LOG OF BOREHOLE

SER No. 77

NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites

Borehole BH1

ground elevation 63.919m

Location : Bridge No. 3/2 km Polgahawela-Kegalle Road

depth of borehole 5.59 m

Boring method : Wash boring

Commenced on : 06.11.1995

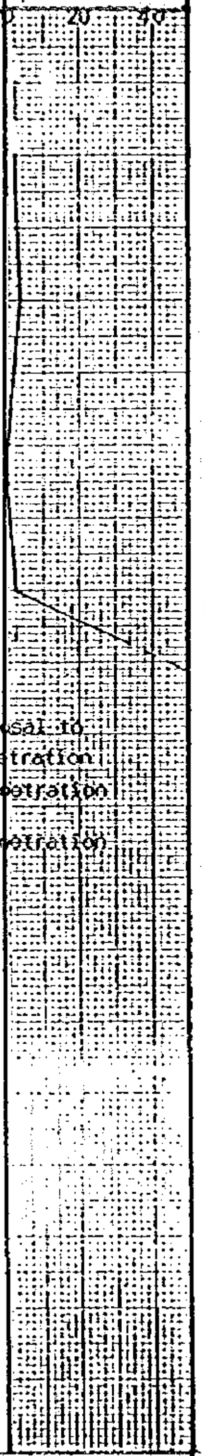
Water struck at GL - m

Drilling mud : Bentonite

Completed on : 06.11.1995

GWL on completion of borehole GL-1.40 m

Depth below GL m	Classification & Description of Soil		Type and Depth of Sampling m		depth tested GL m	STANDARD PENETRATION TEST DATA number of blows per 15cm 1	2	3	N-value for 30cm
0.00	ML SM	Dark brown, low plasticity silt with sand and less percentage of mica.	DS	0.10 0.20	0.10	Auger sample			
0.60	SM	Very loose, brown, medium to fine grained silty sand.							
1.00			DS	1.00 1.45	1.00	1	1	1	2
1.35	SC	Very loose, brown medium to fine grained clayey sand with mica.							
1.70									
2.00	SM	Very loose, brown, medium to fine grained silty sand with mica.	DS	2.00 2.45	2.00	2	2	1	3
2.80									
3.00	SM	Very loose, brown, fine grained silty sand.	DS	3.00 3.45	3.00	1	0	0	0
4.00		Sample not recovered.		4.00 4.45	4.00	2	1	1	2
5.00	SM	Very dense, whitish grey, silty sand with mica.	DS	5.00 5.35	5.00	2	37	21/5	Refusal to penetration
5.59		Sample not recovered.	DS	5.45 5.53	5.45	37/8	Refusal to penetration		
				5.57 5.59	5.57	29/2	Refusal to penetration		
		Borehole terminated at 5.59m depth.							



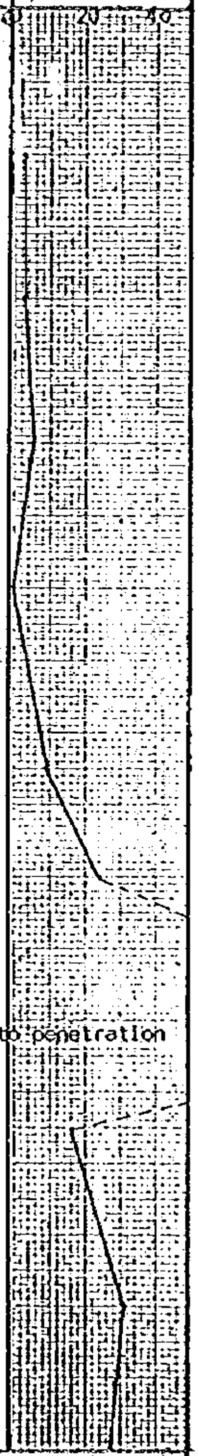
LOGGED BY : Deepthika Herath

GEOTECHNICAL ENGINEERING DIVISION
NATIONAL BUILDING RESEARCH ORGANISATION

DATE :

LOG OF BOREHOLE SER No. 53

NAME OF PROJECT : Geotechnical Investigations for proposed Bridge Sites		Borehole	BH1
		ground elevation	23.238m -
Location : Bridge No. 36/3 km Kegalle-Bulathkohupitiya-Karawanolla Rd.		depth of borehole	13.90 m
Boring method : Wash boring	Commenced on : 01.11.1995	Water struck at GL -	m
Drilling mud : Bentonite	Completed on : 02.11.1995	GWL on completion of borehole	GL- 3.45 m

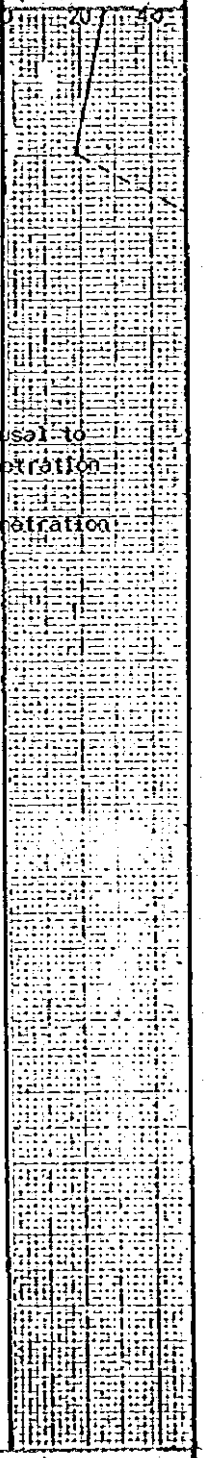
Depth below GL m	Classification & Description of Soil		Type and Depth of Sampling m		STANDARD PENETRATION TEST DATA					
					depth tested GL m	number of blows				
						per 15cm			N-value	
						1	2	3	for 30cm	graphical presentation
0.00	SM	Brown, silty sand.	DS	0.10 0.20	0.10	Auger sample				
1.00 1.30			DS	1.00 1.45	1.00	2	1	2	3	
	ML/SM	Very loose, brown low plasticity silt with sand.								
2.00 2.35			DS	2.00 2.45	2.00	2	1	2	3	
	CL	Soft brown, sandy clay.								
3.00		Sample not recovered.		3.00 3.45	3.00	4	3	2	5	
4.00				4.00 4.74	4.00	1	0	0	0	
	SC	Very loose, brown fine grained clayey sand.								
4.80			UDS	4.90 5.45						
5.00	SM	Loose, brown, fine grained silty sand.								
			DS	5.45 5.90	5.45	1	3	6	9	
5.95										
6.00		Medium dense, brown, medium to fine grained silty sand with quartzitic cobbles.	DS	6.00 6.45	6.00	3	12	12	24	
7.00	SM	Very dense, brown, quartzitic silty sand.	DS	7.05 7.30	7.05	21	17/10	Refusal to penetration		
			DS	7.95 8.40	7.95	2	5	11	16	
8.00	SM	Medium dense brown quartzitic silty sand (significant amount of gravel size quartz particles were observed).								
8.85										
9.00			DS	9.00 9.45	9.00	10	14	17	31	
	ML	Dense, greenish grey, fine grained micasious sandy silt.								

LOGGED BY : Deepthika Herath	GEOTECHNICAL ENGINEERING DIVISION NATIONAL BUILDING RESEARCH ORGANISATION	DATE :

LOG OF BOREHOLE

SER No. 53

NAME OF PROJECT : Geotechnical Investigations for proposed Bridge Sites		Borehole	BH1
		ground elevation	23.238m
Location : Bridge No. 36/3 km Kegalle-Bulathkohupitiya-Karawanella Rd.		depth of borehole	13.90 m
Boring method : Wash boring	Commenced on : 01.11.1995	Water struck at GL -	m
Drilling mud : Bentonite	Completed on : 02.11.1995	GWL on completion of borehole	GL- 3.45 m

Depth below GL m	Classification & Description of Soil		Type and Depth of Sampling m		STANDARD PENETRATION TEST DATA					
					depth tested GL m	number of blows				
						per 15cm			N-value	
						1	2	3	for 30cm	graphical presentation
10.00		Sample not recovered.		10.00 10.45	10.00	12	13	14	27	
11.00	SM	Medium dense, dark greenish fine grained micasious silty sand with some quartz.	DS	11.00 11.45	11.00	9	9	10	19	
11.70										
12.00	SM	Very dense, dark greenish medium to fine grained micasious silty sand	DS	12.00 12.45	12.00	11	22	29	51	
13.00		Very dense, greenish, micasious silty sand with rock fragments.	DS	13.00 13.37	13.00	14	14	36/7	Refusal to penetration	
				13.86 13.90	13.86	34/4	Refusal to penetration			
13.90										
		Borehole terminated at 13.90m depth.								

LOGGED BY : Deepthika Herath

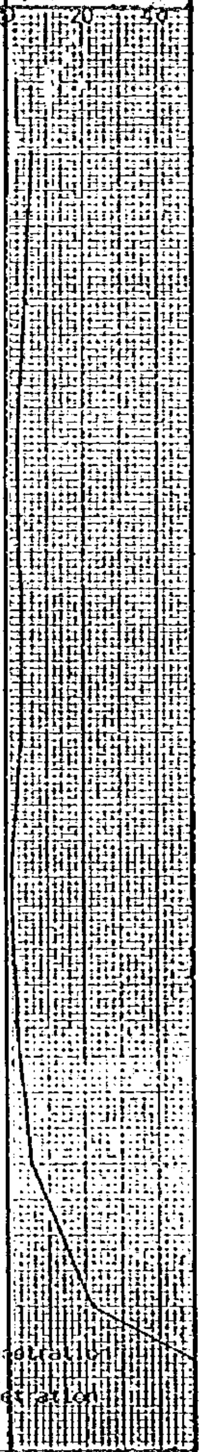
GEOTECHNICAL ENGINEERING DIVISION
NATIONAL BUILDING RESEARCH ORGANISATION

DATE :

LOG OF BOREHOLE SER No. 211

NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites		Borehole	BH1
		ground elevation	11.579m MSL
Location : Bridge No. 8/3 km Hanwella-Pugoda-Weka-Urapola Road		depth of borehole	9.80 m
Boring method : Wash boring	Commenced on : 04.11.1995	Water struck at GL -	m
Drilling mud : Bentonite	Completed on : 04.11.1995	GWL on completion of borehole	GL- 5.80 m

Depth below GL m	Classification & Description of Soil		Type and Depth of Sampling m		STANDARD PENETRATION TEST DATA					
					depth tested GL m	number of blows				
						per 15cm			N-value	
						1	2	3	for 30cm	graphical presentation
0.00	ML	Brown, sandy silt.								
0.60										
1.00			DS	1.00 1.45	1.00	2	3	3	6	
2.00			DS	2.00 2.45	2.00	1	2	2	4	
3.00	MH	Loose to very loose, brown to yellow brown clayey silt with traces of fine sand.	DS	3.00 3.45	3.00	1	1	1	2	
4.00			DS	4.00 4.45	4.00	1	1	2	3	
4.50			UDS	4.60 5.00						
5.00			DS	5.00 5.45	5.00	1	2	1	3	
	MI	Very loose, yellow brown clayey silt.								
6.00			DS	6.00 6.45	6.00	1	0	0	0	
6.50	ML	Very loose to loose, yellow brown clayey silt with traces of sand.								
7.00			DS	7.00 7.45	7.00	1	0	1	1	
8.00			DS	8.00 8.45	8.00	2	2	3	5	
8.35	SM	Medium dense greyish brown silty fine to medium sand.								
9.00			DS	9.00 9.45	9.00	1	3	19	22	
9.35	ML	Very dense, brown clayey silt with traces of sand.		9.65 9.70	9.65	27/5	Refusal to penetration			
				9.77 9.80	9.77	30/3	Refusal to pnetration			
9.80		Borehole terminated at 9.80m depth.								



LOGGED BY :	GEOTECHNICAL ENGINEERING DIVISION NATIONAL BUILDING RESEARCH ORGANISATION	DATE :

LOG OF BOREHOLE

SER No. 33

NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites

Borehole: BH 1

ground elevation: 7.957m MSL

Location : Bridge No. 12/3 km Narthupana-Horana-Aluthgama Road.

depth of borehole: 7.05 m

Boring method : Wash boring

Commenced on : 06.11.1995

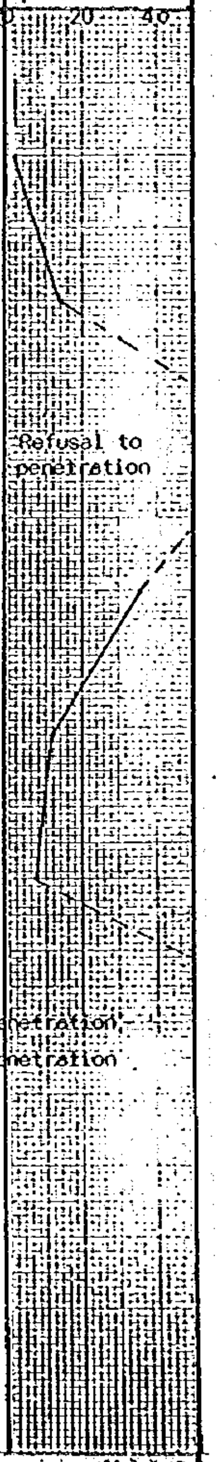
Water struck at GL - m

Drilling mud : Bentonite

Completed on : 06.11.1995

GWL on completion of borehole: GL- 4.80 m

Depth below GL m		Classification & Description of Soil	Type and Depth of Sampling m		depth tested GL m	STANDARD PENETRATION TEST DATA: number of blows per 15cm 1	2	3	N-value for 30cm	graphical presentation
0.00										0 20 40
	MH	Very loose, reddish brown to yellow brown clayey silt with traces of sand and gravel.								
1.00			DS	1.00 1.50	1.00	1	1	1	2	
1.50										
2.00			DS	2.00 2.45	2.00	1	4	10	14	
	SM	Medium dense to dense, yellowish brown to reddish brown, silty fine to medium sand with gravel.								
3.00			DS	3.00 3.45	3.00	17	14	18/0	32	Refusal to penetration
3.50										
	MI	Dense to medium dense yellow brown clayey silt with traces of sand and gravel.								
4.00			DS	4.00 4.95	4.00	25	25	11	36	
5.00			DS	5.00 5.45	5.00	5	6	6	12	
	SM	Medium dense to loose to very dense brown to yellowish brown silty fine to medium sand with occasional gravel.								
6.00			DS	6.00 6.45	6.00	10	5	2	7	
7.00				7.00	7.00	30/0	Refusal to penetration			
7.05				7.05	7.05	30/0	Refusal to penetration			
		Borehole terminated at 7.05m depth.								



LOGGED BY :

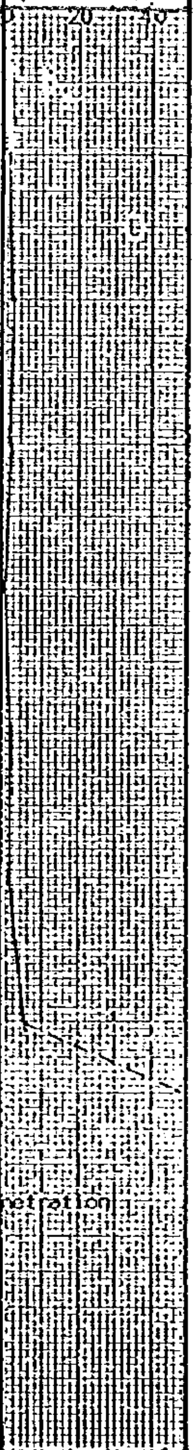
GEOTECHNICAL ENGINEERING DIVISION
NATIONAL BUILDING RESEARCH ORGANISATION

DATE :

LOG OF BOREHOLE SER No. 59

NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites		Borehole	BH1
		ground elevation	0.946m MSL
Location : Bridge No. 43/4 Km - Munamalwatta-Horana-Aluthgama Road.		depth of borehole	8.55 m
Boring method : Wash boring	Commenced on : 04.11.1995	Water struck at GL -	m
Drilling mud : Bentonite	Completed on : 04.11.1995	GWL on completion of borehole	GL- 0.40 m

Depth below GL m		Classification & Description of Soil	Type and Depth of Sampling m		depth tested GL m	number of blows per 15cm 1	2	3	N-value for 30cm
0.00 0.40	SM	Brown silty, fine to medium sand with gravel.							
1.00			DS	1.00 1.50	1.00	1	0	1	1
	SC	Very loose, brown clayey fine to medium sand with gravel.							
2.00			DS	2.00 2.50	2.00	1	1	1	2
3.00			DS	3.00 3.50	3.00	1	0	1	1
			UDS	3.50 4.00					
4.00	Pt	Very soft greyish black peat with partially decomposed vegetation and pockets of organic clay.	DS	4.00 4.50	4.00	1	0	0	0
			UDS	4.50 4.95					
5.00 5.30			DS	5.00 5.50	5.00	1	0	1	1
	SC	Very loose, clayey fine sand with organic matters.							
6.00			DS	6.00 6.50	6.00	1	0	1	1
	SM	Very loose, greyish brown silty fine to medium sand with traces of clay.							
7.00			DS	7.00 7.48	7.00	1	2	3	5
	SM	Very dense, yellowish brown silty fine to medium sand.							
8.00			DS	8.00 8.40	8.00	20	26	36/10	>50
8.55				8.55	8.55	30/0	Refusal	to penetration	
		Borehole terminated at 8.55m depth.							



LOGGED BY :	GEOTECHNICAL ENGINEERING DIVISION NATIONAL BUILDING RESEARCH ORGANISATION	DATE

LOG OF BOREHOLE

SER No. 59

NAME OF PROJECT : Geotecnical Investigation for proposed Bridge Sites	Borehole: BH2
	ground elevation: 1.321m MSL
Location : Bridge No. 43/4 km Munamalwatta-Horana-Aluthgama Road.	depth of borehole: 6.45 m
Boring method : Wash boring / Commenced on : 02.11.1995	Water struck at GL - m
Drilling mud : Bentonite / Completed on : 02.11.1995	GWL on completion of borehole: GL- 0.60 m

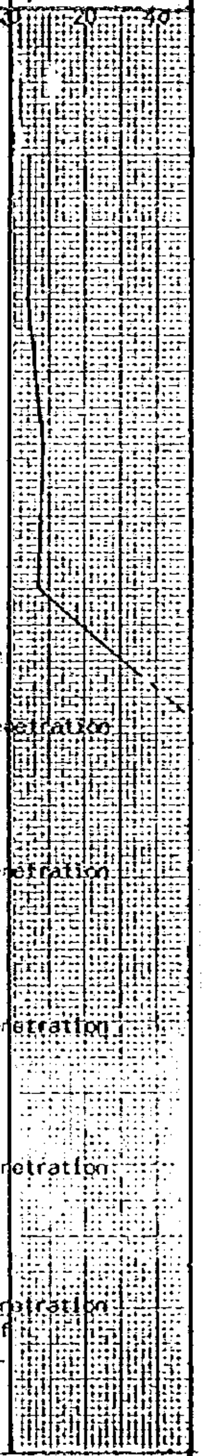
Depth below GL m		Classification & Description of Soil	Type and Depth of Sampling m		STANDARD PENETRATION TEST DATA depth tested GL m	number of blows per 15cm 1	2	3	N-value for 30cm	graphical presentation
0.00	ML	Dark brown silt with fine sand and traces of clay.								0 20 40
0.60										
1.00	ML	Very loose, greyish brown clayey silt with traces of sand and gravel.	DS	1.00 1.65	1.00	1	0	0	0	
1.75										
2.00	Pt OH	Very soft, blackish grey peat with partially decomposed vegetation and pockets of organic clay.	DS	2.00 2.70	2.00	1	0	0	0	
3.00			DS	3.00 3.50	3.00	1	0	1	1	
4.00			DS	4.00 4.50	4.00	1	0	1	1	
4.45										
5.00	SP	Loose, blackish grey fine to medium sand.	DS	5.00 5.45	5.00	2	3	2	5	
6.00			DS	6.00 6.45	6.00	1	3	4	7	
6.45										
		Borehole terminated at 6.45m depth.								

LOGGED BY :	GEOTECHNICAL ENGINEERING DIVISION NATIONAL BUILDING RESEARCH ORGANISATION	DATE :

LOG OF BOREHOLE

SER No. 20

NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites	Borehole: BHI ground elevation 150.5[illegible] m
Location : Bridge No. 25/7km Mallawapitiya-Rambadagalle Road.	depth of borehole 13.00 m
Boring method : Rotary core drilling / Commenced on : 06.11.1995	Water struck at GL - m
Flushing medium: Water / Completed on : 07.11.1995	GWL on completion of borehole GL- 3.25 m

Depth below GL m		Classification & Description of Soil	Type and Depth of Sampling m		STANDARD PENETRATION TEST DATA depth tested GL m	number of blows per 15cm 1	 2	 3	N-value for 30cm	 graphical presentation
0.00	SC	Blackish brown, clayey sand with undecomposed vegetation. (fill)	DS	0.00 0.20	0.00	Auger sample				
0.70										
1.00			DS	1.00 1.45	1.00	1	3	1	4	
	SC	Very loose to loose, yellowish brown, medium to fine grained clayey sand.								
2.00			DS	2.00 2.50	2.00	1	2	2	4	
3.00			DS	3.00 3.48	3.00	3	4	4	8	
3.10	SC	Loose, brown, medium to fine grained clayey sand.								
4.00			DS	4.00 4.45	4.00	3	3	4	7	
4.10	SC	Loose, grey mottled brown, medium to fine grained clayey sand.								
4.98										
5.00				5.00 5.01	5.00	18/1	Refusal to penetration			
6.00				6.00 6.05	6.00	10/5	Refusal to penetration			
7.00		From 5.00m to 9.00m samples not recovered. Washings collected (Blackish silty sand with mica).		7.00 7.05	7.00	18/5	Refusal to penetration			
8.00				8.00 8.08	8.00	48/8	Refusal to penetration			
9.00				9.00 9.05	9.00	20/5	Refusal to penetration			
9.15					RQD	Core Recovery%		Return of Water%		
		Moderatly weathered, hornblend biotite gneiss - Medium rock. (Highly weathered along joints).		9.15 10.15	10	100		100		

LOGGED BY : Deepthika Herath	GEOTECHNICAL ENGINEERING DIVISION NATIONAL BUILDING RESEARCH ORGANISATION	DATE :

LOG OF BOREHOLE

SER No. 20

NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites.

Borehole: BH1

ground elevation 150.320 m

Location : Bridge No. 25/7km Mallawapitiya-Rambadagalle Road

depth of borehole 13.00 m

Boring method : Rotary core drilling

Commenced on : 06.11.1995

Water struck at GL - m

Flushing medium: Water

Completed on : 07.11.1995

GWL on completion of borehole GL- 3.25 m

Depth below GL m	Classification & Description of Soil	Type and Depth of Sampling m	STANDARD PENETRATION TEST DATA					
			depth tested GL m	number of blows				
				per 15cm			N-value	
				1	2	3	for 30cm	graphical presentation
10.00			RQD	Core Recovery%		Return of water%		0 20 40
10.15	Fresh to slightly weathered, hornblend biotite gneiss - Hard rock.	10.15 11.15	82	94		100		
11.00								
11.15	Fresh (moderatly weathered along joints) hornblend biotite gneiss - Hard rock.	11.15 11.95	56	100		100		
11.95	Slightly weathered, highly jointed, hornblend biotite gneiss - Medium rock.	11.95 13.00	62	95		100		
12.00								
12.30	Highly to completly weathered, bornblend biotite gneiss.							
12.45								
12.65	Slightly to moderatly weathered quartz, feldspar rich layer with charnockitic appearence.							
13.00	Slightly weathered (moderatly weathered along joints) hornblend biotite gneiss.							
	Borehole terminated at 13.00m depth.							

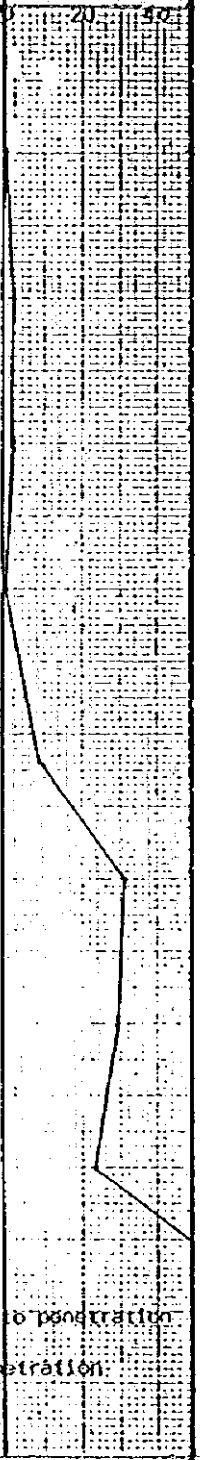
LOGGED BY : Deepthika Herath

GEOTECHNICAL ENGINEERING DIVISION
NATIONAL BUILDING RESEARCH ORGANISATION

DATE :

LOG OF BOREHOLE SBR No. 70

NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites	Borehole: BH1
	ground elevation: 0.859 MSL
Location : Bridge No. 3/6km Moratuwa, Piliyandala Road.	depth of borehole: 14.25 m
Boring method : Rotary core drilling / Commenced on : 30.10.1995	Water struck at GL - m
Flushing medium: Water / Completed on : 01.11.1995	GWL on completion of borehole: GL- 0.35 m

Depth below GL m		Classification & Description of Soil	Type and Depth of Sampling m		STANDARD PENETRATION TEST DATA					
					depth tested GL m	number of blows				
						per 15cm			N-value	
						1	2	3	for 30cm	graphical presentation
0.00										
1.00			DS	1.00 1.60	1.00	1	0	0	0	
	MH	Very loose reddish brown clayey silt with traces of sand and occasional gravel.								
2.00			DS	2.00 2.45	2.00	1	1	1	2	
2.89										
3.00			DS	3.00 3.60	3.00	1	2	0	2	
4.00	OH	Very soft, blackish grey organic clay.	DS	4.00 4.70	4.00	1	0	0	0	
			UDS	4.75 5.15						
5.00										
5.20				5.20 5.65	5.20	1	5	4	9	
	ML	Loose, grey, silt with traces of fine sand.								
5.90										
6.00	SM	Dense to medium dense,grey silty fine sand with pockets of organic silt.	DS	6.00 6.45	6.00	1	10	21	31	
7.00			DS	7.00 7.45	7.00	8	7	21	28	
7.50										
	SP	Medium dense light grey fine to medium sand.								
8.00			DS	8.00 8.45	8.00	6	13	10	23	
8.50										
	SM	Very dense blueish grey silty fine sand.								
9.00			DS	9.00 9.25	9.00	20	32/10	Refusal to penetration		
				9.50	9.50	38/5	Refusal to penetration			
9.60		Highly weathered charnockite [Highly fractured rock.]		9.55 9.60 10.70	RQD%	Core recovery%		Return of water%		
					Nil	30		100		

LOGGED BY :	GEOTECHNICAL ENGINEERING DIVISION NATIONAL BUILDING RESEARCH ORGANISATION	DATE :

LOG OF BOREHOLE

SER No. 70

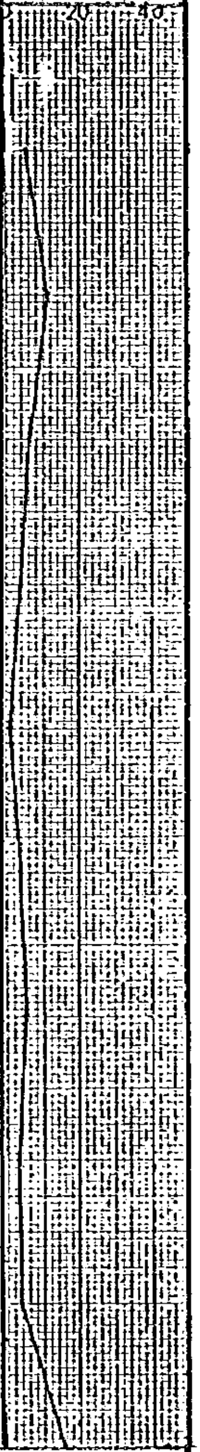
NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites		Borehole	BH1
		ground elevation	0.859 MSL
Location : Bridge No. 3/6km Moratuwa, Piliyandala Road		depth of borehole	14.25 m
Boring method : Rotary core drilling	Commenced on : 30.10.1995	Water struck at GL -	m
Flushing medium: Water	Completed on : 01.11.1995	GWL on completion of borehole	GL- 0.35 m

Depth below GL m	Classification & Description of Soil	Type and Depth of Sampling m	STANDARD PENETRATION TEST DATA					
			depth tested GL m	number of blows				
				per 15cm			N-value	
				1	2	3	for 30cm	graphical presentation
10.00			RQD%	Core Recovery%		Return of water%		0 20 40
	Highly weathered ,highly fractured charnockitic rock.	10.70 11.46	Nil	28		100		
11.00								
11.46	Hard ,highly fractured charnockitic rock with some garnet.	11.46 11.75	Nil	100		100		
12.00		11.75 11.85	Nil	100		100		
		11.85 12.05	Nil	80		100		
		12.05 12.65	Nil	80		100		
13.00		12.65 12.90	Nil	58		100		
		12.90 13.25	25	98		100		
14.00 14.25		13.25 13.55	Nil	100		100		
	Borehole terminated at 14.25m depth.	13.55 14.00	56	100		100		
		14.00 14.25	48	100		100		

LOGGED BY :	GEOTECHNICAL ENGINEERING DIVISION NATIONAL BUILDING RESEARCH ORGANISATION	DATE :

LOG OF BOREHOLE SER No. 7

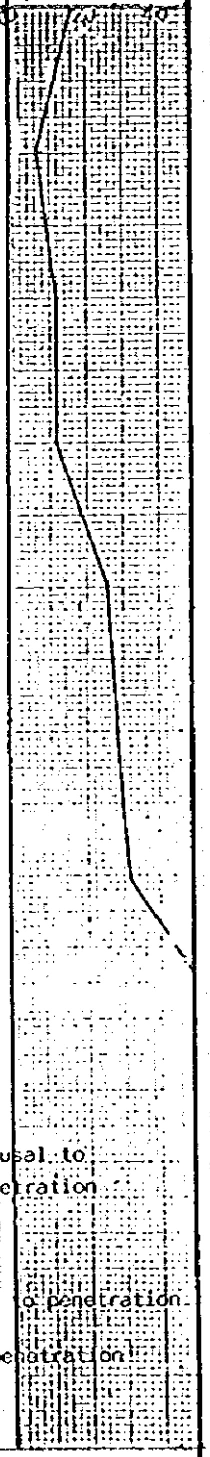
NAME OF PROJECT : Geotechnical Investigation for proposed Bridges Sites.		Borehole	BH1
		ground elevation	1.918m MSL
Location : Bridge No. 20/4 km Tudella-Pamunugama-Talahena-Negombo Rd.		depth of borehole	19.35 m
Boring method : Wash boring	Commenced on : 28.10.1995	Water struck at GL -	m
Drilling mud : Bentonite	Completed on : 30.10.1995	GWL on completion of borehole	GL- 1.65 m

Depth below GL m		Classification & Description of Soil	Type and Depth of Sampling m		depth tested GL m	STANDARD PENETRATION TEST DATA				
						number of blows				
						per 15cm			N-value	
						1	2	3	for 30cm	graphical presentation
0.00 0.60	GM	Reddish brown, silty gravel with sand.	DS	0.00 0.20	0.00	Auger sample				
	ML GM	Reddish, sandy silt with gravel and sand.								
1.00 1.20			DS	1.00 1.45	1.00	1	2	3	5	
	SM	Loose, greenish grey, medium to fine grained silty sand.								
2.00 2.30			DS	2.00 2.45	2.00	2	3	8	11	
	SM	Medium dense, grey, medium to fine grained silty sand.								
3.00			DS	3.00 3.45	3.00	3	3	3	6	
	SP	Loose, yellowish brown, coarse medium to fine grained sand with sea shells.								
4.00			DS	4.00 4.45	4.00	2	2	2	4	
5.00	SM	Loose to very loose, greenish grey fine grained silty sand.	DS	5.00 5.45	5.00	1	0	1	1	
6.00		Sample not recovered.		6.00 6.50	6.00	2	2	2	4	
7.00			DS	7.00 7.45	7.00	1	2	3	5	
	ML	Loose, grey, sandy silt.								
8.00 8.30	OH	Soft blackish, organic clay.	DS	8.00 8.50	8.00	1	1	2	3	
	Pt SM	Black peat with traces of sand.								
9.00			DS	9.00 9.45	9.00	1	2	2	4	
	SM	Very loose to loose, dark grey, medium to fine grained silty sand.								

