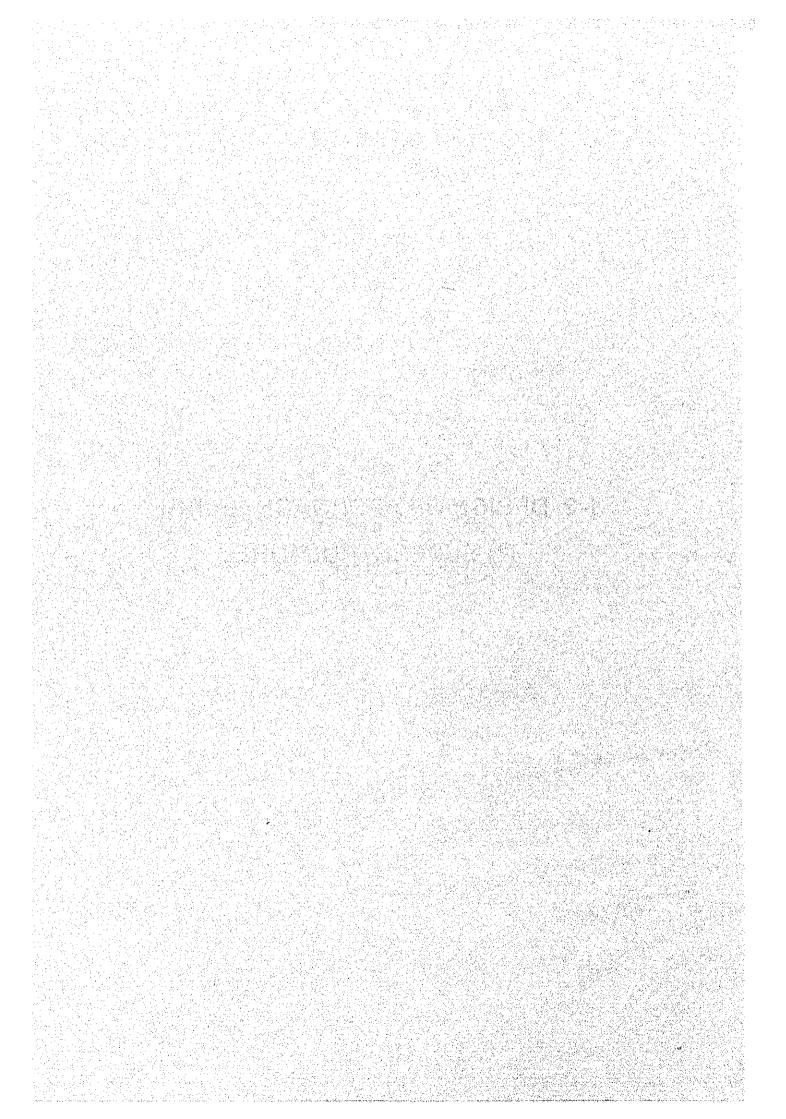
1-2. DESIGN OF APPROACH BRIDGE (1) SUPERSTRUCTURE



CONSTRUCTION OF THE BRIDGE OVER THE RIVER RUPSA (PHASE-2) (DESIGN CALCULATION OF SUPERSTRUCTURE OF APPROACH BRIDGE)

CENEDAL MOUSE				DATE: 2	
GENERAL INPUT			STRESS INPUT		
Total Length of one Girder		29950 mm,	Put 1.0 for Post-Tensioned or 2.0 for Pre-Tensioned	1	.0
Distance of center of Bearing from end of Girder		450			
Live Load		450 mm, IS 20	28 days Concrete Strength for :		
Total no. of Lane		4 Lane,	PC Girder		35 N/mm
Aggregate Size of Chips		19.05 mm,	Deck Slab, Sidewalk, Railing At time of Transfer for:		30 N /mm
Mention Deck type to be used		2	PC Girder		0 N/mm
Total no, of girder below deck		7	Ultimate strength of Prestressing Tendon		0 N/mm
Put 1.0 for working stress design o	r		Put 1.0 for low relaxation wire or strand, 2.0 for stress re		
2.0 for load factor design.		2	ed wire or strand, 3.0 for smooth high strength bar or		
Put 1.0 for non-composite or 2.0 fo			for deformed high strength bar	· ·	.0
composite wearing course Percentage of Jack loss (assumed	,	2	Put 1.0 for wire, 2.0 for strand, 3.0 for bar		.0
Total load due temp. (-ve for compn. +		3 % 200 Kn	Modulus of Elasticity of Prestressing Reinforcement		3 N/mm
Total load due to utility pipe (equally dis		4.0 Kn/m	Yield strength of Non-Prestressing Rebar Put 1 for plain bar or 2 for Deformed bar	, i	4 N/mm
Unit Wt. of concrete for prestressed	and the second of the second o	23,563 kN/m ³	Quantity limiting distribution of flexural reinforcement, z	22766 /	2 8 kN/m
Unit Wt. of concrete for non prestre		23.563 kN/m ³		22100.	IO KIWIJI
Unit Wt. of Wearing Course		23.563 kN/m ³		of the later	
					2 J
CLEAR COVERS					41.15
arapet	40 mm,	Curb			
Pedestal of Parapet	40 mm,	Top of Deck Slab	50 mm.		14, 11, 1
Top of Sidewalk	50 mm,	Bottom of Deck Slab	40 mm,		6 i
Bottom of Sidewalk	50 mm,	Ducts	50 mm,		
INPUT RELATING LOSSES			REDUCTION FACTORS Phi for Moment 0.95		
Wobble coefficient, K	0.000656 per m.		Phi for Moment 0.95 Phi for Shear 0.9		7. 1. 18.
Curvature co-efficient, µ	0.25				
Relative Humidity	70 %		보다 하늘살피는 왕으라면 하는 일 녹음을 살 밤밤을 것		
Amount of slip	8 mm,		Berger Berger (1997) in der Geren der Ge Der Geren der Geren		
	Control of the Contro	化氯化二甲基二甲二甲基二甲基二甲基甲二甲	the first term of a product of the control of the c	and the second of the second o	
ALLOWABLE STRESSES	orest with the reserving of	ระทั่งสายเริ่ม สามพังสาราชาวาศานิส	MINITIPLYING FACTOR FOR DESIGN TO ADJECT DESC	n in a record of the second of	
ALLOWABLE STRESSES	rajati serentek		MULTIPLYING FACTOR FOR DESIGN LOAD/STRESS FOR DESIGN IN LOAD FACTOR METHOD		
			MULTIPLYING FACTOR FOR DESIGN LOAD/STRESS FOR DESIGN IN LOAD FACTOR METHOD		
For normal R.C.C. Member		12.000 N/mm ²	or programment per per control of the conflict of the filter of the collection of the confidence of the collection of th	1.3	
For normal R.C.C. Member Concrete in Compression		12.000 N/mm² 3.411 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD		
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension			FOR DESIGN IN LOAD FACTOR METHOD General		
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear		3,411 N/mm² 0,716 N/mm² 0,432 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa		
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension		3.411 N/mm ² 0.716 N/mm ² 0.432 N/mm ² 165.600 N/mm ²	FOR DESIGN IN LOAD FACTOR METHOD General β _a Dead load Live load, for exterior beam supports S.W. L.L.,		
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8	k	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367	FOR DESIGN IN LOAD FACTOR METHOD General β _a Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact	1.3 1 1 1	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878	k R	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367 1,932 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General β₀ Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition	1.3 1 1 1 1.25	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity	k R	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367 1,932 N/mm² 27572 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General β₀ Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion	1.3 1 1 1	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity	k R	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367 1,932 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General β₀ Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition	1.3 1 1 1 1.25	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member	k R	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367 1,932 N/mm² 27572 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General β₀ Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion	1.3 1 1 1 1.25	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity		3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324	FOR DESIGN IN LOAD FACTOR METHOD General β _d Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail	1.3 1 1 1 1.25 1.67 2.2 1	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity Modulus of Elasticity at time of tran		3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General β _d Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail	1.3 1 1 1 1.25 1.67 2.2 1	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity Modulus of Elasticity at time of tran Basic Creep-coefficient		3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load i from face of rail value For moment For shear	1.3 1 1 1 1.25 1.67 2.2 1	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity Modulus of Elasticity at time of tran Basic Creep-coefficient A. Concrete	isfer	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load i from face of rail value value For moment For shear B. Prestressing Reinforcement	1.3 1 1 1 1.25 1.67 2.2 1 0.9 0.85	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement 1 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of tran Basic Creep-coefficient A. Concrete Stresses immediately after transfer	isfer	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367 1,932 N/mm² 27572 N/mm² 0,8324 29781 N/mm² 27572 N/mm² 2,65	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail value For moment For shear B. Prestressing ReInforcement	1.3 1 1 1 1.25 1.67 2.2 1	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement 1 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of transaic Creep-coefficient A. Concrete Stresses immediately after transfer Compression	isfer	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail 4 value For moment For shear B. Prestressing Reinforcement Yield Strength 1581	1.3 1 1 1 1.25 1.67 2.2 1 0.9 6.85	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement 1 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of transaic Creep-coefficient A. Concrete Stresses immediately after transfer Compression	isfer rcement	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367 1,932 N/mm² 27572 N/mm² 0,8324 29781 N/mm² 27572 N/mm² 2,65	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail 4 value For moment For shear B. Prestressing Reinforcement Yield Strength 1581	1.3 1 1 1 1.25 1.67 2.2 1 0.9 0.85	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement 1 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of tran Basic Creep-coefficient A. Concrete Stresses immediately after transfer J Compression II) Tension with no bonded Reinforce	isfer cement ment	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367 1,932 N/mm² 27572 N/mm² 0,8324 29781 N/mm² 27572 N/mm² 2,65 -16,50 N/mm² 1,36 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail 4 value For moment For shear B. Prestressing Reinforcement Yield Strength 1581	1.3 1 1 1 1.25 1.67 2.2 1 0.9 6.85	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement 1 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of transasic Creep-coefficient A. Concrete Stresses immediately after transfer J Compression J Tension with no bonded Reinforce Stresses at service load after losse	isfer cement ment	3,411 N/mm² 0,716 N/mm² 0,432 N/mm² 165,600 N/mm² 0,367 1,932 N/mm² 27572 N/mm² 0,8324 29781 N/mm² 27572 N/mm² 2,65 -16,50 N/mm² 1,36 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail φ value For moment For shear B. Prestressing Reinforcement Yield Strength 1581 Stress during tensioning 1423	1.3 1 1 1 1.25 1.67 2.2 1 0.9 6.85	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement 1 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of tran Basic Creep-coefficient A. Concrete Stresses immediately after transfer J Compression I) Tension with no bonded Reinforce Stresses at service load after losse J Compression x) under all loads except (y) & (z)	cement ment shave occured	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm² 2.65 -16.50 N/mm² 1.36 N/mm² 3.41 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail φ value For moment For shear B. Prestressing Reinforcement Yield Strength 1581 Stress during tensioning 1423 Stress immediately after seating At anchorage 1302	1.3 1 1 1.25 1.67 2.2 1 0.9 0.85 N/mm²	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of tran Basic Creep-coefficient A. Concrete Stresses immediately after transfer I) Compression II) Tension with no bonded Reinforce Stresses at service load after losse I) Compression x) under all loads except (y) & (z) y) under Prestressed force + all pression	cement ment shave occured	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm² 2.65 -16.50 N/mm² 1.36 N/mm² 3.41 N/mm² 3.41 N/mm² 5 -14.00 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail φ value For moment For shear B. Prestressing Reinforcement Yield Strength 1581 Stress during tensioning 1423 Stress immediately after seating At anchorage 1302	1.3 1 1 1 1.25 1.67 2.2 1 0.9 0.85 N/mm² N/mm²	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement 1 8 1 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of transasic Creep-coefficient A. Concrete Stresses immediately after transfer 1) Compression 1) Tension with no bonded Reinforce Stresses at service load after losse 1) Compression 1) Compression 1) Compression 2) Under Prestressed force + all put 2) under Prestressed force + all put 2) under Live loads + 1/2 of (y)	cement ment shave occured	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm² 2.65 -16.50 N/mm² 1.36 N/mm² 3.41 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail φ value For moment For shear B. Prestressing Reinforcement Yield Strength 1581 Stress during tensioning 1423 Stress immediately after seating At anchorage 1302 At the end of seating loss zone 1312	1.3 1 1 1 1.25 1.67 2.2 1 0.9 0.85 N/mm² N/mm²	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement 1 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of transasic Creep-coefficient A. Concrete Stresses immediately after transfer J Compression I) Tension with no bonded Reinforce Tension with bonded Reinforce Stresses at service load after losse J Compression x) under all loads except (y) & (z) y) under Prestressed force + all p z) under Live loads + 1/2 of (y) ii) Tension	cement ment shave occured	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm² 2.65 -16.50 N/mm² 3.41 N/mm² 3.41 N/mm² -21.00 N/mm² -14.00 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load i' from face of rail value For mornent For shear B. Prestressing Reinforcement Yield Strength 1581 Stress during tensioning 1423 Stress immediately after seating At anchorage 1302 At the end of seating loss zone 1312	1.3 1 1 1 1.25 1.67 2.2 1 0.9 0.85 N/mm² N/mm³ N/mm³	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of tran Basic Creep-coefficient A. Concrete Stresses immediately after transfer J Compression ii) Tension with no bonded Reinforce Stresses at service load after losse i) Compression x) under all loads except (y) & (z) y) under Prestressed force + all p z) under Live loads + 1/2 of (y) ii) Tension x) with bonded Reinforcement	cement ment as have occured ermanent dead loads	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm² 2.65 -16.50 N/mm² 1.36 N/mm² 3.41 N/mm² 3.41 N/mm² -14.00 N/mm² -14.00 N/mm² 2.95 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load 1' from face of rail φ value For moment For shear B. Prestressing Reinforcement Yield Strength 1581 Stress during tensioning 1423 Stress immediately after seating At anchorage 1302 At the end of seating loss zone 1312	1.3 1 1 1 1.25 1.67 2.2 1 0.9 0.85 N/mm² N/mm³ N/mm³	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta 1 For Prestressed Member Modulus of Elasticity at time of tran Basic Creep-coefficient A. Concrete Stresses immediately after transfer I) Compression ii) Tension with no bonded Reinforce Stresses at service load after losse i) Compression x) under all loads except (y) & (z) y) under Prestressed force + all p z) under Live loads + 1/2 of (y) iii) Tension x) with bonded Reinforcement with bonded Reinforcement	cement ment as have occured ermanent dead loads	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm² 2.65 -16.50 N/mm² 1.36 N/mm² 3.41 N/mm² 3.41 N/mm² -14.00 N/mm² 1.400 N/mm² 1.47 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load i' from face of rail value For mornent For shear B. Prestressing Reinforcement Yield Strength 1581 Stress during tensioning 1423 Stress immediately after seating At anchorage 1302 At the end of seating loss zone 1312	1.3 1 1 1 1.25 1.67 2.2 1 0.9 0.85 N/mm² N/mm³ N/mm³	
For normal R.C.C. Member Concrete in Compression Modulus of Rupture Plain Concrete in Tension Concrete in Shear Tension in Reinforcement n 8 J 0.878 Modulus of Elasticity Beta1 For Prestressed Member Modulus of Elasticity at time of tran Basic Creep-coefficient A. Concrete Stresses immediately after transfer I) Compression II) Tension with no bonded Reinforce Stresses at service load after losse I) Compression x) under all loads except (y) & (z) y) under Prestressed force + all p z) under Live loads + 1/2 of (y) III) Tension x) with bonded Reinforcement	cement ment as have occured ermanent dead loads	3.411 N/mm² 0.716 N/mm² 0.432 N/mm² 165.600 N/mm² 0.367 1.932 N/mm² 27572 N/mm² 0.8324 29781 N/mm² 27572 N/mm² 2.65 -16.50 N/mm² 1.36 N/mm² 3.41 N/mm² 3.41 N/mm² -14.00 N/mm² -14.00 N/mm² 2.95 N/mm²	FOR DESIGN IN LOAD FACTOR METHOD General βa Dead load Live load, for exterior beam supports S.W. L.L., traffic L.L. + Impact Live load + Impact, Normal condition Live load + Impact, Over load criterion Live load i' from face of rail value For mornent For shear B. Prestressing Reinforcement Yield Strength 1581 Stress during tensioning 1423 Stress immediately after seating At anchorage 1302 At the end of seating loss zone 1312	1.3 1 1 1 1.25 1.67 2.2 1 0.9 0.85 N/mm² N/mm³ N/mm³	

137113

LEFT/SYMMETRIC PARAPET

Put 1.0 for traffic, 2.0 for combination
& 3.0 for Pedestrian Railing = 2
Width of each pre-cast segment of parapet = 1500 mm,
Maximum c/c spacing of metalic post = 1500 mm,
Total no. of post required = 21 Nos.

