1.868
P9	" ×	2×3.798 + " = 11.856	1.838
P10	" ×	2×3.605 + " = 11.470	1.778



Sectional area of rib.
$.6 \times .05 = .03$
$.25 \times .5 = .125$
$.155^{om}$

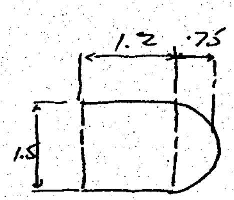
Total Volume of concrete for Shaft.

piers	Top area	Bott. area	average	height	vol. Shaft.	Coping	Beam	Rib	Total.
P7+8	5.367	8.215	6.791 ×	5.475 =	37.181	3.820	8.050	1.868	50.919 $^{m^3}$
P9	"	8.159	6.763 ×	5.378 =	36.371	"	"	1.838	~~48.241~~ 50.790
P10	"	8.051	6.709 ×	5.185 =	34.786	"	"	1.778	48.434

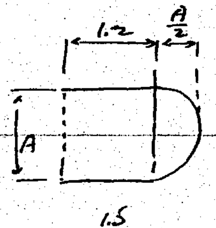
Girder seats for P7 only $2 @ .6 \times .83 \times .75 = .748^{m^3}$ Conc. for P7 = 50.919 + .748 = 51.667 $^{m^3}$
〃 18, P9, P10 平均 50.05

Forms for Shaft.

Coping Straight portions. $6.7 \times 2 = 13.40$
Circular ends. $1.6^{\phi} = 5.03$
$18.43^{m} \times .35 = 6.45$ sq. meters.

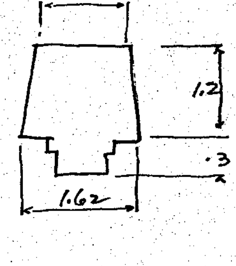


Shaft. top perimeter $1.2 \times 4 = 4.80$
$1.5^{\phi} = 4.71$
9.51^{m} for all piers 7, 8, 9, +10



Bottom perimeter

Piers	width A	Sides	Circular ends	total
P7+P8	2.048	1.2×4=4.8	+ 6.43 =	11.23 m
P9	2.038	4.8	+ 6.40 =	11.20 "
P10	2.019	4.8	+ 6.34 =	11.14 "



Top Beam
Sides $1.2 \times 2 = 2.4$
bottom $1.62 + .6 = 2.22$
$4.62 \times 4.3 = 19.88$ sq meters

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CALCULATIONS FOR

Material list of Eian-Bashi for Okayama prefecture.

Inside face of shaft.

Piers	top width	bottm width	average	projection	total	height	total area
P7+P8	1.658	2.048	1.853	.60	2.453	× 3.895 × 2 =	19.10
P9	〃	2.038	1.848	.60	2.448	× 3.798 × 2 =	18.60
P10	〃	2.019	1.839	.60	2.439	× 3.605 × 2 =	17.58

Total area of forms for Shaft.

Piers	top.	Bottom	average.	height	area	Coping	Top beam	Inside face	Total
P7+P8	9.51	11.23	10.37 ×	5.475 =	56.75 +	6.45 +	19.88 +	19.10 =	102.18 m²
P9	〃	11.20	10.36 ×	5.378 =	55.70 +	〃 +	〃 +	18.60 =	100.63
P10	〃	11.14	10.33 ×	5.185 =	53.60 +	〃 +	〃 +	17.58 =	97.51

For P7 only 2 girder seats (.6×2+.83×2).75 = 4.3 m²
form for P7 = 102.18+4.3 = 106.48 m²
〃 P8, P9, P10 平均 100.11

Reinforcements for shaft.

For Pier P7 Shaft. 1 e 1.0767 = 1.0767
Girder seats 2 e .0331 = .0662
1.1429 kg tons.

For Pier P8, P9, P10 Shaft 1.0767 〃

Materials for 3.5ϕ × 7.0 m well for Pier P.7.

Concrete 1:2:4 mixture.

Shell, top 1.2 meters
Sectional area 3.5ϕ = 9.621 □m
3.1ϕ = −7.548
2.073 × 1.2 = 2.488 cub. meters.

Shell, intermediate 4.3 meters
Sectional area. 3.5ϕ = 9.621 □m
2.9ϕ = −6.605
3.016 × 4.3 = 12.969 cub. meters

Shell, bottom 1.5 meters.
Sectional area, average. 3.6ϕ = 10.179 □m
3.0ϕ = −7.069
3.110 × 1.2 = 3.732
$3.\overset{7}{6}$ϕ = 10.752
3.24ϕ = −8.245
2.507 × .21 = .526
4.258 cub. meters

Total concrete for shell = 19.715 cub. meters

Top filling 3.1ϕ × 1.2 = 9.058 cub. meters
Bottom filling 3.0ϕ × 1.2 = 8.483
3.24ϕ × .21 = 1.731
3.7ϕ × .09 = .968
11.182
20.24

Total concrete for one well = 39.955 cub. meters

Sand filling 2.9ϕ × 4.3 = 28.40 cub. meters

Forms outside $3.\overset{5}{4}$ϕ × 5.5 = 60.48 □m.
〃 3.6ϕ × 1.2 = 13.57
inside 3.1ϕ × 1.2 = 11.69
〃 2.9ϕ × 4.3 = 39.18
〃 3.0ϕ × 1.2 = 11.31
〃 3.24ϕ × .21 = 2.14
bottom of top fill 2.9ϕ = 6.61
144.98 sq. meters. for one well.

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CALCULATIONS FOR

Material List of Eian-Bashi for Okayama Prefecture

Reinforcements plain Bars 1.7448 kg tons for 2 wells
or 0.8724 " " one well.

Structural Steel for curb shoe. 0.5037 kg ton. for one shoe.

Materials for $3.8^{\phi} \times 7.0^{m}$ well for piers P8, P9, + P10.

Concrete 1:2:4 mixture.

Shell, top 1.2 meters

Sectional area $3.8^{\phi} = 11.341^{\square m}$
$3.4^{\phi} = -9.079$
$2.262 \times 1.2 = 2.714^{m^3}$

Shell intermediate 4.3 meters

Sectional area $3.8^{\phi} = 11.341^{\square m}$
$3.2^{\phi} = -8.042$
$3.299 \times 4.3 = 14.186^{m^3}$

Shell bottom 1.5 meters

Sectional area $3.9^{\phi} = 11.946$
$3.3^{\phi} = -8.553$
$3.393 \times 1.2 = 4.072^{m^3}$

$4.0^{\phi} = 12.566$
$3.54^{\phi} = -9.842$
$2.724 \times .21 = .572^{m^3}$

Total vol. of shell = 21.544 cub meters

Top filling $3.4^{\phi} = 9.079 \times 1.2 = 10.895$
Bottom filling $3.3^{\phi} = 8.553 \times 1.2 = 10.264$
$3.54^{\phi} = 9.842 \times .21 = 2.067$
$4.0^{\phi} = 12.566 \times .09 = 1.131$
13.462

24.357
~~38.71~~

Total Concrete for one well = 45.901 cub meters.

Sand filling $3.2^{\phi} = 8.042 \times 4.3 = 34.581$

Forms:

outside $3.8^{\phi} = 11.94 \times 5.5 = 65.67^{\square m}$
" $3.9^{\phi} = 12.25 \times 1.2 = 14.70$
inside $3.4^{\phi} = 10.68 \times 1.2 = 12.82$
" $3.2^{\phi} = 10.05 \times 4.3 = 43.22$
" $3.3^{\phi} = 10.37 \times 1.2 = 12.44$
" $3.54^{\phi} = 11.12 \times .21 = 2.34$
bottom area of top fill $3.2^{\phi} = 8.04$
159.23 sq. meters

Reinforcements, plain Bars. 1.8207 kg tons for 2 wells.
0.9104 " " 1 well.

Structural steel for Curb shoe. 0.5369 kg tons for one shoe.

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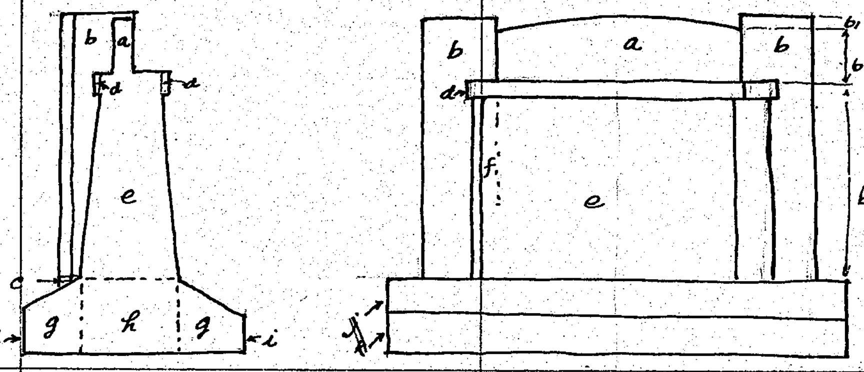
CALCULATIONS FOR

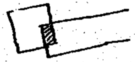
Material List of Eian Bashi for Okayama prefecture.

Materials for Abutment A1. (Girder span side)

Total Height = 9.1m (crown of R/w to bottom).
" width = 7.3m (Perpendicular to ℄ Bridge).



Concrete :- 1:2:4 mixture.

Names.	Section	Length	Volume	Reqd. no	Total volume	Remarks.	
parapet wall.	.3×1.49 ×	5.484 =	2.45	× 1 =	2.45 m³	a.	
Columns.	.95×.95 ×	7.493 =	6.76	× 2 =	13.52	b.	total length.
"	.2×.95 ×	.09 =	.02	× 1 =	.02	c	wedge shape, bottom right
"	.06×.8 ×	.03 =	.00	× 1. =	.00	c	" " left.
Coping	.05×.3 ×	12.88 =	.19	× 1 =	.19	d	total length. front + rear
Shaft.	1.48×6.09 ×	5.88 =	53.01	× 1 =	53.01	e	
less. column	.3×.57 ×	5.88 =	1.01	× 2 =	(−) 2.02	f	
Base	1.3×1.621 ×	8.2 =	17.28	× 2 =	34.56	g	footings
"	1.758×1.7 ×	8.2 =	24.50	× 1 =	24.50	h	center.

total concrete = 126.23 m³

Forms.

names.	Width	Length	reqd. no.	total area.		
parapet wall.	1.49 ×	5.484 ×	2 =	16.35 □m	a	front and rear faces.
Columns	.95×4 ×	.15 ×	2 =	1.14	b1	top.
"	$(.95\times\overset{3.5}{4-.3})$ ×	1.46 ×	2 =	10.22	b2	above coping
"	$(.95\times\overset{2.83}{4-.97})$ ×	5.88 ×	2 =	33.30	b3	under coping
"	.18 ×	1.2 ×	1 =	.22	c	bottom right col.
Coping	.35 ×	12.98 ×	1 =	4.55	d	total length front + rear
Shaft.	6.09 ×	5.58 ×	1 =	33.97	e	front.
"	.79 ×	5.58 ×	2 =	8.82	e	both sides
"	5.48 ×	5.58 ×	1 =	30.59	e	rear
Base	0.9 ×	8.2 ×	2 =	14.76	i	front + rear
"	1.3 ×	1.62 ×	4 =	8.43	g	both sides
"	1.7 ×	1.76 ×	2 =	5.98	h	"
				168.33 □m		

Reinforcements. Plain Bars. 3.0099 Kg.tons. see drawings.

Foundation Piles 内地産赤松 末口 18cm 長5.5m. ----- 40 piles for A1.

人造仕上.

Column.	outside	0.95×4.0 =	3.80
"	front.	1.2×1.6 =	1.92
"	Curb line	.95×.11 =	0.11
"	front	.65×2.4 =	1.56
"	top	$.95^2-.85^2$ =	.18
			7.56 □m × 2 = 15.12 □m for A1.

Excavation.

Bottom area of Base 8.2×5.0 = 41.0 □m
Mean depth of excavation = 6.0
Excavation = 6.0×41.0 = 246 cub meters.

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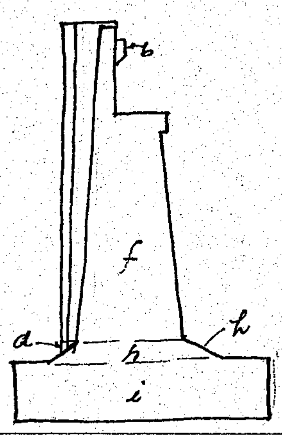
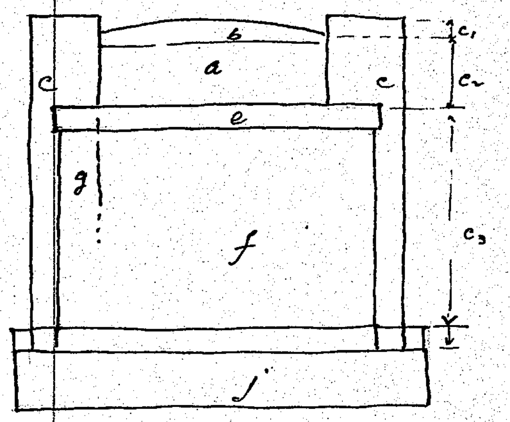
CALCULATIONS FOR

Material List of Eian Bashi for Okayama prefecture

Abutment A2. (Arch span side).
Total height 10.0 meters from crown of roadway to bottom.
Total width 7.9 " out to out of columns.





Concrete: 1:2:4 mixture.

Names.	Section		Length		Required. no			Total volume.	Remarks.
Parapet wall.	2.24 × .357	×	6.09	×	1	=		4.87 m^3	a
" Bracket	.15 × .5	×	6.09	×	1	=		.46	b
Columns.	.95 × .95	×	8.29	×	2	=		14.95	c
"	.95 × .32	×	.18	×	1	=		.05	d bottom of col. right
"	.95 × .2	×	.08	×	1	=		.02	d " left.
Coping	.3 × .05	×	9.60	×	1	=		.14	e total length
Shaft.	1.70 × 5.93	×	7.70	×	1	=		77.65	f
Less, column	.80 × .594	×	5.93	×	2	=	(−)	4.78	g
Base, upper layer	2.89 × .6	×	9.00	×	1	=		15.61	h
" , lower "	6.0 × 1.2	×	9.00	×	1	=		64.80	i
								173.77 m^3	

Reinforcements: Plain Bars. 2.6215 Kg. tons See drawings

Forms:_

Names.	width		Length		Reqd. no		Total area	Remarks.
Parapet wall.	2.24	×	6.09	×	2	=	27.30 $^{\square m}$	a.
" bracket bott.	.22	×	6.09	×	1	=	1.34	b.
Columns	.95 × 4	×	.15	×	2	=	1.14	c_1 above coping
"	$(.95 \times 4 - .36)$ 3.44	×	2.21	×	2	=	15.21	c_2 below "
"	$(.95 \times 4 - 1.41)$ 2.39	×	5.93	×	2	=	28.34	c_3
" bottom	.15	×	1.66	×	1	=	.25	d right col. bottom
" "	.10	×	1.25	×	1	=	.13	" left. " "
Coping	.35	×	9.7	×	1	=	3.40	e
Shaft.	7.70	×	5.63	×	1	=	43.35	f front face
"	6.09	×	5.93	×	1	=	36.12	" rear "
"	1.14	×	5.63	×	2	=	12.84	" both sides.
Base.	1.10	×	9.00	×	2	=	19.80	h front + rear
"	2.89	×	.60	×	2	=	3.47	" both ends.
"	1.20	×	6.00	×	2	=	14.40	i "
"	1.20	×	9.00	×	2	=	21.60	j front + rear.
							228.69 $^{\square m}$	

Foundation piles :- 内地産赤松末口 18 $^{cm\phi}$ 長 5.5 ^{m} — 54 piles required for A2.

. 人造 仕上

Column outside $0.95 \times 4.0 = 3.80$
" frontside $1.0 \times 2.37 = 2.37$
" " $0.1 \times 1.63 = 0.16$
" curb line $0.95 \times .11 = .11$
" top $.95^2 - .85^2 = .18$
$6.62^{\square m} \times 2 = 13.24^{\square m}$ for A_2.

