ES. KUMAR, M.Tech., (Structures)
Civil & Structural Engineering Consultant
CMDA Regd. Structural Engineer Grade I, No.SE/Gr-I/24/02/007
DTCP Regd. Structural Engineer Grade I, No.CGLDIST/SE/Gr-I/2019/04/001
Greater Chennai Corporation Regd. Structural Engineer, No. RSE 100092024

Chennai Office:
KIRTE ASSOCIATES
Plot No. 1233, First Floor, Flat No. 1B
Priya Ajay Arcade, 18th Main Road,
Anna Nagar West, Chennai – 600 040.

FORM-4B

STRUCTURAL STABILITY CERTIFICATE BY REGISTERED STRUCTURAL ENGINEER

(To be furnished on completion of building)

I/We **E.S.Kumar** (CMDA Registration No.SE/GR.I/2024/02/007) have undertaken assignment as Structural Engineer for the detailed design and construction of NHRB with Stilt floor (Mechanized Pit Puzzle Parking) + 3 Floors Commercial (Retail / Showroom) Building (14.00 m Height) at Plot No. 4951, Old Door No. AC-64/1, New Door No. 254/1, AC Block 4th Avenue and 5th Avenue, Anna Nagar, Chennai – 600040, Comprised in Old S.No. 69 part, 73 part & 74 part, T.S.No. 158, Block No. 1B of Naduvakkarai Village within the limits of Greater Chennai Corporation

It is certified that the building has been designed as a RCC structure to withstand the dead load, imposed load and wind load/earthquake loads ensuring structural safety and serviceability as per the norms and guidelines prescribed in applicable relevant standards and Codes of Bureau of Indian Standards.

Further after the detailed inspection of the building, it is certified that the constructed building is structurally safe and sound for the purpose for which it is constructed.

Date: 16.03.2026

Yours faithfully,

Signature, Name and seal of the
Registered Structural Engineer

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 98404 48613

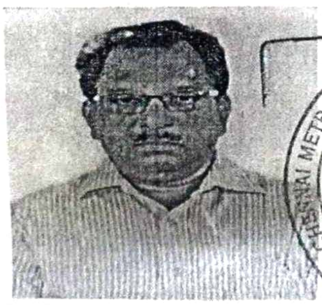
CHENNAI METROPOLITAN DEVELOPMENT AUTHORITY
'ThalamuthuNatarajan Building', No.1, Gandhi Irwin Road,
Egmore, Chennai-600 008.

PROFESSIONAL REGISTRATION CERTIFICATE

This is to certify that the **Thiru. E.S. KUMAR** has enrolled as **Registered Structural Engineer Grade-I (SE)** with Chennai Metropolitan Development Authority (CMDA) who has the qualification of **M.Tech.,(Structural Engg.)** and experience in the relevant field for **27years**.

He has been given the Registration No.**SE/GR-I/2024/02/007** on **27.02.2024.** He shall be required to comply with the duties and responsibilities prescribed by the Government of Tamil Nadu in Part-V, "Registration of Professionals", Tamil Nadu Combined Development and Building Rules, 2019 approved vide G.O.(Ms) No.18 MAWS (MA1) Dept. dated 04.02.2019.

This Registration is valid upto **26.02.2029**.