Unit wt. Of metalic part of po-	st & rail	A	·	7 kN/m³,
Bottom dimension of top pos	t = 1		. 12	5 mm,
Top dimension top post	To Self-te	la de la composição de	-10	0 mm,
Thickness of top post		Alleria		8 mm,
Weight of metalic post per lin	iear mete	er	0.01	2 kN/m,
	100		100	

Dia of metalic rail 100 mm.
Thickness rail pipe 6 mm.
Weight of rail per linear meter 0.07 kN/m,

P	. S	\$1,50	13.74	44.4822 kM	ł,
h	100	47.7		820 m	m,
c		200			

garate bio

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				0		Part-2
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110	40.			11		Part-5
	13.5					Part-6
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			(6)			Total
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Item	Thickness	Weight	Dist. of cg from a
nem	(mm)	(kN/m)	(mm)
Metal	13 6.14	0.083	1340
Part-1	180.00	3.478	1340
Part-2	60.00	0.580	1450
Part-3	313.56	0.554	837.5
Part-4	397.44	· 4.495	A 190 April 115 (0.00)
Part-5	100.00	1.791	1405 (6.47)
Part-6	35.00	0.313	1467
1/2 of F.C.	75.00	0.530	838 🕔 🕟
Total	4 4	11.824	1226.972
yan da 186	aka sa sa		ng Palawaya ng baya di ay ika
	May 35		g Honeldweiter
	ntit Un		The Contract of the
A4 2 3 3	mza de		์ พลากสัสด์แกลสามารถที่

10		MARKET BELLEVILLE	38 (1945 F.S.) 184	Call the recorded pages a page, the first and a con-
	12,45,674	Weight	of Curb with	1/2 F.C.
	Item	Thickness	Weight	Dist. of cg from a
	#91550K	(mm)	(kN/m)	(mm)
	1/2 of F.C.	75.00/	·/ 0.530	313
å	Triangle	360.50	0.212	33
÷	Rect. Left	295.19	0.522	313
ì	Rect. Right	364.94	1.935	163
T	Total	1361134	3.199	203.25
1	200		100	and the contract of the contract of

٠.	50. W	1.5	1	* 1.5	1.150	100		
	40.0			, dili s	1		1	14.5
	243	250	1000		1.5			. 4.5

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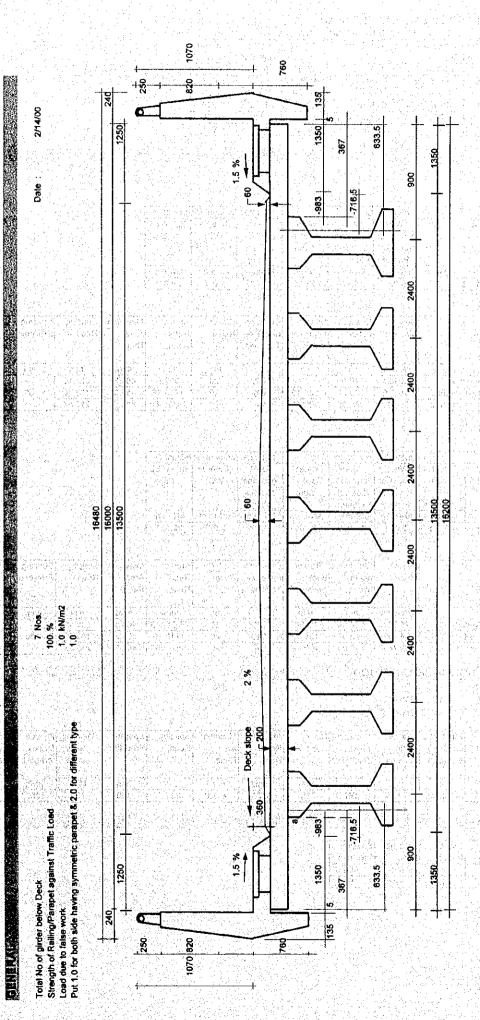
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ikingan di 1853. Berjada ng mga Tili Bershi ka Igara, kanang ikit ang atawasan ing kanang 1968. Berjada ng mga Tili Bershi ka Igara, kanang ikit ang atawasan kanang iking kanang 1968.

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PARAPET

LEFT / SYMMETRIC PARAPET

Put 1.0 for traffic, 2.0 for combination		
& 3.0 for Pedestrian Railing	=	2
Width of each pre-cast segment of parapet	=	1500 mm,
Maximum c/c spacing of metalic post	=	1500 mm,
Total no. of post required	=	21 Nos.,

Unit wt. Of metalic part of post & rail	77 kN/m³,
Bottom dimension of top post	125 mm,
Top dimension top post	100 mm,
Thickness of top post	8 mm,
Weight of metalic post per linear meter	0.012 kN/m

Dia of metalic rail 100 mm, Thickness rail pipe 6 mm, Weight of rail per linear meter 0.07 kN/m,

Weight of End Parapet with 1/2 F.C.

ю	44 4922 141
P	44.4822 kN,
h	820 mm,
С	1

				1 1				, v	veignt of L	no Parapet	With 1/2 F.C.
								Item	Thickness	Weighl	Dist. of cg from a
		3			1490			ILCIII	(mm)	(kN/m)	(mm)
		60	180	1	1200		50	Metal		0,083	1340
4							1 1	Part-1	180.00	3.478	1340
			(0)					Part-2	60.00	0.580	1450
2	250		11 1					Part-3	313,56	0.554	837.5
ĺ		ll	$T \Box$	44.48 Kn,			1. 1	Part-4	397.44	4.495	1115
								Part-5	100.00	1.791	1405
			1					Part-6	35.00	0,313	1467
ļ			Į.				: [1/2 of F.C.	75.00	0.530	838
į			II.					Total		11.824	1226.972
1070			(1)				: 1				
1	820	ll	240	450 6			i :			of Curb witl	
		1	7	1			, .	Item	Thickness	Weight	Dist. of cg from a
			Ī				. !	L	(mm)	(kN/m)	(mm)
			į.					1/2 of F.C.	75.00	0.530	313
		1 1	1			1	225	Triangle	360.50	0.212	33
1		0	2)	375			١;	Rect. Left	295.19	0.522	313
				· [Rect. Right	364.94	1.935	163
- 1		[0	5)	<u>L.</u>		25	\ \ \ \	Total		3.199	203.25
	405	\	(5)	(4)		10 275	1	360	ļ		
760	355			475	450	75	275		200	}	
				A constant and the second and the se		<u></u>				,	
		35	100	550 5				1 1			
				ວ '				┸┛┸			

GENERAL 135 2/14/00 1250 633.5 1350 367 1350 1.5 % 006 Date : ر ا 60 -716.5 2400 DESIGN OF DECK SLAB (For Approach Bridge) 2400 2400 13500 80 15480 15000 13500 2400 7 Nos. 100 % 1.0 kN/m2 1.0 2400 2 % Total No of girder below Deck Strength of Railing/Parapet against Traffic Load Load due to false work Put 1.0 for both side having symmetric parapet & 2.0 for different type Deck slope r 200 2400 380 716.5 -983 800 1.5 % 1250 633.5 1350 367 240 250 1070 820 760

1070

760

DEAD LOAD MOMENT AT SPAN END OF CANTILEVERE SLAB

ROM LEFT PARAPE	ET SYSTEM	A ·	27 200	FROM RIGHT PARAP	FISASIE	M	
Components	Weight	Dist. from	Mom. at	Components	Weight	Dist. from	Mom. at
		span end	Span end			span end	Span end
2 4	kN/m	(mm)	kN-m/m	and the second second	kN/m	(mm)	kN-m/m
Parapet	11.824	510.5	6,036	Parapet	11,824	510.5	6,036
Curb	3.199	-513.3	0,000	Curb	3.199	-513.3	0,000
C. Slab	2.985	316.8	0.946	C. Slab	2.985	316.8	0.946
Wearing Course 1	0.000	-358.3	0.000	Wearing Course 1	0.000	-358.3	0.000
Total	18.008	 	6.981	Total	18.008		6,981

DESIGN OF CANTILEVER SLAB

LEFT CANTILEVER SLAB

Moment	Table		10 mm 20 mm	"连","我们是这个人,我们就是我们的一个人的人 <u>是一种的一个人的。"他们</u>					the state of the s			
Effective Width for			Moment due to					Factored Moment Combination				
Post	Wheel	Wheel	Dead	Post	Wheel	Wheel	Wheel	D+P	(D+WD+	((D+WS)/		
Load	Load over			Load	load over	load over	load at	(working	WC)	1.5)		
	Deck Slab			11.5	Deck Slab	Sidewalk	Curb	combin-				
		54743		100	+ Impact	+ Impact		ation)		24.5		
m	т	m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m	kN-m/m		
2.0468	0.000	1.3260	6.981	27.818	0.000	15.958	0.000	34.799	9,076	29.821		

Design lable	Market Committee of the	the property of		200	- 11 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 -		· · · · · · · · · · · · · · · · · · ·			
Item with design procedure	Design	Effective	Effective	Comment	Dia. of	Area of	Minimum	Spacing	Spacing	Z value
Activities and a second	Moment	depth	depth	regarding	Rebar	Rebar	Area	Regd.	Provided	for flex.
		available	required	Thickness	12/2	Reqd.	Regd.		1.00	cracking
	kN-m/m	mm	mm		mm	mm2	mm2	mm	mm	Mpa
Other combination, designed by method directed	29,821	140.00	66	OK.	20	592.5	466.31	530.19		13874
Due to Post Load, designed by working stress method.	34,799	140.00	134.19	OK	20	1710.19	466.31	183.70	150	
Due to Post Load, designed by ultimate stress method.	69.468	140.00	100.49	ОК	20	1457.588	466.31	215.53	<u> </u>	O.K.

RIGHT CANTILEVER SLAB

Mament Table

Eff	ctive Width	for	40	Moment due to Factored Moment Combi						
Post Load	Wheel Load over Deck Slab	Wheel Load over	Dead Load	Post Load	Wheel load over Deck Slab	Wheel load over Sidewalk	Wheel load at Curb	D+P (working combin-	(D+WD+	((D+WS)/ 1.5)
<u> </u>		2:	131 (-	kN-m/m	+ Impact kN-m/m	+ Impact kN-m/m	kN-m/m	ation):	kN-m/m	kN-m/m
0.0020	0,000	1.326	kN-m/m 6.981	27.818	0.000	15.958	0.000	34.799	9.076	29.821

Dacida Table

Item with design procedure	Design Moment	depth	Effective depth required	Comment regarding Thickness	Rebar	Area of Rebar Reqd.	Minimum Area Regd	Spacing Reqd.		Z value for flex cracking
	kN-m/m	mm	mm		mm	mm2	mm2	mm	mm	Mpa
Other combination, designed by method directed	29.821	140.00	65.84	OK	20	592.5442	466.31	530.19	and the	13874
Due to Post Load, designed by working stress method.	34,799	140.00	134.19	OK	20	1710.19	466.31	183.70	150	1.356
e to Post Load, designed by ultimate stress method.	69.468	140.00	100.49	OK	20	1457.588	466.31	215.53	1	O.K.

DESIGN OF INTERIOR SLAB BY COEFFICIENT METHOD

Dead Load from Wearing Course

1.414 kN/m²

Dead Load from self wt. of Slab

4.713 kN/m²

Span of	Momen	t due to	Design	Effective	Effective	Comment	Dia. of	Area of	Minimum	Spacing	Spacing	Working	Z value	Comment
1'	Dead		Moment	depth	depth	regarding	Rebar	Rebar	Area	Regd.	Provided	steel	for flex.	regarding
0.00		with		available	required	Thickness		Reqd.	Reqd.			stress	craqcking	crack
	130 145	Impact		100				11 9 15 5			177, 134	Apr 1767		
m	kN-m/m	kN-m/m	kN-m/m	mm	mm	·	mm	mm2	mm2	mm	mm	Mpa	kN/m	3 m 277
1.867	2.135	18.794	43.579	142.00	79,59	OK	16	867.7	473.0	231.7	150	125	12567	O.K.

DISTRIBUTION REINFORCEMENT

			. A	
% of +ve	Area of Rebar	Dia. of	Spacing	Spacing
reinforcement	Required	Rebar	Reqd.	Provided
	mm2	mm	mm	mm
67	501	12	105	150

DATA FOR PRESTRESSED GIRDER

Effective web width for Exterior Girder:

(1)	2841.0
(2)	1066.0
Final	1066.0

Effective width of flange for Exterior Girder:

(1)	Ċ		3320.8
(2)			2633.0
(3)	•		2100.0
Final		٠.	2100.0

Fraction of Wheel Load on left exterior Girder	0,5585
Fraction of Wheel Load on right exterior Girder	0.5585
Fraction of Wheel load on left exterior Girder when Wheel over Side	1.650667
Fraction of Wheel load on right exterior Girder when Wheel over Sid	1.650667
Fraction of Wheel Load on interior Girder	1,431639

Effective web width for interior Girder:

	٠.			
(1)		٠.		2841
(2)				1066
Final			10	1066.0

Effective width of flange for Interior Girder:

(1)		7262.5
(2)	77.74	3466
(3)		2400
Final	100	2400

LOAD FROM ADJUSTING CONCRETE ON GIRDER DUE TO SLAB INCLINATION

Extension of slab below		1.00	11 11 11 11	Girder No.	4 - 5 - 5 - 5 - 5		
due to slope of deck slab	1 9	2	3	4	5	6	7
	Left Exterior	1st Interior	2nd Interior	3rd Interior	4th Interior	4th Interior	Right Exterior
Rise for each girder / flange		48	21,32		1, 1	9 9 1	
Increase of weight due to girder s	pacing	1,206	1.4.15	- 1			
Increase due to flange width		0.268		41.4		100	y per eller
Weight	0.268	0.268	0.268	0.268	0,268	0.268	0.268

SUMMERY OF LOAD ON GIRDERS

Load Type		1.2.1.1.1.1.		Girder No.		22,27 1.2	e in all marine
	1	2	3	4	5	6	7
	Left Exterior	1st Interior	2nd Interior	3rd Interior	4th Interior	5th Interior	Right Exterior
		, fr			4.7		
Non-composite dead load	9.977	11.816	11.511	11.612	11.511	11.816	9.977
Composite dead load	20.239	-1.894	4.896	2.649	4.896	-1.894	20.239

MERY OF LOADS FROM DECK SYSTEM ON GIRDERS

Press 1.0 for distribution of load from deck system using moment distribution method or 2.0 for AASHTO method Mention Girder you Like to design now

INTERIOR GIRDER

Non-composite dead load on girder 11.31031	kN/m,
Composite dead load on girder + utility pipe 9.636861	kN/m,
Effective web width	mm, - 12, 18, 21, 18, 18-48-77 (1984-1997)
Effective flange width 2400	mm,
Fraction of wheel load on girder 1.431639	医耳氏神经畸形的 医乳蛋白 医多耳氏氏管
Fraction of wheel load on girder, when wheel over sidewalk 0	
Effective width for sidewalk 0	mm,
Difference in elev. Between girder top and slab bottom	mm, 10.66 Concentrate
Uniform load on girder from false work 2.4	kN/m,

2

EXTERIOR GIRDER

LATERIOR GIRDER		or the first of
Non-composite dead load on girder	9.89652	kN/m,
nposite dead load on girder + utility pipe	9.636861	kN/m,
Errective web width	1066	mm,
Effective flange width	2100	mm,
Fraction of wheel load on girder	0.5585	
Fraction of wheel load on girder, when wheel over sidewalk	1.650667	200
Effective width for sidewalk	1200	mm,
Difference in elev. Between girder top and slab bottom	11	mm,
Uniform load on girder from false work	2.1	kN/m,

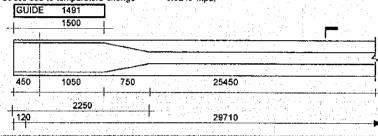
DESIGN OF INTERIOR PRESTRESSED GIRDER m curtyrail/pedestal equally distributed on girder

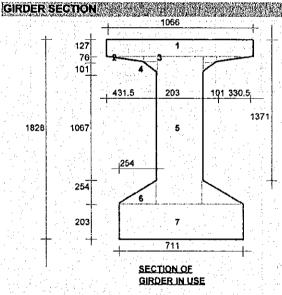
DESIGNED BY: PRASANTA KUMAR BHOWMIK GENERAL! Total Length of one Girder 29950 mm. Distance of center of Bearing from end of Girder 450 mm, Span of Girder 29050 mm, Distance of center of Diaphragm from end of Girder 350 mm. C/c. spacing between girders 2400 mm, Effective width of sidewalk 0 mm, Fraction of wheel load on Girder 1.432 Fraction of wheel load on girder when wheel over sidewalk 0.000 Live Load Magnitude HS 20 Put 1.0 or 2.0 for Deck type Mention AASHTO or WASHINGTON Pattern AASHTO Mention Type 6 Mention end section 1 or 2? 2 Anchorage indent 120 mm. 0.0246 Mpa, Tensile Stress due to temparature change

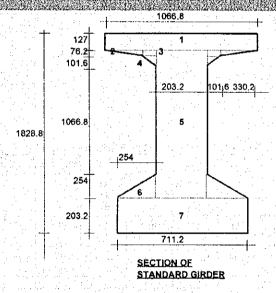
DIAPHRAGM DETAILS Total no. of interior diaphragm reqd 2 Total no. of interior diaphragm provided Width of diaphragm 300 mm, Depth of end diaphragm 1887 mm. Depth of interior diaphragm 1625 mm, Wt. of exterior diaphragm 26.78 kN, Wt. of interior diaphragm 23.65 kN,

NOTE: In designing this girder following references are followed:

- 1. AASHTO 16th edition, 1996.
- 2. Design of Prestressed Concrete, 2nd edition,
 - by : Arthur H. Nilson.
- 3. Design of Prestressed Concrete Structure, 3rd edition,
 - by : T. Y. Lin & Ned H. Burns.
 - All notations are followed from reference-2

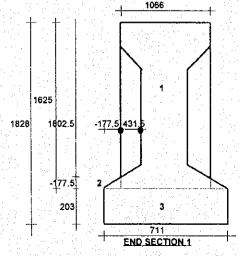


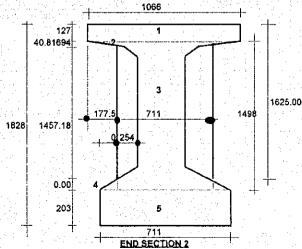


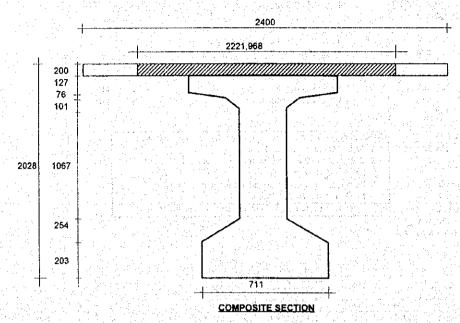


Effectiveness ratio of non-composite section Effectiveness ratio of composite section

0.521613 0.538194 Note: For effectiveness ratio < 0.45, indicate too heavy a section For effective ness ratio > 0.55, indicate an excess slender section.







