DESIGN OF HIGH LEVEL BRIDGE

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

Hydraulic Calculation

Computation of Discharge

1 Flood calculation by Area Velocity Method (As per Article- 5 of IRC SP-13)

Q =	A x V	Where		
A =	751.37 m2		A =	Cross sectional area in m^2
P =	89.44 m		P =	Perimeter calculated in m
S =	1 IN	8333	S =	Slope as per drain LS taken at Proposal site
n =	0.033		n =	Rugosity coefficient (As per IRC SP-13)
V =	$I/n x (A/P)^{2/3} x (S)^{1/2}$		V =	Velocity in m/sec.
=	1.38 m/sec.			
Q =	1036.89 Cumecs			

Linear Water Way Calculation

Regime Surface width of the stream is given by :- L = 4.8 (Q)1/2

= 154.57 m

Looking to the approach gradient constraints adopt 8 Spans of 10 M each.

This will cause contraction and afflux. Calculation is done for the same to fix deck level.

Effective linear water way proposed = 8 x 10 = 80 M

Total 80 M

Scour Depth Calculation

(As per clause no. 703.2.2.1 of IRC : 78.1983)

$dsm = 1.34x (Db^2 /Ksf)^{1/3}$

Where

Db = The discharge in Cumecs per meter width

Ksf = the silt factor

= 1.5

Effective linear waterway = Width of waterway - Obstructed width of piper

= 78.80 - (7 x 1.2)

= 70.40 m

Db = 1036.89 / 70.40

= 14.73 Cumecs per metre width

dsm = 7.04 m

As per Clause No. 703-2-3-1 of IRC 78-1983 considering Scour at the pier two times of calculated scour depth below the highest flood level. But hard rock is available in foundation so the foundation will be anchored in the rock as per IRC guidelines.

Afflux Calculation

As per IS: 7784 (Part -I) 1975

Molesworth Formula for Afflux

Afflux h = $((V^2/17.85) + 0.0152) x (A^2/a^2 - 1)$

Where,

h = afflux in m,

v = Velocity in the unobstructed stream in m/s,

A = the unobstructed sectional area of the river in m^2

a = the obstructed sectional area of the river at the cross drainage work in m^2.

As per Annexure- 1

Unobstructed Area of Flow after Bridge Construction = 739.74 m^2

A = 739.744 m^2

V = 1.38 m/sec.

Computation of Area obstructed by Deck Slab

HFL :	98.500	m
Top Level of Deck slab :	100.755	m
Free Board	1.200	m
Thickness of Slab and Wearing Coat	0.975	m
Length Of Slab	78.800	m
Height of Obstruction	0.975	m
Area obstructed by deck slab	78.800 x 0.98	
=	76.83	m^2

Computation of Area obstructed by Piers

HFL :	98.500	m
Soffit of Deck slab :	99.780	m
Average river bed level =	89.816	m
Nos. of pier =	7	
Height of Obstruction	98.500 - 89.816 = 8.684	m
Area obstructed by one pier : =	1.2 x 8.68	
=	10.421	m^2
For 7 Nos. of piers =	7 x 10.421	
A1 =	72.94	m^2

Computation of Area obstructed by Abutments

Average ground level =		89.816 m			
Height of Obstruction		98.500 -	91.316 =	7.184 m	
Area obstructed by one Abutment : A2 =	(0.40+0.75)/2	x	7.18		
	=	4.13 m^2			
For two Abutments =		2 x	4.13		
	=	8.26 m^2			
Total area of obstruction due to slab, piers and abutments A	=	A0 +A1 + A2			
	=	76.83 +	72.94 +		8.26
	=	158.04 m^2			
Actual Area of flow a =		739.744 -	158.04		
	=	581.71 m^2			
Afflux h =		0.08 m			
Afflux flood level =		98.500 +	0.08 =	98.580 m	
Obstructed Velocity	V	=	Q/a	-	
Obstructed Velocity	=	1036.89 /		581.71	
	=	1.79 m/sec			
However we consider design velocity		2.00 m/sec.			
Afflux flood level	=	**98.580 M**			
Soffit of deck slab	=	**99.780 M**			

This is well above the Afflux flood level.

Though it is not a high level bridge; there shall be no hindrance to traffic during high floods.

Hence OK.

DETERMINATION OF VELOCITY AT PROPOSED SUBMERSIBLE BRIDGE

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

AS PER UP-STREAM SECTION

HIGHEST FLOOD LEVEL 98.500 M

CHAINAGE	G.L.	DEPTH OF FLOW IN M	LENGTH OF FLOW	AVERAGE DEPTH OF FLOW	CROSS SECTIONAL AREA OF FLOW	WETTED PERIMETER
0	91.96	1.30	0.00	0.00	0.00	0.00
5	90.5	8.00	5.00	4.65	23.24	8.36
10	88.76	9.74	5.00	8.87	44.35	5.29
15	87.91	10.59	5.00	10.17	50.83	5.07
20	87.61	10.89	5.00	10.74	53.70	5.01
25	87.3	11.20	5.00	11.05	55.23	5.01
30	86.99	11.51	5.00	11.36	56.78	5.01
35	86.69	11.81	5.00	11.66	58.30	5.01
40	86.38	12.12	5.00	11.97	59.83	5.01
45	86.07	12.43	5.00	12.28	61.38	5.01
50	85.77	12.73	5.00	12.58	62.90	5.01
55	86.76	11.74	5.00	12.24	61.18	5.10
60	88.76	9.74	5.00	10.74	53.70	5.39
65	90.81	7.69	5.00	8.72	43.58	5.40
70	93.08	5.42	5.00	6.56	32.78	5.49
75	95.36	3.14	5.00	4.28	21.40	5.50
80	97.43	1.07	5.00	2.11	10.53	5.41
83.18	98.55	0.00	3.18	0.53	1.70	3.36
		TOTAL	83.18		751.37	89.44

A		751.37	SQM
P		89.44	M
R		8.40	M
N		0.033	
S	1 IN	8333	
V		1.37	M/SEC

CHAINAGE	G.L.	DEPTH OF FLOW IN M	LENGTH OF FLOW	AVERAGE DEPTH OF FLOW	CROSS SECTIONAL AREA OF FLOW	WETTED PERIMETER

Q 1030.74 CUMECS

The design engineer visually observed the river to ascertain the Roughness Coefficient n for the Manning's formula. Upon visual inspection of the river in the vicinity of the proposed bridge site it was found that the River bed surface is good with clean straight banks, no rifts or deep pools however containing some weeds and stones. Roughness Coefficient pertaining to these characteristics is 0.033

Design Discharge = 1030.74 CUMECS

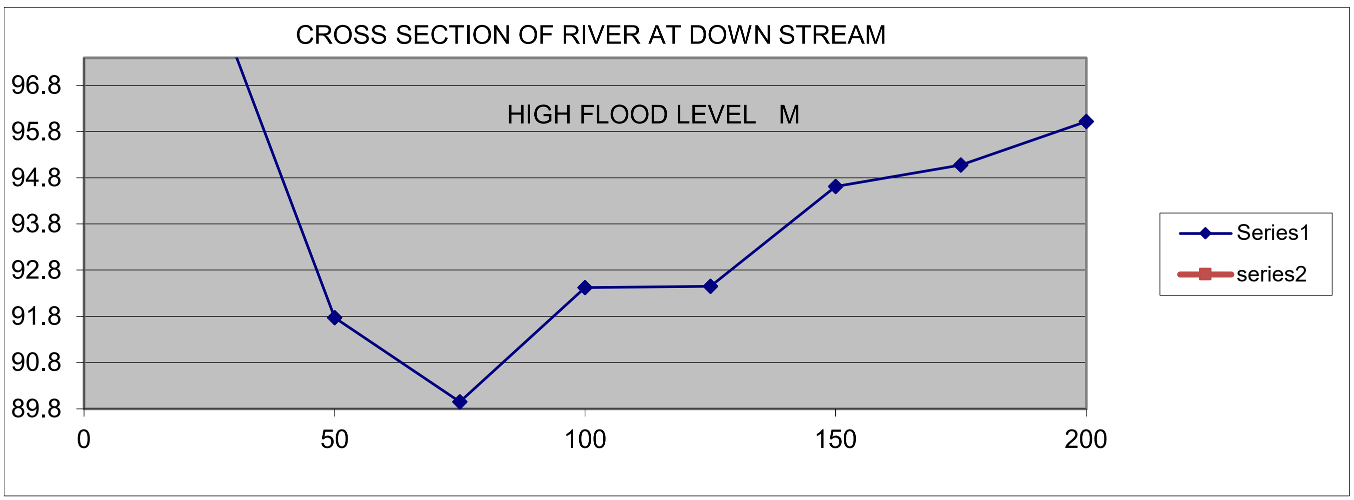
Critical Levels		
Road top level (RTL)	100.755	M
Average Ground Level(AGL)	89.816	M
Average Height Of Bridge	4.500	M
Nala Bed level (NBL)	82.570	M
Ordinary flood level (OFL)	96.000	M
Foundation level (FL)	79.000	M
Ht. of bridge h= (RTL-NBL)	18.185	M
Ht. of bridge H=(RTL-FL)	21.755	M

**** Needs Rational Evaluation w.r.t. afflux.**

**** Average of GL for points lying below HFL.**

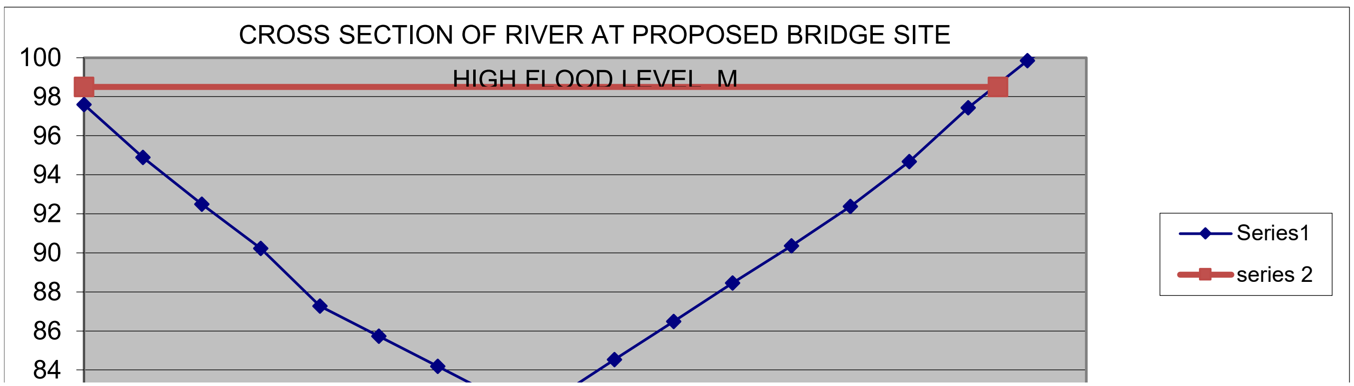
CROSS SECTION OF RIVER DOWN-STREAM
Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

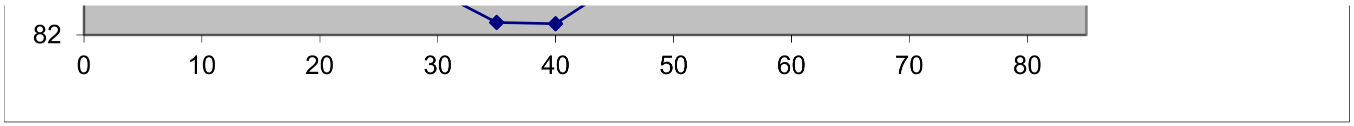
Chainage in M (u/s or d/s)	RL in M
0	100.63
25	98.976
50	91.771
75	89.95
100	92.423
125	92.451
150	94.611
175	95.074
200	96.02
30.00	98.50
180.00	98.50



CROSS SECTION OF RIVER AT PROPOSED BRIDGE SITE

		HIGHEST FLOOD LEVEL			98.500	M
Chainage in M (u/s or d/s)	RL in M	DEPTH OF FLOW IN M	LENGTH OF FLOW	AVERAGE DEPTH OF FLOW	CROSS SECTIONAL AREA OF FLOW	WETTED PERIMETER
0	97.59	0.91	0.00	0.00	0.00	0.00
5	94.89	3.61	5.00	2.26	11.30	5.68
10	92.49	6.01	5.00	4.81	24.05	5.55
15	90.23	8.27	5.00	7.14	35.70	5.49
20	87.27	11.23	5.00	9.75	48.75	5.81
25	85.73	12.77	5.00	12.00	60.00	5.23
30	84.18	14.32	5.00	13.55	67.73	5.23
35	82.63	15.87	5.00	15.10	75.48	5.23
40	82.57	15.93	5.00	15.90	79.50	5.00
45	84.53	13.97	5.00	13.37	66.85	5.70
50	86.48	12.02	5.00	13.17	65.85	5.06
55	88.46	10.04	5.00	9.16	45.78	5.30
60	90.36	8.14	5.00	9.09	45.45	5.35
65	92.38	6.12	5.00	8.08	40.40	6.35
70	94.68	3.82	5.00	5.98	29.90	7.98
75	97.43	1.07	5.00	3.60	17.98	8.66
80	99.84	0.00	5.00	3.06	15.30	9.55
83.18	101.08	0.00	3.18	3.06	9.74	8.74
		TOTAL	83.18		739.74	105.93
0.00	98.50					
77.50	98.50					





82
0
10
20
30
40
50
60
70
80

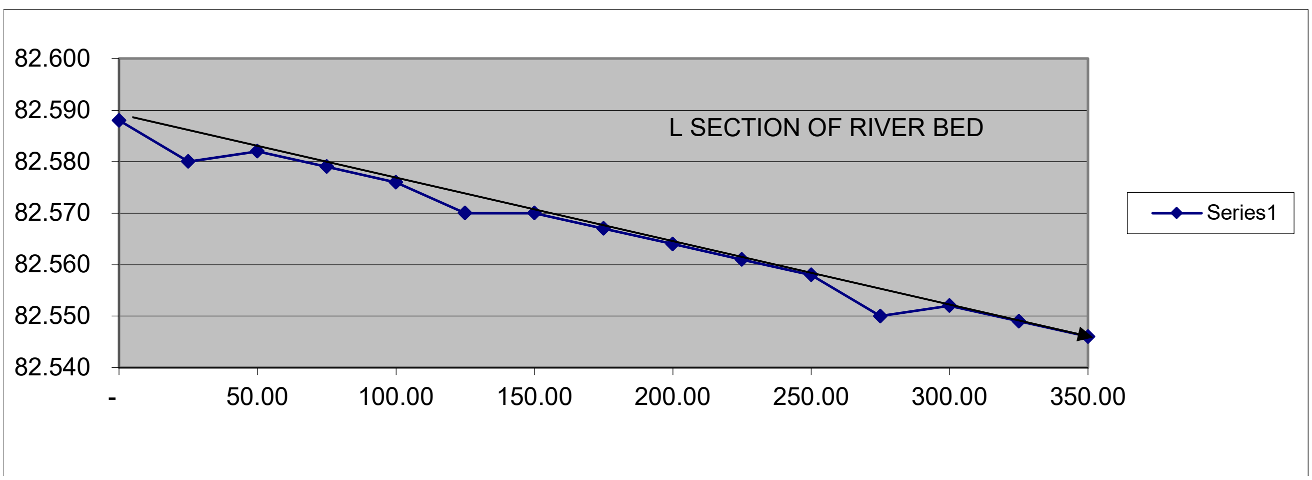
DETERMINATION OF BED SLOPE OF THE RIVER

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

Chainage in M (u/s or d/s)	RL in M
-	82.588
25.00	82.580
50.00	82.582
75.00	82.579
100.00	82.576
125.00	82.570
150.00	82.570
175.00	82.567
200.00	82.564
225.00	82.561
250.00	82.558
275.00	82.550
300.00	82.552
325.00	82.549
350.00	82.546

