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AARA INFRA

Date: 26-10-2024

Structural Stability Certificate

It is certified that the Proposed Residential Building comprising **Stilt Floor + 5 floors with 30 Dwelling Units** to be set up at **T. S. Nos. 462/1C and 462/12, Block No. - 16, ward No. - 1 and T.S. No. 465/3B21, Block No. 17, Ward No. 1 of Kumbakonam Corporation, Thanjavur district, Tamil Nadu State,** have been designed as a RCC structure to withstand the Dead Load, Imposed Load and Wind Load / Earthquake loads ensuring Structural Safety and service ability as per the norms and guidelines prescribed in the following codes of Bureau of Indian Standards.

1. The minimum grade of concrete is M20
2. IS: 456-2000 Code of Practice for Plain and Reinforced concrete.
3. IS: 875-1987 Code of Practice for Design loads for Buildings and Structures.
4. IS: 1893-2002 Criteria for Earthquake resistant design of StructuresPart-1 general Provisions and Buildings.
5. IS: 4236-1993 Code of Practice for Earthquake resistant design & Construction of Buildings.
6. IS: 13920-1993 Code of Practice for Ductile detailing of concrete structures subjected to seismic forces and
7. National Building code of India – 2005 are carried out.

It is further certified that the above structures will be constructed from foundation to roof as per structural designs and detailing of reinforcements furnished in our drawings.

Hence, I hereby certify that the building will be structurally safe and sound when used for the purpose for which it is designated.

Signature with Seal

Er. D.S... M.E. (Structural)
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Reg. No. Coimbatore LPA/SE/GR-II/19/05/026
2/536 AB, VKV Peoples Park, Thiruvalluvar Nagar,
Thadagam Road, Coimbatore-641 108.
Ph. No: 9488064606, 8072342513

Enclosure: Structural Design Calculations

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Design Data Report :

Footing Data

Soil Parameters

SBC of soil	= 280.00 KN/m²
SBC Increment Factor for SeismicLoad	1
SBC Increment Factor for Wind Load	1
Angle offriction,(phi)	= 30.00 degrees
Cohesion	60
Permissiblefactorofsafety	2
Depth From Ground	= 2000 mm

PlaterialProperties:

Fck	= 20.00 N/mm²
Fy	= 500.00 N/mm²

Footing Parameters :

Minimum steel Percentage	0.12
Minimumspacingbetweenbars	=100mm
Maximum spacing between bars	= 200mm
Diameters of selected bars	= 10 mm ,12 mm,
BottomCover	=50mm
EndCover	=50mm
Minimum FootingDepth	=300mm
TopOffset	=50mm
Bottom PCCOffset	=150mm
Bottom PCCDepth	=150mm
Pedestal Offset	=150mm
Pedestal Depth	=450mm
Size RoundingFactor	=25mm
Depth RoundingFactor	=25mm

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Column Data:

Material Properties :

Fck	= 20.00 N/mm²
Fy	=500.00N/mm²
fy- Stirrups	=500.00N/mm²

Column Parameters :

Effective Length Factor (X - Axis)	1
Effective Length Factor (Y - Axis)	1
Minimum Eccentricity @ X-Axis	= 20mm
Minimum Eccentricity @ Y-Axis	= 20mm
Clear Cover	=40mm

Design Considerations :

Design is done as per IS 456.

IS13920provisionsarenotconsideredfor detailing.

LiveLoadreductionisnotconsidered.

Minimumsteelpercentageisbasedon providedsection.

Reinforcement Details :

Steel Ratio(AstX/AstY)	1
ReinforcementPlacing(ontwosidesoron four sides)	2
Minimum steel Percentage	0.8
Maximum steel Percentage	4
Minimum spacing betweenmainbars	= 50 mm
Maximumspacingbetweenmainbars	= 300 mm
Minimum spacingbetweenStirrups	= 150 mm

Diameters of selectedbars

Slab Data:

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Thadagam Road, Coimbatore-641 109.
Ph. No.: [illegible]

UseBent-upbarsforsteeldetailing.

Material Properties :

RCC Grade	= 20.00 N/mm²
Main Steel Grade	=500.00N/mm²
Distribution Steel Grade	=500.00N/mm²

Slab Parameters :

Effective Cover	= 25 mm
MinimumspacingbetweenMainbars	= 100 mm
MaximumspacingbetweenMainbars	= 200 mm
Minimum spacing between Dist.bars	= 100 mm
MaximumspacingbetweenDist.bars	= 300 mm
Maximum Main/Dist. bar Diameter	= 10 mm
Minimum Main bar Diameter	= 8 mm
Minimum Dist.barDiameter	= 8 mm
Maximum Dimension to be used for Trapizoidal Slab	#NAME?

Beam Data•

Shear Design Data :

Minimum Stirrup Dia	= 8 mm
Stirrup Legs	2
Minimum Stirrups Spacing	= 100 mm
Stirrups Steel Grade Name	0

EffectofConcreteinshearconsidered

Beam Covers :

Tension Cover	= 25 mm
Compression Cover	= 25 mm
Side Cover	= 25 mm

Beam sections are designed as Doubly

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1.DiaofReinforcementbarsisinmm

2.SpacingofReinforcementbarsisinmm

3.ThicknessofSlabisinmm

Design Of Slabs : Default Level

DesignMethod:LimitStateMethod

Design Code : IS456:2000

Design Of Slab

SlabName :S1

SlabType : TwoWay Slab

Grade of Concrete:M20

GradeofSteel :Fe500

Dimensions :Lx=3.300m,Ly=3.900m,Thickness—125mm

SpanRatio : Longer/Shorter = 1.182

Boundary Condition : One short and one long edge continuous

Loading on Slab :(Table 1)

Total DL = (I) + (2) + (3)

Self weight (1) Kn/m²	Floor finish (2) Kn/m²	Sunk load (3) Kn/m²	Total DL (4) Kn/m²	Live Load (5) Kn/m²	Total LL (6) Kn/m²	Total Design Load 1.5 x [(4)+(6)] Kn/m²
3.125	1.000	0.000	4.125	2.000	2.000	9.188

Deflection Check : (Table 2)

Asperclause23.2.1of Is456:2000:

(Spam/Depth)Ratio	Motification Factor (a)	Basic Factor (b)	Permissable Ratio (a) x (b)	Status
33.000	1.556	32.000	49.778	OK

MomentAndSteelCalculations:

Coefficients ($\alpha_x$andα_y) are **computed as per** clause, D-1.1 of IS **456:2000:**

$M_x = \alpha_x \times W_x \times L_x \times L_x$:

$M_y = = \alpha_x \times W_x \times L_x \times L_x$:

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Reinforced Section

Main Bars Detailing Straight + Curtailed

DetailingprovisionAsperIS13920not

followed

EndMomentsarecalculatedatfaceofthe

column fordesign

TorsionMomentsarenotconsideredin

beamdesign

Anchor bar Dia = 12 mm

Main Steel Grade =500.00N/mm²

No. of division of the beam 10

Max. Aggregate size 25

Slab Schedule Report:

Slab Group	Slab ID	Concrete Grade	Thk	Bottom Reinforcement c/c)		Top Reinforcement extra steel (c/c)				Remark
SG1	S1	M20	150	#10 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG2	S2	M20	150	#10 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG3	S3	M20	150	#10 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG4	S4	M20	150	#10@ 200	#8 @ 200	--	--	--	--	TwoWay
SG5	S5	M20	150	#10 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG6	S6	M20	150	#10 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG7	S7	M20	150	#10 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG8	S8	M20	150	#8 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG9	S9	M20	150	#8 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG10	S10	M20	150	#8 @ 200	#8 @ 200	--	--	--	--	TwoWay
SG11	S11	M20	150	#8 @ 200	#8 @ 200	--	--	---	--	TwoWay