3.46.				MIDDLE SE	27 and 44 and 5 and 5 and 5			22/4/20/20/20 12/2		F PRECAS	free and the second	2.2.2	Service A
Segment	Area	Distance of	CG mom	Moment of	inertia of	Radius of	Segment	Area	Distance of	CG from	Moment of	nertia of	Radius of
		Тор	Bottom	Components	Block	Gyration		100	Тор	Bottom	Components	Block	Gyration
		C _{1P}	C ²⁶	lo	lc	l r²		100	C1P	C2P	10	lc	r2
	mm ²	mm	mm	mm ⁴	mm ⁴	mm²		mm2	mm	mm	mm4	mm4	mm2
1	135382		1.3 1.4	1.82E+08			1	135382			1.82E+08	10 LN	
2	12559			4030044			2	3623	gret kreteer vie	managa yan sas	335287		
3	7676	1 4 4 1		3694715			3	1065078	881.49	946,51	1.99E+11	3.98E+11	294002
4	5100.5	904.62	923.38	2890567	3.05E+11	435707.6	4	0			0		
5	304094			5.69E+10			5	144333			4.96E+08		
6	32258			1.16E+08			Total	1352038				100	
7	144333			4.96E+08				Terral Science		a gent e la filla		Section 1	

Weight per linear metre = 31.86 kN/r

12272 AV 18		4 1 1 1	397 1.2		134.	
Weight pe	r linear	metre =	16	47	kN/m	ı.

Segment	Area	Distance	of CG of Co	imposite sect	ion from	Moment of	Inertia of	Radius of
		Top of precast	Bot. of precast	Bottom of	Top	Components	Block	Gyration
		section C ₁₀	section C _{2C}	Slab C _{4C}	Slab C₃c	l _o	lc	r ²
	mm²	mm	mm	mm	mm	mm ⁴	mm ⁴	mm ²
Precast Section	698996					3.05E+11		
Eqv. Slab	444393.6	514.1593	1313.841	525,1593	725.1593	1.48E+09	5.86E+11	512761.
Total	1143390				2.5			

Segment	Area	Distance	of CG of Co	mposite sec	ion from	Moment of	Inertia of	Radius of
		Top of precast section C _{1C}	Bot. of precast section C _{2c}	Bottom of Slab C ₄₀	Top of Slab C _{3C}	Components	Block Ic	Gyration r ²
	mm²	mm .	mm;	mm	mm	mm ⁴	mm ⁴	. mm²
Precast Section	1352038				5 M 4	3.98E+11		
Eqv.	4440000	638.6969	1189.303	649.6969	849.6969	4 6	7.28E+11	405502.
Slab Total	444393.6 1796432					1.48E+09		19.5

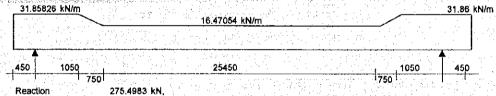
Weight per linear ft.

42.33 kN/m.

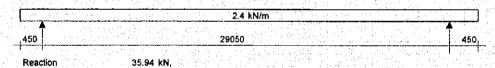
SUMMERY OF SECTION PROPERTIES AT DIFFERENT LOCATIONS

Section		the sub-	1	2	3	4	5	6	7	8	9
Distance from end	4 1 2 1	(mm)	0	450.00	2250	7712.5	14975	22237,5	27700	29500	29950
Total area of duct		(mm)	13273.23	13273,23	13273,23	13273.23	13273.23	13273.23	13273.229	13273.23	13273.23
Dist.of c.g. of duct from	bot.	(mm)	895.27	849.24	679,17	300.61	117.73	300.61	679,17	849,24	895.27
Duc pro	Аср	(mm)	1338765	1338765	685723	685723	685723	685723	685723	1338765	1338765
Duct prop nonce	C _{1P}	(mm)	880.99	880.53	899,89	892.56	889.02	892.56	899.89	880.53	880,99
Juct deducted properties of noncomposite section	C _{2P}	(mm)	947.01	947.47	928,11	935.44	938.98	935.44	928.11	947,47	947.01
	lc	(mm ⁴)	3.97E+11	3.97E+11	3.04E+11	2.99E+11	2.96E+11	2.99E+11	3.04E+11	3.97E+11	3.97E+11
# C (60 ·	2-	(mm²)	296888	296819	442937	436335	431081	436335	442937	296819	296888
350	Аср	(mm²)	1352038	1352038	698996	698996	698996	698996	698996	1352038	1352038
Duct prop noncc	C _{1P}	(mm)	881,49	881.49	904.62	904.62	904.62	904.62	904.62	881.49	881.49
Duct grouted properties of toncomposite section	C _{2P}	(mm)	946,51	946.51	923.38	923.38	923.38	923.38	923,38	946.51	946.51
osit Osit	lc .	(mm ⁴)	3.98E+11	3.98E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.98E+11	3.98E+11
8 ♣ ♣	, , , 2	(mm²)	294002	294002	435707.6	435707.6	435707.6	435707.6	435707.6	294002.2	294002.2
٥٥	Acc	(mm²)	1796432	1796432	1143390	1143390	1143390	1143390	1143390	1796432	1796432
2 8	Cic	(mm)	638,70	638.70	514.16	514.16	514.16	514.2	514.2	638.7	638.7
Duct grouted of composit	C _{2c}	(mm)	1189.30	1189,30	1313.84	1313,84	1313.84	1313.8	1313.8	1189.3	1189.3
	C _{4c}	(mm)	649.70	649.70	525.16	525.16	525.16	525.2	525.2	649.7	649.7
2 2	C _{3c}	(mm)	849.70	850	725.2	725.2	725.2	725.2	725,2	849.7	849.7
opertie	lc	(mm ⁴)	7.28E+11	7.28E+11	5.86E+11	5.86E+11	5.86E+11	5.86E+11	5.86E+11	7.28E+11	7.28E+11
es n	r ²	(mm²)	405502.1	405502.1	512761.3	512761.3	512761.3	512761.3	.512761.3	405502.1	405502.1

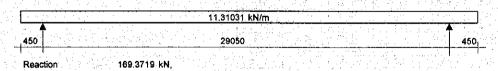
LOAD DIAGRAM FOR SELF WEIGHT



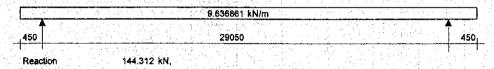
LOAD DIAGRAM FOR FALSE-WORK DEAD LOAD



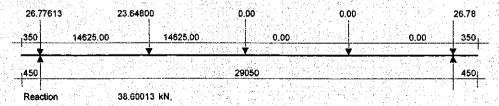
LOAD DIAGRAM FOR NON-COMPOSITE DEAD LOAD



LOAD DIAGRAM FOR COMPOSITE DEAD LOAD



LOAD DIAGRAM FOR CONCENTRATED LOAD FROM DIAPHRAGM



Section	Distance	10 T	10.044.000.00	- 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1	, ,	loment due	to	Hall to the			Total	Total
	from end	Self wl	Cross	Non	Composi-	Live Load	Live Load	Sidewalk	Total	Falsework	Dead	Factored
100			Girder	Composi-	te dead	with	with Impa-	Live	(Non-Co-	(Non-Co-	load	Moment
100		100	(Non-Co-	te dead	load	Impact	ct when	Load	mposite	mposite	Moment	1 4 4 4
			mposite	load			wheel on		dead load)	dead load		
			dead load				Sidewalk		12.72		4. 4. 4.	
		Mo			Mpc			i i sand	M _{DP}	1.3626.75		
	5,700	(a)	(b)	(c)	(d)	(e)	(f)	(g)	(b+c)			
	(mm)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	· (kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)
1	0	0.00	0.00	0.00	0.00	0.00	0,00	0.00	0.00	0.00	0.00	0.00
2	450	+3.23	-2.68	-1.15	-0.98	0,00	0.00	0.00	-3.82	-0,24	8.02	-10.43
2a	1364	222.17	8.13	144.28	122.94	223,80	0.00	0,00	152.41	30.62	497.52	1132,64
3	2250	416.70	18.61	276.24	235.37	425,29	0.00	0.00	294.85	58.62	946.91	2154.30
4	7712.5	1315.84	83.19	893.68	761.45	1331.87	0.00	0,00	976.87	189.64	3054.17	6861.91
5	14975	1750.20	169.07	1191,95	1015.60	1709.16	0.00	0.00	1361.02	252.93	4126,82	9075,44
6	22237.5	1315.84	83.19	893.68	761.45	1331.87	0.00	0.00	976.87	189.64	3054.17	6861.91
7	27700	416.70	18.61	276.24	235.37	425.29	0.00	0.00	294.85	58.62	946.91	2154.30
8	29500	-3.23	-2.68	-1.15	-0.98	0.00	0.00	0.00	-3.82	-0.24	-8.02	-10.43
9	29950	0.00	0.00	√ 0,00	0.00	0.00	0.00	0,00	0.00	0.00	0.00	0.00

rictional loss per unit length

144.20

14.20246 kN/m,

for 8mm Anchorage draw in, x(dist. Of anchorage loss zone) ΔP_{A}

21.7313 mm, 617.2771 kN,

Prestress force after friction losses (before elastic shortening) :-

5333 kN,

Tensioning end Seating loss zone

5641 kN,

No. of cable read

3.64

5545 kN,

Midspan Dead end

5525 kN,

SUMMERY OF INITIAL LOSSES

Section	Distance from end	Jacking Force	Losses due to	Prestress Force	Conc. Str- ess at the	Loss due to	Final initial	Final initial
			friction	after	level of	elastic	loss	prestress
		H (1)	and anc-	friction	steel	shortening		force
	Part St.		horage	loss	centroid at			(after
		4 124	pull-in	(before	sec.of ma-		(øfter	immediate
1.1			17 17 17	elastic sh-	xm. mom.	111	immediate	losses)
				ortening)	(f _{cir})		losses)	(Pi)
	(mm)	(kN)	(kN)	(kN)	(mPa)	(kN)	(kN)	(kN)
1	0		617	5333	3.935	59,82	677	5273
2	450		611	5339	4.030	61.27	672	5278
2a	1364		598	5352	4.269	64.90	663	5287
3	2250	1.0	585	5365	8.189	124.50	710	5240
- 4	7712.5	5950	508	5442	11,551	175.61	683	5267
5	14975	1 - 1 - 1	405	5545	14.345	218.10	623	5327
6	22237.5		316	5634	12.049	183.19	499	5451
7	27700	1	393	5557	8.493	129.13	523	5427
8	29500	25 5 4	419	5531	4.175	63.47	482	5468
9	29950		425	5525	4.076	61.98	487	5463

Section	Distance	Conc. Str-	Loss	Loss	Loss	Final	Total	Loss	Final
	from end	ess at the	due to	due to	due to	Time	loss	in	effective
		level of st-	creep	shrinkage	relaxation	dependent		percent	pre-stress
		eel cg. Du-		1.3.25	of pre-	loss		10000	force
		e to dead		. 7 ·	stressing			10 to	
		loads exc-		194	steel		1.5		1,50.75
		ept self wt.	2000	57.64	9.5	3 4 5			
		(f _{cds})	Safe per		1.5	1,114.5	1.00		(Pe)
11, 11	(mm)	(mPa)	(kN)	. (kN)	(kN)	(kN)	(kN)	the sales	(kN)
1	70	0.000	205.06		317.59	678.34	1355.44	22.78	4594.56
2	450	-0.002	210.09	200 12 45	317.92	683.71	1355.86	22.79	4594.14
2a	1364	0.162	217.52		318.88	692.10	1354.90	22.77	4595.10
- 3	2250	0.574	409.30		260.46	825.45	1535.27	25,80	4414.73
4	7712.5	3.004	510.63	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	243.02	909.35	1592.70	26 ,77	4357.30
5	14975	4.849	600.18	155,70	239.06	994.94	1617.53	27.19	4332.37
- 6	22237.5	3.004	536.61	1	292.36	984.67	1483.69	24.94	4466.31
7	27700	0.574	425.17	1	313.00	893,87	1416.41	23.81	4533.59
8	29500	-0.002	217.63	1	373.10	746.44	1228.88	20.65	4721.12
9	29950	0.000	212.44	1	372.82	740.96	1228.30	20.64	4721.70

EFFECT OF DIFFERENTIAL SHRINKAGE

Ultimate shrinkage co-efficient, ϵ_{SHU} 0.0008 Correction by humidity, (RH = 70) FsH,H 0.7 Assumed day of slab casting after girder concrete placed, t 60 days Shrinkage of girder at t days, $\epsilon_{\text{SH,t}}$ 0.632 Differential (remaining) shrinkage, $\epsilon_{\text{SH,D}}$ 0.000206 Ultimate creep coefficient for girder, ϕ_{cgu} 2.6 Ultimate creep coefficient for slab, ϕ_{csu} 2.8 Correction by humidity for creep coefficient, (RH = 70), Fc,H 0.801 Remaining creep of girder after t days, ϕ_{eg} 0.961267

Remaining creep of slab, φ _{cs}	2.24	
Part Later and L	Share & Carolina Markett An Atlanta Hara A	210

Hor	izontal She	ar & Bendin	g Moment o	on contact s	urface of oir	del & slab	ue to Diffe	ential Shrin	kage .
W 10 10 10 10 10 10 10 10 10 10 10 10 10	6.00	6000000	Mech A	all section	ke mid sec	The state of the s	e de la companya de	A. Leafella be	ALDON MARKET
m				70	υ	Gir	der	S	lab
111		oda Vije	a de la Sil	Ψας	Ψα	V _{SH}	M _{SH}	V _{SH}	M _{SH}
14.11		(m)	(m2)	tak nigiti.	San	(kN)	(kN-m)	(kN)	(kN-m)
222.073	223.073	-23.746	4.730	0.961	2.243	433.145	46.107	261.969	27.886

	Gir	der	S	lab
	Bottom	Тор	Bottom	Тор
Stress at concrete due to differential shrinkage	0.708	-1.920	0.629	0.471

Hor	izontál Shé	ar & Bendin	g Moment c	n contact s	utace of ou	der & slab	due to Diffe	ential Shini	kege
		1900	A. A.	all section I	ike end sec	bon .			
			1	9190	la e la	Gir	rder	S	lab
, m	В	C	.	Ψα	Ψes	VsH	M _{SH}	V _{SH}	M _{SH}
	100	(m)	(m2)			(kN)	(kN-m)	(kN)	(kN-m)
289.845	290.845	-31.402	5.608	0.961	2.243	561.488	60.623	339.592	36.665

	1	Girder	Slab
		Bottom Top	Bottom Top
Stress at concrete due to		0.908 -1.647	0.779 0.643
differential shrinkage	5.55	Larry March 1985 (1985)	

DESIGN FOR MOMENT AT MID SECTION

Total Jacking tension canbe taken by supplied cable (after jack loss) 5993.99

		ande diveri	-,	seed (Bite	. ,,		0000.00	,			<u>`` </u>	
1,4,27%	Initial T	ension		Total	Jacking	Effective	(Row) ρ	f _{ps}	а	Ultimate	Ultimate	Comment
Maximum	Maximum	: Can be	Actual	no. of	Tension	Tension				Design	Moment	
(1)	(2)	taken by	at mid	Cable	(after	at mid				Moment	Capacity	
Barrier 1	11.9	supplied	Section	reod.	Jack	section				17.15.47	3.5 0.5	
	4	Cable	150000		ioss)		37 W 1	15 2 2				
. (kN)	(kN)	(kN)	(kN)		(kN)	(kN)	5.0554	(N/mm²)	(mm)	(kN-m)	(kN-m)	4 M 4 M
6552	5424	5654	5327	3.64	5950	4332	0.000947	1807.51	128.26	9075.4436	13757.43	OK .

PRESTRESSING REINFORCEMENT DETAILS

	PRESTRESSING REINI ONCEMENT DETAILS												
	Total no.	Total no:	Area	Total no.	Dia	A Page 175	Area of		Minimum	Outer	C/	c. Spacing	of
1	of cable	of duct	factor	of strand	of one	One	One cable	Total	inside	diameter	44 N X	Ducts	dept such
	rego.	used	10 to	in one	strand	Strand		Cable	diameter	of Duct	Reqd.	Allowed	Allowed
	1.5			cable		4.545.7	4.25	(Ap)	of Ducts	Provided	11.14	mid	end
	(nos)	(nos)		State of	(mm)	(mm2)	(mm2)	→ (mm2)	(mm)	(mm)	(mm)	(mm)	(mm)
	3.53	4	3.666667	12	12.7	98.7	1184.40	4342.80	54.92	65	103.1	125	125

Reinforcement Index 0.057072 OK against maxm. reinforcement.

Cracking Moment 3311.776 kN-m. OK against minm. reinforcement. 8036.54 2.29

STRESSES AT DIFFERENT LOCATION OF MID-SECTION OF GIRDER

SIKESSE	S ALL DIFFE	VEW	MINUTE UT	(MILVISEU)	tou of all	心 巨大 语 第	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	Land Straight	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	<u>al individual de la companya de la</u>	<u> </u>	4 4 4 4 4		<u> </u>
	A girls	Initial 7	Tension	4. 特别的人们	7 (8)	MARGARITY	ar switcher in a		Effectiv	e Tension	医乳色性蛋白蛋白	5 To 6.5	14 1 1 1 1 1 1	81 Taylor 8 To
	Pi (a	lone)	Pi-	Mo	P.+M	l₀+Map	P _e +M _o +	Map+Mac	Pe+Mo+N	lap+Mac+Mi	0.5(Pe+Mo	+M _{dp} +M _{dc})	At top of	At bot, of
10, 10, 10, 21		100	(À) & Paris	(1	B) - 1 - 1	((C)	+SH	(D)	+MI	(E)	Deck	D eck
	Тор	Bottom	Тор	Bottom	Τόρ	Bottom	Тор	Bottom	Тор	Bottom	Тор	Bottom	Slab	Slab
	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm ²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm ²)	(N/mm ²)
Stress	5.13	-20.63	0.13	~16.11:	-5.80	-6.56	-6,69	-4.28	+9,36	-0.51	-4.46	1.32	-2.62	-1.61
All. stress	Sec. 12. 25. 2		1.36	i-16.50	-14.00	1.47	-14.00	1.47	-21.00	1.47	-14.00	1.47	-12	0.72
Comment	1.4		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK.	OK

CHECK AT MID SECTION FOR EXTERIOR GIRDER WHEN WHEEL ON SIDEWALK

	ndawa ili	Eft.	ective Tens	ion		Belleville:	
	P _e +M _o +M +SH	op+Moc+Mi (D)	0.5(P.+M. +MI	The state of the s	At top of Deck	At bot, of D eck	
	Top (N/mm²)	Bottom (N/mm²)	Top (N/mm²)	Bottom (N/mm²)	Slab (N/mm²)	Slab (N/mm²)	
Stress	0.00	0.00	0.00	0.00	0.00	0.00	
All. stress	-31.50	2.21	-21.00	2.21	-18	1.07	
Comment	ОК	OK	OK:	OK	OK	OK	

Factor of safety against cracking

the second of th				28.			:		
Total no, of Row at mid Section	Row 1	Row 2			A W ₁				
No. of Cable in	3	1			14 4 2 7				
Area factor	3	0,666667	44.4		Security of				t at utility
Cumilitive No.	3	4	0	0	0	0	0	0	0
Maxm. no. of cable allowed	5	5	5	5	5	5	5	5	5
Distance between Row	95	125	1 1						
Cumilitive distance	95	220	0	0	0	0	0	0	0
Distance of C.G. from bot, face of girder	100		2.25		117.73	<u> </u>		1 12542 1	nanda ti
Eccentricity for non composite section				4,5 45.	805,6555		3 4 4 M 4 4 4 4 4	<u> </u>	<u> </u>
Eccentricity for Composite section					1196.11	· · · · · · · · · · · · · · · · · · ·			
Total no. of Row	3 Tab (1)	the state of			1 g 7 g 5		1.41		
at end Section	Row 1	Row 2	Row 3	Row 4					4, 5
1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 -		1995		[5 B. G.					9 July 19
No. of Cable in	1	1 1	1	1			4144		
Area factor	1	.1	1	0.666667				inge and	19 11 11
Maxm. no, of cable allowed							1-4-5	47 3 44.	- 1879ga - 1
Distance between Row	418	350	350	350					
Cumilitive distance	418	768	1118	1468	0	0	0	0	0
Distance of C.G. from bot, face of girder		. 34 17		<u> </u>	895.3			<u> </u>	<u> </u>
Eccentricity for non composite section		·	44 791 gr		51.23		As the street of the	ay Project	
Eccentricity for Composite section			5 8/2	<u> </u>	294.03		ente de la companya d La companya de la co	atti e satti legi,	977 J.