Reference Poits	
Ch	RL
0.00	82.588
350.00	82.546

DISTANCE			350	M
FALL			0.042	M
SLOPE	1	IN	8333	



DESIGN OF PIER AND CHECK FOR STABILITY- SUBMERSIBLE BRIDGE

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

DESIGN DATA

1	RIGHT EFFECTIVE SPAN	=	9.60	M
2	SPAN C/C OF PIERS	=	10.80	M
3	OVERALL WIDTH OF PIER CAP	=	12.00	M
4	H.F.L.	=	98.50	M
5	BUOYANCY			
6	AT FOOTING LEVEL	=	100.00	%
7	AT PIER LEVEL	=	100.00	%
8	AQUEDUCT FALLS UNDER ZONE-II SO SEISMIC CASE IS NOT GOVERNING HERE.			
9	FLOOD DISCHARGE	=	**1030.74**	**CUMECS**
10	RIVER BED SLOPE	=	1	IN 8333
11	DESIGN VELOCITY	=	2.00	m/sec
12	BED LEVEL OF THE HEIGHEST PIER	=	82.57	M
13	SAFE BEARING CAPACITY	=	**25.00 t/m2**	**250.00** **kN/m^2**
14	TOP LEVEL OF FOUNDING ROCK	=	80.50	M
15	EMBEDMENT OF PIER IN HARD ROCK	=	1.50	M
16	FOUNDATION LEVEL OF THE HIGHEST PIER	=	79.000	M
17	DECK LEVEL OF THE BRIDGE	=	100.755	M
18	TOP LEVEL OF THE PIER CAP	=	99.780	M
19	LEVEL DIFFERENCE OF PIER CAP TOP AND FOUNDING LEVEL	=	20.78	M

CHECKING STABILITY OF PIER AT R.L.79 M FOOTING LEVEL

A DEAD LOAD CALCULATION

SUPER STRUCTURE

Self Weight of Slab =	10.80 x	12.00 x	0.90 x	24.00 =	2799.36 kN		
Self Weight of Wearing Coat =	10.80 x	12.00 x	0.075 x	24.00 =	233.28 kN		
Railings and Footpath					62.00 kN		
TOTAL					**3094.64 kN**		

SUB STRUCTURE

Pier Cap

Pier Cap =	1.50 x	12.00 x	0.60 x	24.00		=	259.200 kN
Flared Portion Sides =	0.50 x	0.15 x	0.60 x	12.00 x	2.00 x	24.00 =	25.920 kN
=	0.50 x	0.15 x	0.60 x	3.14 x	1.20 x	24.00 =	4.069 kN
Flared Portion u/s & d/s Sides =	0.60 x	0.60 x	1.50 x	24.00		=	12.960 kN
=	3.14 /	4.00 x	1.20 x	1.20 x	0.60 x	24.00 =	16.278 kN
TOTAL							**318.427 kN**

Pier

Flared Portion Top =	**0.50** x	0.15 x	0.60 x	12.00 x	**2** x	24.00 =	25.920 kN
=	**0.50** x	0.15 x	0.60 x	**3.14** x	1.20 x	24.00 =	4.069 kN
Pier Rectangular portion =	1.20 x	12.00 x	17.93 x	24.00		=	6196.608 kN

Pier Curved portion =	**3.14** /	4 x	1.20 x	1.20 x	17.93 x	24.00 =		486.434 kN
Flared Portion bottom =	0.50 x	0.60 x	0.30 x	24.00		=		2.160 kN
=	**3.14** /	4 x	1.20 x	1.20 x	0.60 x	24.00 =		16.278 kN
TOTAL								**6736.895 kN**
Weight of Pier Above H.F.L. =								**0.000 kN**
Weight of Pier Below H.F.L. =	6736.89 -	0.00				=		**6736.895 kN**
Weight of Sub Structure with 15% Buoyancy =	0.00 + (	6736.89 x	22.50 /	24.00)		=		**6315.839 kN**
Footing	**SIZE**	**15.60**	**M x**	**3.80**	**M x**	**1.50**	**M**	
Weight without Buoyancy =	15.60 x	3.80 x	1.50 x	24.00		=		2134.080 kN
Weight with 100% Buoyancy =	15.60 x	3.80 x	1.50 x	**14.00**		=		1244.880 kN

Total Weight of Substructure Without Buoyancy

= 318.43 + 6736.89 + 2134.08 = **9189.402 kN**

Total Weight of Substructure With Buoyancy

= 318.43 + 6315.84 + 1244.88 = **7879.146 kN**

B LIVE LOAD CALCULATION

Maximum Reaction due Live Load including Impact

= 788.27 x 1.00 = **788.27 kN**

Refer Live load Computation sheet showing maximum reaction

= 78.83 T which is = 788.27 kN

Haunch	0.60	M
PCC Offset	0.20	M
Length Variant	1.00	M
Width Variant	0.50	M

TOTAL LONGITUDINAL MOMENT DUE TO LIVE LOAD & BREAKING FORCE

Maximum Longitudinal moment due to Live Load including Impact and Breaking Force

= 122.13 x 2.00 = **244.25 kN-m**

Refer Live load Computation sheet showing maximum reaction **= 12.21 T- m**

which is = 122.13 kN-m

243.85	Stress
144.78	

TOTAL TRANSVERSE MOMENT DUE TO LIVE LOAD & BREAKING FORCE

Maximum Transverse moment due to Live Load including Impact and Breaking Force

= 1123.94 x 2.00 = **2247.88 kN-m**

Refer Live load Computation sheet showing maximum reaction **= 112.39 T- m**

which is = 1123.94 kN-m

C LOADS DUE TO WATER CURRENT

WATER CURRENT IN LONGITUDINAL DIRECTION (ALONG THE BRIDGE)

As per IRC- II (6-1966) clause 213.5 For V= 2.00 m/sec

Since the bridge is at Zero Degrees skew from the direction of current as per IRC- II (6-1966) clause 213.5 it should be designed for (20+0) =20 Degrees or (20-0) = 20 Degrees whichever gives higher quantum of water current forces.

Obstructed Velocity = V Sin 20^0 = **2.00 x** **Sin 20^0**

= 0.68

$2v^2$ = 0.93

Total SUBMERGED Height = 18.00 M 0.93 0.88 0.88 0.00

FORCE ON DECK SLAB BETWEEN Deck Level 100.755 M to Soffit Level 99.93 M

$2v^2$ = (0.93 + 0.88) /2 = 0.91

Area Obstructed = 8.00 x 0.00 = 0.00 Sqm

Force on Pier = 52.00 x k x v^2 x Area Obstructed

= 52.00 x 1.50 x 0.91 x 0.00 / 100 = **0.00 kN** at R.L. 100.343 M

Moment @ R. L. 80.60 M = 0.00 x 19.74 = **0.00 kN-m**

Moment @ R. L. 80.00 M = 0.00 x 20.34 = **0.00 kN-m**

Moment @ R. L. 79.00 M = 0.00 x 21.34 = **0.00 kN-m**

FORCE ON PIER CAP BETWEEN 99.93 M to Soffit Level 99.33 M

$2v^2$ = (0.88 + 0.88) /2 = 0.88

Area Obstructed = 8.00 x 0.60 = 4.80 Sqm

Force on Pier = 52.00 x k x v^2 x Area Obstructed

= 52.00 x 1.50 x 0.88 x 4.80 / 100 = **3.30 kN** at R.L. 89.465 M

Moment @ R. L. 80.60 M = 3.30 x 8.86 = **29.24 kN-m**

Moment @ R. L. 80.00 M = 3.30 x 9.46 = **31.22 kN-m**

Moment @ R. L. 79.00 M = 3.30 x 10.47 = **34.52 kN-m**

FORCE ON PIER BETWEEN 99.33 M to 80.5 M

$2v^2$ = (0.88 + 0.00) /2 = 0.44

Area Obstructed = 7.33 x 13.20 = 96.82 Sqm

Force on Pier = 52.00 x k x v^2 x Area Obstructed

= 52.00 x 1.50 x 0.44 x 96.82 / 100 = **33.15 kN** at R.L. 89.165 M

Moment @ R. L. 81.10 M = 33.15 x 8.07 = **267.32 kN-m**

Moment @ R. L. 80.50 M = 33.15 x 8.66 = **287.21 kN-m**

Moment @ R. L. 79.00 M = 33.15 x 10.17 = **336.92 kN-m**

TOTAL LONGITUDINAL MOMENT DUE TO WATER CURRENT

Moment @ R. L. 81.10 M = 0.00 + 29.24 + 267.32 = **296.56 kN-m**

Moment @ R. L. 80.50 M = 0.00 + 31.22 + 287.21 = **318.43 kN-m**

Moment @ R. L. 79.00 M = 0.00 + 34.52 + 336.92 = **371.44 kN-m**

WATER CURRENT IN TRANSVERSE DIRECTION (ACROSS THE BRIDGE)

As per IRC- II (6-1966) clause 213.5 For V= 2.00 m/sec Maximum velocity being 1.414 x mean velocity (1.414= Root of 2)

Obstructed Velocity = V Cos 20 0 = **2.00 x Cos 20 0**

= 1.88

2v2 = 7.07

Total Height = 18.00 M 7.07 6.68 6.63 0.00

FORCE ON DECK SLAB BETWEEN Deck Level 100.755 M to Soffit Level 99.93 M

$2v^2$ = (7.07 + 6.68) /2 = 6.87

Area Obstructed = 10.80 x 0.000 = 0.00 Sqm

Force = 52.00 x k x v^2 x Area Obstructed

= 52.00 x 1.50 x 6.87 x 0.00 / 100 = **0.00 kN** at R.L. 100.343 M

Moment @ R. L.	80.60 M =	0.00 x	19.74 =	**0.00 kN-m**
Moment @ R. L.	80.00 M =	0.00 x	20.34 =	**0.00 kN-m**
Moment @ R. L.	79.00 M =	0.00 x	21.34 =	**0.00 kN-m**

FORCE ON PIER CAP BETWEEN 99.93 M to Soffit Level 99.33 M

$2v^2$ = (	6.68 +		6.63) /2 =	6.66					
Area Obstructed =	1.50 x		0.60 =	0.90 Sqm					
Force on Pier =	52.00 x	k	x	v^2 x	Area Obstructed				
=	52.00 x		1.50 x	6.66 x	0.90 / 100	=	**4.67 kN**	at R.L.	89.465 M
Moment @ R. L.	80.60 M =		3.30 x	8.86 =	**29.24 kN-m**				
Moment @ R. L.	80.00 M =		3.30 x	9.46 =	**31.22 kN-m**				
Moment @ R. L.	79.00 M =		3.30 x	10.47 =	**34.52 kN-m**				

FORCE ON PIER BETWEEN 99.33 M to 80.5 M

$2v^2$ = (	6.63 +		0.00) /2 =	3.32					
Area Obstructed =	7.33 x		1.20 =	8.80 Sqm					
Force on Pier =	52.00 x	k	x	v^2 x	Area Obstructed				
=	52.00 x		1.50 x	3.32 x	8.80 / 100	=	**22.77 kN**	at R.L.	89.165 M
Moment @ R. L.	80.60 M =		33.15 x	8.57 =	**283.89 kN-m**				
Moment @ R. L.	80.00 M =		33.15 x	9.16 =	**303.78 kN-m**				
Moment @ R. L.	79.00 M =		33.15 x	10.17 =	**336.92 kN-m**				

TOTAL TRANSVERSE MOMENT DUE TO WATER CURRENT

Moment @ R. L.	80.60 M =	0.00 +	29.24 =	
		+	283.89	**313.13 kN-m**
Moment @ R. L.	80.00 M =	0.00 +	31.22 =	
		+	303.78	**335.00 kN-m**
Moment @ R. L.	79.00 M =	0.00 +	34.52 =	
		+	336.92	**371.44 kN-m**

D SEISMIC CONDITION

According to clause 222.1 of IRC : 6- 1966 the Aqueduct is situated in the standard Zone- II ; therefore the aqueduct need not to be designed for Seismic Forces.

E WIND FORCE

Slab

Area =	11.10 x	0.98				=	10.82 Sqm
height of C.G. above Bed level =	100.34 -	82.57 =		17.77 m			
According to Clause 212.3 IRC -6 -1966	Wind pressure =	114.10 Kg/Sqm	=	1.14	kN/Sqm		
Wind Force =	10.82 x	1.14				=	**12.35 kN**
Moment @ R. L.	80.60 M =	12.35 x		19.74 =	**243.79 kN-m**		
Moment @ R. L.	80.00 M =	12.35 x		20.34 =	**251.20 kN-m**		
Moment @ R. L.	79.00 M =	12.35 x		21.34 =	**263.55 kN-m**		

Pier Cap

Area A1 =	1.50 x	0.60			=	0.90 Sqm
Area A2 =	1.35 x	0.60			=	0.81 Sqm
					Total	**1.71 Sqm**
¥ = (	0.90 x	0.90)+ (	0.81 x	0.30) /	1.71	0.62 M
height of C.G. above Bed level =	89.47 -	82.57 =	6.90 m			

According to Clause 212.3 IRC -6 -1966	Wind pressure =	90.17 Kg/Sqm	=		0.90	kN/Sqm		
Wind Force =	1.71 x	0.90				=		**1.54 kN**
Moment @ R. L.	80.60 M =	1.54 x	8.86 =	**13.67 kN-m**				
Moment @ R. L.	80.00 M =	1.54 x	9.46 =	**14.59 kN-m**				
Moment @ R. L.	79.00 M =	1.54 x	10.47 =	**16.14 kN-m**				
(I) **Pier from R.L.**	**99.780 to**	**82.57 M**						
Area =	1.20 x	17.21				=		20.65 Sqm
height of C.G. above Bed level =	91.18 -	82.57 =	8.61 m					
According to Clause 212.3 IRC -6 -1966	Wind pressure =	93.93 Kg/Sqm	=		0.94	kN/Sqm		
Wind Force =	20.65 x	0.94				=		**19.40 kN**
Moment @ R. L.	80.60 M =	19.40 x	10.58 =	**205.14 kN-m**				
Moment @ R. L.	80.00 M =	1.54 x	11.18 =	**17.23 kN-m**				
Moment @ R. L.	79.00 M =	1.54 x	12.18 =	**18.77 kN-m**				
TOTAL TRANSVERSE MOMENT DUE TO WIND FORCE								
Moment @ R. L.	80.60 M =	243.79 +	13.67 +	205.14 +				
				=		462.60 **kN-m**		
Moment @ R. L.	80.00 M =	251.20 +	14.59 +	17.23 +				
				=		283.02 **kN-m**		
Moment @ R. L.	79.00 M =	263.55 +	16.14 +	18.77 +				
				=		298.45 **kN-m**		

BASE PRESSURE CALCULATION

CASE- 1 FOR SERVICE CONDITION AT R. L.79 M

VERTICAL LOADS

DEAD LOAD CALCULATION

SUPER STRUCTURE	=	**3094.64 kN**		
SUB STRUCTURE	=	**9189.40 kN**	**Without Buoyancy**	
SUB STRUCTURE	=	**7879.15 kN**	**With Buoyancy**	
LIVE LOAD	=	**788.27 kN**		
Total Load without Buoyancy	=	**13072.31 kN**		
Total Load with Buoyancy	=	**11762.05 kN**		
Total LONGITUDINAL MOMENT	=	**371.44 +**	**244.25 =**	**615.70 kN-m**
Total TRANSVERSE MOMENT	=	**371.44 +**	**2247.88 =**	**2619.32 kN-m**

C.S.A. = 15.60 x 3.80 = 59.28 m^2

I_{xx} = 1/6x 15.60 x 3.80^2 = 37.54 m^3

I_{yy} = 1/6x 15.60^2 x 3.80 = 154.13 m^3

STRESS with Buoyancy = (11762.05 / 59.28)+ / - (615.70 / 37.54)+ / - (2619.32 / 154.13)

= 198.42 + / - 16.40 + / - 16.99

P_{max} = 198.42 + 16.40 + 16.99

= **231.81** **kN/m^2**

< 250 kN/m2 Hence O.K.