LOGGED BY : Deepthika Herath	GEOTECHNICAL ENGINEERING DIVISION NATIONAL BUILDING RESEARCH ORGANISATION	DATE :

LOG OF BOREHOLE SER No. 7

NAME OF PROJECT : Geotechnical Investigation for proposed Bridge Sites
Borehole BH1
ground elevation 1.918m MSL
Location : Bridge No. 20/4 km Tudella-Pamunugama-Talahena-Negombo Rd.
depth of borehole 19.35 m
Boring method : Wash boring
Commenced on : 28.10.1995
Water struck at GL - m
Drilling mud : Bentonite
Completed on : 30.10.1995
GWL on completion of borehole GL- 1.65 m

Depth below GL m		Classification & Description of Soil	Type and Depth of Sampling m		depth tested GL m	STANDARD PENETRATION TEST DATA				
						number of blows				
						per 15cm			N-value	
						1	2	3	for 30cm	graphical presentation
10.00	SM	Medium dense, yellow mottled grey, medium to fine grained silty sand with plastic fines.	DS	10.00 10.45	10.00	6	7	9	16	
11.00	CH	Medium stiff, yellow mottled grey, high plasticity clay.	DS UDS	11.00 11.45 11.50 11.95	11.00	2	3	4	7	
12.00			DS	12.00 12.45	12.00	3	6	6	12	
13.00	CH	Stiff, yellow mottled grey, high plasticity clay with traces of sand.	DS	13.00 13.45	13.00	3	4	8	12	
14.00			DS	14.00 14.45	14.00	5	10	16	26	
15.00	SM	Medium dense to dense brownish to greyish fine grained silty sand.	DS	15.00 15.45	15.00	5	11	17	28	
16.00			DS	16.00 16.45	16.00	8	12	20	32	
17.00			DS	17.00 17.45	17.00	14	18	36	54	
	SM	Very dense, greyish yellow medium to fine grained silty sand.								
18.00			DS	18.00 18.35	18.00	25	36	16/5	Refusal to penetration	
	SM SP	Very dense, yellowish coarse medium to fine grained sand.								
19.00		Sample not recovered.		19.00 19.29	19.00	44	59/14	Refusal to penetration		
19.35				19.35	19.35	30/0	Refusal to penetration			
		Borehole terminated at 19.35m depth.								

LOGGED BY : Duptk.. Hend..

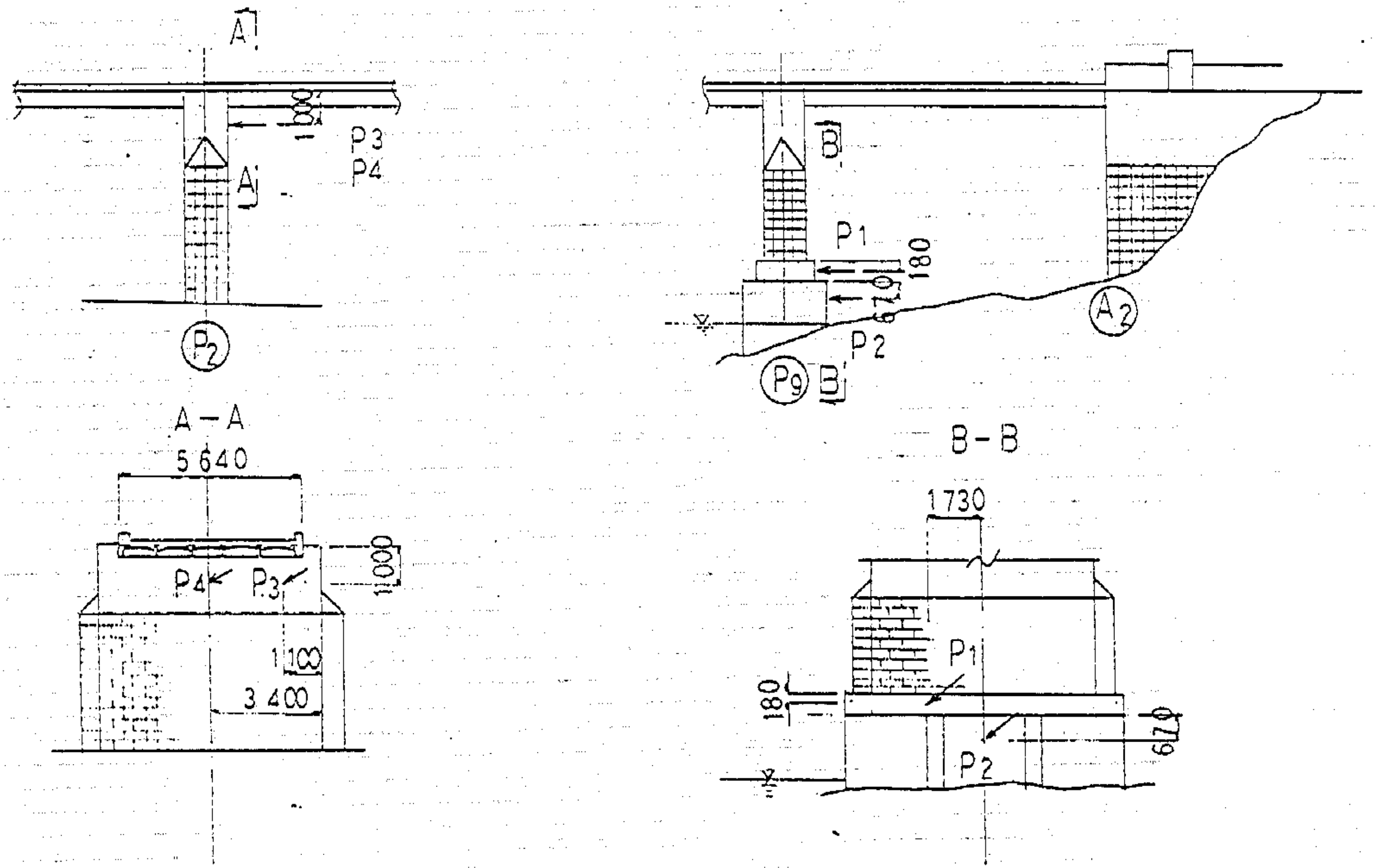
GEOTECHNICAL ENGINEERING DIVISION
NATIONAL BUILDING RESEARCH ORGANISATION

DATE :

Appendix - I

SCHMIDT HAMMER TEST RESULTS

POSITION OF SCHMIDT HAMMER TEST for NO. 77



SCHMIDT HAMMER TEST

SER No. 77		Route No AA019		Bridge No. 3/2 km		Year of Completion 1869					Date of Survey 07-11-1995				
No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
P-1 (P9)	41	38	36	39	41	39	0	0	39	0.63	323	203	37	23	
	44	45	35	38	39										
	42	37	40	34	40										
	37	37	35	44	38										
	34	44	35	34	41										
P-2 (P9)	28	29	24	44	45	35	0	0	35	0.63	271	171	31	20	
	26	52	35	32	35										
	33	41	34	39	35										
	33	37	39	47	48										
	28	35	34	30	31										(297)
P-3 (P2)	41	44	46	42	40	44	0	0	44	0.63	388	244	45	28	
	55	35	48	44	52										
	42	33	34	50	55										
	45	43	45	44	40										
	46	36	44	43	47										
P-4 (P2)	45	43	45	52	43	47	0	0	47	0.63	427	269	49	31	
	52	45	47	42	41										
	37	47	43	46	50										
	59	47	40	55	46										
	54	57	52	44	46										(407)

P-1, P-2

(Footing, Caisson)

→

P-3, P-4

(Pier)

→

R = Average Rebound Value

α = Impact Angle (deg.)

ΔR = Adjusting Value in α

R0 = Standard Value

αn = Coefficient of Concrete Age : more than days

σc = Compressive Strength at Present (kg/cm^2)

$\sigma c = (-184 + 13\,R0)$

σck = Compressive Strength at 28 days (kg/cm^2)

$\sigma ck = \alpha n \times \sigma c$

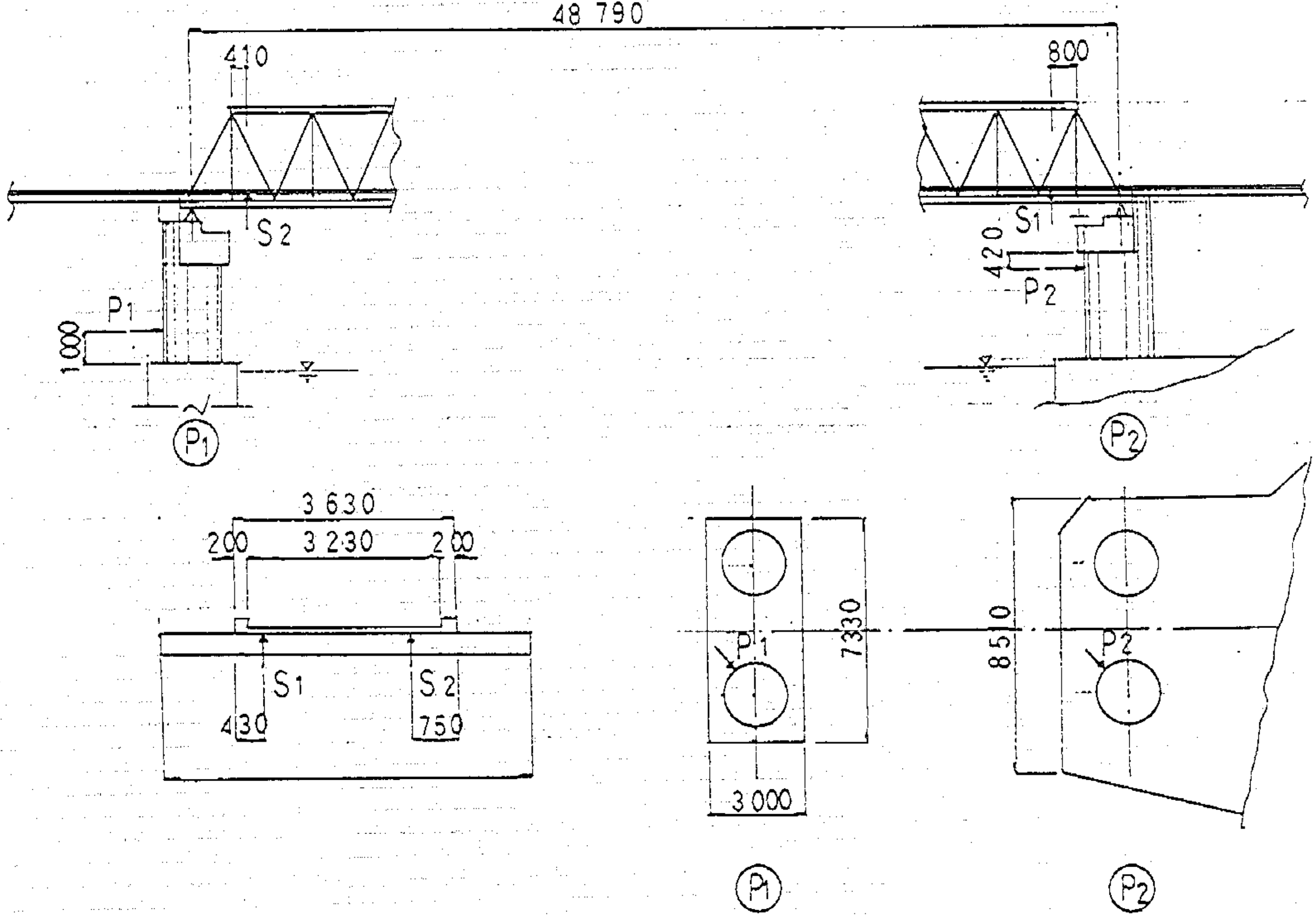
fc = Compressive Strength at Present (N/mm^2)

$fc = \sigma c \times 9.8/100/0.85$

fcu = Compressive Strength at 28 days (N/mm^2)

$fcu = \alpha n \times fc$

POSITION OF SCHMIDT HAMMER TEST for NO.33



SCHMIDT HAMMER TEST

SER No.	Route No.	Bridge No.	Year of Completion	Date of Survey
33	B157	12/3 km	1945	13-11-1995

No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
	60	58	58	60	62						kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
	60	62	59	62	58										
S-1	62	59	56	62	62	60	+90	-4	56	0.63	544	343	63	40	
	59	60	60	60	58										
	62	61	62	58	60										
	61	62	61	59	58										
	60	61	61	58	60										
S-2	61	59	59	61	60	60	+90	-4	56	0.63	544	343	63	40	
	62	62	62	60	64										
	58	60	62	59	62										(544)

S-1, S-2

(RC SLAB)

↑

R = Average Rebound Value

α = Impact Angle (deg.)

ΔR = Adjusting Value in α

R0 = Standard Value

αn = Coefficient of Concrete Age : more than days

σc = Compressive Strength at Present (kg/cm^2)

σc = (-184 + 13 R0)

σck = Compressive Strength at 28 days (kg/cm^2)

σck = αn x σc

fc = Compressive Strength at Present (N/mm^2)

fc = σc x 9.8/100/0.85

fcu = Compressive Strength at 28 days (N/mm^2)

fcu = αn x fc

SCHMIDT HAMMER TEST

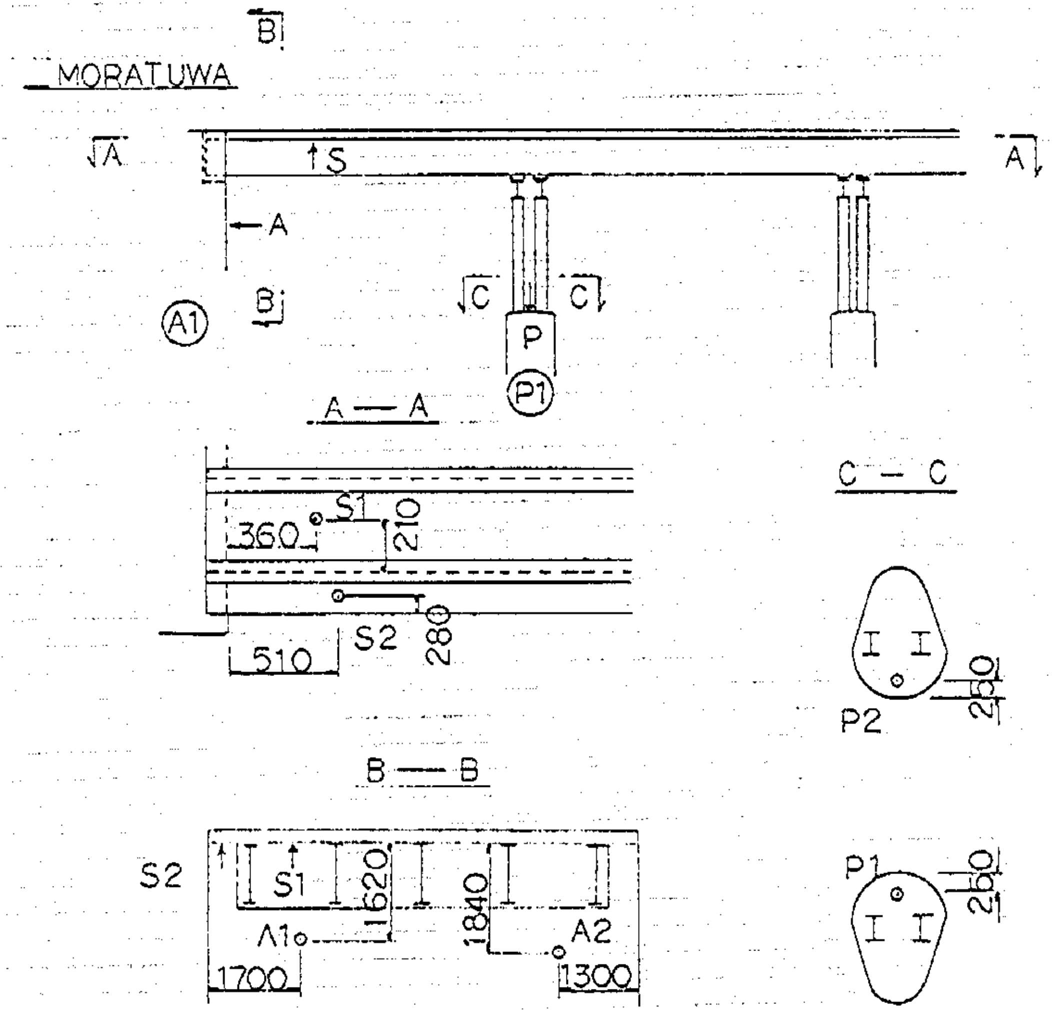
SER No.	Route No	Bridge No.	Year of Completion	Date of Survey
33	B157	12/3 km	1945	13-11-1995

No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
	55	56	50	36	43										
	49	39	42	51	40										
P-1	41	40	48	46	36	43	0	0	43	0.63	375	236	43	27	
	44	41	52	36	50										
	32	37	38	35	50										
	42	40	41	40	46										
	44	43	46	42	42										
P-2	40	48	43	45	48	43	0	0	43	0.63	375	236	43	27	
	46	40	42	40	46										
	40	44	48	46	43										(375)

P-1, P-2
(PIER)
→

R = Average Rebound Value
α = Impact Angle (deg.)
ΔR = Adjusting Value in α
R0 = Standard Value
αn = Coefficient of Concrete Age : more than days
σc = Compressive Strength at Present (kg/cm^2)
σc = (-184 + 13 R0)
σck = Compressive Strength at 28 days (kg/cm^2)
σck = αn x σc
fc = Compressive Strength at Present (N/mm^2)
fc = σc x 9.8/100/0.85
fcu = Compressive Strength at 28 days (N/mm^2)
fcu = αn x fc

POSITION OF SCHMIDT HAMMER for NO.70



SCHMIDT HAMMER TEST

SER No. 70		Route No B295		Bridge No. 3/6 km			Year of Completion 1960				Date of Survey 30-10-1995				
No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
	49	48	50	56	51										
	51	54	58	49	47										
S-1	48	50	55	52	45	50	+90	-4	46	0.63	414	261	47	30	
	52	55	48	45	46										
	50	50	50	46	46										
	58	51	54	58	35										
	51	54	57	57	64										
S-2	48	58	58	58	45	54	+90	-4	50	0.63	466	294	54	34	
	44	59	57	56	52										
	51	54	58	58	48										(440)
	38	31	35	40	36										
	37	42	35	38	34										
A-1	41	45	38	38	38	38	0	0	38	0.63	310	195	36	22	
	36	44	44	36	36										
	46	38	36	40	34										
	31	34	32	32	36										
	38	33	33	37	38										
A-2	33	30	40	37	39	35	0	0	35	0.63	271	171	31	20	
	36	37	40	28	41										
	33	28	29	32	40										(290)
	41	44	46	44	44										
	45	44	48	44	44										
P-1	43	45	48	47	44	45	-90	+3	48	0.63	440	277	51	32	
	41	43	51	51	45										
	44	40	49	46	44										
	41	35	46	50	44										
	44	48	48	46	52										
P-2	45	51	43	50	44	47	-90	3	50	0.63	466	294	54	34	
	45	52	40	52	51										
	43	46	50	52	50										(453)

R = Average Rebound Value

α = Impact Angle (deg.)

ΔR = Adjusting Value in α

R0 = Standard Value

αn = Coefficient of Concrete Age : more than days

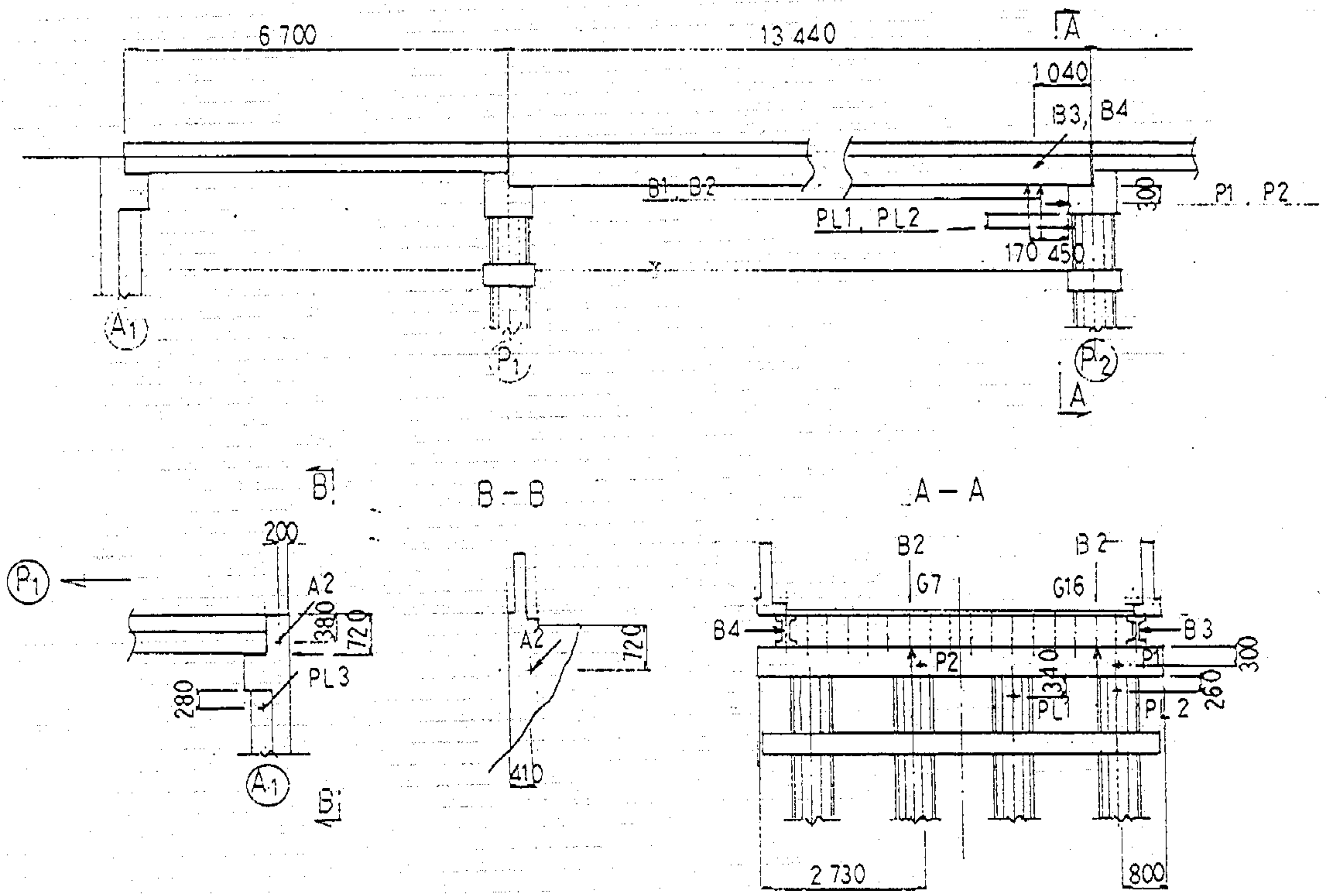
σc = Compressive Strength at Present (kg/cm^2)
$\sigma c = (-184 + 13\ R0)$

σck = Compressive Strength at 28 days (kg/cm^2)
$\sigma ck = \alpha n \times \sigma c$

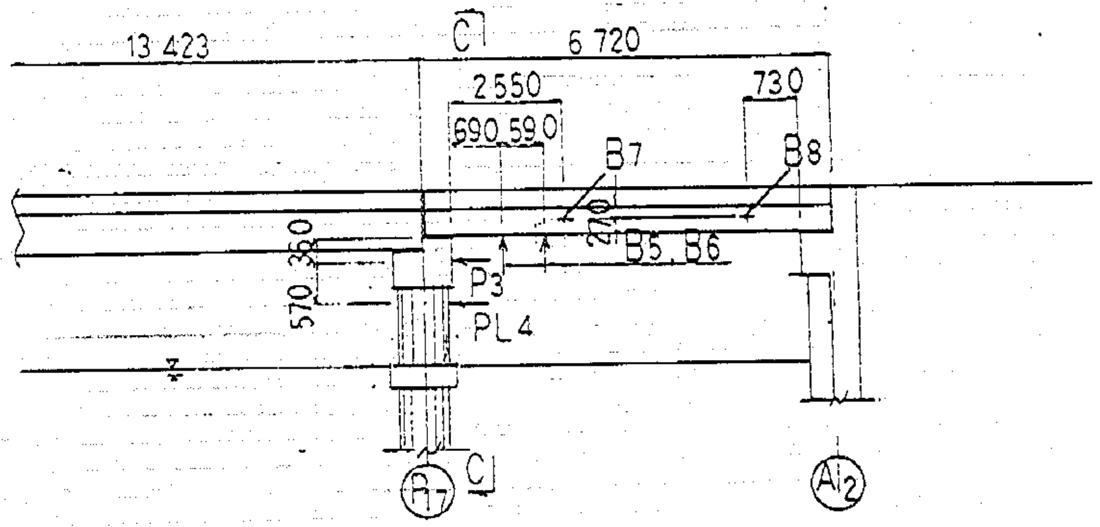
fc = Compressive Strength at Present (N/mm^2)
$fc = \sigma c \times 9.8/100/0.85$

fcu = Compressive Strength at 28 days (N/mm^2)
$fcu = \alpha n \times fc$

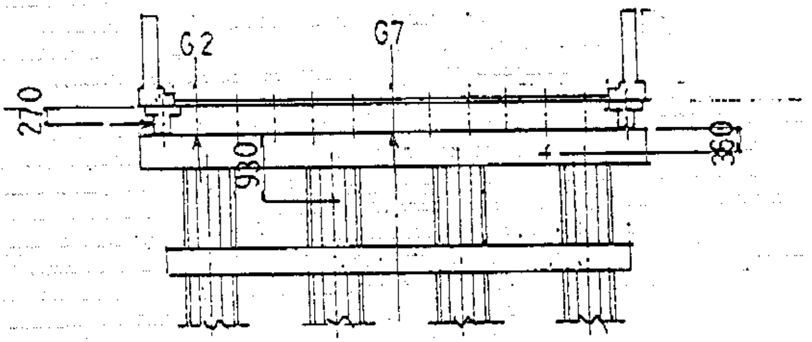
POSITION OF SCHMIDT HAMMER TEST for NO.7 (1)



POSITION OF SCHMIDT HAMMER TEST for NO.7 (2)