	Mention	No. of the	in Nation	11.		Ordinates fr	om bottom	Agentaly and the			g Natifier in		
ıble	Row No.	at end	At mid			at fo	llowing dist	ances from	end	oo ka ka jarah ili salah ili s Harapatan ili salah		34 4 A Y	74 s.d.s.
o.	at end	section	section	450	2250	7712.5	14975	22237.5	27700	29500	1364		
i	section	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)		
	.1	418	95	398.88	328.23	170.97	95.00	170.97	328.23	398.88	361.84	29959.29	
?	2	768	95	728,16	580.96	253.29	95.00	253.29	580.96	728.16	650.98	29990.33	30690.3
3	- 3	1118	95	1057.44	833.68	335.61	95.00	335.61	833,68	1057.44	940.13	30043.18	30743.1
4	4	1468	220	1394,12	1121.15	513.53	220.00	513.53	1121.15	1394.12	1251.01	30088.68	30788.6
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	والمرافأ ويدين	122881.
0	3.25	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		(1. f) T.
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0	41 4 38	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0		Ö	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		The state of
0	12 (4.24)	. 0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0	1.1	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	m milita ka	
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N 967 - 1274	
0	4577	22.00	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0.	n 1. sy	0 :	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	Santani ja k	
0	1 1 1	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0,00	0.00		
0	12.01	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	597 0.00		
0	3.37	. 0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00`	0.00	VALUE OF	
st. of c	g from bot.	895.27	117.73	849.24	679.17	300.61	117.73	300.61	679.17	849.24	760.08		
	C1p	881.49	904.62	881.49	904.62	904.62	904.62	904.62	904.62	881.49	881:49	* ett 3 - 12	i de la compa
	C2p	946.51	923.38	946.51	923.38	923.38	923.38	923.38	923.38	946.51	946.51		ed applica
. (C1c	638.70	514.16	638.70	514.16	514.16	514.16	514.16	514.16	638.70	638.70	figure 1	4.3.
(C2c	1189.30	1313.84	1189.30	1313.84	1313,84	1313.84	1313.84	1313.84	1189.30	1189.30	}	医原性炎
(C3¢	849.70	725.16	849.70	725.16	725.16	725.16	725.16	725.16	849,70	849.70	Jacoba	
(C4c	649.70	525.16	649,70	525.16	525,16	525.16	525.16	525.16	649.70	649.70		2011
- 1	Аср	1352038		1352038	698996	698996	698996	698996	698996	1352038	1352038		. Fr. 37
	Acc	1796432		1796432	1143390	1143390	1143390	1143390	1143390	1796432	1796432	1 0.000	
	lcp	3.98E+1		3.98E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	.3.05E+11	3.98E+11	3.98E+11	1	
	Icc	7.28E+1		7.28E+11	5.86E+11		5.86E+11			7.28E+11.	7.28E+11	4	The Mills
	ер	51.23	805.66	97.26	244.21	622.78	805.66	622.78	244.21	97,26	186.43	1	
	ec	294.03	1196.11	340.06	634.67	1013,23	1196.11	1013.23	634,67	340.06	429,23		
177	rp ²	294002	435708	294002	435708	435708	435708	435708	435708	294002	294002		34.25.3
r Loac	top fibre	Allowable	stresses	-2.786	-5.012	-1.632	0.125	-1.552	+5.147	-2.887	1 1	- - 21 - 6-34 8	
se (A) :	s bot fibre	Tension	1.36	-5.186	-10.355	-14.020	-16.107	-14.654	-10.770	-5.372			
sses	comment	Compn.	-16.50	ок-ок	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK			Y
or Load	top fibre	Allowable	stresses	-2.37	-5.50	-5.52	-5.80	-5.48	-5.46	-2.43		50.00	Section 1
se (B)		_	1.47	-4.45	-7.22	-6.91	-6.56	-7.27	-7.48	-4.58		er a s	21.2
esses a		Compn.	-14.00	OK-OK	ок-ок	OK-OK	ок-ок	OK-OK	ок-ок	OK-OK			
or Load		Allowable	stresses	-2.34	-5.68	-6.17	-6.66	-6.12	:-5.64	-2.41			
se (C)			1.47	-4.43	-6.67	-5.18	-4.26	-5.54	-6.93	-4.55			
esses a			-14.00	OK-OK	OK-OK	ок-ок	OK-OK	OK-OK	OK-OK	OK-OK	85 3 157	g 14 Gard	grading the gr
For	top fibre		stresses	-3.99	-7.80	-8,69	-9.33	-8.64	-7.76	-4.06			
l.oad	bot fibre		1.47	-3.52	-5.19	-2.06	-0.48	-2.42	-5.45	-3.65	Marks (C		377
case	commen		-21.00	ок-ок	ок-ок	ок-ок	OK-OK	OK-OK	ок-ок	OK-OK	1.00		1.44
(D)	top of sla		stresses	0.64	-0.29	-1.93	-2.65	-1.93	-0.29	0.64	1		
tresse			0.72	0.78	0.08	-1.11	-1.63	-1.11	0.08	0,78	1	E. J. E.	
at	commer	—	-12.00	OK-NOK		ок-ок	ок-ок	ок-ок	ок-ок	OK-NOK	1		790
or Loa		_	e stresses	-1.17	-3.07	-3.97	-4.46	-3.95	-3.11	-1.20	1		
se (E)			1.47	-2.22	-2.47	0.11	1.32	-0.07	-2.60	-2.28	i jarren e		
						ок-ок	ок-ок	OK-OK	OK-OK	OK-OK	-4 (1) (1) (1)	100	10 July 24 17 N

Yield strength of rebar

414 Mpa,

Allowable strength of rebar

165.6 Mpa,

On Girder top for Girder casting

Stre	Stress at		Total	Rebar	Dia. of	No. of
Top fibre	Bot. fibre	ten. Area	tension	Area regd.		rebar regd.
(Mpa)	(Mpa)	(mm)	(kN)	(mm²)	(mm)	
0.125	-16.107	14.12	0.94	5.69	12	0.05

On Girder bottom for service load

Stre	ss at	Depth of	Total Rebar		Dia. of	No. of
Top fibre	Bot. fibre	ten. Area	tension	Area regd.	Rebar	rebar read.
(Mpa)	(Mpa)	(mm)	(kN)	(mm²)	(mm)	
-4.455	1,319	417.52	180,17	1087.98	20	3.46

Yield strength of rebar

276 Mpa.

Allowable strength iof rebar

124.2 Mpa,

ANALYSIS FOR SHEAR

Section	Distance		4.50 E. 124 F. 1	No. activity	· 133.13.4.	Shear due	to	Company of the		Tanga at a car
	from end	Self wt	Cross	Non	Composi-	Live Load	Live Load	Sidewalk	Total	Factored
100			Girder	Composi-	te dead	with	with Impa-	Live	Dead	Design
			(Non-Co-	te dead	load	Impact	ct when	Load	Load	Shear
			mposite dead load	load			wheel on Sidewalk			J
		V _o			V _{DC}				Bry Try	Vυ
		(a)	(b)	(c)	(d)	(e)	(1)	(9)	*	9.0
4.1.4.4.4.4	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
2	450	261.16	11.824	164.282	139.975	253.74	0.00	0.00	577.24	1301.29
2a	1364	232.04	11.824	153,945	-131.167	245.52	0.00	0.00	528.98	1220.69
3	2250	209.59	11.824	143.924	122.629	237.52	0.00	0.00	487.96	1150.00
4	7712,5	119.62	11.824	82.141	69.988	187.53	0.00	0.00	283.57	775.78
5	14975	0.00	11.824	0.000	0.000	118.87	0.00	0.00	11.82	273.44
6	22237.5	119.62	11.824	82.141	69.988	187.53	0.00	0.00	283.57	775.78
7	27700	209.59	11.824	143,924	122.629	237,52	0.00	0.00	487.96	1150,00
_ 8	29500	261.16	11.824	164.282	139,975	253,74	0.00	0.00	577.24	1301.29

Section	Distance	Diameter	No. of	Effective	f2p	fo	Mcr	Vci	Vp	Vcw	Vc	Spacing
	from	of	Leg	depth for	(stress in	(tensile	(moment	34.	100		(shear	Read.
	end	stimup		Shear	the conc.	stress at	causing		1.75		strength	
				(d)	due to Pe)	bot due to	flexural		1 1 1 1 1 1 1 1	1	provided	A 9 E
4.5					A. J. J. A. A.	self wt.)	crack)				by conc.)	
					(N/mm²)	(N/mm²)	(kN-m)	(kN)	(kN)	(kN)	(kN)	(mm)
2	450	12	2	1462	-4.461894	-0.0077	3114.91	479905	469.91	1282.95	1282.95	337
2a	1364	12	2	1462	-5.438445	0.5290	3299.64	3913.31	455:45	1268.55	1268.55	337
3	2250	12	2	1462	-9.584544	1.2634	3716.70	2319.64	424.01	1496.91	1496.91	337
4	7712.5	12	2	1527	-14.46102	3.9895	4425.96	743.94	335.97	1448.91	743.94	344
5	14975	12	2	1710	-16.78044	5.3064	4756.62	293.60	224.95	1467.43	293.60	364
- 6	22237.5	12	2	1527	-14.82281	3.9895	4545.29	758.31	344.37	1471.82	758.31	344
7	27700	12	2	1462	-9.842597	1.2634	3801.82	2365.96	435.43	1523.47	1523,47	337
8	29500	12	2	1462	-4.585219	-0.0077	3166.70	487891	482.90	1304.30	1304.30	337

Section	Distance	Effective	Contact	Factored	Allowable	Comment	Diameter	No. of	Minimum
	from	depth for	surface	Design	shear		of	Leg	spacing
	end	Shear	width	Shear	strength		stirrup	tag Egyl	required
	(mm)	(d) (mm)	(mm)	(kN)	(∳Vnh) (kN)		(mm)		
2	450	979	1066	1301.29	2263	О.К.	12	2	(mm) 169.75
3	2250	1149	1066	1150.00	2656	0.K.	12	2	169.75
4	7712.5	1527	1066	775.78	3532	0.K.	12	2	169.75
5	14975	1710	1066	273,44	3954	0.K.	12	2	169.75
6	22237.5	1527	1066	775,78	3532	O.K.	12	2	169.75
· 7 :	27700	1149	1066	1150.00	2656	O.K	12	2	169.75
- 8	29500	979	1066	1301.29	2263	O.K.	12		169.75

ELONGATION OF CABLE

Outside jack cable length (assumed), I; =

700 mm, 70 mm,

Dead anchor width, I. =

Cable stress at jack (after jack loss), fi' = 1370 N/mm²,

Cable stress at dead anchor, f_{LP} ≈

1258 N/mm²,

Average cable length of anchor to anchor, L_p = Average elongation of cables by one side jacking, $\Delta L_p =$ 30004 mm. 210 mm,

ANCHORAGE ZONE DESIGN

Minimum longitudinal extent of anchorage zone Maximum longitudinal extent of anchorage zone Selected longitudinal extent of anchorage zone 1066 mm, 1599 mm, 1500 mm,

- DESIGN of GENERAL ZONE

angle of inclination of the resultant of the tendon

5.93 dea.

or endie or	incente don't	0, 100 10000	Dill Di 1010 10			0.00	weg,	<u> </u>	<u> </u>		
Size of	Thickness	Diameter	Effective	Factored	Total fact-	C/c	Longitudi-	Correction	Concrete	Allowable	Comment
bearing	of bearing	of cone	bearing	tandon	ored tend-	spacing of	nal extent	factor	compres-	Concrete	
plate	plate		area	load	on load	anchorage	of local	1 3 1 4 3	sive stress	compres-	
		50.00	2/3/201	11.4		10.7	zone		257 5 7	sive stress	115
	1.5	10 m 10 m	(Ab)	and said		100,000	(lc)	(k)	(fca)	975 1	200
(mm)	(mm)	(mm)	(mm²)	(kN)	(kN)	(mm)	(mm)		(Mpa)	(Mpa)	
230	35	152	49581.69	1947.273	7140	350	264.5	1.271	16.85	21	- O.K.

DESIGN FOR VERTICAL DIRECTION

DESIGN FO	NZ ACICIIO	WE DIVE'C	HOIT .						11 11 11 11		
Cable no.	ī	2	3	. 4	0	0	0	0	D	0	0
Eccentricit y	528,51	178.51	171.49	521.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00
dburst	658.44	827.68	831.07	661.83	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Reinforcem	ent distribu	tion distanc	е	15000	= -	1861.895	mm,	Maximum	ag 11 ¥	2742	mm,
Bursting for	ce, Tourst	April 1985		: 1991	್ ≢ಿ	1929.157	kN,				
Area of rein	forcement	required for	bursting for	rce .		5482.117	mm²,	the first and		11 () 1 () 1 ()	A MARKET
Diameter of	bursting s	tirrup	to the first	100	= :	16	mm,		1000	and the state	
Total no of	2 legged st	irrup require	ed		=	13.63	nos.	100			

DESIGN FOR HORIZONTAL DIRECTION

	O	J						
Eccentri-	Lateral	Angle of	Bursting	Bursting	Area of	Dia. Of	No. of	Rebar
city	dimension	tendon	distance	force	rebar	rebar	rebar regd	distribution
		19 1 4			required			distance
(e)	(h)	(α)	(dburst)	(Tourst)		15/2	1.00.1	11.47.13
(mm)	(mm)	(deg)	(mm)	(kN)	(mm²)	(mm)	1.27.2.27	(mm)
0	711	0	355.5	1207.574	3431.582	16	8.53	888.75

DESIGN FOR EDGE TENSION

010.011	0.1.2002		
Spalling	Area of	Dia. Of	No. of
force	rebar	rebar	rebar regd
(Tspal)	required	1	1.3443
	1 211		
(kN)	(mm²)	(mm)	1.4
142.8	406	12	0.03

DEFLECTION CALCULATION

	200	Deflection	on due to	200		Instanta-	Deflection	Long-time		
Initial	Effective	Self wt.	Self wt.	Non-co-	Compos-	neous	at	Deflection	Live	Live
Prestress	Prestress			mposite	ite dead	Deflection	erection		load	load
		(at initial	(after 28	dead load	load				2.5	rotation
		period)	days)	W. 1						at suppor
	4.7	100	1.2					9.4		1.0
(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(rad.)
45.2	36.8	-18 322	-16.96	-13.2	-5.1	26.9	47.5	-5.4	-7.3	0.001234

DESIGN OF LEFT EXTERIOR PRESTRESSED GIRDER m:curb/rail/pedestal equally distributed on girde DESIGNED BY: PRASANTA KUMAR BHOWMIK

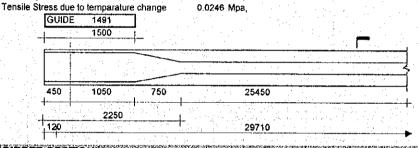
Total Length of one Girder 29950 mm, Distance of center of Bearing from end of Girder 450 mm. Span of Girder 29050 mm, Distance of center of Diaphragm from end of Girder 350 mm, C/c. spacing between girders 2400 mm, Effective width of sidewalk 1200 mm, Fraction of wheel load on Girder 0.559 Fraction of wheel load on girder when wheel over sidewalk 1.651 Live Load Magnitude HS 20 Put 1.0 or 2.0 for Deck type Mention AASHTO or WASHINGTON Pattern AASHTO Mention Type Mention end section 1 or 2? 2 Anchorage indent 120 mm,

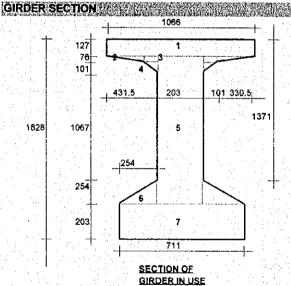
DIAPHRAGM DETAILS Total no. of interior diaphragm reqd Total no. of interior diaphragm provided Width of diaphragm 300 mm, Depth of end diaphragm 1887 mm, Depth of interior diaphragm 1625 mm. Wt. of exterior diaphragm 13.39 kN, Wt. of interior diaphragm 11.82 kN,

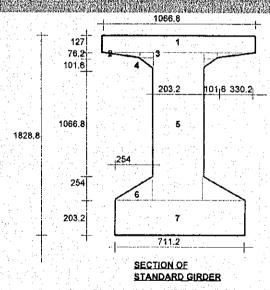
NOTE: In designing this girder following references are followed:

- 1. AASHTO 16th edition, 1996.
- 2. Design of Prestressed Concrete, 2nd edition, by : Arthur H. Nilson.
- Design of Prestressed Concrete Structure, 3rd edition.
- by : T. Y. Lin & Ned H. Burns.