Excavation. Bottom area of Base $= 6.0 \times 9.0 = 54.0^{\square m}$
average penetration $= 6.0^m$
Excavation $= 54 \times 6 = 324$ cub. meters.

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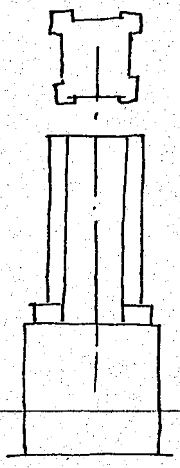
CALCULATIONS FOR

Material List of Eian Bashi for Okayama Prefecture.



Materials for Light pedestals.

Concrete 1:2:4 mixture.

Name	Section	Length	Reqd. no	Volume.
Shaft.	.65 × .65	× 1.5	× 1	= .63
less depression	.05 × .4	× 1.5	× 4	= (−) .12
base	.85 × .85	× 1.1	× 1	= .79
				1.30 cub m./pedestal.

Reinforcements, Plain Bars. 0.0592 ^{kg} tons./pedestal.

Forms.

name	width	Length	Reqd no	area.
Shaft.	.75 × 4	× 1.5	× 1	= 4.50
base	.85 × 4	× 1.1	× 1	= 3.40
				7.90 m^2
			miscellaneous moulding say	0.60
				8.50 m^2 for one pedestal.

Artificial Granite finishing.

form area		= 8.50
top of shaft.	.65 × .65	= .42
〃 base.	.85 × .85 − .65 × .65	= .30
		9.22 m^2 for one pedestal.

Gas pipes for Electric wiring.

1 − gas pipe 7.6 ^{cm}φ × 1.25 ^{m}

1 − 〃 6.3 $^{"}$ × 4.00 ^{m}

Bronze Lamps.

Girder span	6	Brackets.	(2 lights)	wt. of Bronze	12. ^{kg}/bracket.	(3.2′)
Truss span	16	Lamps.	(1 light)	〃	7 〃	(1.9′)
Light pedestals.	4	pedestals	(4 lights)	〃	33 ^{kg}/pedestal	(8.8′)

Total bronze reqd.

6 @ 12 = 72.

16 @ 7 = 112

4 @ 33 = 132

316 ^{kg} (ss84.3′)

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CALCULATIONS FOR

Material List of Eian-Bashi for Okayama-Prefecture

		Handrails				
No	Description	Length		Unit wts	Weights	Remarks
80	cast iron	Length	@	49.0 lbs.	3,920.0	Grate G1
158	〃		@	49.0	7,742.0	〃 G2
94	〃		〃	〃	4,606.0	〃 G3
2	〃		〃	53.4	106.8	〃 G4
6	〃		〃	18.5	111.0	〃 G5
16	〃		〃	32.1	513.6	〃 G6
16	〃		〃	27.6	441.6	〃 G7
16	〃		〃	29.6	473.6	〃 G8
16	〃		〃	31.9	510.4	〃 G9
16	〃		〃	29.8	476.8	〃 G10
2	〃		〃	11.1	22.2	〃 G11
2	〃		〃	21.6	43.2	〃 G12
2	〃		〃	11.3	22.6	〃 G13
6	〃		〃	48.8	292.8	〃 G14
6	〃		〃	49.2	295.2	〃 G15
316	Gas pipe 63φ(2½")	1.650		8.6	4,484.0	Top rail T1
16	〃	2.155		〃	296.5	〃 T2
2	〃	2.270		〃	39.0	〃 T3
6	〃	2.065		〃	106.6	〃 T4
8	〃	1.535		〃	105.6	〃 T5
16	〃	1.545		〃	212.6	〃 T6
16	〃	1.500		〃	206.4	〃 T7
2	〃	2.165		〃	37.2	〃 T8
2	〃	0.320		〃	5.5	〃 T9
6	〃	1.540		〃	79.5	〃 T10
6	〃	1.670		〃	86.2	〃 T11
2	cast iron		@	31.6	63.2	Post P1
278	〃		〃	〃	8,784.8	〃 P2
6	〃		〃	38.8	232.8	〃 P3
104	〃		〃	29.5	3,068.0	〃 P4
6	〃		〃	47.3	283.8	〃 P5
1	〃		〃	37.7	37.7	〃 $P6^R$
1	〃		〃	〃	37.7	〃 $P6^L$
3	〃		〃	121.3	363.9	Lamp past LP^R
3	〃		〃	〃	[illegible]	〃 LP^L
6	Gas pipe 114φ(4½")		〃	18.7	112.2	〃
6	〃 76φ(3")		〃	11.3	106.4	〃
6	Cast iron		〃	2.0	12.0	〃 Cap
16	〃		〃	6.1	97.6	Collar
16	〃		〃	〃	〃	〃
					38,898.5 Kgs	
4,514	Screws	8φ		@.018	81.2	
1,572	Set screws	〃		〃.020	31.4	
128	Tapped bolts	10φ×.050		〃.046	5.9	
128	Adjusting washers			〃.071	9.1	
208	Bolts	9φ×.065		〃.049	10.2	
208	Adjusting washers			〃.261	54.3	
104	Pl	50×8	.150	3.140	49.0	
692	Anchor bolts	16φ	.225	@.045	31.1	
294	Anchor Pl.	60×8	200	3.768	221.6	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Material List of Eian-Bashi for Okayama-Prefecture

104	Anchor Pl.	60×8	.100	3.768	39.2	
24	Bolts	19$^\phi$	.040	@ .270	6.5	
332	Bars	50×13	1.640	5.103	2,778.5	for G1,2,3
2	〃	〃	1.980	〃	20.2	〃 G4
6	〃	〃	.510	〃	15.6	〃 G5
16	〃	〃	.470	〃	38.4	〃 G6
16	〃	〃	1.480	〃	120.8	〃 G7
16	〃	〃	.970	〃	79.2	〃 G8
16	〃	〃	.400	〃	32.7	〃 G9
16	〃	〃	1.450	〃	118.4	〃 G10
2	〃	〃	.240	〃	2.4	〃 G11
2	〃	〃	.610	〃	6.2	〃 G12
2	〃	〃	.260	〃	2.7	〃 G13
6	〃	〃	1.530	〃	46.8	〃 G14
6	〃	〃	1.660	〃	50.8	〃 G15
					3,852.7 Kgs.	

Total wt. of Handrail	Gaspipes + Castings	38.8985
	Bolts, screws &c	3.8522
		42.7507 kg tons

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CALCULATIONS FOR

Material list for Yuian-Bashi, Okayama Ken.

/

No.	Description.	Section in Mm.	Length in Mm.	Wt. of One Meter.	Wt. of Main Section in Kgs.	Wt of Details in Kgs.	Total wts.	Remarks.
			Main girder	MG 1 R/L	2 Req'd.			
2	Web. Pls.	1,200 x 9	9,165	84.78	1,554.0			提出原図 / 原稿
8	Flg. Ls.	150x150x11	9,170	24.95	1,830.3			
4	Cov. Pls.	340 x 13	5,590	34.697	775.8			
8	End Stif Ls.	125x90x10	1,188	16.09		152.9		
8	Fills.	90 x 11	905	7.772		56.3		
26	Stif. Ls.	125x75x10	1,210	14.91		469.1		
2	〃 〃	〃	1,188	〃		35.4		
2	Fills.	75 x 11	905	6.476		11.7		
1	Stif. Ls.	100x75x10	1,158	12.95		15.5		
1	〃 〃	125x75x10	1,158	14.91		17.3		
2	Fills.	75 x 15	880	8.831		15.5		
4	Spl. Ls.	165 x 11	930	14.248		53.0		
2	〃 〃	480 x 11	566	41.448		46.9		
2	〃 〃	340 x 13	1,080	34.697		74.9		
4	Spl. Ls.	150x140(Cut150x150)x15	930	33.55		124.8		
2	Pls.	225 x 13	325	22.961		14.9		
2	Sole. Pls.	370 x 19	530	55.186		58.5		
2	Ls.	90x60x 9	145	9.96		2.9		
4	〃	〃	295	〃		11.8		
2	〃	〃	220	〃		4.4		
2	〃	90 x 75 x 9	230	11.02		5.1		
2	Bed. Pls.	400 x 35	530	109.9		116.5		
2	Ls.	75x75x9	350	9.96		7.0		
6	Anchor Bolts.	32$^\phi$	700	@ 5.2		31.2		
6	Washers.	150 x 9	150	10.598		9.5		
					4,160.1 +	1,334.6 =	5,494.7	
							x 2	
							10,989.4	
			Floor beam	FB 1	4 Req'd.			
1	Web. Pl.	670x 8	4,830	42.076	203.2			
2	Flg. Ls.	75x75x9	4,830	9.96	96.2			
2	〃 〃	〃	4,570	〃	91.0			
2	End Stif Ls.	130x90(Cut150x90)x9	520	16.32		17.0		
8	Stif Ls.	75x65x8	680	8.28		45.0		
2	Pls.	320 x 9	340	22.608		15.4		
					390.4 +	77.4 =	467.8	
							x 4	
							1,871.2	
				FB 2	1 Req'd.			
1	Web. Pl.	670 x 8	4,800	42.076	202.0			
2	Flg. Ls.	75x75x9	4,800	9.96	95.6			
2	〃 〃	〃	4,540	〃	90.4			
2	End Stif Ls.	140x90(Cut150x90)x9	510	16.32		16.6		
8	Stif Ls.	75x65x8	680	8.28		45.0		
2	Pls	320 x 9	340	22.608		15.4		
					388.0 +	77.0 =	465.0	
							x 1	
							465.0	

CALCULATIONS FOR ,

Material list for Yeian Bashi, Okayama-Ken

Stringer 1 Req'd.

2	Is	300x150 @ 48.34 Kgs	4,775		461.6		S1
2	"	"	4,775		459.7		S2
4	"	"	4,390		848.9		S3
12	Connection. Pls.	230 x 9	320	16.25		62.4	

1,770.2 + 62.4 = 1,832.6
x 1
1,832.6

Lateral bracing & Gusset Pls 1 Req'd.

4	Ls.	125x75x10	5,550	14.91	331.0		LB1
8	"	"	3,290	"	392.4		LB2
4	Pls.	270 x 9	510	19.076		38.9	
4	"	345 x 9	555	24.374		54.1	P1&P2
4	"	345 x 9	615	"		60.0	P3&P4
2	"	330 x 9	630	23.315		29.4	P5&P6

723.4 + 182.4 = 905.8
x 1
905.8

Weight of Rivet Heads for Girder spans. 7 Req'd.

3,860	Shop rivets heads	22^{ϕ}	@ 0.0964	372.1
990	Field " "	"	@ "	95.4
1,430	Shop " "	19^{ϕ}	@ 0.0646	92.4
780	Field " "	"	@ "	50.4

610.3
x 7
4,272.1 Kgs or 4.2721 Kg Tons

Summary of Wt. for 1 Girder span.

Maingirders	10,989.4
Floor beams	2,336.2
Stringers	1,832.6
Lateral bracings & Gussets	905.8
	16,064.0
Rivet heads	610.3
	16,674.3

Summary for 7 Spans.
16,674.3 x 7 = 116,720.1

Brackets for Lamppost 6 Req'd.

1	L.	75x75x9	460	9.96	4.6
1	"	"	370	"	3.7
1	Pl.	380 x 8	460	23.864	11.0

19.3
x 6
115.8

Grand Summary of Wt. for 7 girder spans (Rivet head ヲ含ム)

116,720.1
115.8
116,835.9 or 116.8359 Kg. tons

CALCULATIONS FOR

Material list for Yoian-Bashi, Okayama-Ken.

3

			Top Chord	TC 1	4 Reqd. Kgs		
1	Cov Pl.	560 x 8	6,650	35,168	233.9		
2	∟s.	150x100x9	6,750	17.020	229.8		
2	Pls.	250 x 9	4,900	17.663	173.1		
2	∟s.	150x100x12	6,250	22.410	280.1		
2	Pls.	250 x 9	3,980	17.663	140.6		
2	∟s.	90x70x9	1,740	11.020		38.3	
2	〃	〃	1,810	〃		39.9	
1	Pl.	315 x 9	1,740	22.255		38.7	
2	∟s.	75x75x9	304	9.960		6.1	
2	Fills.	75 x 9	165	5.299		1.7	
2	∟s.	90x90x10	1,185	13.340		31.6	
2	Fills.	90 x 4	210	2.826		1.2	
2	∟s.	150x100x12	1,580	22.410		70.8	
1	Tie Pl.	315 x 9	380	22.255		8.5	
2	∟s.	150x100x12	1,800	22.410		80.7	
1	Tie Pl.	315 x 9	380	22.255		8.5	
2	Pls.	2.150 x 9	2,550	151.898		5593	
2	〃	300 x 9	2,060	21.195		87.3	
2	〃	740 x 13	1,440	75.517		217.5	
4	Pin Pls.	380 x 13	545	38.779		84.5	
1	Tie Pl.	455 x 9	540	32.146		17.4	
2	Fills.	90 x 13	250	9.185		4.6	
4	Pls.	250 x 9	580	17.663		41.0	
4	〃	100 x 9	380	7.065		10.7	
4	∟s.	75x75x9	1,120	9.960		44.6	
2	Tie Pls	210 x 9	315	14.837		9.3	
3	Lac bars.	60 x 9	320	4.239		4.1	
4	∟s.	75x75x 9	1,860	9.960		74.1	
4	〃	〃	1,410	〃		56.2	
4	〃	〃	1,780	〃		70.9	
4	〃	〃	1,370	〃		54.6	428.3
4	〃	〃	1,710	〃		68.1	
4	〃	〃	1,330	〃		53.0	
4	〃	〃	1,290	〃		51.4	
2	Fills.	150 x 9	350	10.598		7.4	
2	Tie Pls.	160 x 8	275	10.048		5.5	
4	〃 〃	〃	290	10.048		11.7	
1	Tie Pl.	530 x 9	540	37.445		20.2	
3	Tie Pls.	305 x 9	〃	21.548		34.9	
4	Lac. bars.	70 x 11	690	6.045		16.7	
2	〃 〃	〃	730	〃		8.8	
4	Washers	70ϕ x 11		@ 0.332		1.3	
1	Pl.	560 x 11	640	48.456		31.0	
1	Fill.	320 x 3	560	7.536		4.2	
2	∟s.	150x100x12	720	22.410		32.3	
2	Pls.	250 x 9	480	17.663		17.0	
2	〃	100 x 9	320	7.065		4.5	
2	Fills	130 x 3	360	3.063		2.2	
2	〃	80 x 3	〃	1.884		1.4	
2	∟s.	150x100x15	720	27.670		39.8	
2	Pls.	250 x 9	480	17.663		17.0	
2	〃	100 x 9	320	7.065		4.5	
1	Pl.	320 x 9	540	22.608		12.2	
2	Fills.	130 x 3	360	3.063		2.2	

CALCULATIONS FOR

Material list for Yeian Bashi, Okayama Ken 1

2	Fills.	80 x 3	360	1.884		1.4
1	L.	100x75x10	〃	12.950		4.7

1,057.5 + 2,115.5 = 3,173.0
x 4
12,692.0

TC 2 4 Req'd.

1	Cov. Pl.	560 x 11	5,500	48.456	266.5		
2	Ls.	150x100x12	5,500	22.410	246.5		
2	Pls	250 x 9	5480	17.663	193.6		
2	Ls.	150x100x15	5,600	27.670	309.9		
2	Pls	250 x 9	5600	17.663	197.8		
4	Ls.	75x75x 9	1,580	9.960		62.9	
4	〃	〃	1,260	〃		50.2	
4	〃	〃	1,530	〃		61.0	
4	〃	〃	1,230	〃		49.0	476.8
4	〃	〃	1,470	〃		58.6	
4	〃	〃	1,200	〃		47.3	
4	〃	〃	1,410	〃		56.2	
4	〃	〃	1,160	〃		46.2	
4	〃	〃	1,140	〃		45.4	
8	Tie Pls.	160 x 8	290	10.048		23.3	
4	〃 〃	230 x 9	540	16.250		35.1	
1	Tie Pl.	305 x 9	〃	21.548		11.6	
12	Lac bars.	70 x 11	690	6.045		50.1	
2	〃 〃	〃	730	〃		8.8	
8	Washers.	70$^{\phi}$ x 11		@ 0.332		2.7	
1	Pl.	560 x 11	640	48.456		31.0	
2	Ls	150x100x15	720	27.670		39.8	
2	Pls.	250 x 9	480	17.663		17.0	
2	〃	100 x 9	320	7.065		4.5	
2	Ls.	150x100x15	720	27.670		39.8	
2	Pls.	250 x 9	480	17.663		17.0	
2	〃	100 x 9	320	7.065		4.5	
1	Pl.	320 x 9	540	22.608		12.2	

1,214.3 + 774.7 = 1,989.0
x 4
7,956.0

TC 3 4 Req'd.