C – C



SCHMIDT HAMMER TEST

SER No. 7	Route No. B425		Bridge No. 20/4 km				Year of Completion 1960				Date of Survey 08-11-1995				
No.	Rebound Value					R	α	ΔR	R0	αn	σc kg/cm^2	σck kg/cm^2	fc N/mm^2	fcu N/mm^2	Remark
B-1 (PC)	57	57	57	55	55	55	+90	-4	51	0.63	479	302	55	35	
	51	54	56	58	54										
	50	56	52	58	58										
	52	60	58	55	54										
	50	58	54	52	50										
B-2 (PC)	55	62	54	54	53	57	+90	-4	53	0.63	505	318	58	37	
	60	59	60	59	57										
	53	59	63	61	55										
	58	56	56	56	54										
	54	54	61	59	59										
B-3 (PC)	56	58	52	56	58	53	0	0	53	0.63	505	318	58	37	
	54	52	50	55	56										
	56	54	54	54	54										
	52	54	54	48	52										
	50	50	48	48	48										
B-4 (PC)	54	56	54	52	50	51	0	0	51	0.63	479	302	55	35	
	52	52	48	49	50										
	49	49	53	52	44										
	54	54	53	54	48										
	50	54	48	50	52										(492)

B-1, B-2
(PCB)
↑
B-3, B-4
(PCB)
→

R = Average Rebound Value
α = Impact Angle (deg.)
ΔR = Adjusting Value in α
R0 = Standard Value
αn = Coefficient of Concrete Age : more than days
σc = Compressive Strength at Present (kg/cm^2)
$\sigma c = (-184 + 13\ R0)$
σck = Compressive Strength at 28 days (kg/cm^2)
$\sigma ck = \alpha n \times \sigma c$
fc = Compressive Strength at Present (N/mm^2)
$fc = \sigma c \times 9.8/100/0.85$
fcu = Compressive Strength at 28 days (N/mm^2)
$fcu = \alpha n \times fc$

SCHMIDT HAMMER TEST

SER No.	Route No	Bridge No.	Year of Completion	Date of Survey
7	B425	20/4 km	1960	08-11-1995

No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm²	kg/cm²	N/mm²	N/mm²	
B-5 (RC)	54	60	59	54	58	58	+90	-4	54	0.63	518	326	60	38	
	60	61	54	64	61										
	60	57	63	61	57										
	54	56	55	59	48										
	58	60	61	61	44										
B-6 (RC)	60	59	58	56	60	57	+90	-4	53	0.63	505	318	58	37	
	54	60	58	58	56										
	58	60	57	54	54										
	56	54	50	55	57										
	56	57	56	59	54										
B-7 (RC)	47	45	30	42	44	48	0	0	48	0.63	440	277	51	32	
	44	42	44	52	54										
	42	50	53	49	48										
	45	55	57	53	48										
	47	44	51	46	50										
B-8 (RC)	42	46	45	50	43	45	0	0	45	0.63	401	253	46	29	
	42	50	42	42	44										
	52	54	46	50	49										
	42	46	40	42	42										
	46	41	41	43	44										(466)

B-5, B-6
(RCB)
↑
B-7, B-8
(RCB)
→

R = Average Rebound Value
α = Impact Angle (deg.)
ΔR = Adjusting Value in α
R0 = Standard Value
αn = Coefficient of Concrete Age : more than days
σc = Compressive Strength at Present (kg/cm²)
σc = (-184 + 13 R0)
σck = Compressive Strength at 28 days (kg/cm²)
σck = αn x σc
fc = Compressive Strength at Present (N/mm²)
fc = σc x 9.8/100/0.85
fcu = Compressive Strength at 28 days (N/mm²)
fcu = αn x fc

SCHMIDT HAMMER TEST

SER No.	Route No	Bridge No.	Year of Completion	Date of Survey
7	B425	20/4 km	1960	08-11-1995

No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
A-1 (A1)	38	45	45	54	52	48	0	0	48	0.63	440	277	51	32	
	40	47	52	48	43										
	35	51	52	51	48										
	42	52	39	50	44										
	55	51	53	53	49										
A-2 (A1)	44	47	42	43	43	42	0	0	42	0.63	362	228	42	26	
	48	44	45	32	41										
	43	40	36	43	38										
	47	43	35	38	40										
	41	42	38	39	38										(401)
P-1 (P2)	55	44	53	52	44	51	0	0	51	0.63	479	302	55	35	
	49	46	46	50	52										
	56	53	44	58	53										
	51	58	55	54	57										
	55	55	50	44	53										
P-2 (P2)	38	40	34	34	34	47	0	0	47	0.63	427	269	49	31	
	44	34	42	38	43										
	50	52	46	48	54										
	52	54	55	54	58										
	44	44	50	50	54										
P-3 (P17)	56	55	54	54	38	53	0	0	53	0.63	505	318	58	37	
	52	54	51	49	57										
	53	52	49	50	57										
	59	52	54	55	54										
	55	43	53	33	55										(470)

A-1, A-2
(ABUTMENT)
→
P-1, P-2, P-3
(PIER)
→

R = Average Rebound Value

α = Impact Angle (deg.)

ΔR = Adjusting Value in α

R0 = Standard Value

αn = Coefficient of Concrete Age : more than days

σc = Compressive Strength at Present (kg/cm^2)

$\sigma c = (-184 + 13 R0)$

σck = Compressive Strength at 28 days (kg/cm^2)

$\sigma ck = \alpha n \times \sigma c$

fc = Compressive Strength at Present (N/mm^2)

$fc = \sigma c \times 9.8/100/0.85$

fcu = Compressive Strength at 28 days (N/mm^2)

$fcu = \alpha n \times fc$

SCHMIDT HAMMER TEST

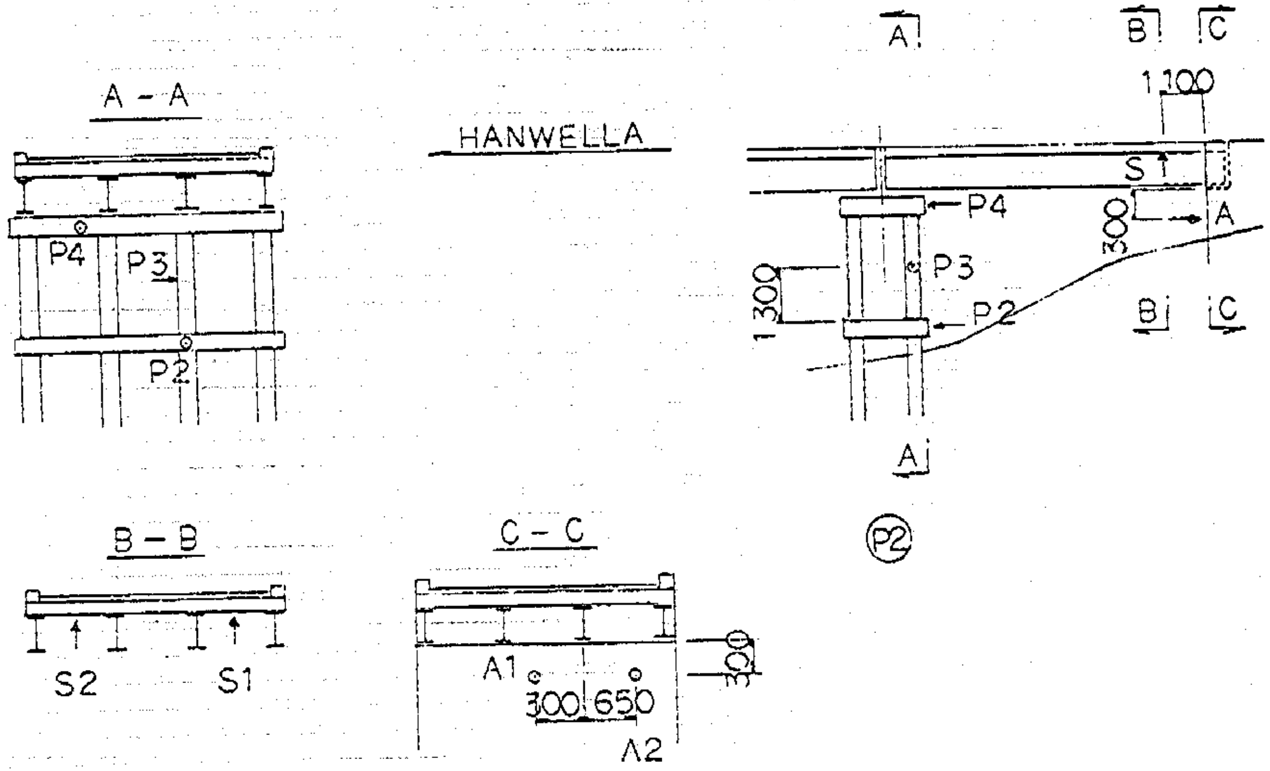
SER No.	Route No	Bridge No.	Year of Completion	Date of Survey
7	B425	20/4 km	1960	08-11-1995

No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
PL-1 (P2)	54	53	53	52	48	51	0	0	51	0.63	479	302	55	35	
	54	53	52	54	57										
	51	56	53	56	50										
	38	44	49	57	47										
	40	43	48	52	47										
PL-2 (P2)	44	48	44	52	42	44	0	0	44	0.63	388	244	45	28	
	50	42	47	38	42										
	44	48	42	48	38										
	44	48	40	40	42										
	42	47	42	40	42										
PL-3 (A1)	56	55	50	54	48	51	0	0	51	0.63	479	302	55	35	
	52	51	53	55	42										
	43	56	54	54	46										
	48	56	54	44	54										
	52	52	56	44	44										
PL-4 (P17)	52	55	50	52	53	51	0	0	51	0.63	479	302	55	35	
	55	53	49	54	54										
	56	54	46	53	46										
	50	52	46	55	51										
	42	52	50	54	50										(456)

PL-1 to PL-4
(RC PILE)
-->

R = Average Rebound Value
α = Impact Angle (deg.)
ΔR = Adjusting Value in α
R0 = Standard Value
αn = Coefficient of Concrete Age : more than days
σc = Compressive Strength at Present (kg/cm^2)
$\sigma c = (-184 + 13 R0)$
σck = Compressive Strength at 28 days (kg/cm^2)
$\sigma ck = \alpha n \times \sigma c$
fc = Compressive Strength at Present (N/mm^2)
$fc = \sigma c \times 9.8/100/0.85$
fcu = Compressive Strength at 28 days (N/mm^2)
$fcu = \alpha n \times fc$

POSITION OF SCHMIDT HAMMER for NO. 211



SCHMIDT HAMMER TEST

SER No.	Route No.	Bridge No.	Year of Completion	Date of Survey
211	B146	8/3 km	1942	28-10-1995

No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
	42	46	53	47	44						kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
	55	48	54	41	46										
S-1	45	42	56	49	53	48	+90	-4	44	0.63	388	244	45	28	
	43	44	54	51	47										
	47	46	46	50	50										
	56	49	46	48	46										
	56	50	48	46	50										
S-2	49	46	48	50	54	49	+90	-4	45	0.63	401	253	46	29	
	48	48	47	51	49										
	49	47	41	46	49										(394)

R = Average Rebound Value

α = Impact Angle (deg.)

ΔR = Adjusting Value in α

R0 = Standard Value

αn = Coefficient of Concrete Age : more than days

σc = Compressive Strength at Present (kg/cm^2)
σc = (-184 + 13 R0)

σck = Compressive Strength at 28 days (kg/cm^2)
σck = αn x σc

fc = Compressive Strength at Present (N/mm^2)
fc = σc x 9.8/100/0.85

fcu = Compressive Strength at 28 days (N/mm^2)
fcu = αn x fc

SCHMIDT HAMMER TEST

SER No. 211		Route No B146	Bridge No. 8/3 km				Year of Completion 1942				Date of Survey 28-10-1995				
No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
A-1	27	43	44	42	38	38	0	0	38	0.63	310	195	36	22	
	43	41	36	46	40										
	33	40	37	41	36										
	46	34	35	33	40										
	34	26	38	33	42										
A-2	34	35	37	30	30	37	0	0	37	0.63	297	187	34	22	
	39	44	40	40	40										
	30	40	45	42	42										
	31	39	40	31	35										
	32	41	40	41	27										(303)
P-2	28	26	20	37	24	28	0	0	28	0.63	180	113	21	13	
	35	32	24	18	26										
	26	32	25	26	22										
	26	66	29	31	24										
	21	27	38	30	27										
P-3	29	34	30	29	31	31	0	0	31	0.63	219	138	25	16	
	29	31	36	32	29										
	34	40	35	28	29										
	29	35	33	35	36										
	26	29	30	30	33										
P-4	28	25	29	32	26	34	0	0	34	0.63	258	163	30	19	
	38	36	32	40	35										
	42	38	32	40	38										
	39	45	28	32	35										
	31	28	28	39	41										(219)

R = Average Rebound Value

α = Impact Angle (deg.)

ΔR = Adjusting Value in α

R0 = Standard Value

αn = Coefficient of Concrete Age : more than days

σc = Compressive Strength at Present (kg/cm^2)

$\sigma c = (-184 + 13\ R0)$

σck = Compressive Strength at 28 days (kg/cm^2)

$\sigma ck = \alpha n \times \sigma c$

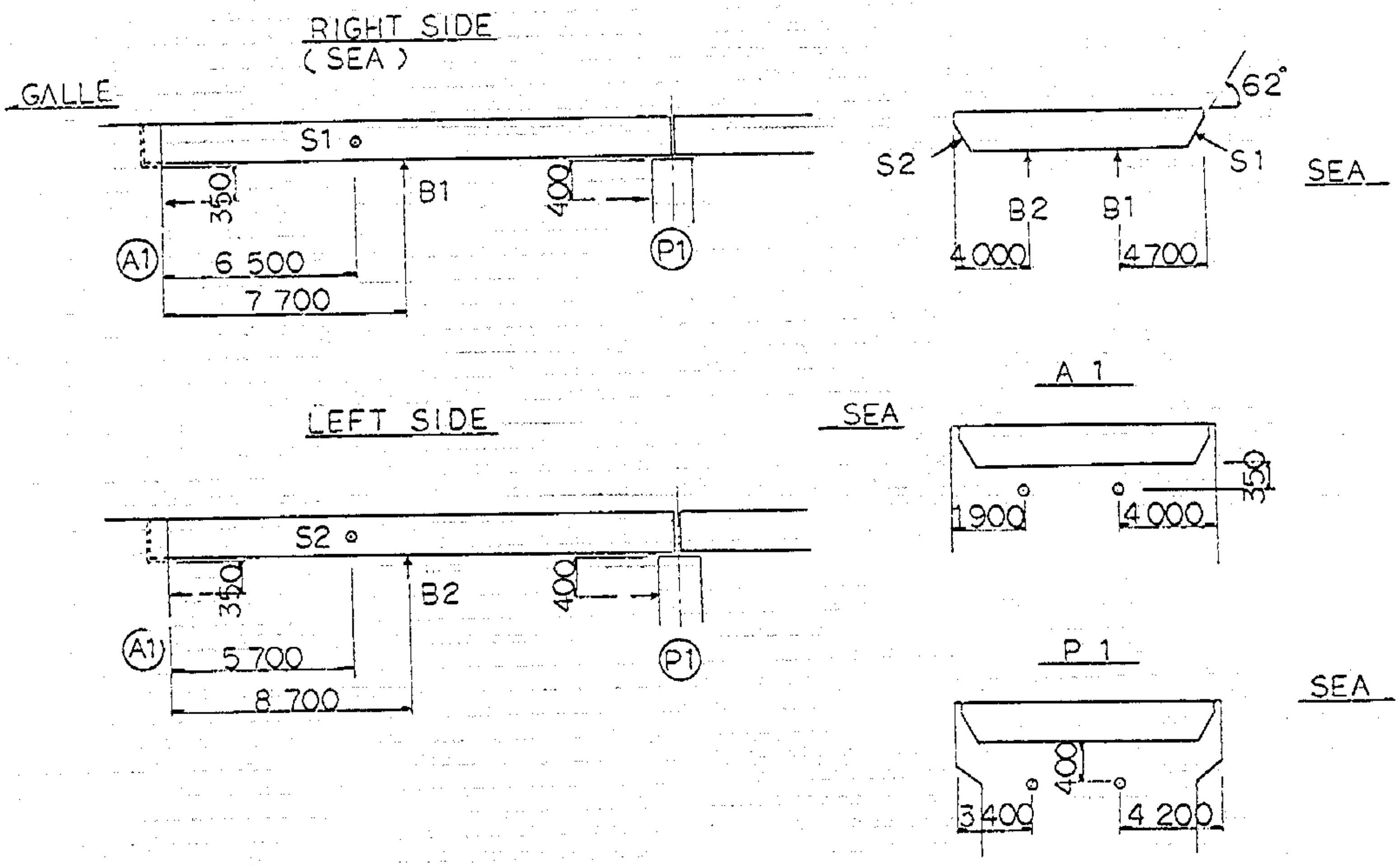
fc = Compressive Strength at Present (N/mm^2)

$fc = \sigma c \times 9.8/100/0.85$

fcu = Compressive Strength at 28 days (N/mm^2)

$fcu = \alpha n \times fc$

POSITION OF SCHMIDT HAMMER for NO. 212



SCHMIDT HAMMER TEST

SER No. 212	Route No AA002		Bridge No. 138/1 km				Year of Completion 1992				Date of Survey 25-10-1995				
No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm^2	kg/cm^2	N/mm^2	N/mm^2	
S-1	55	58	60	56	58	56	+60	-4	52	0.65	492	320	57	37	
	56	54	63	55	55										
	53	53	56	55	55										
	55	58	56	56	54										
	54	55	58	57	58										
S-2	56	54	54	56	51	55	+60	-4	51	0.65	479	311	55	36	
	64	56	51	53	61										
	57	58	54	57	56										
	61	54	53	52	54										
	57	52	52	58	54										
B-1	56	64	61	62	59	57	+90	-4	53	0.65	505	328	58	38	
	56	57	52	60	58										
	57	62	53	60	57										
	54	58	52	52	55										
	58	60	62	55	45										
B-2	54	59	48	59	59	54	+90	-4	50	0.65	466	303	54	35	
	54	60	52	62	60										
	57	56	53	59	52										
	54	50	53	50	43										
	50	54	48	48	54										(485)

S-1, S-2

SLAB = 60°

B-1, B-2

PC BEAM = 90°

↑

R = Average Rebound Value

α = Impact Angle (deg.)

ΔR = Adjusting Value in α

R0 = Standard Value

αn = Coefficient of Concrete Age : more than days

σc = Compressive Strength at Present (kg/cm^2)

$\sigma c = (-184 + 13 R0)$

σck = Compressive Strength at 28 days (kg/cm^2)

$\sigma ck = \alpha n \times \sigma c$

fc = Compressive Strength at Present (N/mm^2)

$fc = \sigma c \times 9.8/100/0.85$

fcu = Compressive Strength at 28 days (N/mm^2)

$fcu = \alpha n \times fc$

SCHMIDT HAMMER TEST

SER No.	Route No.	Bridge No.	Year of Completion	Date of Survey
212	AA002	138/1 km	1992	25-10-1995

No.	Rebound Value					R	α	ΔR	R0	αn	σc	σck	fc	fcu	Remark
											kg/cm²	kg/cm²	N/mm²	N/mm²	
A-1	41	43	43	36	44	49	0	0	49	0.65	453	294	52	34	
	56	56	52	56	51										
	54	55	57	55	40										
	49	51	51	49	46										
	45	49	48	46	52										
A-2	49	46	42	41	45	49	0	0	49	0.65	453	294	52	34	
	46	42	49	43	44										
	46	36	55	56	54										
	50	54	53	54	56										
	45	56	57	55	56										
P-1	44	47	51	52	54	45	0	0	45	0.65	401	261	46	30	
	47	48	55	52	58										
	42	44	30	40	43										
	32	43	52	43	35										
	44	41	44	43	41										
P-2	54	44	50	49	53	48	0	0	48	0.65	440	286	51	33	
	41	46	49	53	44										
	48	51	56	43	44										
	50	48	49	50	52										
	52	44	48	44	42										(420)

A-1, A-2
ABUTMENT = 0°
→

P-1, P-2
PIER = 0°
→

R = Average Rebound Value
α = Impact Angle (deg.)
ΔR = Adjusting Value in α
R0 = Standard Value
αn = Coefficient of Concrete Age : more than days
σc = Compressive Strength at Present (kg/cm²)
σc = (-184 + 13 R0)
σck = Compressive Strength at 28 days (kg/cm²)
σck = αn x σc
fc = Compressive Strength at Present (N/mm²)
fc = σc x 9.8/100/0.85
fcu = Compressive Strength at 28 days (N/mm²)
fcu = αn x fc

Appendix - J

LOADING TEST RESULTS

LIST OF CONTENTS

1. Scope of Bridges

The locations of the bridges tested are shown in Fig. 1.

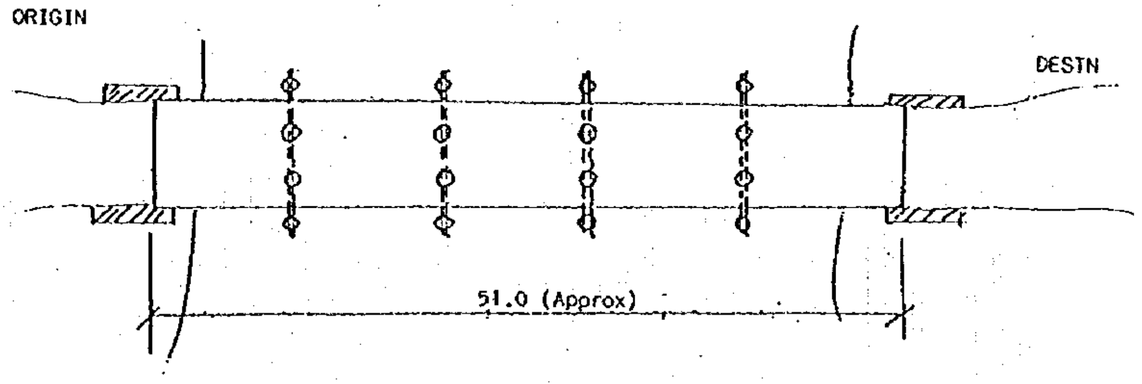
The particular of these bridges are summarized in Table 1.

Table 1 Bridges for Static Load Test

SER No.		59	211	212
Route No.		B 157	B 146	AA002
Bridge No.		43/4K	8/3K	138/1K
Traffic Volume	/ day	750	2620	4120
	year of census	1991	1994	1994
Year of Const.		1924	1942	1975
Type of Bridge	Superstructure	RSJ/BUC	RSJ/RCS	PSC/PRE
	Abutment	RSJ in Conc.	Conc.	RC Conc.
	Pier	RSJ in Conc.	Conc.	RC Conc.
Bridge Length	(m)	51	23.7	62.48
Bridge Width	(m)	3.2	3.5	10.4
No. of Span		5	3	3
Span Length	(m)	8.5	7.7 - 8.1	16.6

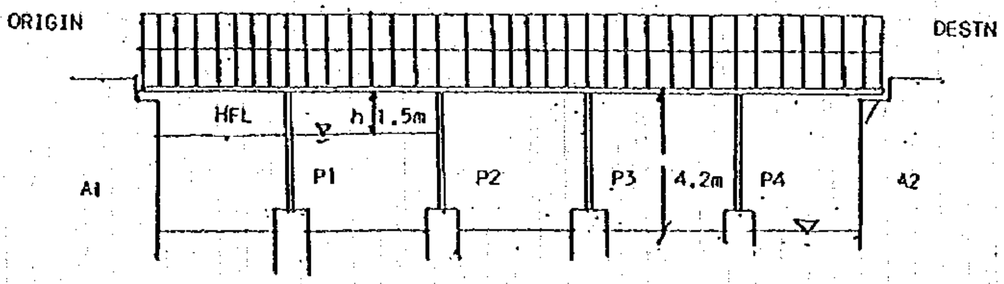
Sketches of bridges are shown on Figs. 1, 2 and 3.

Plan View



h H.F.L. =1.5m
N.F.L. = 2.1m
O.W.L. = 2.4m

Side Elevation



φ75 cm cylindrical mass concrete

Cross Section

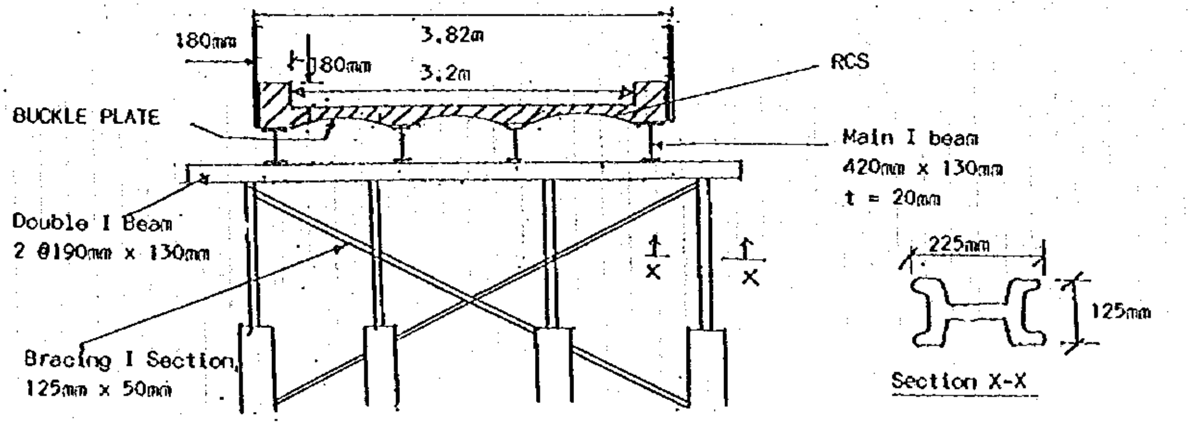
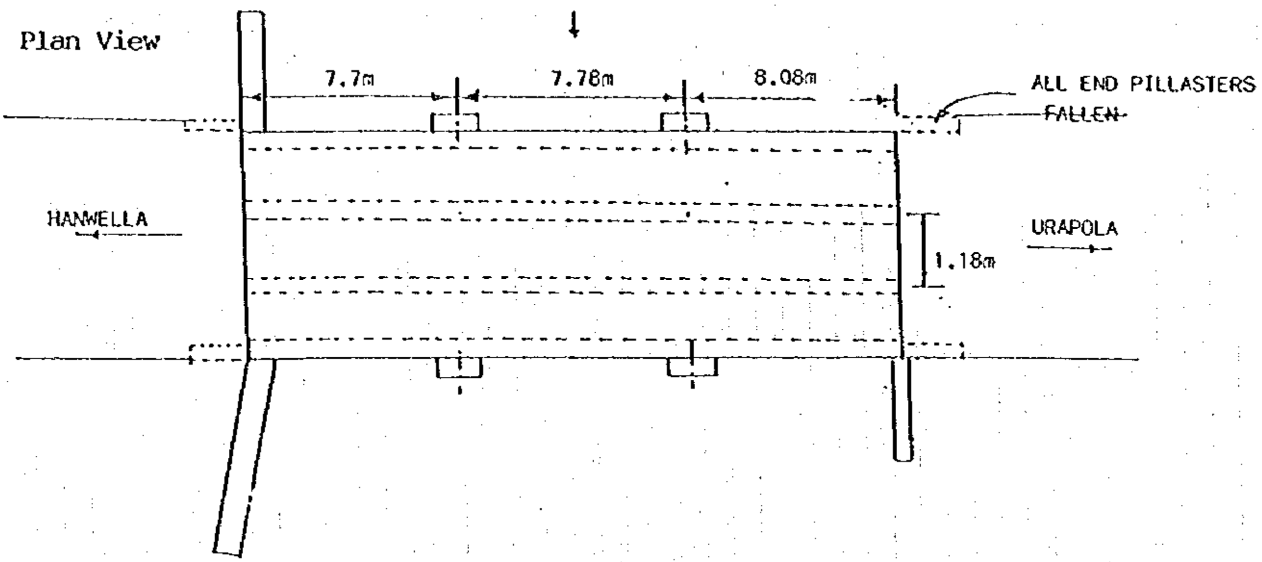
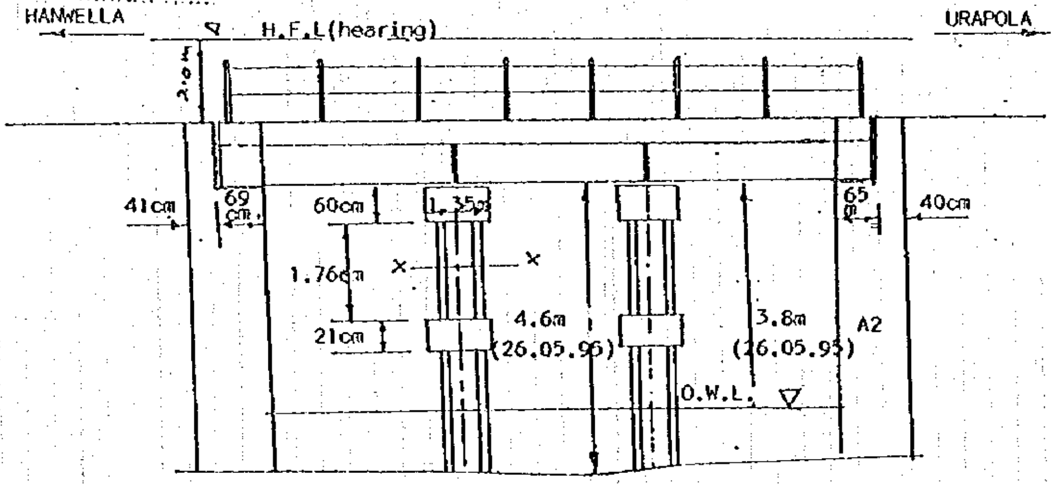