All notations are followed from reference-2

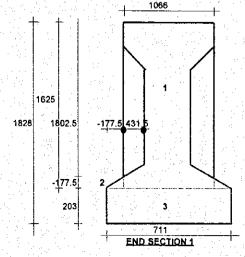


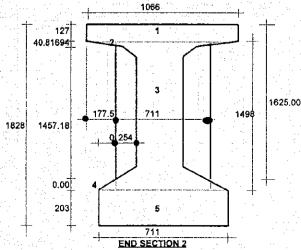


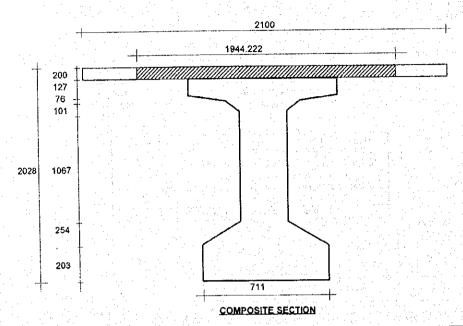


Effectiveness ratio of non-composite section Effectiveness ratio of composite section

0.521613 0.533982 Note: For effectiveness ratio < 0.45, indicate too heavy a section For effective ness ratio > 0.55, indicate an excess slender section.







Segment	Area	Distance of 0	CG from	Moment of	Inertia of	Radius of	Segment	Area	Distance of (CG from	Moment of		Radius of
		Top	Bottom	Components	Block	Gyration	A. N	and the state of	Top	Bottom	Components	Block	Gyration
	3 77	Cip	C _{2P}	lo lo	lc	r ²		1.15	C1P	C2P	10	lc	12
	mm²	mm	mm	mm ⁴	mm ⁴	mm²		mm2	mm	mm	mm4	mm4	mm2
1	135382			1.82E+08			: 1	135382			1.82E+08	2.59	
	12559	1		4030044			- 2	3623	[]		335287		
	7676	1		3694715	1	1	3	1065078	881.49	946.51	1.99E+11	3.98E+11	294002
	5100.5	904.62	923.38	2890567	3.05E+11	435707.6	4	0		1.77	0		1.5
	304094	{ ***	***	5.69E+10	2.5	1.00	5	144333	1	1 1 52	4.96E+08		
	32258	┨		1.16E+08	1 6		Total	1352038	1		2 2 11		

Weight per linear metre = 16.47 kN/m.

Segment	Area	Distance	of CG of Co	mposite secti	on from	Moment of I	nertia of	Radius of	
		Top of precast section	Bot. of precast section C _{2C}	Bottom of Slab C ₄₀	Top of Slab C _{3C}	Components	Block	Gyration r ²	
	mm²	mm	mm	mm	mm	mm ⁴	mm ⁴	mm²	
Precast Section	698996					3.05E+11			
Eqv. Slab	388844.4	545,5206	1282.479	556 5206	756.5206	1,30E+09	5.64E+11	518080	
Total	1087840				5 19		10.0		

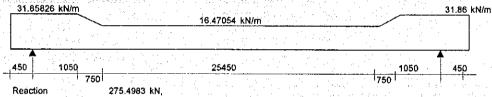
Segment	Area	Distance	of CG of Cor	nposite secti	on from	Moment of	Inertia of	Radius of
•		Top of precast section C _{1C}	Bot. of precast section C _{2C}	Bottom of Slab C ₄₀	Top of Slab C _{3C}	Components 1 ₀	Block Ic	Gyration L ²
1 1	mm²	mm	mm	mm	mm	mm ⁴	mm ⁴	mm²
Precast Section	1352038					3.98E+11		
Eqv. Slab	388844.4	662,2677	1165.732	673,2677	873.2677	1.30E+09	6.96E+11	399960.8
Total	1740882							

Weight per linear ft. = 41.02 kN/m.

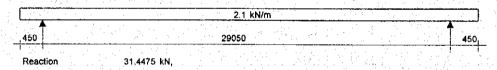
SUMMERY OF SECTION PROPERTIES AT DIFFERENT LOCATIONS

Section			1 1	2	3	4	5	6	7	8	9
Distance from end		(mm)	0	450.00	2250	7712.5	14975	22237.5	27700	29500	29950
Total area of duct		(mm)	13273.23	13273.23	13273,23	13273,23	13273.23	13273.23	13273.229	13273,23	13273.23
Dist.of c.g. of duct from	i bot.	(mm)	895,27	849.24	679,17	300.61	117.73	300,61	679.17	849.24	895.27
Duc pron	Аср	(mm)	1338765	1338765	685723	685723	685723	685723	685723	1338765	1338765
8 50 cg	CiP	(mm)	880,99	880.53	899.89	892,56	889.02	892,56	899.89	880,53	880,99
Duct deducted properties of noncomposite section	C _{2P}	· (mm)	947.01	947.47	928.11	935,44	938.98	935.44	928.11	947.47	947.01
ucted as of sosite	lc	(mm ⁴)	3,97E+11	3.97E+11	3.04E+11	2.99E+11	2.96E+11	2.99E+11	3.04E+11	3.97E+11	3.97E+11
ត÷៥	r ²	(mm²)	296888	296819	442937	436335	431081	436335	442937	296819	296888
8-0	Аср	(mm²)	135203B	1352038	698996	698996	698996	698996	698996	1352038	1352038
onco se	C _{1P}	(mm)	881.49	881,49	904.62	904.62	904.62	904.62	904.62	881.49	881,49
Duct grouted properties of noncomposite section	C _{2P}	(mm)	946.51	946.51	923,38	923.38	923,38	923.38	923,38	946.51	946.51
uted as of osite	lc lc	(mm ⁴)	3.98E+11	3.98E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3,05E+11	3.98E+11	3.98E+11
% → □	, p²	(mm²)	294002	294002	435707.6	435707.6	435707.6	435707.6	435707.6	294002.2	294002.2
Duct of c	Acc	(mm²)	1740882	1740882	1087840	1087840	1087840	1087840	1087840	1740882	1740882
88	C _{1c}	(mm)	662.27	662.27	545.52	545,52	545.52	545.5	545.5	662.3	662.3
grouted omposit	C _{2c}	(mm)	1165.73	1165.73	1282.48	1282.48	1282,48	1282.5	1282.5	1165.7	1165.7
	C _{4c}	(mm)	673.27	673.27	556.52	556.52	556.52	556.5	556.5	673.3	673,3
se of	C _{3c}	(mm)	873.27	873	756.5	756.5	756.5	756.5	756.5	873.3	873.3
properties e section	lc	(mm⁴)	6.96E+11	6.96E+11	5.64E+11	5.64E+11	5.64E+11	5.64E+11	5.64E+11	6.96E+11	6.96E+11
⊃ li čs	r ²	(mm²)	399960.8	399960,8	518080.8	518080.8	518080.8	518080.8	518080.8	399960.8	399960,8

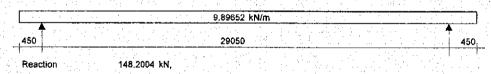
LOAD DIAGRAM FOR SELF WEIGHT



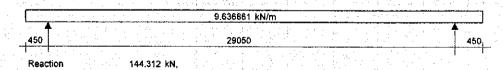
LOAD DIAGRAM FOR FALSE-WORK DEAD LOAD



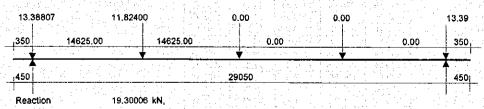
LOAD DIAGRAM FOR NON-COMPOSITE DEAD LOAD



LOAD DIAGRAM FOR COMPOSITE DEAD LOAD



LOAD DIAGRAM FOR CONCENTRATED LOAD FROM DIAPHRAGM



ANALYSIS MOR MOMENT

Section	Distance	2.3	27 1 1 2 1 1 1		٨	Noment due	to	1.44		. Tall v	Total	Total
	from end	Self wt	Cross	Non	Composi-	Live Load	Live Load	Sidewalk	Total	Falsework	Dead	Factored
		1.0	Girder	Composi-	te dead	with .	with Impa-	Live	(Non-Co-	(Non-Co-	load	Moment
			(Non-Co-	te dead	load	Impact	ct when	Load	mposite	mposite	Moment	
			mposite	load			wheel on	454	dead load)	dead load		
			dead load				Sidewalk					
		M _o		7. A.	M _{DC}	13 M			Moe		2.0	
1.0		(a)	(b)	(c)	(d)	(e)	(f)	(g)	(b+c)	1000	1 4.45	
	(mm)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)	(kN-m)
1	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	450	-3.23	-1.34	-1.00	0.98	0,00	0.00	0.00	-2.34	-0.21	-6.54	-8.50
2a	1364	222.17	4.06	126.25	122.94	87.31	258.03	44,32	130.31	26.79	475,42	953,49
3	2250	416.70	9.30	241.71	235.37	165.91	490.36	84.53	251.01	51.29	903,08	1811,47
4	7712.5	1315,84	41.60	781.97	761,45	519.58	1535,64	272.70	823.57	165.93	2900.86	5767.45
5	14975	1750.20	84.53	1042.96	1015.60	666.76	1970.64	363.60	1127.49	221.31	3893.29	7623.11
6	22237.5	1315.84	41.60	781.97	761.45	519.58	1535.64	272.70	823.57	165,93	2900.86	5767.45
7	27700	416.70	9.30	241.71	235.37	165.91	490.36	84.53	251.01	51.29	903.08	1811.47
8	29500	-3.23	-1.34	-1.00	-0.98	0.00	0.00	0.00	-2.34	-0.21	-6.54	-8.50
9	29950	0.00	0.00	0.00	0,00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

No. of cable read

LOSSES

 f_{ps} 144.20 rictional loss per unit length 14.20246 kN/m, For 8mm Anchorage draw in, x(dist. Of anchorage loss zone) 21.7313 mm, ΔP_A 617.2771 kN,

Prestress force after friction losses (before elastic shortening) :Tensioning end

Tensioning end 5333 kN, Seating loss zone 5641 kN,

Midspan 5545 kN, Dead end 5525 kN,

SUMMERY OF INITIAL LOSSES

			·	4 4 5	4.4.4			
Section	Distance	Jacking	Losses	Prestress	Conc. Str-	Loss	Final	Final
	from end	Force	due to	Force	ess at the	due to	initial	initial
			friction	after	level of	elastic	ioss	prestress
	1. 1.		and anc-	friction	steel	shortening	2.04	force
			horage	loss	centroid at		11. 20	(after
4.1.34			pull-in	(before	sec.of ma-		(after	immediate
				elastic sh-	xm, mom.		immediate	losses)
				ortening)	(f _{cir})		losses)	(Pi)
	(mm)	(kN)	(kN)	(kN)	(mPa)	(kN)	(kN)	(kN)
1	0		617	5333	3.935	59.82	677	5273
2	450]	611	5339	4.030	61.27	672	5278
2a	1364]	598	5352	4.269	64.90	663	5287
3	2250		585	5365	8,189	124.50	710	5240
4	7712.5	5950	508	5442	11.551	175,61	683	5267
5	14975]	405	5545	14.345	218.10	623	5327
6	22237.5]	316	5634	12.049	183.19	499	5451
7	27700]	393	5557	8.493	129.13	523	5427
8	29500		419	5531	4.175	63.47	482	5468
9	29950	· · · · · · · · · · · · · · · · · · ·	425	5525	4.076	61.98	487	5463

SUMMERY OF TIME-DEPENDENT & FINAL LOSSES

Section	Distance	Conc. Str-	Loss	Loss	Loss	Final	Total	Loss	Final
4 to 1	from end	ess at the	due to	due to	due to	Time	loss	in	effective
		level of st-	creep	shrinkage	relaxation	dependent		percent	pre-stress
	40.5	eel cg. Du-		1.75	of pre-	loss			force
		e to dead			stressing				
		loads exc-			steel				
1.5		ept self wt.	100	100	1.10			100	
		(f _{cds})	1407	200				V-	(Pe)
	(mm)	(mPa)	(kN)	(kN)	(kN)	(kN)	(kN)	J. 18 J.	(kN)
1	0	0.000	205.06	100	317.59	678.34	1355.44	22.78	4594.56
2	450	-0.002	210.06		317.92	683.69	1355.85	22.79	4594.15
2a	1364	0.148	217.97		318.79	692.46	1355,26	22.78	4594.74
3	2250	0.521	410.92]	260.13	826.75	1536.57	25.82	4413.43
4	7712.5	2.761	518.01		241.54	915,25	1598.61	26.87	4351.39
5	14975	4.429	612.94	155.70	236,51	1005.14	1627.84	27.36	4322.16
6	22237.5	2.761	543,99		290,89	990,58	1489.60	25.04	4460.40
7	27700	0.521	426.79	1	312.68	895.17	1417.71	23.83	4532.29
: 8	29500	-0.002	217.61		373.11	746.42	1228.87	20.65	4721.13
	29950	0.000	212.44		372.82	740.96	1228.30	20.64	4721.70

EFFECT OF DIFFERENTIAL SHRINKAGE

Ultimate shrinkage co-efficient, ϵ_{SHU} 0.0008 Correction by humidity, (RH = 70) . FSH,H 0,7 Assumed day of slab casting after girder concrete placed, t 60 days Shrinkage of girder at t days, $\epsilon_{\text{SH,t}}$ 0.632 Differential (remaining) shrinkage, $\,\epsilon_{\text{SH,D}}$ 0.000206 Ultimate creep coefficient for girder, ϕ_{cgu} 2.6 Ultimate creep coefficient for slab, φ_{csú} 28 Correction by humidity for creep coefficient, (RH = 70), For 0.801 Remaining creep of girder after t days, ϕ_{cg} 0.961267 Remaining creep of stab, ϕ_{cs}

					like mid sec			ertial Shrin	acerus de
					1 11	Gi	rder	S	lab
m	В	. C		Ψсд	φα	V _{SH}	M _{SH}	V _{SH}	: M _{SH}
a tagan a	19 m	(m)	(m2)	Fat. 1	1. (4.1)	(kN)	(kN-m)	(kN) .	(kN-m)
253.798	254.798	-27.267	5.227	0.961	2.243	413,201	44,218	249.907	26.744

	Gir	der	Slab			
	Bottom	Тор	Bottom	Тор		
Stress at concrete due to differential shrinkage	0.676	-1.833	0.674	0.524		

Ho	izontal Sne	er & Bendir	A	Service Control of the Control	A STATE OF THE PARTY OF THE PARTY OF	C. C. Con. 10034 79	due to Diffe	rential Shrin	kage
oki elek Ni	47.4	indian de de	A. A	all section	ike end sed	tion -			Pales Sales
			war 🚅 🗓		A 1	Gi	rder	Š	lab
, m	27 S			Ψαρ	Ψα	V _{SH}	: M _{SH}	V _{SH}	M _{SH}
44.70		(m)	(m2)			(kN)	(kN-m)	(kN)	(kN-m)
331.252	332.252	-35,998	6.257	0.961	2.243	528.518	57.263	319.651	34,633

	1981	Gir	der	S	lab
		Bottom	Тор	Bottom	Тор
Stress at concre differential sh		0.855	-1.551	0.828	0.700

DESIGN FOR MOMENT AT MID SECTION

Total Jacking tension canbe taken by supplied cable (after jack loss) 5993,99 kl

1 OTDI SECVI	nă rension c	Allue lakell	by supplied	canie (alle	H JOUR 1033)		3953.59	NFT.			The second second	<u></u>
11.674	Initial T	ension		Total	Jacking	Effective	(Row) ρ	f _{ps}	а	Ultimate	Ultimate	Comment
Maximum	Maximum	Can be	Actual	no. of	Tension	Tension	30 m	. 186		Design	Moment	
(1)	(2)	taken by	at mid	Cable	(after	at mid				Moment	Capacity	
		supplied	Section	reqd.	Jack	section						
4.5		Cable			loss)			1981 In.				
(kN)	(kN)	(kN)	(kN)	1075	(kN)	(kN)		(N/mm ²)	(mm)	(kN-m)	(kN-m)	
6552	5424	5654	5327	3.64	5950	4322	0.001083	1800.012	145.98	7623.112	13633.27	ÖK

PRESTRESSING REINFORCEMENT DETAILS

١	Total no.	Total no.	Area	Total no.	Dia	Awar da	Area of		Minimum	Outer	a - C/	c. Spacing a	of
Í	of cable	of duct	factor	of strand	of one	One	One cable	Total	inside	diameter		Ducts	
1	reqd.	used		in one	strand	Strand		Cable	diameter	of Duct	Reqd.	Allowed	Allowed
1			2.1	cable				(Ap)	of Ducts	Provided		mid	end
- [(nos)	(nos)			(mm)	(mm2)	(mm2)	(mm2)	(mm)	(mm)	(mm)	(mm)	(mm)
	3.53	4	3.666667	12	12.7	98.7	1184.40	4342.80	54.92	65	103.1	125	125

Reinforcement Index Cracking Moment 0.064955

OK against maxm. reinforcrment.

3379.928 kN-m,

OK against minm. reinforcement.

