CALCULATION OF LOAD RATING

PROJECT TITLE: PASIG-MARIKINA RIVER BRIDGE INSPECTION, LAMBINGAN BRIDGE
ITEM: ANALYTICAL ASSESMENT OF BRIDGE STRUCTURAL **EXTERIOR GIRDERS**

RATING METHOD: ALLOWABLE STRESS AT INVENTORY LEVEL					
STRESSES	SECTION		SPAN A1 TO P2 / A2 TO P3	SPAN A1 TO P2 / A2 TO P3	SPAN P1 TO P2
			MIDSPAN	SUPPORT	MIDSPAN
TDL=PS+DL+SDL	BOTTOM	Mpa	-7.83	6.06	-15.49
	TOP		-4.75	-13.37	-3.37
LL (HS20)	BOTTOM	Mpa	1.02	1.98	-3.05
	TOP		-1.69	-2.54	5.07
Allowable Stress	Compression	Mpa	-21.00	-21.00	-21.00
	Tension		2.96	2.96	2.96
RATING FACTOR (RF=(Cap-TDL)/LL	BOTTOM		4.78	**-1.57**	1.81
	TOP		4.55	3.01	1.25
Equivalent LL(HS20)	RF*(HS20)	tons	145.48	**-50.20**	39.97

RATING METHOD: LOAD FACTOR				
FORCES / SECTION		SPAN A1 TO P2 / A2 TO P3	SPAN A1 TO P2 / A2 TO P3	SPAN P1 TO P2
		MIDSPAN	SUPPORT	MIDSPAN
Moment, DL+SDL	kN-m	2326.00	8674.00	9060.00
Moment, LL+I	kN-m	843.16	1805.19	2560.80
Width of Flange, b	mm	2000.00	2000.00	2000.00
Depth of Composite Section, d	mm	1325.00	1779.00	1828.87
Comp. Strength of Conc., f_c'	Mpa	35.00	35.00	35.00
Ultimate Stress of PS Strands., f_s'	Mpa	1862.00	1862.00	1862.00
Area of PS Strands, A_s*	mm^2	4737.60	4737.60	5922.00
Steel Ratio, ρ*		0.0018	0.0013	0.0016
f_{su}*	Mpa	1773.45	1796.05	1781.81
Neutral Axis, NA Bottom	mm	1.29	1.15	1.29
$R = \phi M_n = \phi A_s{}^* f_{su}{}^* d(1-0.6\rho^* f_{su}{}^*/f_c')$	kN-m	10527.46	14516.86	18343.66
RATING FACTOR: INVENTORY LEVEL RF=(R-1.3(DL+SDL))/1.3*1.67LL		4.10	**0.83**	1.18
RATING FACTOR: OPERATING LEVEL RF=(R-1.3(DL+SDL))/1.3LL		6.85	1.38	1.97

CALCULATION OF LOAD RATING

PROJECT TITLE: PASIG-MARIKINA RIVER BRIDGE INSPECTION, LAMBINGAN BRIDGE
ITEM: ANALYTICAL ASSESMENT OF BRIDGE STRUCTURAL **INTERIOR GIRDERS**

RATING METHOD: ALLOWABLE STRESS AT INVENTORY LEVEL					
STRESSES	SECTION		SPAN A1 TO P2 / A2 TO P3	SPAN A1 TO P2 / A2 TO P3	SPAN P1 TO P2
			MIDSPAN	SUPPORT	MIDSPAN
TDL=PS+DL+SDL	BOTTOM	Mpa	-4.73	5.50	-13.94
	TOP		-7.21	-12.94	-1.33
LL (HS20)	BOTTOM	Mpa	0.76	1.48	-2.35
	TOP		-1.55	-2.27	4.78
Allowable Stress	Compression	Mpa	-21.00	-21.00	-21.00
	Tension		2.96	2.96	2.96
RATING FACTOR	BOTTOM		2.33	**-1.72**	3.00
(RF=(Cap-TDL)/LL	TOP		6.58	3.55	**0.90**
Equivalent LL(HS20)	RF*(HS20)	tons	74.58	**-54.95**	**28.69**

RATING METHOD: LOAD FACTOR				
FORCES / SECTION		SPAN A1 TO P2 / A2 TO P3	SPAN A1 TO P2 / A2 TO P3	SPAN P1 TO P2
		MIDSPAN	SUPPORT	MIDSPAN
Moment, DL+SDL	kN-m	2240.00	8434.00	8772.00
Moment, LL+I	kN-m	757.73	1577.37	2378.40
Width of Flange, b	mm	2000.00	2000.00	2000.00
Depth of Composite Section, d	mm	1325.00	1779.00	1828.87
Comp. Strength of Conc., f_c'	Mpa	35.00	35.00	35.00
Ultimate Stress of PS Strands., f_s'	Mpa	1862.00	1862.00	1862.00
Area of PS Strands, A_s*	mm^2	4737.60	4737.60	5922.00
Steel Ratio, ρ*		0.0018	0.0013	0.0016
f_{su}*	Mpa	1773.45	1796.05	1781.81
Neutral Axis, NA Bottom	mm	1.29	1.15	1.29
$R = \phi M_n = \phi A_s{}^* f_{su}{}^* d(1-0.6\rho^* f_{su}{}^*/f_c')$	kN-m	10527.46	14516.86	18343.66
RATING FACTOR: INVENTORY LEVEL RF=(R-1.3(DL+SDL))/1.3*1.67LL		4.63	1.04	1.34
RATING FACTOR: OPERATING LEVEL RF=(R-1.3(DL+SDL))/1.3LL		7.73	1.73	2.24

EVALUATION FOR PRESTRESSED CONCRETE GIRDER BRIDGE USING ALLOWABLE STRESS

FOR SPAN A1 TO P1 / A2 TO P2 - AT MIDSPAN
TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type IV	0.743	0.406	1.044	0.996
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.100	0.696	1.399	0.841
Live Load MS-18	1.100	0.696	1.399	0.841

TABLE B: MOMENT DEMAND FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	1449.00
Due to Weight of Girder + Slab + Diaphragm	2071.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	255.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact *Distribution factor	681.00
With Impact	843.16
Load Combination at Service Condition	
DL + (LL+I)	3169.16

I = 100*(15.24 / L + 38) = 24 %

TABLE C: STRESSES AT MIDSPAN FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

Prestressing Force, Pf = 4631.24 kN (Assumed: 4-12T 12.7mmØ)

Eccentricity:

For Basic Section = 0.615 m

For Composite Section = 0.541 m (Superimposed Loads)

For Composite Section = 0.541 m (Live Loads)

After Transfer:

f_c' = 35 MPa

Allowable Stress in Compression = 0.60 f_c' = -21.00 MPa

Allowable Stress in Tension = 0.5 $\sqrt{f_c'}$ = 2.96 MPa

LOAD DESCRIPTION	STRESSES (MPa)			
	Top Fiber		Bottom Fiber	
Sresses due Dead Loads (Girder+Slab+Diaphragm Weight+Prestressing)	-8.14	C	-4.23	C
Sresses due to Superimposed Loads	0.31	T	-0.51	C
Sresses due to all Live Load + Impact	1.02	T	-1.69	C

$$RF = \frac{\text{Allowable Stress - (Stress due to Dead Loads + Stress due to Superimposed Loads)}}{\text{Stress due to Live Load + Impact}}$$

RF = 4.78 -At Top fiber

RF = 4.55 -At Bottom fiber

EVALUATION FOR PRESTRESSED CONCRETE GIRDER BRIDGE USING ALLOWABLE STRESS

FOR SPAN A1 TO P1 / A2 TO P2 - AT SUPPORT
TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type IV	1.518	0.552	1.051	0.989
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.874	0.895	1.258	0.982
Live Load MS-18	1.874	0.895	1.258	0.982

TABLE B: MOMENT DEMAND FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	5550.00
Due to Weight of Girder + Slab + Diaphragm	7808.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	866.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact *Distribution factor	1458.00
With Impact	1805.19
Load Combination at Service Condition	
DL + (LL+I)	10479.19

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 24 %

TABLE C: STRESSES AT SUPPORT FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

Prestressing Force, Pf = 4939.99 kN (Assumed: 4-12T 12.7mmØ)
Eccentricity:
For Basic Section = 0.635 m
For Composite Section = 0.703 m (Superimposed Loads)
For Composite Section = 0.703 m (Live Loads)
After Transfer:
f_c' = 35 MPa
Allowable Stress in Compression = 0.60 f_c' = -21.00 MPa
Allowable Stress in Tension = 0.5 $\sqrt{f_c'}$ = 2.96 MPa

LOAD DESCRIPTION	STRESSES (MPa)			
	Top Fiber		Bottom Fiber	
Sresses due Dead Loads (Girder+Slab+Diaphragm Weight+Prestressing)	5.11	T	-12.15	C
Sresses due to Superimposed Loads	0.95	T	-1.22	C
Sresses due to all Live Load + Impact	1.98	T	-2.54	C

$$RF = \frac{\text{Allowable Stress - (Stress due to Dead Loads + Stress due to Superimposed Loads)}}{\text{Stress due to Live Load + Impact}}$$

RF = -1.57 -At Top fiber
RF = 3.01 -At Bottom fiber

EVALUATION FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI BRIDGE USING ALLOWABLE STRESS

FOR SPAN P1 TO P2 - AT MIDSPAN
TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type VI Modified	0.743	0.406	1.044	0.996
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.100	0.696	1.399	0.841
Live Load MS-18	1.100	0.696	1.399	0.841

TABLE B: MOMENT DEMAND FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI (D=1.829M; L=46.100M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	5297.00
Due to Weight of Girder + Slab + Diaphragm	7965.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	1095.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact *Distribution factor	2134.00
With Impact	2521.17
Load Combination at Service Condition	
DL + (LL+I)	3616.17

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 18 %

TABLE C: STRESSES AT MIDSPAN FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI (D=1.829M; L=46.100M)

Prestressing Force, P = 7409.99 kN (Assumed: 5-12T 12.7mmØ)
Eccentricity:
For Basic Section = 0.844 m
For Composite Section = 1.199 m (Superimposed Loads)
For Composite Section = 1.199 m (Live Loads)
After Transfer:
f_c' = 35 MPa
Allowable Stress in Compression = 0.60 f_c' = -21.00 MPa
Allowable Stress in Tension = 0.5 $\sqrt{f_c'}$ = 2.96 MPa

LOAD DESCRIPTION	STRESSES (MPa)			
	Top Fiber		Bottom Fiber	
Sresses due Dead Loads (Girder+Slab+Diaphragm Weight+Prestressing)	-14.17	C	-5.57	C
Sresses due to Superimposed Loads	-1.32	C	2.20	T
Sresses due to all Live Load + Impact	-3.05	C	5.07	T

$$RF = \frac{\text{Allowable Stress - (Stress due to Dead Loads + Stress due to Superimposed Loads)}}{\text{Stress due to Live Load + Impact}}$$

RF = 1.81 -At top fiber
RF = 1.25 -At bottom fiber

EVALUATION FOR PRESTRESSED CONCRETE GIRDER BRIDGE USING LOAD FACTOR

FOR SPAN A1 TO P1 / A2 TO P2 - AT MIDSPAN

TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type IV	1.38	0.308	0.949	0.88
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.058	0.551	1.290	0.770
Live Load MS-18	1.058	0.551	1.290	0.770

TABLE B: MOMENT DEMAND FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	1449.00
Due to Weight of Girder + Slab + Diaphragm	2071.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	255.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact	681.00
With Impact	843.16
Load Combination at Service Condition	
DL + (LL+I)	3169.16

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 24 %

CALCULATION OF MOMENT CAPACITY AT MIDSPAN
CONSIDERING PRESTRESSING STEEL ONLY:

$A_S{}^* =$ 4737.60 mm² $f'_c =$ 35 MPa $f'_s =$ 1862.00 MPa b = 2000.00 mm d = 1325.00 mm

$\rho^* =$ 0.00179 $\gamma^* =$ 0.40 - for stress-relieved steel $\beta_1 =$ 0.80 - for f'c = 35.00 Mpa $\phi =$ 1.00

$$f_{su}{}^* = f'_s \ \{1 - [(\gamma^*/\beta_1)(\rho^* f'_s / f'_c)]\}$$

$f_{su}{}^* =$ 1773.45 MPa

$$\text{Compression Block} = \frac{A_S{}^* f_{su}}{0.85 f'_C b} = 141.2 \text{ mm} < t_{slab} = 200 \text{ mm}$$ -Consider rectangular section

$$\phi M_n = \phi A_s{}^* f_{su}{}^* d \ [1 - (0.6 (\rho^* f_{su}{}^* / f'_c)]$$

$\phi M_n =$ 10527.5 kN

LOAD RATING:

$\gamma_D =$ 1.30 $\gamma_L =$ 2.17 (Inventory Level) D = 2326.00 kN LL + I = 843.16 kN-m

$\gamma_L =$ 1.30 (Operating Level)

INVENTORY LEVEL:

$$RF = \frac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)} = 4.10$$

OPERATING LEVEL:

$$RF = \frac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)} = 6.85$$

EVALUATION FOR PRESTRESSED CONCRETE GIRDER BRIDGE USING LOAD FACTOR

FOR SPAN A1 TO P1 / A2 TO P2 - AT SUPPORT

TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type IV	1.38	0.405	0.885	0.944
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.694	0.828	1.153	0.876
Live Load MS-18	1.694	0.828	1.153	0.876

TABLE B: MOMENT DEMAND FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	5550.00
Due to Weight of Girder + Slab + Diaphragm	7808.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	866.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact	1458.00
With Impact	1805.19
Load Combination at Service Condition	
DL + (LL+I)	10479.19

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 24 %

CALCULATION OF MOMENT CAPACITY AT SUPPORT
CONSIDERING PRESTRESSING STEEL ONLY:

$A_S^* =$ 4737.60 mm^2 $f'_c =$ 35 MPa $f'_s =$ 1862.00 MPa $b =$ 2000.00 mm $d =$ 1779.00 mm
$\rho^* =$ 0.00133 $\gamma^* =$ 0.40 - for stress-relieved steel $\beta_1 =$ 0.80 - for f'c = 35.00 Mpa $\phi =$ 1.00

$$f_{su}^* = f'_s \{1 - [(\gamma^*/\beta_1)(\rho^* f'_s / f'_c)]\}$$
$f_{su}^* =$ 1796.05 MPa

$$\text{Compression Block} = \frac{A_S^* f_{su}}{0.85 f'_C b} = 143.0 \text{ mm} < t_{slab} = 200 \text{ mm}$$ -Consider rectangular section

$$\phi M_n = \phi A_s^* f_{su}^* d \; [1 - (0.6(\rho^* f_{su}^* / f'_c)]$$
$\phi M_n =$ 14516.9 kN-m

LOAD RATING:

$\gamma_D =$ 1.30 $\gamma_L =$ 2.17 (Inventory Level) D = 8674.00 kN-m LL + I = 1805.19 kN-m
$\gamma_L =$ 1.30 (Operating Level)

INVENTORY LEVEL:

$$RF = \frac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)} = 0.83$$

OPERATING LEVEL:

$$RF = \frac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)} = 1.38$$

EVALUATION FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI

FOR SPAN P1 TO P2 - AT MIDSPAN

TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type VI Modified	0.701	0.308	0.949	0.880
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.058	0.551	1.290	0.770
Live Load MS-18	1.058	0.551	1.290	0.770

TABLE B: MOMENT DEMAND FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI (D=1.829M; L=46.100M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	5297.00
Due to Weight of Girder + Slab + Diaphragm	7965.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	1095.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact	2134.00
With Impact	2560.80
Load Combination at Service Condition	
DL + (LL+I)	3655.80

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 18 %

CALCULATION OF MOMENT CAPACITY AT MIDSPAN:
CONSIDERING PRESTRESSING STEEL ONLY:

$A_S{}^* =$ 5922.00 mm^2 $f'_c =$ 35 MPa $f'_s =$ 1862.00 MPa b = 2000.00 mm d = 1828.87 mm

$\rho^* =$ 0.00162 $\gamma^* =$ 0.40 - for stress-relieved steel $\beta_1 =$ 0.80 - for f'c = 35.00 Mpa $\phi =$ 1.00

$f_{su}{}^* = f'_s \; \{1 - [(\gamma^* / \beta_1)(\rho^* f'_s / f'_c)]\}$

$f_{su}{}^* =$ 1781.81 MPa

Compression Block $= \dfrac{A_S{}^* f_{su}}{0.85 f'_C b}$ = 177.3 mm < t_{slab} = 200 mm -Consider rectangular section

$\phi M_n = \phi A_s{}^* f_{su}{}^* d \; [1 - (0.6(\rho^* f_{su}{}^* / f'_c)]$

$\phi M_n =$ 18343.7 kN

LOAD RATING: D = 9060.00 kN LL + I = 2560.80 kN-m

$\gamma_D =$ 1.30 $\gamma_L =$ 2.17 (Inventory Level)