P_{min} = 198.42 - 16.40 - 16.99

= **165.02** **kN/m^2**

> 0 Hence O.K.

STRESS without Buoyancy = (13072.31 / 59.28)+ / - (615.70 / 37.54)+ / - (2619.32 / 154.13)

= 220.52 + / - 16.40 + / - 16.99

P_{max} = 220.52 + 16.40 + 16.99

= **241.91** **kN/m^2**

< 250 kN/m2 Hence O.K.

P_{min} = 220.52 - 16.40 - 16.99

= **187.12** kN/m^2

> 0 Hence O.K.

CASE- 2 FOR IDLE CONDITION AT R. L.79 M **(WHEN THERE IS NO LIVE LOAD)**

A CHECK OF STABILITY DUE TO BUOYANCY EFFECT

SUPER STRUCTURE	=	**3094.64 kN**	
SUB STRUCTURE	=	**9189.40 kN**	**Without Buoyancy**
SUB STRUCTURE	=	**7879.15 kN**	**With Buoyancy**
LIVE LOAD	=	**0.00 kN**	
Total Load without Buoyancy	=	**12284.04 kN**	
Total Load with Buoyancy	=	**10973.79 kN**	

STRESS with Buoyancy = (10973.79 / 59.28)+ / - (371.44 / 37.54)+ / - (371.44 / 154.13)

= 185.12 + / - 9.89 + / - 2.41

P_{max} = 185.12 + 9.89 + 2.41

= **197.42** kN/m^2

< 250 kN/m2 Hence O.K.

P_{min} = 185.12 - 9.89 - 2.41

= **172.81** kN/m^2

> 0 Hence O.K.

STRESS without Buoyancy = (12284.04 / 59.28)+ / - (371.44 / 37.54)+ / - (371.44 / 154.13)

= 207.22 + / - 9.89 + / - 2.41

P_{max} = 207.22 + 9.89 + 2.41

= **219.52** kN/m^2

< 250 kN/m2 Hence O.K.

P_{min} = 207.22 - 9.89 - 2.41

= **194.92** kN/m^2

> 0 Hence O.K.

CASE- 3 FOR WIND FORCE AT SERVICE CONDITION AT R. L.79 M

SUPER STRUCTURE	=	**3094.64 kN**				
SUB STRUCTURE	=	**9189.40 kN**	**Without Buoyancy**			
SUB STRUCTURE	=	**7879.15 kN**	**With Buoyancy**			
LIVE LOAD	=	**788.27 kN**				
Total Load without Buoyancy	=	**13072.31 kN**				
Total Load with Buoyancy	=	**11762.05 kN**				
Total LONGITUDINAL MOMENT	=	**371.44 +**	**244.25**		**=**	**615.70 kN-m**
Total TRANSVERSE MOMENT	=	**371.44 +**	**298.45 +**	**2247.88**	**=**	**2917.78 kN-m**

STRESS with Buoyancy = (11762.05 / 59.28)+ / - (615.70 / 37.54)+ / - (2917.78 / 154.13)

= 198.42 + / - 16.40 + / - 18.93

P_{max} = 198.42 + 16.40 + 18.93

= **233.75** kN/m^2

< 250 kN/m2 Hence O.K.

P_{min} = 198.42 - 16.40 - 18.93

= **163.08** kN/m^2

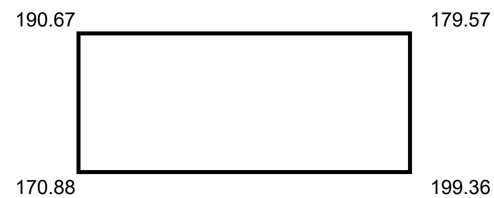
> 0 Hence O.K.

STRESS without Buoyancy = (13072.31 / 59.28)+ / - (615.70 / 37.54)+ / - (2917.78 / 154.13)
= 220.52 + / - 16.40 + / - 18.93
P_{max} = 220.52 + 16.40 + 18.93
= **243.85** **kN/m²**
< 250 kN/m2 Hence O.K.
P_{min} = 220.52 - 16.40 - 18.93
= **185.19** **kN/m²**
> 0 Hence O.K.

<u>CASE- 4 FOR WIND FORCE AT IDLE CONDITION AT R. L.79 M</u> **<u>[NO LIVE LOAD]</u>**

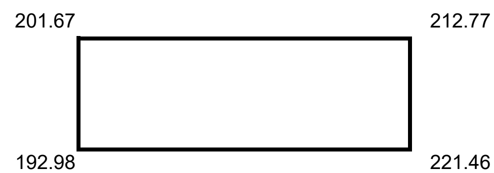
SUPER STRUCTURE	=	**3094.64 kN**	
SUB STRUCTURE	=	**9189.40 kN**	**Without Buoyancy**
SUB STRUCTURE	=	**7879.15 kN**	**With Buoyancy**
LIVE LOAD	=	**0.00 kN**	
Total Load without Buoyancy	=	**12284.04 kN**	
Total Load with Buoyancy	=	**10973.79 kN**	
Total LONGITUDINAL MOMENT	=	371.44 **kN-m**	
Total TRANSVERSE MOMENT	=	**371.44 +**	**298.45 = 669.90 kN-m**

STRESS with Buoyancy = (10973.79 / 59.28)+ / - (371.44 / 37.54)+ / - (669.90 / 154.13)
= 185.12 + / - 9.89 + / - 4.35
P_{max} = 185.12 + 9.89 + 4.35
= **199.36** **kN/m²**
< 250 kN/m2 Hence O.K.
P_{min} = 185.12 - 9.89 - 4.35
= **170.88** **kN/m²**
> 0 Hence O.K.
P_3 = 185.12 + 9.89 - 4.35
= **190.67** **kN/m²**
< 250 kN/m2 Hence O.K.
P_4 = 185.12 - 9.89 + 4.35
= **179.57** **kN/m²**
> 0 Hence O.K.



Stress Diagram

STRESS without Buoyancy = (12284.04 / 59.28)+ / - (371.44 / 37.54)+ / - (669.90 / 154.13)
= 207.22 + / - 9.89 + / - 4.35
P_{max} = 207.22 + 9.89 + 4.35
= **221.46** **kN/m²**
< 250 kN/m2 Hence O.K.
P_{min} = 207.22 - 9.89 - 4.35
= **192.98** **kN/m²**
> 0 Hence O.K.



Stress Diagram

<u>CASE- 5 FOR ONE SPAN DISLODGED CONDITION AT R. L.79 M</u>

SUPER STRUCTURE	=	**1547.32 kN**	
SUB STRUCTURE	=	**9189.40 kN**	**Without Buoyancy**
SUB STRUCTURE	=	**7879.15 kN**	**With Buoyancy**
LIVE LOAD	=	**0.00 kN**	

Total Load without Buoyancy	=	**10736.72 kN**											
Total Load with Buoyancy	=	**9426.47 kN**											
Total LONGITUDINAL MOMENT	=	**371.44 kN-m**											
Total TRANSVERSE MOMENT	=	**371.44 +**		**298.45 =**			**669.90 kN-m**						
STRESS with Buoyancy	= (	9426.47 /		59.28	)+ / - (		371.44	/	37.54	)+ / - (	669.90 /	154.13	)
	=	159.02	+ / -	9.89	+ / -		4.35						
P_{max}	=	159.02	+	9.89	+		4.35						
	=	**173.26** kN/m^2							153.47			164.56	
		< 250 kN/m2 Hence O.K.											
P_{min}	=	159.02	-	9.89	-		4.35						
	=	**144.78** kN/m^2											
P_3	=	159.02	+	9.89	-		4.35						
	=	**164.56** kN/m^2							144.78			173.26	
											Stress Diagram		
P_4	=	159.02	-	9.89	+		4.35						
	=	**153.47** kN/m^2											

STRESS without Buoyancy	= (	10736.72 /		59.28	)+ / - (	371.44	/	37.54	)+ / - (	0.00 /	154.13	)
	=	181.12	+ / -	9.89	+ / -	0.00						
P_{max}	=	181.12	+	9.89	+	0.00						
	=	**191.01** kN/m^2						171.23			191.01	
P_{min}	=	181.12	-	9.89	-	0.00						
	=	**171.23** kN/m^2										
P_3	=	181.12	+	9.89	-	0.00						
	=	**191.01** kN/m^2						171.23			191.01	
										Stress Diagram		
P_4	=	181.12	-	9.89	+	0.00						
	=	**171.23** kN/m^2										

CASE- 6 FOR SERVICE CONDITION AT R. L.80 M

VERTICAL LOADS

DEAD LOAD CALCULATION

SUPER STRUCTURE	=	**3094.64 kN**				
SUB STRUCTURE	=	**7055.32 kN**	**Without Buoyancy**			
SUB STRUCTURE	=	**6634.27 kN**	**With Buoyancy**			
LIVE LOAD	=	**788.27 kN**				
Total Load without Buoyancy	=	**10938.23 kN**				
Total Load with Buoyancy	=	**10517.17 kN**				
Total LONGITUDINAL MOMENT	=	**296.56 +**	**244.25 =**	**540.81 kN-m**		
Total TRANSVERSE MOMENT	=	**313.13 +**	**2247.88 =**	**2561.01 kN-m**		

C.S.A. = 12.00 x 1.20 = 14.40 m^2

I_{xx} = 1/6x 12.00 x 1.20^2 = 2.88 m^3

I_{yy} = 1/6x 12.00^2 x 1.20 = 28.80 m^3

STRESS with Buoyancy = (10517.17 / 14.40)+ / - (540.81 / 2.88)+ / - (2561.01 / 28.80)

=	730.36	+ / -	187.78	+ / -	88.92						
P_{max} =	730.36	+	187.78	+	88.92						
=	**1007.07** kN/m^2										
	< 8000 kN/m^2 (that is 8 N/mm^2) Hence O.K.										
P_{min} =	730.36	-	187.78	-	88.92						
=	**453.65** kN/m^2										
	> (- 3600 kN/m^2 (that is 3.6 N/mm^2) Hence O.K.										
STRESS without Buoyancy = (	10938.23 /		14.40	)+ / - (	540.81	/	2.88	)+ / - (	2561.01	/	28.80)
=	759.60	+ / -	187.78	+ / -	88.92						
P_{max} =	759.60	+	187.78	+	88.92						
=	**1036.31** kN/m^2										
	< 8000 kN/m^2 (that is 8 N/mm^2) Hence O.K.										
P_{min} =	759.60	-	187.78	-	88.92						
=	**482.89** kN/m^2										
	> (- 3600 kN/m^2 (that is 3.6 N/mm^2) Hence O.K.										

CASE- 7 FOR IDLE CONDITION AT R. L.80 M

SUPER STRUCTURE	=	**3094.64 kN**	
SUB STRUCTURE	=	**7055.32 kN**	**Without Buoyancy**
SUB STRUCTURE	=	**6634.27 kN**	**With Buoyancy**
LIVE LOAD	=	**0.00 kN**	
Total Load without Buoyancy	=	**10149.96 kN**	
Total Load with Buoyancy	=	**9728.91 kN**	

STRESS with Buoyancy = (	9728.91 /		14.40	)+ / - (	296.56	/	2.88	)+ / - (	313.13 /		28.80)
=	675.62	+ / -	102.97	+ / -	10.87						
P_{max} =	675.62	+	102.97	+	10.87						
=	**789.46** kN/m^2										
	< 8000 kN/m^2 (that is 8 N/mm^2) Hence O.K.										
P_{min} =	675.62	-	102.97	-	10.87						
=	**561.77** kN/m^2										
	> (- 3600 kN/m^2 (that is 3.6 N/mm^2) Hence O.K.										
STRESS without Buoyancy = (	10149.96 /		14.40	)+ / - (	296.56	/	2.88	)+ / - (	313.13 /		28.80)
=	704.86	+ / -	102.97	+ / -	10.87						
P_{max} =	704.86	+	102.97	+	10.87						
=	**818.70** kN/m^2										
	< 8000 kN/m^2 (that is 8 N/mm^2) Hence O.K.										
P_{min} =	704.86	-	102.97	-	10.87						
=	**591.01** kN/m^2										
	> (- 3600 kN/m^2 (that is 3.6 N/mm^2) Hence O.K.										

CASE- 8 FOR WIND FORCE AT SERVICE CONDITION AT R. L.80 M

SUPER STRUCTURE	=	**3094.64 kN**	
SUB STRUCTURE	=	**7055.32 kN**	**Without Buoyancy**
SUB STRUCTURE	=	**6634.27 kN**	**With Buoyancy**
LIVE LOAD	=	**788.27 kN**	

Total Load without Buoyancy	=	**10938.23 kN**										
Total Load with Buoyancy	=	**10517.17 kN**										
Total LONGITUDINAL MOMENT	=	**296.56 +**		**244.25**				**=**	**540.81 kN-m**			
Total TRANSVERSE MOMENT	=	**313.13 +**		**462.60 +**		**2247.88 =**			**3023.61 kN-m**			
STRESS with Buoyancy	= (	10517.17 /		14.40	)+ / - (	540.81	/		2.88	)+ / - (	3023.61 /	28.80)
	=	730.36	+ / -	187.78	+ / -	104.99						
P_{max}	=	730.36	+	187.78	+	104.99						
	=	**1023.13** kN/m^2										
		< 8000 kN/m^2 **(that is 8** N/mm^2 **) Hence O.K.**										
P_{min}	=	730.36	-	187.78	-	104.99						
	=	**437.59** kN/m^2										
		> (- 3600 kN/m^2 **(that is 3.6** N/mm^2 **) Hence O.K.**										
STRESS without Buoyancy	= (	10938.23 /		14.40	)+ / - (	540.81	/		2.88	)+ / - (	3023.61 /	28.80)
	=	759.60	+ / -	187.78	+ / -	104.99						
P_{max}	=	759.60	+	187.78	+	104.99						
	=	**1052.37** kN/m^2										
		< 8000 kN/m^2 **(that is 8** N/mm^2 **) Hence O.K.**										
P_{min}	=	759.60	-	187.78	-	104.99						
	=	**466.83** kN/m^2										
		> (- 3600 kN/m^2 **(that is 3.6** N/mm^2 **) Hence O.K.**										

CASE- 9 FOR WIND FORCE AT IDLE CONDITION AT R. L.80 M

SUPER STRUCTURE	=	**3094.64 kN**										
SUB STRUCTURE	=	**7055.32 kN**		**Without Buoyancy**								
SUB STRUCTURE	=	**6634.27 kN**		**With Buoyancy**								
LIVE LOAD	=	**788.27 kN**										
Total Load without Buoyancy	=	**10938.23 kN**										
Total Load with Buoyancy	=	**10517.17 kN**										
Total LONGITUDINAL MOMENT	=	**296.56 kN-m**										
Total TRANSVERSE MOMENT	=	**313.13 +**		**462.60 =**		**775.73 kN-m**						
STRESS with Buoyancy	= (	10517.17 /		14.40	)+ / - (	296.56	/		2.88	)+ / - (	775.73 /	28.80)
	=	730.36	+ / -	102.97	+ / -	26.94						
P_{max}	=	730.36	+	102.97	+	26.94						
	=	**860.27** kN/m^2										
		< 8000 kN/m^2 **(that is 8** N/mm^2 **) Hence O.K.**										
P_{min}	=	730.36	-	102.97	-	26.94						
	=	**600.45** kN/m^2										
		> (- 3600 kN/m^2 **(that is 3.6** N/mm^2 **) Hence O.K.**										
STRESS without Buoyancy	= (	10938.23 /		14.40	)+ / - (	296.56	/		2.88	)+ / - (	775.73 /	28.80)
	=	759.60	+ / -	102.97	+ / -	26.94						
P_{max}	=	759.60	+	102.97	+	26.94						
	=	**889.51** kN/m^2										
		< 8000 kN/m^2 **(that is 8** N/mm^2 **) Hence O.K.**										
P_{min}	=	759.60	-	102.97	-	26.94						

= **629.69** **kN/m^2**

> (- 3600 kN/m^2 (that is 3.6 N/mm^2) Hence O.K.