8024.59 6.20

STRESSES AT DIFFERENT LOCATION OF MID-SECTION OF GIRDER

(A. V.) A. (A. (A. (A. (A. (A. (A. (A. (A. (A.					2.00	in risk six six s	All the state	1690 1922 A 14 B						
14 (14)		Initial 1	ension	la sinte		ji bili shipi ta	Addition to	hanan ger	Effective	e Tension	100		The section	: : .
	Pi (a	lone)	Pi	Mo	P.+M	o+M _{do}	P.+M.+	M _{do} +M _{dc}	P.+M.+M	l _{ap} +M _{oc} +M,	0.5(Pe+Me	+M _{dp} +M _{dc})	At top of	At bot, of
			(A)	(3)	(6	C)``	+SH	(D)	+MI	(E)	Deck	D eck
	Тор	Bottom	Тор	Bottom	Тор	Bottom	Тор	Bottom	Тор	Bottom	Тор	Bottom	Slab	Slab
	(N/mm²)	(N/mm²)	(N/mm ²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm ²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm ²)	(N/mm ²)
Stress	5.13	-20.63	0.13	-16.11	-5.02	-7.32	-6.00	-5.01	-7.82	-3.49	-3.31	-1.31	-1.54	-0.84
All. stress			1,36	-16,50	-14.00	1.47	-14.00	1.47	-21.00	1.47	-14.00	1.47	-12	0.72
Comment	4, 5,8		OK	OK	OK	OK	OK:	OK	OK	OK	OK:	OK	OK	ΟK

CHECK AT MID SECTION FOR EXTERIOR GIRDER WHEN WHEEL ON SIDEWALK

5 S S	Art Style	Eff	ective Tens	ion		Asia da
	P _e +M _o +M +SH	_{do} +M _{de} +M _i (D)	0.5(P _e +M _o +Ml	+M _{dp} +M _{dc}) (E)	At top of Deck	At bot, of D eck
	Top (N/mm²)	Bottom (N/mm²)	Top (N/mm²)	Bottom (N/mm²)	Slab (N/mm²)	Slab (N/mm²)
Stress	-9.09	-0.52	-4.57	1.66	-3.16	-2.03
Ali. stress	-31.50	2.21	-21. 0 0	2,21	-18	1.07
Comment	OK	OK	OK	OK	OK	OK

Factor of safety against cracking

5.0

CABLE LAYOUT

Total no. of Row at mid Section	Row 1	Row 2							
2									
No. of Cable in	3	1			44.4				
Area factor	3	0.666667		20.00					
Cumilitive No.	3	4	0	0	0	0	0	0	0
Maxm. no. of cable allowed	5	5	5	5	5	5	5	5	5
Distance between Row	95	125						44	
Cumilitive distance	95	220	0	0	0	0	0	0	0
Distance of C.G. from bot, face of girder			3.5		117.73	A ROSE NO	<u>a na katani</u>	4 g 4 d	Take et al., and
Eccentricity for non composite section					805,6555	4. 4		1 4 4 4	1
Eccentricity for Composite section			4000		1164.75	y .	<u> Parkana</u>	1,200	
Total no. of Row				1.1					
at end Section	Row 1	Row 2	Row 3	Row 4	73. Page 1				
1 1887 4 11 18 18 18 18 18 18 1		100	1.47						1
No. of Cable in	1	. 1	1.5	1	12.00		9.7	100	
Area factor	1	1	1	0.666667	100000		The late		
Maxm, no, of cable allowed	100 1 Mil	1.4 900	: .		2000		4 4 4		15.4
Distance between Row	418	350	350	350	2.0	125, 54	180 180		100
Cumilitive distance	418	768	1118	1468	0	0	0	0	0
Distance of C.G. from bot, face of girder			el e e	5 6.00	895.3	100	, e i a i a 1,271	22 J 2 2 Z	
Eccentricity for non composite section	1	- V		1	51.23	- 11541 Y		1 1	
Eccentricity for Composite section	1		y satisfaction and		270.46		B. Carrier	1000	ALC: 18 19 19 19

STRESSES AT DIFFERENT LOCATION OF GIRDER

4 3	Mention			<u> </u>		Ordinates for					
Cable	Row No.	at end	At mid		<u> </u>	at fo	llowing dist	ances from			
No.	at end	section	section	450	2250	7712.5	14975	22237.5	27700	29500	1364
100	section	(mm)	(mm)	(mm):	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
1	1	418	95	398.88	328.23	170.97	95.00	170.97	328.23	398.88	361.84
2	2	768	95	728.16	580.96	253.29	95.00	253.29	580,96	728.16	650.98
3	3	1118	95	1057.44	833.68	335.61	95.00	335.61	833.68	1057.44	940.13
4	4	1468	220	1394.12	1121.15	513.53	220.00	513.53	1121.15	1394.12	1251.01
0	-	0	0	0.00	0.00	0.00	0.00	0,00	0,00	0.00	0.00
0 /		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0		0	ō	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	 	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0		0	ő	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0		0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	 	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	 		0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	 	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	1	0								0.00	0.00
0 °	 	0	0.0	0.00	0.00	0.00	0.00	0.00	0.00		
0		0:	0	0.00	0.00	0.00	0.00	0.00	0,00	0.00	0.00
0	1	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	1 1 1 1	0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	9.54	0	0	0.00	0,00	0.00	0.00	0.00	0.00	0.00	0.00
0		0	0	0.00	0.00	0.00	0.00	0.00	0,00	0.00	0.00
	g from bot.	895.27	117.73	849.24	679.17	300.61	117.73	300.61	679.17	849.24	760.0
	>1p	881.49	904.62	881.49	904.62	904.62	904,62	904.62	904.62	881.49	881.4
	C2p	946.51	923.38	946.51	923,38	923.38	923.38	923.38	923.38	946,51	946.5
(C1c	662.27	545.52	662.27	545.52	545.52	545,52	545.52	545.52	662.27	662.2
(32c	1165.73	1282.48	1165.73	1282.48	1282.48	1282.48	1282.48	1282.48	1165.73	/1165.7
20 NO C	.3c	873.27	756.52	873.27	756.52	756.52	756.52	756.52	756.52	673,27	873.2
10.0	Ç4c	673.27	556.52	673.27	556.52	556.52	556.52	556.52	556,52	673.27	673.2
	Аср	1352038	698996	1352038	698996	698996	698996	698996	698996	1352038	135203
	Acc	1740882	1087840	1740882	1087840	1087840	1087840	1087840	1087840	1740882	174088
	lcp	3.98E+11		3.98E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.05E+11	3.98E+11	3.98E+
	lcc	6.96E+11		6.96E+11	5.64E+11	5.64E+11	5.64E+11	5.64E+11	5.64E+11	6,96E+11	6,96E+
	ep	51.23	805.66	97.26	244.21	622.78	805.66	622.78	244.21	97.26	186.4
	ec	270.46	1164.75	316.49	603.31	981.87	1164.75	981.87	603.31	316.49	405.6
	Γρ ²	294002	435708	294002	435708	435708	435708	435708	435708	294002	29400
				1	1.000	4 4 4 4 4 4		and a confidence of	A	and the state of the state of	
or Load			4 1 1 4	-2.786	-5.012	-1.632	0.125	-1,552	-5.147	-2.887	French.
se (A) s			1.36	-5.186	-10.355	-14.020	-16.107	-14.654	-10.770	-5.372	
esses a			-16.50	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	
or Loac	top fibre	Allowable		-2.37	-5.35	-5.00	-5.02	-4.95	-5.31	-2.44	lovi i
se (B)	s bot fibre	Tension	1.47	-4.45	-7.38	-7.43	-7.32	-7.79	7.64	-4.57	
esses a	t comment	Compn.	-14.00	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK]
or Load	top fibre	Allowable	stresses	-2.34	-5.55	-5.71°	-5.98	-5.67	-5.51	-2,41]
ase (C)		-1	1.47	-4.43	-6.82	-5.67	-4.98	-6.03	-7.08	-4.55	1
esses a		_	-14.00	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	1 //
For	top fibre	<u> </u>		-3.90	-7.40	-7.55	-7.80	-7.51	-7.35	-3.96	104.5
Load	bot fibre		1.47	-3.57	-5.92	-4.31	-3.46	+4.68	-6.18	-3.70	
case	comment		-21.00	OK-OK	OK-OK	ок-ок	OK-OK	OK-OK	OK-OK	OK-OK	1
				0.70	0.03	-1.07	-1.57:	-1.07	0.03	0.70	t en in
(D)		Allowable		0.83	0.03	-0.50	-0.86	-0.50	0.03	0.83	1
stresse:		Tension	0.72								4
at	commer		-12.00	OK-NOK		OK-OK	OK-OK	OK-OK	OK-OK	OK-NOK	
For Loa				-1.17	-2.80	-3.11	-3.31	-3.09	-2.84	+1.21	4 3 % - 3
ase (E)			1.47	-2.21	-3.11	-1.90	-1.31	-2.09	-3.24	7/i +2.28	40000
resses	at commen	l Compn.	-14.00	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	OK-OK	

29959.29 30659.29 29990.33 30690.33 30043.18 30743.18 30088.68 30788.68 122881.5

TENSION REINFORGEMENT

Yield strength of rebar

414 Mpa,

Allowable strength of rebar

165.6 Mpa,

On Girder top for Girder casting

Stre	Stress at		Total	Rebar	Dia. of	No. of
Top fibre	Bot, fibre	ten. Area	tension	Area regd.	Rebar	rebar rego.
(Mpa)	(Mpa)	(mm)	(kN)	(mm²)	(mm)	
0.125	-16.107	14.12	0.94	5.69	12	0.05

On Girder bottom for service load

Stre	Stress at		Depth of Total Rebar		Dia. of	No. of
Top fibre	Bol. fibre	ten. Area	tension	Area reod.	Rebar	rebar regd.
(Mpa)	(Mpa)	(mm)	(kN)	(mm²)	(mm)	23.1
-3,306	-1.311	0.00	#VALUE!	#VALUE!	20	#VALUE!

DESIGN FOR SHEAR

Yield strength of rebar

276 Mpa,

Allowable strength iof rebar

124.2 Mpa,

ANALYSIS FOR SHEAR

Section	Distance			The section		Shear due	to		12 44 34	rug in estimation
1 1 2	from end	Self wt	Cross	Non	Composi-	Live Load	Live Load	Sidewalk	Total	Factored
4.00	1		Girder	Composi-	te dead	with	with Impa-	Live	Dead	Design
		**	(Non-Co-	te dead	load	Impact	ct when	Load	Load	Shear
		1.4	mposite	load	19		wheel on	1 1		34.4
	[[4 + 4 + 1		dead load	2.0	1 // 1 / 1	200	Sidewalk	A - 5		
A 1		Vo		454	V _{DC}			4 T. O. C.		Vυ
2000	9.5 (4)	(a)	(b)	(c)	(d)	(e)	(f)	(g)	1.5	A ALMANA
1945	(mm)	. (kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
2	450	261.16	5,912	143.747	139.975	98.99	292.56	0,16	550.80	1096.37
2a	1364	232.04	5.912	134.702	131.167	95.78	283.08	0.15	503.82	1022.97
3	2250	209,59	5.912	125.933	122.629	92.66	273,85	0.14	464.06	959.29
4	7712.5	119.62	5.912	71.873	69.988	73.16	216.23	0.09	267.39	628.70
5 :	14975	0.00	5.912	0.000	0.000	46.37	137.06	0.04	5.91	185.86
6	22237.5	119.62	5.912	71.873	69.988	73.16	216.23	0.09	267.39	628.70
7	27700	209.59	5.912	125.933	122.629	92.66	273.85	0.14	464.06	959.29
8	29500	261.16	5.912	143.747	139.975	98.99	292.56	0.16	550,80	1096,37

Section	Distance	Diameter	No. of	Effective	f2p	fo	Mcr	Vci	Vp	Vcw	Vc	Spacing
1	from	of	Leg	depth for	(stress in	(tensile	(moment			1 1 1 Mg 4 3	(shear	Reod.
	end	stirrup		Shear	the conc.	stress at	causing	1 1 4			strength	100
12.00	1,81	10000		(d)	due to Pe)	bot due to	flexural				provided	
				100		self wt.)	crack)	1000			by conc.)	
14 4		f			(N/mm ²)	(N/mm²)	(kN-m)	(kN)	(kN)	(kN)	(kN)	(mm)
2	450	12	2	1462	-4.461911	-0.0077	3114.92	546451	469.92	1282.95	1282.95	337
2a	1364	12	2	1462	-5.438022	0.5290	3299.46	3900.21	455.41	1268.49	1268.49	337
3	2250	12	2	1462	-9.581729	1.2634	3715.78	2306.99	423.89	1496.62	1496.62	337
4	7712.5	12	2	1527	-14.44142	3.9895	4419.50	726.50	335.51	1447.67	726.50	344
. 5	14975	12	2	1710	-16,74092	5.3064	4743.58	289.94	224.42	1465.38	289.94	364
6	22237.5	12	2	1527	-14.80321	3.9895	4538.83	740.42	343.92	1470.58	740.42	344
7	27700	12	2	1462	-9.839782	1.2634	3800.89	2353.02	435.30	1523.18	1523.18	337
8	29500	12	2	1462	-4.585236	-0.0077	3166.71	555543	482.90	1304.31	1304.31	337

HORIZONTALISHEAD TRANSFER

Section	Distance	Effective	Contact	Factored	Allowable	Comment	Diameter	No. of	Minimum
	from	depth for	surface	Design	shear		of	Leg	spacing
	end	Shear	width	Shear	strength		stimup	142 6 3	required
		(d)	100		(¢Vnh)				na aren
	1000	100	114	200	1,412				
	(mm)	(mm)	(mm)	(kN)	(kN)		(mm)		(mm)
2	450	979	1066	1096.37	2263	0.K.	12	2	169.75
3	2250	1149	1066	959.29	2656	O.K.	12	2	169.75
4	7712.5	1527	1066	628.70	3532	O.K.	12	2	169.75
5	14975	1710	1066	185.86	3954	O.K.	12	· · · 2	169.75
6	22237,5	1527	1066	628.70	3532	0.K.	12	2	- 169.75
7	27700	1149	1066	959.29	2656	0.K.	12	2	169.75
8	29500	979	1066	1096.37	2263	0.K.	12	2	169.75

ELONGATION OF CABLE

Outside jack cable length (assumed), l_i =

700 mm

Dead anchor width, I, =

70 mm,

Cable stress at jack (after jack loss), fi =

1370 N/mm²,

Cable stress at dead anchor, f_{LP} =
Average cable length of anchor to anchor, L_p =

1258 N/mm², 30004 mm,

Average elongation of cables by one side jacking, $\Delta L_p =$

210 mm,

ANCHORAGE ZONE DESIGN

Minimum longitudinal extent of anchorage zone Maximum longitudinal extent of anchorage zone Selected longitudinal extent of anchorage zone = 1066 mm, = 1599 mm, = 1500 mm,

- DESIGN of GENERAL ZONE

angle of inclination of the resultant of the tendor

5.93 deg.

Size of	Thickness	Diameter	Effective	Factored	Total fact-	C/c	Longitudi-	Correction	Concrete	Allowable	Comment
bearing	of bearing	of cone	bearing	tandon	ored tend-	spacing of	nal extent	factor	compres-	Concrete	
plate	plate		area	load	on load	anchorage	of local	17.0	sive stress	compres-	
				60	197	activity.	zone		1. HART (1. 1	sive stress	
		1 4	(Ab)	1.5			(lc)	(k) .	(fca)	Sylation .	4 (4)
(mm)	(mm)	(mm)	(mm²)	(kN)	(kN)	(mm)	(mm)	4.5	(Mpa)	(Mpa)	200
230	35	152	49581.69	1947,273	7140	350	264.5	1.271	16.85	21	O.K.

DESIGN FOR VERTICAL DIRECTION

DESIGN FO	OK VER HU	AL DIKEU	ION	atti karala inter	10 10 10 10 10 10 10 10 10 10 10 10 10 1	1000	3 3		ter die er tree.	and the second	
Cable no.	1	2	3	4	0	0	0	0	0	0	, 0
Eccentricit v	528.51	178.51	171.49	521.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00
dburst	658.44	827.68	831.07	661.83	0.00	0.00	0.00	0.00	0.00	0.00	. 0,00
Reinforcem	ent distribu	tion distanc	e r. 61 - 55	3 to 1 to 1		1861.895	mm,	Maximum	<u> 1960. J. 1960</u>	2742	mm,
Bursting for	ce, Tourst	et nati.	1.19.00	F 9:4 Y		1929,157	kN,	12.34		dalik perjetan	20.0
Area of rein	forcement	required for	bursting fo	rce		5482.117	mm²			<u> </u>	
Diameter of				19 759 151		16	mm,	<u> </u>			1971
Total no of	2 legged st	irrup require	d		(= 1_0	13.63	nos.		2000	grade garage	

DESIGN FOR HORIZONTAL DIRECTION

Eccentri-	Lateral	Angle of	Bursting	Bursting	Area of	Dia Of	No. of	Rebar
city	dimension	tendon	distance	force	rebar	rebar	rebar rego	distribution
	H1 1 4 3 1	ta di Ka	100		required	ar ti	1 - 1 - 1 - 1	distance
(e)	(h)	(a)	(dburst)	(Tburst)		1992 <u>- 1</u>		
(mm)	(mm)	(deg)	(mm)	(kN)	(mm²)	(mm)		(mm)
0	711	0	355.5	1207.574	3431.582	16	8.53	888.75

DESIGN FOR EDGE TENSION

Spalling force (Tspal)	Area of rebar required		No. of rebar reqd
(kN)	(mm²)	(mm)	
142.8	406	12	0.03

DEFLECTION CALCULATION

			and process	erande in filozof	a and Artif	North Control		1.2.2	<u> </u>	1 1 1	
į		.): ' .	Deflection	on due to	and the second	. 14.5 1 2 1	Instanta-	Deflection	Long-time	Target A	
	Initial	Effective	Self wt.	Self wt.	Non-co-	Compos-	neous	at	Deflection	Live	Live
	Prestress	Prestress	7 7	200	mposite	ite dead	Deflection	erection		load	load
			(at initial	(after 28	dead load	load	e alter e		1. 1. 11. 11. 11.	p 22 1	rotation
			period)	days)	100						at support
		1.7							387	14 C Y	1 3 AV 1 A 1
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(rad.)
	45.2	36.7	-18 322	-16.96	-10.9	-5.3	26.9	47.5	-1.0	3.0	0.000482

DESIGN OF REINFORCED ELASTOMERIC BEARING (AASHTO: 1992 METHOD: A)

Developed & designed by : Prasanta Kumar Bhowmik.