Notes :

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where

M_x,M_y=momentsonstripsofunitwidthspanning $L_x$and L_y.yrespectively,

W = Total **Design Load** = 9.188 kN/m²

L_x = Shorter span = 3.300 m

Position	Moment coeff$\alpha_{x\ or}$ α_y	Moment KN-m	$A_{s\text{-}rqd}$ Per meter	Steel detail Dia @spc m/m	$A_{s\text{-}prv}$ Per meter	Remark
MidShort	0.036	3.611	84.809	#8 @ 200	251.327	M4ain
MidLong	0.028	2.801	65.473	#8 @ 200	251.327	Other
SuppDown	0.037	3.702	86.994		0.000	ExtraatTop
SuppTop	0.037	5.702	86.994		0.000	ExtraatTop
SuppLeft	0.000	0.000	0.000		0.000	ExtraatTop
SuppRight	0.048	4.815	113.926		Q.000	ExtraatTop

Minimum Steel Check

As per clause 26.5.2.1 of IS 456:2000:
Minimum area of steel For Main Steel = 0.12 x C/S Area
— 150.000 mm²

Notes:

ExtrasteelatTopsupportiscomputedconsidering

thebent-ups,ifany,comingfromtheadjoiningspan.

Itisthemaximumoftheextrasteelrequired

for each slab at that commonsupport.

SlabName : S2

SlabType : TwoWay Slab

Grade of Concrete:M20

GradeofSteel :Fe500

Dimensions :Lx=3.300m,Ly=5.800m,Thickness=125mm

SpanRatio : Longer/Shorter =1.152

Boundary Condition : One long edge discontinuous

Loading on Slab :(Table I)

Total DL = (1) + (2) + (3)

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Self weight (1) Kn/m²	Floor finish (2) Kn/m²	Sunk load (3) Kn/m²	Total DL (4) Kn/m²	Live Load (5) Kn/m²	Total LL (6) Kn/m²	Total Design Load 1.5 x [(4)+(6)] Kn/m²
3.125	1.000	0.000	4.125	2.000	2.000	9.188

Deflection Check: (Table 2)

Asperclause23.2.1of Is456:2000:

(Spam/Depth)Ratio	Motification Factor (a)	Basic Factor (b)	Permissable Ratio (a) x (b)	Status
33.000	1.556	32.000	49.778	OK

MomentAndSteelCalculations:

Coefficients ($\alpha_x$andα_y) are computed as perclause, D-1.1 of IS 456:2000:

$M_x = \alpha_x W_x L_x L_x$:

$M_y = = \alpha_x W_x L_x L_x$:

where

M_x, M_y=momentsonstripsofunitwidthspanning $L_x$and L_y.yrespectively,

W = Total Design Load= 9.188 kN/m²

L_x = Shorter span = 3.300 m

Position	Moment coeff$\alpha_{x\ or}\ \alpha_y$	Moment KN-m	$A_{s\text{-}rqd}$ Per meter	Steel detail Dia @spc m/m	$A_{s\text{-}prv}$ Per meter	Remark
MidShort	0.036	3.611	84.809	#8 @ 200	251.327	M4ain
MidLong	0.028	2.801	65.473	#8 @ 200	251.327	Other
SuppDown	0.037	3.702	86.994		0.000	ExtraatTop
SuppTop	0.037	5.702	86.994		0.000	ExtraatTop
SuppLeft	0.000	0.000	0.000		0.000	ExtraatTop
SuppRight	0.048	4.815	113.926		Q.000	ExtraatTop

Minimum Steel Check •

As per clause 26.5.2.1 of IS 456:2000:
Minimum area of steel For Main Steel = 0.12 x C/S Area

— 150.000 mm²

Notes :

ExtrasteelatTopsupportiscomputedconsidering

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the bent-ups,if any,coming from the adjoiningspan.

Itisthemaximumoftheextrasteelrequired

foreachslabatthatcommonsupport.

Building Version:a.55O ETRUDS Vezalon :2.0.0

SlabName :S3

'SlabType : TwoWaySlab

Grade of Concrete:M20

GradeofSteel :Fe500

Dimensions :Lx=4.200m,Ly=2.200m,Thickness=125mm

SpanRatio : Longer/Shorter =1.909

Boundary Condition : One short edge discontinuous

Loading on Slab :(Table I)

Total DL = (1) + (2) + (3)

Self weight (1) Kn/m²	Floor finish (2) Kn/m²	Sunk load (3) Kn/m²	Total DL (4) Kn/m²	Live Load (5) Kn/m²	Total LL (6) Kn/m²	Total Design Load 1.5 x [(4)+(6)] Kn/m²
3.125	1.000	0.000	4.125	2.000	2.000	9.188

Deflection Check : (Table 2}

Asperclause23.2.1of Is456:2000:

(Spam/Depth)Ratio	Motification Factor (a)	Basic Factor (b)	Permissable Ratio (a) x (b)	Status
33.000	1.556	32.000	49.778	OK

Beam Design Schedule Default Level:6.000 m

Beam Name	Size Mm	Bottom Steel					Top Steel				Stirrups			Remarks
		Straight	Bent /Curt	Left At	Right At	Straigh t	left	L1,L2 mm	Right	L1, L2 mm	Left Dia-spc Nos	Middle Dia-spc Nos	Right Dia-spc Nos	
B1	230x375	4-#16 --	---	---	---	2-#12 +2-#10	4-#12 --	0.000, 0.825	2-#12 --	0.000, 0.096	--	#8@250 14nos (Total)	--	---
B2	230x375	2-#12 +2-#10	---	---	---	2-#12 ---	-- --	--- ---	-- --	0.000, 0.096	#8@225 3nos	#8@250 22nos	--	---
B3	230x375	2-#12	---	---	---	2-#16	4-#12	0.825,	3-#12	---	--	#8@250	#8@175	---