$\gamma_L =$ 1.30 (Operating Level)

INVENTORY LEVEL:

$RF = \dfrac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)}$ = 1.18

OPERATING LEVEL:

$RF = \dfrac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)}$ = 1.97

EVALUATION FOR PRESTRESSED CONCRETE GIRDER BRIDGE USING ALLOWABLE STRESS

FOR SPAN A1 TO P1 / A2 TO P2 - AT MIDSPAN
TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type IV	0.743	0.406	1.044	0.996
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.053	0.670	1.367	0.673
Live Load MS-18	1.053	0.670	1.367	0.673

TABLE B: MOMENT DEMAND FOR PRESTRESSED CONCRETE GIRDER
(D=1.829M; L=26.000M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	1446.00
Due to Weight of Girder + Slab + Diaphragm	1985.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	255.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact *Distribution factor	612.00
With Impact	757.73
Load Combination at Service Condition	
DL + (LL+I)	2997.73

I = 100*(15.24 / L + 38) = 24 %

TABLE C: STRESSES AT MIDSPAN FOR PRESTRESSED CONCRETE GIRDER
(D=1.829M; L=26.000M)

Prestressing Force, Pf = 4322.49 kN (Assumed: 5-12T 12.7mmØ)

Eccentricity:

For Basic Section = 0.381 m

For Composite Section = 0.449 m (Superimposed Loads)

For Composite Section = 0.449 m (Live Loads)

After Transfer:

f_c' = 35 MPa

Allowable Stress in Compression = 0.60 f_c' = -21.00 MPa

Allowable Stress in Tension = 0.5 $\sqrt{f_c'}$ = 2.96 MPa

LOAD DESCRIPTION	STRESSES (MPa)			
	Top Fiber		Bottom Fiber	
Sresses due Dead Loads (Girder+Slab+Diaphragm Weight+Prestressing)	-4.99	C	-6.69	C
Sresses due to Superimposed Loads	0.26	T	-0.52	C
Sresses due to all Live Load + Impact	0.76	T	-1.55	C

$$RF = \frac{\text{Allowable Stress - (Stress due to Dead Loads + Stress due to Superimposed Loads)}}{\text{Stress due to Live Load + Impact}}$$

RF = 2.33 -At Top fiber

RF = 6.58 -At Bottom fiber

EVALUATION FOR PRESTRESSED CONCRETE GIRDER BRIDGE USING ALLOWABLE STRESS

FOR SPAN A1 TO P1 / A2 TO P2 - AT SUPPORT
TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type IV	1.518	0.552	1.051	0.989
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.828	0.858	1.236	0.804
Live Load MS-18	1.828	0.858	1.236	0.804

TABLE B: MOMENT DEMAND FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	5599.00
Due to Weight of Girder + Slab + Diaphragm	7568.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	866.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact *Distribution factor	1274.00
With Impact	1577.37
Load Combination at Service Condition	
DL + (LL+I)	10011.37

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 24 %

TABLE C: STRESSES AT SUPPORT FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

Prestressing Force, Pf = 4939.99 kN (Assumed: 5-12T 12.7mmØ)
Eccentricity:
For Basic Section = 0.635 m
For Composite Section = 0.703 m (Superimposed Loads)
For Composite Section = 0.703 m (Live Loads)
After Transfer:
f_c' = 35 MPa
Allowable Stress in Compression = 0.60 f_c' = -21.00 MPa
Allowable Stress in Tension = 0.5 $\sqrt{f_c'}$ = 2.96 MPa

LOAD DESCRIPTION	STRESSES (MPa)			
	Top Fiber		Bottom Fiber	
Sresses due Dead Loads (Girder+Slab+Diaphragm Weight+Prestressing)	4.68	T	-11.69	C
Sresses due to Superimposed Loads	0.81	T	-1.25	C
Sresses due to all Live Load + Impact	1.48	T	-2.27	C

$$RF = \frac{\text{Allowable Stress - (Stress due to Dead Loads + Stress due to Superimposed Loads)}}{\text{Stress due to Live Load + Impact}}$$

RF = -1.72 -At Top fiber
RF = 3.55 -At Bottom fiber

EVALUATION FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI BRIDGE USING ALLOWABLE STRESS

FOR SPAN P1 TO P2 - AT MIDSPAN
TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type VI Modified	0.743	0.406	1.044	0.996
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.053	0.670	1.367	0.673
Live Load MS-18	1.053	0.670	1.367	0.673

TABLE B: MOMENT DEMAND FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI (D=1.829M; L=46.100M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	5311.00
Due to Weight of Girder + Slab + Diaphragm	7677.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	1095.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact *Distribution factor	1982.00
With Impact	2341.59
Load Combination at Service Condition	
DL + (LL+I)	3436.59

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 18 %

TABLE C: STRESSES AT MIDSPAN FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI (D=1.829M; L=46.100M)

Prestressing Force, P = 6174.99 kN (Assumed: 5-12T 12.7mmØ)
Eccentricity:
For Basic Section = 0.944 m
For Composite Section = 1.267 m (Superimposed Loads)
For Composite Section = 1.267 m (Live Loads)
After Transfer:
f_c' = 35 MPa
Allowable Stress in Compression = 0.60 f_c' = -21.00 MPa
Allowable Stress in Tension = 0.5 $\sqrt{f_c'}$ = 2.96 MPa

LOAD DESCRIPTION	STRESSES (MPa)			
	Top Fiber		Bottom Fiber	
Sresses due Dead Loads (Girder+Slab+Diaphragm Weight+Prestressing)	-12.84	C	-3.56	C
Sresses due to Superimposed Loads	-1.10	C	2.23	T
Sresses due to all Live Load + Impact	-2.35	C	4.78	T

$$RF = \frac{\text{Allowable Stress - (Stress due to Dead Loads + Stress due to Superimposed Loads)}}{\text{Stress due to Live Load + Impact}}$$

RF = 3.00 -At top fiber
RF = 0.90 -At bottom fiber

EVALUATION FOR PRESTRESSED CONCRETE GIRDER BRIDGE USING LOAD FACTOR

FOR SPAN A1 TO P1 / A2 TO P2 - AT MIDSPAN

TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type IV	1.38	0.308	0.949	0.88
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.058	0.551	1.290	0.770
Live Load MS-18	1.058	0.551	1.290	0.770

TABLE B: MOMENT DEMAND FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	1446.00
Due to Weight of Girder + Slab + Diaphragm	1985.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	255.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact	612.00
With Impact	757.73
Load Combination at Service Condition	
DL + (LL+I)	2997.73

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 24 %

CALCULATION OF MOMENT CAPACITY AT MIDSPAN
CONSIDERING PRESTRESSING STEEL ONLY:

$A_S^* =$ 4737.60 mm² $f'_c =$ 35 MPa $f'_s =$ 1862.00 MPa b = 2000.00 mm d = 1325.00 mm
$\rho^* =$ 0.00179 $\gamma^* =$ 0.40 - for stress-relieved steel $\beta_1 =$ 0.80 - for f'c = 35.00 Mpa $\phi =$ 1.00

$f_{su}^* = f'_s \{1 - [(\gamma^* / \beta_1)(\rho^* f'_s / f'_c)]\}$
$f_{su}^* =$ 1773.45 MPa

Compression Block $= \dfrac{A_S^* f_{su}}{0.85 f'_C b}$ = 141.2 mm < t_{slab} = 200 mm -Consider rectangular section

$\phi M_n = \phi A_s^* f_{su}^* d \; [1 - (0.6 (\rho^* f_{su}^* / f'_c)]$
$\phi M_n =$ 10527.5 kN

LOAD RATING:

$\gamma_D =$ 1.30 $\gamma_L =$ 2.17 (Inventory Level) D = 2240.00 kN LL + I = 757.73 kN-m
$\gamma_L =$ 1.30 (Operating Level)

INVENTORY LEVEL:

$RF = \dfrac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)}$ = 4.63

OPERATING LEVEL:

$RF = \dfrac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)}$ = 7.73

EVALUATION FOR PRESTRESSED CONCRETE GIRDER BRIDGE USING LOAD FACTOR

FOR SPAN A1 TO P1 / A2 TO P2 - AT SUPPORT

TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type IV	1.38	0.405	0.885	0.944
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.694	0.828	1.153	0.876
Live Load MS-18	1.694	0.828	1.153	0.876

TABLE B: MOMENT DEMAND FOR PRESTRESSED CONCRETE GIRDER (D=1.829M; L=26.000M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	5599.00
Due to Weight of Girder + Slab + Diaphragm	7568.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	866.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact	1274.00
With Impact	1577.37
Load Combination at Service Condition	
DL + (LL+I)	10011.37

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 24 %

CALCULATION OF MOMENT CAPACITY AT SUPPORT
CONSIDERING PRESTRESSING STEEL ONLY:

$A_S^* =$ 4737.60 mm^2 $f'_c =$ 35 MPa $f'_s =$ 1862.00 MPa b = 2000.00 mm d = 1779.00 mm

$\rho^* =$ 0.00133 $\gamma^* =$ 0.40 - for stress-relieved steel $\beta_1 =$ 0.80 - for f'c = 35.00 Mpa $\phi =$ 1.00

$f_{su}^* = f'_s \ \{1 - [(\gamma^* / \beta_1)(\rho^* f'_s / f'_c)]\}$

$f_{su}^* =$ 1796.05 MPa

Compression Block = $\dfrac{A_S^* f_{su}}{0.85 f'_C b}$ = 143.0 mm < t_{slab} = 200 mm -Consider rectangular section

$\phi M_n = \phi A_s^* f_{su}^* d \ [1 - (0.6(\rho^* f_{su}^* / f'_c)]$

$\phi M_n =$ 14516.9 kN-m

LOAD RATING:

$\gamma_D =$ 1.30 $\gamma_L =$ 2.17 (Inventory Level) D = 8434.00 kN-m LL + I = 1577.37 kN-m

$\gamma_L =$ 1.30 (Operating Level)

<u>INVENTORY LEVEL:</u>

$RF = \dfrac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)}$ = 1.04

<u>OPERATING LEVEL:</u>

$RF = \dfrac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)}$ = 1.73

EVALUATION FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI

FOR SPAN P1 TO P2 - AT MIDSPAN

TABLE A: SECTION PROPERTIES

DESCRIPTION	Area (m^2)	Moment of Inertia (m^4)	Y Bottom of Girder (m)	Y Top of Girder (m)
Basic Section				
PSCG Type VI Modified	0.701	0.308	0.949	0.880
Deck Slab	0.400	N/A	N/A	N/A
Diaphragm	0.425	N/A	N/A	N/A
Composite Section				
Suprimposed Loads	1.058	0.551	1.290	0.770
Live Load MS-18	1.058	0.551	1.290	0.770

TABLE B: MOMENT DEMAND FOR SIMPLY SUPPORTED PRESTRESSED CONCRETE GIRDER TYPE VI (D=1.829M; L=46.100M)

DESCRIPTION	MIDSPAN
Dead Load Moment per Girder (kN-m)	
Basic Section	
Due to Weight of Girder	5311.00
Due to Weight of Girder + Slab + Diaphragm	7677.00
Composite Section	
Due to Weight of Superimposed Loads (railing, sidewalk, median and wearing surface)	1095.00
MS-18 Live Load Moment per Girder (kN-m)	
Without Impact	1982.00
With Impact	2378.40
Load Combination at Service Condition	
DL + (LL+I)	3473.40

Distribution Factor = S / 1.68 = 1.19
I = 100*(15.24 / L + 38) = 18 %

CALCULATION OF MOMENT CAPACITY AT MIDSPAN:
CONSIDERING PRESTRESSING STEEL ONLY:

$A_S^* =$ 5922.00 mm² $f'_c =$ 35 MPa $f'_s =$ 1862.00 MPa b = 2000.00 mm d = 1828.87 mm

$\rho^* =$ 0.00162 $\gamma^* =$ 0.40 - for stress-relieved steel $\beta_1 =$ 0.80 - for f'c = 35.00 Mpa $\phi =$ 1.00

$f_{su}^* = f'_s \ \{1 - [(\gamma^* / \beta_1)(\rho^* f'_s / f'_c)]\}$

$f_{su}^* =$ 1781.81 MPa

Compression Block $= \dfrac{A_S^* f_{su}}{0.85 f'_C b} =$ 177.3 mm $< t_{slab} =$ 200 mm -Consider rectangular section

$\phi M_n = \phi A_s^* f_{su}^* d \ [1 - (0.6 (\rho^* f_{su}^* / f'_c)]$

$\phi M_n =$ 18343.7 kN

LOAD RATING: D = 8772.00 kN LL + I = 2378.40 kN-m

$\gamma_D =$ 1.30 $\gamma_L =$ 2.17 (Inventory Level)

$\gamma_L =$ 1.30 (Operating Level)

INVENTORY LEVEL:

$RF = \dfrac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)} =$ 1.34

OPERATING LEVEL:

$RF = \dfrac{\phi M_n - \gamma_D D}{\gamma_L (LL+I)} =$ 2.24

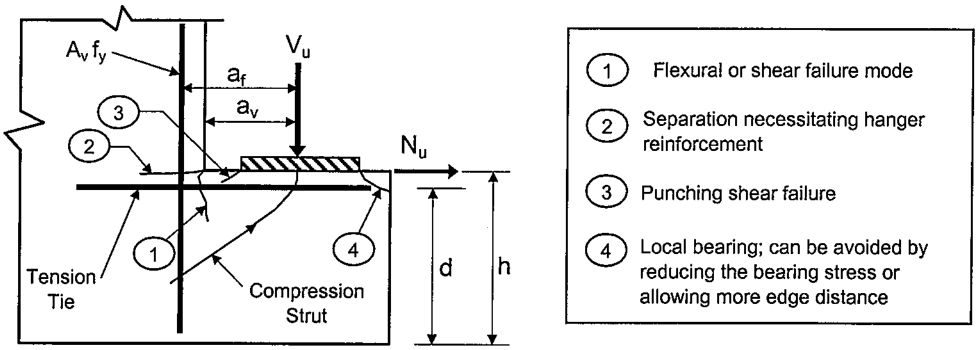
PROJECT TITLE: **PASIG-MARIKINA RIVER BRIDGE INSPECTION**
LAMBINGAN BRIDGE

ITEM: LIVE LOAD RATING

BEAM LEDGE CAPACITY INVESTIGATION FOR GERBER HINGE

1. BEAM LEADGE FAILURE MECHANISM

Beam ledges have to be designed for overall member actions and local failure modes as follows:

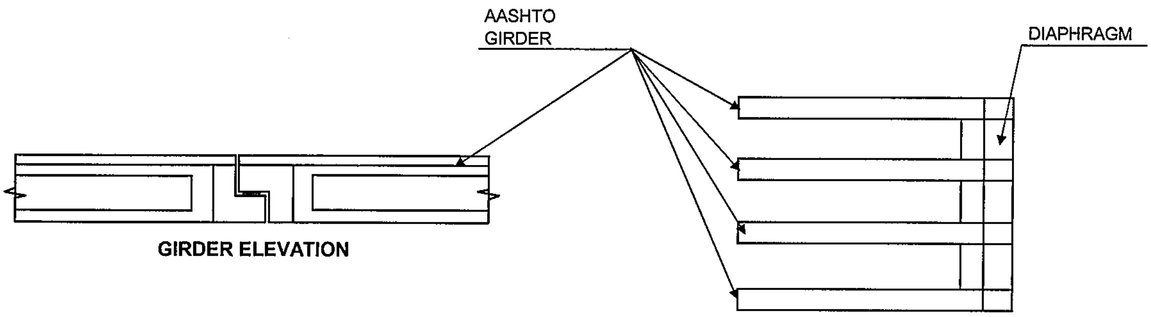


Failure Modes and Potential Cracks

Forces and actions acting on the ledge includes shear (Vu), horizontal tensile force (Nuc), and moment (Mu):

Vu = Factored Shear (Dead load + Live load + Impact)
Nuc ≥ 0.2Vu, but less than 1.0Vu
Mu = Vu (a_f) + Nu (h-d)
a_f = Flexural moment arm; distance from reaction centerline to centerline of hanger reinforcement
h - d = Moment arm for the horizontal load, Nuc