ABSTRACT OF BASE PRESSURE AND STRESSES

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

CASE- 1 FOR SERVICE CONDITION AT R. L.79 M	231.81	165.02	241.91	187.12		
CASE- 2 FOR IDLE CONDITION AT R. L.79 M	197.42	172.81	219.52	194.92		
CASE- 3 FOR WIND FORCE AT SERVICE CONDITION AT R. L.79 M	233.75	163.08	243.85	185.19		
CASE- 4 FOR WIND FORCE AT IDLE CONDITION AT R. L.79 M	199.36	170.88	190.67	179.57	221.46	192.98
CASE- 5 FOR ONE SPAN DISLODGED CONDITION AT R. L.79 M	173.26	144.78	164.56	153.47	181.12	171.23

Maximum 243.85 **144.78 Minimum**

CASE- 6 FOR SERVICE CONDITION AT R. L.80 M	1007.07	453.65	1036.31	482.89
CASE- 7 FOR IDLE CONDITION AT R. L.80 M	789.46	561.77	818.70	591.01
CASE- 8 FOR WIND FORCE AT SERVICE CONDITION AT R. L.80 M	1023.13	437.59	1052.37	466.83
CASE- 9 FOR WIND FORCE AT IDLE CONDITION AT R. L.80 M	860.27	600.45	889.51	629.69

Maximum 1052.37 **437.59 Minimum**

REINFORCEMENT CALCULATION IN PIER IN LOWER FLARED PORTION
Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

R.L. 80.50 M TO 81.10 M

FOR SERVICE CONDITION

VERTICAL LOADS				
SUPER STRUCTURE	=	3094.64 kN		
SUB STRUCTURE	=	7055.32 kN	Without Buoyancy	
SUB STRUCTURE	=	6634.27 kN	With Buoyancy	
LIVE LOAD	=	788.27 kN		
Total Load without Buoyancy	=	10938.23 kN		
Total Load with Buoyancy	=	10517.17 kN		

Total LONGITUDINAL MOMENT

Moment @ R. L. 80.00 M = 540.81 kN-m

Total TRANSVERSE MOMENT

Moment @ R. L. 80.00 M = 3023.61 kN-m

CONCRETE MIX M-25

CHARACTERISTIC STRENGTH OF REINFORCEMENT 415 N/mm2

PERMISSIBLE STRESSES

IN STEEL 190

IN CONCRETE

CHARACTERISTIC STRENGTH OF Concrete	fck	=	30 N/mm2
Permissible Compressive Stress in Bending	σcbc	=	8 N/mm2
Permissible Compressive Stress in Direct Compression	σcc	=	8 N/mm2
	σct	=	3.6 N/mm2

Ultimate Axial Load P_U	=	1.5 X	10938.23	=	16407.34	kN
Ultimate Longitudinal Moment M_U	=	1.5 X	540.81	=	811.2195	kN-m
Ultimate Transverse Moment M_U	=	1.5 X	3023.61	=	4535.417	kN-m

INCREASE WHEN WIND CONDITION IS CONSIDERED 33.33 %

Neglecting area of Cut and Ease water parts Rectangular Section considered is

12001 mm x 1201 mm

Assume cover as 75

d^1/d = 87.5 / 1201.2 = 0.0728

$P_U/(f_{ck}\ b\ d)$ = 16407.34 x 1000 / (30 x 12001 x 1201.2)

= 0.0379

FOR LONGITUDINAL MOMENT

$Mu/(f_{ck}\ b\ d^2)$ = 811.22 x 1000000 / (30 x 12001 x 1201.2^2)

= 0.0016

Refer Chart 31 & 32 of Design Aids for Reinforced concrete SP-16 the point lies below the range of applicability. Hence provide minimum percentage of steel.

The point lies below the range of applicability. Hence provide minimum percentage of steel
CRITERIA 1 FOR MINIMUM STEEL Pt = 0.8 % OF CROSS SECTION AREA OF COLUMN REQUIRED FOR COMPRESSION

Area Required due to Compression =		10517.17 x	1000 /	8	
	=	1314647 mm^2			
Area of steel @ 0.8% =		0.8 x	1314647 /	100	
	=	**10517** $\mathbf{mm^2}$			

CRITERIA 2 FOR MINIMUM STEEL Pt = 0.3 % OF GROSS SECTION AREA OF COLUMN

Area of steel @ 0.3% =		0.3 x	12001.2 x	1201.2 /	100
	=	**43248** $\mathbf{mm^2}$			
PROVIDE STEEL AREA	=	**43248** $\mathbf{mm^2}$			
NO. OF		25 MM BARS =	88 Nos.		
SPACING	=	290 MM			
FOR TRANSVERSE MOMENT					
$Mu/(f_{ck}\ b\ d^2)$	=	4535.42 x	1000000 / (	30 x	
			12001.2 x	1201.2^2)	
	=	0.0087			

Refer Chart 31 & 32 of Design Aids for Reinforced concrete SP-16 the point lies below the range of applicability. Hence provide minimum percentage of steel.

TRANSVERSE REINFORCEMENT

Shear Force to be resisted by the pier In Accordance to IS 1893

3023.61 / 11.87 = 254.67 kN

Check for Shear

Nominal Shear Stress =	254.67	x	1000	/ (	12001 x	1201)
	=	0.02 N/mm^2				
Pt	0.30					
Permissible Shear Stress =	0.40 N/mm^2	Refer table 61				

Nominal Shear Reinforcement will suffice

According to IRC 21-1987 Clause 306.3

Dia of Transverse Reinforcement	=	25 /	4 =	6.25 mm	
Provide		12 mm dia rings			
Pitch of the Transverse should be least of					
a) Least lateral Dimension =		1201.2 mm			
b) 12 d =		12 x	12 =	144 mm	
c) 300 mm =		300 mm			
d) As per IS IS 13920:1993 Cl. 7.4.6	< or =	100 mm			
Provide		12 mm dia rings @	100 mm c/c.		

This spacing is in accordance to IS 13920:1993 Cl. 7.4.6

CODE OF PRACTICE FOR DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES

Check for Size of Hoop Reinforcement **Refer IS 13920:1993 Cl. 7.4.8**

Ash= 0.18 Sh (Fck/Fy)x(Ag/Ak-1)

S	=	100.00	mm	
h	=	300.00	N/mm^2	(Spacing of long. bars+ effective cover) or 300 mm whichever is less
Fck	=	30.00	N/mm^2	**Cover 75 mm to main reinforcement**
Fy	=	415.00	N/mm^2	
Ag	=	1201.20	mm^2	Considering 1 mm Wide Pier
Ak	=	1100.20	mm^2	Considering 1 mm Wide Pier Effective
Hence Ash	=	35.84	mm^2	
Ash ProvideD	=	113.04	mm^2	Which is OK

d) As per IS IS 13920:1993 Cl. 7.4.6 < or = 100 mm

Provide 12 mm dia rings @ 100 mm c/c.

This spacing is in accordance to IS 13920:1993 Cl. 7.4.6

CODE OF PRACTICE FORDUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES

ABSTRACT

LONGITUDINAL REINFORCEMENT	**25**	**MM BARS**	**290**	**MM**	**However Adopt spacing as 250 mm**
TRANSVERSE REINFORCEMENT	**12mm dia rings @100mm c/c.**				

REINFORCEMENT CALCULATION IN PIER

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

R.L. 81.10 M TO 100.80 M

FOR SERVICE CONDITION

VERTICAL LOADS

SUPER STRUCTURE	=	**3094.64**	kN	
SUB STRUCTURE	=	**9189.40**	kN	Without Buoyancy
SUB STRUCTURE	=	**7879.15**	kN	With Buoyancy
LIVE LOAD	=	**788.27**	kN	
Total Load without Buoyancy	=	**13072.31**	kN	
Total Load with Buoyancy	=	**11762.05**	kN	

Total LONGITUDINAL MOMENT

Moment @ R. L. 81.10 M = 615.70 kN-m

Total TRANSVERSE MOMENT

Moment @ R. L. 81.10 M = 2619.32 kN-m

CONCRETE MIX M-25

CHARACTERISTIC STRENGTH OF REINFORCEMENT 415 N/mm2

PERMISSIBLE STRESSES

IN STEEL 190

IN CONCRETE

CHARACTERISTIC STRENGTH OF Concrete	fck	=	30	N/mm2
Permissible Compressive Stress in Bending	σcbc	=	8	N/mm2
Permissible Compressive Stress in Direct Compression	σcc	=	8	N/mm2
	σct	=	3.6	N/mm2

Ultimate Axial Load P_U	=	1.5	X	13072.31 =	19608.46	kN
Ultimate Longitudinal Moment M_U	=	1.5	X	615.70 =	923.5442	kN-m
Ultimate Transverse Moment M_U	=	1.5	X	2619.32 =	3928.986	kN-m

INCREASE WHEN WIND CONDITION IS CONSIDERED 33.33 %

Neglecting area of Cut and Ease water parts Rectangular Section considered is

12000 mm x 1200 mm

Assume cover as 75

d^1/d = 87.5 / 1200 = 0.0729

$P_U/(f_{ck}\ b\ d)$ = 19608.46 x 1000 / (30 x 12000 x 1200)

= 0.0454

FOR LONGITUDINAL MOMENT

$Mu/(f_{ck}\ b\ d^2)$ = 923.54 x 1000000 / (30 x 12000 x 1200^2)

= 0.0018

Refer Chart 31 & 32 of Design Aids for Reinforced concrete SP-16 the point lies below the range of applicability. Hence provide minimum percentage of steel.

The point lies below the range of applicability. Hence provide minimum percentage of steel

CRITERIA 1 FOR MINIMUM STEEL Pt = 0.8 % OF CROSS SECTION AREA OF COLUMN REQUIRED FOR COMPRESSION

Area Required due to Compression =		11762.05 x		1000 /	8	
	=	1470257 mm^2				
Area of steel @ 0.8% =		0.8 x	1470257 /	100		
	=	**11762** **mm^2**				

CRITERIA 2 FOR MINIMUM STEEL Pt = 0.3 % OF GROSS SECTION AREA OF COLUMN

Area of steel @ 0.3% =		0.3 x	12000 x	1200 /	100	
	=	**43200** **mm^2**				
PROVIDE STEEL AREA	=	**43200** **mm^2**				
NO. OF		25 MM BARS	=	88 Nos.		
SPACING	=	290 MM				
FOR TRANSVERSE MOMENT						
$Mu/(f_{ck}\ b\ d^2)$	=	3928.99 x	1000000 / (	30 x		
			12000 x	1200^{2})		
	=	0.0076				

Refer Chart 31 & 32 of Design Aids for Reinforced concrete SP-16 the point lies below the range of applicability. Hence provide minimum percentage of steel.

TRANSVERSE REINFORCEMENT

Shear Force to be resisted by the pier In Accordance to IS 1893

2619.32 / 11.87 = 220.62 kN

Check for Shear

Nominal Shear Stress =	220.62	x	1000	/ (	12000	x	1200)
	=		0.02 N/mm^2				
Pt		0.30					
Permissible Shear Stress =		0.40 N/mm^2	Refer table 61				

Nominal Shear Reinforcement will suffice

According to IRC 21-1987 Clause 306.3

Dia of Transverse Reinforcement	=	25 /	4 =	6.25 mm	
Provide		12 mm dia rings			
Pitch of the Transverse should be least of					
a) Least lateral Dimension =		1200 mm			
b) 12 d =		12 x	12 =	144 mm	
c) 300 mm =		300 mm			
d) As per IS IS 13920:1993 Cl. 7.4.6	< or =	100 mm			
Provide		12 mm dia rings @	100 mm c/c.		

This spacing is in accordance to IS 13920:1993 Cl. 7.4.6

CODE OF PRACTICE FOR DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES

Check for Size of Hoop Reinforcement **Refer IS 13920:1993 Cl. 7.4.8**

Ash= 0.18 Sh (Fck/Fy)x(Ag/Ak-1)

S	=	100.00	mm	
h	=	300.00	N/mm^2	(Spacing of long. bars+ effective cover) or 300 mm whichever is less
Fck	=	30.00	N/mm^2	**Cover 75 mm to main reinforcement**
Fy	=	415.00	N/mm^2	

Ag	=	1200.00	mm^2	Considering 1 mm Wide Pier
Ak	=	1099.00	mm^2	Considering 1 mm Wide Pier Effective
Hence Ash	=	35.87	mm^2	
Ash ProvideD	=	113.04	mm^2	Which is OK

d) As per IS IS 13920:1993 Cl. 7.4.6 < or = 100 mm

Provide 12 mm dia rings @ 100 mm c/c.

This spacing is in accordance to IS 13920:1993 Cl. 7.4.6

CODE OF PRACTICE FORDUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES

ABSTRACT

LONGITUDINAL REINFORCEMENT	**25**	**MM BARS**	**290**	**MM**	**However Adopt spacing as 250 mm**
TRANSVERSE REINFORCEMENT	**12mm dia rings @100mm c/c.**				

DESIGN OF PIER FOOTING SUBMERSIBLE BRIDGE

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

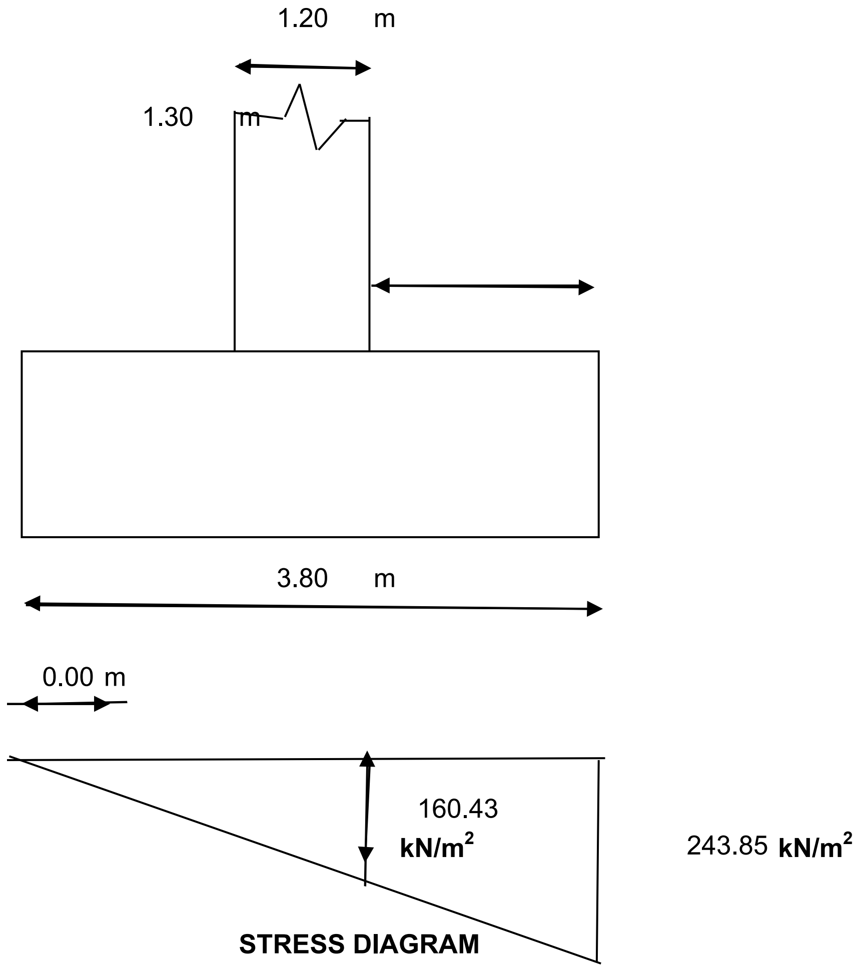
FOR WIND AT SERVICE CONDITION

Length of footing	l_f	15.60	m	
Width of Footing	l_b	3.80	m	
Width of Pier		1.20	m	
Vertical Load	P	13072.31	kN	
Longitudinal Moment	M_e	615.70	kN-m	
Transverse Moment	M_b	2917.78	kN-m	
Area in Tension = y x l_b		0.00	**m^2**	0.00 %
Maximum Pressure before Redistribution		243.85	**kN/m^2**	
Maximum Pressure After Redistribution = pxK		243.85	**kN/m^2**	
Maximum Stress at Edge of Pier		243.85	**kN/m^2**	
Distance From Face of Pier to the Edge		1.30	m	
Stress at the Edge of Pier		160.43	**kN/m^2**	
Average Stress on Cantilevered Area		202.14	**kN/m^2**	
Area of the Cantilever Portion		1.30	**m^2**	
Distance of Centroid of the Stress in Cantilever Portion		0.69	m	
Moment about the Face of Pier		**182.55**	**kN-m**	
CONCRETE GRADE		**M-25**		
FOR THIS GRADE σcbc		**10**	N/mm2	
m		**9.33**		
σst		**200**		
factor k		0.318		
j		0.894		
R		1.422		
Effective Depth Required		358	mm	
Adopt Total Depth		1500	mm	
Cover		50	mm	
Assume Bar Dia		25	mm	
Keeping A Cover Of 50 mm Effective Depth		1438	mm	
Adopt Effective Depth		1437.5	mm	
Steel Required Ast		710	mm^2	
Area Of One Bar		491	mm^2	
Spacing S		**691**	**mm**	