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		--				+2-#10	--	1.450	--	---		19nos	3nos	
B4	230x375	2-#12 +2-#10	---	---	---	2-#12 +2-#16	2-#12 --	--- ---	-- --	--- ---	--	#8@250 14nos (Total)	--	---
B5	230x375	2-#12 +2-#10	---	---	---	2-#12 +2-#10	-- --	--- ---	4-#12 --	0.000, 0.096	--	#8@250 6nos (Total)	--	---
B6	230x375	2-#12 --	---	---	---	2-#12 +2-#10	3-#12 --	0.825, 1.450	-- --	0.825, 1.450	--	#8@250 14nos	--	---
B7	230x375	3-#16 --	---	---	---	3-#12 +2-#10	-- --	0.825, 1.450	4-#12 --	0.000, 0.096	#8@250 3nos	#8@250 14nos (Total)	--	---
B8	230x375	4-#16 --	---	---	---	2-#12 +2-#10	4-#12 --	0.000, 0.096	2-#12 --	0.000, 0.096	--	#8@250 7nos (Total)	--	---
B9	230x375	2-#12 +2-#10	---	---	---	2-#12 ---	-- --	--- ---	-- --	0.000, 0.096	--	#8@250 16nos	--	---
B10	230x375	2-#12 --	---	---	---	2-#16 +2-#10	4-#12 --	--- ---	3-#12 --	--- ---	#8@225 3nos	#8@250 14nos (Total)	--	---
B11	230x375	2-#12 +2-#10	---	---	---	2-#12 +2-#16	2-#12 --	--- ---	-- --	--- ---	--	#8@250 10nos	#8@200 3nos	---
B12	230x375	2-#12 +2-#10	---	---	---	2-#12 +2-#10	-- --	0.825, 1.450	4-#12 --	0.000, 0.096	--	#8@250 13nos (Total)	--	---
B13	230x375	2-#12 --	---	---	---	2-#12 +2-#10	3-#12 --	0.000, 0.096	-- --	0.825, 1.450	--	#8@250 14nos (Total)	--	---
B14	230x375	3-#16 --	---	---	---	3-#12 +2-#10	-- --	--- ---	4-#12 --	0.000, 0.096	--	#8@250 22nos	--	---
B15	230x375	4-#16 --	---	---	---	2-#12 +2-#10	4-#12 --	0.000, 0.096	2-#12 --	0.000, 0.096	#8@225 3nos	#8@250 17nos (Total)	--	---
B16	230x375	2-#12 +2-#10	---	---	---	2-#12 ---	-- --	0.000, 0.096	-- --	0.000, 0.096	--	#8@250 16nos (Total)	--	---
B17	230x375	2-#12 --	---	---	---	2-#16 +2-#10	4-#12 --	--- ---	3-#12 --	--- ---	--	#8@250 16nos (Total)	--	---
B18	230x375	2-#12 +2-#10	---	---	---	2-#12 +2-#16	2-#12 --	--- ---	-- --	--- ---	--	#8@250 4nos (Total)	--	---
B19	230x375	2-#12 +2-#10	---	---	---	2-#12 +2-#10	-- --	0.000, 0.096	4-#12 --	0.000, 0.096	--	#8@250 15nos (Total)	--	---
B20	230x450	2-#12 --	---	---	---	2-#12 +2-#10	3-#12 --	0.825, 1.450	-- --	0.825, 1.450	--	#8@250 5nos	--	---
B21	230x450	3-#16 --	---	---	---	3-#12 +2-#10	-- --	0.000, 0.096	4-#12 --	0.000, 0.096	#8@200 3nos	#8@250 4nos (Total)	#8@25 15nos	---
B22	230x450	4-#16 --	---	---	---	2-#12 +2-#10	4-#12 --	--- ---	2-#12 --	0.000, 0.096	--	#8@250 3nos	--	---
B23	230x450	2-#12 +2-#10	---	---	---	2-#12 ---	-- --	--- ---	-- --	0.000, 0.096	--	#8@250 5nos (Total)	--	---
B24	230x450	2-#12 --	---	---	---	2-#16 +2-#10	4-#12 --	--- ---	3-#12 --	--- ---	--	#8@250 10nos (Total)	--	---
B25	230x450	2-#12 +2-#10	---	---	---	2-#12 +2-#16	2-#12 --	--- ---	-- --	--- ---	--	#8@250 10nos	--	---
B26	230x450	2-#12 +2-#10	---	---	---	2-#12 +2-#10	-- --	0.825, 1.450	4-#12 --	0.000, 0.096	#8@225 3nos	#8@250 9nos (Total)	--	---
B27	230x375	2-#12 --	---	---	---	2-#12 +2-#10	3-#12 --	0.000, 0.096	-- --	0.825, 1.450	--	#8@250 13nos	--	---

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												(Total)		
B28	230x375	3-#16 --	---	---	---	3-#12 +2-#10	-- --	0.825, 1.450	4-#12 --	0.000, 0.096	--	#8@250 14nos (Total)	--	---
B29	230x375	4-#16 --	---	---	---	2-#12 +2-#10	4-#12 --	0.000, 0.096	2-#12 --	0.000, 0.096	--	#8@250 14nos (Total)	--	---
B30	230x375	2-#12 +2-#10	---	---	---	2-#12 ---	-- --	--- ---	-- --	0.000, 0.096	--	#8@250 9nos (Total)	--	---
B31	230x375	2-#12 --	---	---	---	2-#16 +2-#10	4-#12 --	0.825, 1.450	3-#12 --	--- ---	--	#8@250 14nos (Total)	--	---
B32	230x375	2-#12 +2-#10	---	---	---	2-#12 +2-#16	2-#12 --	0.000, 0.096	-- --	--- ---	--	#8@250 15nos (Total)	--	---

Beam Detail Report

FF- Default Level

Design load combination : 1.50 DL + g.50 LL

Design of Beam Group : BG1

Beam: BI

Material Properties

fck = 20.000 N/mmB

fy = 500.000 N/mmB

1.Design Data '

I.I)Dimensions:

BeamType = 'RectangularBeam'

Width(b) = 230mm

TotaldepthtD) = 375mm

EffectiveTensionCover = 33mm

Effective Compression Cover = 33mm

EffectivesideCover =25mm

Effectivedepth(d) =342mm

Span(L) =3.300ai

I.2)Loading. on the Beam :

I.2.I)Selfwt.ofbeam:-2.156kN/m

1.2.2)SlabLoad

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Dead Load :

Triangular Load in kN/m

St.Pt 0.000 m L = 3.300 m Max. Value = -6.806 kN/m

LiveLoad

Triangular Load in kN/m

St.Pt 0.000 m L = 3.300 m Man. Value = -3.300 kN/m

1.2.3)Wall Load

UDL in kN/m

St.Pt 0.000 m L = 3.300 m Max. Value = -13.110 kN/m

1Computation of Area of steel : 2.

1)FlexuralSteelatTopandBottom

2.1.1)Computation of theoreticalsteel

At length	0.000L	0.I00L	0.200L	0.300L	0.400L	0.500L	0.600 L	0.700L	0.800L	0.900L	I.000L
B.M. Hog in kN-m	19.175	11.095	1.058	0.000	0.000	0.000	0.000	3.905	15.653	30.556	41.805
B.M. Sag in kN-m	0.000	0.000	0.000	5.826	9.225	8.809	4.360	0.000	0.000	0.000	0.000
Torsion in kN-m	6.910	6.910	6.910	6.910	6.910	6.910	6.910	6.910	6.910	6.910	6.910
Eq.B.M. Hog in kN-m	19.175	11.095	1.058	0.000	0.000	0.000	0.000	3.905	15.653	30.556	41.805
Eq.B.M. Sag in kN-m	0.000	0.000	0.000	5.826	9.225	8.809	4.360	0.000	0.000	0.000	0.000
Top A_{st_req} In mmB	134.652	76.438	7.127	0.000	0.000	0.000	0.000	26.468	108.991	220.896	3I 1.925
Bot A_{st_req} In mmB	0.000	0.000	0.000	39.658	63.279	60.374	29.58 3	0.000	0.000	0.000	0.000

Max.Area of steel req. at Top = 311.925 mmB at 1.000L

Max.Areaofsteelreq.atBottom=63.279mmBat0.400L

2.I.2)Minimum and Maximum Steel Checks:

a)Minimum Steel Check as per IS 456:2000 Clause26.5.1.1(a)

$A_{st} = (0.85 \times b \times d) / f_y$

= 133.722 mmB

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MaximumSteelCheckasperIS456:2000Clause2li.5.1.1(b)

A_{st} = 0.04 x b xD

= 3450.000 mmB

2.1.3)Provided Steel

2.1.4)Provided Bottom Steel :

Straight bars 2 - #12

Bent up bars

Total Area of Steel at bottom at 0.000L = 226.195 mmB > A_{st_req} (37K)