GERBER HINGE LAYOUT

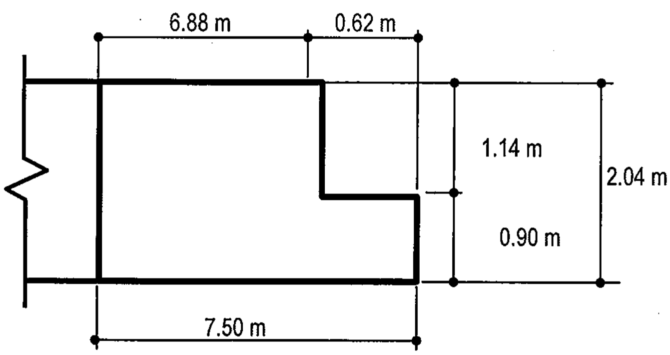


2. DIMENSION AND PROPERTIES OF LEDGE:

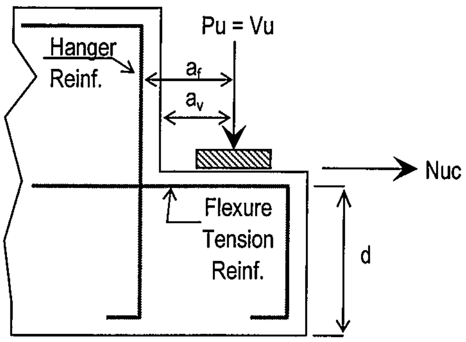
b = 7500 mm
D = 2040 mm
h = 900 mm

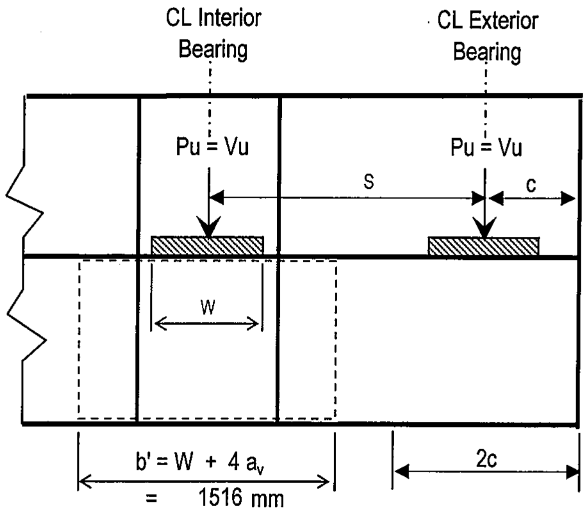
f'c = 35 Mpa
fys = 275 Mpa Reinforcing Steel
fyp = 1860 Mpa Prestressing Steel

concrete cover = 50 mm



CALCULATION OF LOAD RATING - GERBER HINGE



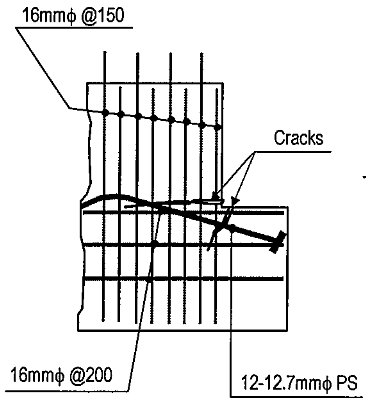


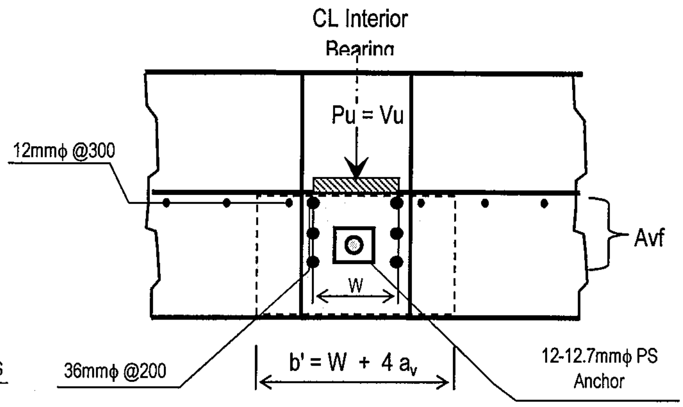
Bearing Pad Dimensions :

Width of bearing pad, W	=	500 mm	a_v	=	254	mm
Length of bearing pad, L	=	200 mm	a_f	=	304	mm
Width of Girder End Block	=	711 mm	d	=	850	mm
Girder Spacing, S	=	2000 mm (CL of bearing)	b	=	711	mm
Edge dist. of ext bearing,c	=	300 mm	b'	=	1516	mm

Bearing Edge Width:	
2c	= 600 mm

3. SHEAR FRICTION





Reinforcements Provided

Location	Bar Φ, mm	No. Pcs.	Area, mm²
Interior	20	22.00	6911.5
Exterior	20	22.00	6911.5

Note: 10-ϕ36 diagonal bars are included (7-ϕ36 eq.)

μ = friction coefficient = 1.40

b' = W+4a_b = 1516 mm

<u>Contribution of Prestressing Tendons:</u>

Prestress, As (12-d12.7) = 3553 mm^2

Equiv. Rebar, As' = 21629 mm^2

Effective Area of PS = 5197 mm^2 (2-tendons)

θ = 6.9 deg

<u>Ledge Capacity Under Shear Friction:</u>

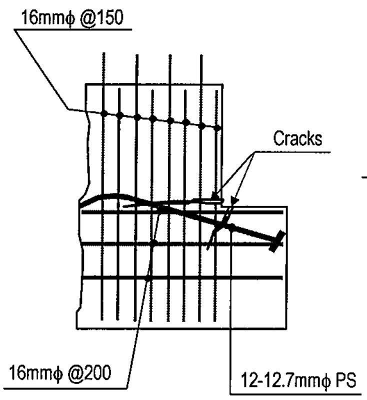
For Interior Bearing:

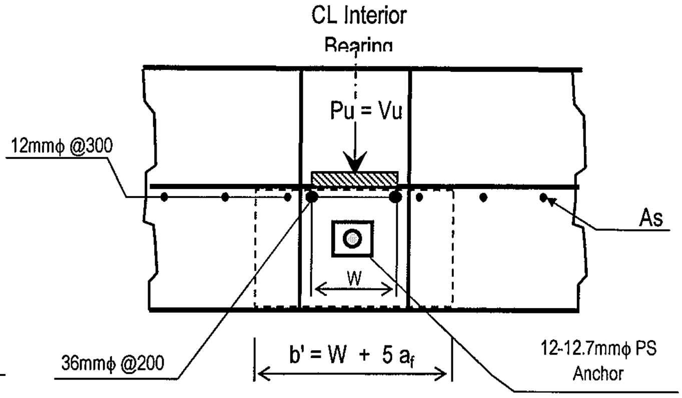
a_v / d = 0.30	=	1
Vu ≤ ϕ (0.2f'c) (W+4a_v) (d)	=	7667 kN
Vu ≤ $\phi\mu A_{vf}$ fy	=	2262 kN
With Prestress, Vu	=	3962 kN

For Exterior Bearing:

a_v / d = 0.30	=	1
Vu ≤ ϕ (0.2f'c) (K) (d)	=	3035 kN
Vu ≤ $\phi\mu A_{vf}$ fy	=	2262 kN
With Prestress, Vu	=	3962 kN
K = 2c	=	600 mm

4. FLEXURE





Reinforcements Provided

Location	Bar Φ, mm	No. Pcs.	Area, mm^2
Interior	20	10.00	3141.6
Exterior	20	10.00	3141.6

Note: 10-φ36 diagonal bars are included (5-φ36 eq.)

Prestressing :

Ap = 12-φ12.7mm = 3553.2 mm²
Equiv. Rebar, As' 24033 mm²
Effective Area of PS = 21260 mm²
θ = 10.6 deg

W + 5a_f = 2020 mm
2c = 600 mm

Ledge Capacity Under Flexure:

Reinforcing Bars Only

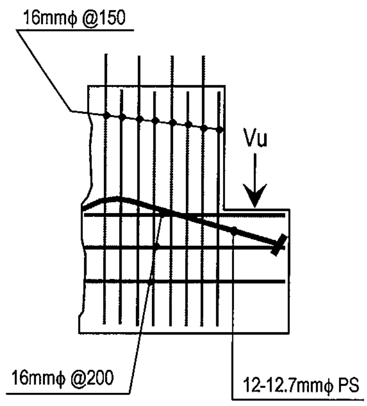
Strength:	Interiror	=	1590 kN
Vu ≥ φ Af fy jd /[af +0.2(h-d)]	Exterior	=	1590 kN

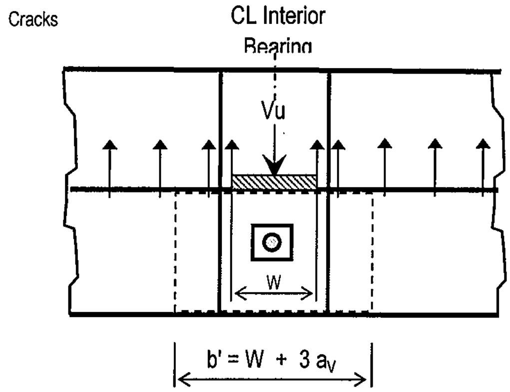
As ≥ 2(A_{vf})/3 + An = 4609 mm^2 NOT OK
As ≥ ρmin (W + 5af)(d) = 8741 mm^2 NOT OK
ρmin = 0.04(f'c/fy) = 0.0051

Reinforcing Bars Plus Prestressing Bars

Strength	Interiror	=	12352 kN
Vu ≥ φ Af fy jd /[af +0.2(h-d)]	Exterior	=	12352 kN

5. HANGER REINFORCEMENT





Reinforcements Provided

Location	Bar Φ, mm	No. Pcs.	Area, mm^2
Interior	20	20.00	6283.2
Exterior	20	20.00	6283.2

Note: 10-φ36 diagonal bars are included (25-φ16 eq.)

Contribution of Prestressing Tendons:

Prestress, As (12-d12.7) = 1184 mm^2
Equiv. Rebar, As' = 7210 mm^2
Effective Area of PS = 2652 mm^3
θ = 10.6 deg

Ledge Capacity Under Hanger Tension:

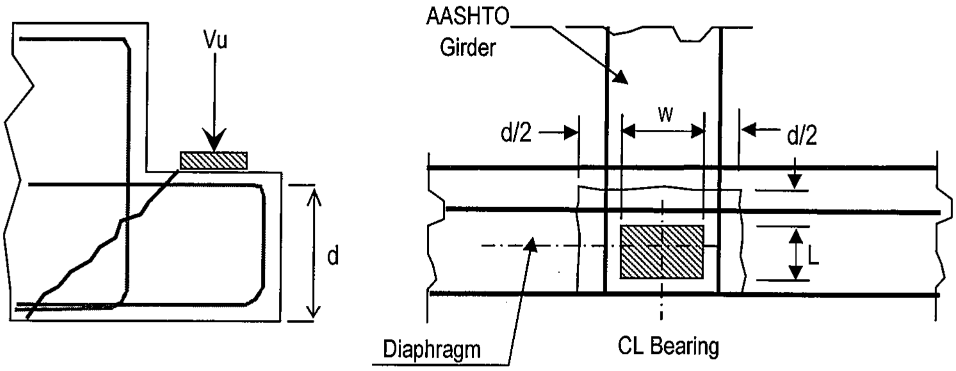
Reinforcing Bars Only

Strength	Vu = φ Av fy S / s	Interior	=	1469 kN
		Exterior	=	1469 kN
Serviceability		Interior	=	864 kN
	V = Av (0.5 fy) (W+3a) / s	Exterior	=	864 kN

Reinforcing Bars Plus Prestressing Bars

Strength	Vu = φ Av fy S / s	Interior	=	2089 kN
		Exterior	=	2089 kN
Serviceability		Interior	=	1229 kN
	V = Av (0.5 fy) (W+3a) / s	Exterior	=	1229 kN

6. PUNCHING SHEAR



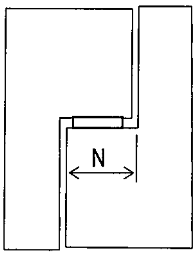
Allow. Tensile Strength for Puching = 0.33 $\sqrt{f'c}$ = 1.95 MPa

Ledge Capacity Under Punching Shear:

Interior Bearing: Vu $\leq \phi$ (0.33 $\sqrt{f'c}$)(W+2L'+2d)(d) = 3136 kN

Exterior Bearing: Vu $\leq \phi$ (0.33 $\sqrt{f'c}$)(W+L'+d)(d) = 1859 kN

7. AVAILABLE SEAT WIDTH



From AASHTO 7.3.1 DIVISION 1A

N = (305 + 2.5L + 10H) (1 + 0.000125S^2)

L = length in meters of the bridge deck to the adjacent expansion joint

S = angle of skew of support in degrees measured from a line normal to the span.

H = is the column or pier average height in meters

Seat width provided at gerber hinge = 800 mm

L = 37.5 m (Total length of deck from expansion joint to the of the expansion)

S = 0 degrees

H = 5.3 m (Average Height of Column at Main Bridge)

N = (305 + 2.5L + 10H) (1 + 0.000125S^2)

N = 451.75 mm **OK, Seat Width Sufficient**

8. SUMMARY OF CAPACITY

Demand / Reaction:

	EXT		INT	
Dead Load =	878.0	kN	856.0	kN
Live Load =	213.3	kN	259.2	kN

Load Factors (Service)			Load Factors		
γ_D Dead Load	=	1.00	γ_D Dead Load	=	1.30
γ_L Inventory	=	1.00	γ_L Inventory	=	2.17
γ_L Operating	=	1.00	γ_L Operating	=	1.30

Calculated Capacity (Load Factor)

Girder Location	Shear Friction		Flexure		Hanger		Punching Shear
	Rebar	W/ PS	Rebar	W/ PS	Rebar	W/ PS	
Interior	2262	3962	1590	12352	1469	2089	3136
Exterior	2262	3962	1590	12352	1469	2089	1859

Load Rating:

By Serviceability State Limit Method:

Girder Location	Considering Reinforcing Bars Only				Reinforcing Bars Plus Prestress			
	Inventory				Inventory			
	RF	LL_{EQUIV} (HS20)			RF	LL_{EQUIV} (HS20)		
Interior	**2.36**	**77.3 tons**			**4.76**	**155.5 tons**		
Exterior	**2.77**	**90.6 tons**			**1.55**	**50.7 tons**		

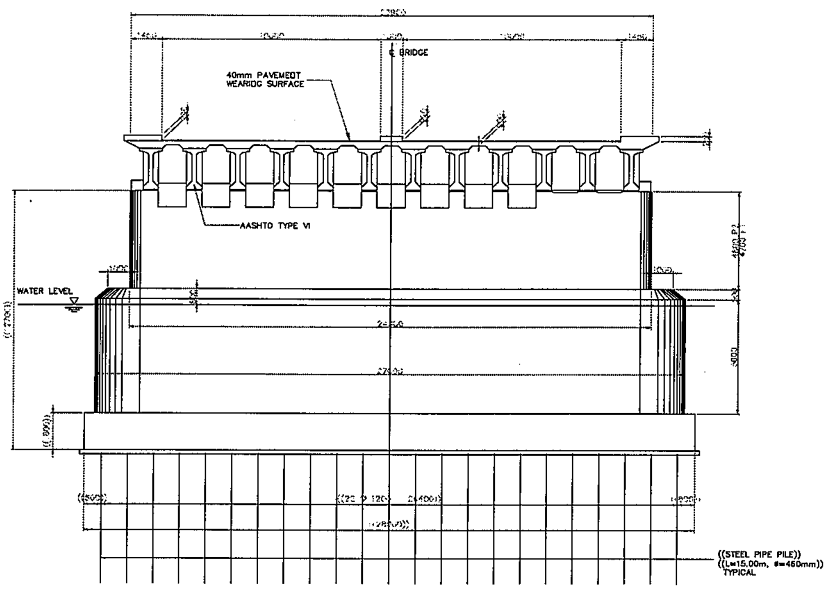
Load Rating:

By Load Factor Method:

Girder Location	Considering Reinforcing Bars Only				Reinforcing Bars Plus Prestress			
	Inventory		Operating		Inventory		Operating	
	RF	LL_{EQUIV} (HS20)	RF	LL_{EQUIV} (HS20)	RF	LL_{EQUIV} (HS20)	RF	LL_{EQUIV} (HS20)
Interior	**0.63**	**20.7 tons**	**1.06**	**34.5 tons**	**1.74**	**56.7 tons**	**2.90**	**94.7 tons**
Exterior	**0.71**	**23.1 tons**	**1.18**	**38.6 tons**	**1.55**	**50.7 tons**	**2.59**	**84.6 tons**

CALCULATION OF CAPACITY-DEMAND RATIO OF PIER WALL

CAPACITY/DEMAND RATIO FOR LAMBINGAN BRIDGE



Seismic Design Condition :