Provide Bars Of Dia And Spacing	25 mm	**Adopt spacing as 250 mm**
Area Of Distribution Steel		2000 mm^2
Dia Of Bar For Distribution Steel		20 mm
Area Of One Bar In Distribution Reinforcement		314 mm^2
Using The Bars Spacing Required		157 mm
Provide Bars Of Dia And Spacing	20 mm	150 mm
Provide Bars Of Dia And Spacing for Top Main Steel	12 mm	150 mm
Provide Bars Of Dia And Spacing for Top Distribution Steel	12 mm	150 mm

CHECK FOR SHEAR (As per IRC 21-1987 Cl. 304.7)

Critical Section is at a distance equal to effective depth from pier face		1437.5 mm
Section of Shear from end of pier		-0.14 m
Maximum Stress at Edge of Pier		243.85 **kN/m^2**
Stress at the Section for Shear Check		253.16 **kN/m^2**
Average Stress on Cantilevered Area		248.51 **kN/m^2**
Shear Force		-34.17 kN
V=V' + M/d tanB	(B=0) Hence V =V'	
Actual Shear Stress		-0.02 **N/mm^2**
Percentage Steel	100As/bd	0.05
Tc		0.23 **N/mm^2**
k=1		
Permissble Shear Stress = k Tc		0.23 **N/mm^2**
		< Actual Shear Stress hence Shear Reinforcement should be provided
Dia Of two Legged Stirrups		16 mm
Area Of One Bar In Distribution Reinforcement		201 mm^2
Using The Bars Spacing Required s= Asw ts d/V		-3382 mm
Provide Bars Of Dia And Spacing	**16 mm**	**Adopt spacing as 250 mm**



STRESS DIAGRAM

DESIGN OF PIER FOOTING

LIVE LOAD CALCULATION :-

[1] CLASS AA TRACKED VEHICLE :-

(a) Dispersion width along the span

According to clause 305.13 IRC- 21-2000

= Length of Contact + 2 (Wearing coat + depth of Slab)

= 3.6 + 2 (0.075 + 0.775)

= 5.3 M

(b) Dispersion width across the span

According to clause 305.13 IRC- 21-2000

be = K x (1 - x/Le) +bw

K = A Constant having the value depending upon the ratio
(L1/Le where.
be = the effective width of the slab on which the load acts.
Le = Effective Span
x = the distance of c.g. of concentrate load from the near support
bw = The breadth of concentration area of the load i.e. Dimension of the tyre or track contact area over the road surface

Heve ,

Le = 10.00 M & L1 = 7.00 M

$= \frac{L1}{Le} = \frac{7.00}{10.0} = 0.7$

Value of K = 2.4

bw = 0.85 + 2 x 0.075 = 1.0 M

$X = \frac{L}{2} = \frac{10}{2} = 5.0$ M

be = 2.4 x 4 (1 - 5/10) + 1

= 5.8 M

Impact factor is 13.75% as pere IRC Section-II, Clause - 211-3 (a) (i)

DISPERSION ACROSS SPAN (CLASS AA TRACKED VEHICAL

The tracked vehicle is placed at a distance of minimum clearence of 1-2 m from Kerb

Dispersion across span

= C/C distance between wheels

+ width from centre of wheel on clearence side

+ Least on other side or halp the dispersion of one wheel.

= 2.05 + 1.93 + Least of 2.715 OR 5.8/2

= 2.05 + 1.93 + 2.715

= 6.695

Impact factor = 1.1375

Total load with impact

= 70 x 1.1375

= 79.63 T

= Intensity of Load

$$= \frac{79.63}{5.30 \times 6.695} = 2.24 \text{ T/M}$$

Maximum Reaction

For Maximum reaction at support the Centre of gravity of the loads should be adjacent to one support should be adjacent to one support

Reaction R_A= 2.24x 3.00 x 1.50 /10.00

= 1.01 T

Reaction R_B= 2.24x 3.00 -1.01

= 5.71 T

DISPERSION ALONG SPAN (CLASS AA TRACKED VEHICLE

(a) Dispersion width along the span :-

tp = tc = 2 (tw + ts)

tp = width of dispersion parallel to span

tc = width of tyre contact area parallel to span

ts = Overall depth of slab

tw = Thickness of Wearing coat

Dispersion along the span

= 0.15 + 2 (0.075 + 0.775)

= 1.9 M

Dispersion between two wheel is overlapping hence restricted to 1-2 M

= Dispersion combined for two wheels

= C/c distance between two wheels + Longitudinal dispersion

= 1-2 + 1.9

= 3.1 M (along the span)

Impact factor = 1.1375

Total load with impact

= 70 x 1.1375

= 79.63 T

= Intensity of Load

$$= \frac{79.63}{1.90 \times 5.30} = 7.91 \text{ T/M}$$

Maximum Reaction

For Maximum reaction at support the Centre of gravity of the loads should be adjacent to one support should be adjacent to one support

Reaction R_A= 7.91x 3.00 x 1.50 /10.00

= 3.56 T

Reaction R_B= 7.91x 3.00 -3.56

= 20.17 T

DESIGN OF PIER CAP :-

D.L./ M Width along bridge

DL. Of Slab =	0.75 x	8.40 x.	2.4 =	15.12 T
D.L. of Wearing coat =	0.08 x	8.40 x.	2.4 =	1.51 T
			TOTAL	16.63 T
D.L. of Slab & Wearing coat on half of the pier	=			
		16.63 /	2 =	8.32 T
L.L. on Pier cap including impact along bridge				
	=	82.50 x	1.1375 =	93.84 T
(Refer Live Load Computation)				
Dispersion width across the span for 70 T TRACKED VEHTCLE	=	6.695 M		
(Refer Solid slab design page SS-16)				
Live Load u.d.l. on Pier Per M width	=	93.84 /	6.695 =	14.02 T
Total Load on Half = of pier along bridge	8.32 +	14.02	=	22.33 T Per M width
Effective depth of slab =90-2.5-2.5/2 =	71.25 cm			

Placement of the live load at effective depth from the support (taking support width 750 mm)

Eccentricity = 71.25 -75/2	=	33.75 cm	=	0.34 M
Bending Moment along the bridge =	22.33 x	0.34		7.54 T - M/M width
=	7.54 x	10.00 =		**75.4 kN-M/M width**

This moment is too small hence it will not/be the governing B.M.

Moment in pier cap		**75.40 kN-m**		
CONCRETE GRADE		**M30**		
FOR THIS GRADE σcbc		**10** N/mm2		
m		**9.33**		
σst		**200**		
factor k		0.318		
j		0.894		
R		1.422		
Effective Depth Required		230 mm		
Adopt Total Depth		1200 mm		
Cover		50 mm		
Assume Bar Dia		25 mm		
Keeping A Cover Of 50 mm Effective Depth		1138 mm		
Adopt Effective Depth		1137.5 mm		
Steel Required Ast		371 mm^2		
Area Of One Bar		491 mm^2		
Spacing S		1323 mm		
Provide Bars Of Dia And Spacing	25 mm	**100** mm	**Adopt spacing as 100 mm**	
Provide Bars Of Dia And Spacing for Top Main Steel	25 mm	100 mm		
Provide Bars Of Dia And Spacing for Bottom Steel	16 mm	100 mm		

PIER SECTION ACROSS BRIDGE

DEAD LOAD MOMENT PER METRE Width across bridge :-

Slab D.L.	0.975 x	15 x.	2.4 =	35.10 T
D.L. of Wearing coat =	0.075 x	12 x.	2.4 =	2.16 T
			TOTAL	**37.26** T
D.L. of Slab & Wearing coat on half of the pier	=			
		37.26 /	2 =	18.63 T/ M width

L.L on pier		=		64.69 T
Dispersion width along the span for				
70 T Tracked vehical	=	5.3 M		
L.L. . per M width on pier =		64.69 /	5.3 =	12.21 T/ M width
Total D.L. + L.L. on half of Pier across		18.63 +	12.21 =	30.84 T
bridge per M width				Per M width

The Live Load is with clearance from the Footpath and kerb. The cantilever portion of pier cap and width of footpath is 1500 mm
Hence There is no eccentricity.

Bending Moment across the bridge =

30.84 x 0 0.00 T - M/M width

Provide Minimum steel

Minimum Reinforcement calculation for Pier cap :-

As per clause 710.8.2, IRC- 78 - 2000, the thickness of pier cap shall be at least 200 mm However the thickness of Pier cap here is 1200 MM.

Grade of Concrete M 30

Minimum Shrinkage and Temperature reinforcement required as per Clause 305.10 IRC 21-2000 in any RC structure is 250 Sq mm per m in each direction. Allowable maximum spacing is 300 mm.

Shrinkage and Temperature reinforcement required = 250 x 1.2 = 300 mm^2

Provide 25 mm tor reiforcement @ 100 mm c/c (14 Nos.) in top along the pier cap
Provide 16 mm tor reiforcement @ 100 mm c/c (14 Nos.) in bottom along the pier cap

Area of Steel Provided at top
= (14x 491) = 6874 mm^2 > 300 mm^2 OK

Area of Steel Provided at bottom
= (14x 201) = 2814 mm^2 > 300 mm^2 OK

CHECK FOR SHEAR ALONG BRIDGE DIRECTION

V =		30.84 T		
Shear Force			308.40 kN	
V=V' + M/d tanB	(B=0) Hence V =V'			
Actual Shear Stress			0.27 **N/mm^2**	
Percentage Steel	100As/bd		0.25	
Tc			0.23 **N/mm^2**	
k=1				
Permissble Shear Stress = k Tc			0.23 **N/mm^2**	
			< Actual Shear Stress hence Shear Reinforcement should be provided	
Dia Of two Legged Stirrups			16 mm	
Area Of One Bar In Distribution Reinforcement			201 mm^2	
Using The Bars Spacing Required s= Asw ts d/V			296 mm	
Provide Bars Of Dia And Spacing		16 mm	100 mm	**Adopt spacing as 100 mm**

HOWEVER
Provide 16 mm tor 2 legged vertical stirrups @ 100 mm centre to centre along the pier cap
Provide 16 mm tor 2 legged horizontal stirrups @ 100 mm centre to centre along the pier cap

SHEAR CHECK ACROSS BRIDGE DIRECTION

V =		20.3 T	
Shear Force			203.00 kN
V=V' + M/d tanB	(B=0) Hence V =V'		

Actual Shear Stress		0.18	**N/mm^2**
Percentage Steel	100As/bd	0.25	
Tc		0.23	**N/mm^2**
k=1			
Permissble Shear Stress = k Tc		0.23	**N/mm^2**

> Actual Shear Stress hence No Shear Reinforcement is required.

HOWEVER
Provide 16 mm tor 2 legged vertical stirrups @ 100 mm centre to centre along the pier cap
Provide 16 mm tor 2 legged horizontal stirrups @ 100 mm centre to centre along the pier cap

CALCULATION OF LIVE LOAD REACTION FOR PIER SUBSTRUCTURE
FOR SIMPLY SUPPORTED SPANS OF A TWO LANE BRIDGE STRUCTURE

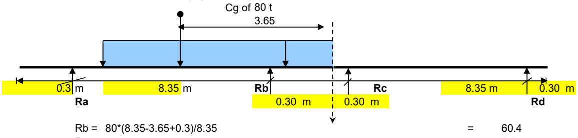
Centre line of pier w.r.t. the bearings :-

Rb = **0.3** m
Rc = **0.3** m

Reaction has been calculated for the following cases
1. One lane of class 70-R(W)
2. One lane of class - A
3. Two lane of class - A
4. Three lane of class - A
5. One lane of class 70-R(W) + One lane of class - A

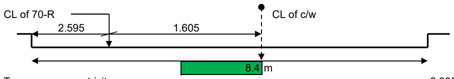
Condition A: **MAXIMUM LONGITUDINAL MOMENT CASE**

Case 1: **One lane of class 70-R(W)**



Rb = 80*(8.35-3.65+0.3)/8.35		=	60.4	t
Rc =		=	0.0	t
Ra=		=	19.6	t
Vert.Reaction=	60.4 + 0	=	60.4	t
Braking Force, B =	0.2*80	=	16.0	t
Dead load reaction on the pier , Rg		=	485.0	t
Value of "μ" =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	16.0	t

(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)

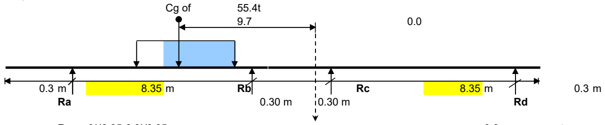


Transverse eccentricity		=	2.905	m
Transverse moment =	2.905*(60.4 + 0)	=	175.3	t.m
Long. moment =	60.4*0.3-0*0.3	=	18.1	t.m
Long. Eccentricity (for input)		=	0.300	m

span	load	cg
4.42	51	1.93
5.79	68	2.895
7.92	80	3.65
9.44	92	4.4
13.4	100	5.12

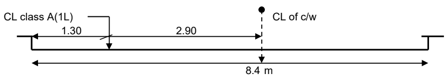
8.78

B) **One lane of class-A**



Rc = 0*(8.35-0.3)/8.35		=	0.0	t
Rb = 55.4*(8.35-9.7+0.3/2)/8.35		=	7.0	t
Ra=		=	48.4	t
Vert.Reaction =	48.4 + 7	=	55.4	t
Braking Force, B =	0.2*(0+55.4)	=	11.1	t
Dead load reaction on the pier , Rg		=	485.0	t
Value of "μ" =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	11.1	t

(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)



Transverse eccentricity		=	2.90	m
Transverse moment =	2.9*55.4	=	160.7	t.m
Long. moment =	7*0.3-0*0.3	=	2.1	t.m
Long. Eccentricity (for input)		=	0.038	m

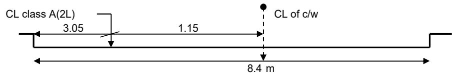
SPAN	LOAD	CG
5.5	29.6	1.73
8.5	36.4	2.99
11.5	43.2	4.33
14.5	50	5.71
24	50	5.71
8.78		