2.1.2)Provided Top Steel:

At length	0.000L	0.100L	0.200L	0.300L	0.400L	0.500L	0.600L	0.700L	0.800L	0.900L	1.000L
S.F. in kN	40.225	34.862	25.805	15.747	4.688	-7.371	-19.430	-30.489	-40.547	-49.605	-54.967
T, in kN-m	6.910	6.910	6.910	6.910	6.910	6.910	6.910	6.910	6.910	6.910	6.910
Vuc in kN	29.986	29.986	29.986	29.986	29.986	29.986	29.986	29.986	46.407	46.407	46.407
Design Shear in kN	10.239	4.877	0.000	0.000	0.000	0.000	0.000	0.503	0.000	3.198	8.561
Spacing in mm for 8 Dia	257	257	257	257	257	257	257	257	257	257	257

Straightbars = 2 - #12

Extra bars atLeft

Extra barsatRight - 4 - #12 0.825,1.450

Total Area of Steel atTop

AtLeftSupport - 226.195mmB

AtRightSupport = 678.584 mmB

2.2)ComputationofShearReinforcement(Stirrups):

τ_{auc} ' 0.590 N/mmB

S_{vmin} = 257 mm

Note :

Design Shear = (Equivalent S.F.)-(V_{uc})

2.2.1) Computation of Steel

2.2.2)Provided Shear Steel (Stimips) :

Nominal Zone :#8 @ 250 mm c/c from 0.000 to 3.250 .

Note : Nominal zone is where No Shear Reinforcement is needed.

HenceprovideminimumShearReinforcementof0.75d

(As per Clause 26.5.1.5 of IS456:2000)

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Design of Beam Group : BG2

Beam: B2

htaterial Properties : '

fck = 20.000N/mmB

fy = 500.000 N/mmB

1.DesignData:

1.1)Dimensions:

BeamType = 'RectangularBeam'

Width(b) = 230mm

Totaldepth(D) = 375mm

EffectiveTensionCover = 33mm

Effective Compression Cover = 33mm

EffectivesideCover =25mm

Effectivedepth(d) =342mm

Column Detail Report:

SPACEFRAMESTRUCTURE

As Per IS 456 : 2000

CG1	C1	sf	300x230	532.15	7.1	25.43	679	0.983
		tf	300x230	356.25	9.65	27.19	679	0.983
		ter	300x230	179.97	8.86	28.14	679	0.983
		hr	300x230	38.12	1.37	32.89	679	0.983
CG2	C2	pl	230x380	743.98	1.24	3.95	1030	1.493
		S	230x380	680.07	2.8	11.01	1608	2.331
		sf	230x380	489.01	5.5	17.48	679	0.983
		tf	230x380	297.04	6.75	16.99	679	0.983
		ter	230x380	104.5	5.48	15.92	679	0.983
CG3	C3	pl	450x230	885.56	5.04	2.18	1608	0.983

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								2.331
		ff	450x230	788.42	12.22	2.04	1885	2.732
		sf	450x230	593.37	14.6	1.28	679	0.983
		tf	450x230	381.39	13.63	0.23	679	0.983
		ter	450x230	170.84	10.24	0.85	679	0.983
		hr	450x230	15.39	16.39	4.08	679	0.983
CG4	C4	pl	450x230	907.53	2.68	12.62	1608	2.331
		ff	450x230	754.87	3.1	14.69	1608	2.331
		sf	450x230	562.89	6.39	19.01	679	0.983
		tí	300x230	390.77	5.85	16.41	679	0.983
		ter	300x230	216.11	8.16	15	679	0.983
		hr	300x230	47.13	3.25	16.95	679	0.983
CG5	C5	pl	450x230	758.44	0.97	16.98	1206	1.748
		ff	450x230	642.05	0.81	16.49	1608	2.331
		sf	450x230	480.94	0.32	24.22	679	0.983
		tf	450x230	320.56	1.53	21.86	679	0.983
		ter	300x230	159.33	2.64	23.25	679	0.983
		hr	300x230	33.59	1.53	24.4	679	0.983
CG6	C6	pl	450x230	816.24	1.47	8.6	1257	1.821
		ff	450x230	745.31	1.94	16.16	1885	2.732
		sf	230 x 300	545.07	6.54	21.63	804	1.166
		tf	230 x 300	336.82	8.24	17.21	679	0.983
		ter	230 x 300	129.93	8.26	17.99	679	0.983
CG7	C7	pl	450x230	803.13	5.88	17.21	1608	2.331
		ff	300 x 230	757.27	1.36	14.7	1885	2.732
		sf	300 x 230	555.98	23.8	19.67	1030	1.493
		tf	300 x 230	348.03	31.01	9.72	905	1.311
		ter	300 x 230	138.52	28.19	8.99	804	1.166
CG8	C8	pl	230 x 300	595.35	0.81	2.05	679	0.983
		ff	230 x 300	543.96	1.56	1.64	804	1.166
		sf	230 x 300	403.43	1.21	4.61	679	0.983
		tf	230 x 300	248.44	0.98	6.53	679	0.983
		ter	230 x 300	92.72	1.28	5.57	679	0.983
CG9	C9	pl	300x 450	506.77	4t47		679	0.983
		ff	300x 450	463.81	14.64	7.19	905	1.311
		sf	300x 450	353.37	30.26	14.06	804	1.166
		tf	300x 450	215.56	33.57	13.71	804	1.166
		ter	300x 450	76.6	33.16	13.84	1206	1.748
CG10	C10	pl	300 x 230	918.94	9.09	3.98	1608	2.331
		lî	375x300	831.73	25.16	14.93	2011	2.331
		sf	375x300	567.39	22.7	31.32	1608	2.331
		tf	375x300	350.92	5.85	30.56	679	0.983
		ter	375x300	136.25	5.15	29.53	679	0.983
CG11	C11	pl	450x300	617.51	1.79	8.61	679	0.983
		ff	450x300	567.38	5.28	26.9	1885	2.732
		sf	450x300	420.06	9.99	45.72	1608	2.331
		tf	450x230	256.08	9.13	46.29	1257	1.821
		ter	450x230	92.17	8.1	45.37	12S7	1.821

Terms Used in Calculation

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In biaxial column design,

$P_u=P_{uc}+P_{us(Total)}$

where,

P_u = external axial compressiveload,

P_{uc} = axial compressive resistance offered byconcrete,

$P_{us(Total)}$ = total axial compressive resistance offered by steel at different levels In the section.

$M_u=M_{uc}+M_{us(Total}$

P_u = external moment about centroidal axis,

P_{uc} = momentof resistance offered byconcrete in compression,

$P_{us(Total)}$ = total momentof resistance offered by steel at different levels In the section.,

I = serial no. of row of reinforcement,

A_{si} = cross-sectional area of reinforcement in the i th row,

f_{si} = stress in the reinforcement in the i th row (compression + ve, tension - ve),

f_{ci} =compressive stress in concrete at the level of i th row of reinforcement,

e_i = strain at the I th row oF reinforcement from the stress-strain curve of steel and concrete,

x_i = distance of the bars in the i th row from the centroid of the section

Values of stress in steel :

Design strength in bending compression (f_{yd}) = 0.87 f_y

a)Fe250,

For$e_i >= f_{yd} / E_s$, $f_{si} = f_{yd}$

For $e_i < f_{yd} / E_s$, $f_{si} = e_i x f_{yd}$

b)Fe415,

For $e_i > = 0.8 x f_{yd} / E_s$, f_{si}= value obtained from stress-strain curve

For $e_i > = 0.8 x f_{yd} / E_s$, $f_{si} = e_i x f_{yd}$

Stress	Stress level (N/mm²)	Strain
0.800 f_{yd}	288.7	0.00144
o.850f_{yd}	306.7	0.00163

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0.900 f_{yd}	324.8	0.00t92
0.950 f_{yd}	342.8	0.0024J
0.975 f_{yd}	351.8	0.00276
1.000 f_{yd}	360.9	0.00380

Values of stress in concrete :

For e_i> = 0.002,f_{ci} =0 .446 f_{ck}

For e_i> = 0.002,f_{ci} =(446.e_i x (1-250e_i)) x f_{ck}

Values of strain :

Values of strain at different levels areobtained by taking maximumstrain as 0.0035 as the reference value at the highly compressededge.