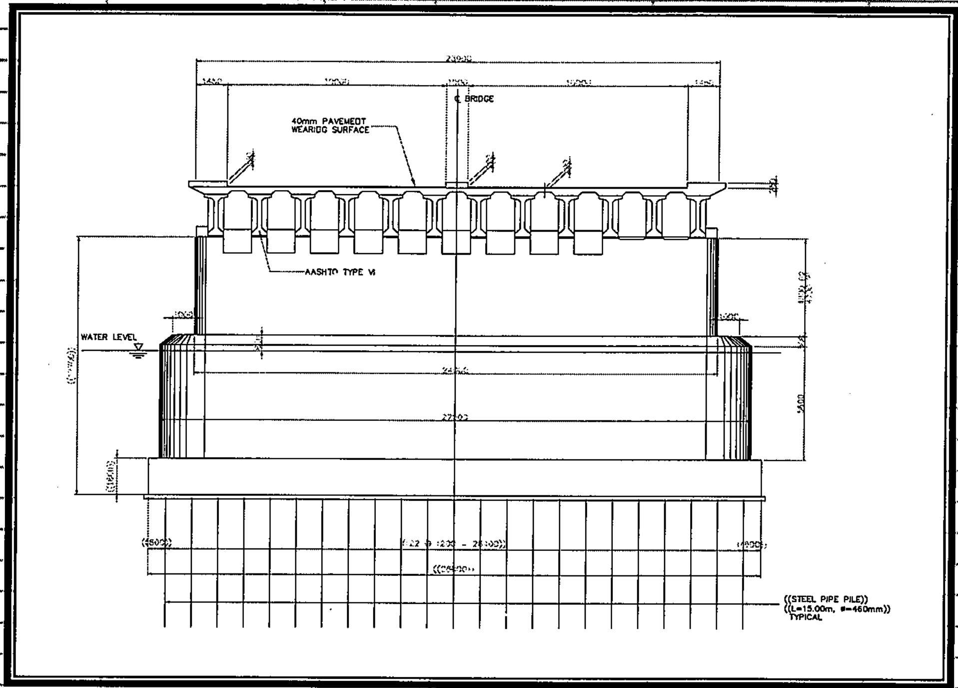
Seismic Performance Category, SPC	=	D
Importance Classification, IC (Essential Bridge)	=	I
Site Coefficient (Soil Profile Type II)	=	1.20

	Particular		Longitudinal Direction	Transverse Direction	Unit
A = 0.4g	Elastic Design Forces	P_{max}	25,200.67	25,200.67	kN
		P_{min}	25,200.67	25,200.67	kN
		M_{DESIGN}	47,421.06	38,087.68	kN•m
		V_{DESIGN}	9,838.39	7,902.01	kN
	Plastic Forces	$M_{PLASTIC}$	39,499.85	720,177.77	kN•m
		$V_{PLASTIC}$	8,194.99	149,414.48	kN
	Number of Rebars		166 - ϕ20mm		
	Rebar Ratio		0.17%		
	Flexural Capacity	$M_{CAPACITY}$	30,384.50	553,982.90	kN•m
	C/D Ratio		0.64	14.54	

CALACULATION OF CAPACITY-DEMAND RATIO OF PIER WALL

CAPACITY/DEMAND RATIO FOR LAMBINGAN BRIDGE

(PEDESTAL)



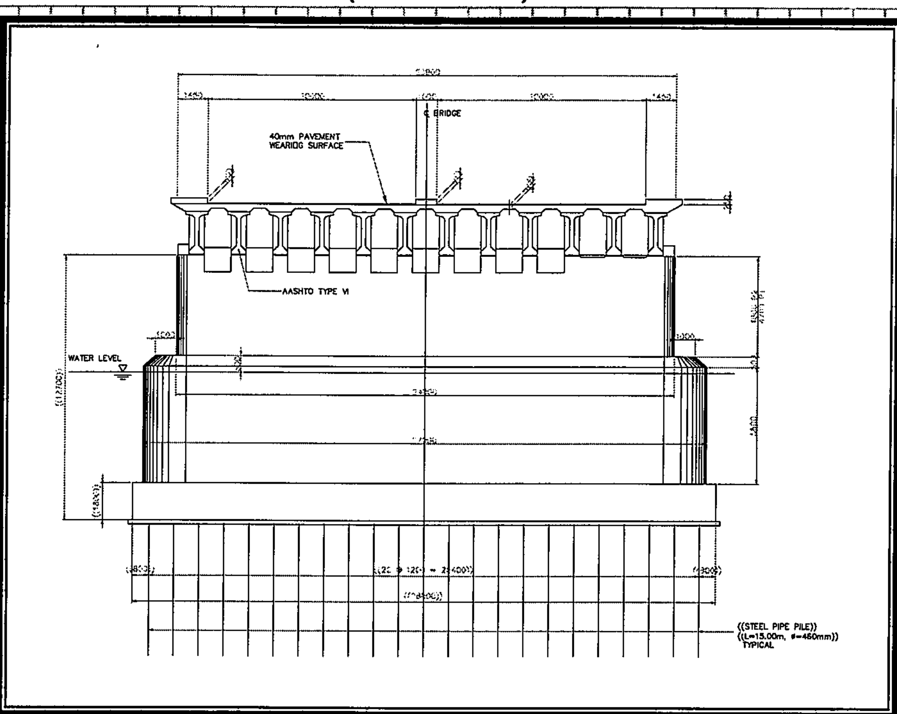
Seismic Design Condition :

Seismic Performance Category, SPC	=	D
Importance Classification, IC (Essential Bridge)	=	I
Site Coefficient (Soil Profile Type II)	=	1.20

	Particular		Longitudinal Direction	Transverse Direction	Unit
Pedestal @ 0.4g	Elastic Design Forces	P_{max}	44,974.38	44,974.37	kN
		P_{min}	44,974.36	44,974.37	kN
		M_{DESIGN}	113,767.80	98,901.60	kN•m
		V_{DESIGN}	23,603.28	20,519.00	kN
	Plastic Forces	$M_{PLASTIC}$	224,907.02	1,125,034.95	kN•m
		$V_{PLASTIC}$	46,661.21	233,409.74	kN
	Number of Rebars (Assumed)		198 - ϕ20mm		
	Rebar Ratio		0.05%		
	Flexural Capacity	$M_{CAPACITY}$	173,005.40	865,411.50	kN•m
	C/D Ratio		1.52	8.75	

	Particular		Longitudinal Direction	Transverse Direction	Unit
Pedestal @ 0.22g	Elastic Design Forces	P_{max}	44,974.38	44,974.37	kN
		P_{min}	44,974.36	44,974.37	kN
		M_{DESIGN}	62,572.31	54,395.92	kN•m
		V_{DESIGN}	12,981.81	11,285.46	kN
	Plastic Forces	$M_{PLASTIC}$	224,907.02	1,125,034.95	kN•m
		$V_{PLASTIC}$	46,661.21	233,409.74	kN
	Number of Rebars (Assumed)		198 - ϕ20mm		
	Rebar Ratio		0.05%		
	Flexural Capacity	$M_{CAPACITY}$	173,005.40	865,411.50	kN•m
	C/D Ratio		2.76	15.91	

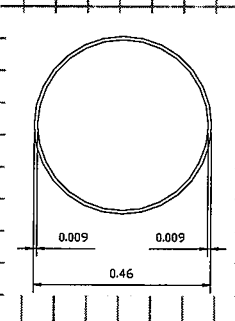
CAPACITY/DEMAND RATIO FOR LAMBINGAN BRIDGE
(STEEL PILE)



Type of Pile	=	Steel Pipe Pile
Acceleration Coefficient	=	0.22g

Dimensions:

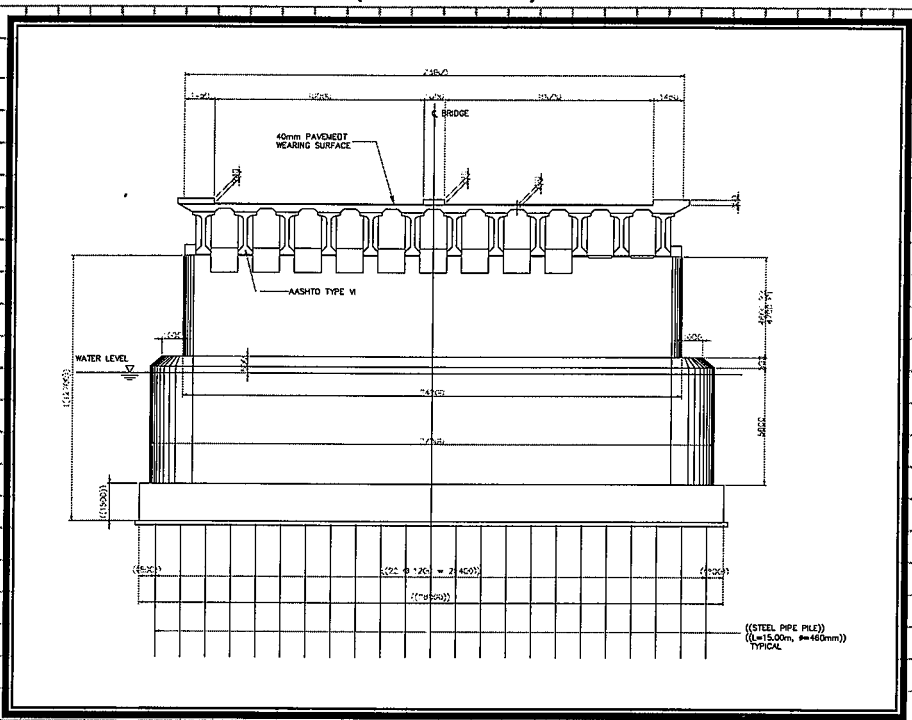
Assumed thickness = 9mm



Item	Longitudinal	Transverse	Unit
Area, A	0.0128	0.0128	m^2
Moment of Inertia, I	0.0014	0.0014	m^4
Section Modulus, S	0.0061	0.0061	m^3

Item	Longitudinal	Transverse	Unit
Moment Demand	211.00	125.00	kNm
Moment Capacity	905.74	905.74	kNm
C/D Ratio	4.29	7.25	

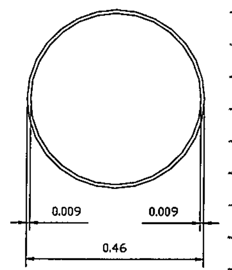
CAPACITY/DEMAND RATIO FOR LAMBINGAN BRIDGE
(STEEL PILE)



Type of Pile	=	Steel Pipe Pile
Acceleration Coefficient	=	0.4g

Dimensions:

Assumed thickness = 9mm



Item	Longitudinal	Transverse	Unit
Area, A	0.0128	0.0128	m^2
Moment of Inertia, I	0.0014	0.0014	m^4
Section Modulus, S	0.0061	0.0061	m^3

Item	Longitudinal	Transverse	Unit
Moment Demand	383.00	228.00	kNm
Moment Capacity	905.74	905.74	kNm
C/D Ratio	2.36	3.97	

ANALYSIS OF WALL-PIER

SEISMIC DESIGN CRITERIA

Acceleration Coefficient, A = 0.20 | Importance Classification, IC **(Essential Brdg)** = I (I or II)

Seismic Performance Category, SPC = C | Site Coefficient, : **(Soil Profile Type II)** = 1.20

ELASTIC SEISMIC FORCES

From STAAD-III Multi-Modal Dynamic Analysis

	LONGITUDINAL		TRANSVERSE		
	SHEAR kN	MOMENT kN-m	SHEAR kN	MOMENT kN-m	AXIAL kN
DEAD LOAD	0.00	0.02	0.00	0.00	25,200.67
LONG EQ	12,421.51	69,597.48	0.00	0.00	0.00
TRAN EQ	0.00	0.00	5,886.97	31,854.79	0.00

LOAD COMBINATION

Load Case 1 = 1.0 LONG EQ + 0.3 TRAN EQ

Load Case 2 = 0.3 LONG EQ + 1.0 TRAN EQ

	LONGITUDINAL		TRANSVERSE		
	SHEAR kN	MOMENT kN-m	SHEAR kN	MOMENT kN-m	AXIAL kN
Load Case 1	12,421.51	69,597.48	1,766.09	9,556.44	0.00
Load Case 2	3,726.45	20,879.24	5,886.97	31,854.79	0.00

GROUP LOADING OF DESIGN FORCES

Group Load = 1.0 (D + B + SF + E + EQ)

	LONGITUDINAL		TRANSVERSE		AXIAL	
	SHEAR kN	MOMENT kN-m	SHEAR kN	MOMENT kN-m	max kN	min kN
Load Case 1	12,421.51	69,597.50	1,766.09	9,556.44	25,200.67	25,200.67
Load Case 2	3,726.45	20,879.26	5,886.97	31,854.79	25,200.67	25,200.67

MODIFIED DESIGN FORCES

Group Load = 1.0 (D + EQ / R)

R = 2 (for wall-pier)*** — *** Check H / Dmax < 2.5

R = 3 (for single column)

R applied only to elastic seismic moments	LONGITUDINAL		TRANSVERSE		AXIAL	
	SHEAR kN	MOMENT kN-m	SHEAR kN	MOMENT kN-m	max kN	min kN
Weak Direction	12,421.51	23,199.18	1,766.09	3,185.48	25,200.67	25,200.67
Load Case 1	12,421.51	34,798.76	1,766.09	4,778.22	25,200.67	25,200.67
Load Case 2	3,726.45	10,439.64	5,886.97	15,927.40	25,200.67	25,200.67

PIER ELASTIC DESIGN FORCES

	STRONG DIR	WEAK DIR	
M DESIGN =	19,043.85	35,125.28	kN-m
V DESIGN =	6,967.27	12,546.43	kN
Pmax DES =	25,200.67	25,200.67	kN
Pmin DES =	25,200.67	25,200.67	kN

COLUMN ELASTIC DESIGN FORCES

	ABOUT THE WEAK AXIS	
M DESIGN =	23,416.86	kN-m
V DESIGN =	12,546.43	kN
Pmax DES =	25,200.67	kN
Pmin DES =	25,200.67	kN

DESIGN OF WALL-PIER MAIN REINFORCEMENT BARS

(AS PIER ABOUT THE STRONG AXIS)

MATERIAL SPECIFICATIONS

A) Concrete

Compressive Strength of Concrete, f'c	=	28.00 MPa
Modulus of Elasticity of Concrete, Ec = 4730 sqrt(f'c)	=	25,028.81 MPa
Concrete Cover, cc	=	50 mm

B) Reinforcing Steel

Tensile Strength of Steel, fy	=	303.00 MPa
Main Bar Diameter, db	=	20 mm
Lateral Tie Diameter, ds ≤ 20 mm	=	20 mm

CAPACITY-DEMAND RATIO OF PIER WALL

WALL-PIER PROPERTIES

Base, B	=	1.30 m
Depth, D	=	24.00 m
Unsupported Length, Lu	=	5.82 m
Clear Height, H	=	4.82 m
Gross Area, Ag = BD	=	31.20 m^2
Core Area, Ac = BcDc	=	28.68 m^2
Moment of Inertia, Ig = $BD^3/12$	=	1497.60 m^4
Radius of Gyration, r = sqrt(Ig/Ag)	=	6.93 m

End Condition		EFFECTIVE LENGTH FACTOR, K					
Top	Rotation	FIXED	FREE	FIXED	FREE	FREE	FIXED
Top	Translation	FIXED	FIXED	FREE	FIXED	FREE	FREE
Bottom	Rotation	FIXED	FIXED	FIXED	FREE	FIXED	FREE
Bottom	Translation	FIXED	FIXED	FIXED	FIXED	FIXED	FIXED
Theoretical K		0.50	0.70	1.00	1.00	2.00	2.00
Design Value, K		0.65	0.80	1.20	1.00	2.10	2.00

Effective Length Factor, k = 2.10

SLENDERNESS EFFECT

22.00 < k Lu / r < 100.00 neglect slenderness effect 22.00 > 1.76 < 100.00

Lu / r < 35 / sqrt[Pu / (f'cAg)] 0.84 < 206.07

MOMENT MAGNIFICATION

Maximum Dead Load Moment, Mdl	=	0.02 kN-m
Maximum Total Load Moment, Mmax	=	19,043.85 kN-m
Ratio βd = Mdl / Mmax	=	0.00
Flexural Stiffnes of Column, EI = (EcIg/2.5)/(1+βd)	=	1.50E+07 MN-m2
Factored Axial Load, Pu = Pmax	=	25,200.67 kN
Buckling Load, Pc = π^2 EI / $(kLu)^2$	=	9.91E+08 kN
Spiral as Lateral Reinforcement, ϕ	=	0.70
Moment Magnification Factor not braced against sidesway, $\delta s = 1 / [1 - (\Sigma Pu / \phi\ \Sigma Pc)]$	=	1.00 $\geq$ 1.00
Magnified Design Moment, Mc = δs Mmax	=	19,043.85 kN-m