Case 3 : **Two lane of class-A**

Rc = 2*0		=	0.0	t
Rb = 2*7		=	13.9	t
Ra=		=	96.9	t
Vert.Reaction =	0 + 13.9	=	13.9	t
Braking Force(For single lane only)		=	11.1	t
Dead load reaction on the pier , Rg		=	485.0	t
Value of "μ" =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t

Design horizontal force is higher of either (B/2+T) or (B-T)		=	11.1	t

(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)

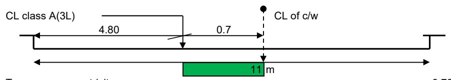


Transverse eccentricity		=	1.15	m
Transverse moment =	1.15*13.9	=	16.0	t.m
Long. moment =	13.9*0.3-0*0.3	=	4.2	t.m
Long. Eccentricity (for input)		=	0.300	m

Case 4 : **Three lane of class-A**

Rc = 90% of 3*0		=	0.0	t
Rb = 90% of 3*7		=	18.8	t
Ra=		=	61.1	t
Vert.Reaction =	0 + 18.8		18.8	
Braking Force, B =	(0.2)*55.4+0.05*55.4	=	13.9	t
(5% extra taken for third lane)				
Dead load reaction on the pier , Rg		=	485.0	t
Value of " μ " =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	13.9	t

(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)

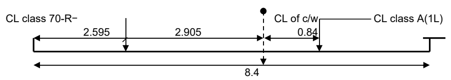


Transverse eccentricity		=	0.70	m
Transverse moment =	0.7*18.8	=	13.2	t.m
Long. moment =	18.8*0.3-0*0.3	=	5.6	t.m
Long. Eccentricity (for input)		=	0.300	m

Case 5 : **One lane of class-70R(W)+One lane of class-A**

Rc = 90% of(0+0)		=	0.0	t
Rb = 90% of(6.97+60.36)		=	60.6	t
Ra=		=	61.3	t
Braking Force =	16 + 5% of 55.4	=	18.8	t
(5% extra taken for class A)				
Dead load reaction on the pier , Rg		=	485.0	t
Value of " μ " =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	18.8	t

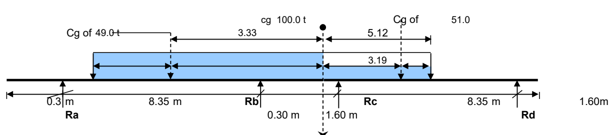
(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)



Transverse ecc.(class 70 R)		=	2.905	m
Transverse ecc.(class A)		=	-0.84	m
Trans. moment =	0.9*(60.4*2.905-55.4*-0.84)	=	115.9	
Net transverse ecc. (for input)		=	1.913	t.m
Long. moment =	60.6*0.3-0*0.3	=	18.2	t.m
Long. Eccentricity (for input)		=	0.300	m

Condition B: **MAXIMUM TRANSVERSE MOMENT / REACTION CASE**

CASE 1: **ONE LANE OF CLASS 70-R(W)**



first span		
SPAN	LOAD	CG
8.28	49	3.33
5.04	58	2.18
8.95		

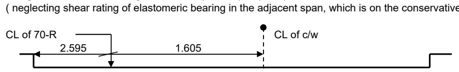
second span		
SPAN	LOAD	CG
3	80	3.65
4.52	92	4.4
8.48	100	5.12
24	100	5.12
8.95		

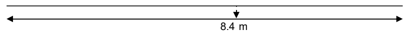
second span		
4.4	34	3.715
5.12	51	3.19
11.55		

first span		
3	17	0.87
4.52	29	1.75
8.48	41	2.56
24	49	3.53
8.95		

Rb = 49*(8.35 - 3.33 + 0.3)/8.35		=	31.22	t
Rc = 51*(8.35-3.19+1.6)/8.35		=	21.74	t
Ra=		=	27.3	t
Vert. Reaction =	31.2 + 21.7	=	53.0	t
Braking Force, B =	0.2*100	=	20.0	t
Dead load reaction on the pier , Rg		=	485.0	t
Value of " μ " =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	20.0	t

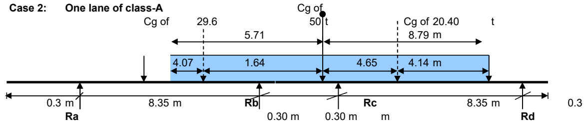
(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)





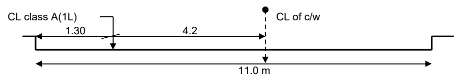
Transverse eccentricity		=	2.905	m
Transverse moment =	2.905*(31.2 + 21.7)	=	154.0	t.m
Long. moment =	31.22*0.3-21.74*0.3	=	2.8	t.m
Long. Eccentricity (for input)		=	0.054	m

Case 2: **One lane of class-A**



Rc = 20.4*(8.35-4.65+0.3)/8.35		=	9.77	t
Rb = 50*(8.35-1.64+0.3)/8.35		=	24.85	t
Ra=		=	4.8	t
Vert.Reaction =	9.77 + 24.85	=	34.62	t
Braking Force, B =	0.2*(50)	=	10.0	t
Dead load reaction on the pier , Rg		=	485.0	t
Value of " μ " =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	10.0	t

(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)

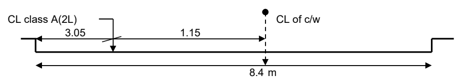


Transverse eccentricity		=	4.20	m
Transverse moment =	4.2*34.6	=	145.4	t.m
Long. moment =	24.8*0.3-9.8*0.3	=	4.5	t.m
Long. Eccentricity (for input)		=	0.131	m

Case 3 : **Two lane of class-A**

Rc = 2*9.8		=	19.5	t
Rb = 2*24.8		=	49.7	t
Ra=		=	9.5	t
Vert.Reaction =	19.5 + 49.7	=	69.2	t
Braking Force(For single lane only)		=	11.1	t
Dead load reaction on the pier , Rg		=	485.0	t
Value of " μ " =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	11.1	t

(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)

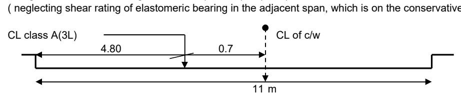


Transverse eccentricity		=	1.15	m
Transverse moment =	1.15*69.2	=	79.6	t.m
Long. moment =	49.7*0.3-19.5*0.3	=	9.0	t.m
Long. Eccentricity (for input)		=	0.131	m

Case 4 : **Three lane of class-A**

Rc = 90% of 3*9.8		=	26.4	t
Rb = 90% of 3*24.8		=	67.1	t
Ra=		=	12.8	t
Vert.Reaction =	26.4 + 67.1		93.5	
Braking Force, B =	(0.2)*55.4+0.05*55.4	=	13.9	t
(5% extra taken for third lane)				
Dead load reaction on the pier end , Rg		=	485.0	t
Value of " μ " =		=	0.00	
Horizontal force due to temperature, T μ*(Rg+Ra)		=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	13.9	t

(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)



Transverse eccentricity		=	0.70	m
Transverse moment =	0.7*93.5	=	65.4	t.m
Long. moment =	67.1*0.3-26.4*0.3	=	12.2	t.m
Long. Eccentricity (for input)		=	0.131	m

Case 5 : **One lane of class-70R(W)+One lane of class-A**

Rc = 90% of(9.77+21.74)	=	28.4	t
Rb = 90% of(24.85+31.22)	=	50.5	t
Ra=	=	29.5	t

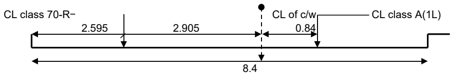
two span length	load	cg6.8 end	cg2.7 end
9	27.2	4.5	4.5
13.3	38.6	7.1	6.2
14.5	50	8.79	5.71
18.7	52.7	9.24	9.46
18.8	55.4	9.71	9.09
17.6	55.4	9.71	9.09

load	Span2load	cg 6.8	load	Span2 load	cg 6.8
27.2	13.6	1.5	55.4	27.2	4.5
38.6	20.4	4.14	52.7	27.2	4.5
50	20.4	4.14	50	20.4	4.14
52.7	27.2	4.5	38.6	20.4	4.14
55.4	27.2	4.5	27.2	13.6	1.5
			span2	8.78	

load 1	Cg 2.7 end	load 1	Cg 2.7 end
13.6	1.5	28.2	4.07
18.2	1.81	25.5	3.4
25.5	3.4	29.6	1.73
28.2	4.07	18.2	1.81

Braking Force =	16 + 5% of 55.4	=	18.8	t
(5% extra taken for class A)				
Dead load reaction on the pier , Rg		=	485.0	t
Value of " μ " =		=	0.00	
Horizontal force due to temperature, T	μ*(Rg+Ra)	=	0.0	t
Design horizontal force is higher of either (B/2+T) or (B-T)		=	18.8	t

(neglecting shear rating of elastomeric bearing in the adjacent span, which is on the conservative side)



Transverse ecc.(class 70 R)		=	2.905	m
Transverse ecc.(class A)		=	-0.84	m
Trans. moment =	0.9*(60.4*2.9-0*-0.8)	=	112.4	
Net transverse ecc. (for input)		=	1.426	t.m
Long. moment =	50.5*0.3-28.4*0.3	=	6.6	t.m
Long. Eccentricity (for input)		=	0.084	m

Summary of Loads

Max. Longitudinal Moment			Design horizontal force (t)	Transverse ecc. (m)	Longitudinal ecc. (m)
Max. vertical reaction (t)	Transverse moment (t.m)	Longitudinal moment (t.m)			
60.4	175.3	18.1	16.0	2.905	0.300
55.4	160.7	2.1	11.1	2.900	0.038
13.9	16.0	4.2	11.1	0.700	0.300
18.8	13.2	5.6	13.9	0.700	0.300
60.6	115.9	18.2	18.8	1.913	0.300

	Max.Transverse Moment			Design horizontal force (t)	Transverse ecc. (m)	Longitudinal ecc. (m)
Load case	Max. vertical reaction (t)	Transverse moment (t.m)	Longitudinal moment (t.m)			
1L class 70 - R	53.0	154.0	2.8	20.0	2.905	0.054
1L class - A	34.6	145.4	4.5	10.0	4.200	0.131
2L class - A	69.2	79.6	9.0	11.1	9.046	0.131
3L class - A	93.5	65.4	12.2	13.9	0.700	0.131
1L class 70 - R + 1L class - A	78.8	112.4	6.6	18.8	1.426	0.084

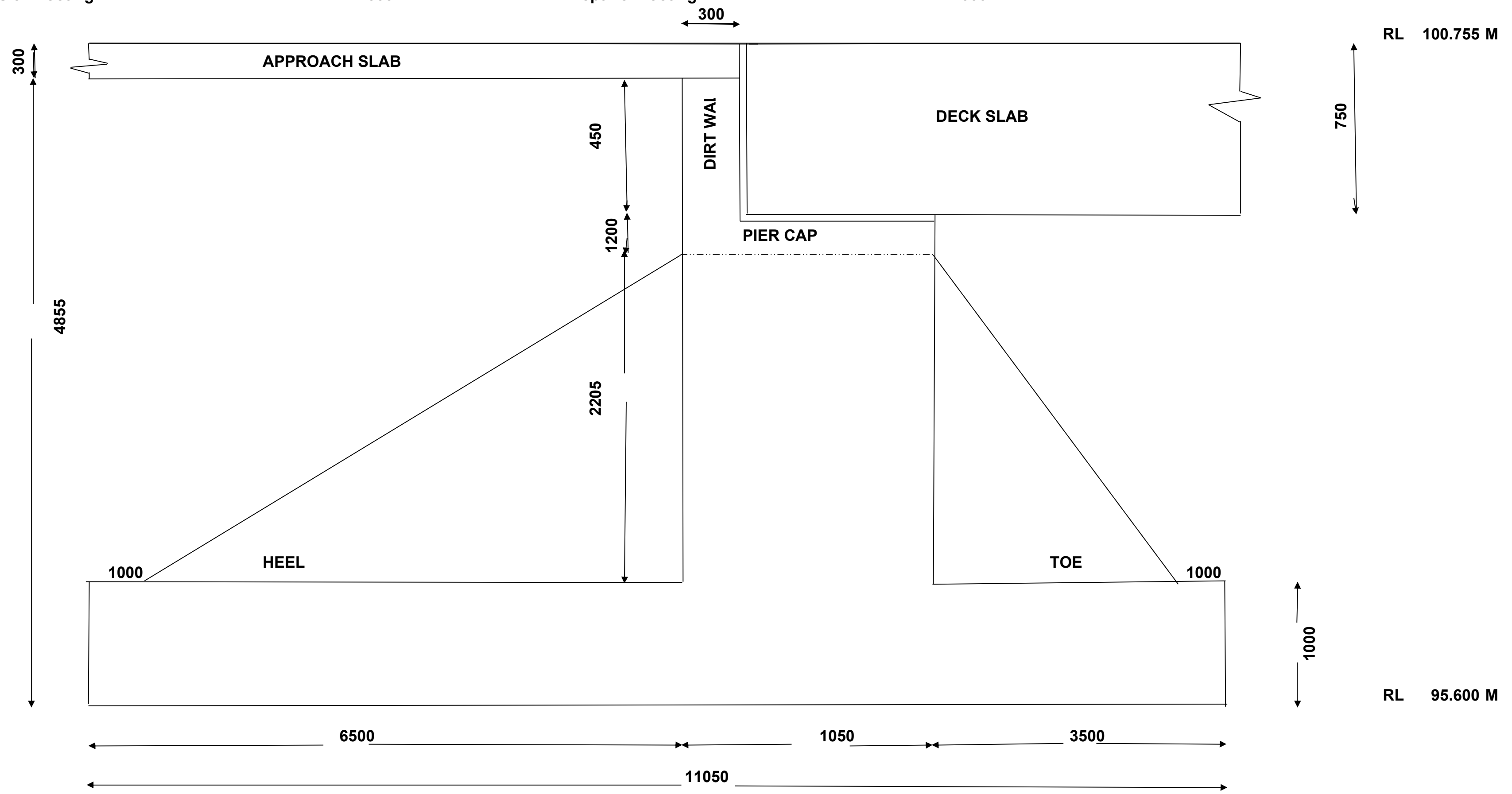
Vertical reaction due to braking has been neglected.

Maximum Reaction due Live Load including Impact	78.83	MT	=	788.27	KN
Maximum Longitudinal moment due to Live Load including Impact and Breaking Force	12.21	T-M	=	122.13	KNM
Maximum Transverse moment due to Live Load including Impact and Breaking Force	112.4	T-M	=	1123.94	KNM

Component	Chainage	NSL
Central Pier at Chainage	40	82.57
A1	-3.2	98.6
P1	7.6	
P2	18.4	
P3	29.2	
P4	40	82.57
P5	50.8	
P6	61.6	
P7	72.4	
A2	83.2	101

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

Deck Level	100.755 M	Length of Heel projection	6500 mm
Foundation Level	95.600 M	Length of Toe projection	3500 mm
Thickness of Deck Slab	750 mm	Width of Stem	1050 mm
Thickness of Approach Slab	300 mm	Thickness of Abutment Cap	1200 mm
Height below Approach Slab	4855 mm	Thickness of Dirt Wall	300 mm
Offsets on Footing	1000 mm	Depth of Footing	1000 mm



TYPICAL SECTION OF THE ABUTMENT TYPABUT-01

Design of ABUTMENT

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

(a) Data

Preliminary dimensions	: Assumed as in Fig. **TYPABUT-01**		
Superstructure	: RCC Slab Bridge	Total Width of Slab =	12.00 M
	overall length = 10.80 m		
Type of abutment	: Reinforced concrete		
Loading	: As for National Highway		
Back fill	: Gravel with angle of repose Φ =		35 °
	Unit weight of back fill, w =	18 kN/m3	
	Angle of internal friction of soil on wall, z =		17.5 °
Approach slab	: R.C. slab 300 mm thick, adequately reinforced		

Load from superstructure per running foot of abutment wall:

Dead load	=	802.01	kN/m
Live load	=	93.84	kN/m

(Refer Stability Analysis for sub structure. The above two values are obtained from the calculations for superstructure, and are taken to act over a width of 15 m).