Design of CG1 :

General Design Parameters:

Below ter - Default Level at t5 000 m

Column size, (B x D) = 230 x 300 mm Column height,(L) = 2625 mm

From analysis results,loads on column

Axial load,(P) = 63.63 kN

MomentX,(M_x) =19.84kN-m

PlomentY,(M_y) =22.86kN-m

F_{ck} = 20.00 N/mm²

500.00 N/mm²

f_y=

Load combination = 1.50 DL + .50 LL

Column Name	Orientation Angle
CI	0.00

Check For Slendemesr :

Slenderness RatioX =(L_ox Effective Length FactorX) /Depth

= (2625 x 1.000) / 300 = 8.75 <= 12.00,

column is not slender in this direction.

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Slenderness Ratiov = (L_o x Effective Length Factory) / Width

= (2625 x 1.000) / 230 = 11.41 <= 12.00,

column Is not slender in this direction.

$M_{x_mineccen}$ =1.273kN.m

$M_{y_mineccen}$ =1.273kN.m

$M_x = \max(M_x, M_{x_mineccen}) + M_{uddx} = 19.836kN.m$

$M_y = \max(M_y, M_{y_mineccen}) + M_{uddy} = 22.859kN.m$

Calculation Of Eccentricities:

As per IS 456: 2000 Clause 25.4,all columns shall be designed for minimum eccentricityequaltotheunsupportedlengthofthecolumn/500pluslateraldimension/30,subjecttoaminimumof20mm.

Actual $eccen_x = M_x / P$ = 19.84/63.63 = 312mm

Actual $eccen_y = M_x / P$ = 22.86/63.63 = 359mm

$eccen_{xmin}$ = (L/500) + (D/30) = (2625/500) + (300/30) = 15mm

$eccen_{xmin}$ = max(15,20)= 20mm

$eccen_{ymin}$ = (L/500) * (B/30) = (2625/500) + (230/30) = 13mm

$eccen_{ymin}$ = max(13,20) = 20mm

$eccen_x$ =max(Actual $eccen_x$, $eccen_{xmin}$)= max(312,20) = 3t2mm

$eccen_y$ =max(Actual $eccen_y$,$eccen_{ymin}$) = max(359,20) = 359 mm

As per IS 456: 2000 Clause39.3,

. when the minimum eccentricity does not exceed 0.OS times the lateral dimension,

thecolumnwlllbedesignedasanaxlallyloadedcolumn.

eccenX(3 12 mm) > 0.05 x 230(12 mm)

andeccenY(359 mm) > 0.05 x 300(15 mm)

Hence the column wlll be designed as a column with biaxial moments.

Provided steel :

Provide #16 - 6 nos. (1206 mm^2)

biaxial Creek Calculations :

For X axis :

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Row no (1)	A_{si} (Mm^2)	e_i	F_{si} (N/mm^2)	F_{ci} (N/mm^2)	(F_{si} -F_{ci}) (N/mm^2)	P_{usi} (kN)	X_i (mm)	M_{usi} (kN-m)
1	402	0.00201	373.57	8.92	364.65	146.63	102	14.96
2	402	-0.00117	- 233.33	0.00	-233.33	-93.83	0	0.00
3	402	-0.00434	- 434.80	0.00	-434.80	-174.84	- 102	17.83
Total						-122.04		32.79

X_{ux} =113mm $P_{uc} = C_1, F_{ck}, B, D$

C_1 = $0.36k_u$ = 0.36 x 113/300 = 0.135

P_{uc} = 0.135 x 20.00 x 230 x300

= 186.30 kN

P_{ux1} = $P_{uc} + P_{us(Total)}$

186.30 + (-122.04)

=64.26kN>P,henceO.K.

M_{uc} = $P_{uc}.(0.5.D – C_2. D)$

C_2 = 0.416 K_u=0.416 x 113/300

= 0.156

M_{uc} = 186.30 x (0.5 x 300 - 0.156 x 300) = 19.23 kN-m

M_{ux1} = $M_{uc} + M_{us(Total))}$

= 19.23 + (32.79)

= 52.02 kN-m

For Y axis :

Row no (1)	A_{si} (Mm^2)	e_i	F_{si} (N/mm^2)	F_{ci} (N/mm^2)	(F_{si} -F_{ci}) (N/mm^2)	P_{usi} (kN)	X_i (mm)	M_{usi} (kN-m)
1	603	0.00135	269.52	7.90	261.55	157.76	67	10.57
2	603	-0.00466	- 434.80	0.00	- 434.80	-262.27	- 67	17.57
Total						-104.50		28.14

X_{uy} = 78mm $P_{uc}=C_1, F_{ck}, B, D$

C_1 =0.36 K_u=0.36x78/230=0.122

P_{uc} = 0.122 x 20.00 x 300 x 230

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= 168.59kN

P_{u1} = P_{uc} + $P_{us(Total))}$

= 168.59 + (-104.50)

=64.09kN>P,henceO.K.

M_{uc} = P_{uc}.(0.5.B – C_2. B)

C_2 = 0.416 K_u = 0.416 x78/230

= 0.141

M_{uc} = 168.59 x (0.5 x 230 - 0.141 x 230) = 13.91 kN-m

M_{u1} = M_{uc} + $M_{us(Total))}$

= 13.91 + (28.14)

P_u/P_{uz} = 0.060

For P_u/P_{uz}< = 0.2, $\alpha\eta$ = 1.0

For P_u/P_{uz}> = 0.8, $\alpha\eta$ = 2.0

For P_u/P_{uz} = 0.060, $\alpha\eta$ = 1.000

$((M_{ux}/M_{ux1})^{\alpha\eta}+M_{uy}/M_{uy1})^{\alpha\eta})$

= $((19.84/52.02)^{1.000})$ + $((22.86/42.06)^{1.000})$

= 0.925 < 1.0,hence O.K.

Ties Details :

Loadcombinationfortiesdesign=t.50DL+1.50LL

Calculation of diameter of ties:

AsperIS456:2000Clause26.5.3.2(c),thediameterofthepolygonalLinksor lateral ties shall be not less than thefollowing

i)one-fourth the diameter of the largest longitudinal bar = t6/4 = 4mm

Ii)in no case less than = 6 mm

Requireddiameter=maximumof(4,6)=6mm

#Provide ties 8 @230 mm c/c

Calculation of spacing of tiles :

AsperIS456:2000Clause26.5.3.2(c),thepitchoftransversereinforcementshallbe

not more than the least of the followingdistances:

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i)the least lateral dimension of the compression member = 230mm

ii)sixteen times the smallest diameter of the Longitudinal reinforcement bar to betied

= 16 x 16 = 256 mm

iii)300mm

Requiredspacing=minimumoF(230,256,300)=230mm

#Provideties 8 @ 230 mmc/c

SUMMARY:

CGI :ter - Default Level at 15.000
mProviderectangularsection:230x300mm
Provide #416 - 6 nos. (1206mm')
Provide ties #8 @ 230 mm c/c
Provide2leggedtiesalongwidthand
2 legged ties alongdepth.