MODIFIED STRENGTH REDUCTION FACTOR ϕ

Maximum Axial Stress, σPmax = Pmax / Ag	=	807.71 kPa
10% of Compressive Strength of Concrete, 0.1f'c	=	2,800.00 kPa
Approximate Balanced Axial Load, ϕPb = 0.1 f'c Ag	=	87,360.00 kN
Modified Strength Reduction Factor, $\phi = 0.90 - 0.20\ [\sigma Pmax / (0.10f'c)] \geq 0.70$	=	0.842

MAGNIFIED ELASTIC DESIGN FORCES

M DESIGN =	19043.85	kN-m
Pmax DES =	25200.67	kN
Pmin DES =	25200.67	kN

****** DESIGN COLUMN USING PCACOL PROGRAM ...***

****** NOTE 0.01 < As / Ag < 0.06***

DESIGN OF WALL-PIER MAIN REINFORCEMENT BARS

(AS COLUMN ABOUT THE WEAK AXIS)

MATERIAL SPECIFICATIONS

A) Concrete

Compressive Strength of Concrete, f'c	=	28.00 MPa
Modulus of Elasticity of Concrete, Ec = 4730 sqrt(f'c)	=	25,028.81 MPa
Concrete Cover, cc	=	50 mm

B) Reinforcing Steel

Tensile Strength of Steel, fy	=	303.00 MPa
Modulus of Elasticity of Steel, Es	=	200,000.00 MPa
Main Bar Diameter, db	=	20 mm

WALL-PIER PROPERTIES

Base, B	=	24.00 m
Depth, D	=	1.30 m
Unsupported Length, Lu	=	5.82 m
Clear Height, H	=	4.82 m
Gross Area, Ag = BD	=	31.20 m^2
Core Area, Ac = BcDc	=	28.68 m^2
Moment of Inertia, Ig = $BD^3/12$	=	4.39 m^4
Radius of Gyration, r = sqrt(Ig/Ag)	=	0.38 m

End Condition		EFFECTIVE LENGTH FACTOR, K					
Top	Rotation	FIXED	FREE	FIXED	FREE	FREE	FIXED
Top	Translation	FIXED	FIXED	FREE	FIXED	FREE	FREE
Bottom	Rotation	FIXED	FIXED	FIXED	FREE	FIXED	FREE
Bottom	Translation	FIXED	FIXED	FIXED	FIXED	FIXED	FIXED
Theoretical K		0.50	0.70	1.00	1.00	2.00	2.00
Design Value, K		0.65	0.80	1.20	1.00	2.10	2.00

Effective Length Factor, k = 2.10

SLENDERNESS EFFECT

22.00 < k Lu / r < 100.00 22.00 < 32.57 < 100.00

Lu / r < 35 / sqrt[Pu / (f'cAg)] 15.51 < 206.07

Appendix 22.1.4-4 (7/12)
CAPACITY-DEMAND RATIO OF PIER WALL

MOMENT MAGNIFICATION

Maximum Dead Load Moment, Mdl	=	0.02 kN-m
Maximum Total Load Moment, Mmax	=	23,416.86 kN-m
Ratio βd = Mdl / Mmax	=	0.00
Flexural Stiffnes of Column, EI = (EcIg/2.5)/(1+βd)	=	4.40E+04 MN-m2
Factored Axial Load, Pu = Pmax	=	25,200.67 kN
Buckling Load, Pc = π^2 EI / (kLu)^2	=	2.91E+06 kN
Spiral as Lateral Reinforcement, ϕ	=	0.70
Moment Magnification Factor not braced against sidesway, δs = 1 / [1 - (ΣPu / ϕ ΣPc)]	=	1.01 $\geq$ 1.00
Magnified Design Moment, Mc = δs Mmax	=	23,710.54 kN-m

MODIFIED STRENGTH REDUCTION FACTOR ϕ

Maximum Axial Stress, σPmax = Pmax / Ac	=	878.68 kPa
20% of Compressive Strength of Concrete, 0.2f'c	=	5,600.00 kPa
Approximate Balanced Axial Load, ϕPb = 0.2 f'c Ac	=	160,608.00 kN
Modified Strength Reduction Factor, ϕ = 0.90 - 0.40 [σPmax / (0.20f'c)] $\geq$ 0.50	=	0.837

MAGNIFIED ELASTIC DESIGN FORCES

M DESIGN =	23710.54	kN-m
Pmax DES =	25200.67	kN
Pmin DES =	25200.67	kN

****** DESIGN COLUMN USING PCACOL PROGRAM ...***

****** NOTE 0.01 < As / Ag < 0.06***

ULTIMATE (Nominal) DESIGN FORCES FOR COLUMN

M ULTIMATE =	28319.99	kN-m
Pmax ULT =	30099.81	kN
Pmin ULT =	30099.81	kN

****** INVESTIGATE COLUMN PLASTIC CAPACITY FROM PCACOL INTERACTION DIAGRAM...***

FORCES RESULTING FROM PLASTIC HINGING

Pmax DES =	25,200.67	kN		M Nominal Cap =	28,431.80	kN-m
M Plastic =	36,961.34	kN-m	<	M Elastic =	69,597.50	kN-m
V Plastic =	7,668.33	kN				
Pmax Plastic =	25,200.67	kN				
Pmin Plastic =	25,200.67	kN				

COLUMN CONNECTIONS REQUIREMENTS

Anchorage for Uplift Forces, Ptens = 1.25 As fy			=	118.99 KN
Development Length, L_d $L_a = 0.04\ A_b f_y / f_c'^{\ 1/2}$ =	13.431945		=	341.26 mm
$L_a = 0.0004\ d_b f_y$ =	13.840909	**GOVERNS**	=	351.65 mm
Shear Stress @ Joint, vu = Vu / ϕAg $\leq$ sqrt(f'c) =	5,291.50 kPa	**ok**	=	289.15 kPa

Appendix 22.1.4-4 (8/12)
CAPACITY-DEMAND RATIO OF PIER WALL

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02/21/04 PCACOL V3.00 - PORTLAND CEMENT ASSOCIATION -                        Page   1
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        ========================================================================
        Computer program for the Strength Design of Reinforced Concrete Sections
        ========================================================================
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02/21/04 PCACOL V3.00 - PORTLAND CEMENT ASSOCIATION -                        Page   2
16:08:20 Licensed to: KEI, Pasig City, PI                                        L-2G
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General Information:
====================

File Name: C:\MSNAVAL\PASIG--3\FROMKE~1\FINAL--1\APPEND~2\LAMBIN~1\L-2G.COL
Project: Lambingan Bridge
Column: Column — Engineer: AHR
Code: ACI 318-95 — Units: Metric

Run Option: Investigation — Slenderness: Not considered
Run Axis: Biaxial — Column Type: Structural

Material Properties:
====================

f'c = 28 MPa — fy = 415 MPa
Ec = 24870.1 MPa — Es = 200000 MPa
fc = 23.8 MPa — Rupture strain = Infinity
Ultimate strain = 0.003 mm/mm
Beta1 = 0.846954

Section:
========

Rectangular: Width = 23561 mm — Depth = 1300 mm

Gross section area, Ag = 3.06293e+007 mm^2
Ix = 4.31363e+012 mm^4 — Iy = 1.41691e+015 mm^4
Xo = 0 mm — Yo = 0 mm

Reinforcement:
==============

Rebar Database: User-defined

Size	Diam (mm)	Area (mm^2)	Size	Diam (mm)	Area (mm^2)	Size	Diam (mm)	Area (mm^2)
# 6	6	28	# 8	8	50	# 10	10	79
# 12	12	113	# 14	14	154	# 16	16	201
# 20	20	314	# 25	25	491	# 28	28	616
# 32	32	801	# 36	36	1018	# 40	40	1256

Confinement: User-defined; #20 ties with #20 bars, #20 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.837

Layout: Rectangular
Pattern: Equal Bar Spacing (Cover to transverse reinforcement)
Total steel area, As = 52124 mm^2 at 0.17%
166 #20 Cover = 50 mm

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
===

No.	Pu kN	Mux kN-m	Muy kN-m	fMnx kN-m	fMny kN-m	fMn/Mu
1	25200.7	0.0	19043.8	0.0	488390.4	25.646
2	25200.7	23710.5	0.0	26807.6	2.7	1.131

*** Program completed as requested! ***

ANALYSIS OF WALL-PIER

SEISMIC DESIGN CRITERIA

Acceleration Coefficient, A = 0.40 Importance Classification, IC **(Essential Brdg)** = I (I or II)
Seismic Performance Category, SPC = D Site Coefficient, : **(Soil Profile Type II)** = 1.20

ELASTIC SEISMIC FORCES

From STAAD-III Multi-Modal Dynamic Analysis

	LONGITUDINAL		TRANSVERSE		
	SHEAR kN	MOMENT kN-m	SHEAR kN	MOMENT kN-m	AXIAL kN
DEAD LOAD	0.00	0.02	0.00	0.00	25,200.67
LONG EQ	24,843.03	139,194.95	0.00	0.00	0.01
TRAN EQ	0.01	0.01	11,773.94	63,709.58	0.00

LOAD COMBINATION

Load Case 1 = 1.0 LONG EQ + 0.3 TRAN EQ
Load Case 2 = 0.3 LONG EQ + 1.0 TRAN EQ

	LONGITUDINAL		TRANSVERSE		
	SHEAR kN	MOMENT kN-m	SHEAR kN	MOMENT kN-m	AXIAL kN
Load Case 1	24,843.03	139,194.95	3,532.18	19,112.87	0.01
Load Case 2	7,452.92	41,758.50	11,773.94	63,709.58	0.00

GROUP LOADING OF DESIGN FORCES

Group Load = 1.0 (D + B + SF + E + EQ)

	LONGITUDINAL		TRANSVERSE		AXIAL	
	SHEAR kN	MOMENT kN-m	SHEAR kN	MOMENT kN-m	max kN	min kN
Load Case 1	24,843.03	139,194.97	3,532.18	19,112.87	25,200.68	25,200.66
Load Case 2	7,452.92	41,758.52	11,773.94	63,709.58	25,200.67	25,200.67

MODIFIED DESIGN FORCES

Group Load = 1.0 (D + EQ / R)
R = 2 (for wall-pier)*** *** Check H / Dmax < 2.5
R = 3 (for single column)

R applied only to elastic seismic moments	LONGITUDINAL		TRANSVERSE		AXIAL	
	SHEAR kN	MOMENT kN-m	SHEAR kN	MOMENT kN-m	max kN	min kN
Weak Direction	24,843.03	46,398.34	3,532.18	6,370.96	25,200.68	25,200.66
Load Case 1	24,843.03	69,597.50	3,532.18	9,556.44	25,200.68	25,200.66
Load Case 2	7,452.92	20,879.27	11,773.94	31,854.79	25,200.67	25,200.67

PIER ELASTIC DESIGN FORCES

	STRONG DIR	WEAK DIR	
M DESIGN =	38,087.68	70,250.53	kN-m
V DESIGN =	13,934.55	25,092.88	kN
Pmax DES =	25,200.67	25,200.68	kN
Pmin DES =	25,200.67	25,200.66	kN

COLUMN ELASTIC DESIGN FORCES

	ABOUT THE WEAK AXIS	
M DESIGN =	46,833.69	kN-m
V DESIGN =	25,092.88	kN
Pmax DES =	25,200.68	kN
Pmin DES =	25,200.66	kN

DESIGN OF WALL-PIER MAIN REINFORCEMENT BARS
(AS PIER ABOUT THE STRONG AXIS)

MATERIAL SPECIFICATIONS

A) Concrete
- Compressive Strength of Concrete, f'c = 28.00 MPa
- Modulus of Elasticity of Concrete, Ec = 4730 sqrt(f'c) = 25,028.81 MPa
- Concrete Cover, cc = 50 mm

B) Reinforcing Steel
- Tensile Strength of Steel, fy = 303.00 MPa
- Main Bar Diameter, db = 20 mm
- Lateral Tie Diameter, ds ≤ 20 mm = 20 mm

CAPACITY-DEMAND RATIO OF PIER WALL

WALL-PIER PROPERTIES

Base, B	=	1.30 m
Depth, D	=	24.00 m
Unsupported Length, Lu	=	5.82 m
Clear Height, H	=	4.82 m
Gross Area, Ag = BD	=	31.20 m^2
Core Area, Ac = BcDc	=	28.68 m^2
Moment of Inertia, Ig = $BD^3/12$	=	1497.60 m^4
Radius of Gyration, r = sqrt(Ig/Ag)	=	6.93 m

End Condition		EFFECTIVE LENGTH FACTOR, K					
Top	Rotation	FIXED	FREE	FIXED	FREE	FREE	FIXED
Top	Translation	FIXED	FIXED	FREE	FIXED	FREE	FREE
Bottom	Rotation	FIXED	FIXED	FIXED	FREE	FIXED	FREE
Bottom	Translation	FIXED	FIXED	FIXED	FIXED	FIXED	FIXED
Theoretical K		0.50	0.70	1.00	1.00	2.00	2.00
Design Value, K		0.65	0.80	1.20	1.00	2.10	2.00

Effective Length Factor, k = 2.10

SLENDERNESS EFFECT

22.00 < k Lu / r < 100.00	neglect slenderness effect	22.00 > 1.76 < 100.00
Lu / r < 35 / sqrt[Pu / (f'cAg)]		0.84 < 206.07

MOMENT MAGNIFICATION

Maximum Dead Load Moment, Mdl	=	0.02 kN-m
Maximum Total Load Moment, Mmax	=	38,087.68 kN-m
Ratio βd = Mdl / Mmax	=	0.00
Flexural Stiffnes of Column, EI = (EcIg/2.5)/(1+βd)	=	1.50E+07 MN-m2
Factored Axial Load, Pu = Pmax	=	25,200.67 kN
Buckling Load, Pc = π^2 EI / $(kLu)^2$	=	9.91E+08 kN
Spiral as Lateral Reinforcement, ϕ	=	0.70
Moment Magnification Factor not braced against sidesway, $\delta s = 1 / [1 - (\Sigma Pu / \phi \Sigma Pc)]$	=	1.00 $\geq$ 1.00
Magnified Design Moment, Mc = δs Mmax	=	38,087.68 kN-m

MODIFIED STRENGTH REDUCTION FACTOR ϕ

Maximum Axial Stress, σPmax = Pmax / Ag	=	807.71 kPa
10% of Compressive Strength of Concrete, 0.1f'c	=	2,800.00 kPa
Approximate Balanced Axial Load, ϕPb = 0.1 f'c Ag	=	87,360.00 kN
Modified Strength Reduction Factor, $\phi = 0.90 - 0.20\ [\sigma Pmax / (0.10f'c)] \geq 0.70$	=	0.842

MAGNIFIED ELASTIC DESIGN FORCES

M DESIGN =	38087.68	kN-m
Pmax DES =	25200.67	kN
Pmin DES =	25200.67	kN

****** DESIGN COLUMN USING PCACOL PROGRAM ...***

****** NOTE 0.01 < As / Ag < 0.06***

DESIGN OF WALL-PIER MAIN REINFORCEMENT BARS

(AS COLUMN ABOUT THE WEAK AXIS)

MATERIAL SPECIFICATIONS

A) Concrete

Compressive Strength of Concrete, f'c	=	28.00 MPa
Modulus of Elasticity of Concrete, Ec = 4730 sqrt(f'c)	=	25,028.81 MPa
Concrete Cover, cc	=	50 mm

B) Reinforcing Steel

Tensile Strength of Steel, fy	=	303.00 MPa
Modulus of Elasticity of Steel, Es	=	200,000.00 MPa
Main Bar Diameter, db	=	20 mm

WALL-PIER PROPERTIES

Base, B	=	24.00 m
Depth, D	=	1.30 m
Unsupported Length, Lu	=	5.82 m
Clear Height, H	=	4.82 m
Gross Area, Ag = BD	=	31.20 m^2
Core Area, Ac = BcDc	=	28.68 m^2
Moment of Inertia, Ig = $BD^3/12$	=	4.39 m^4
Radius of Gyration, r = sqrt(Ig/Ag)	=	0.38 m

End Condition		EFFECTIVE LENGTH FACTOR, K					
Top	Rotation	FIXED	FREE	FIXED	FREE	FREE	FIXED
Top	Translation	FIXED	FIXED	FREE	FIXED	FREE	FREE
Bottom	Rotation	FIXED	FIXED	FIXED	FREE	FIXED	FREE
Bottom	Translation	FIXED	FIXED	FIXED	FIXED	FIXED	FIXED
Theoretical K		0.50	0.70	1.00	1.00	2.00	2.00
Design Value, K		0.65	0.80	1.20	1.00	2.10	2.00

Effective Length Factor, k = 2.10

SLENDERNESS EFFECT

22.00 < k Lu / r < 100.00	22.00 < 32.57 < 100.00
Lu / r < 35 / sqrt[Pu / (f'cAg)]	15.51 < 206.07