Fig. 1 Bridge SER NO. 59


Plan View


Side Elevation

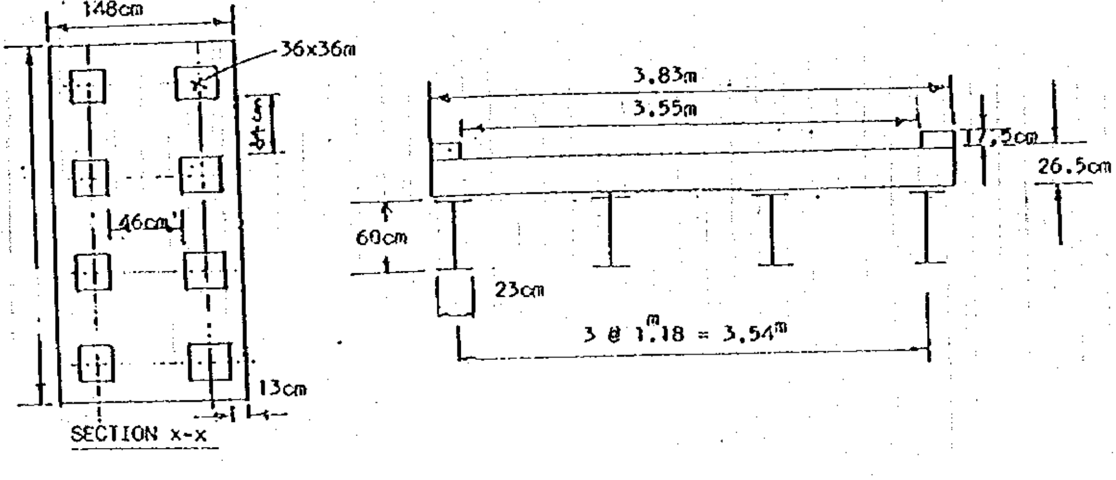
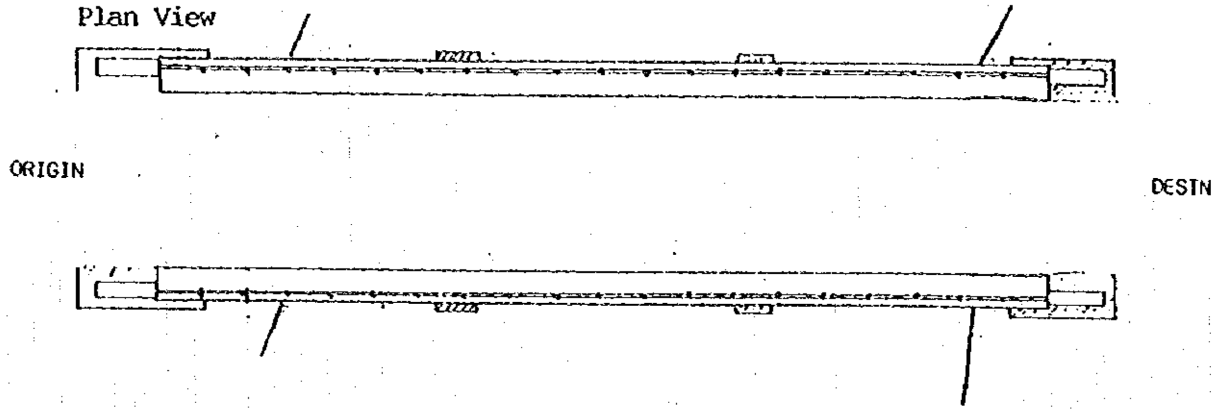
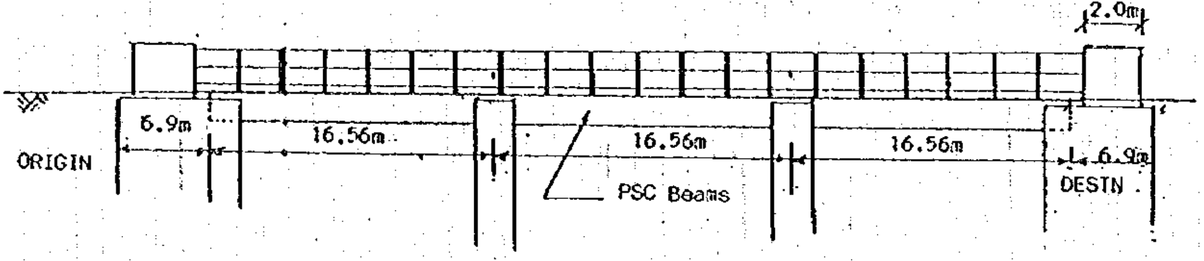

Cross Section

Fig. 2 Bridge SER NO. 211

Plan View



Side Elevation



Cross Section

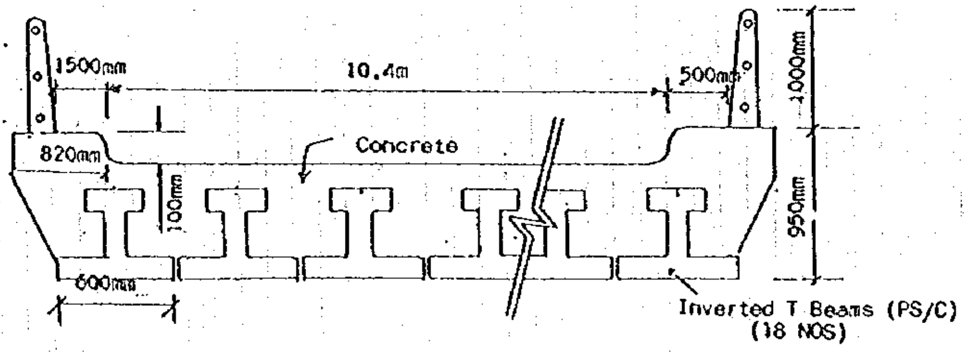


Fig.3 Bridge SER NO. 212

2. SCOPE OF WORK

2.1 Load Test

The static load test was carried out to evaluate bearing capacity and disintegration of girder of the bridges. This test was achieved by measure the sag of girder when load is applied on the bridge.

a) Apparatus

The apparatuses used for load test are listed below:

(1)	Dial Gauges	Maker	:	Peacock
		Model	:	307 G
		Accuracy	:	1/100 mm
(2)	Dead Load		:	Lorry with Load

b) Load

Two axles of Lorry with load (total weight 11 tons) was used as dead load. Two lorries were used for bridge SER No. 212 and one lorry for bridge SER No. 59 and No.211.
The test were carried out on three loading stages by changes of loading location as shown in Figs. 5, 7 and 9.
The weight of lorries were measured by portable scale before the load test.

c) Measuring Points

The measuring points are at center and at quarter half of the span using dial gauges as shown in Figs. 4, 6 and 8.
Extra gages were installed at pier 4 of the bridge of SER No. 59 as shown in Fig. 4.

d) Procedure

1 Set the measuring apparatus

Set supporters for dial gauge under the girder and set dial gauge on top of the supporters using magnet base.

2 Close traffic flow and read zero

Close the traffic flow at the both ends of the bridge and immediately read the dial gauges.

3 Apply load

Shift the lorry onto the proposed first location (Case 1) and read the dial gages after reading became stable. The process of Case 2 and Case 3 was repeated.

4 Remove load

Drive the lorry out of the bridge and read the dial gages.

5 Open traffic flow

Open the traffic flow

6 Retest

The test was repeated at bridge SER No.59 and No.211.

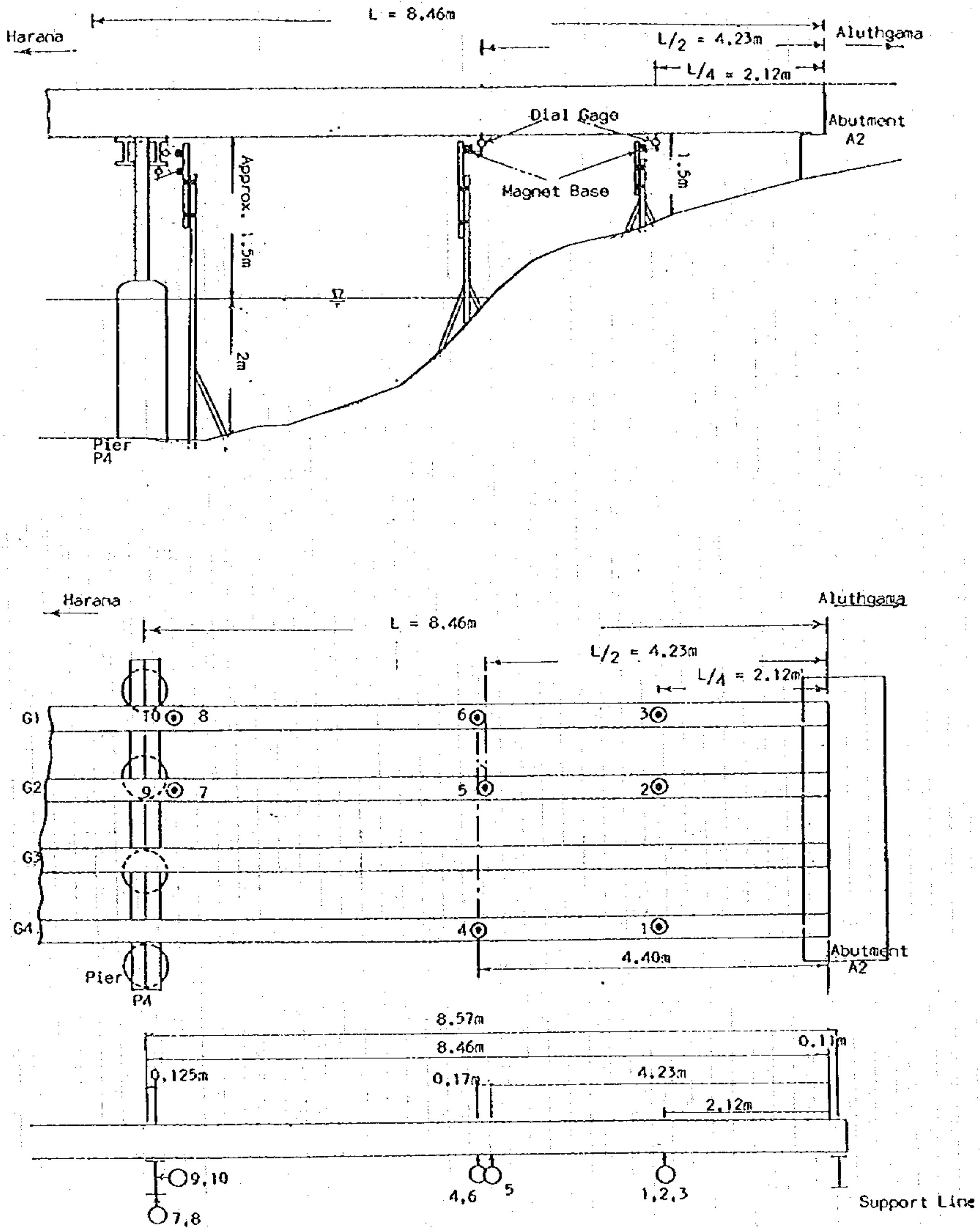
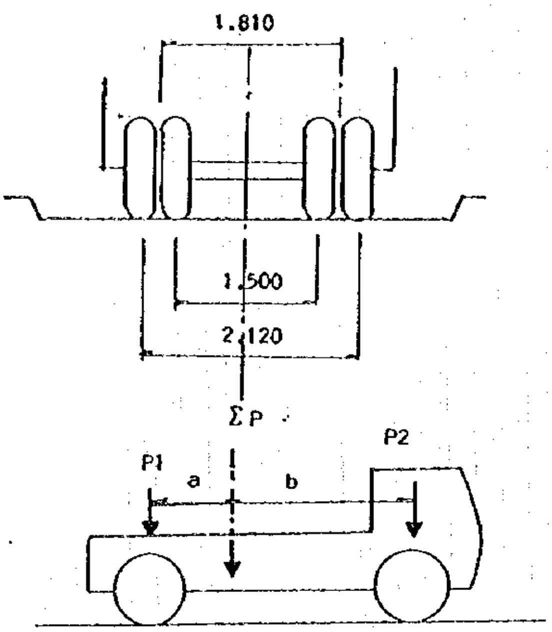


Fig. 4 Measuring Points Bridge SER 59



For Vehicle No: 43/8249

P1 + P2 = 10.93 ton
P1 > P2

$a = \frac{P2}{P1 + P2} \times 4.210 = 1.456^{m}$

$b = 4.210 - a = 2.754^{m}$

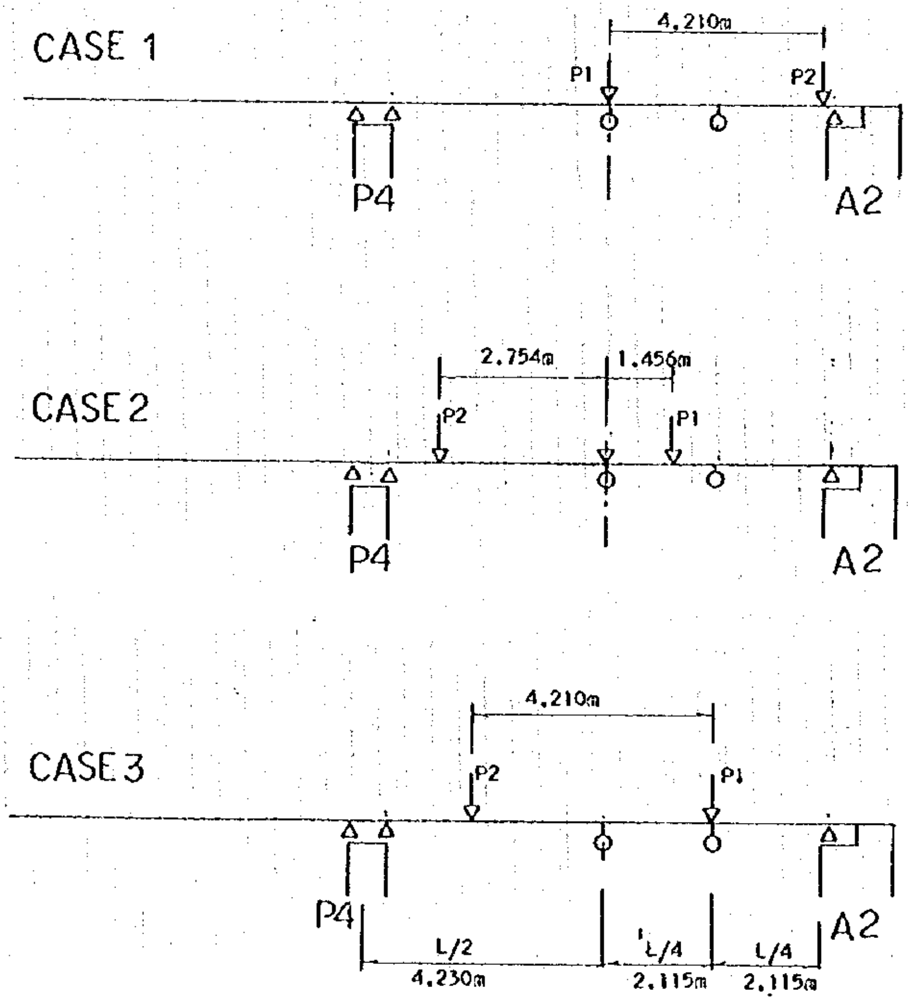


Fig. 5 Loading Stage   Bridge SER 59

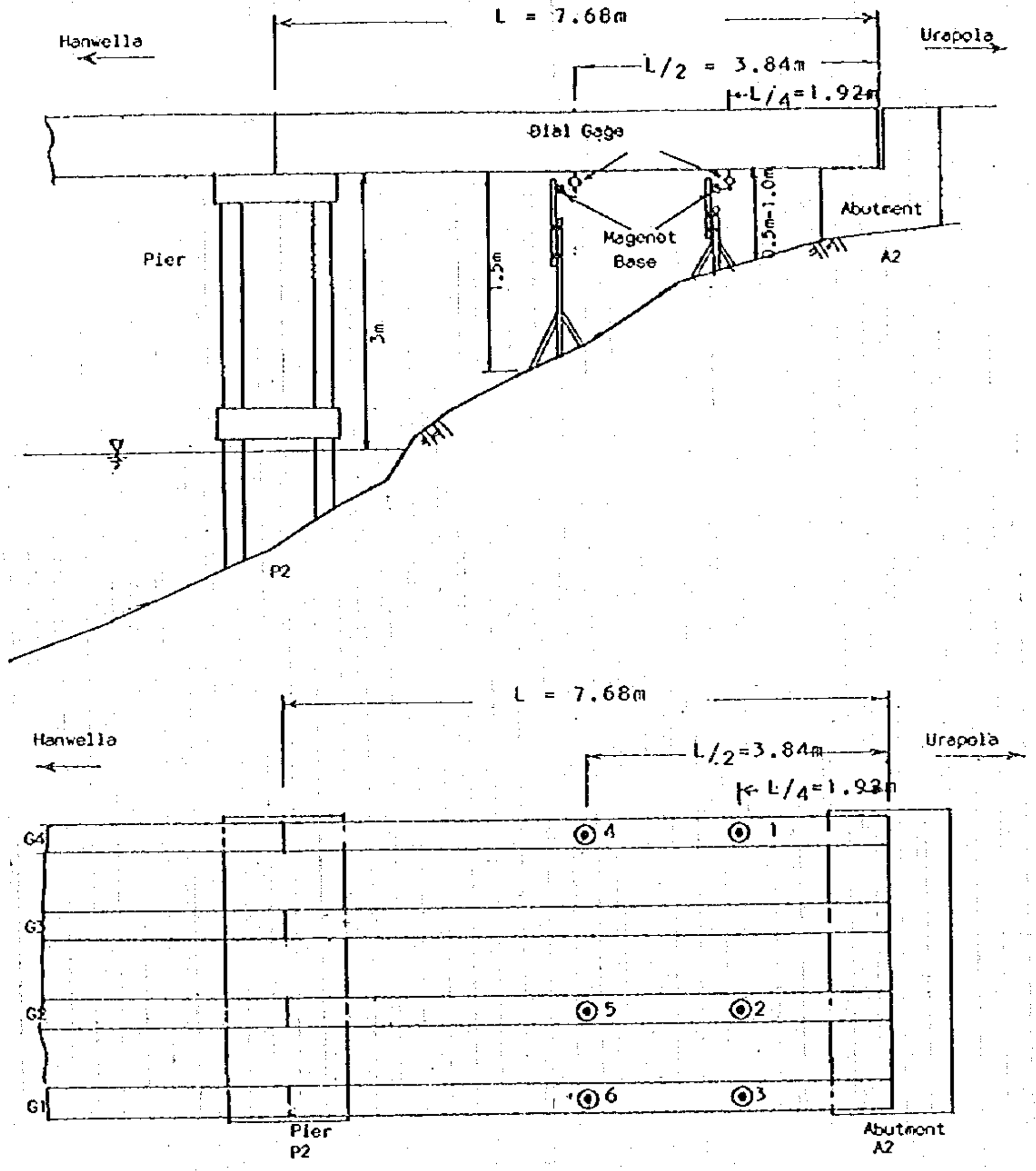
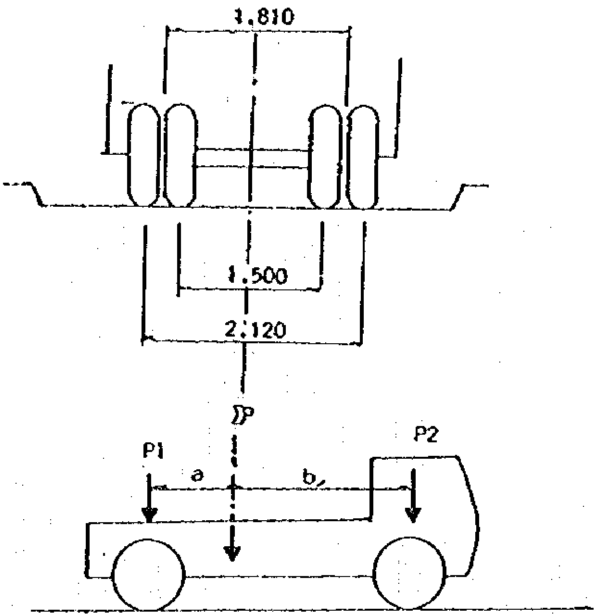


Fig.6 Measuring Points Bridge SER 211



For Vehicle No: 43/8249

P1 + P2 = 10.93ton

P1 > P2

$$a = \frac{P2}{P1 + P2} \times 4.210 = 1.456^{m}$$

$$b = 4.21 - a = 2.754^{m}$$

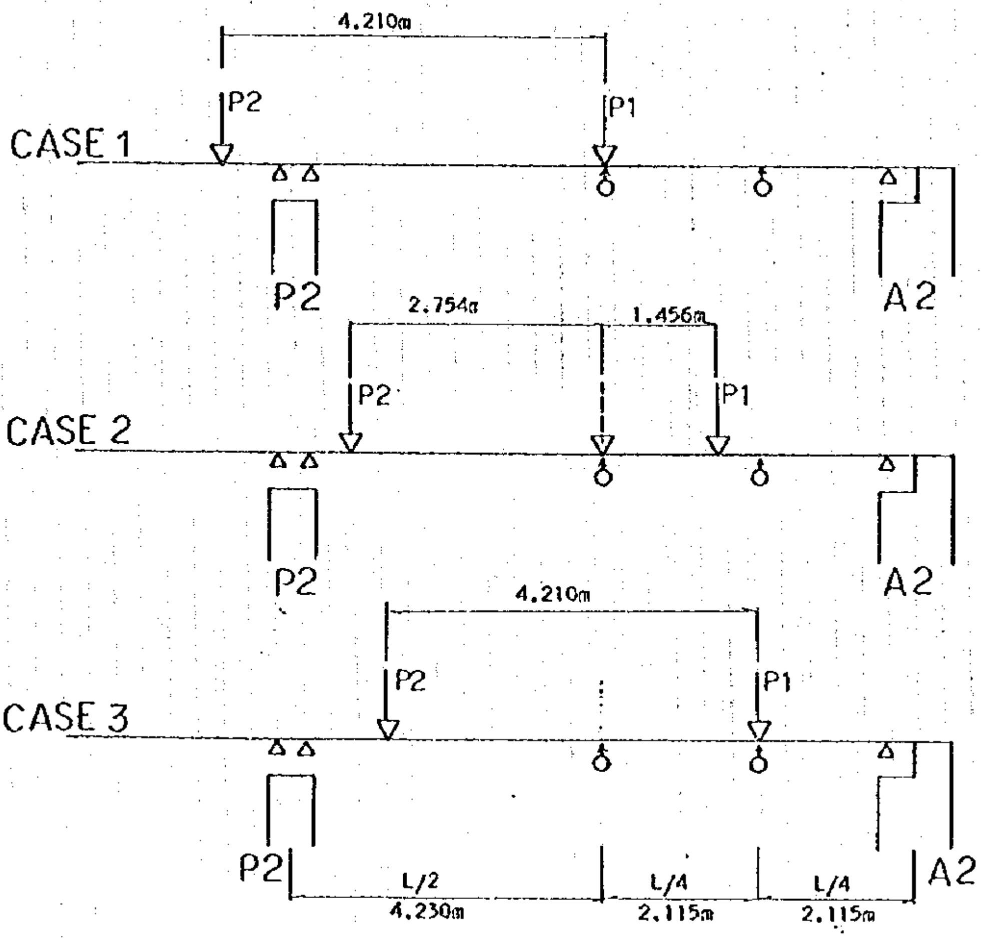


Fig.7 Loading Stage Bridge SER 211

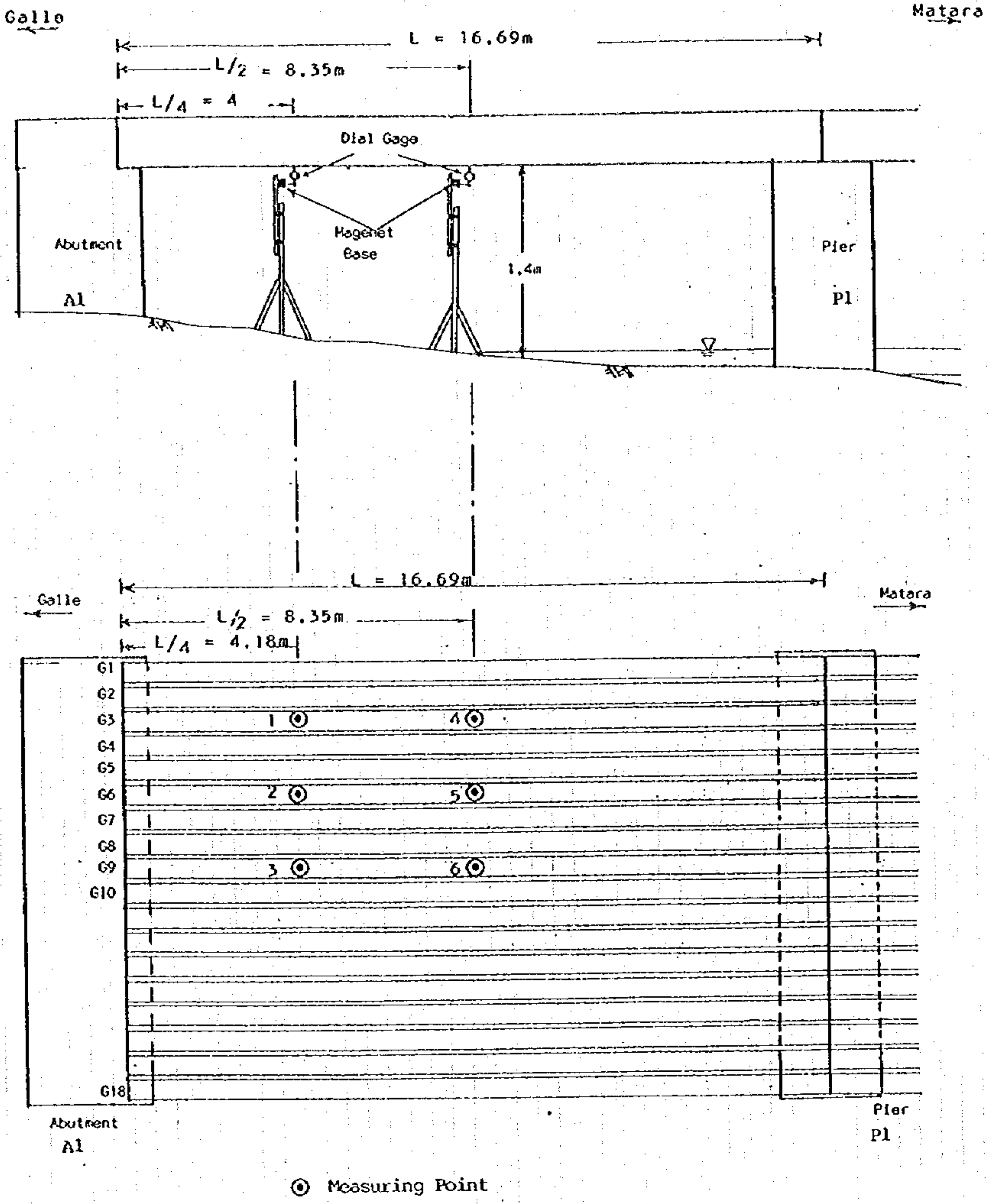
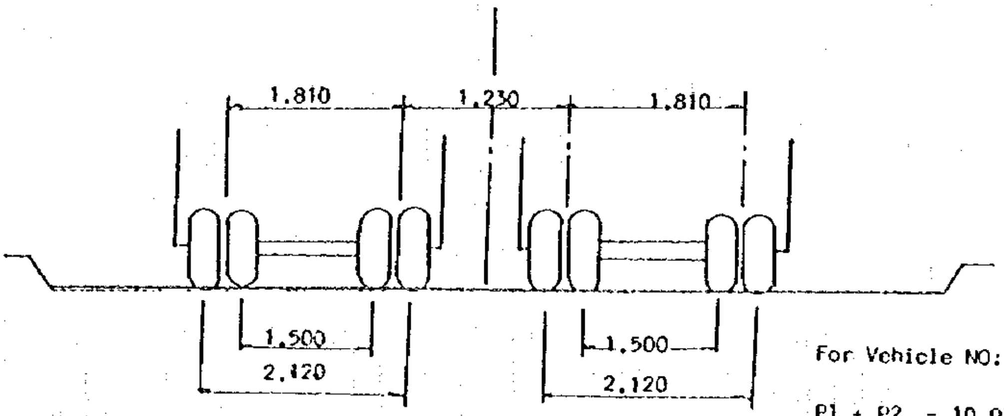
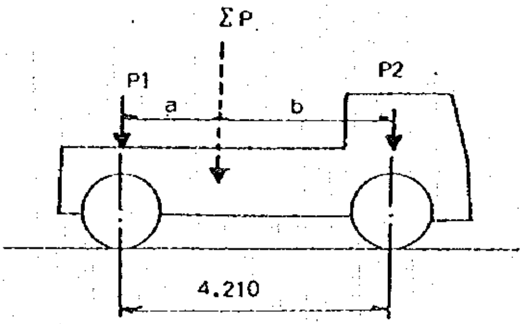


Fig. 8 Measuring Points Bridge SER 212





For Vehicle NO: 43/824

P1 + P2 = 10.93 ton

P1 > P2

$a = \frac{P2}{P1 + P2} \times 4.210 = 1.456$ m

$b = 4.21 - a \quad = 2.754$ m

For Vehicle NO: 43/8647

P1 + P2 = 10.96 ton

a = 1.44m

b = 2.77m

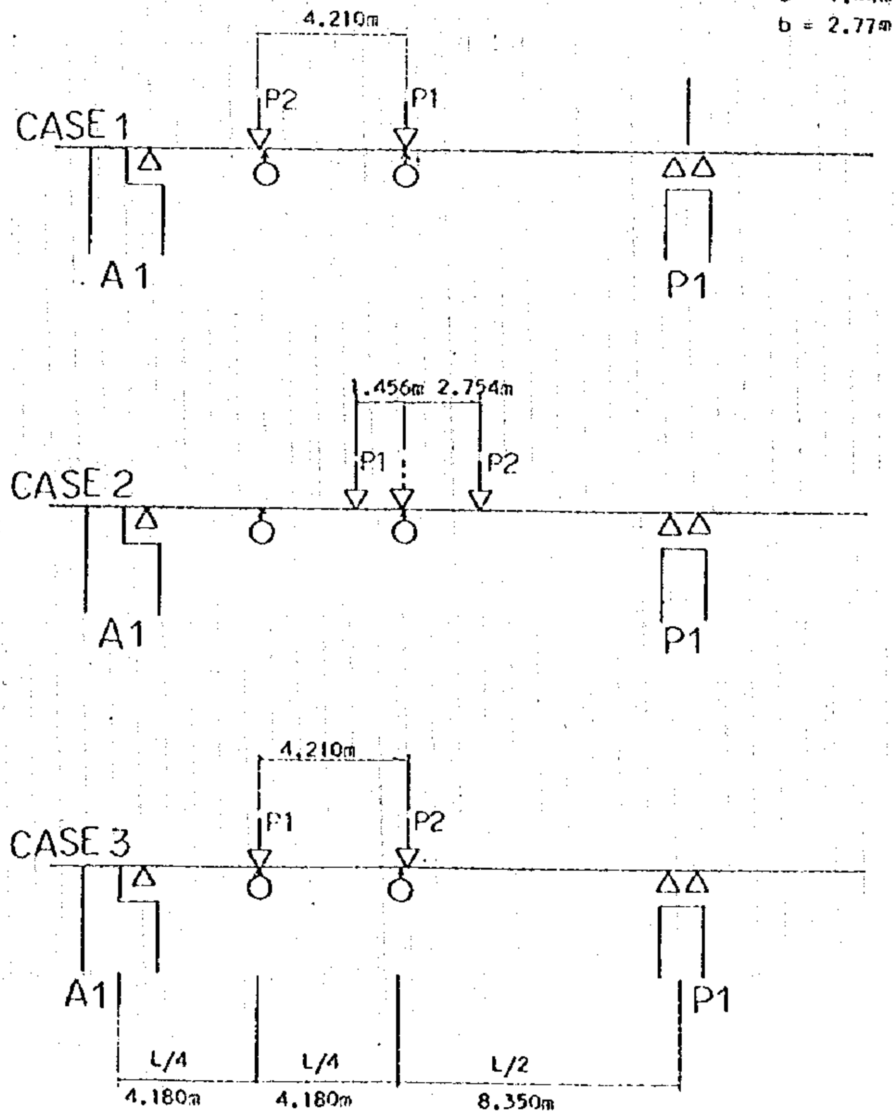


Fig. Loading Stage Bridge SER 212

2.2 FEM Analysis

The finite-element method (FEM) was used for evaluate the sag of slab and girders of three bridges. This calculation was done with computer.
The basic element of FEM is given in Figs. 10, 11 and 12.