General Design Parameters :

BelowIf-DefaultLevelat12.000m

Column size, (B x D) = 230 x 300 mm

From analysis results,Loads on column

Axial load,(P) = 179.65 kN

MomentX,(M_x) = 16.90 kN-m

MomentY,(My) = 19.02 kN-m

F_{ck} =20.00 N/mm²

FOOTING DESIGNSCHEDULE:

(As Per IS 456 :2OOO)

Schedule of Footings -FlatFootings:

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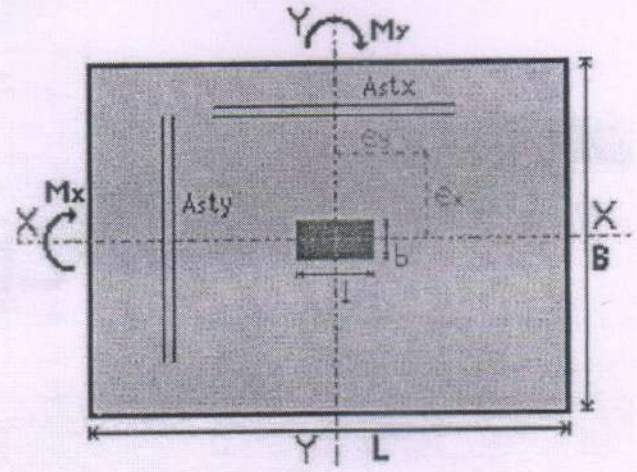
Footing Group	Column Nos	Concrete Grade	Steel Grade	Footing Size I I to X I I to Y (mm)	Total Depth (D) (mm)	steel	
						I I to X	I ItoY
FG1	C1	M20	Fe500	1575 x 1650	325	9 - #10	9 - #10
FG2	C2	M20	Fe500	2550 x 2700	550	16 - #12	15 - #12
FG3	C3	M20	Fe500	1975 x 1900	400	12 - #10	12 - #10
FG4	C4	M20	Fe500	2175 x 2250	450	17 - #10	17 - #10
FG5	C5	M20	Fe500	2100 x 2175	450	15 - #10	15 - #10
FG6	C6	M20	Fe500	2450 x 2375	500	14 - #12	20 - #10
FG7	C7	M20	Fe500.	2475 x 2475	500	14 - #12	20 - #10
FG8	C8	M20	Fe500	2200 x 2150	450	16 - #10	16 - #10
FG9	C9	M20	Fe500	2050 x 2100	450	14 - #10	13 - lt10
FG10	C10	M20	FeS00	2300 x 2225	475	17 - #10	17 - #10
FG11	C11	M20	Fe500	2325 x 2275	475	18 - #!0	18 - #10

FOOTING DESIGNDetail Report:

(Design by Limit State Method as per IS 456 : 2000)

Standard Conventions:

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Individual Footing

Design of FGI :

Size of col./ped,(l x b) = 0.230 x 0.300 m

SBCofsoil = 28000kN/m²

F_{ck} = 20.00N/mm²

Fy = 500.00N/mm²

Load combination = 1.50DL + 1.50LL

LoadsOn Footing

Factored axial loadoncolumn,(P) = 443.94kN

Workingaxialloadoncolumn,{P_w) =295.96kN(Fromanalysisresults)

SelfWeightoffootingassumed(S_w) = 10.00 % of Pg = 29.60kN

Total Working load,(P_1) = {P_w)+ (S_w)= 325.55 kN

Total Ultimate load,(P_{u1}) = (P)+ 1.5x(S_w) = 488.33 kN

$BM_{@X-X}$,(WM_X) = 0.80kN-m

$BM_{@Y-Y}$,(WM_Y) = 1.32kN-m

Factored BM $_{@X-X}$,(c) =i.20lrN-m

FactoredBM $_{@Y-Y}$,(M_Y) = 1.98kN-m

In Loadcombination with DL load case :

If load factor for DL loadcase is less than 1.0,this factor will be retained while computing the working load.

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Footing Area

Areaforaxialload,(A_f) = P_1/SBC = 325.55 / 130.00 = 2.504 m^2

Eccentricity @x-x,(e_x)=WM_x /P_w= 0.0027 m

= 0.003 m

Eccentricity @y-y,(e_y) =WM_y /P_w= 0.0045 m

= 0.004 m

Footing size Y,(B) =(0.5 x (b-1))

+ ($\sqrt{((((0.5 \times (b-I)) \times (0.5 \times (b-I)))+A_f)}$)

Footing size X,(L) =Aj/B = 1.548 m

ModifiedareaforMx,(A_{fx}) = (P I /SBC) x (1+6.e_x /B)
= (325.55/130.00) x (I + 6 x 0.0027/1.618)

= 2.529m^2

ModifiedareaforMy,(A_{fy}) = (P I /SBC) x (1+6.e_y /L)

= (325.55/I30.00) x (I + 6 x 0.0045/1.548)

= 2.547 m'

Size,(LxB) = 1.575 x 1.650m

Areaprovided ,(A_P) = 2.599m^2>A_F

SBC Check:

Upward pressure due to axial load,(P_{r1w}) = P1/ (L x B) = 125.27 kN/m^2

Upward pressure due to M_x,(P_{r2w}) = WM_x / (L x B x B/6) = 1. 1161kN/m^2

Upward pressure due to M_y,(P_{r3w}) = WM_y /(BxLxL/6)=1.9316kN/m^2

Minimumupwardpressure,(Up_{Maxw}) = P_{r1w} +P_{r2w}+ P_{r3w} =128.32kN/m^2

Minimumupwardpressure,(Up_{Minw}) = P_{r1w} +P_{r2w}+ P_{r3w} =122.23kN/m^2

Up_{Maxw}(128.32kN/m^2) <SBC(130.00kN/m^2),Hence,safe.

Pressures For Different Load Combinations:

Load Combinations	Design Pressures(kN/m^2)		Working Pressures(kN/m^2)	
	Maximum	Minimum	Maximum	Minimum
1.50 DL + 1.50 LL	192.48	183.34	128.32	122.23

Bending Moment Calculations:

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As per IS 456: 2000 Clause 34.2.3.2,ctitical section for checking bending momentinthedesign'ofanisolatedconcretefootingwhichsupportsacolumnshallbe asection1ocat.-datthefaceofthecolumnorpedestal.