Bearing : Tar Paper Bearings

(C) Self weight of abutment

Treating the section as composed of 6 elements as shown in Fig. 1the weight of each element and moment about the point O on the front toe are computed as in Table 1

(d) Longitudinal forces

(i) Force due to braking

Force due to 70 R wheeled vehicle =	0.2 x	1000 =	200 kN

This force acts at 1.2 m above the road level(Clause 214.3).

Force on one abutment wall =	200 /	2 =	100 kN
Horizontal force per m of wall =	100 /	12.00 =	8.34 kN/ m

(ii) Force due to temperature variation and shrinkage

Assuming moderate climate, variation in temperature is taken as + 17 oC as per Clause 218.5 of Bridge Code.

Coefficient of Thermal expansion =	1.17E-05 /^{o}C		
Strain due to temperature variation =	17 x	1.17E-05 =	1.99E-04
From Clause 220.3, strain due to concrete shrinkage =	2.00E-04		
Total strain due to temperature and shrinkage =	1.99E-04 +	2.00E-04 =	3.99E-04

Horizontal deformation of deck due to temperature and shrinkage affecting one abutment = 3.99E-04 x 10800 /2 = 2.15E+00 mm

Modulus of Elasticity Ec = $5000 \times fck^{1/2}$ = 31220.19 N/mm2

Horizontal Stress due to strain in longitudinal direction at bearing level = 3.99E-04 x 31220.19 = 12.45 N/mm2

Horizontal Force due to strain in longitudinal direction at bearing level (For 1 m width of Slab) = 1.25E+01 x 900 = 11208.36 N/m

= 11.21 kN/m

(iii) Vertical reaction due to braking

$$\text{Vertical reaction at one abutment} = \frac{200(1.2 + 0.975)}{11.10 \times 15} = 2.61 \text{ kN/m}$$

(d)Earth pressure

Active earth pressure $P = 0.5\, wh^2 K_a$

where K_a is obtained from Equation (3.5)

$K_a = \sec\Theta \sin(\Theta-\Phi)/[(\sin(\Theta+z)^{1/2} + \{\sin(\Phi+z)\sin(\Phi-\delta)/\sin(\Theta-\delta)\}^{1/2]}$

Where P= Total active pressure, acting at a height of 0.42 h inclined at z to the normal to the wall on the earth side

w = unit weight of earth fill

h = height of wall

Θ = Angle subtended by the earthside wall with thw horizontal on the earth side

Φ = Angle of internal friction of the earthfill

z = angle of friction of the earthside wall with the earth

δ = Inclination of earthfill surface with the horizontal

$\Theta = 90^0$ $\Phi = 35^0$

$z = 17.5^0$ $\delta = 0^0$

Substituting values in Equation (3.5), we get K_a = 0.496 Coefficient

Height of backfill below approach slab = 4.86 m

Active earth pressure = 0.5 x 18 x 4.86^2 x 0.496

= 105.23 kN/m

Height above base of centre of pressure = 0.42 x 4.86 = 2.04 m

Passive pressure in front of toe slab is neglected.

(e) Live load surcharge and approach slab

Equivalent height of earth for live load surcharge as per clause 714.4 is 1.20 m

Horizontal force due to L.L. surcharge =1.2 x 18 x 0.496 x 9.20 =		52.02	kN/m
Horizontal force due to approach slab = 0.3 x 24 x 0.496 x 9.20 =		17.34	kN/m
	Total	**69.36**	**kN/m**

The above two forces act at **2.4275 m above the base.**

Vertical load due to L.L. surcharge and approach slab

= (1.2 x 18 + 0.3 x 24) x 6.5 = 187.2 kN/m

(f) Weight of earth on heel slab

Vertical load = 18 x6.5x (4.855 - 1) = 34.7 kN/m

(g) Check for stability - overturning

The forces and their position are as shown in Fig. 1

The forces and moments about the point O at toe on the base are tabulated as in

Table 1 Two cases of lading condition are examined (i) Span loaded condition and (ii) Span unloaded condition.

Case (i) Span loaded condition

See Row 15 of Table 12.3

Overturning moment about toe =	469.25 kN-m		
Restoring moment about toe =	9783.99 kN-m		
Factor of safety against overturning =	9783.99 /	469.25 =	**20.85**
Location of Resultant from O			**> 1.5 Hence Safe**

X_0 = (M_V - M_H) / V= (1740.9 - 623.1) / 691.4 = 1.62 m

=(9783.986 - 469.25) / 1904.726) = 4.89 m

Eccentricity of resultant

e_{max} = B/6 =	11.05 /6 =	1.84 m	
e = (B/2 -$X_{0)}$ = 0.78 m < 0.80 m	5.53 -	4.89 =	0.64 m
		<	1.84 m

Case (ii) Span unloaded condition

See Row 11 of Table 12.3

Overturning moment about toe =	432.47 kN-m		
Restoring moment about toe =	9410.21 kN-m		
Factor of safety against overturning =	9410.21 /	432.47 =	**21.76**
Location of Resultant from O			**> 1.5 Hence Safe**

X_0 = (M_V - M_H) / V=

=(9410.214 - 432.47) / 1808.272) = 4.96 m

(h)Check for stresses at base

For Span loaded condition

Total downward forces = 1904.73 kN

1904.73 6 x 0.78

Extreme stresses at base =

Maximum Stress = 1904.726/(11.05x1)(1 +(6x0.64/11.05)) = 232.28 kN/m2

Minimum Stress = 1904.726/(11.05x1)(1 -(6x0.64/11.05)) = 112.48 kN/m2

Table 1 Forces and Moments About Base for Abutment.

Sl. No.	Details	Force, kN		Moment about O, kn-m		
		V	H	Arm m	Mv	M_H
1.	D.L. from superstructure	802.01	-	3.88	3111.810	-
2.	Horizontal force due to temperatre and shrinkage	0	11.21	4.41	-	49.429
3.	Active earth pressure	0	105.23	2.04	-	214.669
4.	Horizontal force due to L.L surcharge and approach slab	0	69.36	2.4275	-	168.371
5.	Vertical load due to L.L. surcharge and approach slab	187.20	-	7.8	1460.16	-
6.	Self weight - part 1 11.05x1x 24 =	265.20	-	5.525	1465.23	-
7.	Self weight - part 2 2.205x1.05x 24 =	55.57	-	4.03	223.9471	-
8.	Self weight - part 3 1.2x1.05x 24 =	30.24	-	1.68	50.8032	-
9.	Self weight - part 4 0.3x0.45x 24 =	3.24	-	2.05	6.642	-
9.	Self weight - part 5 Triangular River Side 1/2x3x2.655x24=	95.58	-	2.50	238.95	-
9.	Self weight - part 5 Triangular Earth Fill Side 1/2x6x2.855x24=	191.16	-	6.55	1252.098	-
10.	Weight of earth on heel slab part 1 Rectangular Portion 0.5 x 3.855 x 18=	34.7	-	10.8	374.76	-
10.	Weight of earth on heel slab part 2 Triangular Portion 1/2x6x3.855x18=	143.37	-	8.55	1225.814	-
11.	**Items 1 to 10**	**1808.27**			**9410.21**	**432.47**

	(Span unloaded condition)					
12.	L.L. from Superstructure Class 70 R wheeled vehicle	93.84	-	3.875	363.6348	-
13.	Vertical force due to braking	2.61	-	3.88	10.137	-
14.	Horizontal force due to braking	0.00	8.34	4.41		36.7794
15.	**Items 11 to 14** **(Span loaded condition)**	**1904.73**	**194.14**	**-**	**9783.99**	**469.25**

NET LONGITUDINAL MOMENT 9783.99 - 469.25 = 9314.74

Maximum pressure = 232.28 kN/m2 < 250.00 kN/m2 permissible HENCE OK.

Minimum pressure = 112.48 kN/m2 >0 (No tension) HENCE OK.

(i) Check for sliding

See Row 15 of Table 1

Sliding force = 194.14 kN

Force resisting sliding = 0.6 x 1904.73 = 1142.84 kN

Factor of Safety against sliding = 1142.84 / 194.14 = **5.89**

(j) Summary

> 1.5 Hence Safe

The assumed section of the abutment is adequate.

DESIGN OF ABUTMENT FOOTING

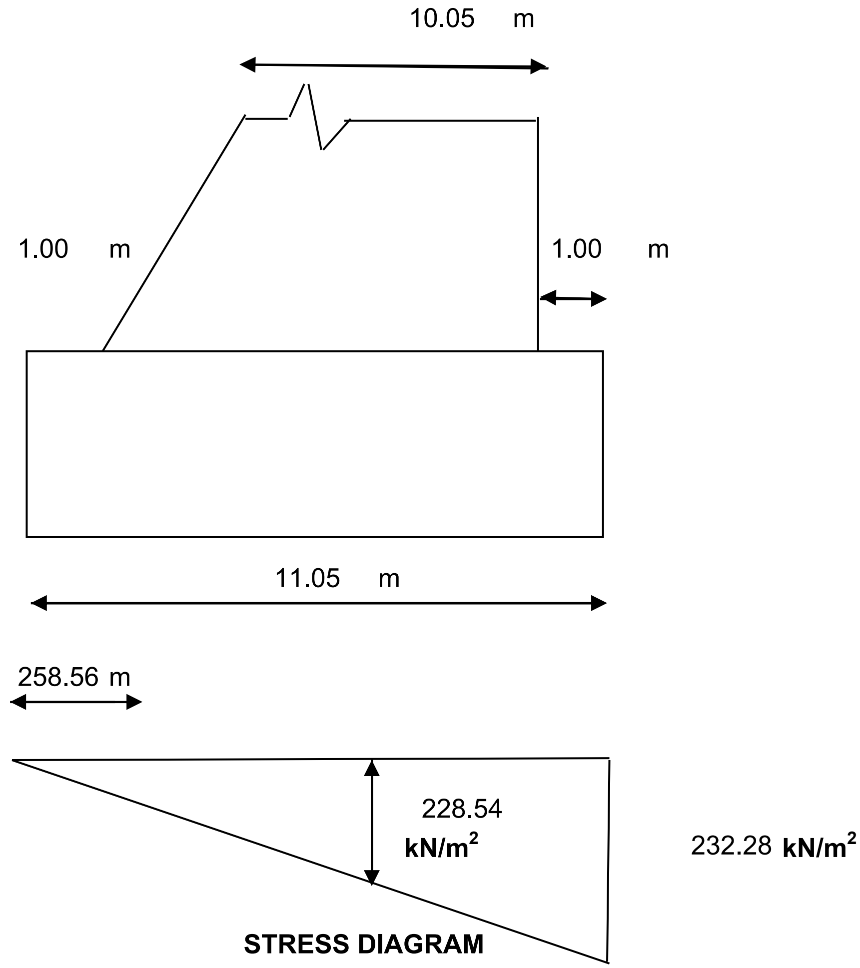
Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

REDISTRIBUTION OF PRESSURE

FOR WIND AT SERVICE CONDITION

Length of footing	l_f	15.20	m		
Width of Footing	l_b	11.05	m		
Width of Abutment just above footing		9.05	m		
Vertical Load	P	1904.73	kN		
Longitudinal Moment	M_e	9314.74	kN-m		
Transverse Moment	M_b	0.00	kN-m		
Area in Tension = y x l_b			0.00	**m^2**	0.00 %
Maximum Pressure before Redistribution			232.28	**kN/m^2**	
Maximum Pressure After Redistribution = pxK			232.28	**kN/m^2**	
Maximum Stress at Edge of Pier			232.28	**kN/m^2**	
Distance From Face of Pier to the Edge			1.00	m	
Stress at the Edge of Pier			211.26	**kN/m^2**	
Average Stress on Cantilevered Area			221.77	**kN/m^2**	
Area of the Cantilever Portion			1.00	**m^2**	
Distance of Centroid of the Stress in Cantilever Portion			0.51	m	
Moment about the Face of Pier			**112.64**	**kN-m**	
CONCRETE GRADE			**M-25**		
FOR THIS GRADE σcbc			**10**	N/mm2	
m			**9.33**		
σst			**200**		
factor k			0.318		
j			0.894		
R			1.422		
Effective Depth Required			281	mm	
Adopt Total Depth			1000	mm	
Cover			50	mm	
Assume Bar Dia			16	mm	
Keeping A Cover Of 50 mm Effective Depth			942	mm	
Adopt Effective Depth			942	mm	
Steel Required Ast			669	mm^2	
Area Of One Bar			201	mm^2	

Spacing S			300 mm	
Provide Bars Of Dia And Spacing		16 mm	**150** mm	**Adopt spacing as 150 mm**
Area Of Distribution Steel			1884 mm^2	
Dia Of Bar For Distribution Steel			20 mm	
Area Of One Bar In Distribution Reinforcement			314 mm^2	
Using The Bars Spacing Required			167 mm	
Provide Bars Of Dia And Spacing		16 mm	160 mm	**Adopt spacing as 150 mm**
Provide Bars Of Dia And Spacing for Top Main Steel		12 mm	150 mm	
Provide Bars Of Dia And Spacing for Top Distribution Steel		12 mm	150 mm	
CHECK FOR SHEAR	(As per IRC 21-1987 Cl. 304.7)			
Critical Section is at a distance equal to effective depth from pier face			942 mm	
Section of Shear from end of pier			0.06 m	
Maximum Stress at Edge of Pier			232.28 **kN/m^2**	
Stress at the Section for Shear Check			228.54 **kN/m^2**	
Average Stress on Cantilevered Area			230.41 **kN/m^2**	
Shear Force			13.36 kN	
V=V' + M/d tanB	(B=0) Hence V =V'			
Actual Shear Stress			0.01 **N/mm^2**	
Percentage Steel	100As/bd		0.14	
Tc			0.23 **N/mm^2**	
k=1				
Permissble Shear Stress = k Tc			0.23 **N/mm^2**	
			< Actual Shear Stress hence Shear Reinforcement should be provided	
Dia Of two Legged Stirrups			16 mm	
Area Of One Bar In Distribution Reinforcement			201 mm^2	
Using The Bars Spacing Required s= Asw ts d/V			5666 mm	
Provide Bars Of Dia And Spacing		16 mm	150 mm	**Adopt spacing as 150 mm**



DESIGN OF ABUTMENT FOOTING

REINFORCEMENT CALCULATION IN ABUTMENT SUBMERSIBLE BRIDGE

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

Minimum Shrinkage and Temperature reinforcement required as per Clause 305.10 IRC 21-2000
in any RC structure is 250 Sq mm per m in each direction. Allowable maximum spacing is 300 mm.

Shrinkage and Temperature reinforcement required per metre =			250 mm^2			
Area Of One Bar	12 mm dia		113 mm^2			
Spacing S			452 mm			
Provide Bars Of Dia And Spacing	12 mm		**125** mm			
Provide Bars Of Dia And Spacing	12 mm		125 mm			
HORIZONTAL SHRINKAGE &TEMPERATURE REINFORCEMENT	**12**	**MM BARS**	**125**	**MM**	**In Vertical direction on all FOUR faces**	
VERTICAL SHRINKAGE &TEMPERATURE REINFORCEMENT	**12**	**MM BARS**	**125**	**MM**	**In Lateral direction on all FOUR faces**	

DESIGN OF Abutment CAP SUBMERSIBLE BRIDGE

Name Of Work :- Construction Of High Level Bridge on Kelwara Kumbhalgarh Road Over Kelwara Lake

DESIGN OF Abutment CAP :-

D.L./ M Width along bridge				
DL. Of Slab =	0.975 x	15 x.	2.4 =	35.10 T
D.L. of Wearing coat =	0.075 x	12 x.	2.4 =	2.16 T
			TOTAL	**37.26 T**
D.L. of Slab & Wearing coat on half of the Abutment	=			
		37.26 /	2 =	18.63 T
L.L. on Abutment cap including impact along bridge				
	=	82.50 x	1.1375 =	93.84 T
(Refer Live Load Computation)				
Dispersion width across the span for				
70 T TRACKED VEHTCLE	=	6.695 M		
(Refer Solid slab design page SS-16)				
Live Load u.d.l. on Abutment	=	93.84 /	6.695 =	14.02 T
Per M width				
Total Load on Half =	18.63 +	14.02	=	32.65 T
of Abutment along bridge				Per M width
Effective depth of slab =90-2.5-2.5/2 =	86.25 cm			
Placement of the live load at effective depth from the support (taking support width 750 mm)				
Eccentricity = 71.25 -75/2	=	33.75 cm	=	0.34 M
Bending Moment along the bridge =				
	32.65 x	0.34		11.02 T - M/M width
=				
	11.02 x	10.00 =	**110.2 kN-M/M width**	
This moment is too small hence it will not/be the governing B.M.				