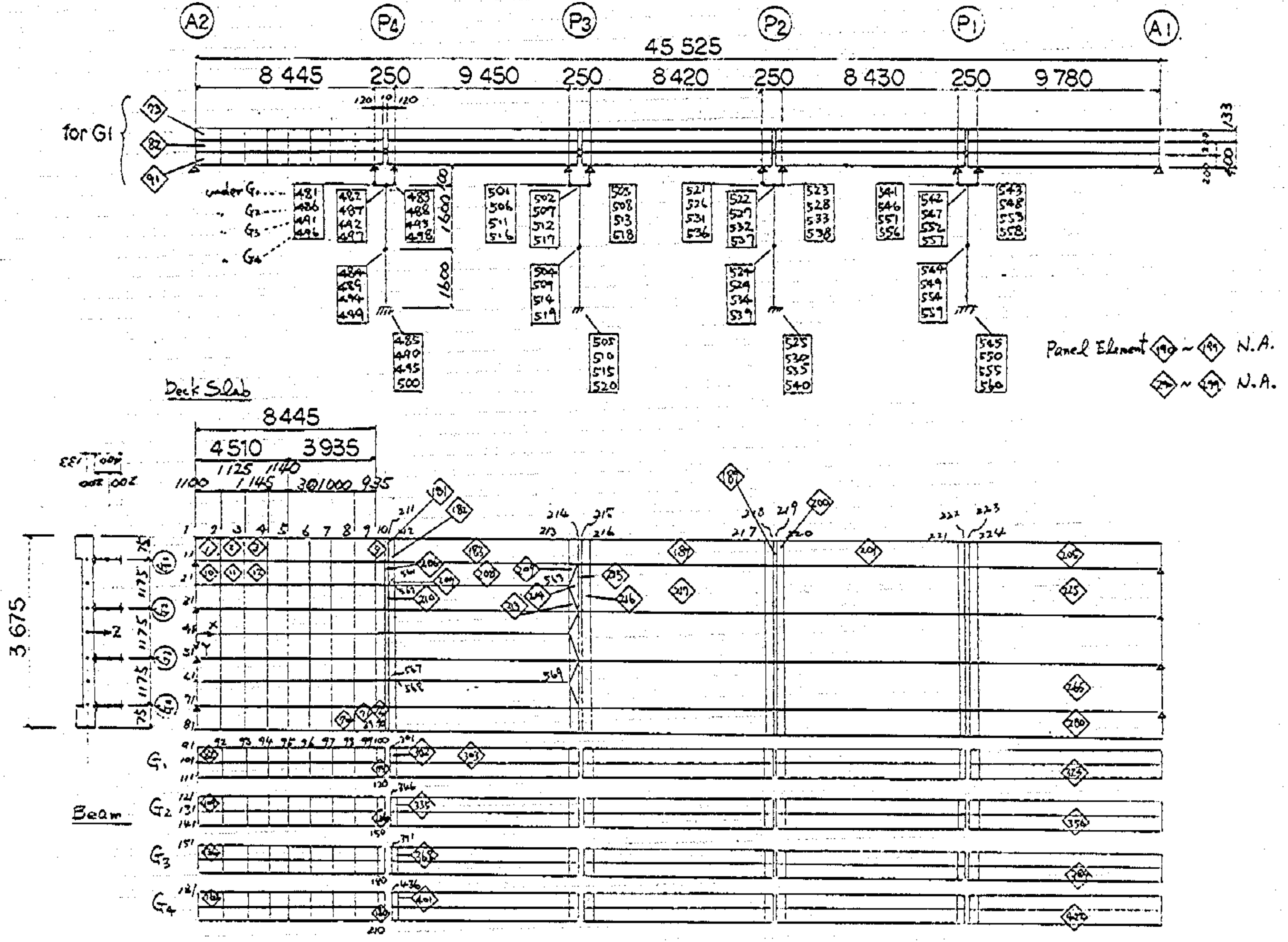


Fig. 10 Elements for FEM Bridge SER No.59

RC slab σ ck=330 kgf/cm²
σ s=400,388=394 kgf/cm²
σ ck=0.85 σ s=0.85X394
E=2890000 t/m² V=0.167(1/6)
Steel
E=21000000 t/m², V=0.30
Year of const 1942

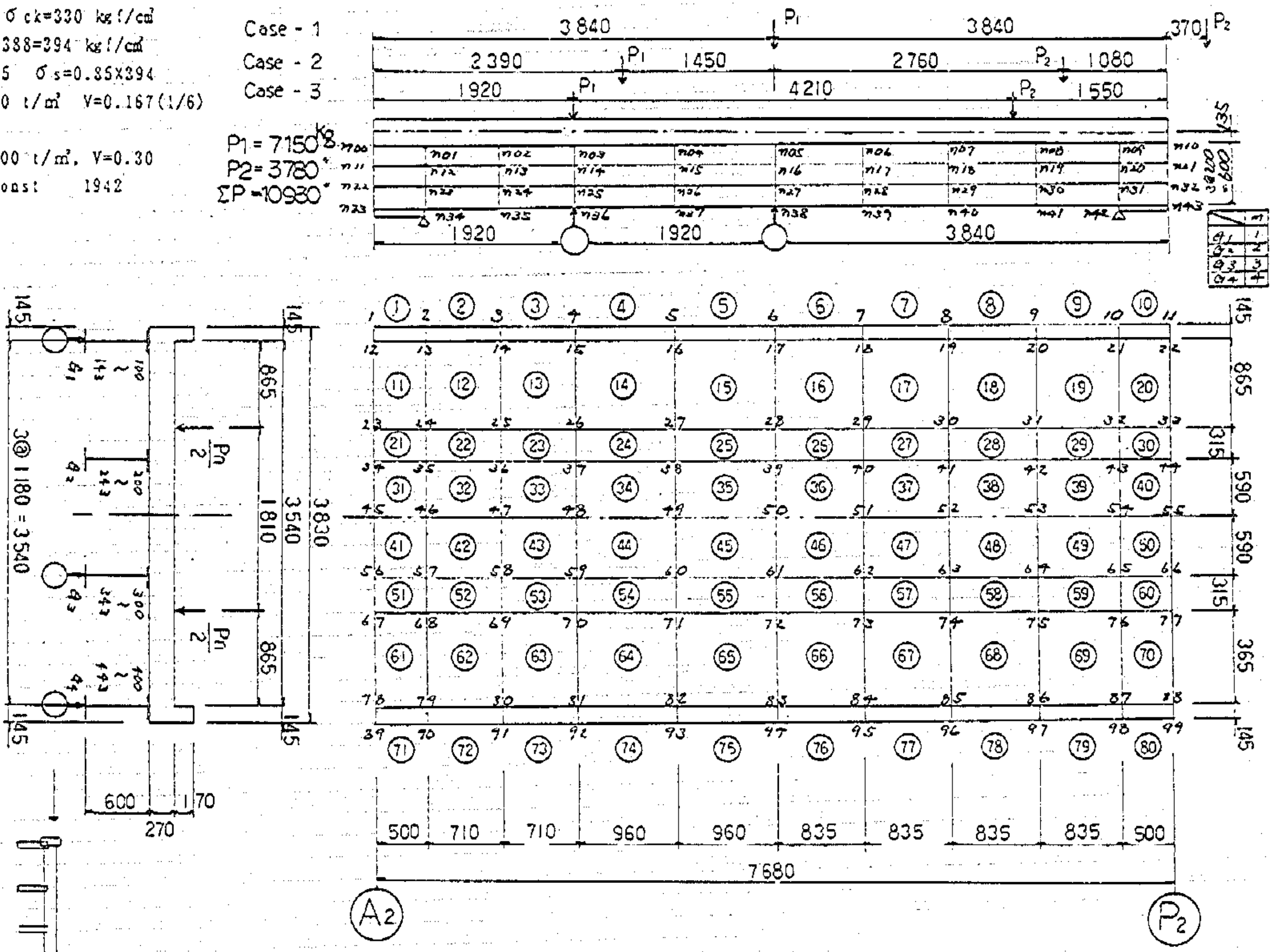


Fig. 11 Elements for FEM   Bridge SER No.211

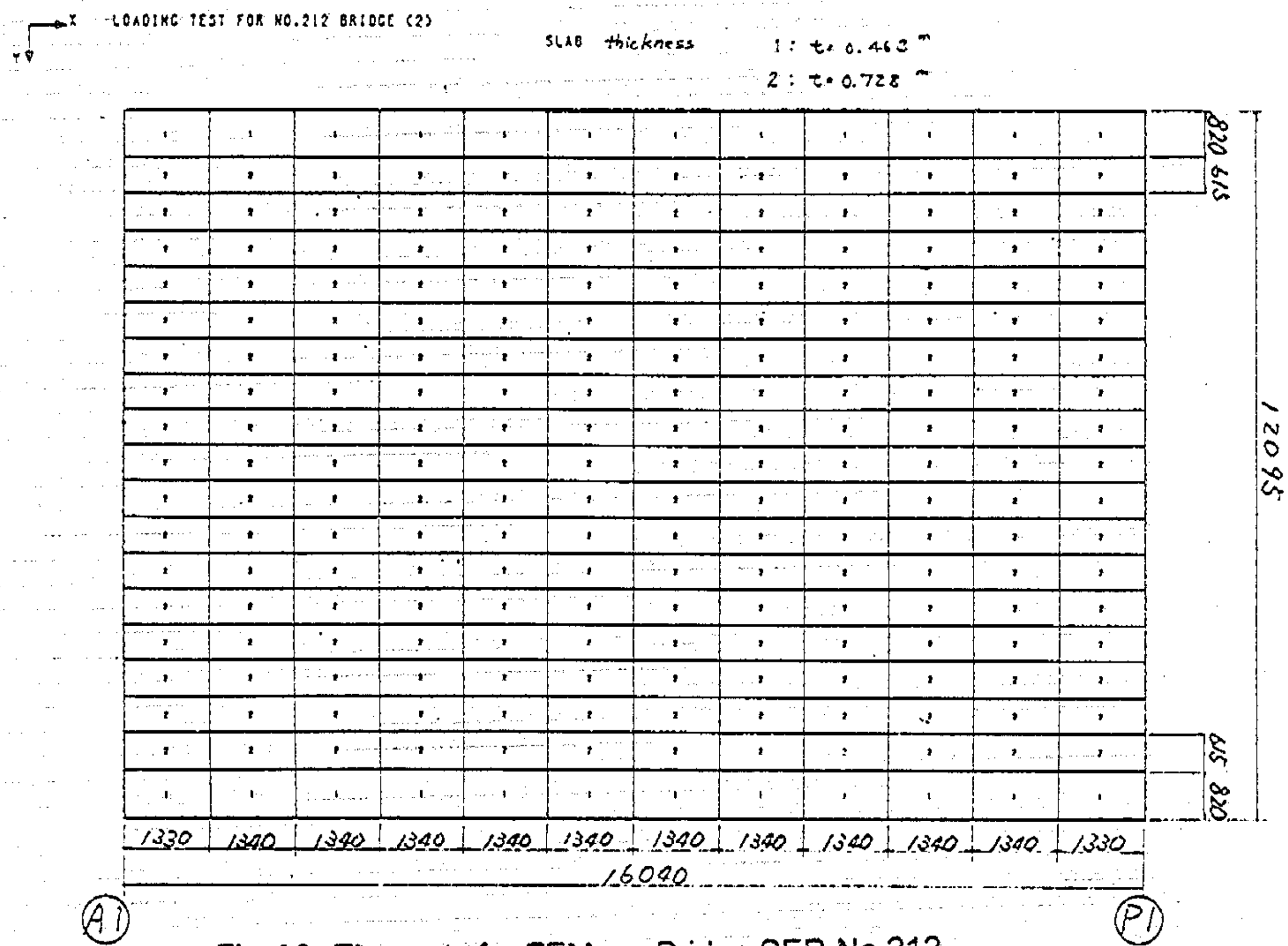


Fig. 12 Elements for FEM Bridge SER No.212

3. RESULT OF INVESTIGATION

3.1 Load Test

In this test, precise measurements were obtained without particular problem.
At bridge SER 59 and SER 211, the tests have done 2 times.
At bridge SER 212, the test has done only 1 time because the test affected the traffic flow.
Measured weight of lorries is as follows:

Lorry No.	43/8249	43/8247
Total Weight at Front Tires	3.78 ton	3.75 ton
Total Weight at Rear Tires	7.15 ton	7.21 ton
Total Weight of Lorry	10.93 ton	10.96 ton

The results of the loading tests are summarized in Tables 2, 3 and 4.
On the test at bridge SER 212, the gage No.6 did not work smoothly at the case 3.

Table 2 Summary of Load Test

Bridge SER N0.59

Case	Location	Girder No.	Gage No.	Deformation (mm) First	Deformation (mm) Second	Deformation (mm) Average
Case 1	L/4	G4	1	0.50	0.50	0.50
		G2	2	0.50	0.51	0.51
		G1	3	0.48	0.50	0.49
	L/2	G4	4	0.66	0.65	0.66
		G2	5	0.67	0.70	0.69
		G1	6	0.67	0.51	0.59
	L - Vertical	G2	7	0.11	0.12	0.12
		G1	8	0.06	0.08	0.07
	L - Horizontal	G2	9	-0.14	-0.15	-0.15
		G1	10	-0.14	-0.14	-0.14
Case 2	L/4	G4	1	0.61	0.59	0.60
		G2	2	0.59	0.60	0.60
		G1	3	0.55	0.55	0.55
	L/2	G4	4	0.77	0.75	0.76
		G2	5	0.73	0.75	0.74
		G1	6	0.65	0.68	0.67
	L - Vertical	G2	7	0.19	0.17	0.18
		G1	8	0.07	0.10	0.09
	L - Horizontal	G2	9	-0.15	-0.16	-0.16
		G1	10	-0.15	-0.15	-0.15
Case 3	L/4	G4	1	0.59	0.57	0.58
		G2	2	0.59	0.59	0.59
		G1	3	0.53	0.55	0.54
	L/2	G4	4	0.73	0.70	0.72
		G2	5	0.71	0.71	0.71
		G1	6	0.63	0.67	0.65
	L - Vertical	G2	7	0.16	0.14	0.15
		G1	8	0.10	0.07	0.09
	L - Horizontal	G2	9	-0.13	-0.17	-0.15
		G1	10	-0.14	-0.16	-0.15
Last	L/4	G4	1	0.01	0.01	0.01
		G2	2	0.00	-0.01	-0.01
		G1	3	-0.01	-0.02	-0.02
	L/2	G4	4	-0.01	0.00	-0.01
		G2	5	0.00	0.00	0.00
		G1	6	-0.01	0.02	0.01
	L - Vertical	G2	7	0.00	-0.03	-0.02
		G1	8	0.00	-0.03	-0.01
	L - Horizontal	G2	9	0.03	0.00	0.02
		G1	10	0.04	0.01	0.02

Table 3 Summary of Load Test

Bridge SER N0.211

Case	Location	Girder No.	Gage No.	Deformation (mm)		
				First	Second	Average
Case 1	L/4	G4	1	0.15	0.15	0.15
		G2	2	0.18	0.21	0.20
		G1	3	0.17	0.16	0.17
	L/2	G4	4	0.22	0.21	0.22
		G2	5	0.25	0.28	0.27
		G1	6	0.26	0.26	0.26
Case 2	L/4	G4	1	0.20	0.19	0.20
		G2	2	0.25	0.27	0.26
		G1	3	0.20	0.18	0.19
	L/2	G4	4	0.24	0.22	0.23
		G2	5	0.28	0.31	0.30
		G1	6	0.26	0.26	0.26
Case 3	L/4	G4	1	0.20	0.20	0.20
		G2	2	0.26	0.28	0.27
		G1	3	0.20	0.19	0.20
	L/2	G4	4	0.24	0.22	0.23
		G2	5	0.28	0.31	0.30
		G1	6	0.26	0.26	0.26
Last	L/4	G4	1	0.00	0.00	0.00
		G2	2	0.00	0.00	0.00
		G1	3	0.03	0.00	0.02
	L/2	G4	4	0.02	0.00	0.01
		G2	5	0.01	0.00	0.01
		G1	6	0.02	0.00	0.01

Table 4 Summary of Load Test

Bridge SER N0.212

Case	Location	Girder No.	Gage No.	Deformation (mm)		
				First	Second	Average
Case 1	L/4	G3	1	0.40	-	0.40
		G6	2	0.41	-	0.41
		G9	3	0.42	-	0.42
	L/2	G3	4	0.62	-	0.62
		G6	5	0.63	-	0.63
		G9	6	0.63	-	0.63
Case 2	L/4	G3	1	0.40	-	0.40
		G6	2	0.43	-	0.43
		G9	3	0.44	-	0.44
	L/2	G3	4	0.66	-	0.66
		G6	5	0.69	-	0.69
		G9	6	0.49	-	0.49
Case 3	L/4	G3	1	0.31	-	0.31
		G6	2	0.35	-	0.35
		G9	3	0.38	-	0.38
	L/2	G3	4	0.47	-	0.47
		G6	5	0.48	-	0.48
		G9	6	* 0.28	-	0.28
Last	L/4	G3	1	-0.10	-	-0.10
		G6	2	-0.12	-	-0.12
		G9	3	-0.10	-	-0.10
	L/2	G3	4	-0.12	-	-0.12
		G6	5	-0.15	-	-0.15
		G9	6	0.00	-	0.00

* Gage 6 did not work smoothly at Case 3.

4. STUDY

4.1 Deformation of Bridge SER 59

Vertical Deformation

Table 5 and Fig. 13 present the deformations given by FEM analysis and Load Test results. The computed deformations are 83 - 99% of the experimental deformation. The computed deformation gives fairly good agreement with the experimental results.

The experimental deformations at P-4 (Pier 4), however, are much larger than the computed deformation. The computed deformation is 34 -40 % of the experimental deformation. This may be because Pier 4 was sunk by load.

Compensated FEM analysis of which deformation at P-4 is adjusted to experimental deformation is shown in Table 7 and Fig.14. The compensated results are very good agree with experimental results.

Horizontal Deformation

As shown in Table 7 and Fig. 15, the computed horizontal deformations of P-4 give good agreement with the experimental deformation.

Table 5 FEM Calculation Results and Test Results Bridge SER No. 59

Distance from A2 (m)			0.000 0	1.110	2.225 L/4	3.370	4.340 L/2	4.510	5.510	6.510	7.510	8.445	8.565
Case1	G1	Deformation by FEM (mm)	0.000	0.222	0.427	0.565	0.583	0.586	0.497	0.342	0.170	0.028	0.008
		Deformation by Test (mm)			0.49			0.59				0.07	
		FEM / Test (%)			87			99				40	
	G2	Deformation by FEM (mm)	0.000	0.237	0.465	0.631	0.660	0.665	0.560	0.387	0.199	0.046	0.024
		Deformation by Test (mm)			0.51		0.69					0.12	
		FEM / Test (%)			91		96					38	
	G4	Deformation by FEM (mm)	0.000	0.222	0.427	0.565	0.583	0.586	0.497	0.342	0.170	0.028	0.008
		Deformation by Test (mm)			0.50			0.66					
		FEM / Test (%)			85			89					
Case2	G1	Deformation by FEM (mm)	0.000	0.272	0.507	0.635	0.630	0.629	0.532	0.376	0.193	0.033	0.011
		Deformation by Test (mm)			0.55			0.67				0.09	
		FEM / Test (%)			92			94				37	
	G2	Deformation by FEM (mm)	0.000	0.298	0.568	0.710	0.697	0.695	0.592	0.434	0.241	0.065	0.039
		Deformation by Test (mm)			0.60		0.74					0.18	
		FEM / Test (%)			95		94					36	
	G4	Deformation by FEM (mm)	0.000	0.272	0.507	0.635	0.630	0.629	0.532	0.376	0.193	0.033	0.011
		Deformation by Test (mm)			0.60			0.76					
		FEM / Test (%)			85			83					
Case3	G1	Deformation by FEM (mm)	0.000	0.270	0.490	0.601	0.595	0.594	0.506	0.358	0.181	0.031	0.009
		Deformation by Test (mm)			0.54			0.65				0.09	
		FEM / Test (%)			91			91				34	
	G2	Deformation by FEM (mm)	0.000	0.301	0.553	0.663	0.654	0.652	0.564	0.412	0.221	0.056	0.032
		Deformation by Test (mm)			0.59		0.71					0.15	
		FEM / Test (%)			94		92					37	
	G4	Deformation by FEM (mm)	0.000	0.270	0.490	0.601	0.595	0.594	0.506	0.358	0.181	0.031	0.009
		Deformation by Test (mm)			0.58			0.72					
		FEM / Test (%)			84			83					

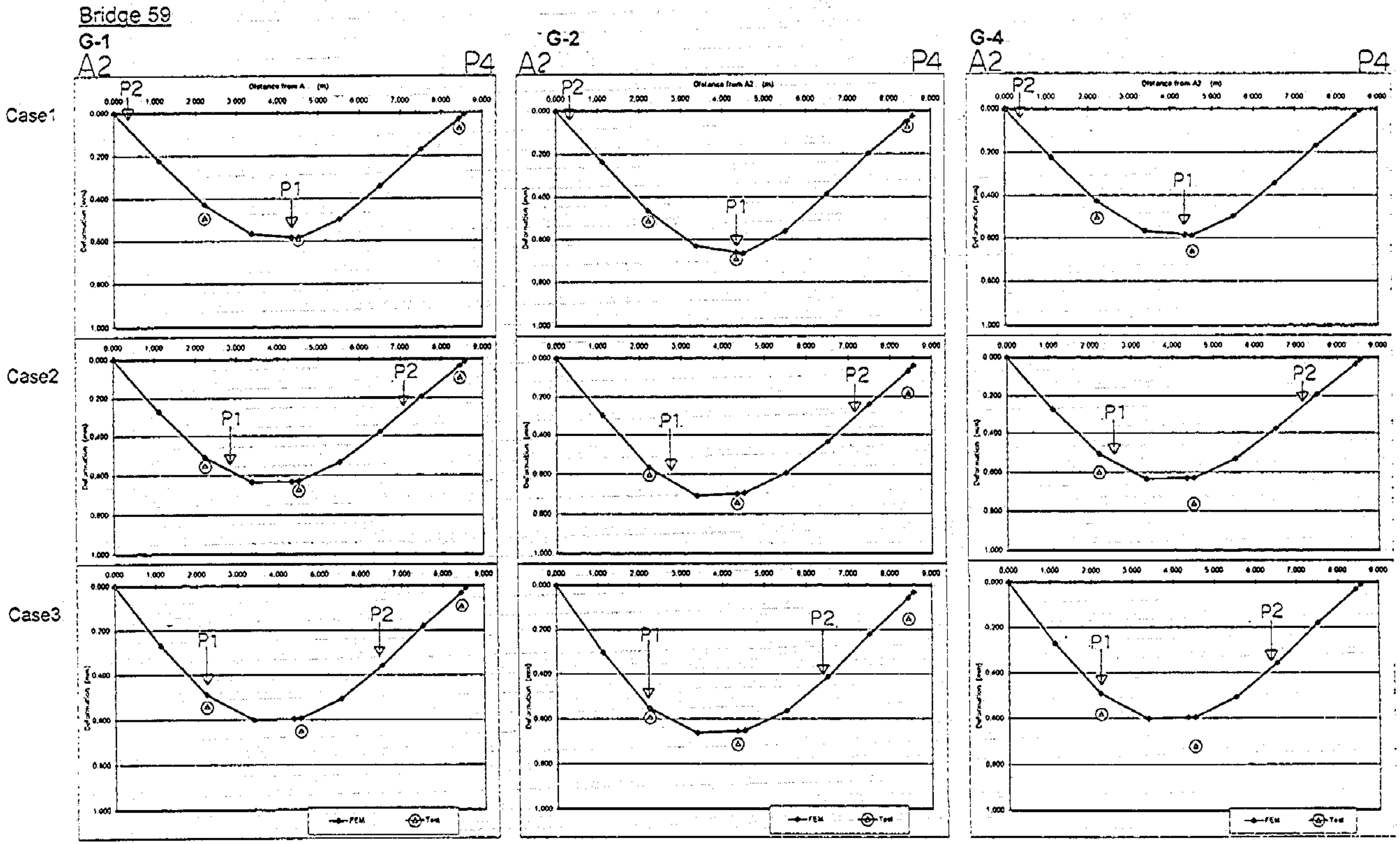


Fig.13 Deformation of Girders Bridge SER No.59

Table 6 Compensated FEM Calculation Results and Test Results Bridge SER No. 59

		Distance from A2 (m)	0.000	1.110	2.225	3.350	4.340	4.510	5.510	6.510	7.510	8.445	8.565
			0		L/4		L/2						
Case1	G1	Deformation by FEM (mm)	0.000	0.222	0.427	0.565	0.583	0.586	0.497	0.342	0.170	0.028	0.008
		Compensated Deformationby FEM (mm)	0.000	0.228	0.438	0.582	0.605	0.608	0.524	0.374	0.207	0.070	0.051
		Deformation by Test (mm)			0.49			0.59				0.07	
		FEM / Test (%)			87			99				40	
		Conpensated FEM / Test (%)			89			103				100	
	G2	Deformation by FEM (mm)	0.000	0.237	0.465	0.631	0.660	0.665	0.560	0.387	0.199	0.046	0.024
		Compensated Deformationby FEM (mm)	0.000	0.247	0.484	0.660	0.698	0.705	0.608	0.444	0.265	0.120	0.120
		Deformation by Test (mm)			0.51		0.69					0.12	
		FEM / Test (%)			91		96					38	
		Conponsated FEM / Test (%)			95		101					100	
Case2	G1	Deformation by FEM (mm)	0.000	0.272	0.507	0.635	0.630	0.629	0.532	0.376	0.193	0.033	0.011
		Compensated Deformationby FEM (mm)	0.000	0.279	0.522	0.658	0.659	0,659	0.569	0.420	0.244	0.090	0.069
		Deformation by Test (mm)			0.55			0.67				0.09	
		FEM / Test (%)			92			94				37	
		Conpensated FEM / Test (%)			95			98				100	
	G2	Deformation by FEM (mm)	0.000	0.298	0.568	0.710	0.697	0.695	0.592	0.434	0.241	0.065	0.039
		Compensated Deformationby FEM (mm)	0.000	0.313	0.598	0.756	0.756	0.756	0.667	0.523	0.343	0.180	0.156
		Deformation by Test (mm)			0.60		0.74					0.18	
		FEM / Test (%)			95		94					36	0.120
		Conpensated FEM / Test (%)			100		102					100	
Case3	G1	Deformation by FEM (mm)	0.000	0.270	0.490	0.601	0.595	0.594	0.506	0.358	0.181	0.031	0.009
		Compensated Deformationby FEM (mm)	0.000	0.278	0.506	0.624	0.625	0.626	0.544	0.403	0.233	0.090	0.069
		Deformation by Test (mm)			0.54			0.65				0.09	
		FEM / Test (%)			91			91				34	
		Conpensated FEM / Test (%)			94			96				100	
	G2	Deformation by FEM (mm)	0.000	0.301	0.553	0.663	0.654	0.652	0.564	0.412	0.221	0.056	0.032
		Compensated Deformationby FEM (mm)	0.000	0.313	0.578	0.700	0.702	0.702	0.625	0.484	0.305	0.150	0.127
		Deformation by Test (mm)			0.59		0.71					0.15	
		FEM / Test (%)			94		92					37	
		Conpensated FEM / Test (%)			98		99					100	