Upward pressure due to axial load,(P_{r1}) = P_{u1} / (L x B) = 187.91 kN/m²

Upward pressure due to M_x,(P_{r2}) =M_x/(LxBxB/6)=1.6741kN/m²

Upward pressure due to M_y,(P_{r3}) =M_y/(BxLxL/6)= 2.8975kN/m²

Maximumupwardpressure,(Up_{Maxw}) =P_{r1} +P_{r2}+ P_{r3} = 192.48kN/m²

Minimumupwardpressure,(Up_{Minw}) = P_{r1} +P_{r2}+ P_{r3} = 183.34kN/m²

Project1on-X,(X_{proj}) = (L -l)/2

= (1.575 - (L230)/2

= 0.673 m

Critical upward pressure,(Up_{cr},) = Up_{Min} + (L - X_{proj}) x ((Up_{Max} . Up_{Min})/L)

= 183.34 -t (1.575 - 0.673) x ((192.48 - 183.34)/1.575)

= 188.58 ky/rnz

Upward pressurearea ,(Up_{crarea},) = X_{proj} x ((Up_{cr} . Up_{Max})/2)

= 0.673 x (188.58+192.48)/2

= 128.13 kN/m

CGdistancefromcolumnface,(CG_{Dist}) = X_{proj} x ((Up_{cr}+(2 x Up_{Max}))

= 0.673 x (188.58 -r 2 x 192.48)

/ (3 x (l 88.58 + 192.48))

= 0.337 m

DesignBM_x,(M_{ux}) = Up_{crarea}, xCG_{Dist} xB

= 128.13x0.337 x 1.650

= 71.33 kN-m

Projection-Y,(Y_{proj}) = (B - b)/2

= (l.650 - 0.300)/2

= 0.675 m

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Critical upward pressure,(Up_{cra}) = Up_{Min} + (B - Y_{proj}) x ((Up_{Max} - Up_{Min})/B)

= 183.34 + (1.650 - 0.675) x ((192.48 - 183.34)/1.650)

= 188.74 kN/m²

Upward pressurearea ,(Up_{crarea},) = Y_{proj} x ((Up_{cr}+ Up_{Max}))/2

= 0.675 x (1g8.74-+ 192.48)/2

= 128.66 kN/

CGdistancefromcolumnface,(CG_{Dist}) = Y_{proj} x ((Up_{cr}+(2 x Up_{Max}))

/3 x (Up_{cr}+(Up_{Max}))

= 0.675 x (188.74 + 2 x 192.48) / (3 x (188.74 + 192.48))

= 0.339 m

Design BM_y,(M_{uy}) = Up_{crarea}, xCG_{Dist} xL

= 128.66 x 0.339 x 1.575

= 68.62 kN-m

Depth Calculations :

Depth required for punching shear:

AsperIS456:2000Clause31.6.3.1,thecalculatedshearstressatthe criticalsection shall not exceed k_Sx τ_{auc}.

where,

K_S= 0.5 +β_{ac}, but not greater than 1,β_{ac} being the ratio of short side to the long side of column, and

τ_{auc}= 0.25 x√(f_{ck})in limit state method of design.

Depthrequiredforpunchingshearis calculatedbyequatingtheactualstress topermissiblestressandbysolvingtheresultingquadraticequation.

Totaldepthassumed,(D) = 0.325m

Bottomcover = 0.050m

Effective Depth Calculation :

Effectivedepth,(d_{effx}) = 0.270m

Effectivedepth,(d_{effy}) = 0.260m

Effectivedepth,(d_{eff}) = 0.270m

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Depth I I to X axis at critical section,(d_{punch_x}) = 0.270m

Depth I I to X axis at critical section,(d_{punch_y}) = 0.270m

Resisting area I I to X axis,(A_{rx}) $=d_{punch_x} x\ 2\ x\ (1+d_{eff})$

$= 0.270 \times 2 \times (0.230 + 0.270)$

$= 0.270\ m^2$

Resisting area I I to Y axis,(A_{ry}) $=d_{punch_y} \times 2 \times (b+d_{eff})$

$= 0.270 \times 2 \times (0.300 + 0.270)$

$= 0.308 m^2$

Resisting area I ,(A_r) $=A_{rx} + A_{ry}$

$= 0.578\ m^2$

Shear force,(V) $= (P/A_p)x(A_p – (1+ d_{eff})x(b+ d_{eff}))$

$= (443.94 / 2.599) \times (2.599 - (0.500 \times 0.570))$

$= 395.25$ kN

Actual shear stress $= V / A_r$

$=395.25/0.578$

$=684.06 kN/m^2$

Permissible shear stress $= 0.25 \times \sqrt{(f_{ck})} \times ks$

ks $= 0.5+betaC$

betaC = short column side/ long column side

$= 0.230 / 0.300 = 0.767$

Ks $= 0.5 + 0.767 = 1.267>I$,ks =I

Permissible shear stress $= 0.25 \times 4.4721 \times 1.000$

$= 1.118\ N/mm^2$

$= 1118.03\ kN/m^2$

Actualshearstress<Permissibleshearstress.

Hence,Safe in punching shear.

CalculationOfF¢omentOfResistance:

Totaldepth,(D) = 0 325m

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Minimum depth,(D_{min}) = 0.000 m

For(Fe500),k =0.456

R = 0.36 x f_{ck} x k x (I - 0.42 xk)

= 0.36 x 20.000 x 0.456 x (I- 0.42 x 0.456)

= 2.655

Moment of resistanceX = R x width at NA x d_{effx}x d_{effx}

= 2.655 x 1.650 x 0.270 x 0.270

= 321.42 kN-m > 71.33 CN-m

Moment of resistance Y = R x width at NA x d_{effy}x d_{effy}

= 2.655 x 1.575 n 0.260 x 0.260

= 284.50 kN-m > 68.62 kN-m

Steel Calculations :

For M_{ux}:

Cross sectional area,(C/S_{area})= width at NA x d_{effx}

= 1.650 x 0.270

= 0.445 m^2

Val = 1 – [(4.6 x Mux)/(f_{ck}x Bxd_{effx}^2)]

Required steel area,(A_{stx}) = [(0.5(f_{ck}/f_y) x (1 – √(val)0]x C/S_{area}

= 0.000630 m^2

= 630 mm^2

Required steel pt. = 0.14139 %

Provide #10 - 9 nos.(707 mm^2)

Spacing = (L - dia. - (2 x end cover))/(no - I)

= (I .650 - 0.010 - (2 x 0.050))/(9 -I)

= 0.192 m < maximum spacing(0.200 m)

For M_{uy}:

Cross sectional area,(C/S_{area})= width at NA x d_{effx}

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=1.575 x 0.260

= 0.410 m^2

Val = $1 - [(4.6 \times Mux)/(f_{ck} \times B \times d_{effx}^2)]$

Required steel area,(A_{stx}) = $[(0.5(f_{ck}/f_y) \times (1 - \sqrt{(val)0}] \times C/S_{area}$

= 0.000630 m^2

= 631 mm^2

Required steel pt. = 0.15417 %

Provide #10 - 9 nos.(707 mm^2)

Spacing = (B- dia. - (2 x end cover))/(no - I)

= (I .575 - 0.010 - (2 x 0.050))/(9 -I)

= 0.183 m < maximum spacing(0.200 m)

Check ForOneWay Shear:

As per IS 456: 2000 Clause 33.2.4.I,thc critical section for this

condition shall be assumed as a vertical section located at a distance equal to the effective depth of the footing from the face of the column or pedestal.