1	Cov Pl.	560 x 11	6,300	48.456	305.3		
2	Ls.	150x100x12	6,300	22.410	282.4		
2	Pls.	250 x 9	6,250	17.663	220.8		
2	Ls.	150x100x15	6,350	27.670	351.4		
2	Pls.	250 x 9	6,350	17.663	224.3		
4	Ls.	75x75x 9	1,300	9.960		51.8	
4	〃	〃	1,120	〃		44.6	
4	〃	〃	1,260	〃		50.2	
4	〃	〃	1,110	〃		44.2	504.2
4	〃	〃	1,220	〃		48.6	
4	〃	〃	1,100	〃		43.8	
4	〃	〃	1,180	〃		47.0	
4	〃	〃	1,090	〃		43.4	

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CALCULATIONS FOR

Material list for Yeian Bashi, Okayama-Ken

5

4	Ls.	75x75x9	1,080	9.960		43.0	
4	〃	〃	1,130	〃		45.0	
4	〃	〃	1,070	〃		42.6	
10	Tie Pls.	160 x 8	290	10.048		29.1	
6	〃 〃	230 x 9	540	16.250		52.7	
1	Tie Pl.	290 x 9	540	20.489		11.1	
12	Lac bars.	70 x 11	660	6.045		47.9	
4	〃 〃	〃	730	〃		17.7	
10	Washers	70ϕ x 11		@ 0.332		3.3	
1	Pl.	560 x 11	640	48.456		31.0	
2	Ls.	150x100x15	720	27.670		39.8	
2	Pls.	250 x 9	480	17.663		17.0	
2	〃	100 x 9	320	7.065		4.5	
2	Ls.	150x100x15	720	27.670		39.8	
2	Pls.	250 x 9	480	17.663		17.0	
2	〃	100 x 9	320	7.065		4.5	
1	Pl.	320 x 9	540	22.608		12.2	
					1,384.2 +	831.8	= 2,216.0
							x 4
							8,864.0
			TC4	4 Req'd.			
1	Cov Pl.	560 x 11	6,150	48.456	298.0		
2	Ls.	150x100x12	6,150	22.410	275.6		
2	pls.	250 x 9	6,100	17.663	215.5		
2	Ls.	150x100x15	6,150	27.670	340.3		
2	Pls	250 x 9	6,100	17.663	215.5		
4	Ls.	75x75x9	1,100	9.960		43.8	
16	〃	〃	1,070	〃		170.5	
4	〃	〃	1,050	〃		41.8	463.6
8	〃	〃	1,060	〃		84.5	
4	〃	〃	1,010	〃		40.2	
4	〃	〃	1,000	〃		39.8	
4	〃	〃	1,080	〃		43.0	
10	Tie Pls.	160 x 8	290	10.048		29.1	
6	〃 〃	230 x 9	540	16.250		52.7	
8	Lac bars.	70 x 11	660	6.045		31.9	
6	〃 〃	〃	730	〃		26.5	
10	Washers.	70ϕ x 11		@ 0.332		3.3	
1	Tie Pl.	290 x 9	540	20.489		11.1	
1	P.l.	560 x 11	640	48.456		31.0	
2	Ls.	150x100x15	720	27.670		39.8	
2	Pls.	250 x 9	480	17.663		17.0	
2	〃	100 x 9	320	7.065		4.5	
2	Ls.	150x100x15	720	27.670		39.8	
2	Pls.	250 x 9	480	17.663		17.0	
2	〃	100 x 9	320	7.065		4.5	
1	Pl.	320 x 9	540	22.608		12.2	
					1,344.9 +	784.0	= 2,128.9
							x 4
							8,515.6

CALCULATIONS FOR

Material list for Yuian Bashi, Okayama-Ken

6

TC5 2 Req'd.

1.	Cov. Pl.	560 ×11	4,950	48.456	239.9	
2	∟s.	150×100×12	4,950	22.410	221.9	
2	Pls.	250 × 9	4,900	17.663	173.1	
2	∟s.	150×100×15	5,200	27.670	287.8	
2	Pls.	250 × 9	〃	17.663	183.7	
8	∟s.	75×75×9	990	9.960		78.9
8	〃	〃	980	〃		78.1
16	〃	〃	1,080	〃		172.1
8	Tie Pls.	160 × 8	290	10.048		23.3
6	〃 〃	230 × 9	540	16.250		52.7
8	Lac bars.	70 × 11	730	6.045		35.3
8	Washers.	70$^{\phi}$ × 11		@ 0.332		2.7

} 329.1

1,100.4 + 443.1 = 1,549.5
2
3,099.0

Summary for Top chord. 41,126.6

TIE T1. 4 Req'd.

2	[s.	300×90@38.13 Kgs.	4,870		371.4	
2	Pls.	240 × 9	4,200	16.956	142.4	
2	Tie Pls.	290 × 9	370	20.489		15.2
3	〃 〃	220 × 9	290	15.543		13.5
1	Pl.	190 × 9	385	13.424		5.2
2	Pls.	300 × 10	765	23.550		36.0
2	〃	240 × 10	465	18.840		17.5
2	〃	290 × 10	755	22.765		34.4
4	∟s.	90×75×9	240	11.020		10.6
1	Pl.	180 × 8	250	11.304		2.8

513.8 + 135.2 = 649.0
× 4
2,596.0

T 2 4 Req'd.

2	[s.	300×90@38.13 Kgs.	8,340		736.0	
2	Pls.	240 × 9	8,340	16.956	282.8	
8	Tie Pls.	220 × 9	290	15.543		36.1
1	Pl.	190 × 9	385	13.424		5.2
4	∟s.	90×75×9	240	11.020		10.6
1	Pl.	180 × 8	250	11.304		2.8
2	Pls.	300 × 10	765	23.550		36.0
2	〃	240 × 10	465	18.840		17.5
2	〃	290 × 10	755	22.765		34.4

1,018.8 + 142.6 = 1,161.4
× 4
4,645.6

T 3 4 Req'd.

2	[s.	300×90@38.13 Kgs.	6,570		501.0	
2	Pls.	240 × 9	6,570	16.956	222.8	

CALCULATIONS FOR

Material list for Yeian Bashi, Okayama-Ken

4	Tie Pls.	220 × 9	290	15.543		18.0	
2	Pls.	190 × 9	385	13.424		10.3	
8	Ls.	90x75x9	240	11.020		21.2	
2	Pls.	180 × 8	250	11.304		5.7	
2	〃	300 × 10	765			36.0	
2	〃	240 × 10	465			17.5	
2	〃	290 × 10	755			34.4	
					7238 +	143.1 =	866.9
							× 4
							3,467.6
			T4		2 Req'd.		
2	[s.	Kgs 300x90x38.13	8490		647.4		
2	Pls.	240 × 9	8490	16.956	287.9		
8	Tie Pls.	220 × 9	290	15.543		36.1	
1	Pl.	190 × 9	385	13.424		5.2	
4	Ls.	90x70x9	240	11.020		10.6	
1	Pl.	180 × 8	250	11.304		2.8	
					9353 +	54.7	990.0
							× 2
							1,980.0
			Summary for Tie		12,689.2		
			VERTICAL	V1.	4 Req'd.		
2	Ls	90x75x9	3,680	11.020	81.1		
2	〃	〃	3,745	〃	82.5		
1	Pl.	290 × 8	3600	18.212	65.6		
2	Fills.	190 × 9	440	13.424		11.8	
2	〃	〃	480	〃		12.9	
					229.2 +	24.7 =	253.9
							× 4
							1,0156
			V 2		4 Req'd.		
2	Ls	90x75x9	5,920	11.020	130.5		
2	〃	〃	5,970	〃	131.6		
2	pls.	290 × 8	2,915	18.212	106.2		
2	Fills.	190 × 9	400	13.424		10.7	
2	〃	〃	450	〃		12.1	
2	pls.	145 × 8	285	9.106		5.2	
					3683 +	28.0 =	396.3
							× 4
							1,5852
			V 3		4 Req'd.		
2	Ls.	90x75x9	7,520	11.020	165.7		
2	〃	〃	7,555	〃	166.5		
2	Pls.	290 × 8	3,690	18.212	134.4		
2	Fills.	190 × 9	370	13.424		9.9	

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2	Fills.	190 × 9	400	13.424			10.7			
2	Pls.	145 × 8	285	9.106			5.2			
					466.6	+	25.8	=	492.4	
								×	4	
									1,969.6	
			V 4	4 Req'd.						
2	Ls.	90×75×9	8,490	11.020	187.1					
2	〃	〃	8,510	〃	187.6					
2	Pls.	290 × 8	4,215	18.212	153.5					
4	Fills.	190 × 9	370	13.424			19.9			
2	Pls.	145 × 8	285	9.106			5.2			
					528.2	+	25.1		553.3	
								×	4	
									2,213.2	
			V 5	2 Req'd.						
4	Ls.	90×75×9	8,800	11.020	387.9					
2	Pls.	280 × 8	4,370	17.584	153.7					
4	Fills.	190 × 9	330	13.424			17.7			
2	Pls.	145 × 8	285	9.106			5.2			
					541.6	+	22.9	=	564.5	
								×	2	
									1,129.0	
			Summary for Verticals.		7,912.6					
			BOTTOM LATERAL BRACINGS.							
2	Ls.	150×90×9	6,510	16.320	212.5					BL 1
2	〃	〃	6,580	〃	214.8					〃
2	Pls.	510 × 9	830	36.032			59.8			〃
4	Ls.	150×90×9	3,680	16.320	240.6					BL 2
4	〃	〃	3,870	〃	252.6					BL 3
8	〃	〃	6,640	〃	866.9					BL 4
4	Pls.	510 × 9	690	36.032			99.4			〃
16	Ls.	150×90×9	3,870	16.320	1,010.5					BL 5
4	〃	150×100×9	6,710	17.020	456.8					BL 6
4	Pls.	380 × 9	580	26.847			62.3			〃
8	Ls.	150×100×9	3,960	17.020	539.2					BL 7
					3,793.8	+	221.5	=	4,015.3	
			BOTTOM LATERAL PLATES.							
2	Pls.	480 × 9	840	33.912			57.0			
2	〃	600 × 9	680	42.390			57.7			
2	〃	640 × 9	800	45.216			72.3			
2	〃	690 × 9	〃	48.749			78.0			
4	〃	640 × 9	780	45.216			141.1			
2	〃	〃	710	〃			64.2			
2	〃	650 × 9	680	45.923			62.5			
6	〃	570 × 9	650	40.271			157.1			
							689.9			

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CALCULATIONS FOR
Material list for Yeian-Bashi, Okayama, Ken 9

Summary for Bottom Lateral Bracings & Plates 4,705.2

TOP LATERAL BRACINGS.

4	Ls.	150x90x9	6,530	16.32	426.3		TB 1
4	"	"	3,800	"	248.1		TB 2
4	"	"	3,725	"	243.2		TB 2
4	"	"	6,420	"	419.1		TB 3
4	"	"	3,770	"	246.1		TB4
4	"	"	3,695	"	241.2		TB4
4	P/s.	590 x 9	710	41.684		118.4	
2	"	365 x 9	680	25.787		35.1	
2	"	"	830	"		42.8	
6	"	"	980	"		151.6	
8	Fills	180 x 25	305	35.325		86.2	
8	"	180 x 30	315	42.39		106.8	
					1,824.0	+ 540.9	= 2,364.9

Summary for Top Lateral Bracings 2,364.9

PORTAL BRACING PB 1 2 Req'd.

2	Ls.	150x90x9	5,550	16.32	181.2
2	"	100x90x10	"	14.13	156.8
2	"	"	7,250	"	204.9
5	P/s.	300 x 8	350	18.84	33.0
6	"	180 x 8	190	11.304	12.9
12	Ls.	65x65x8	1,170	7.66	107.5
2	P/s.	80 x 8	780	5.024	7.8
2	"	"	650	"	6.5
2	"	360 x 8	700	22.608	31.7
2	"	80 x 8	370	5.024	3.7
2	"	350 x 8	450	21.980	19.8
2	"	80 x 8	220	5.024	2.2
1	Pl.	350 x 8	350	21.980	7.7
10	Ls.	65x65x8	1,180	7.66	90.4
5	Washers.	60$^\phi$ x 8		22.19	0.9
2	P/s.	300 x 8	350	18.84	13.2
2	"	260 x 8	355	16.328	11.6
2	"	210 x 8	230	13.188	6.1
8	Ls.	125x90x10	265	16.09	34.1
2	Fills.	100 x 12	230	9.42	4.3
2	"	185 x 8	860	11.618	20.0
2	"	100 x 15	230	11.775	5.4

961.7
x 2
1,923.4

SWAY BRACING SB 1 2 Req'd.

4	Ls.	150x90x9	5,550	16.32	362.3
10	Fills.	145 x 8	180	9.106	16.4
12	Ls.	65x65x8	1,370	7.66	125.9
6	Fills.	60$^\phi$ x 26		22.19	3.5

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Materials list for Yeian-Bashi, Okayama Ken 10

8	Ls.	125x90x10	150	16.09		19.3	
2	Pls	155 x 8	370	9.734		7.2	
2	"	145 x 8	320	9.106		5.8	
2	"	210 x 8	230	13.188		6.1	
4	Ls.	100x90x10	2,150	14.13		121.5	
							668.0
							x 2
							1,336.0

SB 2 1 Req'd.

4	Ls.	150x90x9	5,550	16.32		362.3	
10	Fills.	145 x 8	200	9.106		18.2	
12	Ls.	65x65x8	1,350	7.66		124.1	
6	Fills.	60$^{\phi}$ x 26		22.19		3.5	
8	Ls.	125x90x10	135	16.09		17.4	
2	Pls.	155 x 8	370	9.734		7.2	
2	"	145 x 8	320	9.106		5.8	
2	"	210 x 8	230	13.188		6.1	
4	Ls.	100x90x10	2,150	14.13		121.5	
							666.1
							x 1
							666.1

Summary for Portal Bracings & Sway Bracings 3,925.5

FLOOR BEAM FB 1 2 Req'd.

1	Web Pl.	670 x 8	5,065	42.076	213.1		
4	Flg Ls.	90x90x10	5,560	13.34	296.7		
4	Ls.	125x90x10	770	16.09		49.6	
4	"	90x90x10	380	13.34		20.3	
2	Pls.	380 x 8	1,155	23.864		55.1	
4	Ls.	75x75x9	660	9.96		26.3	
8	"	"	680	"		54.2	
4	Pls.	100 x10	490	12.56		24.6	
4	"	150 x 9	255	10.598		10.8	
					509.8 +	240.9 =	750.7
							x 2
							1,501.4

FB2 & FB3 9 Req'd.

1	Web Pl.	670 x 8	5,065	42.076	213.1		
2	Flg Ls.	90x90x10	5,560	13.34	148.3		
2	" "	"	5,885	"	157.0		
4	Ls.	150x90x9	790	16.32		51.6	
2	Pls.	410 x 8	975	25.748		50.2	
4	Ls.	75x75x9	660	9.96		26.3	
8	"	"	680	"		54.2	
4	Pls.	100 x 10	490	12.56		24.6	
4	"	150 x 9	255	10.598		10.8	
					518.4 +	217.7 =	736.1
							x 9
							6,624.9

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CALCULATIONS FOR

Material list for Yeian-Bashi, Okayama Ken.