50% SHEAR DEFORMATION DUE TO CREEP & SHRINKAGE ALLOWED

INPUT OF LOAD/STRESS/GEOM	ETR'	A SURVINE FOR
Dead Load Reaction	=	627 kN
Live Load Reaction without Impact	=	207 kN
Live Load Rotation	=	0.0013 Radian
Total length of Girder	=	30 m
Width of Girder	=	711 mm
Temperature Change	=	21 °C
Hardness of Bearing Material	= .	60 Dur.
Yield Strength of Steel Laminates, fy	=	248 MPa
Fatigue Strength of Steel Laminates, fsr	=	165 MPa

INPUTEOF BEARING DIMENSIONS	
Width of Elastomeric Bearing (Trans.) =	300 mm
Length of Elastomeric Bearing _(Long.) =	400 mm
Thickness of internal layers, h _{ri} =	10 mm
Thickness of Cover Layer =	8.5 mm
Total no of Internal Layer of Elastomer =	0 O.K.
Thickness of Steel Plates =	3 mm
Total thickness of Bearing =	20 mm

Design Shear Force To		
Substructure, H	=	75.7059 Kn

化氯化物 医乳腺性 医二甲基甲二甲基甲基甲基甲基甲基甲基甲基甲基甲基甲基甲基甲基甲基甲基甲基甲基甲基		After Array of the Control of the
Creep & Shrinkage	=	7.500 mm
Temperature shortening	=	0.000 mm
Shear deformation of the Bearing, Δ_s	=	7.500 mm
Shear Modulus, G _{min}	· · =	0.930 MPa
Shear Modulus, G _{max}	· =	1.430 MPa
Creep deflection at 25 years		35.000 %
Instantaneous Deflection k _b		0.600
Minm. Elastomer Thickness for shear, ha	=	15.00 mm
Minimum Area Required	=	119933 mm ²
Minimum Dimension for Stability (L or W)	≥	45 mm
Minimum length against Compression		400 mm
Minimum Shape Factor	=	7.47
Maxm. thick, of each elastomer layer, his	· =	11.5 mm
Total No. of Layer required, N	= _	-0.20
Thickness Reqd. for Steel Plates	=	0.71 mm
Actual Shape Factor, S	=	8.57 O.K.
Effective Compressive Modulus		
of the Elastomer, E _c	=	382.51 MPa
Instantaneous Compressive	Ž.	
Deflection of Bearing Δ_c	=	0.31 mm
	L =	0.4 mm
생물이 생생님은 이번째나는		О.К.
化热点 化氯化物 医乳腺性病 化二氯化物酶 医氯化物 化二氯化物		

	Sheet of	
	기능 등 기능 시간 기술 기술 기술 등 있다. 기술의 기술 기술 기술 기술 기술 기술 기술	
Project :	Designed:	***************************************
Structure:	Date :	***************
Item :		
DESIGN OF CONNECTION		
Dead Load Reaction from		
- extenion gimden = 27	75.5+148.20+117.89+1	18.95
	60.54 KN.	
- interiore gimdere = 27	75.5+169.37+117.89+3	17.90
	30.66 KN	
:. Total Bead Load Reac	HON	
= 2x560.54+5x600	9.66	
= 4124-38 KM.		
:. Connection Denign fonce		
= 0.2 x4124.38	ARCHTO I.A.5.	.2
= 824.88 KN.		
Let up provide 2x7=14	4 Mor 25 9 bar ac)
connection bare		
ty = 414 MPa		
fu = 0.4 fy (Table 1	10.32.1A Of AASHTO	
= 165 MPa Av = 490.87 mm ²		
i. strength of 14-2500 b	oak against shear	
= 14×490.87×165/1		
= 33 KN > 24.8	BKN O.K.	
一个人,只要你说在这个人,就是一个人,我是不是一个一个人,我们就是一个一个一个人。	重压器 医二氏试验检尿炎反射 医皮肤属性 经工程	

	D11661					
45						
					to the second second	
					1.1	
April 6 Comments						

Structure:

Item

Project

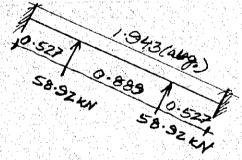
DESIGN OF END DIAPHRAGEM FOR CONCENTRATED LOAD FROM PIN

Longitudinal force in each connection bar = 824.88/14 = 58.92 KN

strennen due to above load will be concentrate at bottom region of diaphragem.

For longitudinal load

t = 300 mm (approximg 45° dintribution)



Fixed end Moment = 58.92x0.527x1.4162

= 22.63 KM.M.

Rea Hon = 58 92KN

Mid abon moment = 58.92 x 0.9715 - 22.63 - 58.92 x 0.4445 = 8.42 KN.m.

			ale separal d		
roject		,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	Designed	•	· · · · · · · · · · · · · · · · · · ·
Structure	•		Date		***************************************

ESONAING ORMENO design

fo'= 00 N/mm²

fo = 12 N/mm²

fy = 414 N/mm²

15 = 105.6 N/mm²

R = 1.932

j = 0.898

K = 0.367

$$d_{negot.} = \sqrt{\frac{p/R6}{1.932 \times 300}^2}$$

$$= 199.60 \text{ mm}$$

davailable = 300-40-12-25/2 = 235-5 > 197-60 O.K.

= 660.90 mm = 760.29 > 660.90 0.K.

d = 300-40-12-22/2 = 237 mm.

Shear Almenn & U = V = 58-92 × 1000 = 300 × 237

= 0:83×/ mm 2

Allowable char nemero = 0.079 Vas = 0.433 N/mm / hays/mm

Alimone megd.

100			
Project		Doolers	
	* *************************************	Designed :	***************************************
Structure	* *************************************	Date :	

Design for alimnuly

U-U_2 = 0.83-0.433 = 0.997 N/mm²

0.332\f'_c' = 1.82 N/mm² > 0.397 N/mm² O.K.

0.166\f'_c = 0.91 N/mm² > 0.937 N/mm²

Normal abacing can be maintained

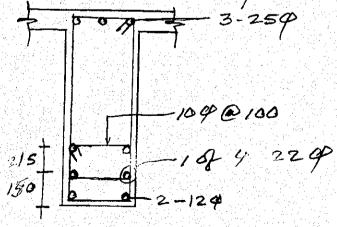
Let un une 109 ban an atimmuly

Sheet of..... of....

Let un une 109 bar an n'immuf Au = 2×78.54 = 157.08 mm²

$$5 \text{ max} = \frac{d}{2} = \frac{237}{2} = 118.5 \text{ mm}$$

:. Provide 100 ntimpub 100 mm c/c



Project	1	Designed:
Structure	:	Date :
Item		

Sheet of

Check For Dearing on Conemete

h = 300 mm

g = 25 mm

Bearing Promote on commete

= 58.92 × 1000

= 7.86 M/A

Allowable Bearing ofman

= 0.9 fc' = 0.3 × 30 = 9 M/A > 7.86 M/A

1-2. DESIGN OF APPROACH BRIDGE (2) SUBSTRUCTURE

 아마는 요즘 그 동생들은 아마를 가는 일하다. 사는 이 작은 사는 사람들이 가는 사람들이 되었다. 그는 사람들이 모든 사람들이 되었다.	1 TO 5 IP 이 보면 경우, IP 보고 있는 다른 가는 다른 하다. 그 보는 사람들은 보고 있는 것은 하는 것은 모든 것은

A. Soil Spring

(i) Pier No. 11

Following the article 16:15 of Foundation Analysis and design written by Bowles (5th. edition) Modulus of Subgrade reaction K_s can be given as follows:

$$K_s = A_s + B_s Z^n$$

where $A_s = F_{w1} C_m C (c N_c + 0.5 γ B_p N_γ)$ and $B_s = F_{w2} C_m C (γ Nq Z^n)$ again where $F_{w1} = 1.3$ to 1.7 say 1.5 $F_{w2} = 2.0$ to 4.4 say 3.2 $C_m = 1 + (457 / D)^{0.75} >= 1.5$ where D is pile diameter $C_m = 1 + (457 / 900)^{0.75} = 1.6$
Assume $C = 40$, $n = 0.6$ and $γ = 10$ kn/m³

Therefore
$$A_s = 1.5 \times 1.6 \times 40$$
 (c $N_c + 0.5 \times 10$ $B_p N_y$) = 90 (c Nc + 5 Bp Ny)
 $B_s = 3.2 \times 1.6 \times 40 \times 10 \text{ Ng } Z_n = 1920 \text{ Ng } Z^n$

Among all the boreholes in the viaduct area BH1VP2 is taken as the basis for estimating Spring constant because the top soil of this borehole is the softest. The Soil parameter is assumed as follows:

Top layer (Upto a depth of 11.5 i.e. Rt. = -8.28)

For avg. SPT = 3,
$$\phi$$
 = 0, c = 6.25 x SPT = 18.75, N_q = 1, N_γ = 0, N_c = 5.14 (See table 4.4.7.1A odf AASHTO Code.)

Therefore, Ks = $90 \times 18.75 \times 5.14 \times + 1920 \times 1 \times Z^{0.6} = 8,674 + 1920 Z^{0.6}$ Like main bridge here also, the spring stiffness is reduced by 35% to account for the group action on laterally loaded pile and hence multiplying the above equation by 0.65,

$$Ks = 5.650 + 1.250 Z^{0.6}$$

Bottom layer (For remaining depth of pile)

Avg. SPT = 13, $\phi = 0$, $c = 6.25 \times SPT = 81.25$, $N_q = 1$, $N_y = 0$, $N_c = 5.14$ Assuming reduction factor of 0.65 for Group action, Ks = $0.65 (90 \times 81.25 \times 5.14 \times + 1920 \times 1 \times Z^{0.6}) = 24,400 + 1,250 Z^{0.6}$

Depth		Spring Constant	
Z (m)	Ks	KFX	KFZ
0.60	6,570	14,783	4,139
1.35	7,147	9,648	9,648
2.85	7,993	10,791	10,791
4.35	8,670	11,704	11,704
5.85	9,257	12,498	12,498
7.35	9,787	13,212	13,212
8.85	10,275	13,871	13,871

Note_

 $Ks = 5,650 + 1250 \times Z^{0.6}$

 $KFX = Ks \times 15.0 \times 1.2 / 8$ for Z = 0.60m (Pilecap)

 $KFZ = Ks \times 4.2 \times 1.2 / 8 \text{ for } Z = 0.60 \text{m} \text{ (Pilecap)}$

KFX = KFZ = Ks \times 0.9 \times 1.5 for Z > 0.60m (Piles)

-						dadalah d
	Spring (Constant		Depth		KFX /
1	KFX	KFZ		Z (m)	Ks∴	KFZ
ס	14,783	4,139		10.35	29,480	39,798
7	9,648	9,648		11.85	29,910	40,378
3	10,791	10,791		13.35	30,318	40,930
þ	11,704	11,704		14.85	30,709	41,457
7	12,498	12,498		16.35	31,084	41,963
7	13,212	13,212		17.85	31,445	42,451
5	13,871	13,871		19.35	31,795	42,923
7.		1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1		20.85	32,133	43,380
Z	0.6			22.35	32,463	43,825
2 ,	/ 8 for Z = 0	.60m (Pilec	ap)	23.85	32,783	44,257
/	8 for $Z = 0.6$	30m (Pileca	p)	25.35	33,096	44,679
9	x 1.5 for Z	> 0.60m (Pil	les)	26.85	33,401	45,091
				28.35	33,699	45,494
	and the second section is					- 06

 $Ks = 24,400 + 1250 \times Z^{0.6}$

in the fire body to the field of the factor in the first of the fire of

(ii) Pier No. 1 to 10

4 times of that of pier no. 11 shall be used as Spring constant with values as follows:

Depth		Spring Constant		
Z (m)	Ks 🥠	KFX	KFZ	
0.60	6,570	59,130	16,556	
1.35	7,147	38,592	38,592	
2.85	7,993	43,164	43,164	
4.35	8,670	46,818	46,818	
5.85	9,257	49,990	49,990	
7.35	9,787	52,850	52,850	
8.85	10,275	55,483	55,483	

 $Ks = 5,650 + 1250 \times Z^{0.6}$

		14 11 11 11 11 11
Depth :	27-18 (1)	KFX /
Z (m)	Ks	KFZ
10.35	29,480	159,193
11.85	29,910	161,513
13.35	30,318	163,719
14.85	30,709	165,827
16.35	31,084	167,852
17.85	31,445	169,804
19.35	31,795	171,691
20.85	32,133	173,521
22.35	32,463	175,298
23.85	32,783	177,029
25.35	33,096	178,716
26.85	33,401	180,364
28.35	33,699	181,975

 $Ks = 24,400 + 1250 \times Z^{0.6}$

(iii) Abutment

6 times of that of pier no. 11 shall be used as Spring constant with values as follows:

Depth		Spring (Constant
Z (m)	Ks	KFX	KFZ
0.50	6,475	62,934	16,316
1.29	7,079	57,341	57,341
2.75	7,944	64,343	64,343
4.25	8,628	69,888	69,888
5.75	9,220	74,685	74,685
7.25	9,753	79,000	79,000
8.75	10,243	82,970	82,970

Note:

 $Ks = 5,650 + 1250 \times Z^{0.6}$

KFX = Ks x 16.2 x 1.2 / 2 for Z = 0.50m (Pilecap)

 $KFZ = Ks \times 4.2 \times 1.2 / 2$ for Z = 0.50m (Pilecap)

KFX = KFZ = 6 x Ks x 0.9 x 1.5 for Z > 0.50m (Piles)

Depth		KFX /
Z (m)	Ks	KFZ
10.25	29,451	238,550
11.75	29,882	242,044
13.25	30,292	245,363
14.75	30,683	248,534
16.25	31,059	251,580
17.75	31,421	254,514
19.25	31,772	257,351
20.75	32,111	260,101
22.25	32,441	262,772
23.75	32,762	265,372
25.25	33,075	267,907
26.75	33,381	270,383
28.25	33,679	272,803

 $Ks = 24,400 + 1250 \times Z^{0.6}$

(iv) End pier

For D > 1200mm,
$$C_m = 1.25$$

Assume C = 40, n = 0.6 and γ = 10 kn/m³

Therefore
$$A_s = 1.5 \times 1.25 \times 40 (c N_c + 0.5 \times 10 B_p N_{\gamma}) = 75 (c Nc + 5 Bp N_{\gamma})$$

 $B_s = 3.2 \times 1.25 \times 40 \times 10 Nq Z_n = 1285 Nq Z^n$

Considering Borehole BH1BA2, the soil parameter is assumed as follows:

Top layer (Upto a depth of 10.5 Le. RL = -6.25)

For avg. SPT = 3,
$$\phi$$
 = 0, c = 6.25 x SPT = 18.75, N_q = 1, N_y = 0, N_c = 5.14 (See table 4.4.7.1A odf AASHTO Code.) Therefore, Ks = 0.65 (75 x 18.75 x 5.14 x + 1285 x 1 x Z $^{0.6}$)= 4,700 + 835 Z $^{0.6}$

Bottom layer (For remaining depth of pile)

Avg. SPT = 20, ϕ = 0, c = 6.25 x SPT = 125, N_q = 1, N_γ = 0, N_c = 5.14 Assuming reduction factor of 0.65 for Group action, Ks = 0.65 (75 x 125 x 5.14 + 1285 x 1 x Z $^{0.6}$) = 31,300 + 835 Z $^{0.6}$

Depth		Spring (Constant
Z (m)	Ks	KFX	KFZ
0.75	5,403	22,286	22,286
2.25	6,058	24,990	24,990
3.75	6,545	49,091	49,091
5.25	6,958	52,187	52,187
6.75	7,326	54,944	54,944
8.25	7,662	57,464	57,464

Note:

 $Ks = 4.700 + 835 \times Z^{0.6}$

KFX = Ks x 11.0 x 1.5 / 4 for Z <= 2.25m (Pilecap)

KFZ = Ks x 11.0 x 1.5 / 4 for Z <= 2.25m (Pilecap)

KFX = KFZ = $2 \times Ks \times 2.5 \times 1.5$ for Z > 2.25m (Piles)

		<u> </u>
Depth		KFX/
Z (m)	Ks	KFZ
9.75	34,574	259,306
11.25	34,868	261,507
12.75	35,146	263,594
14.25	35,411	265,584
15.75	35,666	267,493
17.25	35,911	269,330
18.75	36,147	271,104
20.25	36,376	272,822
21.75	36,599	274,490
23.25	36,815	276,112
24.75	37,026	277,693
26.25	37,231	279,236
27.75	37,433	280,744
Vo = 24 20/		0.6

 $Ks = 31,300 + 835 \times Z^{0.6}$

B. DEAD LOAD FROM SUPERSTRUCTURE

(I) On pier

a. Steel railing	= 50 kg/m
b. Precast Parapet	= 0.469 sqm x 23.56 = 11.05 kn/m
c. Sidewalk Slab	= 0.056 sqm x 23.56 = 1.32 kn/m
d. Curb	= 0.119 sqm x 23.56 = 2.80 kn/m
e. Barrier	$= 0.336 \text{ sqm} \times 23.56$ $= 7.92 \text{ kn/m}$
	23.59 x 2 x 30 = 1415 kn
f. 60mm Premix	= 30 x 13.50 x 0.06 = 24.30 Cum x 23.56 = 573 kn
g. RC Deck	= 30 x 16.20 x 0.20 = 98.10 Cum x 23.56 = 2311 kn
h. Girders	= 7 x 551 kn (See Girder design) = 3857 kn
I. Diaphragm	= (See Girder design) = 455 kn

Dividing equally on each Girder = 8611 / 7 = 1230 km

8611 kn

(ii) On Abutment

From deck & Girder = 7 x 1230 / 2 = 4305 kn From approach Slab =15.1 x 2.5 x 0.36 x 23.56 = 320 kn Acting at Node 13 = 4625 kn

Moment Mz = $4305 \times 0.275 - 320 \times 0.45 = 1039$ knm acting at Node 13.

Back fill = 15.1 x 1.425 x 4.0 x 18.85 = 1623 kn

Return wall = 2 x 1.425 x 4.3 x 0.55 x 23.56 = 159 kn

= 2 x 3.075 x 1.0 x 0.55 x 23.56 = 80 kn

Railing etc. = 23.59 x 2 x 4.5 = 212 kn

2074 kn

UDL acting on member 65 = 2074 / 2.1 = 990 kn/m

C. LIVE LOAD FROM SUPERSTRUCTURE

(I) On Peir

Assuming Lane load and 4 lanes and allowing 8.75 % reduction

a. Live Load = (30 x 9.4 + 116) x 4 x 0.9125 = 1453 kn b. Impact = 1453 * 15.24 / (30+38) = 326 kn c. Sidewalk = 2 x 30 x 1.2 x 2.875 = 207 kn d. Utilities = 4 kn/m x 30.0 = 120 kn

Dividing equally on each Girder = 2106 / 7 = 300 km

(ii) On Abutment

Vertical load = 7 x 300 / 2 = 1050 kn acting at Node 13 Moment Mz = 1050 x 0.275 = 290 knm acting at Node 13

D. WIND LOAD ON SUPERSTRUCTURE

Design Wind Velocity, $V_d = 2.5 \times V_0 \times V_{10} / V_B \log_e (Z / Z_0)$

where V_b = Base wind velocity = 161 km/hr

 V_0 = Friction Velocity = 13.21 km/hr for open country

V₁₀ = Basic Wind Speed = 238 km/hr.