For X-axis :

Project1on-X,(X_{proj}) = $(L - I)/2 - d_{effx}$

= (1.575 - 0.230)/2 - 0.270

=0.403m

Critical upward pressure,(Up_{cr}) = $Up_{Min} + (L - _{proj}) \times ((Up_{Max} - Up_{Min})/L)$

= 183.34 + (1.575 - 0.403) x ((192.48 - 183.34)/1 .575)

= I 90.14 kN/m^2

Shear force,(V_x) = $(_{proj} \times B) \times (Up_{cr} + Up_{Max})/2$

= (0.403 x 1.650) x 191.31

= 127.06 kN

Resisting area I ,(A_r) = $d_{effx} \times B$

= 0.270 x 1.650

= 0.445 m

Percentage of steel at critical section = (0.000707 x 100)/0.445

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= 0.15867

Permissible shear stress = 294.75 kN/m²

Actual shear stress = V_x / A_r = 127.06 / 0.445

= 285.20 kN/m²

<permissible shear stress(294.75 kN/mz)

For Y-axis :

Project1on-Y,($_{proj}$) = (B-b)/2 - d_{effy}

= (I.650 - (1.300)/2 - 0.260

= 0.415 m

Critical upward pressure,(Up_{cr}) = Up_{Min} + (L - $_{proj}$) x ((Up_{Max} - Up_{Min})/L)

= 183.34 + (1.650 - 0.415) x ((192.48 - I 83.34)/1.650)

= 190.18 kN/m

Shear force,(V_y) = ($_{proj}$x L) x (Up_{cr}+ Up_{Max})/2

= (0.415 x 1.575) x I91.33

= 125.06 kN

Resisting area I ,(A_r) =d_{effx} x L

= 0.260 x I .575

= 0.410 m²

Percentage of steel at critical section = (0.000707 x 100)/0.410

= 0.1726 I

Permissible shear stress = 305.90 kN/m'

Actual shear stress =Vy / AT= 125.06 / 0.410

= 305.39 kN/m

<permissible shear stress(305.90 kN/m²)

Check ForBearing:

As per IS 456: 2000 Clause 34.4, the bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value

equal to √ (A I/A2), but not greater than 2.

Supporting area for bearing of footing,(A I) = (I + (4 x D)) x (b + (4 x D))

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= (0.230 + (4 x 0.325)) x

(0.300 + (4 x 0.325))

= 2.448 m²

Loaded area at the column base,(A2)= 0.069 md

√(AI/A2 = /(2.448/0.069) = 5.956 > 2

= 2.000

Actual bearing stress = P/A2 443.94/0.059

6433.56 kN/m²

Permissible bearing stress = 0.45 x f_{ck} x √(A I /A2)

= 0.25 x 20.00 x 2.000

= 18.000 N/mm²

= 18000.00 kN/m²

Actual bearing stress < Permissible bearing strees.

Hence, Safe in beaaring

Check For Sliding Along X axis :

Load combination = I.50DL + I.50LL

Self wt.offooting = 21 .11kN

Stabilizing force,(W) = 465.05kN

Horizontal force,(H) = 2.72 kN

Soilresistance,(S) = 53.21 kN>H

Hence,check for sliding is not required.

Check For Sliding Along Yaxis :

Loadcombination = I.50DL+ I.50LL

Selfwt.offooting = 21.11 kN

Stabilizing force,(W) = 465.05 kN

Horizontal force,(H) = 2.77kN

Soilresistance,(S) = 50.79 kN>H

Hence,check for sliding is not required.

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Bond Check Calculations for Footing Bars:

AsPerIS456:2000,Clause26.2.I,TheRequiredDevelopmentLength,Ldisgivenas

L_d = $(\Phi \times \sigma)/(4 \times \tau_{bd})$

where,

Φ = Nominal diameter of the footing bar in X or Y direction,

σ = Stress in the bar at the section considered, at design load and

τ_{bd} = Design bond stress.

Hence ,L_{dx} = (10 x 0.87 x 500.00) / (4 x 1.92)

= 566.406

Available Development Length=(F_d - C_d) / 2 - End Cover + End Anchorages(If required)

where,

F_d =Footing size in X / Y direction

C_d =Column size in X / Y direction

End Anchorage =4 xΦ, for a L hook

= 8 x Φ, for a 90 degrees bend

In X direction = (1.575 - 0.230) x 0.5 - 0.050

= 622.500 mm

In Y direction = (1.650 - 0.300) x 0.5 - 0.050

= 625.000mm

Hence, the Bond check for footing bars is satisfied.

Bond Check Calculations For Column Bars in Tension:

L_d = $(\Phi \times \sigma)/(4 \times \tau_{bd})$

where, other terms remaining same,

Φ = Nominal diameter of columnbars

τ_{bd} = Design bond stress in ***Tension*** for columnbars

DiameterofColumnbard$_1$ = 12mm

Required Development Length for 'd_1' = (12 x 0.87 x 500.00) / (4 x 1.92)

= 680 mm

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Available development length= Min(d_{effx}/d_{effy}) + H_{proj}

Where,

H_{proj} = Horizontal projection of columns bars in footing

= Min(270.000 / 260.000) + 300.000

= 560.000 mm

Hence, the Bond check for column bars in tension is satisfied

Bond CheckCalculations For Column Bars in Compression:

Required Development Length For Column Bars

L_d = (Φ x σ)/(4 x τ_{bd})

where, other terms remaining same,

Φ = diaof columnbars

τ_{bd} = Design bond stress in ***Tension*** for columnbars

Here,

Diameter of Column bar'd_1' = 12 mm

Required Development Length for 'd_1' ' = (I 2 x 0.87 x 500.00) / (4 x 2.40)

= 544 mm

AvailableDevelopmentlength = Min(d_{effx}/d_{effy})

= Min(270.000 / 260.000)

= 260 mm

Hence, the Bond check for column bars in compression is satisfied.

Summary:
Provide isolated flat footing of size
1.575 x 1.650 x 0.325 m
Provide #l0 - 9 nos along the length of footing
Provide #10 - 9 nos along the width of footing.
(Bars should be bent and carried to top.)

Design of FG2:

Size of col./ped, (l x b) = 0.230 x 0.375 m

SBCof soil =280.00kN/m²

f_{ck} = 20.00N/mm'

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f_y = 500.00 N/mm²

Load combination = 1.50DL +1.50LL

Loads On Footing

Factored axial loadoncolumn,(P) = 1192. 73kN

Working axial loadoncolumn,(P_w) = 79S. 15 kN (From analysis results)

Self Weight of footingassumed(Sq) = 10.00 % of Pg = 79.52kN

Total Working load,(P_1) =(P_w)+(S_w) = 874.67 kN

Total Ultimate load,(P_{u1}) = (P) + I .5 x (S_w) = 1312.00 kN

BM $_{@x-x}$,(WM_x) = 5.8 lkN-m

$BM_{@y-y}$,(WM_y) = 1.79kN-m

Factored $BM_{@x-x}$, (M_x) =8.71kN-m

Factored$BM_{@y-y}$,(M_y) = 2.69kN-m

In Loadcombination with DL load case :

If load factor for DL loadcase is less than 1.0,this factor will be retained while computing the working load.

Footing Area

Area for axial load,(A_f)= P_1/SBC = 874.67 / 130.00 = 6.728 m²

Eccentricity $_{@x-x}$,(e_x) = WM_x /P_w = 0.0073 m

Eccentricity $_{@y-y}$,(e_y) = WM_y/ P_w = 0.0023 m

= 0.002 m

Footing sizeY,(B) = (0.5 x (b-I))

+ (v ((((0.5 x (b-I)) x (0.5 x (b-I)))+A_f)))

= 2.667 m

FootingsizeX,(L) = A_f/B = 2.522 m

Modified area fotM_x,(A_{fx}) = (P_1/SBC)x (1+ 6 e_x/B)

= (874.67/130.00) x (I + 6 x 0.0073/2.667)

= 6.839 m²

Modified area for M_y,(A_{fy})= (P_1/SBC)x (1+ 6e_y/L)

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= (874.67/130.00) x (1 + 6 x 0.0023/2.522)

= 6.764 m

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