Summary for Floor Beams 8,126.3

STRINGERS

2	Is	300x150@48.34 kgs	5,385		520.6		S 1
2	"	"	5,350		517.2		S 2
2	"	"	5,320		514.3		S 3
2	"	"	5,285		511.0		S 4
32	"	"	4,990		7,718.9		S 5
72	Pls	230 x 9	320	16.25		374.4	

9,782.0 + 374.4 = 10,156.4

Summary for Stringer 10,156.4

Weight of Rivet Heads for Truss Spans. 4 Req'd.

2,240	shop rivet head	22ϕ	@ 0.0964	215.9	
9,410	Field " "	"	@ "	907.1	
29,840	Shop " "	19ϕ	@ 0.0646	1,927.7	
3,110	Field " "	"	@ "	200.9	
3,450	Shop " "	16ϕ	@ 0.0390	134.6	
					3,386.2 kgs
					4
					13,544.8 kgs.

SHOES

2	Cast steel shoe			@ 186.0	372.0		RS 1
4	Cast steel dust guard			@ 18.5	74.0		
2	Cast steel bed pls.			@ 168.0	336.0		
4	Dust guard pls.	135 x 8	680	8.478	23.1		
20	Tapped bolts.	7ϕ	025	@ 0.014	0.3		
8	Bolts.	22ϕ	085	@ 0.539	4.3		
8	Rollers.	100ϕ	610	61.650	300.9		
4	Pls.	70 x 13	400	7.144	11.4		
8	Pins.	25ϕ	065	@ 0.20	1.6		
8	"	"	045	@ 0.18	1.4		
2	Cast steel shoes.			@ 342.0	684.0		FS 1
4	Pins.	125ϕ	580	88.78	206.0		PN 1
8	Nuts.			@ 2.09	16.7		"
16	Anchor bolts	32ϕ	800	@ 5.75	92.0		AB 1
16	Pls	150 x 9	150	10.598	25.4		"
						2,149.1	

Summary of Wt. for 1- Truss span

Top chord.	41,126.6	
Tie.	12,689.2	
Verticals.	7,912.6	
Bottom lateral bracing & Plates.	4,705.2	
Top lateral bracings.	2,364.9	
Potal bracings and Sway bracings	3,925.5	
Floor beams.	8,126.3	
Stringers.	10,156.4	
		91,006.7
Shoes.	2,149.1	
Rivet heads.	3,386.2	
		96,542.0

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CALCULATIONS FOR

Material list for Yeian Bashi, Okayama-Ken

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[Materials for through the Bridge]	EXPANSION JOINT			EJ 1	1 Req'd.		
1	Checkered Pls.	251x9	5,545	19.978	110.8		
1	L.	75x65x8	5,535	8.28	45.8	A. 114.1	
1	"	"	5,520	"	45.7		
2	Pls.	175x8	370	10.99	8.1		
2	"	170x8	200	10.676	4.3		
2	"	170x8	210	"	4.5		
1	Pl.	170x8	215	"	2.3		
8	Bolts.	19ᶲ	55	@ 0.3131	2.5		
4	Beveld washers	60x8	60	3.768	0.9		
							224.9
							× 1
							224.9
			EJ 2		1 Req'd.		
1	L.	125x75x10	5,545	14.91	82.7		
1	"	75x65x8	5,520	8.28	45.7	B. 170.5	
1	bar.	50 x 11	5,510	4.318	23.8		
2	Pls.	165 x 8	370	10.362	7.7		
2	"	170 x 8	190	10.676	4.1		
2	"	"	200	"	4.3		
1	Pl.	"	205	"	2.2		
7	Anchor bolts.	19ᶲ	300	@ 0.84	5.9		
							170.4
							× 1
							170.4
			EJ 3		6 Req'd.		
1	checkered Pl.	227x9	5,540	18.068	100.1		
	A.				114.1		
							214.2
							× 6
							1,285.2
			EJ 4		6 Req'd.		
	B.				170.5		
8	Bolts.	19ᶲ	55	@ 0.3131	2.5		
4	Beveled Washer.	60x8	60	3.768	0.9		
							173.9
							× 6
							1,043.4
			EJ 5		3 Req'd.		
1	Checkered Pl.	236x9	5,540	18.784	104.1		
1	L.	75x65x8	5,535	8.28	45.8	C. 115.0	
1	"	"	5,520	"	45.7		
2	Pls.	175x8	370	10.99	8.1		
2	"	170x8	200	10.676	4.3		
2	"	170x8	210	"	4.5		
1	Pl.	"	215	"	2.3		
8	Bolts.	19ᶲ	55	@ 0.3131	2.5		
8	Beveled Washers.	60x8	60	3.768	1.8		

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CALCULATIONS FOR

Material list for Yuan Bashi, Okayama-Ken. 13

						219.1
						× 3
						657.3

EJ 6 3 Req'd.

1	L.	150×90×9	5,545	16.32	90.5	D. 177.2	
1	"	75×65×8	5,520	8.28	45.7		
1	Bar.	50 × 11	5,510	4.318	23.8		
2	Pls.	155 × 8	370	9.734	7.2		
2	"	170 × 8	180	10.676	3.8		
2	"	"	190	"	4.1		
1	Pl.	"	195	"	2.1		
8	Bolts.	19ᵠ	55	0.3181	2.5		
8	Beveled Washers	60×8	60	3.768	1.8		
							181.5
							× 3
							544.5

EJ 7 1 Req'd.

	C.				115.0	
1	Checkered Pl.	286 × 9	5,550	22.764	126.3	
						241.3
						× 1
						241.3

EJ 8. 1 Req'd.

	D				117.2	
7	Bolts	19ᵠ	300	@ 0.84	5.9	
						183.1
						× 1
						183.1

CONSTRUCTION JOINTS CJ 1 & CJ2 2 Req'd.

1	L.	75×65×8	5,965	8.28	49.4	
2	Pls.	170 × 9	2,740	12.011	65.8	
2	"	265 × 9	310	18.722	11.6	
1	Pl.	70 × 9	150	4.946	0.7	
8	Bolts.	19ᵠ	55	@ 0.3181	2.5	
6	Beveled washers.	60 × 8	60	3.768	1.4	
						131.4
						× 2
						262.8

EXPANSION JOINT FOR COPING

6	Pls.	370 × 8	607	23.236	84.6	
12	"	345 × 8	602	21.666	156.5	
2	"	420 × 8	615	26.376	32.4	
84	Anchor bolts.	16ᵠ	150	0.336	28.2	
84	Washers.	60 × 3	60	1.413	7.1	
2	Pls.	330 × 8	550	20.724	22.8	
						331.6

CALCULATIONS FOR

Material list for Yeian Bashi, Okayama Ken

14

Summary for Expansion & Construction Joints 4,950.5

Weight of Rivet Heads for Expansion Joints 1 Reqd.

2,180	Shop rivet head	19^{ϕ}	@ 0.0646	140.8
820	" " "	16^{ϕ}	@ 0.0390	32.0

172.8$^{\text{Kgs.}}$

Summary 13,717.6$^{\text{Kgs.}}$
or 13,717.6$^{\text{Kg Tons.}}$

Summary for Expansion joints & Construction joint.

Expansion & Construction joint	4,950.5
Rivet heads	172.8
	5,123.3 or 5.1233$^{\text{Kg tons}}$

Grand Summary of Weights (Rivet heads ヲ含ム)

	Kg. Tons
Girder for 7 Spans	116.8359
Truss Span 96,542.0×4	386.1680
Expansion & Construction	5.1233
	508.1272

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CALCULATIONS FOR

昭和四年二月

岡山縣吉井川架橋

永安橋予算設計予算書

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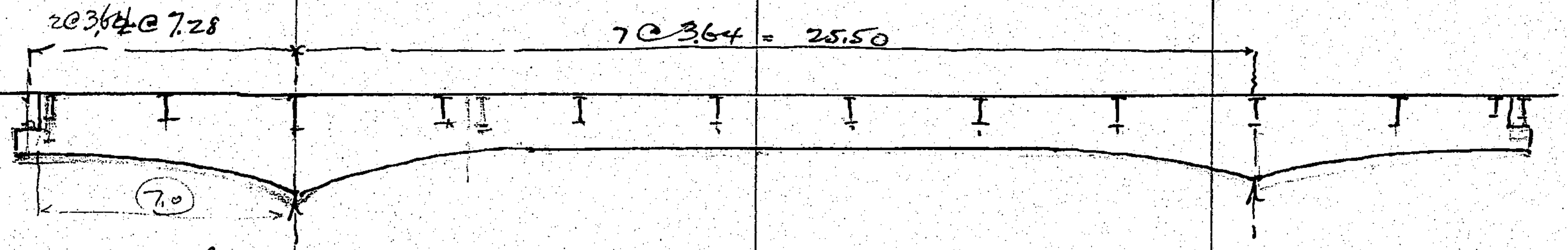
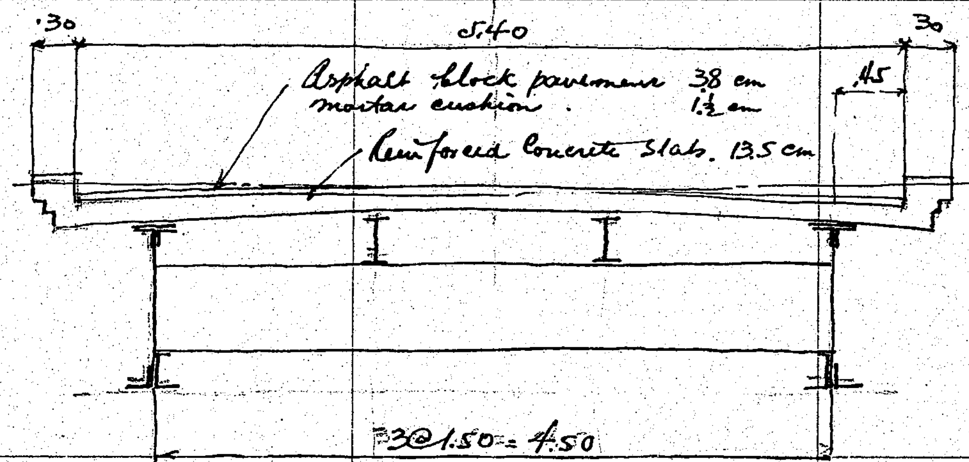
CALCULATIONS FOR

Preliminary Estimate of Eiau Bashi for Okayama-Ken.

Berger Träger span length 25.5 meters @ 7 @ 3.64 meters
Clear Roadway 5.4 meters.



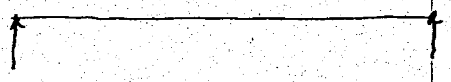
Cross section of bridge.



Stringer Span length 3.64 m spacing 1.5 meters.

Dead Load

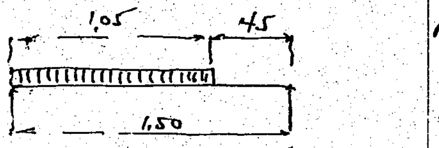
Asphalt block -	3.8 cm	@	~~2~~ 21.0	=	8.0
mortar cushion	1.5	@	22.0	=	33
Concrete Slab.	13.5	@	24	=	324
misc concrete Say					13
					450.0 kg per sq meter.

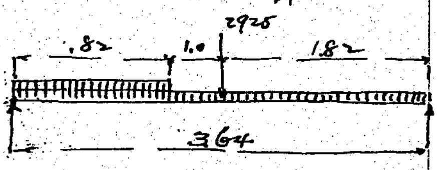


Dead Load ~~moment = 1/10 · 450 · 1.5²~~ = 450 · 1.50 = 675 kg.
stringer say. 50
725

Dead Load moment = $\frac{1}{8} \cdot 725 \cdot 3.64^2$ = 1200

Live load motor truck loading with impact = 2925 kg.
Uniform live load - 500 kg per square meter.
Reaction on A. $\frac{500 \cdot 1.05^2}{2 \cdot 1.50}$ = 184 — M_2
full load 500 · 1.5 750
566 kg. — M_1





moment due to motor truck $\frac{2925}{2} \cdot 1.82$ = 2660
M_2 $\frac{1}{8} \cdot 184 \cdot 3.64^2$ = 305
M_1 $\frac{566 \cdot .82^2}{2 \cdot 3.64} \cdot 1.82$ = 96
3061 kgm.
Dead Load moment 1200
Total moment 4261 kgm

Section modulus reqd = $\frac{426100}{1100}$ = 388.0

Use 250 · 125 · 75 @ 38.29 kg = 48788 o cm³
or try 300 · 150 · 8 @ 48.34 " =

Cross beam approximate weight. 500 kg per beam.

Bottom lateral bracings. 45 kg per lin meter.

2 stringers @ 50 =	100
Floor beam 500 ÷ 3.64 =	138
Lateral Bracing.	45
	283 kg per lin meter

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CALCULATIONS FOR

Preliminary Estimate of Eian Bashi for Okayama-ken

main girder.

Dead Load.

floor	450 × 5.40	=	2430
Copings	2@200	=	400
Handrails	2@60	=	120
			3050

Structural Steel.

Stringer Floor beam + laterals	283
Girders assumed 2@450 =	900
	1183

4233 ÷ 2 = 2116 kg per lin. meter.

Suspended span 11.52 meters

Dead Load moment = $\frac{1}{8} \times 2116 \times 11.52^2$ = 35000 kgm

shear = $\frac{1}{2} \times 2116 \times 11.52$ = 12200 kg.

Live Load 500 × 2.70 = 1350 kg

moment = $\frac{1}{8} \times 1350 \times 11.52^2$ = 22400 kgm

shear = $\frac{1}{2} \times 1350 \times 11.52$ = 7800 kg.

Concentration assumed 4000 kg. at center

moment = $2000 \times \frac{11.52}{2}$ = 11520

shear = 4000

Summary for moments and shears

	moment	shear
Dead Load	35000	12200
Live load	33920	11800
	68920 kgm	24000 kg

web assumed 1200 × 9 = 108.0 cm² 1/8 web = 13.5 cm² Back to back of ∠s = 121 cm

Effective depth = 112.9 cm Flange stress 6892000 ÷ 112.9 = 61000

section reqd = 61000 ÷ 1200 = 50.90

13.50

37.40 □cm net

try 2∠s 150 × 150 × 11 = 63.58 − 11.0 = 52.58 □cm net

Approximate weight of main girder (suspended span)

1 web pl.	1200 × 9	@	84.80	× 11.52	=	985.0
4∠s	150 × 150 × 11	@	24.95	× 11.52	=	1150.0
						2135.0
Details say		40%				855.
						2990 say 30.00 kg

3000 ÷ 11.52 = 260 kg per lin. meter.

Summary for suspended span

laterals, floor complete	283 ×	11.52	=	3260
main girders	2@3000		=	6000
misc details say				340
				9600 kg

Suspended span at abutment span length 18.50 meters between bearings

Dead Load moment = $\frac{1}{8} \times 2116 \times 18.50^2$ = 90200

shear = $\frac{1}{2} \times 2116 \times 18.50$ = 19500

Live Load. moment = $\frac{1}{8} \times 1350 \times 18.50^2$ = 57700

shear = $\frac{1}{2} \times 1350 \times 18.50$ = 12500

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CALCULATIONS FOR

Preliminary Estimate of Eian Bashi for Okayama-Ken.

Concentration say 4000 kg. at center of span
Live Load Conc. moment = 2000 × 9.25 = 18500 kgm
shear = 4000 kg.

Summary for moments and shears

	moments	shears
Dead Load	90200	19500
Live Load	76200	16500
	166400 kgm	36000 kg

Web assumed 1200 × 9 = 108.0 cm² 1/8 web = 13.5 cm² Back to back of Ls = 121.0 cm
Effective depth say 112.0 flange stress = 166400 ÷ 1.12. = 148,600 kg.
Section req'd = $\frac{148.600}{1200}$ = 124.0
13.5
110.5

2Ls 150 × 150 × 11 = 63.58 − 11.0 = 52.58
1Pl. 320 × ~~10~~ 11 = 35.20 − 5.5 = 29.70 14.0 m
1Pl 320 × ~~10~~ 11 = 35.20 − 5.5 = 29.70 10.0 m
111.98 □ cm net.