CAPACITY-DEMAND RATIO OF PIER WALL

MOMENT MAGNIFICATION

Maximum Dead Load Moment, Mdl	=	0.02 kN-m
Maximum Total Load Moment, Mmax	=	46,833.69 kN-m
Ratio βd = Mdl / Mmax	=	0.00
Flexural Stiffnes of Column, EI = (EcIg/2.5)/(1+βd)	=	4.40E+04 MN-m2
Factored Axial Load, Pu = Pmax	=	25,200.68 kN
Buckling Load, Pc = π^2 EI / (kLu)^2	=	2.91E+06 kN
Spiral as Lateral Reinforcement, ϕ	=	0.70
Moment Magnification Factor not braced against sidesway, $\delta s = 1 / [1 - (\Sigma Pu / \phi\ \Sigma Pc)]$	=	1.01 $\geq$ 1.00
Magnified Design Moment, Mc = δs Mmax	=	47,421.06 kN-m

MODIFIED STRENGTH REDUCTION FACTOR ϕ

Maximum Axial Stress, σPmax = Pmax / Ac	=	878.68 kPa
20% of Compressive Strength of Concrete, 0.2f'c	=	5,600.00 kPa
Approximate Balanced Axial Load, ϕPb = 0.2 f'c Ac	=	160,608.00 kN
Modified Strength Reduction Factor, $\phi = 0.90 - 0.40\ [\sigma Pmax / (0.20f'c)] \geq 0.50$	=	0.837

MAGNIFIED ELASTIC DESIGN FORCES

M DESIGN =	47421.06	kN-m
Pmax DES =	25200.68	kN
Pmin DES =	25200.66	kN

****** DESIGN COLUMN USING PCACOL PROGRAM ...***

****** NOTE 0.01 < As / Ag < 0.06***

ULTIMATE (Nominal) DESIGN FORCES FOR COLUMN

M ULTIMATE =	56639.96	kN-m
Pmax ULT =	30099.82	kN
Pmin ULT =	30099.80	kN

****** INVESTIGATE COLUMN PLASTIC CAPACITY FROM PCACOL INTERACTION DIAGRAM...***

FORCES RESULTING FROM PLASTIC HINGING

Pmax DES =	25,200.68	kN		M Nominal Cap =	30,384.50	kN-m
M Plastic =	39,499.85	kN-m	<	M Elastic =	139,194.97	kN-m
V Plastic =	8,194.99	kN				
Pmax Plastic =	25,200.68	kN				
Pmin Plastic =	25,200.66	kN				

COLUMN CONNECTIONS REQUIREMENTS

Anchorage for Uplift Forces, Ptens = 1.25 As fy			=	118.99 KN
Development Length, L_d $L_a = 0.04\ A_b f_v / f_c'^{\ 1/2}$ =	13.431945		=	341.26 mm
$L_a = 0.0004\ d_b f_y$ =	13.840909	**GOVERNS**	=	351.65 mm
Shear Stress @ Joint, vu = Vu / ϕAg $\leq$ sqrt(f'c) :	5,291.50 kPa	**ok**	=	309.01 kPa

CAPACITY-DEMAND RATIO OF PIER WALL

```
02/21/04 PCACOL V3.00 - PORTLAND CEMENT ASSOCIATION -                          Page   1
16:08:11 Licensed to: KEI, Pasig City, PI                                          L-4G
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         ===================================================================
         Computer program for the Strength Design of Reinforced Concrete Sections
         ===================================================================
```

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02/21/04 PCACOL V3.00 - PORTLAND CEMENT ASSOCIATION -                          Page   2
16:08:11 Licensed to: KEI, Pasig City, PI                                          L-4G

 General Information:
 ====================
    File Name: C:\MSNAVAL\PASIG--3\FROMKE-1\FINAL--1\APPEND-2\LAMBIN-1\L-4G.COL
    Project:  Lambingan Bridge
    Column:   Column                       Engineer: AHR
    Code:     ACI 318-95                   Units: Metric

    Run Option: Investigation              Slenderness: Not considered
    Run Axis:   Biaxial                    Column Type: Structural

 Material Properties:
 ====================
    f'c   = 28 MPa                         fy   = 303 MPa
    Ec    = 24870.1 MPa                    Es   = 200000 MPa
    fc    = 23.8 MPa                       Rupture strain = Infinity
    Ultimate strain = 0.003 mm/mm
    Beta1 = 0.846954

 Section:
 ========
    Rectangular: Width = 23561 mm          Depth = 1400 mm

    Gross section area, Ag =  3.29854e+007 mm^2
    Ix =  5.38762e+012 mm^4                Iy =  1.52591e+015 mm^4
    Xo =  0 mm                             Yo =  0 mm

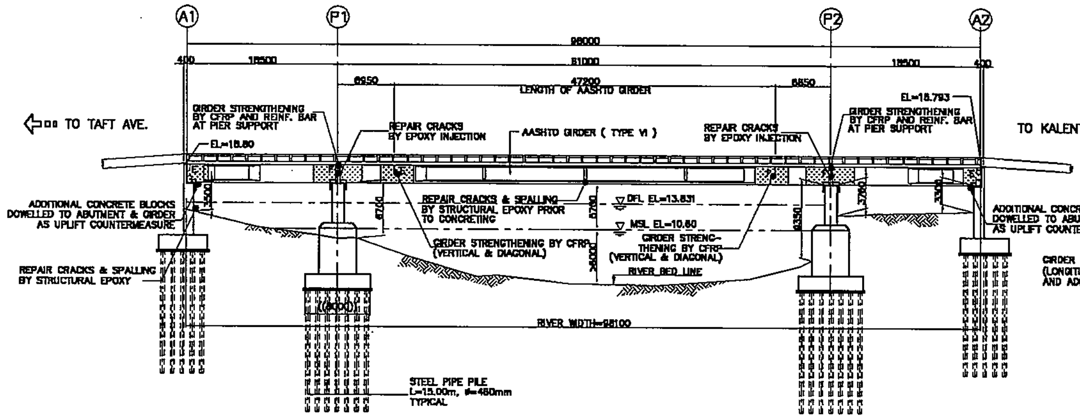
 Reinforcement:
 ==============
    Rebar Database: User-defined
    Size Diam (mm) Area (mm^2)   Size Diam (mm) Area (mm^2)   Size Diam (mm) Area (mm^2)
    ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
    #  6        6          28    #  8        8          50    # 10       10          79
    # 12       12         113    # 14       14         154    # 16       16         201
    # 20       20         314    # 25       25         491    # 28       28         616
    # 32       32         801    # 36       36        1018    # 40       40        1256

    Confinement: User-defined; #20 ties with #20 bars,  #20 with larger bars.
    phi(a) = 1,  phi(b) = 1,  phi(c) = 1

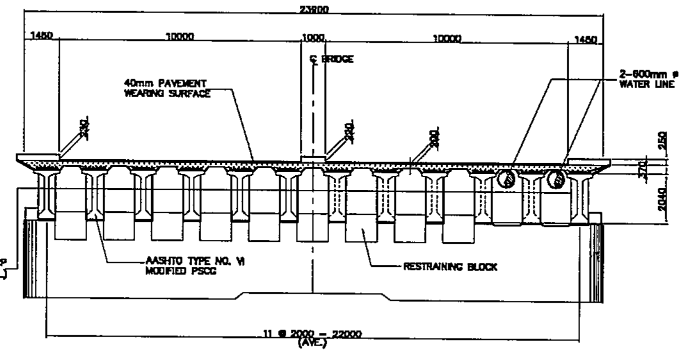
    Layout: Rectangular
    Pattern: Equal Bar Spacing  (Cover to transverse reinforcement)
    Total steel area, As = 52124 mm^2 at 0.16%
    166 #20    Cover = 50 mm

 Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
 ========================================================
              Pu          Mux         Muy        fMnx        fMny
    No.       kN         kN-m        kN-m        kN-m        kN-m    fMn/Mu
    --- ----------- ----------- ----------- ----------- ----------- --------
     1      25491.1         0.0     38000.9         0.0    462268.2   12.165
     2      25491.1     47251.1         0.0     27303.8         3.2    0.578