 Z_0 = Friction Length = 0.07 for open country

 $V_d = 2.5 \times 13.21 \times 238 / 161 (\log Z - \log 0.07) = 130 + 48.82 \log Z$

R.L. on top of deck above the tallest peir = 17.0 - 30 x 0.03 = 16.1 MPWD

R.L. on top of parapet above the tallest peir = 16.1 + 0.82 + 0.36 = 17.28,

R.L. of beam soffit = 16.1 - 2.0 = 14.1 MPWD

Level at C.G. of Wind pressure = (17.28 + 14.1) / 2 = 15.69 MPWD Normal Ground Level = 1.5 MPWD

Z for wind on superstructure = 15.69 - 1.5 = 14.19 = 14.5 (say)

Z for wind on Live load = 16.1 + 1.83 -1.5 = 16.5 (say)

Vd = 130 + 48.82 log 14.5 = 260 km/hr for Superstr. Vd = 130 + 48.82 log 16.5 = 267 km/hr for Live load

Wind on Superstr. = $2.394 \times (260 / 161)^2 = 6.24 \text{ kpa}$ (Transverse) $0.575 \times (260 / 161)^2 = 1.50 \text{ kpa}$ (Longitudinal)

Wind on Live load = $1.460 \times (267 / 161)^2 = 4.00 \text{ kn/m}$ (Transverse)

 $0.584 \times (267 / 161)^2 = 1.60 \text{ kn/m (Longitudinal)}$

Area of Superstructure exposed to wind = $30 \times 3.2 = 96$ sqm

Wind on Superstr. = 96 x 6.24 = 600 kn (Transverse)

96 x 1.50 = 144 kn (Longitudinal)

Wind on Live load = 30 x 4.00 = 120 kn (Transverse)

 $30 \times 1.60 = 48 \text{ kn (Longitudinal)}$

E. LONGITUDINAL FORCES

5% of L.L = $2 \times (30 \times 9.4 + 116) \times 5\% = 40 \text{ kn}$

E. SOIL SPRING TO SIMULATE BACKFILL PRESSURE BEHIND ABUTMENT

In case of increase in temperature, superstructure shall expand and push back the abutment wall against the soil creating a situation something in between active and passive state of earth pressure. To simulate the situation, springs are used behind the abutment wall at different levels and spring reactions are calculated by running STAAD. These reactions are then checked against the passive earth pressure at the corresponding level and if the reactions are found more than the passive pressure, the spring stiffness are reduced untill the reactions become equal to passive earth pressure. Initial Spring constants are found as follows:

Assume
$$F_{w1} = 1$$
 $C_m = 1$ $c = 0$ $N_q = 18.4$ $B = 1$ $F_{w2} = 1$ $C_m = 40$ $\phi = 30$ $N_y = 18.4$ $A_s = 1 \times 40 \times 0.5 \times 18.85 \times 1 \times 15.1 = 5,695$ $B_s = 1 \times 40 \times 18.85 \times 18.4 \times Z^n = 13,875 Z^n$ For $n = 0.4$, $K_s = 5,695 + 13,875 Z^{0.4}$

Depth (m)	Estimated Ks	Spring Constant	Passive Pressure	Maximum Spring Reaction	Spring Constant after adjustment
	2.64 (2.75				
0.50	16,210	131,303	28	229	112,000
1.00	19,570	158,517	57	458	158,517
1.50	22,013	178,306	85	687	178,306
2.00	24,003	218,729	113	1,030	218,729
2.63	26,107	264,333	148	1,502	264,333
3.25	27,927	282,765	184	1,860	282,765
3.88	29,548	299,173	219	2,218	299,173
4.50	31,018	314,059	254	2,575	314,059

G. THERMAL FORCES

Mean temperature is assumed 26°C while the maximum and minimum temperature are assumed 38°C and 7°C respectively.

H. SHRINKAGE DEFORMATION

Ref: CEB - FIP Manual (Relevant pages enclosed in Appendix A)

Assuming age of girder = 15 days at the time of prestressing and 120 days at the time of casting deck and relative humidity = 70%.

Sept. Dynam dan d	ah dagarta ya Kabasa san wasan katawa	lada da ora komba	. i. a. aab lage. i.
	For C	Sirder E	or Deck
X-sectional area 'F'	7000	sqcm 240 x 2	0 = 4800 sqcm
Perimeter 'u'	519	cm 2 x 240	- 107 = 373 cm
Eff. thickness 'dw' =2 I	k F / u 40	cm	40 cm

(kw = 1.5 from table 1.2)

Residual Shrinkage Strain $\epsilon_s' = \epsilon_{so}$ (K_{st} - K_{sto}) where $\epsilon_{so} = 25 \times 10^5$ at 70% humidity

(See Table 1.2)

	For Girder For Deck
	10000 days
	120 days 0 days
K _{st} (Table 1.3c)	0.79
K _{sto} (Table 1.3c)	0.20
ε _ε του της Μαραία (14.7) 14. Απορείου 11. Απορείου 14.79	5 x 10 ⁻⁵ 19.75 x 10 ⁻⁵

Average Shrinkage Strain = $(14.75 \times 7000 + 19.75 \times 4800) / 11800 \times 10^{-5}$ (weighted average) = 16.8×10^{-5}

Shrinkage deformation ' Δ ' = 16.8 x 10⁻⁵ x 360 x 10³ = 60.4 mm

I. CREEP DEFORMATION

Cross sectional area of concrete Girder 'Ag' = 4.90 sqm

deck 'A_d' = 3.27 sqm Total 'A' = 8.17 sqm

Total prestress P' = 4766 x 7 = 33,362 kn (after all loss)

Elastic Modulus 'E' = 27.98 x 10³ mpa

Creep Coefficient ' ϕ ' = $\varphi_{vo} k_{v(t+to)} + \varphi_{to} (k_{tt} - k_{to})$ where,

 $\phi_{vo} = 0.4$ $\phi_{fo} = 2.0$ (from table 1.2) $k_v = 1$ $k_{ft} = 1.40$ (for 10,000 days)

and $k_{flo} = 0.7$

Therefore $\varphi = 0.4 \times 1.0 + 2.0 (1.40 - 0.7) = 1.80$

At service condition $f_0 = P / A = 33,362 / 8.17 = 4.08$ mpa At construction time $f_t = P / Ag = 33,362 / 4.9 = 6.80$ mpa

For Girder =
$$f_t - (f_t - t_o) (1 - e^{-\phi})$$

= 6.8 - (6.8 - 4.08) (1 - $e^{-1.8}$) = 4.52 mpa
For deck = 4.08 (1 - $e^{-1.8}$) = 3.41 mpa

Average prestress 'f' =
$$(f_g A_g + f_d A_d) / (A_g + A_d)$$

= $(4.52 \times 4.9 + 3.41 \times 3.27) / 8.17 = 4.07$ mpa

Therefore 'P' = fA =
$$4.07 \times 8.17 / 7 = 4570 \text{ kn}$$

Strain '\$' = $4.07 / 27.98 \times 10^3 = 14.5 \times 10^{-5}$

Creep Deformation ' Δ ' = 14.5 x 10⁻⁵ x 360 x 10³ = 52mm

J. EARTH PRESSURE BEHIND ABUTMENT

Node	depth P	ressure	
36	0.000	0.0	그림으로 잃었다면 그렇게 되었다.
37	0.500	47.4	Note: Pressure = 18.85 x depth x 15.1 / 3
38	1.000	94.9	
39	1.500	142.3	보고를 하고 얼마가 보고 말한 글로 보고 말했다.
13	2.000	189.8	
40	2.625	249.1	
41	3.250	308.4	
42	3.875	367.7	
35	4.500	427.0	

The viaduct is modeled in STAAD III (file name is Viaduct1.Std) as space frame to carry the loads described in section B to J. Piles are kept supported horizontally by providing the spring as described in section A. Output of this model is given in Appendix 'B'.

Model Viaduct1.Std considers the effect of thermal contraction accompanied with creep, Shrinkage and the backfill pressure behind the abutment. Another model Viaduct2.Std is created to study effect of thermal expansion on the structure and the output is given in appendix 'C'.

K. DESIGN OF PIER HEAD

No. of legs =

6 Nos

From Page 11 of Appendix 'B' =

Max. (-)ive moment = -2094.65 knm for member 14 & load 7

Max. (+) ive moment = 1215.59 knm for member 16/17 & load 7

Max . Shear = 2843.43 kn for member 15 & load 7

Design of Be	am (USD method)		
		Design Moment =	"我们,我看到这么一样,一点一点,我们就把我看了一点,一点一点,我
fc=	30 mpa	Beta1 =	
fy =	410 mpa	ROWMX =	
b =	1650 mm	ROWMIN =	0.0034
Depth =	900 mm		
Bar dia =	28 mm	d =	805 mm
Str. Dia. =	16 mm	As =	7389 sqmm
Cl. Cover =	65 mm	p=	0.0056
Ast =	12 Nos	기가 하는 사람은 생 월 두 년	72.0 mm
Resisting M	loment = 2,096.7	knm O.K.	
그 그림부 11			
		Design Moment =	1216 knm
fc=	30 mpa	Beta1 =	0.8324
fy =	410 mpa	ROWMX =	0.0231
b =	1650 mm	ROWMIN =	0.0034
Depth =	900 mm		
Bar dia =	28 mm	d = /	805 mm
Str. Dia. =	16 mm		4926 sqmm
Cl. Cover =		p≡	0.0037
Ast =	8 Nos		48.0 mm
	REPUBLISH REPUBLISH		
Resisting M	foment = 1,419.6	8 knm > 0.K.	
i viasio ifa			
fc =	30 mpa	Design Shear =	2844 kn
fy≡	410 mpa		0.909 mpa
b =	1650 mm	Vc.=	1,207,7 kn
d =	805 mm	Vs =	2,275.2 kn
Str. dia =	16 mm		
Spacing =	175 mm	Vu =	2,960.5 kn O.K.

L. DESIGN OF COLUMN

PIER NO. 6 TO 11

For forces see Page 12 & 13 and for design see Page 24 of appendix '8'

PIER NO. 1 TO 5

From Page 25 of appendix B, with 1% reinforcement, column can take as much as 5627 kmm (0.9 x 6252.53) of moment, even if there is no axial forces. This is sufficient for column of pier no. 1 to 5. (see forces for column in page 13 of appendix 'B')

M. DESIGN OF PILECAP

Short direction:

From Page 14 of Appendix B and for members 51-54,

Design Shear = (5799.73+5938.55+5918.46+5730.01)/15 = 1559 knm/mDesign Moment = $1559 \times (1.35 - 1.5 / 2) = 935 \text{ knm/m}$ at the face of column

Design of Beam (USD method)

	Design Moment = 935 knm
30/m	pa Beta1 = 0.8324
ify:≓	oa ROWMX = 0.0231
b = 1000 mr	m ROWMIN = 0.0034
Depth = 1200 mr	n a kanakatan katan bada da katan
Bar dia = 25 mr	m d = 1038 mm
Str. Dia. = 0 mr	m As = 3927 sqmm
Cl. Cover = 150 mr	n p= 0.0038
Ast = 30 As 8 No	os a = 63.1 mm

Resisting Moment = 1,457.7 knm O.K.

Beam shear is not critical because, pile lies entirely within critical area. However corner pile is critical for punching shear having a force of 5730.01 kn (See member 480, load 10 at page 22 of appendix 'B').

Shear area = 2 x (1036/2 + 800 + 800) x 1036 =4,388,496 sqmm Shear stress = 5730.01 x 1000 / 4,388,496 = 1.305 mpa Allowable Shear = 0.332 x fc^{0.5} = 1.54 mpa > 1.305 O. K.

Long direction:

From Page 16 of Appendix B and for members 76 & load 8,
Design Moment = 1622.01 /4.2 =386 knm/m
Design Shear = negligible

Design of Beam (USD method)

		Design Moment = 386 knm
fc=	30 mpa	Beta1 = 0.8324
1961 - 197 5 - 1975	410 mpa	ROWMX = 0.0231
b=	1000 mm	ROWMIN = 0.0034
Depth =	1200 mm	
Bar dia =	25 mm	d = 1038 mm
Str. Dia. =	0 mm	As = 1963 sqmm
Cl. Cover =	150 mm	p= 0.0019
Ast =	4 Nos	a = 31.6 mm

Resisting Moment = 740.3 knm O.K.

N. DESIGN OF STEM OF ABUTMENT

From Page 17 of Appendix B and for members 73,

Design Moment = 24151.42 /16.2 = 1491 knm/m for load 12

Design Shear = 9679.87 / 16.2 = 598 kn /m for load 11

Design of Beam (USD method)

		Design Moment = 17 1491 knm	7.5
**	30 mpa	Beta1 = 6/0.8324	
ੇਰ ਹੋਰ ਨੂੰ fy = ਰ ੂੰ	410 mpa	ROWMX = 0.0231	
b=	1000 mm	ROWMIN = 0.0034	Š.
Depth =	1350 mm		
Bar dia =	25 mm	d = 1273 mm	
Str. Dia. =	0 mm	As = 3274 sqm	m
Cl. Cover =	65 mm	p=15 0.0026	140
Ast =	6.67 Nos	ing m a min a dit io a = 1,500 52.6 mm	

Resisting Moment = 1,505.6 knm O.K.

19.	10 161 1 3	494000 186	r kalentii	Traffic Place	455/47		
٠.		20		Dooles (Shaaa	598	les.
fc	and the second	30 mpa		Design	Shear = ∷		The State of the Control
ſy	/ = x - 1	410 mpa	建物学码		∵ .vc =	∍⊬0.909	mpa
t) = 10	000 mm			Vc =	983.4	kn O.K.
	1 - 4	272 mm					

From Page 10 of Appendix C and for members 73 & load 10,
Design Moment = 1967.01 / 16.2 = 122 knm/m in water face
Use nominal reinforcement.

O. DESIGN OF PILECAP OF ABUTMENT

From Page 19 of Appendix B and for members 49 & load 12,

Design Moment = 18210.41 x (1.35 - 0.75) /16.2 = 675 knm/m at wall face

Design Shear = 18210 / 16.2 = 1124 kn /m

Design of Beam (USD method)

		Design Moment =	675 knm
fc = 30) mpa	Beta1 =	0.8324
fy = 410) mpa	ROWMX =	0.0231
b = 1000) mm	ROWMIN =	0.0034
Depth = 1000	mm (
Bar dia = 25	mm	d =	838 mm
Str. Dia. =) mm	As =	3274 sqmm
Cl. Cover = 150) mm	p=	0.0039
Ast = 6.67	Nos	(1971)	52.6 mm

Resisting Moment = 980.0 knm O.K.

Beam shear is not critical because, pile lies entirely within critical area. However corner pile is critical for punching shear having a force of 3035 kn. l.e. 18210.39 / 6 (See member 380, load 12 at page 22 of appendix 'B').

Shear area = 2 x (836/2 + 800 + 700) x 836 =3,206,896 sqmm Shear stress = 3035 x 1000 / 3,206,896 = 0.946 mpa Allowable Shear = 0.332 x fc^{0.5} = 1.54 mpa > 0.946 O. K.

P. DESIGN OF RETURN WALL

Depth	Length	UDL	Moment	Mom/m
0.25	4.50	1.57	15.90	53.60
0.75	4.00	4.71	37.70	73.04
1.25	3.00	7.85	35.34	57.34
1.75	2.00	11.00	21.99	37.90
2.25	1.50	14.14	15.90	35.34
2.75	1.50	17.28	19.44	42,41
3.25	1.50	20.42	22.97	49.48
3.75	1.50	23.56	26.51	a Child

Design Moment = 73.04 x 1.3 = 95 knm/m

Design of Beam (USD method)

		Design	Moment =	95	knm
fc =	30 mpa		Beta1 =	0.8324	
fy =	410 mpa		ROWMX =	0.0231	
b =	1000 mm	n en en en en	ROWMIN =	0.0034	
Depth =	550 mm				
Bar dia =	16 mm		d =	477	mm
Str. Dia. =	0 mm		As =	1146	sqmm
Cl. Cover =	65 mm		p=	0.0024	
Ast =	5.70 Nos		a=	18.4	mm

197.8 knm O.K. Resisting Moment =

Q. DESIGN OF PILES

Structural:

See appendix 'D'

Geotechnical:

Please refer page no. 9 & 10 of appendix B for pile reaction and appendix E for pile capacity. Please note further that reaction of piles under abutment shall have to be divided by 6

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	West abut.	WP1-WP11	EP1-EP7	EP8-EP11	East abut.
Reference bore hole :	BH1 VA1	BH1 VP1	BH1 EA1	BH1 VP2	BH1 EA1
Pile reaction for load 13 (kn):	820	2206	2206	2206	820
Pile reaction for load 14 (kn):	2522	4801	4801	4801	2522
Uplift for load 14 (kn) :	1118	1306	1306	1306	1118
Ultimate capacity of pile (kn):	5805	8621	9542	8888	9542
Depth of pile tip from G.L. (m):	30.00	48.00	58.50	46.50	58.50
Top of bore hole R.L. (MPWD):	2.03	2.16	3.31	3.22	3.31
Pile tip R. L. (MPWD)	-27.97	-45.84	-55.19	-43.28	-55.19
Length of pile (m):	29,81	46.34	55.69	43.78	55.69
Weight of pile (kn):	447.09	695.10	835.35	656.70	835.35
Net uplift deducting pile weight:	670.91	610.90	470.65	649.30	282.65
Skin friction (kn):	3780.00	7184.00	7320.00	6559.00	7320.00
FS against load 13 :	7.08	3.91	4.33	4.03	11.64
FS against load 14:	2.30	1.80	1.99	1.85	3.78
FS against uplift :	5.63	11.76	15.55	10.10	25.90

Minimum bearing capacity: Abutment = 5805 / 2.5 = 2322 kn

Pier = 8621 / 2.5 = 3448 km