Approximate weight of main girder suspended span 18.5 meters

1 web. 1200 × 9 @ 84.80 × 18.7 = 1590.
4Ls 150 × 150 × 11 @ 24.95 × 18.7 = 1870
2Pls. 320 × 11 @ 27.60 × 14.0 = 772
2Pls. 320 × 11 @ 27.60 × 10.0 = 552
4784
Details say 35% 1680
6464 ÷ 18.7 = 346 kg per lin meter 6464 kg.

Total weight 18.5 meter suspended span

Floor and bottom laterals 283 × 18.7 = 5300.
Girders 2 @ 6464 = 12928
misc details say 500
18728 kg.

Anchor span span length 25.50 meters between piers
Over hang 7.00 meters.

Dead Load Cantilever arm. 1/2 × 2116 × 7² = 51800
Concentration 12200 ~~19500~~ × 7 = 85500 ~~136600~~
137300 ~~188400~~ kgm

Live Load. moment 1/2 × 1350 × 7² = 33100
Conc 11800 × 7 = 82600
115700
253000 kgm

Depth ~~req~~ of girder say 2000 × 9 = 180 1/8 web = 22.5 cm² back to back = 181. cm
Effective depth say 173. flange stress = 253000 ÷ 173 = 146000

section same as for 18.5 meter suspended span

Between piers
Dead Load. moment 1/8 × 2116 × 25.5² = 172.000
less cant. moment = 137300
34700 kg m

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CALCULATIONS FOR

Preliminary Estimate of Eian Bashi for Okayama-Ken

Live Load moment Unif. = 1/8 × 1350 × 25.50² = 110.000
Conc. = 2000 × 1275 = 25.500
135.500
Dead Load m 34.700
170200

section same as for 18.5 meter span
Approximate weight of anchor span say 370 kg per lin meter
Total weight - laterals + floor . 283
653
370
1023 × 39.5 = 40500 kg.

Summary for structural steel.
suspended span 5 @ 9.60 = 48.0
" " 2 @ 18.70 = 37.4
Anchor span 6 @ 40.50 = 243.0
328.4 call this 335 tons

Total length of bridge - 332.0 meters
area of pavement 332. × 5.4 = 1790 square meters

Concrete in slab. .135 × 5.4 = .073
Coping. .17
.90 × 332 = say 300. cubic meters.

Reinforcing steel in span 1790 × 23 kg = 41.2 tons
Forms 6.6 × 332 = 2190 square meters.

Handrails. 140 kg × 332 = 46.5 tons for both sides.

Approximate Estimate of Cost of Deck construction.
Concrete slabs. 300 cubic m @ 16.41 = 4920
Reinf. bars. 41.2 tons @ 160.00 = 6600
Forms. 2190 sq m @ 1.50 = 3290
pavement 1790 sq m @ 4.50 = 8050
Finish 660 sq m @ 4.00 = 2640
Handrails. 46.5 tons @ 330.00 = 15350
Pedestals drains &c 2000
Electric wirings 3000
45850

Structural steel 335 tons @ 230.00 = 77800
122850

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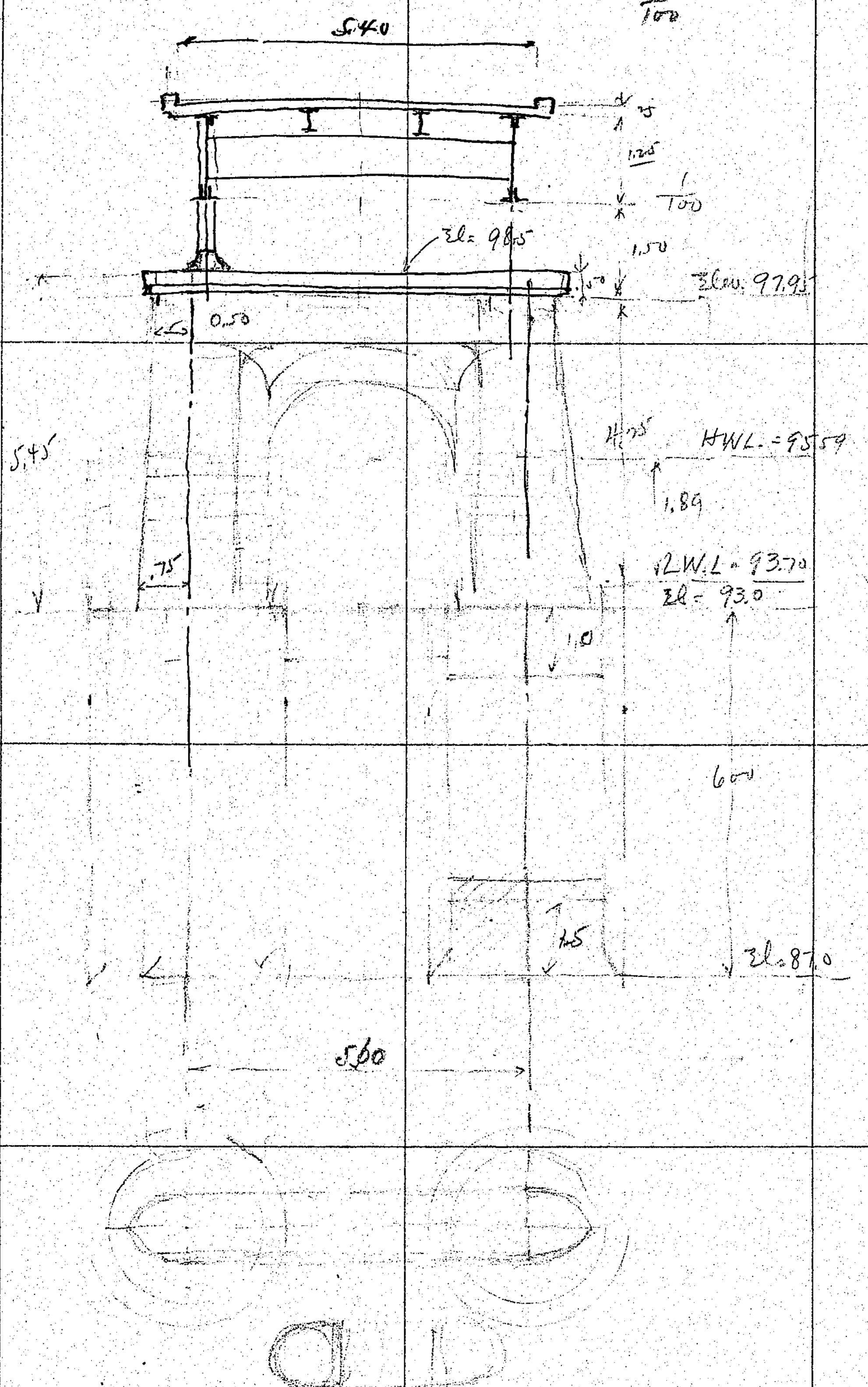
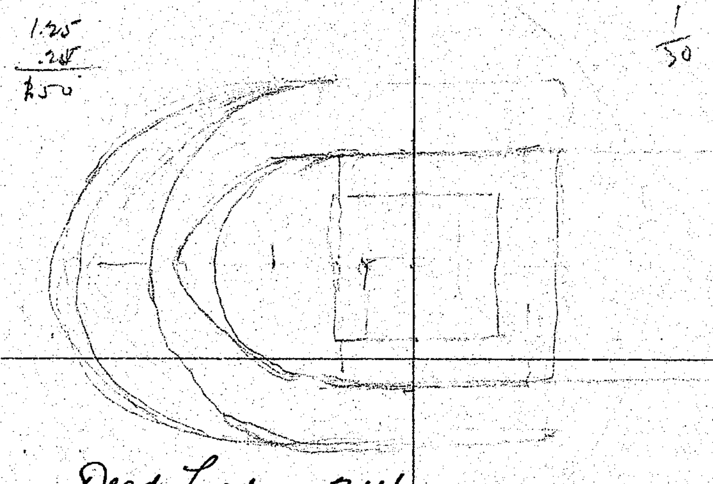
CALCULATIONS FOR

Preliminary Estimate of Cost Eian Bashi for Okayama-ken

Design of pier.





Dead Load 2116
Live Load. 1350
3466 × 25.5 = 88400 kg.

Bearing area = $\frac{88400}{30}$ = 2950 □cm

$\underline{35}$ cm^2

Load on pier say 2 @ 88400 = 176800
call this 180,000 kg

Coping 1.50^{ϕ} = 1.76
1.50 × 5.10 = 7.65
9.41 × .45 = 4.23

Shaft
top area = 1.0^{ϕ} = .78
1.0 × 5.10 = 5.10
5.88
bottom area 9.41
15.29 ÷ 2 = 7.64

vol = 7.64 × 5.00 = 38.20
4.23
42.43

Counting noise - say 45.0 cubic meters.

well. 3.0 meter dia.
area of Ring. 3^{ϕ} = 7.07
2.4^{ϕ} = 4.52
2.55 sq meter. × 6.0 = 15.30 2 @ 15.30 = 30.60

Top and bottom fill 1:2:4 concrete
2 - 4.52 × 2.5 = 22.6

sand fill.
2 - 4.52 × 3.5 = 31.6

Total Concrete shaft and coping 45.0
well - 30.6
filling 22.6
~~54.2~~
~~129.8~~ 98.2 @ 2400 = 236.000 ~~311.000~~
Sand 31.6 @ 1700 = 53.700
289.700
Superimposed load 180.000
469.700 kg.

Bottom area 3.10^{ϕ} = 7.55
15.10

Unit bear = $\frac{469.700}{15.10}$ = 31100

2.9 tons/□'

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CALCULATIONS FOR

Preliminary Estimate of cost Eian Bashi for Okayama-Ken.

area of forms.

shaft 2 @ 5.10 = $\frac{4.5 + 10.2}{14.7}$ × 5.5 = 81.0 sq meter.

well. Outside 9.4, inside 7.55, 2 - 16.95 × 6 = $\frac{204.0}{285.0}$ sq meter.

G.P. 4,100.
T.P. 5,000
Abt. 4,000

Concrete in shaft	45.00 @	17.50	=	788^{00}
Reinforcing bars.	1.7 ton @	130^{00}	=	221^{00}
forms.	81.0 sq @	2^{00}	=	162^{00}
				1171^{00} 1171

well.

Concrete shell	30.6 @	17.50	=	535	624
Concrete filling	22.6 @	"	=	396	396
Sand filling	31.6 @	50	=	16	18.3
Reinforcing bars	1.5 @	130^{00}	=	195	228
forms.	204.0 @	1.50	=	306	358
Curb shoe	0.8 @	230^{00}	=	184	184
				1632	1808

well sinking

Excavation.	400^{00}	433
[illegible]	660	660
[illegible]	140	140
	1120	1233

1200 1233
4003 ~~3041~~ 4211

4100
Call this ~~4200~~^{00}
Say 4,300.

45
20
22

2 abutments height say 9.5 meter cost say 4000^{00}

Piers	12 @ 4200	=	50400
abutment	2 @ 4000	=	8000
sum			58400

Substructures.	58400
Deck construction.	45850
Structural Steel.	77000
	181250

Concrete span over 4 [illegible] 17

2 abutments @	2000	=	4000^{00}
81 sq meters @	42		3400
			7400

25 [illegible]

300 10/17

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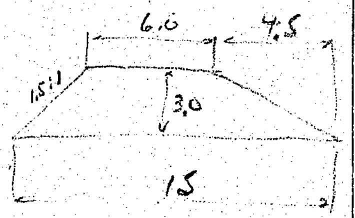
CALCULATIONS FOR

永安橋土工概算

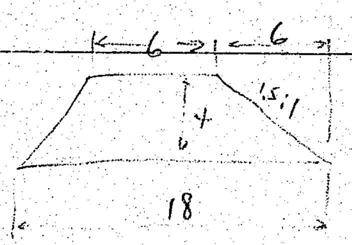
右岸 延長 80m 平均盛土高 3.0m



10.5 × 3 = 31.5 ㎡
盛土 31.5 × 80 = 2520 m^3

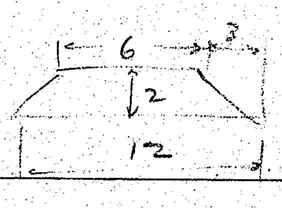
(10 × 80 = 800)

左岸 橋マデ 延長 92m 平均盛土高 4.0m



12.0 × 4 = 48.0 ㎡
盛土 48 × 92 = 4416 m^3

橋ヨリ遠方 延長 60m 平均盛土高 2.0m



9.0 × 2 = 18 ㎡
盛土 18 × 60 = 1080 m^3

盛土計 8016 ㎡
(or 1340 立坪)

7 × .. = 72

5w

路面椎 232m × 5.5 = 1,275 ㎡ or 385 坪
砂利 385 × 0.05 = 19.3 立坪 or 116 m^3

石垣 宮様内 70 × 3.5 = 245 m^2 or 74 面坪
右岸 say 55
300 ㎡ or 91 面坪

土羽 2000 ㎡ or 600 坪

Summary.

盛土	8,000	m^3	@ .8 =	~~6,400~~ 6,400	坪 6.0
表装砂利	120	m^3	@ 2 =	240	" 12.0
石垣	300	m^2	@ 6 4 =	~~1800~~	" 20.
土羽	2,000	㎡	@ .24 =	480	" 0.8

~~10,520~~ 円
8,320

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CALCULATIONS FOR

Preliminary Estimate of Cost. Eian-Bashi for Okayama-Ken

Total length of bridge - 332 meters Clear Roadway 5.4 meters

~~structural~~ Steel		335 tons	@ 230	=		77000	
Deck const							
Concrete Slab.		300 cubic meters	@ 16.41	=	4920		4920
Reinforcing Bars		41.2 tons	@ $\overset{135}{160.00}$	=	6600		5550
forms		2190 sq m	@ 1.50	=	3290		3290
Pavement	1480	1790 " "	@ 4.50	=	8050		6660
Finish	550	~~660~~ " "	@ 4.00	=	2640		2200
Handrails		46.5 tons	@ 330.00	=	15350		15350
Pedestals drains &c					2000		2000
Electric wiring					3000		2000
						45850	41970
Substructure							
abutments		2 @ 4000.00	=	8000			
Piers		12 @ 4200.00	=	50400			
						58400	
							181250

Reinforced Concrete Span over Senda gawa

abutments	2 @ 2000	=	4000
Span	81 sq m @ 42	=	3400

25 坪 @ 300.円

Fills &c on both approaches

7400
8320
~~10520~~
↓
Say 15700

15700
~~17920~~
199170

5.4
4.5

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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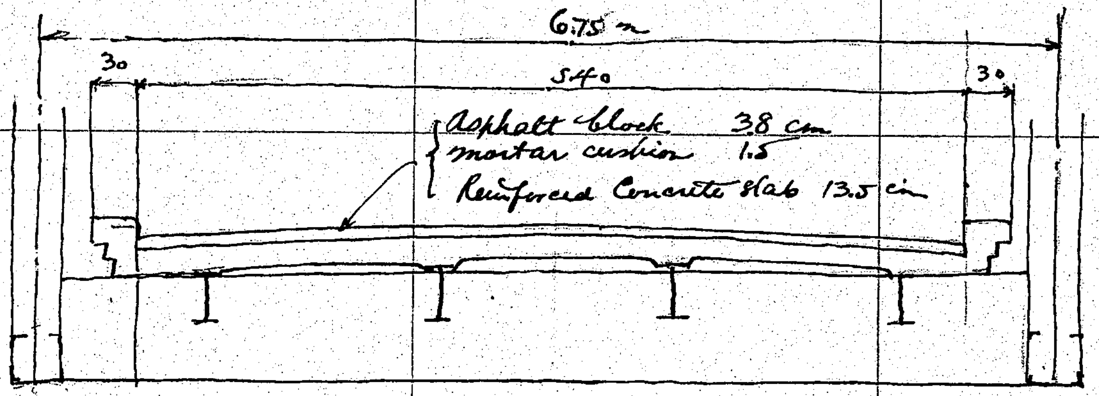
CALCULATIONS FOR

Preliminary Estimate of Cost Eian Bashi for Okayama-Ken : 3

Plate girder spans 10 @ 18.3 meters. (60 ft) = 183.0
3 @ 49.0 = 148.0
330.0 . +

49.0 meter truss span clear roadway 5.40 meters
Cross section of bridge.