    *** Program completed as requested! ***
```



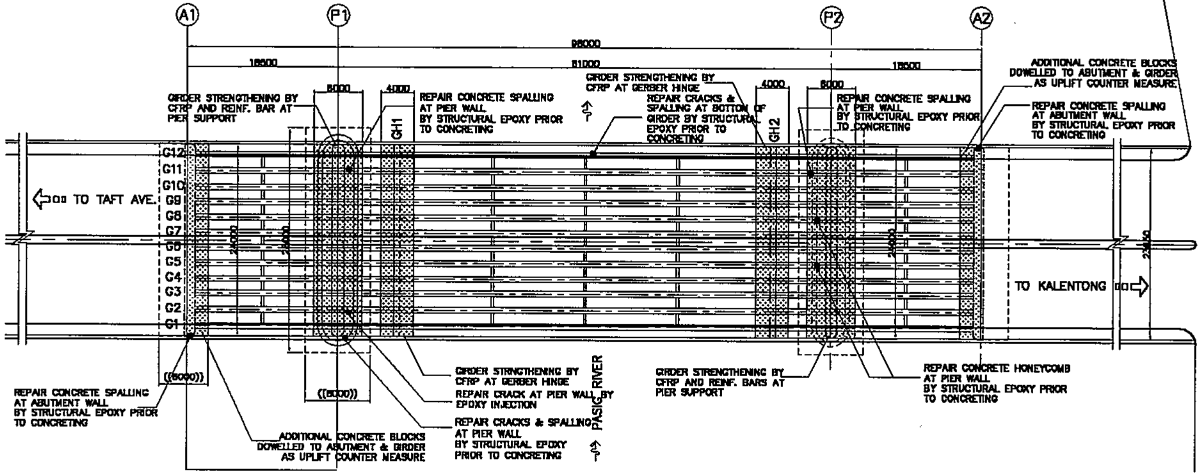
GENERAL ELEVATION
SCALE 1:300



CROSS SECTION @ P2
SCALE 1:100

SCOPE OF WORK

- REPAIR AND SEALING OF CONCRETE CRACKS, HONEYCOMB AND SPALLING.
- ADDITIONAL CONCRETE BLOCK DOWELED TO ABUTMENT AND GIRDER AS UPLIFT COUNTERMEASURE.
- INSTALLATION OF CFRP (CARBON FIBER REINFORCED POLYMER) LONGITUDINALLY AT TOP OF GIRDER AND HORIZONTALLY AT WEB OVER PIER SUPPORT.
- ADDITIONAL REINFORCING BARS AT TOP OG GIRDER OVER PIER SUPPORT.
- REHABILITATION OF GERBER HINGE PORTION WITH SLANTED P/S CABLES.
- RECONSTRUCTION OF DIAPHRAGM AND SLAB AT GERBER HINGE.

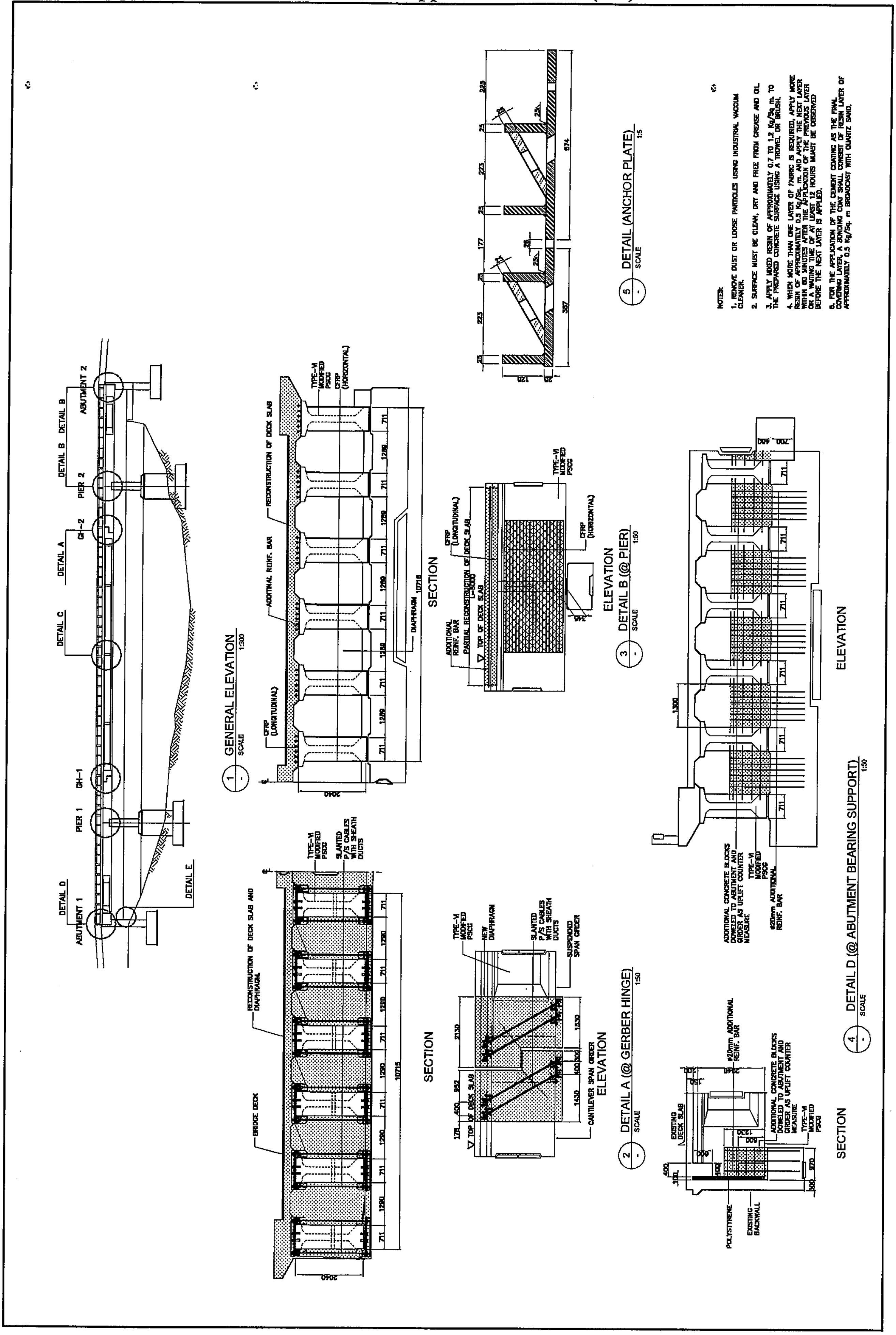


GENERAL PLAN
SCALE 1:300

QUANTITY ESTIMATE

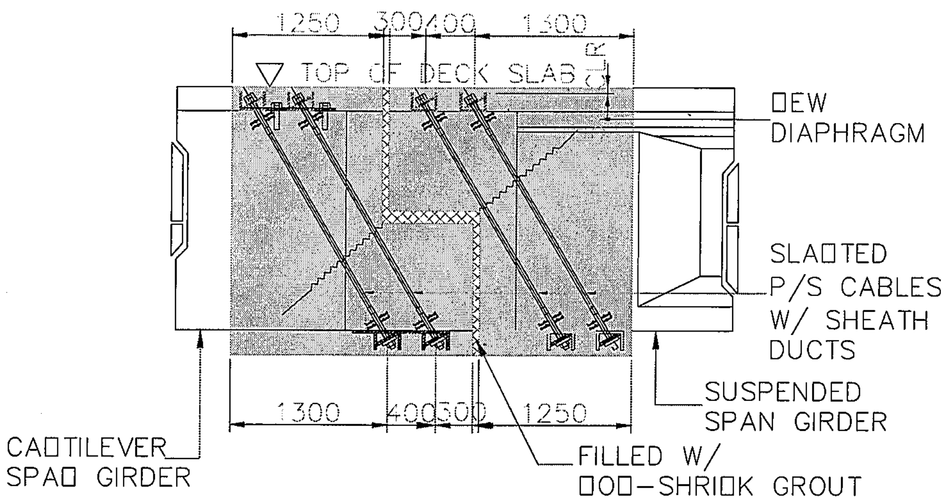
ITEM NO.	DESCRIPTION	UNIT	QUANTITY
1	Epoxy Bonding	l.s.	1.00
2	Concrete Grouting	cu.m.	9.00
3	Removal of Concrete Slab	sq.m.	359.00
4	Removal of Asphalt	sq.m.	2,264.00
5	Tack Coat	ton	1.00
6	Asphalt	ton	77.00
7	Structural Concrete for Slab	cu.m.	73.00
8	Uplift Device at Abutment	each	8.00
9	Installation of CFRP - Wrap	sq.m.	3,100.00
10	Installation of CFRP - Strip	l.m.	666.00
11	Reconditioning Steel Railings	l.m.	166.00
12	Steel Railings	l.m.	8.00
13	Reconditioning of Clogged Drainage W/ Cover	each	1.00
14	Gondola Falsework	l.s.	1.00
15	Traffic Management	l.s.	1.00
16	Contingencies	l.s.	1.00
17	Facilities	l.s.	1.00
18	Mobilization/demobilization	l.s.	1.00

DETAILS OF REHABILITATION (LAMBINGAN BRIDGE)



DAMAGE DETAILS OF REHABILITATION (LAMBINGAN BRIDGE)

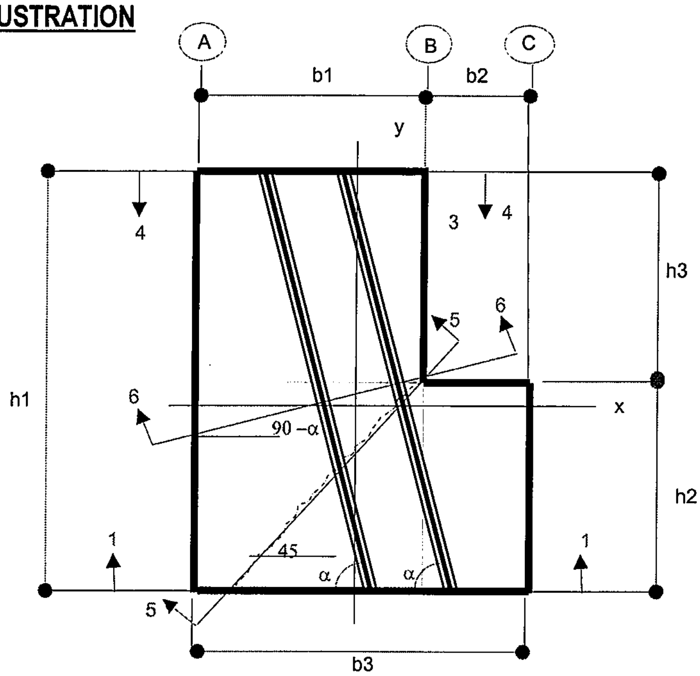
CAPACITY/DEMAND RATIO FOR LAMBINGAN BRIDGE (GERBER HINGE)



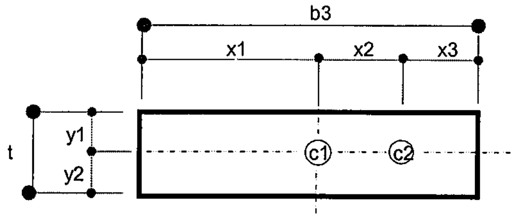
Type of Girder	-	**AASHTO Girder Type VI (Modified)**		
Dead Load Vertical Force at GerberHinge, V_{DL}		=	852.12	kN
Live Load Vertical Force at GerberHinge, V_{LL}		=	237.20	kN
Demand Force		=	1089.32	kN
Existing Condition :				
Reinforcing Bar Diameter		=	16.00	mm
Number of Reinforcing Bar Provided		=	6	pcs
Capacity of of Section due to Reinforcing Bars		=	564.00	kN
C/D Ratio		=	**0.52**	
Retrofit :				
Slanted Prestressing Bar Diameter		=	28.50	mm
Slanted Prestressing Bar Area		=	383.90	mm^2
Total Number of Slanted Prestressing Bar Provided per Girder		=	8	pcs
Capacity of Slanted Prestressing Bars		=	1411.00	kN
C/D Ratio		=	**1.30**	

CALCULATION OF GERBER HINGE REHABILITATION

ILLUSTRATION

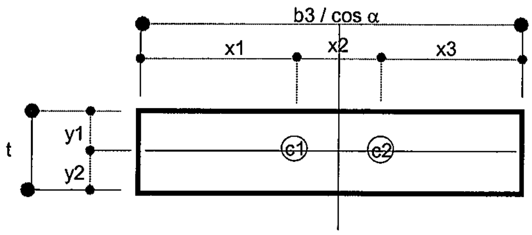


b1 =	1.53	m
b2 =	0.60	m
b3 =	2.13	m
h1 =	2.62	m
h2 =	1.20	m
h3 =	1.42	m
α =	60.00	deg



SECTION

Section	1	4	Unit
t =	0.300	0.300	m
y1 =	0.150	0.150	m
y2 =	0.150	0.150	m
x1 =	1.430	0.178	m
x2 =	0.400	0.400	m
x3 =	0.300	0.952	m
b3 or b1=	2.130	1.530	m



SECTION

Section	5	6	Unit
t =	0.300	0.300	m
y1 =	0.150	0.150	m
y2 =	0.150	0.150	m
x1 =	1.063	1.154	m
x2 =	0.359	0.346	m
x3 =	0.276	0.266	m
base, b =	1.698	1.766	m

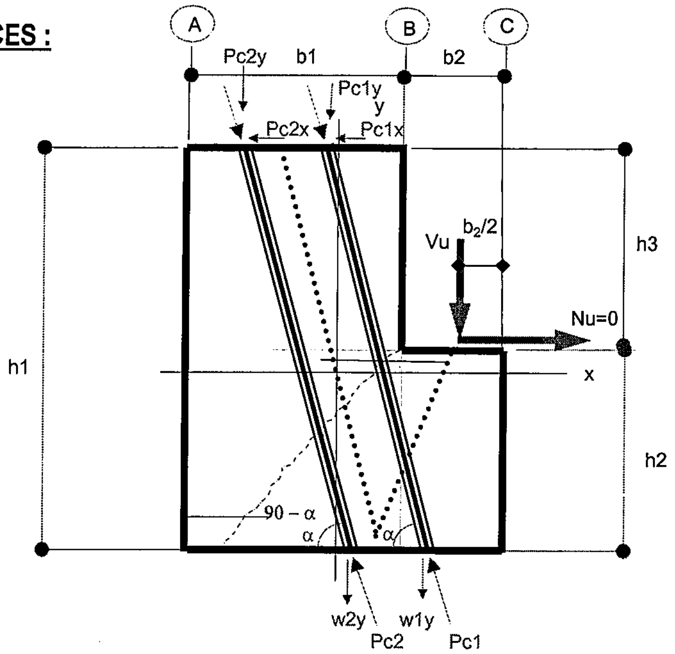
SECTION PROPERTIES

Section	1	5	6	4	Unit
Area, A	0.639	0.509	0.530	0.459	m^2
Dist. from N.A. to edge a, Xa =	1.065	0.849	0.883	0.765	m
Dist. from N.A. to edge c, Xc =	1.065	0.849	0.883	0.765	m
Moment of Inertia, I = t * b^3/12	0.242	0.122	0.138	0.090	m^4
Section modulus @ a, Sa	0.227	0.144	0.156	0.117	m^3
Section modulus @ c, S_c	0.227	0.144	0.156	0.117	m^3

MATERIAL SPECIFICATIONS

Compressive strength of concrete :			
at time of initial prestress, f'ci	=	22.40	Mpa
at 28th day, f'_c	=	28.00	Mpa
Ultimate strength of HTS, f'_s	=	1860.00	Mpa
Elastic modulus of HTS, E_S	=	195000	Mpa
Nominal area of HTS, A_{ps}	=	383.90	mm^2
Jacking stress, $0.70f'_s$	=	1302.00	Mpa
Number of HTS, N	=	1	pcs
.70Pu	=	499.84	kN
Total number of Prestressing steel =		4	pcs

FORCES :



Section	1	5	6	4	Unit
Shear reaction due to Dead Load, Wy =	852.12	852.12	852.12	852.12	kN
Shear reaction due to Live Load, Vll =	237.20	237.20	237.20	237.20	kN
Impact = (15.21 / 38.1 + L) =	0.21	0.21	0.21	0.21	
Wylocal = sin(90-α) * Wy due to DL	737.96	737.96	737.96	737.96	kN
Wxlocal = cos(90-α) * Wy due to DL	426.06	426.06	426.06	426.06	kN
Wylocal = sin(90-α) * Wy due to DL+LL+i	985.77	985.77	985.77	985.77	kN
Wxlocal = cos(90-α) * Wy due to DL+LL+i	569.14	569.14	569.14	569.14	kN
Effective 0.70Pu	432.87	499.84	499.84	432.87	kN

Assumption :
1) Shear,V are carried equally by the oblique prestress cables since spacing is not far apart.
2) Favorable effects of internal prestress tendon in the girders are neglected.
3) Horizontal force, Nu is neglected due to cable restrainer/or slab made continuous, preventing the horizontal force from developing.

ACTUAL ECCENTRICITY "e"

Section	1	5	6	4	No. of HTS
Distance of c.g. of C1 from edge c =	700 mm	635 mm	612 mm	1352 mm	1
Distance of c.g. of C2 from edge c =	300 mm	276 mm	266 mm	952 mm	1
Ya of strands	500 mm	456 mm	439 mm	1152 mm	Total = 2
Eccentricity "e"	565 mm	394 mm	444 mm	-387 mm	

LOSSES

A) Friction and Anchorage Draw-In

Section	1	5	6	4	Unit
Loss due to friction and anchorage draw-in, FS	0.00	0.00	0.00	0.00	Mpa

Note :
-Live End device using SEE (Screw type).
-Tendon profile is straight.

B) Elastic Shortening, ES

$$ES = \frac{0.50E_s f_{cir}}{E_{ci}}$$

where :

f_{cir} = Concrete stress at the center of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer, in mpa.

CALCULATION OF GERBER HINGE REHABILITATION

Section	1	5	6	4	Unit
Eci, modulus of elasticity of concrete in mpa at transfer =	22386.45	22386.45	22386.45	22386.45	Mpa
Concrete stress, f_{cir}	2.50	3.23	3.32	3.33	Mpa
Loss due to elastic shortening, ES	10.88	14.06	14.45	14.52	Mpa

C) Concrete Shrinkage, SH

Mean annual ambient relative humidity in percent, RH = 80.00 %
Loss due to concrete shrinkage, SH = 0.80(117-1.03RH)
= 92.94 Mpa

D) Creep of Concrete, CR_C

$CR_C = 12f_{cir} - 7f_{cds}$

where : f_{cds} = Concrete stress at center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied, in Mpa.

Section	1	2	6	4	Unit
Moment due to dead load (w/o beam weight)	0.00	0.00	0.00	0.00	kN-m
Concrete stress, f_{cds}	0.00	2.36	0.00	0.00	Mpa
Loss due to creep of concrete, CR_C	29.99	22.24	39.82	40.01	Mpa

E) Relaxation of Prestressing Steel, CR_S

$CR_S = 138 - 0.30FR - 0.40ES - 0.20(SH+CR_C)$ *... for stress relieved strands*

Section	1	5	6	4	Unit
Loss due to relaxation of prestressing steel, CR_S	109.06	109.34	105.67	105.60	Mpa

F) Effective Prestress at Initial and Final Condition

Section	1	5	6	4	Unit
Initial losses, FR + ES	10.88	14.06	14.45	14.52	Mpa
Effective prestress at initial condition	**1291.12**	**1287.94**	**1287.55**	**1287.48**	**Mpa**
Final losses, FR + ES + SH + CR_C + CR_S	242.87	238.57	159.94	160.13	Mpa
Effective prestress at final condition	**1059.13**	**1063.43**	**1142.06**	**1141.87**	**Mpa**