Moment in Abutment cap		**110.20**	**kN-m**	
CONCRETE GRADE		**M30**		
FOR THIS GRADE σcbc		**10**	N/mm2	
m		**9.33**		
σst		**200**		
factor k		0.318		
j		0.894		
R		1.422		
Effective Depth Required		278	mm	
Adopt Total Depth		1200	mm	
Cover		50	mm	
Assume Bar Dia		25	mm	
Keeping A Cover Of 50 mm Effective Depth		1138	mm	
Adopt Effective Depth		1137.5	mm	
Steel Required Ast		542	mm^2	
Area Of One Bar		491	mm^2	
Spacing S		905	mm	
Provide Bars Of Dia And Spacing	25 mm	**100**	mm	**Adopt spacing as 100 mm**
Provide Bars Of Dia And Spacing for Top Main Steel	25 mm	100	mm	
Provide Bars Of Dia And Spacing for Bottom Steel	16 mm	100	mm	

Abutment SECTION ACROSS BRIDGE

DEAD LOAD MOMENT PER METRE Width across bridge :-				
Slab D.L.	0.975 x	15 x.	2.4 =	35.10 T
D.L. of Wearing coat =	0.075 x	12 x.	2.4 =	2.16 T
			TOTAL	**37.26** T

D.L. of Slab & Wearing coat on half of the Abutment		=			
		37.26 /		2 =	18.63 T/ M width
L.L on Abutment		=			64.69 T
Dispersion width along the span for 70 T Tracked vehical	=	5.3 M			
L.L. . per M width on Abutment =		64.69 /		5.3 =	12.21 T/ M width
Total D.L. + L.L. on half of Abutment across bridge per M width		18.63 +	12.21	=	30.84 T Per M width

The Live Load is with clearance from the Footpath and kerb. The cantilever portion of Abutment cap and width of footpath is 1500 mm
Hence There is no eccentricity.

Bending Moment across the bridge =

30.84 x 0 0.00 T - M/M width

Provide Minimum steel

Minimum Reinforcement calculation for Abutment cap :-

As per clause 710.8.2, IRC- 78 - 2000, the thickness of Abutment cap shall be at least 200 mm However the thickness of Abutment cap here is 1200 MM.

Grade of Concrete M 30

Minimum Shrinkage and Temperature reinforcement required as per Clause 305.10 IRC 21-2000 in any RC structure is 250 Sq mm per m in each direction. Allowable maximum spacing is 300 mm.

Shrinkage and Temperature reinforcement required = 250 x 1.2 = 300 mm^2

Provide 25 mm tor reiforcement @ 100 mm c/c (14 Nos.) in top along the Abutment cap
Provide 16 mm tor reiforcement @ 100 mm c/c (14 Nos.) in bottom along the Abutment cap

Area of Steel Provided at top = (14x 491)	=	6874 mm^2	> 300 mm^2	OK
Area of Steel Provided at bottom = (14x 201)	=	2814 mm^2	> 300 mm^2	OK

CHECK FOR SHEAR ALONG BRIDGE DIRECTION

V =		30.84 T	
Shear Force		308.40 kN	
V=V' + M/d tanB	(B=0) Hence V =V'		
Actual Shear Stress		0.27 **N/mm^2**	
Percentage Steel	100As/bd	0.25	
Tc		0.23 **N/mm^2**	
k=1			
Permissble Shear Stress = k Tc		0.23 **N/mm^2**	
		< Actual Shear Stress hence Shear Reinforcement should be provided	
Dia Of two Legged Stirrups		16 mm	
Area Of One Bar In Distribution Reinforcement		201 mm^2	
Using The Bars Spacing Required s= Asw ts d/V		296 mm	
Provide Bars Of Dia And Spacing	16 mm	100 mm	**Adopt spacing as 100 mm**

HOWEVER

Provide 16 mm tor 2 legged vertical stirrups @ 100 mm centre to centre along the Abutment cap
Provide 16 mm tor 2 legged horizontal stirrups @ 100 mm centre to centre along the Abutment cap

SHEAR CHECK ACROSS BRIDGE DIRECTION

V = 20.3 T

Shear Force		203.00	kN
V=V' + M/d tanB	(B=0) Hence V =V'		
Actual Shear Stress		0.18	**N/mm^2**
Percentage Steel	100As/bd	0.25	
Tc		0.23	**N/mm^2**
k=1			
Permissble Shear Stress = k Tc		0.23	**N/mm^2**

> Actual Shear Stress hence No Shear Reinforcement is required.

HOWEVER

Provide 16 mm tor 2 legged vertical stirrups @ 100 mm centre to centre along the Abutment cap

Provide 16 mm tor 2 legged horizontal stirrups @ 100 mm centre to centre along the Abutment cap

DESIGN OF DIRT WALL AS COLUMN WITH BENDING

AXIAL LOAD ON THE DIRT WALL	**31.60**	KN		
ASSUME WIDTH OF DIRT WALL	1000	MM	EMIN/B	0.00
ASSUME DEPTH OF DIRT WALL	300	MM	EMIN/D	0.01
MOMENT TRANSFERRED TO DIRT WALL	12.80	KN-M		
FACTORED AXIAL LOAD	**47.40**	KN		
FACTORED MOMENT	19.20	KN-M		
DIA OF LONGITUDINAL REINFORCEMENT	**10**	MM		
CLEAR COVER	40	MM		
d'	45	MM		
d'/D	0.15			
ADOPT d'/D	0.15			
PU/FCKBD	0.01			
$MU/FCKBD^2$	0.01			
REINFORCEMENT EQUALLY DISTRIDUTED ON **TWO** SIDES				
USING CHART NO- OF RCC DESIGN AIDS	**33**		**CONC GRADE M-30**	
P/FCK	0.01			
P	0.3		> Minimum Steel 0.2% Hence OK	
AS	900	SQ MM		
TOTAL NUMBER OF BARS REQUIRED	12			
NUMBER OF BARS ON EACH SIDE	6			
SPACING	**200**	MM		

Alternate design Considering dirt wall as cantilever

B.M. = 12.80 KN-M

$$\text{deff reqd.} = \frac{12.80 \times 10^6}{1000 \times 0.972} = 118.7 \text{ mm}$$

$$\text{dpro} = 300 - 50 - \frac{16}{2} = 245 \text{ mm} > 118.7 \text{ mm (OK)}$$

$$\text{Ast} = \frac{12.80 \times 10^6}{200 \times 0.917 \times 245} = 284.87 \text{ mm}^2$$

This steel is to be provided on back i.e. approach slab side

Provide Vertical steel as follows

On River side 10mm bars @ 150 mm c/c = 524 mm^2

On Approach Slab side 10mm bars @ 150 Mm c/c = 524 mm^2

Minimum steel required in Horizontal direction

= 0.002 x 1000 x 250

= 500 mm^2

i.e. 250 mm^2 on each face

provide 10 @ 250 mm c/c = 314 mm^2

<u>ABSTRACT</u>

VERTICAL REINFORCEMENT IN SHAPE OF STIRRUPS on both faces

DIA	**10**	**mm**
SPACING	**150**	**mm**

HORIZONTAL REINFORCEMENT BAR DIA on both faces

DIA	**10**	**mm**
SPACING	**250**	**mm**

Design of Dirt Wall

Dirt wall is subjected to

(1) Live load

(2) Live load surcharge

(3) Braking force

(3) Earth Pressure

1) Consider 70 T tracked vehicle case is governing & 14 T Axle over dirt wall, Dispersion width at top of DIRT WALL

= 2.90 + (1.2 + 0.305) + 0.83

= 2.90 + 1.53 + 0.825

= 5.255 M

$\frac{\text{Live Load}}{\text{M Length}} = \frac{14}{5.255} = 2.66$ T/M

2) Self wt. of dirt wall

= 0.6 x 0.3 x 2.4

= 0.495 T/M

Say 0.5 T/M

Total direct loads = 2.66 + 0.5 = 3.16 T/M = **31.6 kN**

Here considering that only 70% of Braking force will be on dirt wall & the rest of braking force will be on soil.

= Braking force/mt. = $\frac{0.2 \times 14 \times 1}{5.255}$ = 0.53 T

= B.M. due to Braking force

= 0.53 x (1.2 + [0.83])

= 1.07 T-M

Intensity of Earth Pressure at Deck Level

$= 0.224 \times 1.8 \times 1.2$

$= 0.483$ T/M^2

Intensity of Earth Pressure at top of Abutment Ca =

$= 0.224 \times 1.8 \times (1.2 + 0.825)$

$= 0.816$ T/M^2

B.M. due to Earth Pressure & Live Load
Surcharge/M width

$$= \frac{1}{2} = (0.816 - 0.483) \times 0.825 \times 0.42 \times 0.875$$

$$+ 0.483 \times 0.825 \times \frac{0.528}{2}$$

$= 0.048 + 0.164$

$= 0.21$ T-M

Total BM at top of DIRT WALL

$= 1.07 + 0.21$

$= 1.28$ T-M $=$ **12.8 kN-m**

Direct Stress $= \dfrac{3.16 \times 10^{3}}{30 \times 100}$

$= 1.05$ Kg./Cm^2

Bending Stres: $= \dfrac{1.28 \times 10^{3}}{\frac{1}{6} \times 100 \times 30^{2}}$

$= 0.09$ Kg./Cm^2

For M 30 Grade,

Permissible Bending Stress = 67 Kg./Cm^2

Permissible Direct Compressive Stress = 50 Kg./Cm^2

$$= \frac{1.05}{50} + \frac{0.09}{67} \leq 1$$

$$= 0.021 + 0.001 \leq 1$$

$$= 0.022 \leq 1$$ **HENCE OK.**

DEAD LOAD CALCULATION :-

DEPTH OF DECK SLAB =	925 mm			
DEPTH OF WEARING COAT =	75 mm			
DIA OF MAIN BAR =	25 mm			
Clear cover =	25 mm			
Effective depth of slab deffective =	925 -	25 -	25 /2 =	887.5 mm
Effective SpanLeffective =	10 m			

DESIGN DEAD LOAD :-

(1) Weight / Sqm of Slab	0.925 x	2.4 =	2.22 T/ Sqm
(2) Weight / Sqm of wearing coat	0.075 x	2.4 =	0.18 T/ Sqm
Total DL			**2.4** T/ Sqm
DEAD LOAD BENDING MOMENT		2.4x10x10/8 =	30.00 T-M

LIVE LOAD CALCULATION :-

[1] CLASS AA TRACKED VEHICLE :-

(a) Dispersion width along the span

= Length of Contact + 2 (Wearing coat + depth of Slab)

= 3.6+2(0.075+0.925)= = **5.60** m

(b) Dispersion width across the span

be = K x (1 - x/Le) + Bw

K = A Constant having the value depending upon the ratio (be/Le) where ---

be = the effective width of the slab on which the load acts.

Le = Effective Span

x = the distance of c.g. of concentrate load from the near support

bw = The breadth of concentration area of the load i.e. Dimension of the tyre or track contact area over the road surface

Here ,

be = 7.50 m

Le = 10.00 m

be/Le = 0.75

Value of K = 2.4

Bw = 0.85 + (2 x 0.075) = 1.00 m

x = Le/2 = 10.00 /2 = 5.00 m

be = 2.20 m

Impact factor is 13.75% as pere IRC Section-II, Clause - 211-3 (a) (i)

DISPERSION ACROSS SPAN (CLASS AA TRACKED VEHICAL)

The tracked vehicle is placed at a distance of minimum clearence of 1.2 m from Kerb

Dispersion across span = C/C distance between wheels + width from centre of wheel on clearence side

+ Least on other side or half the dispersion of one wheel.

= 2.05 + 1.93 + **Least of 3.825 Or 5.6/2** 5.75

= 2.05 + 1.93 + 2.8 = 6.78 M

Impact factor = 1.1375

Total load with impact = 70 x 1.1375

= 79.63 T

Intensity of Load = 79.63 /(2.20 x 6.78) =

= 5.34 T

DISPERSION ALONG SPAN (CLASS AA TRACKED VEHICLE

Maximum Bending Moment due to Live load , at centre

$$= 5.34 \times \frac{5.6}{2} \left(10.00 - \frac{5.6}{5}\right)$$

= **132.77** T - M

Class AA wheeled vehicle :-

For Maximum B.M. at Centre of the span, the Centre of gravity of the loads and the centre of the span should coincide

(a) Dispersion width along the span :-

tp = tc = 2 (tw + ts)

tp = width of dispersion parallel to span

tc = width of tyre contact area parallel to span

ts = Overall depth of slab

tw = Thickness of Wearing coat

Dispersion along the span

= 0.15 + 2 (0.075 + 0.75) = 1.8 m

Dispersion between two wheel is overlapping hence restricted to 1.2 M
= Dispersion combined for two wheels

= C/c distance between two wheels + Longitudinal dispersion

= 1.2 + 1.8

= 3.0 m (along the span)

DISPERSION ALONG SPAN (CLASS AA WHEELED VEHICLE)

(B) Dispersion width across the span :-

be = k x (1 - X/L) + w

Le = 10.0 M & L1 = 7.5 M

= Value of K = 2.4

4.8

X = L/2 = 10/2 = 5.00 M

Bw = 0.30 + 2 (0.075) = 0.45 M

be = 2.4 x 5 x (1 - 5.00/10.00) + 0.45 = 6.45 M (For one Wheel)

DISPERSION ACROSS THE SPAN (CLASS AA WHEELED HEVHICLE)

When the wheel is placed at a distnace of minimum clearance of 1-2 M from Kerb,

Combined effective width

= c/c distance between wheels
+ 1/2 of the dispersion of one wheel
+ least of available width from centre of wheel on clearance side or half the dispersion of one wheel

$$= 2.2 + \frac{6.45}{2} + \text{Lesser of } 1.655 \ \& \ \frac{6.45}{2}$$

= 2.2 + 3.225 + 1.655

= **7.08** m

According to clause 211.3 (a) (ii) section-III, IRC 6- 1966

Impact factor = 25% = 1.25

= Total load of tracks with impact

= 20 x 1.25

= 25 T

Intensity = $\frac{\text{Load}}{\text{dispersion along x across the span}}$

$= \frac{25 \times 2}{3.00 \times 7.08}$

= 2.35 T/M

DISPERSION ACROSS THE SPAN (CLASS AA WHEELED HEVHICLE)

Maximum B.M. due to Live load at centre

$= 2.35 \times \frac{7.08}{2} \left(10.00 - \frac{7.08}{5}\right)$

= **71.41** T - M

$= 2.35 \times \frac{3}{2} \left(5 - \frac{3}{2}\right)$

=

Here from bending moment view point class AA tracked vehical is governing

Hence Maximum Bending Moment due to Live load = 15.527 T - M **132.77** T - M

Total B.M = B.M due to Dead load + BM. Due to Live load

= **30.00** + 132.77

= 162.77 T-M