Thickness of slab. 13.5 cm
stringer 300 × 150 @ 48.34 kg.
approximate weight = 4 @ 50 = 200 kg per lin meter.

Cross beam 120 @ 6.43 = 770 kg.
140 @ 6.43 = 900 kg.

9 @ 900 = 8100
2 @ 770 = 1540
9640 kg

Lateral Bracing. Bottom lateral. 4900 100 kg per lin meter

Top Lateral Bracings 7000 kg

Design of truss :—
structural steel in one span
stringers 4 @ 50 = 200
floor beam 900 ÷ 4.90 = 184
Lateral 4900 ÷ 49.0 $\frac{1}{10}$ = 100
Top lateral say 200
trusses assumed 1000
1684 — say 1700

Flooring coping and Handrails 3050
4750 ÷ 2 = 2375 kg.

Dead Load moment $\frac{1}{8} \times 2375 \times 49^2$ = 713000
Live Load. say $\frac{1}{8} \times 1350 \times 49^2$ = 405000
1,118,000

assumed section of ~~bot~~ top chord
1 cov pl 550 × 9 = 49.5
2[300 × 90 × 9 = 97.15
146.65

Height of truss = $\frac{1,118.000}{146650}$ = 7.6

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CALCULATIONS FOR

Preliminary Estimate of Cost Eian Bashi for Okayama-Ken

Height of truss say 8.0 meters.

stress in top chord 1.118.000 ÷ 8.0 = 140.000 140.0 gross

bottom chord 140.000 ÷ 1200 = 117.0 net

Bottom chord. member. 2[s 300 × 90 · 9 = 97.15 − 18.48 = 72.67

2Pls. 300 × 9 = 54.00 − 7.92 = 46.08

151.15 118.75

Approximate weight of one truss.

top chord 146.05

bottom chord 151.15

297.80 — × 50 × 0.785 = 11.700

Diagonals 88.14 @ .785 · 63. = 4350

verticals 66.00 @ .785 · 53.4 = 2770

18820

40% details 7500

26320

Approximate weight of structural steel in one span

stringers. 200 · 49.7 = 9920

floor beams 9640

bottom laterals. 4900

Top laterals. 7000

trusses. 2@26320 = 52640

Shoes say 2500

86600 $\frac{x}{x}$

1400

88000 tons

Girder span 18.3 meters.

same as for Anabuki − Bashi − 17.0 tons for one span.

Total structural steel.

truss spans. 3@ 88.0 = 264. tons

girder spans 10 @ 17.0 = 170.0

434.0 tons.

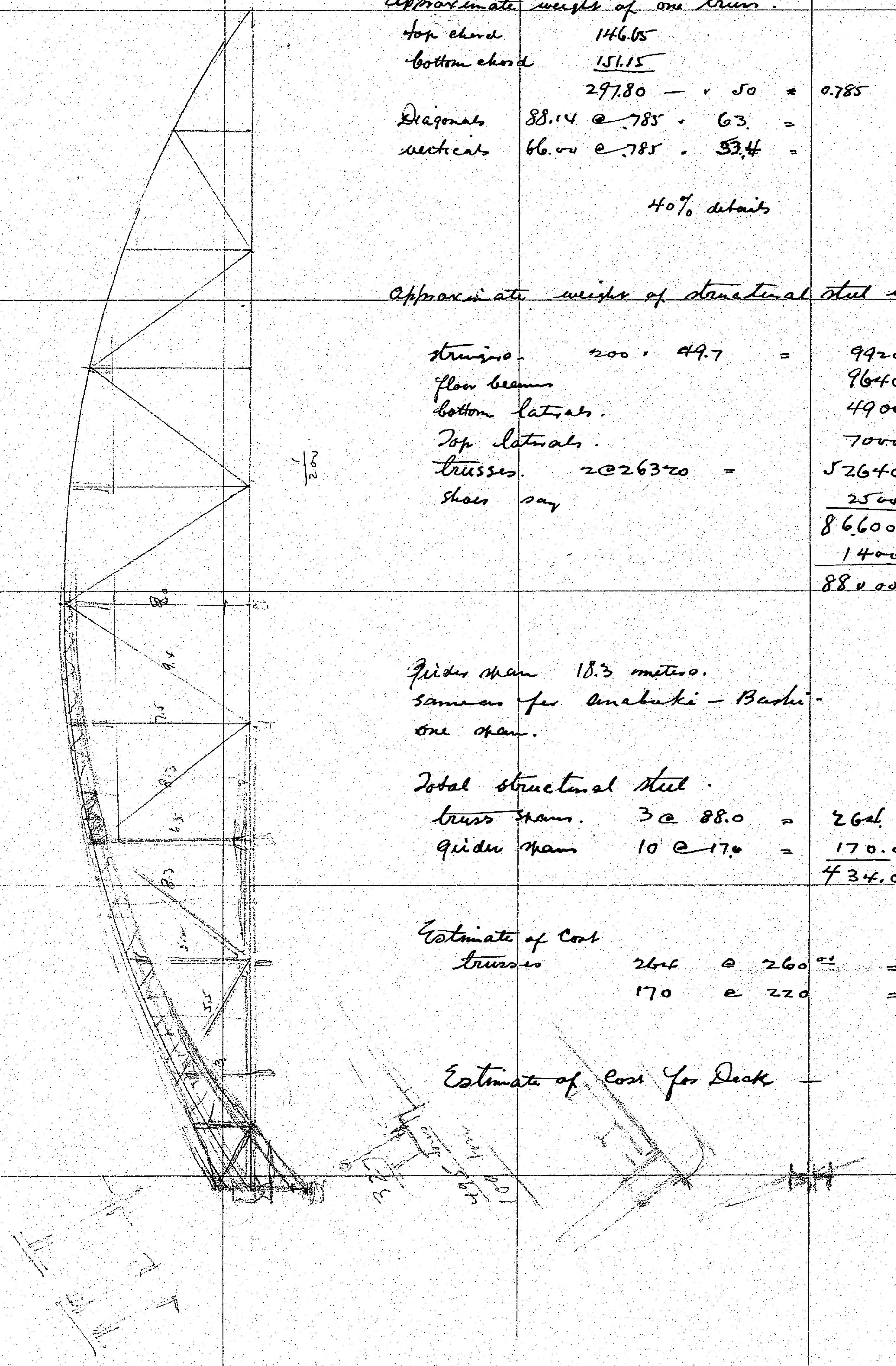
Estimate of Cost

trusses 264 @ 260.00 = 68500

170 @ 220 = 37400

105900

Estimate of Cost for Deck — 45850



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JIJI BLDG, TOKYO

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CALCULATIONS FOR

Preliminary Estimate of Cost Eian-Bashi for Okayama-Ken

Estimate of cost of substructure

For girder piers 8@4200 = 33600
" girders + truss piers 2@5000 = 10000
" truss piers 2@6000 = 12000
abutments 2@4000 = 8000
$63600^{\underline{00}}$

Total cost of bridge -

Structural steel truss span 264 tons @ $260^{\underline{00}}$ = 68500
Girder span 170 tons @ 220 = 37400

$105900^{\underline{00}}$

Deck complete same as for Burger-haiga $40850^{\underline{00}}$
abutment and pier. 63600
$215350^{\underline{00}}$

approaches. 17920
$233270^{\underline{00}}$

4@90. = $360^{\underline{00}}$ × 250 = 90.000
131. × 220 = 28800
11,8800
45850
53700
218350

4@7000 = 28000
4@4300 = 17200
abutment 8500
53700

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CALCULATIONS FOR

Preliminary Estimate of Cost Eian Bashi for Okayama-Ken

All truss span 9 @ 36^00 - 324
weight of truss span 52. tons. 9 @ 52 = 468 tons.

Estimate of cost
Deck construction. 45850
Steel ~~truss~~ span 468 @ 260^00 122000
Substructure
piers 9 @ 4500 = 40500
abutments 2 @ 4500 = 9000
49500
217350^00
approaches. 17920
235270^00

36 -

36 tons @ 260 = 9350
5 × 17, tons @ 220 = 18700
28050

6 @ 20 =
120 × 220 = 26400

1650
1000
2650

Layout No 3.
5 Spans @ 30 = 150 meters
10 " @ 18.0 = 180
330

Structural steel in one truss span 43 × 5 = 215 tons.
" " " girder span 17 × 10 = 170 tons

Estimate of cost
Structural Steel 215 @ 260^00 = 56000
170 @ 220^00 = 37400
93400^00
Deck 45850
Substructure
girder piers 8 @ 4200 = 33600
truss piers 6 @ 5000 = 30000
Abut 8,000
63600
202850
approaches. 17920
220770^00
~~228,770~~

per cubic meter

8
36
268

268 tons @ 260 = ~~5200~~
5 × 17 = 85 @ 220 = 40,000
8 @ 5000 16800
4 @ 4000 56800

6930
1830
88 40
56 60
14 400
45850
20 000
18 000
21 800
25 000
25 500

(16.27)

Yen

予算 216,500

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CALCULATIONS FOR

Preliminary Estimate of Cost Zian-Bashi for Okayama-Ken

Layout No 4

Clear Roadway 5.40 meters

truss Spans 4 @ 50.0 meters

Girder spans 7 @ 18.35 meters.

Girder span same as for Inabuki-Bashi.

weight of structural Steel in one span = 17.0 tons.

for kg ton 17.3 tons call this 17.5 tons

for 7 spans $7 \times 17.5 = 122.5$ tons.

50 meter tied arch span.

structural steel in span

stringer

stringer. see Susai-Bashi p4

Dead Load moment $= \frac{1}{8} \times 770 \times 5.0^2 = 2410$ kg m

Live Load moment

~~Due to~~ motor truck $\frac{2925}{2} \times 2.50 = 3660$

Unif. load M_1 $\frac{593 \times 1.50^2}{2 \times 5.0} \times 2.50 = 334$

Unif. load M_2 $\frac{1}{8} \times 207 \times 5.0^2 = 647.$

4641

2410

7051

Section modulus reqd $= \frac{7051}{1100} = 640.0$

try 300×150 I @ 48.34 kg Sm 633.2

Approximate weight

stringers	$4 @ 50 = 200 \times 50.6 =$	10120
floor beams		10000
bottom laterals		5000
Top Laterals		7000
		32120 kg.

Design of 50 meter tied Arch

structural Steel stringer 4 @ 50.	= 200
floor beams	190
Lateral Bott	100
" Top	200
trusses assumed	1200
	1890 kg

Flooring. $450 \times 5.40 = 2430$

Coping & Handrails. 570

3000

$4890 \div 2 = 2445$ kg per lin meter.

Dead Load stress $M = \frac{1}{8} \times 2445 \times 50^2 = 765000$ kg meter

at crown stress $= \frac{765000}{8.5} = 90.000$ kg.

at End $90000 \times 1.22 = 110.000$ kg.

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CALCULATIONS FOR

Preliminary Estimate of Cost Eian-Bashi for Okayama-Ken

50 meter tied arch
Live Load stress for full load.

500 × 2.70 = 1350

Crown stress 90000 × $\frac{1350}{2445}$ = 49,600
Haunch stress 122.000 × " = 67.500

Max Bending moment due to live load panel concentration 1350 × 2.5 = 3370
for Extra Concentration say 1500
4870
Or say 4500
3450

max bending moment = $\frac{Pl}{2}$ j
3450
= 0.560 × $\frac{4500 × 50}{2}$ = ~~63.000~~
48300
18900
58

At 1/4 point. the thrust say 10.0000
50.000
150.000
2 × 63.000 = 126.000
276.000
Section required = 276.00 cm.

Depth assumed 1.0 meter at neutral axis of top and bottom flange.

Section for bottom chord or tie. 150.000 ÷ 1200 = 125.0 cm net
125 × 1.2 = 150.0 sq cm gross.

Hanger. 4Ls 90 × 90 × 10 @ 13.34 kg. = 53.30
1Pl. 300 × 9 × .785 = 21.20
74.50 kg × 60 m = 4,470 kg.

approximate weight 276.0 @ .785 × 55 = 11,900
Details say 50% - 5950
17,850 — 17.850

Tie. 150 @ .785 × 50 = 5820
30%. 1740
7560

7.560

Vertical hanger 4.470
29.880
For 2 trusses - 59.760
59760 ÷ 50 = 1200 kg per lin. meter

approximate weight
Floor system and laterals - 32120 91.8
~~trusses~~ 59760
shoe &c 2500
94380 call this 95.0 tons
4 @ 95 = 380 tons

approximate cost of structural steel
~~380~~ 396 tons @ ~~265~~ 260 = 101.000 103
122.5 tons @ 210.00 = 25700
126,700

Deck construction - 45.000
171.700
44700
216400

128,700
42000
170700

95
396
4

Load on shoe DL 2445
LL 1350
3795 × 25 = 95.000 for One shoe

216500
170700
45800

227

113
4 48

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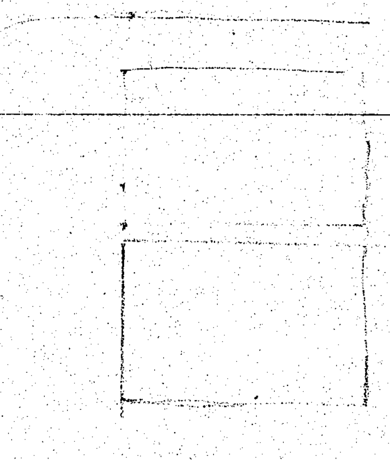
CALCULATIONS FOR

Preliminary Estimate of Cost Eian-Bashi Okayama-Ken.

Girder Pier.
Center to center of main girder. 5.60
Superimposed load Dead load 2100
Live load 1350
3450 × 18.35 = 63.200 kg.

For one bearing. 31.600 kg. size of base – 40 × 60 = 2400 $\frac{31.600}{2400}$ = 13 kg/cm²



coping. 1.40 ϕ = 1.54
1.40 × 5.60 = 7.84
9.38 × .30 = 2.82 cubic meters

strut. 1.20 = 1.13
1.20 × 5.60 = 6.72
7.85 × 1.00 = 7.85

shaft 1.20 = 1.13
1.20 × 1.6 = 1.92
3.05
1.74 ϕ = 2.38
1.74 × 1.9 = 3.30
5.68
8.73 ÷ 2 = 4.36 × 4.45 = 19.40
7.85
2.82
shaft and coping – 30.07

well – 2.75 ϕ – outside dia ~~4.00~~ 5.93
2.15 inside dia 3.63
2.30 × 6.0 = 13.8 2 @ 13.8 = 27.6

Concrete filling 3.63 × 2.5 = 9.1 × 2 = 18.2 cubic meters
Sand filling 3.63 × 3.5 = 12.7 × 2 = 25.4 " "

Total Concrete 30.07
well – 27.60
filling 18.20
75.87 @ 2400 = 182.000
Sand – 25.40 @ 1700 = 43 000
225.000 –
Superimposed load 126.000
351.000 kg.

bottom area 2.90 ϕ – 6.60 × 2 = 13.2
unit bearing $\frac{351.000}{132}$ = 26600 kg/m²
24.7 tons/ft²

Counting friction of well – 1.5 tons per sq meter
2.75 ϕ – 8.64 × 1.5 × 6.0 = 78.000 kg.
2
156.0 –

351.0
– 156.0
195.0

Unit bearing = $\frac{195.000}{132}$ = 14800 kg/m²
1.375 tons/ft²

area of forms. shaft. 75.
well. 186.

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CALCULATIONS FOR

Preliminary Estimate of Cost Eian-Bashi Okayama-Ken.