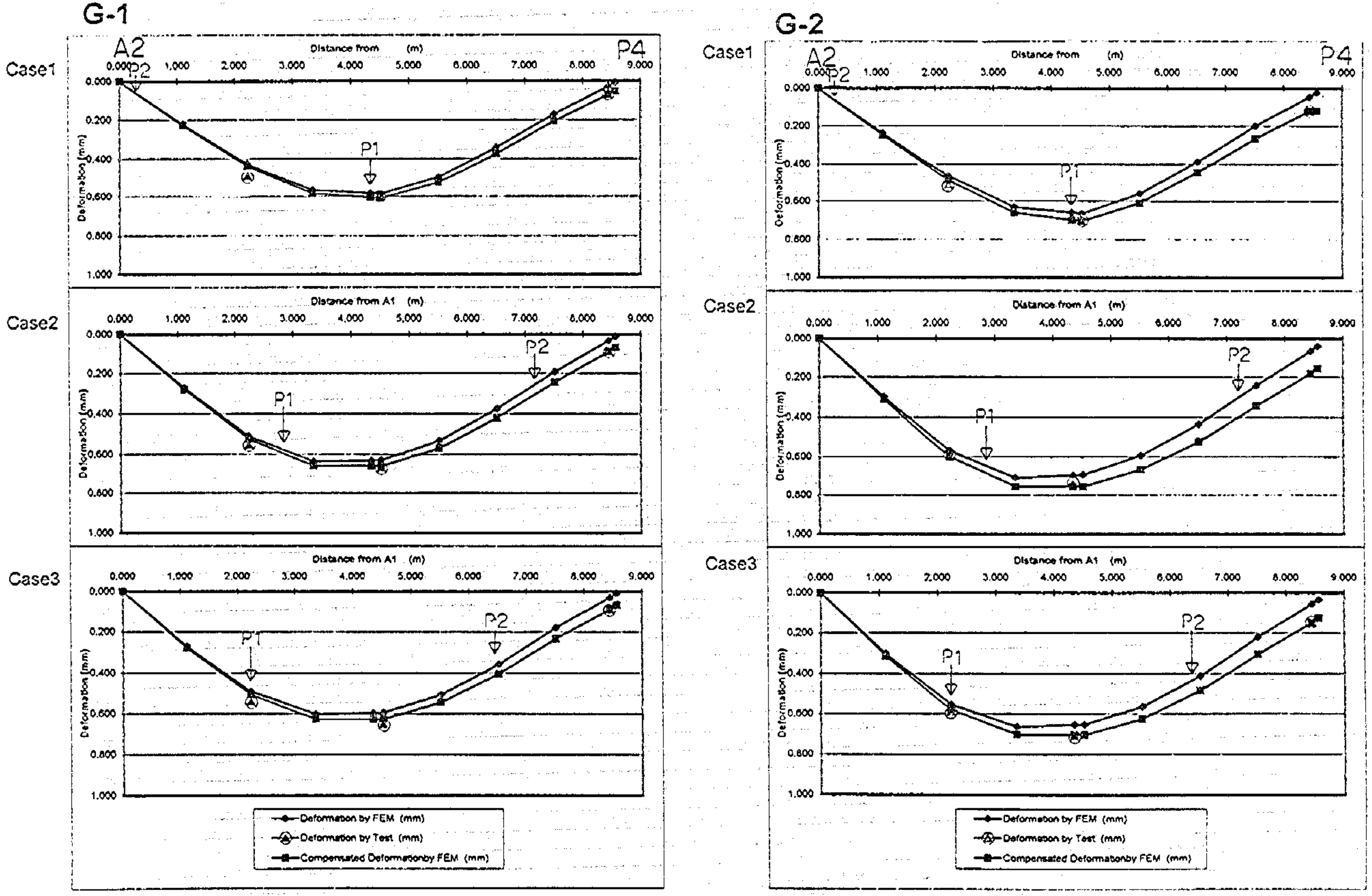


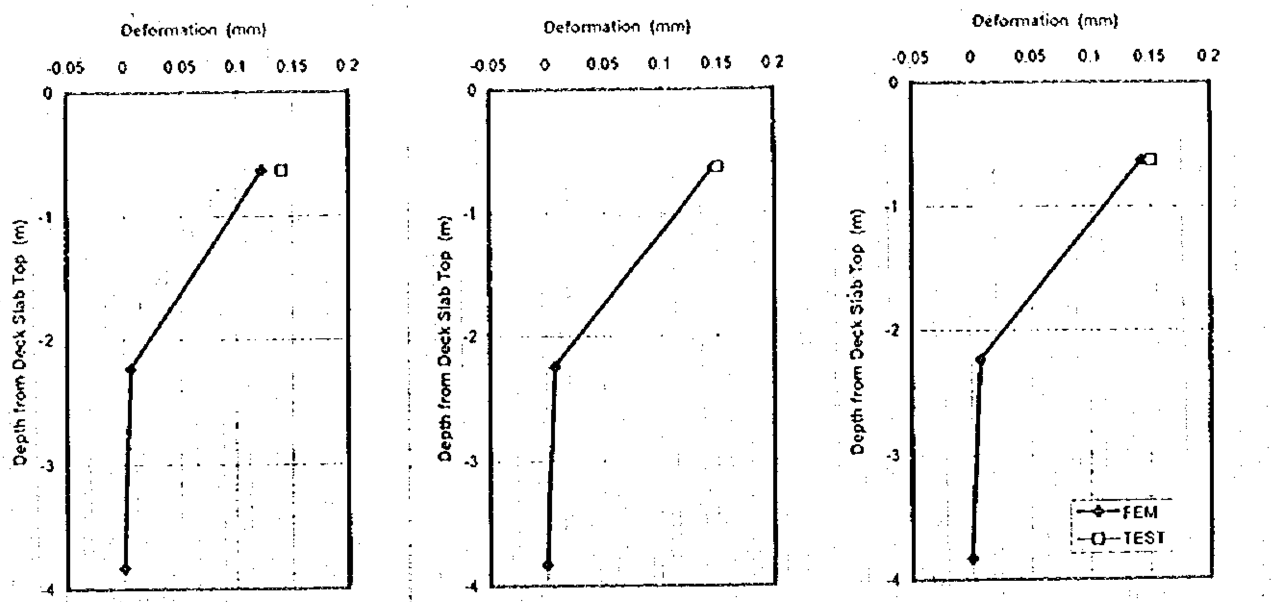
Fig. 14 Compensated Deformation of Girders Bridge SER No.59

Table 7 Horizontal Deformation of Column G1

Depth			-3.833	-2.233	-0.633
Case1	G1	FEM	0	0.006	0.122
		Test			0.14
	G2	FEM	0	0.008	0.128
		Test			0.15
Case2	G1	FEM	0	0.007	0.146
		Test			0.15
	G2	FEM	0	0.010	0.155
		Test			0.16
Case3	G1	FEM	0	0.007	0.142
		Test			0.15
	G2	FEM	0	0.009	0.151
		Test			0.15

G-1

CASE 1 CASE 2 CASE 3



G-2

CASE 1 CASE 2 CASE 3

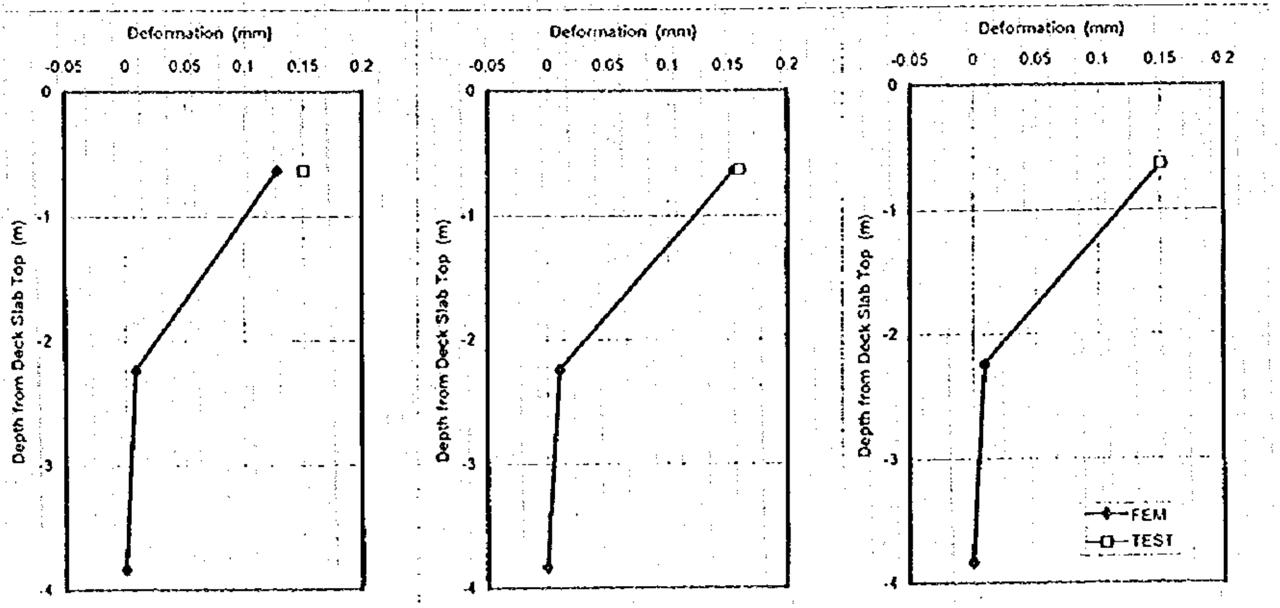


Fig. 15 Horizontal Deformation of Column G2 Bridge SER No.59

4.2 Deformation of Bridge SER 211

FEM analysis is computed under the condition of composite girder and noncomposite girder. The computed result of both conditions and experimental results are shown in Table 8 and Fig. 16. The experimental deformations are more similar to the computed deformation of composite girder than noncomposite girder.

However the deformations of composite girder are still deferent from experimental deformation. Therefore floor slab and girders may not be composite perfectly.

Table 8 FEM Calculation Results and Test Results

Bridge SER No.211

Distance from P1 (m)			0	0.5	1.21	1.92	2.88	3.84	4.675	5.51	6.345	7.18	7.68
Case1	G1	FEM composite	-0.037	0.000	0.055	0.103	0.151	0.169	0.154	0.116	0.063	0.000	-0.038
		FEM non Composite	-0.129	0.000	0.185	0.350	0.513	0.575	0.528	0.399	0.215	0.000	-0.129
		Test				0.17		0.26					
	G2	FEM composite	-0.037	0.000	0.061	0.117	0.176	0.201	0.182	0.135	0.073	0.000	-0.039
		FEM non Composite	-0.132	0.000	0.191	0.364	0.541	0.611	0.557	0.417	0.223	0.000	-0.133
		Test				0.20		0.27					
	G4	FEM composite	-0.037	0.000	0.055	0.103	0.151	0.169	0.154	0.116	0.063	0.000	-0.038
		FEM non Composite	-0.129	0.000	0.185	0.350	0.513	0.575	0.528	0.399	0.215	0.000	-0.129
		Test				0.15		0.22					
Case2	G1	FEM composite	-0.039	0.000	0.058	0.106	0.145	0.153	0.137	0.103	0.057	0.000	-0.032
		FEM non Composite	-0.133	0.000	0.189	0.350	0.489	0.520	0.467	0.353	0.192	0.000	-0.116
		Test				0.190		0.260					
	G2	FEM composite	-0.038	0.000	0.068	0.126	0.171	0.176	0.156	0.119	0.068	0.000	-0.034
		FEM non Composite	-0.140	0.000	0.205	0.379	0.522	0.544	0.484	0.366	0.202	0.000	-0.119
		Test				0.260		0.300					
	G4	FEM composite	-0.039	0.000	0.058	0.106	0.145	0.153	0.137	0.103	0.057	0.000	-0.032
		FEM non Composite	-0.133	0.000	0.189	0.350	0.489	0.520	0.467	0.353	0.192	0.000	-0.116
		Test				0.200		0.230					
Case3	G1	FEM composite	-0.037	0.000	0.056	0.100	0.136	0.144	0.130	0.100	0.056	0.000	-0.031
		FEM non Composite	-0.127	0.000	0.181	0.331	0.458	0.490	0.445	0.340	0.187	0.000	-0.113
		Test				0.200		0.260					
	G2	FEM composite	-0.036	0.000	0.067	0.120	0.158	0.164	0.149	0.116	0.067	0.000	-0.034
		FEM non Composite	-0.135	0.000	0.198	0.360	0.486	0.510	0.462	0.356	0.198	0.000	-0.118
		Test				0.270		0.300					
	G4	FEM composite	-0.037	0.000	0.056	0.100	0.136	0.144	0.130	0.100	0.056	0.000	-0.031
		FEM non Composite	-0.127	0.000	0.181	0.331	0.458	0.490	0.445	0.340	0.187	0.000	-0.113
		Test				0.200		0.230					

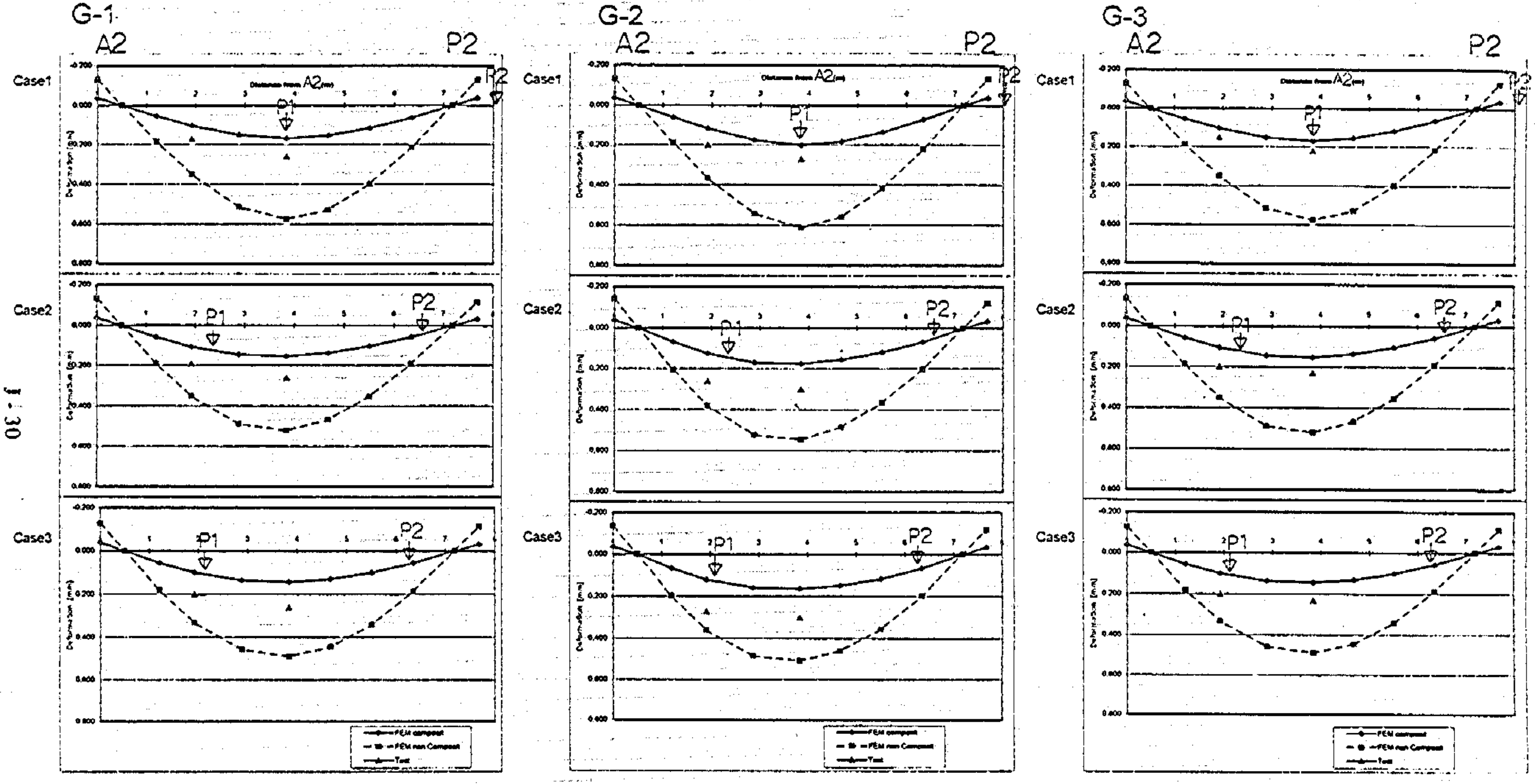


Fig. 16 Deformation of Girders Bridge SER No.211

4.3 Deformation of Bridge SER 212

Table 9 and Fig.17 present the deformations given by FEM analysis and test results. The computed deformations are almost double of the experimental deformation. In FEM analysis, the deformation was computed under the condition that thickness of slab is 728 mm. The computed results may have error by difference between the assumption and actual average thickness of slab. According to measurement at site, the actual thickness of slab may be more than 800 mm.

Table 9 FEM Calculation Results and Test Results

Bridge SER No.212

Distance from A1 (m)			0.00	1.33	2.67	4.01	5.35	6.69	8.02	9.35	10.69	12.03	13.37	14.70	16.04
Case1	G3	FEM	0.000	0.340	0.665	0.952	1.183	1.341	1.407	1.372	1.239	1.017	0.721	0.372	0.000
		Test				0.40			0.62						
	G6	FEM	0.000	0.340	0.664	0.952	1.186	1.347	1.417	1.380	1.246	1.024	0.725	0.374	0.000
		Test				0.41			0.63						
	G9	FEM	0.000	0.340	0.664	0.953	1.189	1.354	1.426	1.390	1.254	1.031	0.730	0.377	0.000
		Test				0.42			0.63						
Case2	G3	FEM	0.000	0.371	0.722	1.027	1.264	1.413	1.459	1.405	1.255	1.020	0.717	0.368	0.000
		Test				0.40			0.66						
	G6	FEM	0.000	0.371	0.723	1.031	1.273	1.425	1.469	1.413	1.262	1.023	0.718	0.369	0.000
		Test				0.43			0.69						
	G9	FEM	0.000	0.372	0.725	1.036	0.280	1.434	1.478	1.420	1.268	1.028	0.720	0.370	0.000
		Test				0.44			0.49						
Case3	G3	FEM	0.000	0.356	0.685	0.957	1.149	1.254	1.269	1.199	1.052	0.844	0.588	0.301	0.000
		Test				0.31			0.47						
	G6	FEM	0.000	0.361	0.696	0.972	1.162	1.263	1.276	1.202	1.051	0.841	0.545	0.299	0.000
		Test				0.35			0.48						
	G9	FEM	0.000	0.365	0.704	0.982	1.171	1.271	1.282	1.206	1.053	0.841	0.585	0.298	0.000
		Test				0.38			0.28						

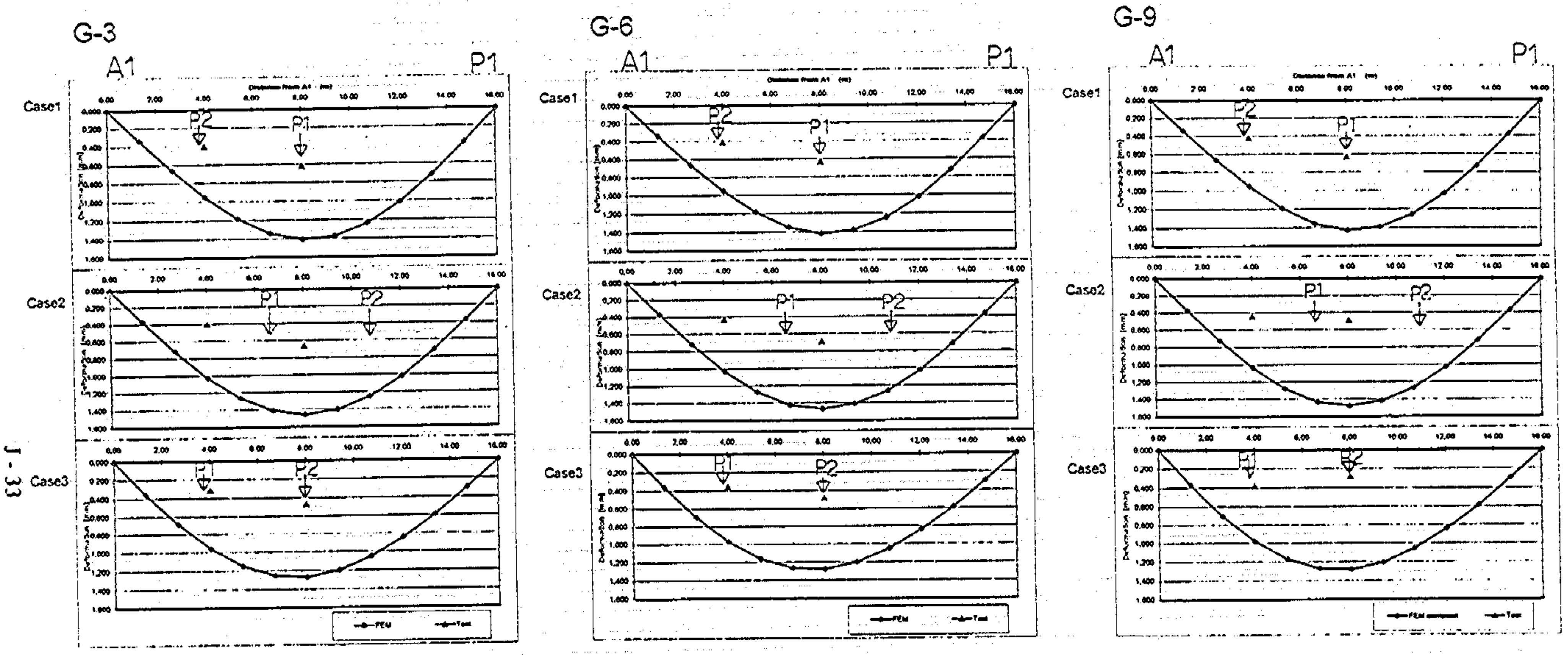
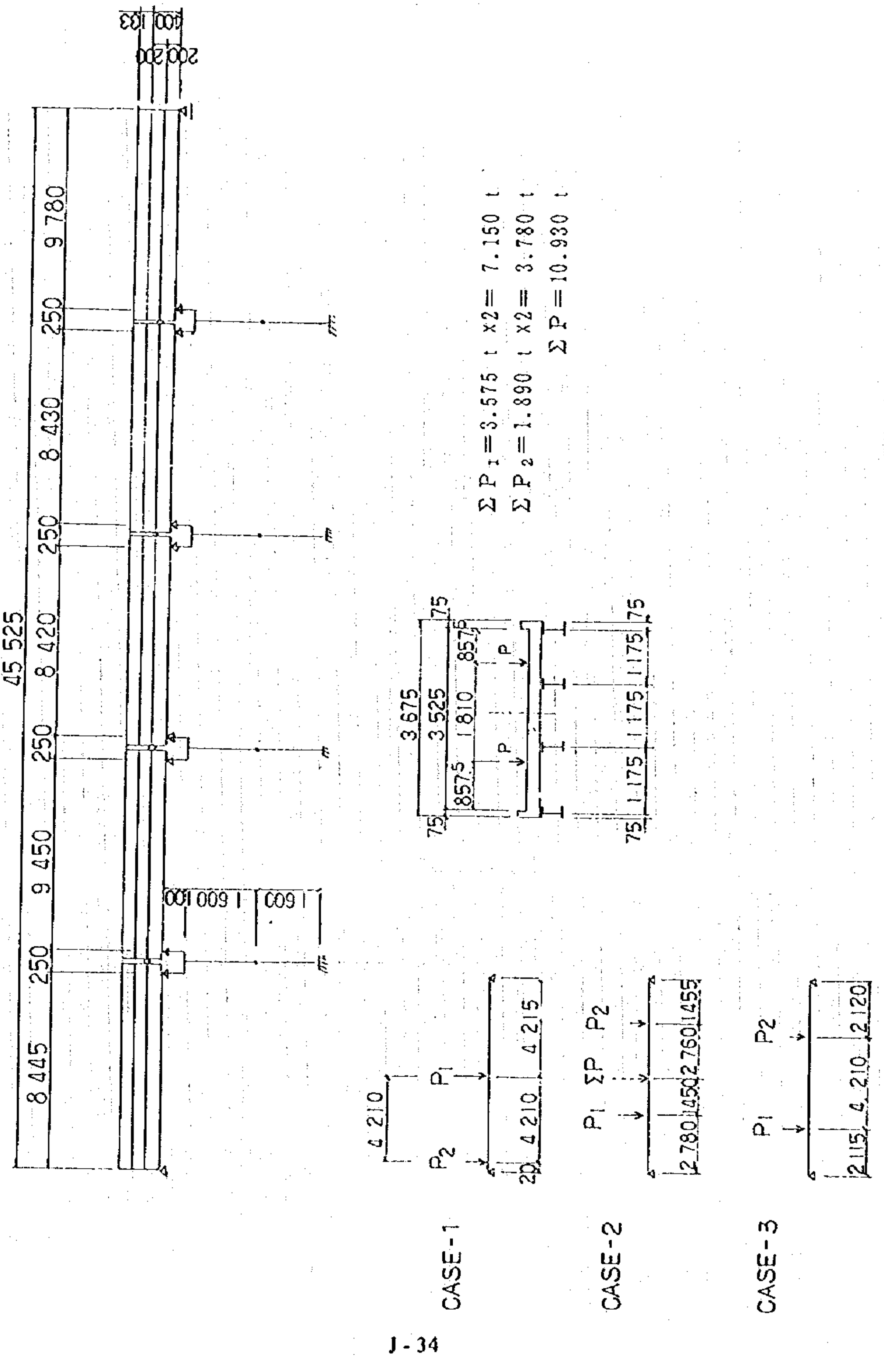


Fig. 17 Deformation of Girders Bridge SER No.212

5. Data of FEM Analysis

5.1 SER No. 59

Live Loading

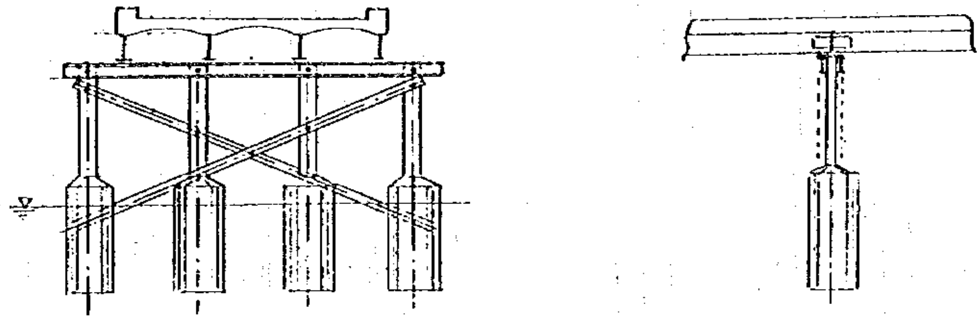


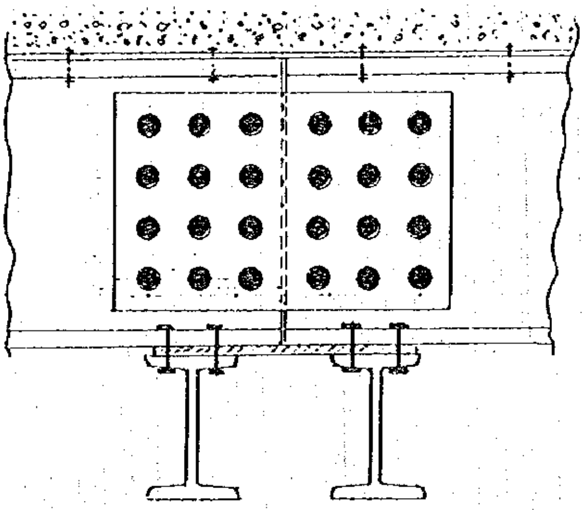
$\Sigma P_1 = 3.575$ t x2 = 7.150 t

$\Sigma P_2 = 1.890$ t x2 = 3.780 t

$\Sigma P = 10.930$ t

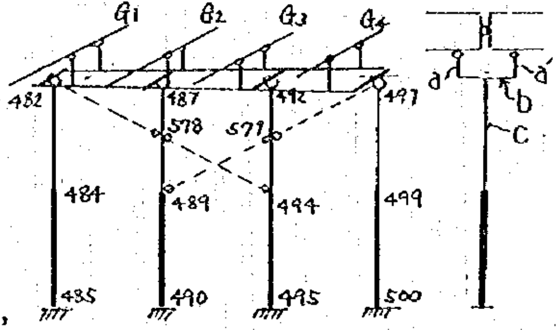
Pier





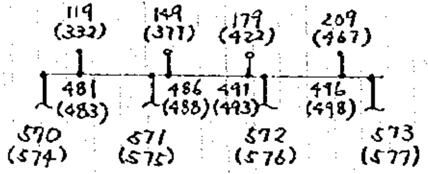
P 4 Elements for FEM

"c"

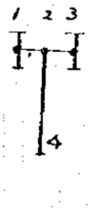


"a, a'"

A2 side (P3 side)

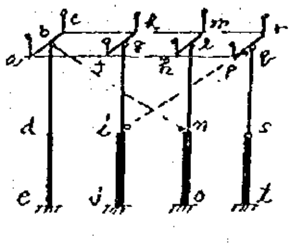


"b"



1	2	3	4
570	482	574	484
571	487	575	489
572	492	576	494
573	497	577	499

P3~1



P3 a~t 501~520
P2 〃 521~540
P1 〃 541~560

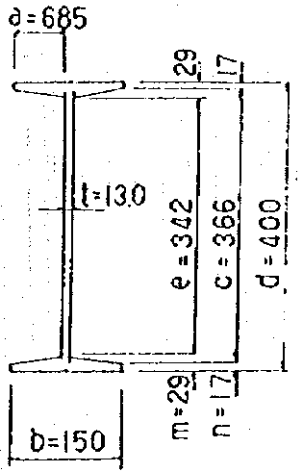
	P3	P2	P1
a	501	521	541
b	502	522	542
c	503	523	543
d	504	524	544
e	505	525	545
f	506	526	546
g	507	527	547
h	508	528	548
i	509	529	549
j	510	530	550
k	511	531	551
l	512	532	552
m	513	523	553
n	514	534	554
o	515	535	555
p	516	536	556
q	517	537	557
r	518	538	558
s	519	539	559
t	520	540	560

Assamptions

Curb ;	Continuous Construction joints are not necessary to be considered.
Deck slad ;	Continovous
BUC ;	Longitudinal direction to the bridge axis 4ft=1.22m Perpendicular direction to the bridge axis 3 3/4ft=1.14m There are joints at 1.22m intervals and connected with main beam by rivets of φ12 at 23cm intervals.
Main beam ;	There are joints on each pier. Webs are connected with splice plates, 2 Nos. height h=30cm, thickness t=10cm. Low flanges are connected with cross beams of piers by 8 Nos. of M24 Bolts, 10mm thickness steel plates inserting between them (single shear connection). The steel plates can be assumed as filler plates. Main beams can be considered as a type of connection beams.