CHECK STRESSES

A) Only Prestress Force Acting.

Section	1		5		6		4		Unit
Number of strands, N	2		2		2		2		pcs.
Effective jacking force @ intial condition, Pj	858.51		988.88		988.58		856.09		kN
Eccentricity, e	0.565		0.394		0.444		-0.387		m
Stress at edge c, f c	3.48	C	4.64	C	4.68	C	-0.97	T	Mpa
Remarks	safe!		safe!		safe!		safe!		
Stress at edge a, f a	-0.79	T	-0.17	T	-0.95	T	4.70	C	Mpa
Remarks	safe!		safe!		safe!		safe!		

Allowable stresses : Compression = $0.55f'_{ci}$ = 12.32 Mpa
Tension = 1.40 Mpa or $0.25(f'_{ci})^{½}$ = -1.18 Mpa

B) If All DL is Acting.

Section	1		5		6		4		Unit
Axial Force due to dead load	-426.06		-368.98		-368.98		-426.06		kN
Number of strands, N	2		2		2		2		pcs.
Effective jacking force, Pj	704.25		816.50		876.88		759.27		kN
Eccentricity, e	0.565		0.394		0.444		-0.387		m
Stress at edge c, f c	1.13	C	2.10	C	2.40	C	-0.38	T	Mpa
Remarks	safe!		safe!		safe!		safe!		
Stress at edge a, f a	-0.26	T	-0.34	T	-0.49	T	1.83	C	Mpa
Remarks	safe!		safe!		safe!		safe!		

Allowable stresses : Compression = $0.40f'_c$ = 11.20 Mpa
Tension = $.50*(fc')^{.5}$ = -2.65 Mpa

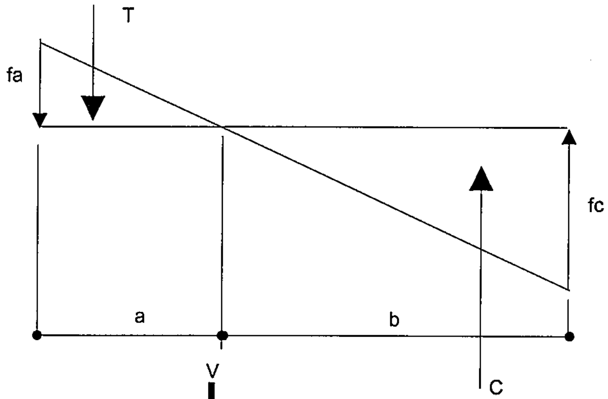
C) Due to All Dead Load and Live Load Plus Impact (Service Condition)

Section	1		5		6		4		Unit
Axial Force due to DL + LL+i	-544.66		-284.57		-284.57		-544.66		kN
Number of strands, N	2		2		2		2		pcs
Effective jacking force, Pj	704.25		816.50		876.88		759.27		kN
Eccentricity, e	0.565		0.394		0.444		-0.387		m
Stress at edge c, f c	0.647	C	2.496	C	2.804	C	-0.242	T	Mpa
Remarks	safe!		safe!		safe!		safe!		
Stress at edge a, f a	-0.15	T	-0.41	T	-0.57	T	1.18	C	Mpa
Remarks	safe!		safe!		safe!		safe!		

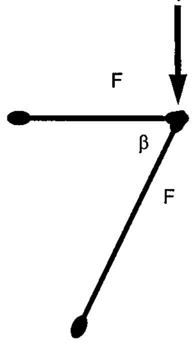
Allowable stresses : Compression = $0.40f'_c$ = 11.20 Mpa

Tension = $.50*(fc')^{.5}$ = -2.65 Mpa

Reinforcement Bars :



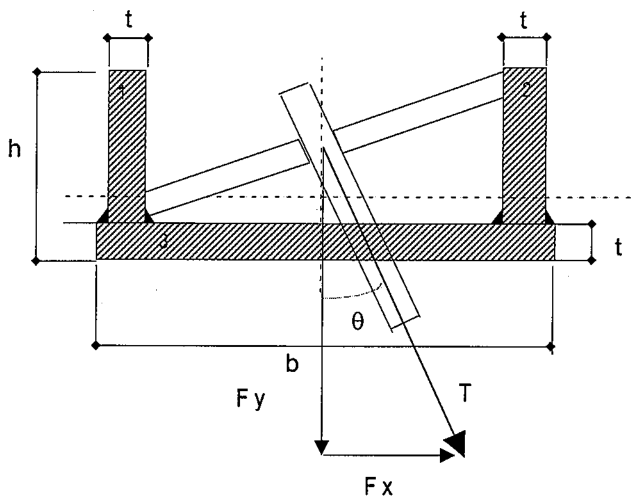
t = 0.30 m
a+b = 1.443 m
fa = 2.050 mpa
fc = 6.590 mpa
b = 1.101 m
a = 0.342 m
T = 105.3 kN
Fy = 414 mpa
As = T / .90 * Fy ;
As = 282.6 mm2
provide = 2 - 16 ; Ap = 402.1239 mm^2
cos (90 - α) * T = 130 kN O.K!



$F_2 = Vu * \sin\beta$; F_2 = 1677.635 kN
$F_1 = Vu / \tan\beta$; F_1 = 431.09 kN ; As = F1 / (.90 * Fy)
β = 76 deg Fy = 414 mpa
Vu = 1728.993 kN As = 1156.969 mm^2
provide = 1 - 36 ; Ap = 1017.876 mm^2

Dimension & Material Properties of steel channel anchorage::

Specified minimum yield stress of structural steel, Fy = 245 mpa



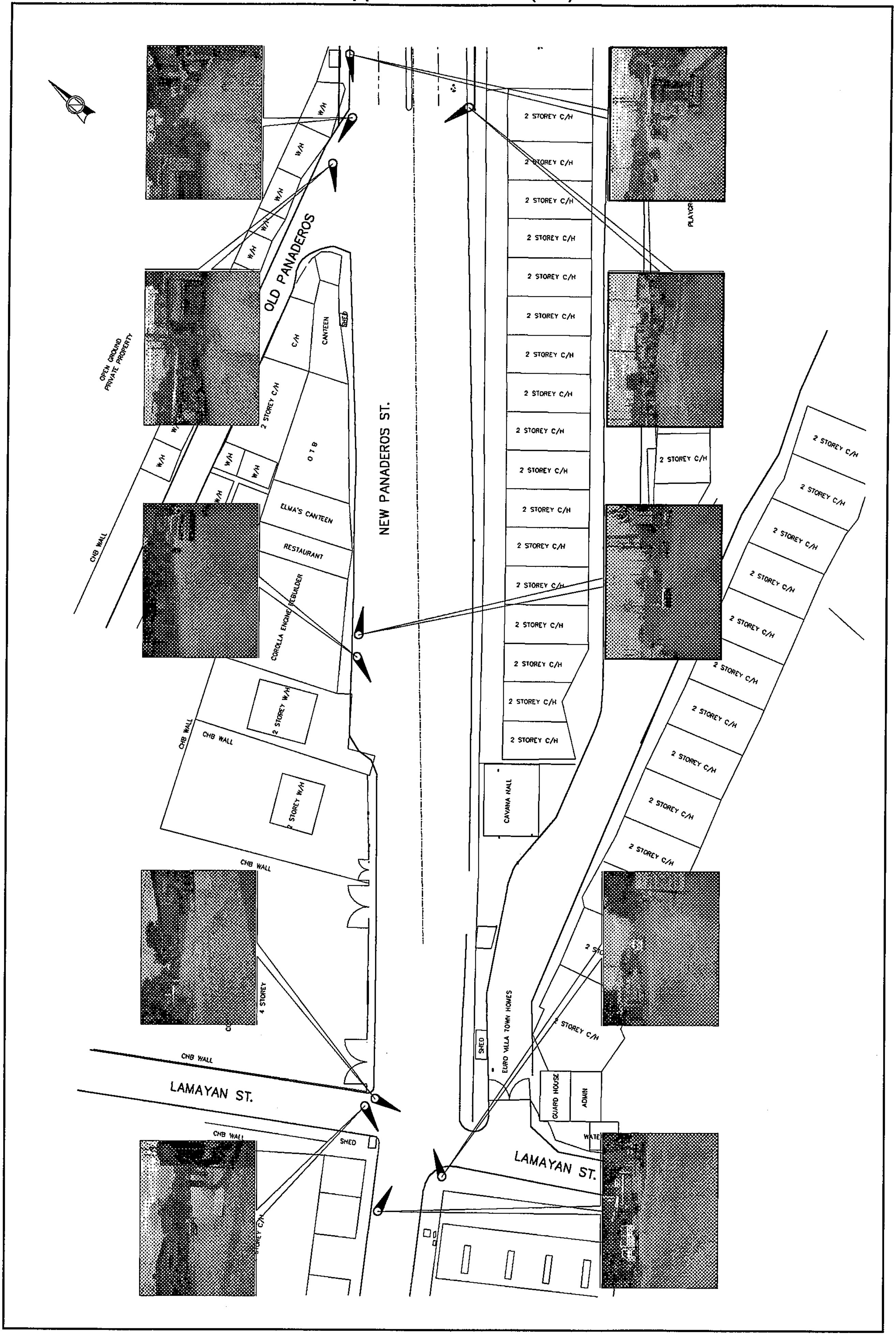
Appendix 22.3.1-2 (6/6)
CALCULATION OF GERBER HINGE REHABILITATION

element	t (mm)	h (mm)	Area (mm2)	y (mm)	A*y (mm3)	Ix = bh3/3 (mm4)	A(Y-y)2 (mm4)
1	22	125	2750	62.5	171875	14322916.67	1236645.978
2	22	125	2750	62.5	171875	14322916.67	1236645.978
3	22	175	3850	11	42350	621133.3333	3533274.221
			9350		386100	29266966.67	6006566.176

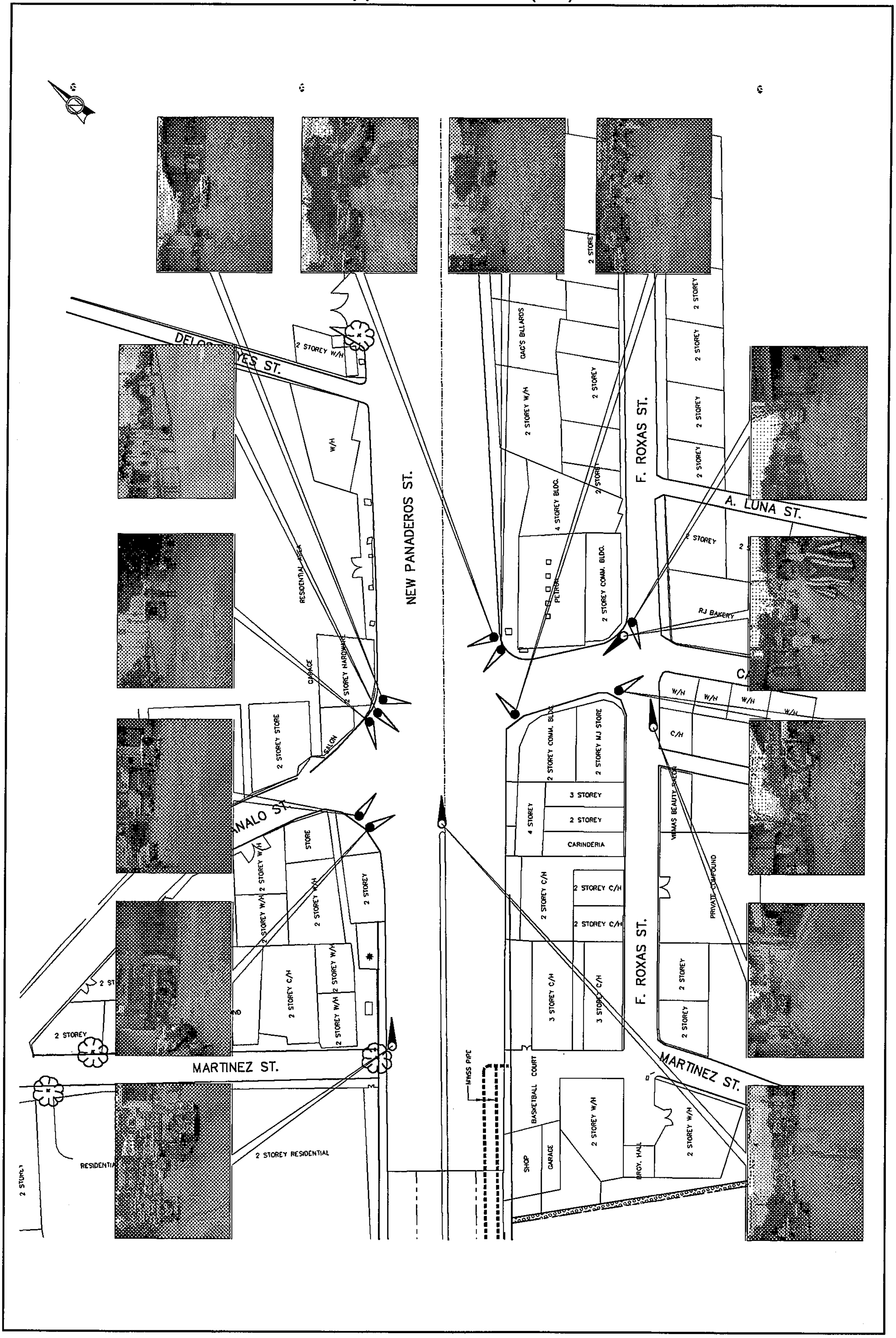
Y =	41.29	mm
Ix =	35273532.84	mm^4
Sx =	854202.36	mm^3

Check bending and shear stress :

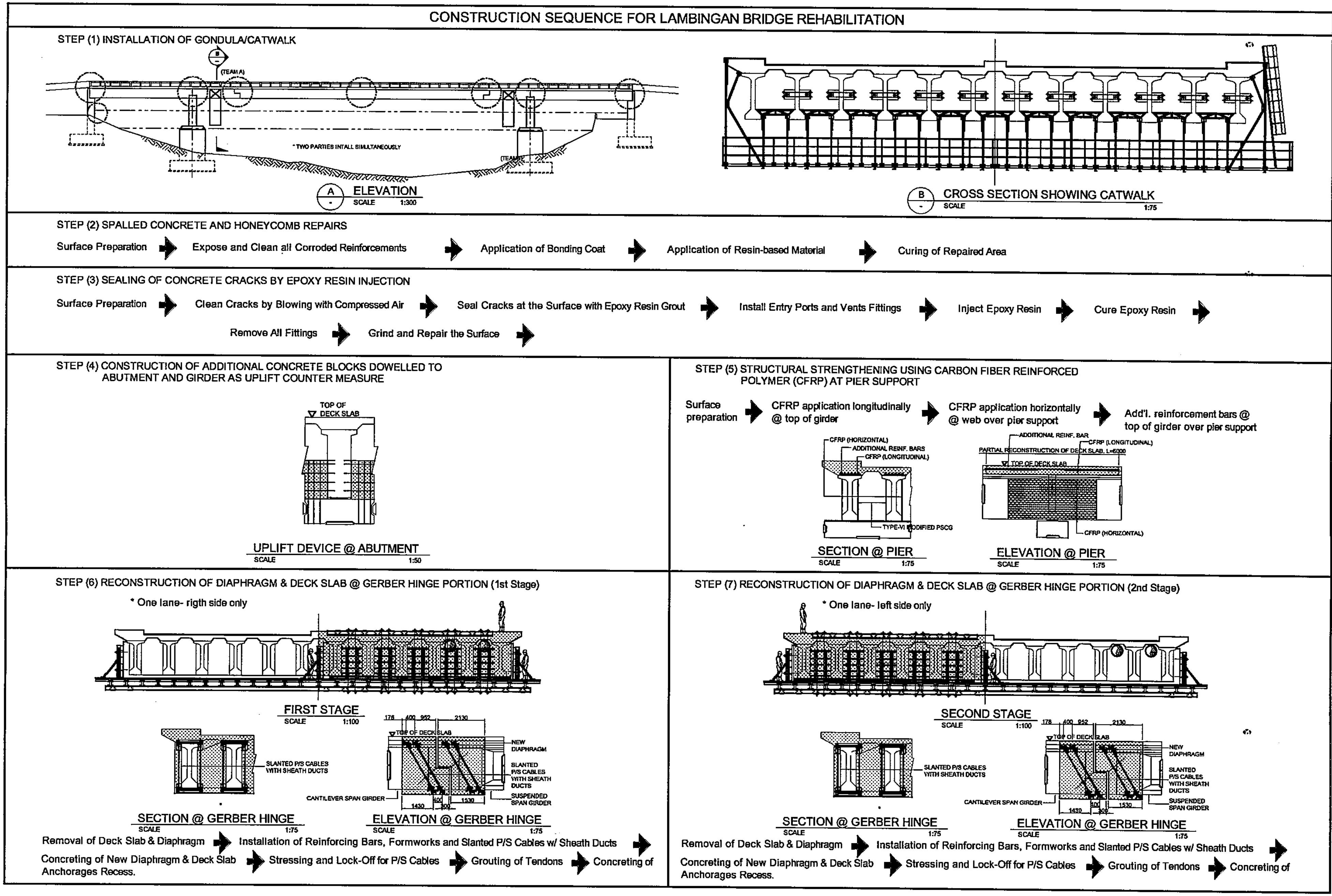
F =	499.84	kN	
θ =	30	deg	
0.70Puy =	432.87	kN	
0.70Pux =	249.92	kN	
cantilever arm =	0	m	
moment =	0.00	kN-m	
.55Fy =	134.75	mpa	
fb = M/Sx =	0.00	mpa	OK!
Fv = .33Fy	80.85	mpa	
Fv = V / A	78.70	mpa	OK!



APPROACH 1 SITE OCCULAR INSPECTION (LAMBINGAN BRIDGE)



APPROACH 2 SITE OCCULAR INSPECTION (LAMBINGAN BRIDGE)



CONTRUCTION SEQUENCE OF LAMBINGAN BRIDGE

Appendix 22.3.3-1 (1/2)
BREAKDOWN OF COSTS

Annex II - Construction Cost for Retrofitting of Lambingan Bridge

Description		Unit	Quantity	Unit Price	Cost	Components		
						Foreign	Local	Taxes
A. Repair/Sealing of Concrete Cracks								
SPL	Epoxy Bonding	l.s.	1.00	9,827,443.90	9,827,443.90	7,174,034.05	1,474,116.59	1,179,293.27
SPL	Concrete Grouting	cu.m.	1.00	2,948,233.17	2,948,233.17	1,916,351.56	737,058.29	294,823.32
B. Partial Reconstruction of Deck Slab/New Diaphragm								
101(3)	Removal of Concrete Slab	sq.m.	399.00	500.00	199,500.00	129,675.00	41,895.00	27,930.00
	Removal of Diaphram	cu.m.	50.46	575.00	29,014.50	18,859.43	6,093.05	4,062.03
101(3)b	Removal of Asphalt	sq.m.	2,264.00	150.00	339,600.00	220,740.00	71,316.00	47,544.00
301(1)	Tack Coat	ton	0.16	25,000.00	4,016.16	3,052.28	401.62	562.26
310	Asphalt	ton	86.00	3,100.00	266,600.00	202,616.00	26,660.00	37,324.00
405(1)a	Structural Concrete for Slab	cu.m.	80.00	4,500.00	360,000.00	234,000.00	75,600.00	50,400.00
405(3)	Structural Concrete for New Diapahragm	cu.m.	224.00	6,000.00	1,344,000.00	873,600.00	282,240.00	188,160.00
404	Reinforcing Steel Bars	kgs	50,507.00	50.00	2,525,350.00	1,641,477.50	530,323.50	353,549.00
416(1)	Prestressing Bar with Anchor	kgs	4,469.00	604.27	2,700,500.51	1,755,325.33	567,105.11	378,070.07
C. Installation of Carbon Fiber Reinforced Polymer								
	Installation of CFRP - Wrap	sq.m.	722.00	7,057.19	5,095,288.81	4,076,231.05	509,528.88	509,528.88
	Installation of CFRP - Strip	l.m.	514.00	5,340.53	2,745,032.38	2,196,025.91	274,503.24	274,503.24
D. Steel Railings								
	Reconditioning Steel Railings	l.m.	188.00	1,500.00	282,000.00	183,300.00	59,220.00	39,480.00
	Steel Railings	l.m.	8.00	12,636.69	101,093.52	65,710.79	21,229.64	14,153.09
E. Drainage								
	Reconditioning of Clogged Drainage with Cover	each	1.00	8,204.76	8,204.76	5,333.09	1,723.00	1,148.67
F. Gondola and Falsework								
SPL	Gondola and Falsework	l.s.	1.00	12,465,544.36	12,465,544.36	8,476,570.17	2,243,797.99	1,745,176.21
G. Traffic Management								
SPL	Traffic Management	l.s.	1.00	2,000,000.00	2,000,000.00	1,420,000.00	300,000.00	280,000.00
H. Contingencies								
	Contingencies	l.s.	1.00	2,162,071.10	2,162,071.10	1,621,553.33	324,310.67	216,207.11
I. Temporary Facilities								
	Facilities	l.s.	1.00	2,279,851.20	2,279,851.20	1,481,903.28	478,768.75	319,179.17
I. Mobilization/Demobilization								
	Mobilization/demobilization	l.s.	1.00	1,297,242.66	1,297,242.66	972,932.00	194,586.40	129,724.27
				Total	**48,980,587.04**	**34,669,290.76**	**8,220,477.71**	**6,090,818.58**
				% Component	**100%**	**71%**	**17%**	**12%**

Annex III - Roadway Improvement (Lambingan Bridge)

Item No.	Description	Unit	Quantity	Unit Cost	Amount	Component		
						Foreign	Local	Tax
	Miscellaneous							
	Concrete Median	m^2	30.00	272.93	8,187.96	5,322.17	1,719.47	1,146.31
600(1)	Concrete Curb	l.m.	104.00	562.46	58,496.15	38,022.50	12,284.19	8,189.46
612(1)	Pavement Markings	m^2	380.00	862.13	327,607.50	212,944.88	68,797.58	45,865.05
	Contingencies	l.s.	1.00	19,714.58	19,714.58	14,785.94	2,957.19	1,971.46
	Traffic Signal (2 Intersection)	l.s.	1.00	3,000,000.00	3,000,000.00	2,250,000.00	450,000.00	300,000.00
				Total	**3,414,006.19**	**2,521,075.48**	**535,758.43**	**357,172.28**
				% Component	**100%**	**74%**	**16%**	**10%**