Estimate of Cost

concrete in shaft	30.0 @	17.00	=	510	
Reinf. bars -	1.6 @	150^{00}	=	240	
forms.	750 @	2^{00}	=	150	
					900^{00}
well concrete	4580 ~~75.87~~ @	17^{00}	=	780 -	
Sand filling.	25.40 @	1^{00}	=	25	
Reinf. bars -	1.5 @	150^{00}	=	225	
forms.	186 @	1^{50}	=	280	
curb shoe				150	
well sinking -				400 -	
sheet piling &c				300	
					2160
					3060
					3100^{00}
	7 @	3250	=	22700	45800

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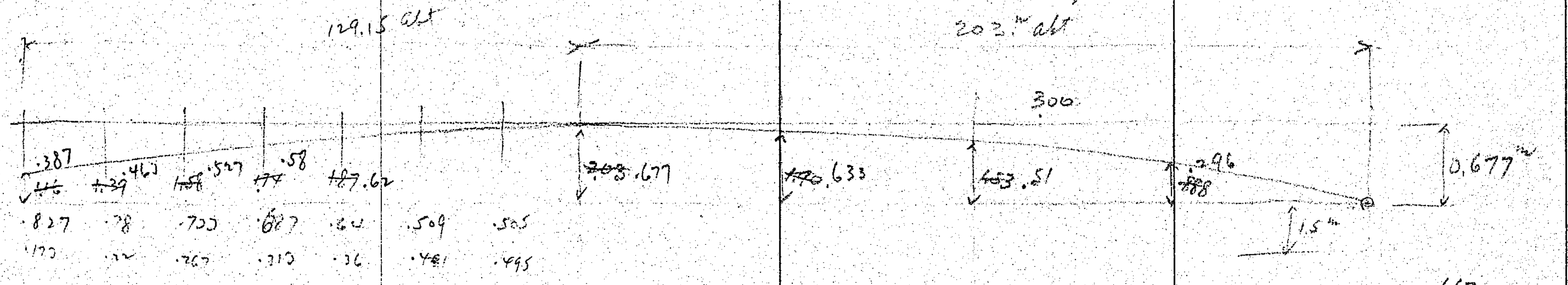
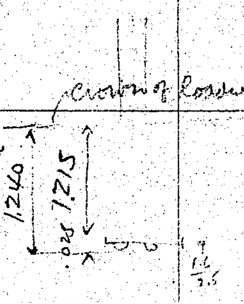
CALCULATIONS FOR

Eian Bashi for Okayama ken.



.667
.387
.280

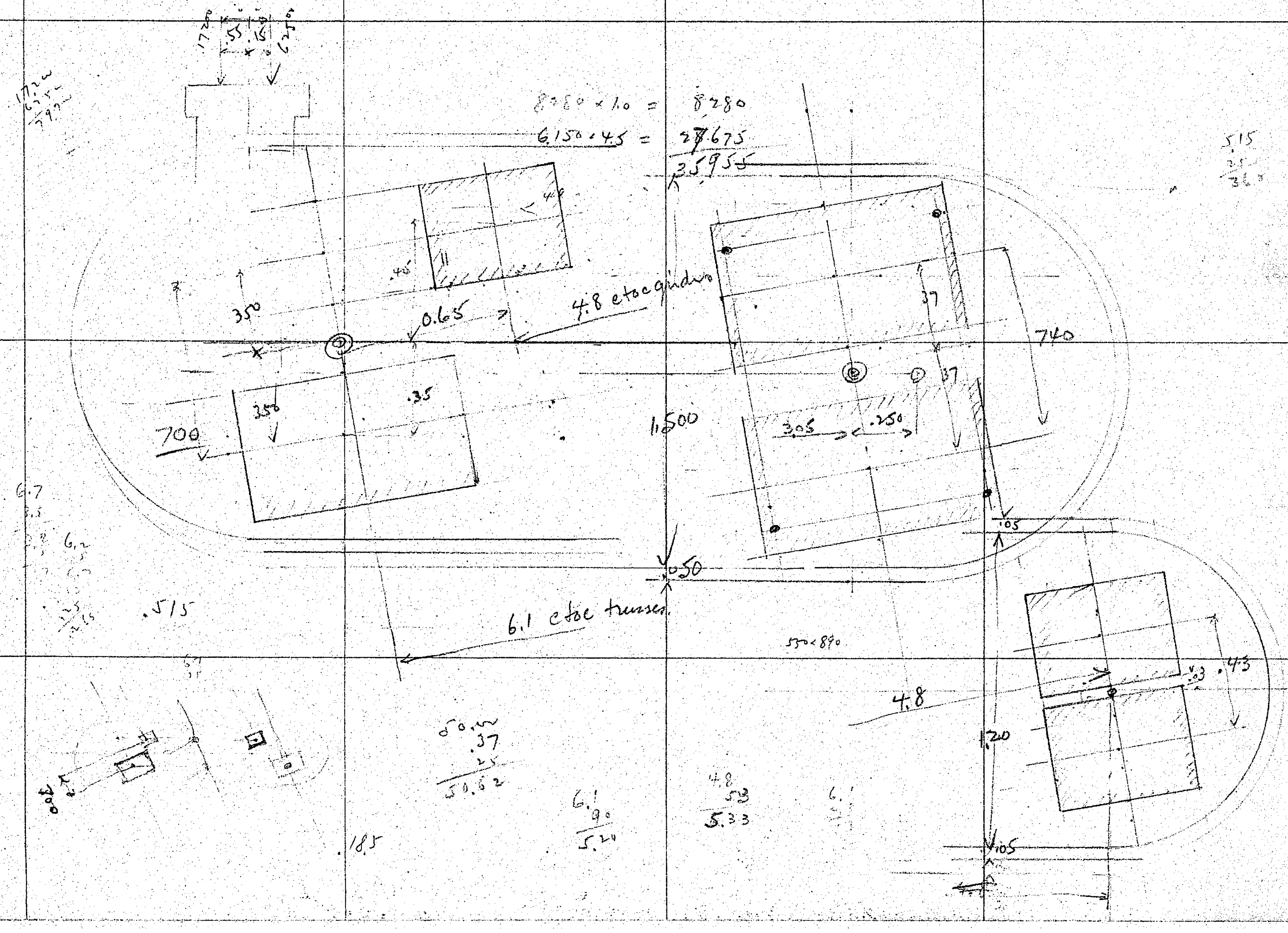
2.00
1.16
.889



Girder pier
Load on shoe. D.L. 18,000
L.L. 16,500
34,500 one shoe
×4

					L.L.	total
max D.L. Girder span		17200 kg	on one shoe.		16,454	33,654 kg
" " truss	12500×5	62500	" "		35,955	98,455
		79700 kg				132,110 kg ×2 = 264,200 kg for one pier

8080×1.0 = 8280
6150×4.5 = 27675
35955



5.15
.35
3.60

50.00
.37
.25
50.62

6.1
.90
5.20

4.8
.53
5.33

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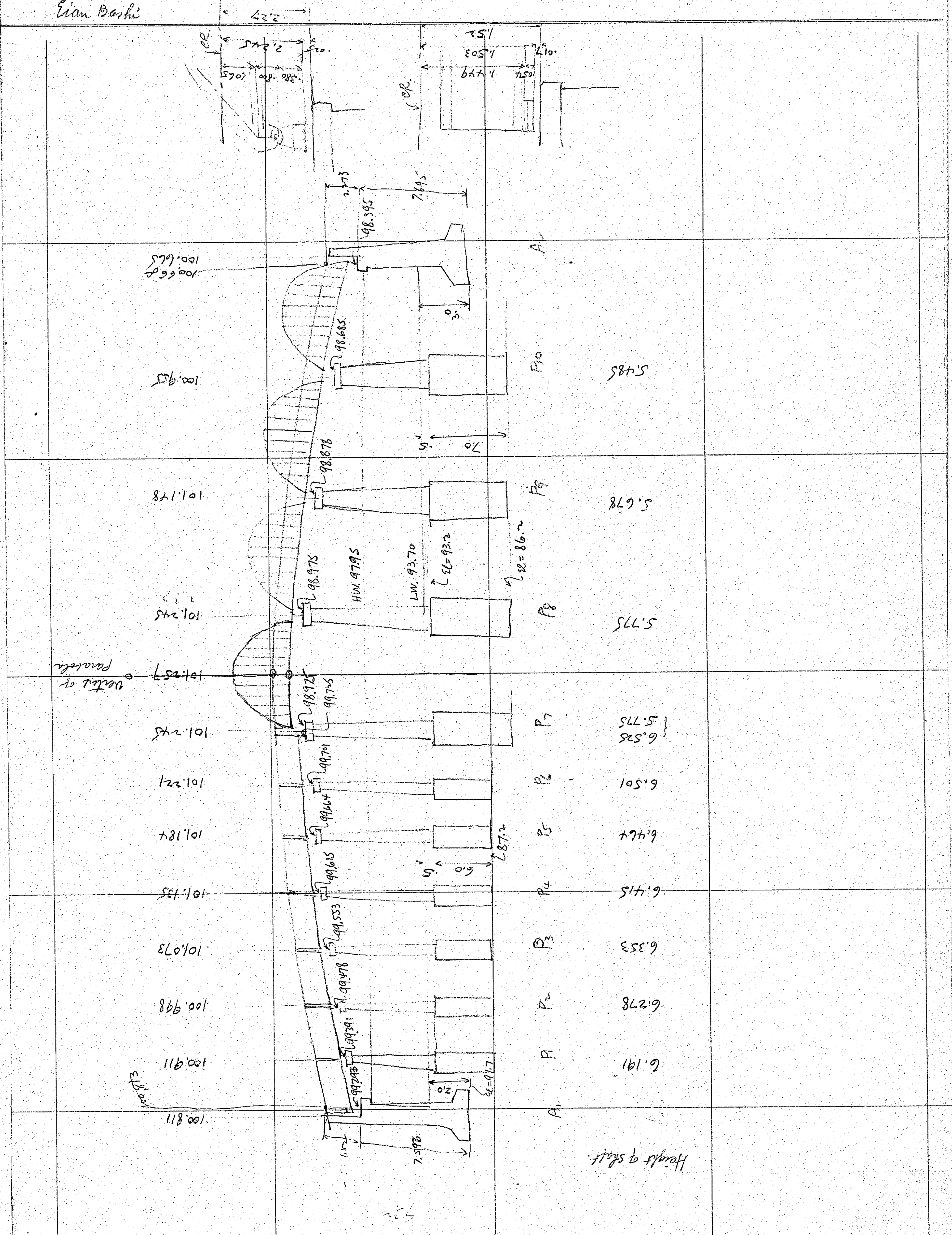
CALCULATIONS FOR

Eian Bashi



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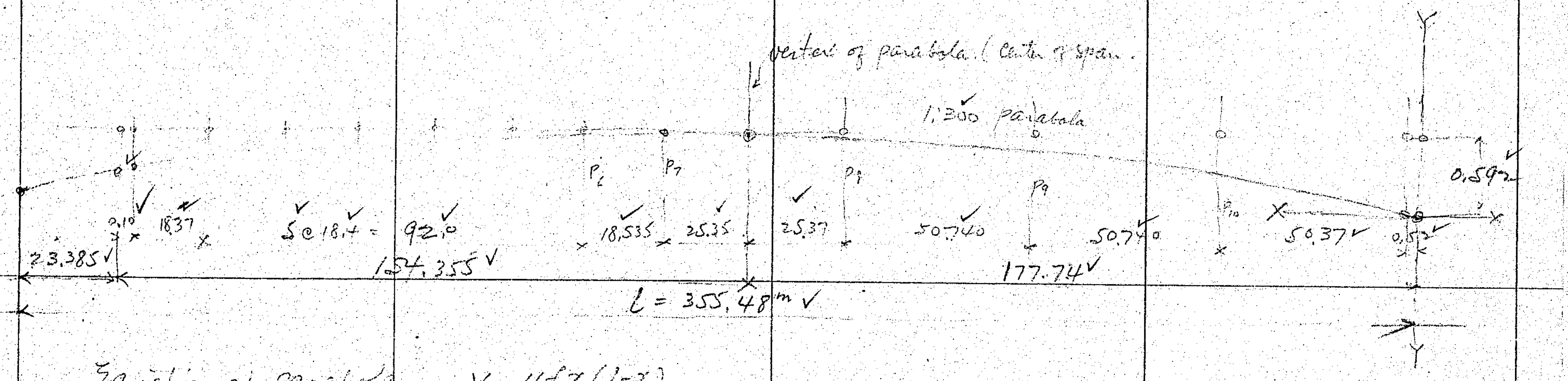
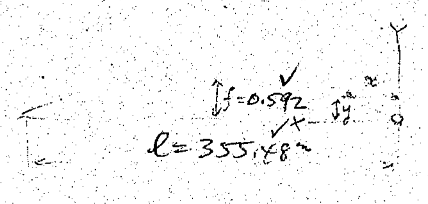
CALCULATIONS FOR

Eian bashi for Okayamaken

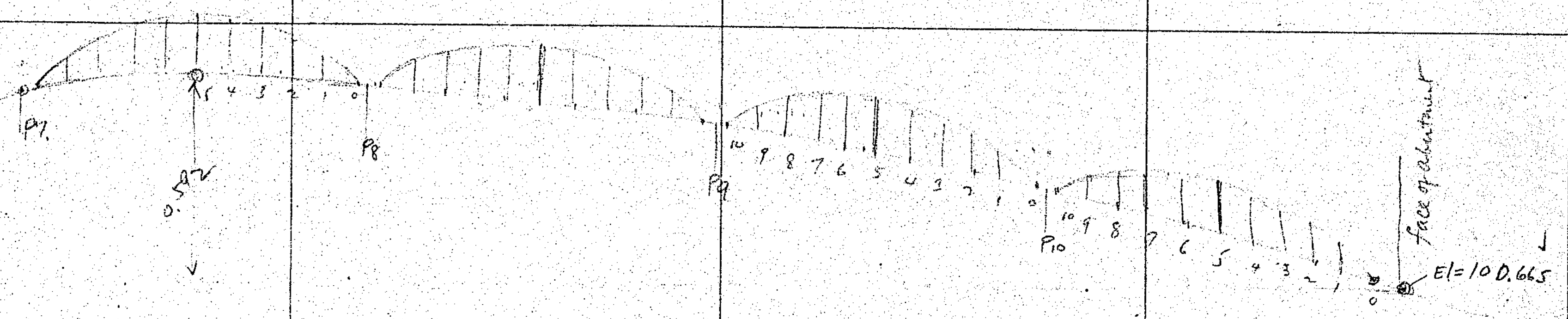
Profile of Roadway



Equation of parabola $y = \frac{4fx(l-x)}{l^2}$



$$= \frac{4\times 0.592}{355.48^2}(l-x)x = 0.00001874(l-x)x$$



Points	x	l-x	y	Crown of Roadway El	
Face of abutment	0 ✓	355.48 ✓	0.000 ✓	100.665 ✓	3
Lo ✓	0.52 ✓	354.96 ✓	0.003 ✓	100.668 ✓	3
1 ✓	5.52 ✓	349.96 ✓	0.036 ✓	100.701 ✓	32
2 ✓	10.52 ✓	344.96 ✓	0.068 ✓	100.733 ✓	31
3 ✓	15.52 ✓	339.96 ✓	0.099 ✓	100.764 ✓	30
4 ✓	20.52 ✓	334.96 ✓	0.129 ✓	100.794 ✓	29
5 ✓	25.52 ✓	329.96 ✓	0.158 ✓	100.823 ✓	28
6 ✓	30.52 ✓	324.96 ✓	0.186 ✓	100.851 ✓	27
7 ✓	35.52 ✓	319.96 ✓	0.213 ✓	100.878 ✓	26
8 ✓	40.52 ✓	314.96 ✓	0.239 ✓	100.904 ✓	25
9 ✓	45.52 ✓	309.96 ✓	0.264 ✓	100.929 ✓	25
10 ✓	50.52 ✓	304.96 ✓	0.289 ✓	100.954 ✓	1
P10 ✓	50.89 ✓	304.59 ✓	0.290 ✓	100.955 ✓	2
0 ✓	51.26 ✓	304.22 ✓	0.292 ✓	100.957 ✓	23
1 ✓	56.26 ✓	299.22 ✓	0.315 ✓	100.980 ✓	23
2 ✓	61.26 ✓	294.22 ✓	0.338 ✓	101.003 ✓	21
3 ✓	66.26 ✓	289.22 ✓	0.359 ✓	101.024 ✓	21
4 ✓	71.26 ✓	284.22 ✓	0.380 ✓	101.045 ✓	19
5 ✓	76.26 ✓	279.22 ✓	0.399 ✓	101.064 ✓	19
6 ✓	81.26 ✓	274.22 ✓	0.418 ✓	101.083 ✓	17
7 ✓	86.26 ✓	269.22 ✓	0.435 ✓	101.100 ✓	17
8 ✓	91.26 ✓	264.22 ✓	0.452 ✓	101.117 ✓	16
9 ✓	96.26 ✓	259.22 ✓	0.468 ✓	101.133 ✓	14
10 ✓	101.26 ✓	254.22 ✓	0.482 ✓	101.147 ✓	1
P9 ✓	101.63 ✓	253.85 ✓	0.483 ✓	101.148 ✓	