S E R №59 Property of Members

Main beam



$A = dt + 2a(m+n)$

$= 40.0 \times 1.3 + 2 \times 6.85 \times (2.9+1.7)$

$= 115.02\ cm^2 \fallingdotseq 115\ cm^2 = 0.0115\ m^2$

$$I_x = \frac{bd^3 \frac{a}{4(m-n)}\ (c^4 - e^4)}{12}$$

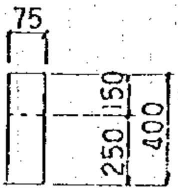
$$= \frac{15.0 \times 40.0^3 - \frac{6.85}{4 \times (2.9-1.7)} \times (36.6^4 - 34.2^4)}{12}$$

$= 29\ 295\ cm^4$

$W = 0.0115 \times 7.850 = 90.3\ kg/m$

$E = 2.1 \times 10^6\ kg/cm^2 = 21\ 000\ 000\ t/m^2$

Curb ($\sigma ck = 210\ kg/cm^2$)

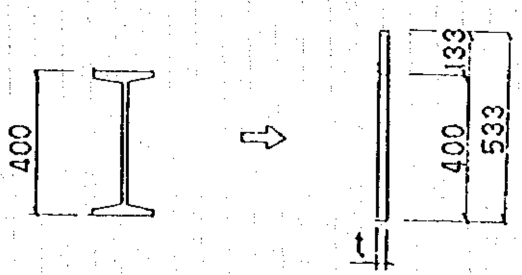


$A = 7.5 \times 40.0 = 300.0\ cm^2 = 0.0300\ m^2$

$$I = \frac{7.5 \times 40.0^3}{12} = 40.000\ cm^4$$

$E = 2.35 \times 10^5\ kg/cm^2 = 2\ 350\ 000 t/m^2$

For FEM Analysis, assuming Inertia of Main I Beam = const

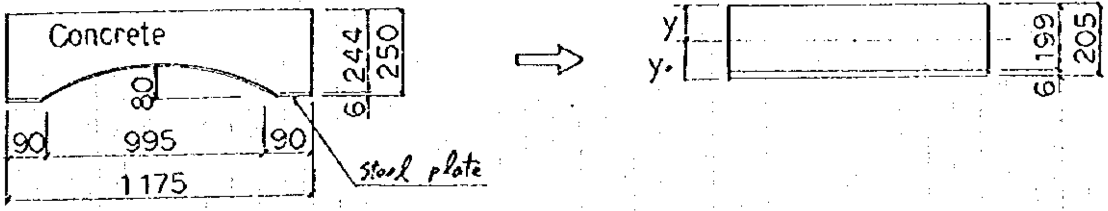


$$I = \frac{t \cdot h^3}{12} = 29\ 295\ cm^4$$

$t = 2.32\ cm$

S E R No.59

Deck Slab ($\sigma ck = 210$ kg/cm²)



$$h = \frac{A_1}{\ell} = \frac{\frac{2}{3} \times 99.5 \times 8.0}{117.5} = 4.5 \text{ cm}$$

poisson ratio n = 7

Property of Deck Slad (per metre)

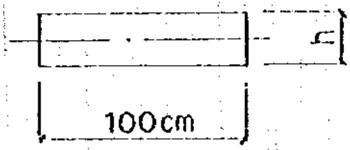
	A	Y	AY	AY²	Io
100×19.9	1990	10.55	20994.5	221492	65672
7×100× 0.6	420	0.3	126	38	13
	2410		21120.5	221530	65685

$$Y_o = \frac{AY}{A} = 8.764 \text{ cm} \fallingdotseq 8.8 \text{ cm}, \quad Y = 11.7 \text{ cm}$$

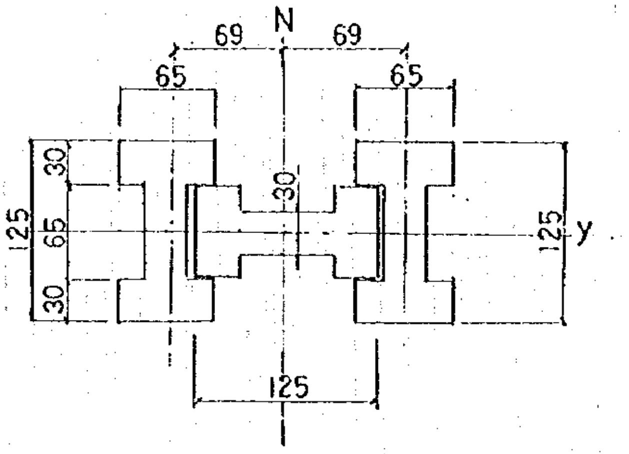
$$I = AY^2 - AY^2{}_o + I_o$$
$$= 221530 - 185094 + 65685$$
$$= 102121 \text{ cm}^4$$
$$E = 2.35 \times 10^5 \text{ kg/cm}^2$$

For FEM Analysis, assuming I = constant

$$I' = \frac{b \cdot h^3}{12} = 102\ 121 \text{ cm}^4$$
$$h = 23.1 \text{ cm}$$



Column-1



$$A = 3 \times \{6.5 \times 12.5 - 6.5 \times (6.5-3.0)\} = 3 \times 58.5 = 175.5\ \text{cm}^2 = 0.01755\ \text{m}^4$$

$$I_y = 2 \times \frac{6.5 \times 12.5^3 - 6.5^3 \times (6.5-3.0)}{12} + \frac{2 \times 3.0 \times 6.5^3 + 6.5 \times 3.0^3}{12}$$

$$= 2 \times 978 + 152 = 2\ 108\ \text{cm}^4 = 0.00002108\ \text{m}^4$$

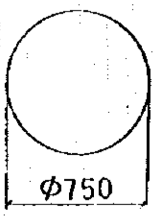
$$I_z = 978 + 2 \times 152 + 258.5 \times 6.9^2 = 6852\ \text{cm}^4 = 0.00006852\ \text{m}^4$$

$$J = 3 \times 6.5 \times 3.0^3 = 527\ \text{cm}^4 = 0.00000527\ \text{m}^4$$

$$E = 2.1 \times 10^6\ \text{kg/cm}^2$$

$$\omega = 0.01755 \times 7850 = 137.8\ \text{kg/m}$$

Coumn-2 $(\sigma_{ck} = 210\ \text{kg/cm}^2)$



$$A = 1/4 \times \pi \times 75.0^2$$

$$= 4418\ \text{cm}^2$$

$$I = \frac{\pi \times 75.0^4}{64} = 1\ 553\ 000\ \text{cm}^4 = 0.01553\ \text{m}^4$$

$$E = 2.35 \times 10^5\ \text{kg/cm}^2$$

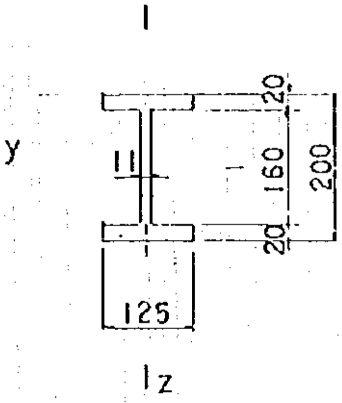
$$J = \frac{1}{2} \times \pi \times \left(\frac{75.0}{2}\right)^4 = 3\ 106\ 000\ \text{cm}^4$$

$$= 0.03106\ \text{m}^4$$

$$G_c = \frac{2.35 \times 10^5}{2.3} = 102\ 174\ \text{kg/cm}^2$$

$$= 1\ 020\ 000\ \text{t/m}^2$$

Cross Beam on the Column



$$A = 12.5 \times 20.0 - 16.0 \times (12.5 - 1.1)$$

$$= 67.6 \text{ cm}^2 = 0.00676 \text{ m}^2$$

$$Iy = 2 \times \frac{12.5 \times 20.0^3 - 16.0^3 \times (12.5 - 1.1)}{12}$$

$$= 4442 \text{ cm}^4 = 0.00004442 \text{ m}^4$$

$$W = 0.00676 \times 7850 = 53.1 \text{ kg/m}$$

$$E = 2.1 \times 10^6 \text{ kg/cm}^2$$

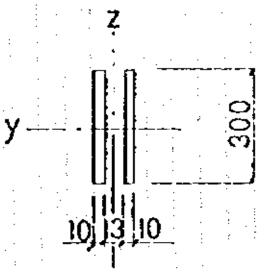
$$Iz = \frac{2 \times 2.0 \times 12.5^3 + 16.0 \times 1.1^3}{12}$$

$$= 653 \text{ cm}^4 = 0.000000653 \text{ m}^4$$

$$J = \frac{2 \times 12.5 \times 2.0^3 + 16.0 \times 1.1^3}{3}$$

$$= 73.8 \text{ cm}^4 = 0.000000738 \text{ m}^4$$

Splice of Main Beam



$$A = 2 \times 1.0 \times 30.0 = 60.0 \text{cm}^2 = 0.0060 \text{ m}^4$$

$$Iy = 2 \times \frac{1.0 \times 30.0^3}{12} = 4500 \text{ cm}^4 = 0.000045 \text{ m}^4$$

$$Iz = 2 \times \frac{30.0 \times 1.0^3}{12} + 2 \times 30.0 \times 1.15^2$$

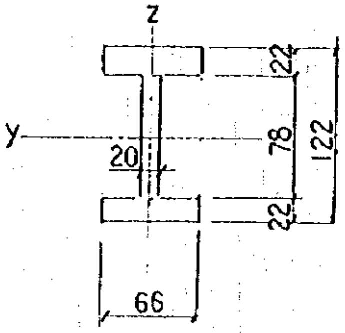
$$= 84.35 \text{ cm}^4 = 0.00000084 \text{ m}^4$$

$$J = 2 \times 15.0 \times 0.5^3 \times \left\{ \frac{16}{3} - 3.36 \times \frac{0.5}{15.0} \times \left(1 - \frac{0.5^4}{15.0^4}\right) \right\}$$

$$= 19.58 \text{ cm}^4 = 0.0000002 \text{ m}^4$$

$$E = 2.1 \times 10^6 \text{ kg/cm}^2$$

Bracing of Columns



$$A = 6.6 \times 12.2 - 7.8 \times (6.6 - 2.0)$$

$$= 44.64 \text{ cm}^2 = 0.0045 \text{ m}^2$$

$$Iy = \frac{6.6 \times 12.2^3 - 7.8^3 \times (6.6 - 2.0)}{12}$$

$$= 817 \text{ cm}^4 = 0.00000817 \text{ m}^4$$

$$Iz = \frac{2 \times 2.2 \times 6.6^3 + 7.8 \times 2.0^3}{12}$$

$$= 111 \text{ cm}^4 = 0.00000111 \text{ m}^4$$

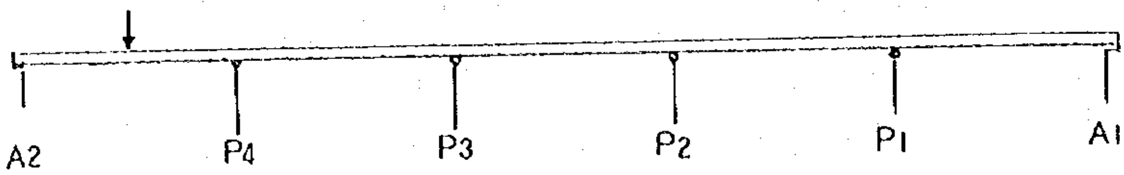
$$J = \frac{2 \times 6.6 \times 2.2^3 + 7.8 \times 2.0^3}{3}$$

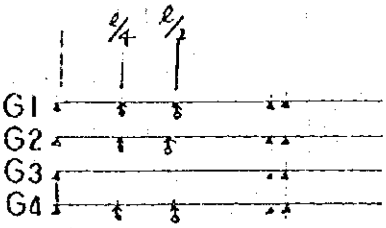
$$= 67.65 \text{cm}^4 = 0.00000068 \text{ m}^4$$

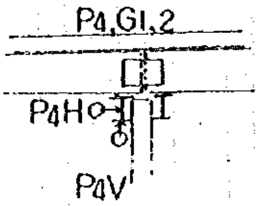
$$E = 2.1 \times 10^6 \text{ kg/cm}^2$$

$$W = 0.004464 \times 7850 = 35.0 \text{ kg/m}$$

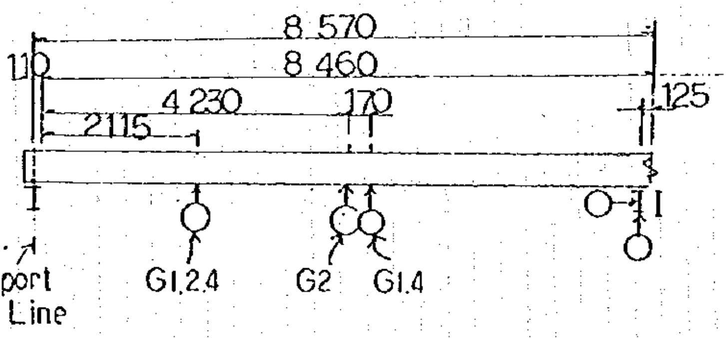
SER No.59 LODING TEST







G_1 ℓ/4 Point … 113 ℓ/2 Point … 115
G_2 ℓ/4 Point … 143 ℓ/2 Point … 145
G_4 ℓ/4 Point … 203 ℓ/2 Point … 205



P4 G_1 H Point 119
V Point 481
H Point 149
V Point 486

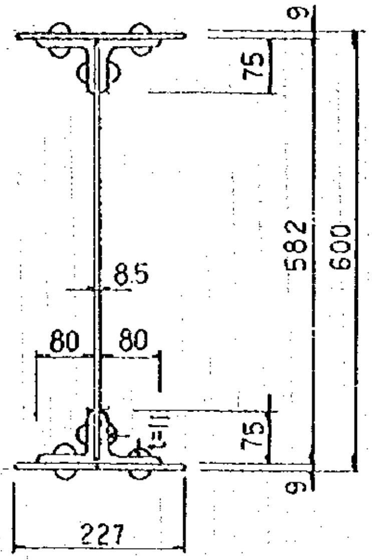
Deflection of SER No.59 Loading Test and FEM Analysis

Loading Test Cases		Outside Girder				Inside Girder			
		L/4		L/2		L/4		L/2	
	Def. (mm)	Def.	F/A	Def.	F/A	Def.	F/A	Def.	F/A
Case-1	Actual	0.49	-	0.59	-	0.51	-	0.69	-
	FEM Analysis 1	0.438	0.894	0.608	1.031	0.484	0.949	0.698	1.012
Case-2	Actual	0.55	-	0.67	-	0.60	-	0.74	-
	FEM Analysis 1	0.522	0.949	0.659	0.984	0.598	0.997	0.756	1.022
Case-3	Actual	0.54	-	0.65	-	0.59	-	0.71	-
	FEM Analysis 1	0.506	0.937	0.626	0.963	0.578	0.980	0.702	0.989
Ave-rage	Actual	0.527	-	0.637	-	0.567	-	0.713	-
	FEM Analysis 1	0.489	0.928	0.631	0.991	0.553	0.975	0.719	1.008

Note: The mount of deflection above derived from FEM analysis are compensated.
FEM Analysis 1 : Composite.

5.2 SER No.211

GIRDER point 12~22, 34~44, 56~66, 78~88



		A cm²	y cm	Ay^2+I cm⁴
U-Flg PL	2×227× 9	40.86	29.55	35679
L(Flg)	4× 80×11	35.20	28.55	28692
L(Web)	4× 64×11	28.16	24.80	17320
Web PL	1×580× 8.5	49.30	——	13820
		153.52cm²		95511cm⁴

$A = 153.52$ cm² $\fallingdotseq 154$ cm²

$I = 95511$ cm⁴ $\fallingdotseq 0.000955$ m⁴

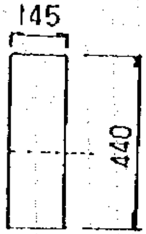
$W = 0.0154 \times 7{,}850 + 47.568/1000 \times 123 = 127$ kgf/m

Rivet Head

Rivet Head $= \phi 30 \sim \phi 32$ mm $\fallingdotseq \phi 30$ mm → $d = 19$ mm

Rivet pich 97.5 mm $(12 \times 1000/97.5 = 123.08)$

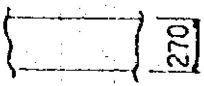
CURB, SLAB ①～⑩, ⑦～(80) $\sigma ck = 350$ kg/cm²



$A = 14.5 \times 44.0 = 638.0$ cm²

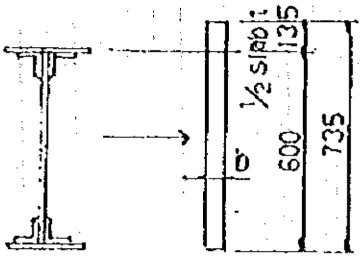
$$I = \frac{14.5 \times 44.0^3}{12} = 102\ 931 \text{ cm}^4 = 0.00103 \text{ m}^4$$

SLAB ⑪～(70)



$t = 270$ mm $\sigma ck = 350$ kg/cm²

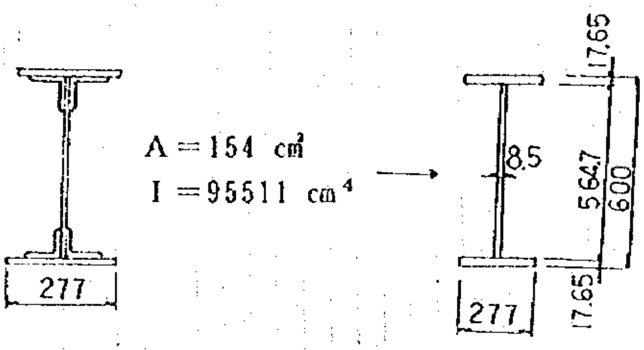
CHANGE TO GIRDER MODEL



$I_o = 95511$ cm⁴

$$I_1 = \frac{bh^3}{12} = \frac{b \times 73.5^3}{12} = 95511 \text{ cm}^4$$

$\therefore b = 2.8865$ cm



		A cm²	y cm	Ay²+I cm⁴
U-Flg PL	227X17.65	48.89	29.12	41457
Web PL	564.7X 8.5	48.00		12755
L-Flg PL	227X17.65	48.89	-29.12	41457
		145.78cm²		95669cm⁴

Deflection of SER No. 211 Loading Test and FEM Analysis

Loading Test Cases		Outside Girder				Inside Girder			
		L/4		L/2		L/4		L/2	
	Def. (mm)	Def.	F/A	Def.	F/A	Def.	F/A	Def.	F/A
Case-1	Actual	0.160	-	0.240	-	0.200	-	0.270	-
	FEM Analysis 1	0.103	0.644	0.169	0.704	0.117	0.585	0.201	0.744
	FEM Analysis 2	0.350	2.138	0.575	2.396	0.364	1.820	0.611	2.263
Case-2	Actual	0.195	-	0.245	-	0.260	-	0.300	-
	FEM Analysis 1	0.106	0.544	0.153	0.624	0.126	0.485	0.176	0.587
	FEM Analysis 2	0.350	1.795	0.520	2.122	0.379	1.458	0.544	1.813
Case-3	Actual	0.200	-	0.245	-	0.270	-	0.300	-
	FEM Analysis 1	0.100	0.500	0.144	0.588	0.120	0.444	0.164	0.547
	FEM Analysis 2	0.331	1.655	0.490	2.000	0.360	1.333	0.510	1.700
Averege	Actual	0.185	-	0.243	-	0.243	-	0.290	-
	FEM Analysis 1	0.103	0.557	0.154	0.634	0.121	0.498	0.180	0.621
	FEM Analysis 2	0.344	1.859	0.528	2.173	0.368	1.514	0.555	1.914

FEM Analysis 1 ; Composite. FEM Analysis 2 ; Non-composite.

Appendix - K

DATA OF STRENGTH TEST OF STEEL SAMPLES

LIST OF CONTENTS

1. Steel/Iron Piece Sampling

Sample No.	Name of Bridge	Route No. Bridge No.	Description
1	Giriulla Bridge	B322 38/3km	Negombo-Giriulla Rd. Year of Cons. : 1880 Type of Bridge : ST.TR/COR Sample taken : Plate from and post of main girder Size of Sample : b=400mm, h=620mm, t=10mm
2	Kuruwita Bridge	AA004 87/1km	Colombo-Ratnapura Rd. Year of Cons. : 1934 (Rehabilitated) Type of Bridge : ST.TR/COR Sample taken : Plate and angle from new structure rehabilitated in 1934. Size of Sample : b=600mm, h=225mm, t=6.5mm L-75x75x10, l=620mm
3	Matara Old Bridge	AA002 160/7km	Colombo-Galle-Hambantota Rd. Year of Cons. : 1860-70 (estimated) Type of Bridge : RSJ/COR + ST.TR/COR Sample taken : U-Flange and Web from main I-beam (I-450x155, tw=16) Size of Sample : h=410mm, l=510mm
4	Warakatota Bridge	AA004 102/3km	In Ratnapura town Year of Cons. : 1909 Type of Bridge : ARCH/S (Steel Arch Bridge) + ST.TR/COR Sample taken : End plate of End post from main girder of ST.TR Size of Sample : b=470mm, h=500mm

Survey of Construction Year for Sample No.3

Matara Old Bridge was constructed before the following 2 bridges.

1) B309 1/1km Akuressa-Kumburutitiya Rd., Nilwalaa Ganga
ST.TR/T - Lower Cord Pin-Type
- Records on the Newel Post -

OCT 28TH 1879
W.C. RANASINGHA
OVER P.

W. WRICHTSONCE
SUP ç OFFICER

2) AA17 Galle-Deniyaya-Madampe Rd., Polwatta Ganga
- Records on the Newel Post -

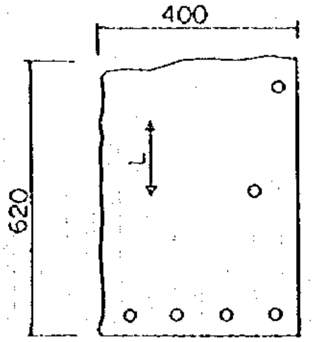
F M TEMPCER
1874
R BROTHER HOOD
GHIPPENHAM
WILTS. 1868

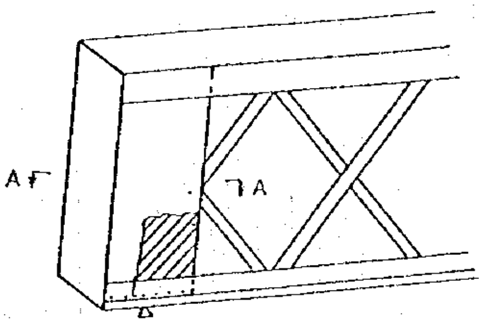
RECONSTRUCTED
GOVERNMENT FACTORY
1916 G.H.M.HYDE

in 1868:Commencement
in 1874:Completed
in 1916:Re-superstructure

B322 38/3km ST·TR/T/COR

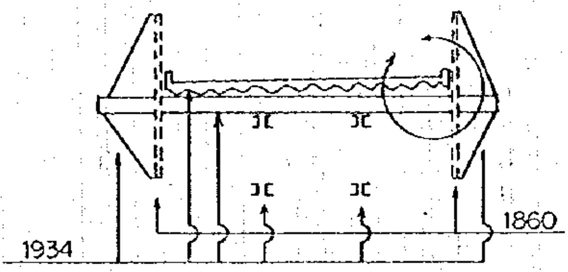
①Giriulla Bridge Year of construction 1880

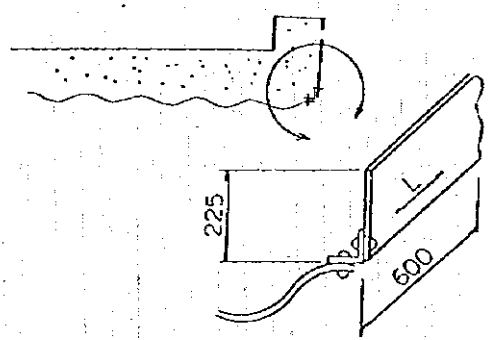




AA004 87/1km ST·TR/T/COR

②Kuruwita Bridge Year of construction OLD 1860 REP 1934

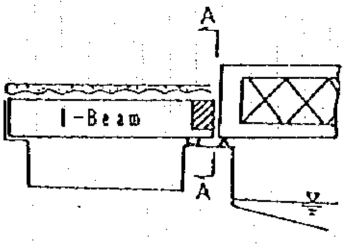


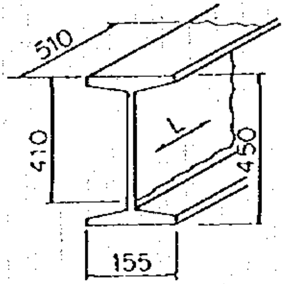


AA002 160/7km RSJ/COR,ST·TR/T/COR

③Matara Old Bridge Year of construction 1860~1874(estimated)

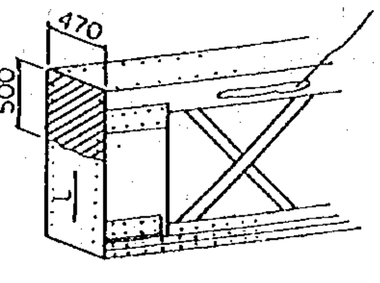
elder 1)Akuressa-Kumburutiya
Nilwalaa Ganga const 1879
2)AA017 Polwatta Ganga const 1874





AA004 102/3km ST·TR/T/COR,ARCH/S

④Warakatota Bridge Year of construction 1909



LANARKSHIPE STEEL CO
SCOTLAND

SIEMENS MARTIN
ACID PROCESS

2. Results of Strength Test of Steel Samples

Table 1 Chemical analysis (w t %)

Sample	C	S i	M n	P	S	C u	N i	C r	M o	V	N b	A l	O
Typical data of wronght iron	0. 02 ~0. 08	0. 10 ~0. 20	<0. 05	0. 10 ~0. 25	<0. 03	—	—	—	—	—	—	—	—
①	0. 003	0. 16	0. 044	0. 29	0. 016	0. 020	0. 029	<0. 005	<0. 005	0. 007	<0. 005	0. 015	—
②	0. 22	0. 048	0. 50	0. 025	0. 022	0. 050	0. 036	0. 034	<0. 005	<0. 005	<0. 005	0. 004	0. 0153
③	0. 051	0. 004	0. 44	0. 036	0. 082	0. 010	0. 040	0. 020	<0. 005	<0. 005	<0. 005	0. 001	0. 0285
④	0. 051	0. 003	0. 54	0. 015	0. 013	0. 036	0. 048	0. 018	<0. 005	<0. 005	<0. 005	0. 001	0. 0088

Re. : ① ; Giriulla Br. 1880 ② ; Kuruwita Br. 1934 ③ ; Matara Br. 1860 ~1870 ④ ; Warakatota Br. 1909

Teble 2-1 Tensile test result

Sample	Thickness(mm)	Direction	Specimen No	YS (N/mm^2)	TS (N/mm^2)	EL (%)
①	7.0	L	1L1	247	334	13
			〃2	253	334	13
			〃3	247	339	12
		T	1C1	244	257	5
			〃2	242	259	7
②	4.5	L	2L1	272	459	32
			〃2	273	457	37
			〃3	283	469	35
		T	2C1	285	466	27
			〃2	282	457	35
③	13.0	L	3L1	249	357	41
			〃2	260	359	42
			〃3	256	356	41
		T	3C1	282	366	44
			〃2	257	366	45
④	6.5	L	4L1	317	357	43
			〃2	320	356	45
			〃3	322	364	44
		T	4C1	340	365	40
			〃2	305	355	42