Points	x	l-x	y		
0 ✓	102.00 ✓	253.48 ✓	0.485 ✓	101.150 ✓	2
1 ✓	107.00 ✓	248.48 ✓	0.498 ✓	101.163 ✓	13
2 ✓	112.00 ✓	243.48 ✓	0.511 ✓	101.176 ✓	12
3 ✓	117.00 ✓	238.48 ✓	0.523 ✓	101.188 ✓	11
4 ✓	122.00 ✓	233.48 ✓	0.534 ✓	101.199 ✓	10
5 ✓	127.00 ✓	228.48 ✓	0.544 ✓	101.209 ✓	9
6 ✓	132.00 ✓	223.48 ✓	0.553 ✓	101.218 ✓	8
7 ✓	137.00 ✓	218.48 ✓	0.561 ✓	101.226 ✓	7
8 ✓	142.00 ✓	213.48 ✓	0.568 ✓	101.233 ✓	6
9 ✓	147.00 ✓	208.48 ✓	0.574 ✓	101.239 ✓	6
10 ✓	152.00 ✓	203.48 ✓	0.580 ✓	101.245 ✓	0
P8 ✓	152.37 ✓	203.11 ✓	0.580 ✓	101.245 ✓	0
0 ✓	152.74 ✓	202.74 ✓	0.580 ✓	101.245 ✓	4
1 ✓	157.74 ✓	197.74 ✓	0.585 ✓	101.250 ✓	3
2 ✓	162.74 ✓	192.74 ✓	0.588 ✓	101.253 ✓	2
3 ✓	167.74 ✓	187.74 ✓	0.590 ✓	101.255 ✓	2
4 ✓	172.74 ✓	182.74 ✓	0.592 ✓	101.257 ✓	0
5 ✓	177.74 ✓	177.74 ✓	0.592 ✓	101.257 ✓	

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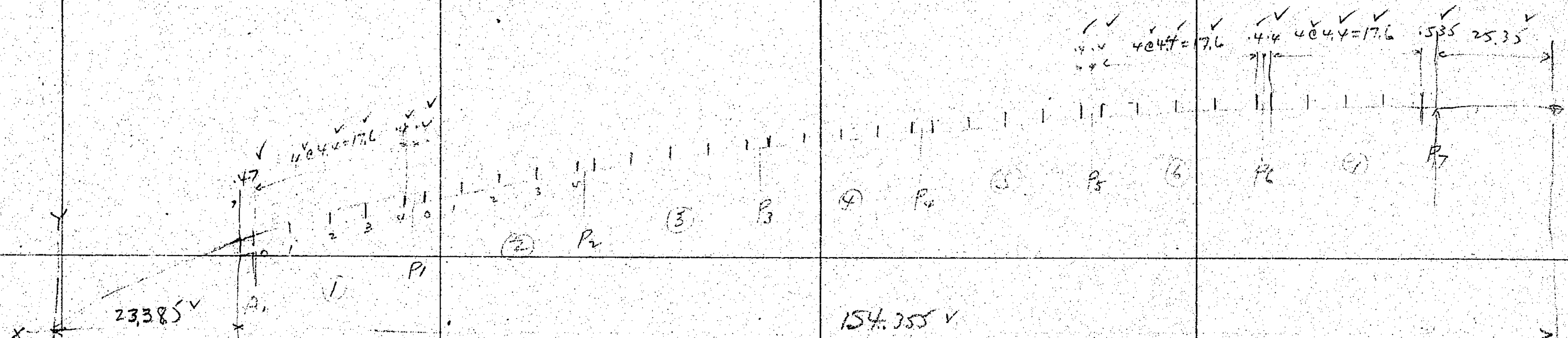
CALCULATIONS FOR

Eian-Bashi in Okayama Ken

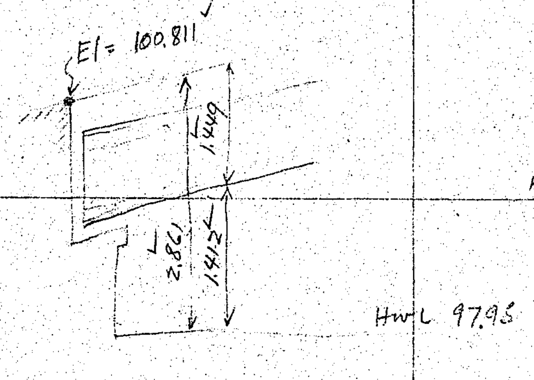


154.355
L = 177.743 × 2 = 355.48

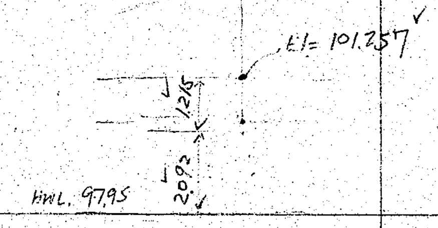
$$y = \frac{4fx(l-x)}{l^2} = 0.00001874\,(l-x)x$$

Points	x	$l-x$	y	
Face of Abutment	23.385	332.095	0.146	100.811
0	23.855	331.625	0.148	100.813
1	28.255	327.225	0.173	100.838
2	32.655	322.825	0.198	100.863
3	37.055	318.425	0.221	100.886
4	41.455	314.025	0.244	100.909
P1	41.855	313.625	0.246	100.911
0	42.255	313.225	0.248	100.913
1	46.655	308.825	0.270	100.935
2	51.055	304.425	0.291	100.956
3	55.455	300.025	0.312	100.977
4	59.855	295.625	0.332	100.997
P2	60.255	295.225	0.333	100.998
0	60.655	294.825	0.335	101.000
1	65.055	290.425	0.354	101.019
2	69.455	286.025	0.372	101.037
3	73.855	281.625	0.390	101.055
4	78.255	277.225	0.407	101.072
P3	78.655	276.825	0.408	101.073
0	79.055	276.425	0.410	101.075
1	83.455	272.025	0.425	101.090
2	87.855	267.625	0.441	101.106
3	92.255	263.225	0.455	101.120
4	96.655	258.825	0.469	101.134
P4	97.055	258.425	0.470	101.135

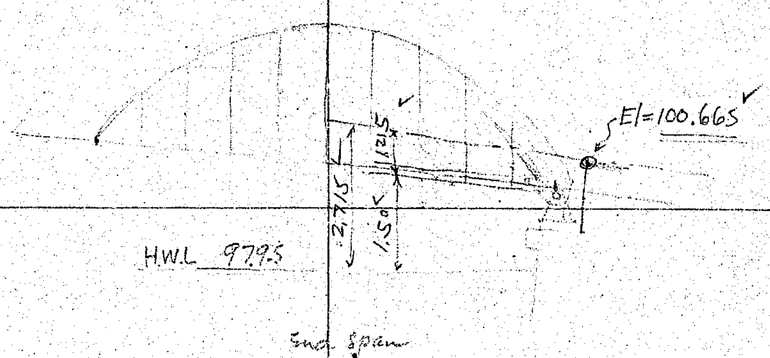
points	x	$l-x$	y	
0	97.455	258.025	0.471	101.136
1	101.855	253.625	0.484	101.148
2	106.255	249.225	0.496	101.161
3	110.655	244.825	0.508	101.173
4	115.055	240.425	0.518	101.183
P5	115.455	240.025	0.519	101.184
0	115.855	239.625	0.520	101.185
1	120.255	235.225	0.530	101.195
2	124.655	230.825	0.539	101.204
3	129.055	226.425	0.548	101.213
4	133.455	222.025	0.555	101.220
P6	133.855	221.625	0.556	101.221
0	134.255	221.225	0.557	101.222
1	138.655	216.825	0.563	101.228
2	143.055	212.425	0.569	101.239
3	147.455	208.025	0.575	101.240
4	151.855	203.625	0.579	101.244
P7	152.390	203.090	0.580	101.245
0	152.740	202.740	0.580	101.245
1	157.740	197.74	0.585	101.250
2	162.740	192.74	0.588	101.253
3	167.740	187.74	0.590	101.255
4	172.740	182.74	0.592	101.257
5	177.740	177.740	0.592	101.257



Endspan



Between P7 + P8



End span

97.95
2.74
100.69

34
22
56.m 50
28m

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CALCULATIONS FOR Eian Bashi for Okayama Ken

Estimate of Cost for Girder Pier.

Concrete 1:2:4	93.65 m^3	@ 16.0 =	1500	
Sand fill	34.4 "	@ 1.0 =	34	
Reinforcements.	2.8 tons	@ 135 =	378	~~3005~~
Area of forms.	310 m^2	@ 1.2 ~~1.5~~ =	372 ~~465~~	2850
Curb shoes.	0.831 ton	@ 250 =	208	
well sinking 6m @ 7.07 = 42.4 m^3 42.4 84.8 m^3	2×6 = 12m	@ 30 ~~35~~ =	360 ~~420~~	

~~3005~~ 2850
400
3250

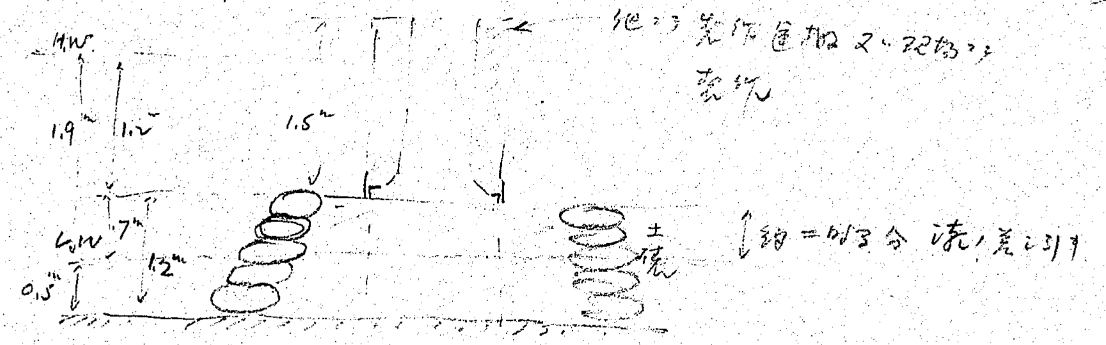
土俵 盛土 其他 設置費

19φ 5×38 = 190 @ 2.22 = 421
16φ 4×38 = 152 @ 1.57 = 239
Shaft 13φ 7.5×34 =
" 6.0×5 =
" 1×25 =
} 310 @ 1.04 = 323
983

well. 13φ
11.3×60
8.8×64
1.7×32
64×64
} 1710 @ 1.04 = 1780
2763 kg

.35×14 = 5
4.4×3.4 = 15
13.0×6.2 = 81
− 4.2×1.1 = −5
96

3.φ = 10
2.4φ = 7.5
17.5×6 = 105×2 = 210
96
306



Estimate for Truss pier.

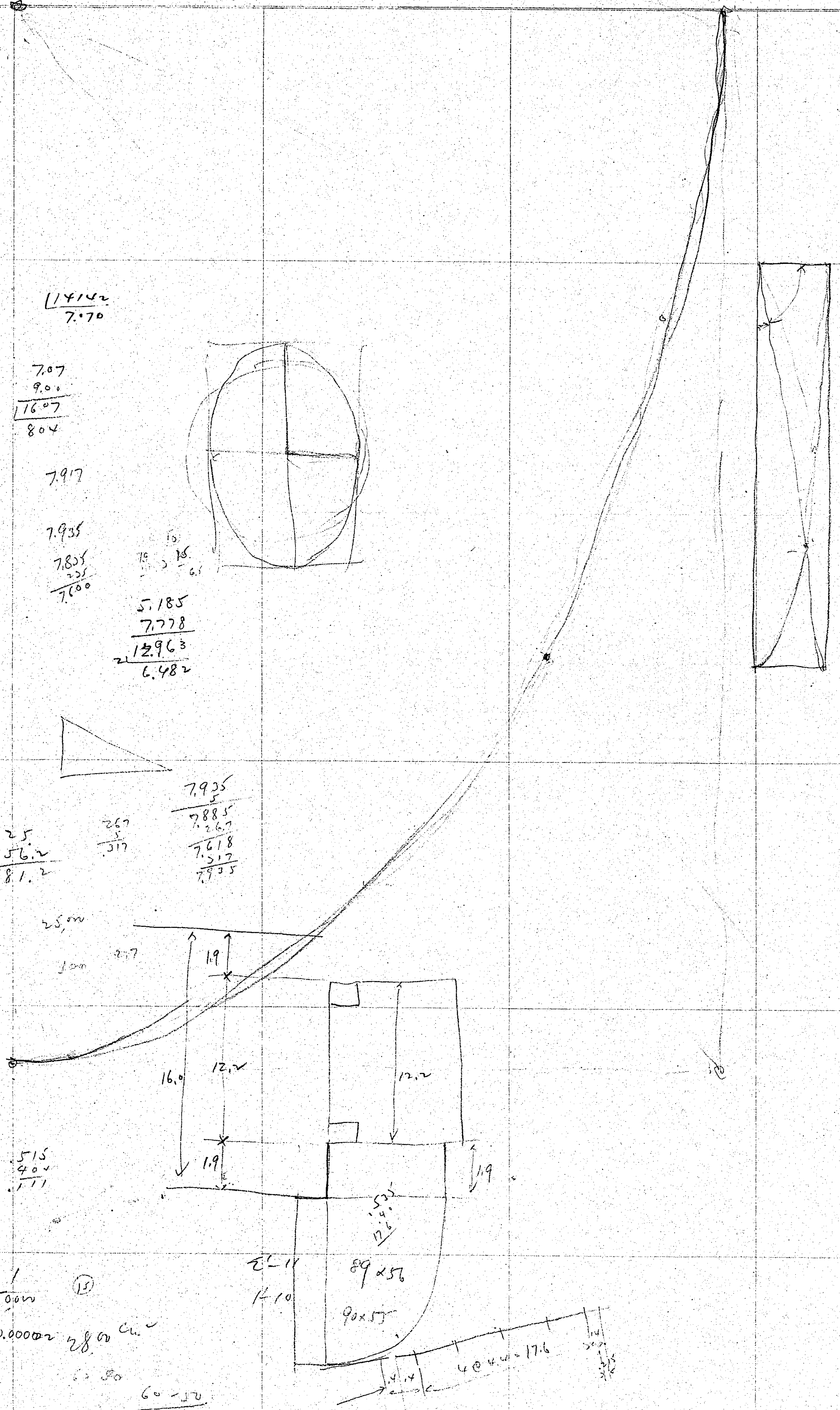
Concrete		131.5 m^3	@ 16 =	2100
Sand fill		57	@ 1.0 =	57
Reinf.	say	3.4	@ 135 =	460
Area of form		370	@ 1.2 =	445
Curb shoes		1	@ 250 =	250
well sinking		1.4	@ 500 =	700

4010
490
4500 円

40
10
10
13
73
43
116
60

6 @ 3000 18000
4 @ 5000 = 20000
10 piers — 38000 円

7 @ 3250 = 22,700
4 @ 4500 = 18,000
1 @ 4000 = 4000
44700



1.4142
7.70

7.07
9.00
16.07
8.04

7.917

7.935

7.835
.235
7.600

5.185
7.778
12.963
6.482

7.935
5
7.885
.267
7.618
.317
7.935

.267
.5
.317

25
56.2
81.2

25.0m

.515
.404
.111

1
500m

⑮

0.00002 2800 cm

60 - 32