Re. 1. ① ; Giriulla Br. 1880 ② ; Kuruwita Br. 1934
③ ; Matara Br. 1860 ~1870 ④ ; Warakatota Br. 1909

2. Specimen, in accordance with JIS Z 2201 (Type 5, GL=50mm)
3. Thickness, measured after machine descaling from both surfaces

Table 2-2 Tensile stregth and referrence data

Sample	Direction	* TS N/mm²	Ratio to Sample ②	** Ratio to SS400
①	L	336	0.73	0.84
	T	258		
②	L	462	1.00	——
	T	462		
③	L	357	0.77	0.89
	T	366		
④	L	359	0.78	0.90
	T	360		

＊ ; Average of measured value in Table 2-1

＊＊; Ratio to the specified minimum TS of SS400

Table 3 Young's moduli measured

Sample	Specimen No	Young's moduli kgf/mm²	Sample	Specimen No	Young's moduli kgf/mm²
①	1L1	2.07×10^4	③	3L1	2.11×10^4
	1L2	2.06×10^4		3L2	2.21×10^4
	Av.	2.07×10^4		Av.	2.16×10^4
②	2L1	2.08×10^4	④	4L1	2.08×10^4
	2L2	2.16×10^4		4L2	2.08×10^4
	Av.	2.12×10^4		Av.	2.08×10^4

Re.; Young's moduli are calculated from the tangent line of $\sigma-\varepsilon$ chart

Table 4 Results of Brinell hardness test

Sample	Specimen No	HBS 10/500		Sample	Specimen No	HBS 10/500	
		n 1	n 2			n 1	n 2
①	1 L 1	68.6	69.1	③	3 L 1	66.8	60.5
	〃 2	66.8	69.1		〃 2	62.5	66.8
	〃 3	71.5	74.1		〃 3	64.6	62.5
	1 C 1	71.0	74.1		3 C 1	66.8	64.6
	〃 2	69.1	66.8		〃 2	62.5	62.5
	Av.	70.0			Av.	64.0	
②	2 L 1	76.8	79.6	④	4 L 1	66.8	69.1
	〃 2	80.8	85.7		〃 2	67.3	65.0
	〃 3	88.4	85.7		〃 3	65.0	69.1
	2 C 1	85.1	85.7		4 C 1	69.6	66.8
	〃 2	85.7	79.6		〃 2	64.6	60.5
	Av.	83.4			Av.	66.4	

Re. ; for each specimen two points are selected for test from end of tensile specimen

Table 5 Results of Rockwell hardness test

Sample	HRB					
	n 1	n 2	n 3	n 4	n 5	Av.
①	73.5	74.8	74.7	73.8	75.0	74.4
②	87.8	81.0	81.6	85.0	88.8	84.8
③	54.5	57.0	56.0	54.5	59.2	56.2
④	61.5	61.6	62.3	62.3	62.3	62.0

Re. ; for each sample five points are selected for test from end of a tensile specimen

Table 6 Results of Vickers hardness test

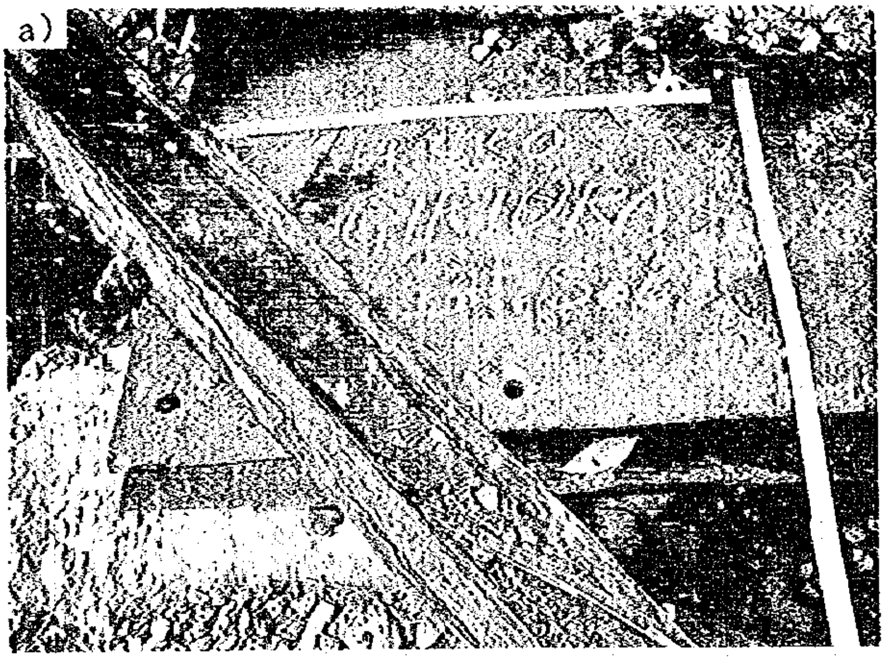
Sample	HV30					
	n 1	n 2	n 3	n 4	n 5	Av.
①	122	118	133	120	138	126
②	126	125	118	129	132	126
③	105	105	106	102	100	104
④	114	115	111	115	113	114

Re. ; for each sample five points are selected from a photomicrograph for hardness test

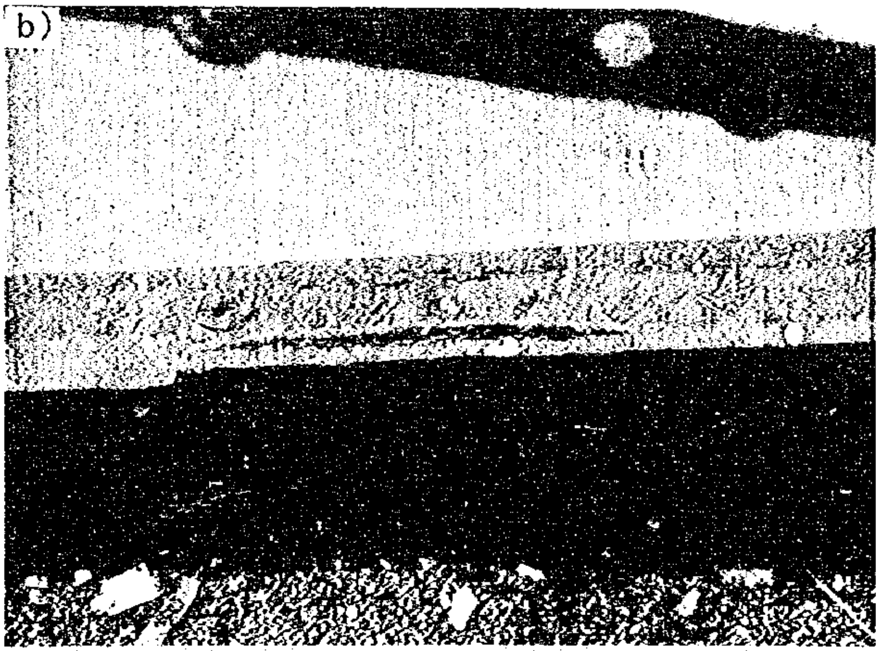
Table 7 Results of Ultrasonic hardness test

Surface condition of measured spesimen	Sample	Diamond prove for micro-vickers hardness test					
		n 1	n 2	n 3	n 4	n 5	Av.
Scale on surface	①	——	——	——	——	——	——
	②	485	476	160	129	——	——
	③	353	293	871	102	——	——
＊1	④	264	79	331	——	——	——
Scale off with grinder	①	142	195	206	149	168	172
	②	101	106	126	116	154	121
	③	67	79	78	91	89	81
＊1	④	102	95	100	100	102	100
Machined	①	162	136	151	144	—	148
	②	133	154	150	149	—	147
	③	85	125	92	99	120	104
＊2	④	82	88	91	77	—	85

Re. ; ＊1 Samples ; Cut blocks for chemical analysis
＊2 Samples ; End part of tensile Specimens



a) Rust of multi-layered state



b) Cracking in lamellar state

Photo. 1 Rust on the wrought iron. Sample ① (Giriulla Bridge 1880)

Appendix - L

STRUCTURAL ASSESSMENT CRITERIA

LIST OF CONTENTS

1. General

The national highways network of Sri Lanka comprises around 11,000 km and the number of existing bridges in this network is about 3,700. Some of the bridges have been built in colonial times and oer 50 to100 years old. Throughout the years various revision of the British Standard, and modification on the application of standard to suit Sri Lankan condition has been carried out resulting in bridges being designed to various loading and design specification. For the purpose of this Study, the design criteria to be applied are based on RDA bridge design practice and the results of meeting between RDA and the Study Team except where the specification is not clear then the Japanese Bridge Design Specification will suffice. The design criteria covers the following aspect of design:-

- Geometric design standard
- Bridge clearance
- Bridge width
- Bridge loading
- Design method
- Material and allowable stress
- Superstructure design
- Substructure design
- Applicable bridge design standard

2. Geometric Design Standard

The geometric design standard to be applied in this Study is based on the RDA standard shown in Table L-1 and head room (vertical clearance) is 5.5m.

Table L-1 Geometric Design Condition

Class of Road	Terrain	Design Traffic Volume (Vhc. per day)	Design Speed (km/hr)	Carriageway Width (m)	Shoulder Width (m)	Absolute Minimum Radium of Curfature (m)	Maximum Gradient (%)	Minimum Sight Distance (m)
A,B	F(Rural)	0 - 500	70	3.7	1.8	100	4	70
C,D	F	0 - 500	50	3.7	1.8	80	4	70
A,B,C,D	R/M	0 - 500	40	3.2	1.5	50	8	50
			50	3.2	1.5	80	8	50
A,B,C	F	501 - 2500	60	6.2	1.8	120	4	90
A,C	R/M	501 - 2500	40	5.5	1.5	60	8	50
			50	5.5	1.5	60	8	50
A,B,C	F/R	2501 - 5000	60	6.2	1.8	120	4	90
A,B	F	5001 - 8000	60	6.8	1.8	120	4	90
			80	6.8	1.8	120	4	140
A,B	F	8001 - 12000	60	7.4	1.8	120	4	90
			80	7.4	1.8	230	4	140
	F	12001 - 32000	60	13.5	1.8	120	4	90
			80	13.5	2.4	230	4	140
	F	32001 - 4000	60	2 / 7.4	2.4	120	4	90
			80	2 / 7.4	2.4	230	4	140
	F	over 40000	60	2 / 11.0	2.4	120	4	90
			80	2 / 11.0	2.4	230	4	140

3. Bridge Width

The width of carriageway and footway on road shall b designed according to the Geometric Design Condition of RDA as shown in Table L-1.

The component of width on bridge was determined in accordance with the results of hearing from RDA as follows:-

Table L-2 Compound of Width on Bridge (Draft)

	Footway (m)	Carriageway (m)	Footway (m)	Total
Important Trunk Road (Inner-city)	1.8	3.7 x 2	1.8	11.0
Important Trunk Road (others)	1.2	3.7 x 2	1.2	9.8
Trunk Road	1.2	3.4 x 2	1.2	9.2

However, Table L-2 is only applicable when the bridges need reconstruction. In case the bridges need repair and reinforcement, Table L-3 shall be applied in order to save construction cost though service level and design speed will be reduced.

Table L-3 Width of Bridge for Repair and Reinforcement (Sub-standard)

Case	Class of Road	Traffic Volume (vpd)	Footway Existed	Footway not Existed
			Carriageway Width	Effective Width
1	A, AB	ADT > 700	W ≧ 6.0m	W ≧ 6.5m
2	B	ADT > 3000	W ≧ 6.0m	W ≧ 6.5m
3	B	1000 < ADT ≦ 3000	W ≧ 5.5m	W ≧ 6.0m
4	A, AB	ADT ≦ 700	W ≧ 3.6m	W ≧ 4.0m
5	B	ADT ≦ 100	W ≧ 3.6m	W ≧ 4.0m

The Table shown above is prepared under conditions as follows:-

- Traffic data of RDA includes 30% of motor cycles, thus the actual amount of the vehicles are around 500 vpd. on A and AB class roads.
- Traffic data of RDA includes 50% of motor cycle, thus the actual amount of vehicles are around 500 vpd. on B class roads.
- The widening shall be carried out along with the schedule of road rehabilitation plan when case 4 or 5 above is applied.

Besides, the width of diversion road during construction was determined in accordance with the hearing from RDA as follows:-

- Nos. of lane : 1 lane
- Width of carriageway : 14 ft to 12 ft = 4.2 m to 3.6 m

4. Free Board

Although hydraulic analysis shall be carried out based on RDA design practice in the cast of reconstruction, the free board requirement is not clearly stated in the RDA Standard, thus the proposed elevation on the 10 bridges to be designed was decided considering the calculation result, the highest flood level and the site condition.

5. Bridge Loading

Loads acting on the bridge structure includes Dead Load, Live Load, Load due to Tractire / Traking Force, Collision Load on bridge parapet, Collision Load on bridge support, Wind Load, Load due to creep, Shrinkage and Temperature, Buoyancy or Uplift Force and Forces of steam current. Since the Study is only concerned on the preliminary design, the bridge loading which is not critical to all type of bridges in the Study will not be considered in the analysis. Hence the load to be considered shall be limited to the following type of loadings:-

- Dead Loads
- Primary Live Loads
- Tractire / Braking Force
- Collision Load on bridge support
- Collision Load on bridge parapet
- Pedestrian Load (footway loading)
- Forces due to stream current
- Forces due to earth pressure

(1) Design Criteria for Live Loading

1) Design live loading for repair and reinorcement

The design data and information, to determine loading capacity of bridges, such as strength of material and design conditions at the date of construction are not existed. The Study Team setted up the criteria under the conditions below:-

- Specific live loading is not determined
- The repair is assumed only to remain existing loading capacity

As the results of study on loading capacity of the existing bridges, it was found that T-16 to 18 live loading in accordance with Japanese Bridge Design Standard was applicable for repair and reinorcement of the existing bridges. The design live loading for widening also shall be applied in accordance with T-16 to 18 live loading above mentioned.

2) Design live loading for replacement and reconstruction

RDA Standard which follows BS 5400 shall be applied basically.

(2) Dead Loads

The unit weight of bridge construction material as given in Table L-4 below may be used for calculation of the dead load:-

Table L-4 The Unit Weight of Bridge Construction Material

Material	Unit Weight (KN/cu.m)
Reinforced Concrete	25
Prestessed Concrete	25
Asphalt Pavement	23
Steel or Cast Steel	77
Cast Iron	71
Alluminium Alloys	28
Timber	8
Stone Masonry	27
Bituminous Water Proofing Material	11
Compacted Sand, Earth or Gravel	19
Loose Sand, Earth or Gravel	16

(3) Primary Live Loads

Live load to be applied in the design of reconstruction shall be HA and HB loading.

the number of notional lanes for the 7.4 m wide carriageway is 2 in accordance with Part 2, Clauses 3.2.9.3.1 of BS 5400.

As this is a theoretical assumption, the calculations with the likewise theoretical uniformly distributed load (HA) are straight forward (see Fig. L-1) while special consideration has to be given to the application of the abonrmal HB-vehicle (see Fig. L-2 & 3).

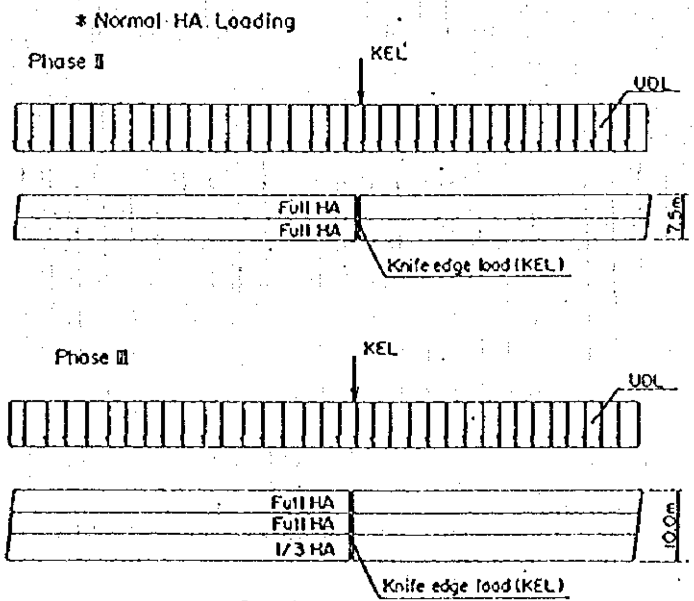
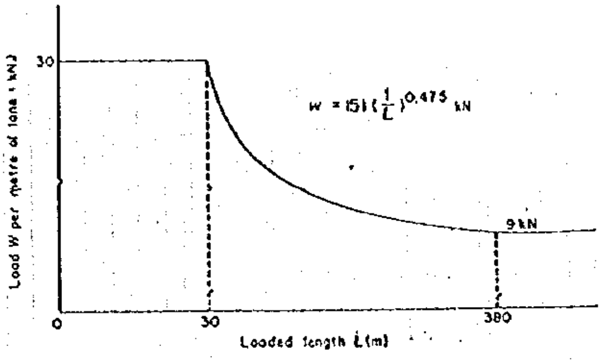


Fig. L - 1

The values to applied for uniformly distributed load (UDL) depending on the loaded length and for the knife edge load (KEL) at 120 kN per lane shall be considered as specified in Clause 6.2.1 (Table 13) and Clause 6.2.2 respectively.

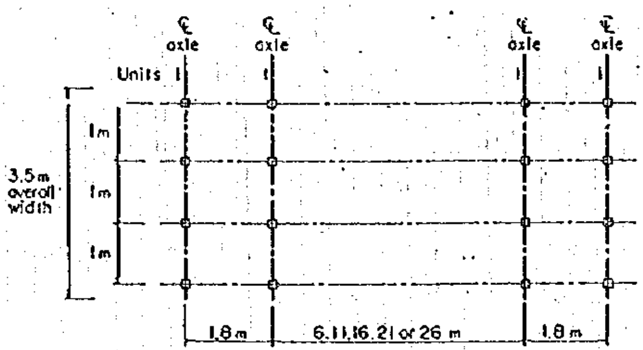
Fig. L - 2



LOADING CURVE FOR HA(UDL)

* Abnormal HB loading

Fig. L - 3



Dimension and axle arrangement of the abnormal HB vehicle as shown. For one unit of HB-loading, all four axle loads are to be considered equally at 10 kN (2.5 kN per wheel).

HB loading requirements derive from the possible occurrance of exceptional industrial loads as, for instance, electrical transformers, generators, pressure vessels, etc. Generally, such loads will be trnasported on the roads only under police escort and due to the oversized dimensions of the vehicle cum load, ordinary traffic will be restricted, while such transport is passing. For a bridge in this Study, it appears advisable to consider the exceptional HB loading at 45 units, however, by reason of the above, no accompanying HA loads would have to be considered.

Longitudinal Direction

It easily can be shown that the strain to the main girder under HA loading in all respects is bigger than for 45 units HB loading alone or for 30 units HB together with accompanying HA loading in accordance with Clause 6.4.2.

(4) Drift Wood Collision

Assuming that the weight of drift wood is 100kN, and the current velocity at the surface is 3.0 m/sec. the collision force is estimated as 100kN. The acting point of it is the highest water level.

(5) Footway Live Loading

Footway live loading to be used in the Study shall be taken as 80% of 5.0 kN/m² (see 7.2.1, Part 2).

(6) Forces due to Earth Pressure

The earth pressure acting on a wall surface shall be calculated by Coulomb's formula, taking account of soil cohesion for cohesive soil.

6. Design Method Applied

The assessment of the existing bridges and the design of rehabilitation work on the existing bridges were carried out in accordance with elastic design method (allowable stress design method), while for adding a footway which is not attached to the existing bridge or a new bridge for total replacement, the design was carried out using limit state design method.

The reasons for adopting thest two different design methods in the preliminary rehabilitation design are:

- For assessment of the load carrying capacity, bridge history (year constructed, materials used, specification adopted), and design data and information are essential, while many bridges studies have not information and some of them were constructed in colonial times and are over 50 to 100 years old. These bridges were designed to various specifications which follows the elastic design principle.

- Quality of materials used in the studies bridges are also various. (That is strength variation is very wide.)

Thus, it is safe to apple elastic design method for the assessment and rehabilitation work. Contrarily, quality of material and accuracy of design for a new independent structure can be controlled properly within a very low tolerance. Accordingly, it is rational to apply limit state design methods only for an indepenent structure which will not be attached to the existing bridge.

The elastic design method was based on the Bridge Design Specification in Japan, while for limit state design the provisions prescribed in BS 5400 was applied.

Free board under a bridge shall be determined taking into consideration the necessary space needed for river navigational ways and maintenance, etc. The river administrative clearance from the bottom of the bridge girder or beam to design flood water level will vary from 0.5 m to 1.5 m depending on the size of river if the River Design Standard in Japan would be adopted.

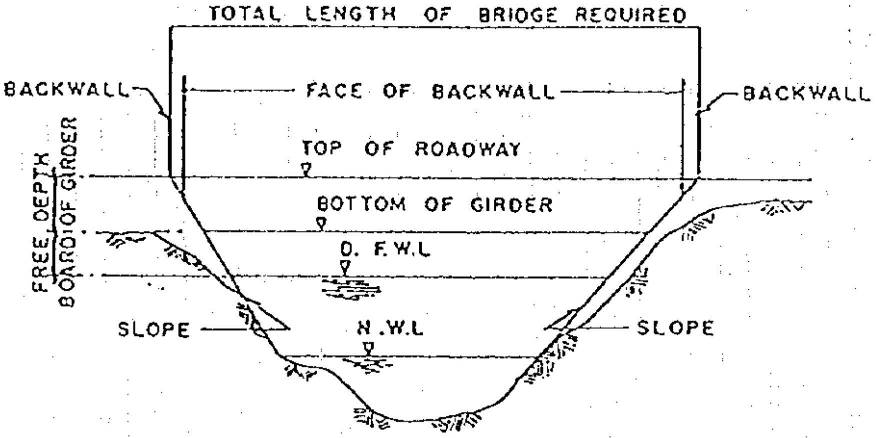


Figure L-4

The design elevation of the bottom of bridge shall not be lower than the highest water level plus the free board.

Free board (below the bridge) - For non navigable river, general clearance between D.F.W.L. and the bottom of the lowest member of superstructures shall comply with 0.6 m.

However, it was difficult in some case of peliminary design of 10 bridges to suffice the above criteria by the surrounding condition of bridge site. Therefore, the design elevation of the bottom of bridge was adjusted considering a situation of each bridge.

(1) Applicable Bridge Types

To selct the applicable types of superstructure, substructure and foundation, the basic and important factors to be taken into consideration shall include economical construction, stability and safety, shorter construction period and ease of maintenance and operation.

7. Bridge Planning (Applicable to only replacement or reconstruction)

(1) Determination of Bridge Length

The clearances of a bridge controls the bridge's length as indicated in the following. From the intersection of ordinary water level and ground surface as shown in the sketch below, the proposed slopes of protection work follow the slope of the bank as close as possible, having in mind not to constrict the area of the water way required. Then the top of roadway elevation was determined based on the Design Flood Water Level (DFWL).

The distance between the intersections of the slopes of protection work and the top of roadway elevation represents the length of bridge required, which is the total distance between the back of backwalls. Minor adjustments shall be made, if necessary, to suit the length of standard type of superstructure to be adopted.

Table L-5 Applicable Types of Concrete Bridge

TYPE	SPAN LENGTH (m)	HEIGHT /SPAN
1. R.C. Slab	3 – 8	1/20
2. R.C. Hollow slab	6 – 20	1/20
3. R.C. T-Beam	10 – 20	1/13
4. P.C. Hollow (pretention)	10 – 21	1/14
5. P.C. I-Beam (pretention)	6 – 13	1/15
6. P.C. T-Beam (pretension)	10 – 21	1/15
7. P.C. I-Beam (post)	20 – 36	1/16
8. P.C. T-Beam (post)	20 – 46	1/17.5
9. Simple Box Girder	30 – 41	1/2.0
10. R.C. Arch	40 – 70	1/6.5

TYPE	SPAN LENGTH (m)	HEIGHT /SPAN
11. Cantilever Box Girder	60 – 240	1/13
12. P.C. Cable Stayed Girder	130 – 330	–

Table L-6 Applicable Type of Steel Bridge

TYPE	SPAN LENGTH (m)	HEIGHT SPAN
1. Steel I Beam (non-comp.)	0 – 10	1/200
2. Steel I Beam (Comp.)	10 – 25	1/22
3. Simple Plate Girder	15 – 40	1/17
4. Continous Plate Girder	25 – 65	1/18
5. Simple Comp. Girder	20 – 50	1/18
6. Simple Box Girder	30 – 60	1/22
7. Continuous Comp. Girder	40 – 65	1/19
8. Continuous Box Girder	50 – 100	1/23
9. Simple Truss	50 – 100	1/8
10. Continous Truss	60 – 150	1/9
11. Stabogen Bridge	60 – 150	1/8.5
12. Cable Stayed Girder	100 – 600	1/100 1/15

Table L-5 and Table L-6 show the relationship between the superstructure type and the span length based on the samples of bridges. The following items are fundamental in the selection of superstructure types:

- Reinforced concrete structures re initially considered except for special requirements of steel structure because of easier maintenance.
- Reinforced concrete beam and steel I-beam types are applicable for short span length (10m to 15m).
- Prestressed concrete girder, and steel plate girder types are applicable for medium span length (20m to 50m).
- Prestressed concrete box girder, steel through truss and ranger girder types are to be applied for long span length (60m to 150m).

Table L-7 Applicable Types of Pier

TYPE		HEIGHT (m) 0 10 20 30 40	REMARKS
P-1	Column Type	0 15	
P-2	Rigid Frame Type (1 storey)	5 15	
P-3	Rigid Frame Type (2 storey)	15 25	
P-4	Wall Type	10 30	
P-5	Wall Type (1 storey)	25 40	

Table L-8 Applicable Types of Abutment

TYPE		HEIGHT (m) 0 10 20 30	REMARKS
A-1	Chair Type	3	
A-2	Gravity Type	4	
A-3	Semi Gravity Type	4 6	
A-4	Inverse T-Type	5 10	
A-5	Buttressed Type	10 15	
A-6	Box Type	10 20	
A-7	Sustaining Wall Type	10 15	

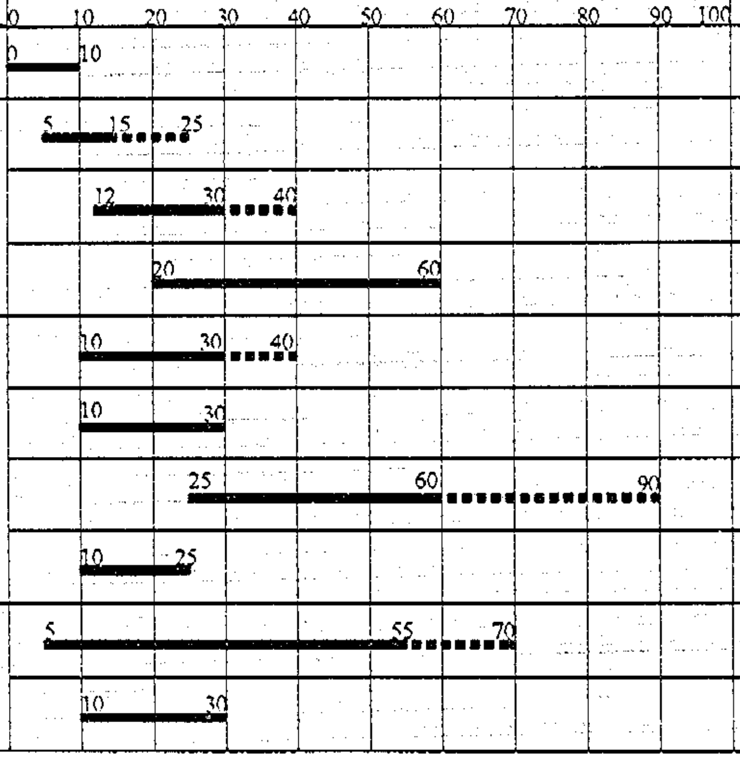
Table L-7 and L-8 show the applicable substructure types in accordance with the required structural height of a bridge. The selection of substructure types is based not only on specified figures but also on the following considerations:

- Reinforced concrete structures
- The cross section of peir column in the river is circular or elliptical and rectangular shape with no restricted conditions.
- Non sliding of the back fill materials behind abutment structure is considered in the selection in the abutment type to avoid the approach settlement.

Table L-9 shows the applicable foundation types in accordance with the required effective depth to sustain the upper-structures. The followings are considered in selecting the foundation type:-

- Possible construction depth is studies in consideration of soil conditions.
- The advantageous type is considered for works above water e.g. reverse circulation drill pile.
- the prefabricated pile types are advantageous when the bearing stratum is within a shallow range.

Table L-9 Applicable Types of Foundation



TYPE / DEPTH			DEPTH (m)	USABLE DIA. (m)	SOIL CONDITION	
					CLAYEY	SANDY
F-1	Spread Foundation		0 10	–	○	○
F-2	R.C. Pile	DRIVEN PILE	5 15 25	0.3-0.5	△	△
F-3	P.C. Pile		12 30 40	0.35-0.5	△	△
F-4	Steel Pipe Pile		20 60	0.5-0.8	○	○
F-5	Cast in Place W/Casing	CAST-IN-PLACE PILE	10 30 40	1.0-1.2		△
F-6	Earth Auger		10 30	1.0-1.5	○	×
F-7	Reverse Circulation Drill		25 60 90	1.0-1.2	○	×
F-8	Shinso Pile		10 25	2.0-5.0	—	—
F-9	Open Caisson	CAISSON	5 55 70	–	—	—
F-10	Pneumatic Caisson		10 30	–	—	—