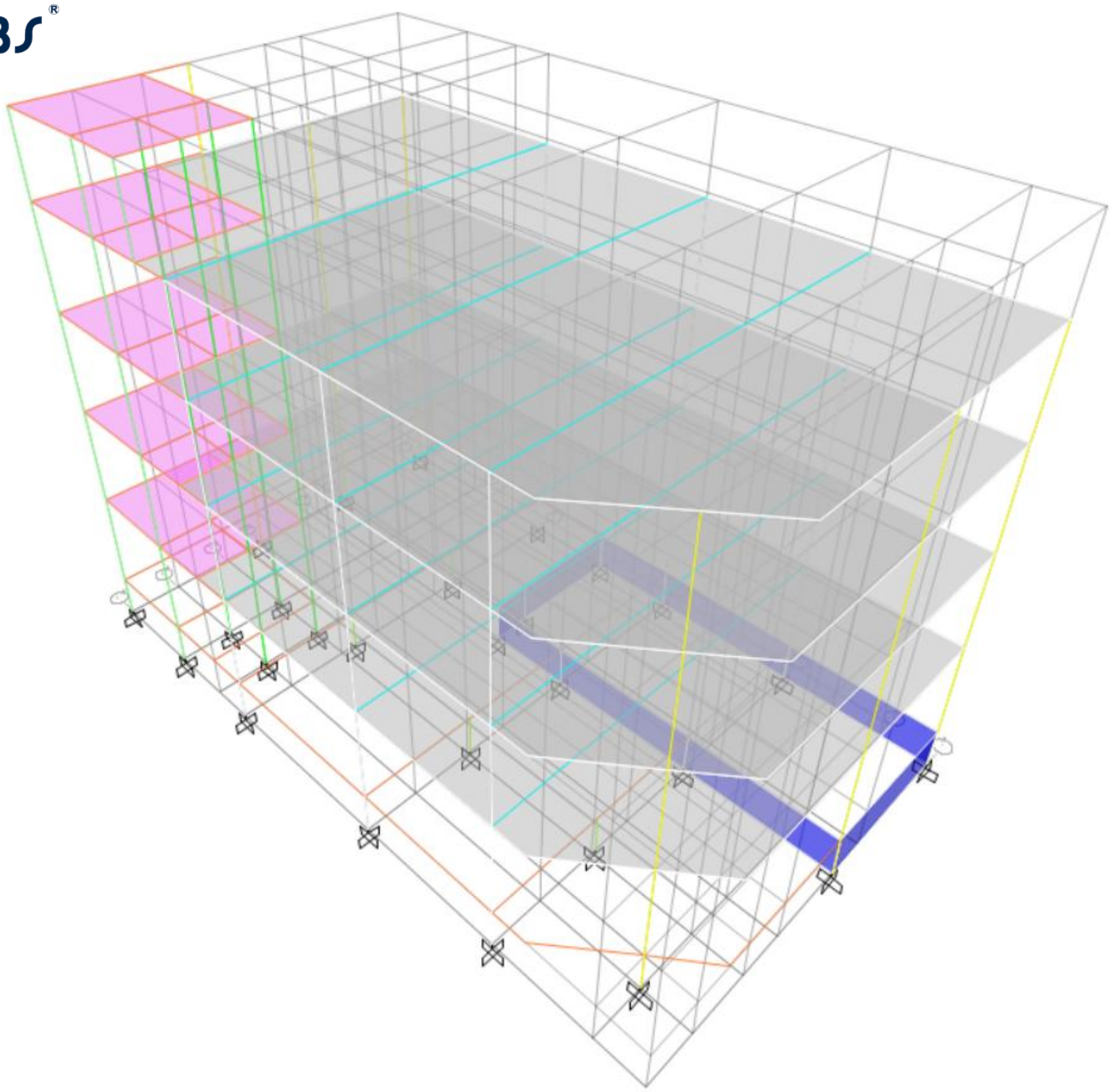


CHENNAI METROPOLITAN DEVELOPMENT AUTHORITY
CHENNAI-600 008.

22/2/24

(A. MEGANATHAN)
Superintendent

ETABS



PROJECT TITLE: THE PROPOSED CONSTRUCTION OF COMMERCIAL (RETAIL) BUILDING AT PLOT NO 4951, OLD DOOR NO: AC-64/1, NEW DOOR NO: 254/1, AC BLOCK 4TH AVENUE & 5TH AVENUE, ANNA NAGAR, CHENNAI - 600 040. COMPRISED IN OLD S.NO: 69 Pt & 73 Pt, T.S.NO: 158 OF BLOCK NO: 1B OF NADUVAKKARAI VILLAGE, AMINJIKARAI TALUK, ZONE VIII, DIVISION 103 OF GREATER CHENNAI CORPORATION

STRUCTURAL CONSULTANT

E.S. Kumar., M.Tech.,

Principal Structural Consultant

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS

Plot No. 1233, First Floor, Flat No. 1B, Priya Ajay Arcade, 18th Main Road, Anna Nagar west, Chennai – 600040

Mob: 98404 48613

EMail Id: kirtiassociates@gmail.com

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS

Plot No. 1233, First Floor, Flat No. 1B, Priya Ajay Arcade, 18th Main Road, Anna Nagar west, Chennai – 600040

Mob: 98404 48613 eMail Id: kirtiassociates@gmail.com

STABILITY CERTIFICATE

For,

This is to certify THE PROPOSED CONSTRUCTION OF COMMERCIAL (RETAIL) BUILDING AT PLOT NO 4951, OLD DOOR NO: AC-64/1, NEW DOOR NO: 254/1, AC BLOCK 4TH AVENUE & 5TH AVENUE, ANNA NAGAR, CHENNAI - 600 040. COMPRISED IN OLD S.NO: 69 Pt & 73 Pt, T.S.NO: 158 OF BLOCK NO: 1B OF NADUVAKKARAI VILLAGE, AMINJIKARAI TALUK, ZONE VIII, DIVISION 103 OF GREATER CHENNAI CORPORATION

Is designed to resist earthquake forces for Zone III. It is hereby certified that

1. The minimum grade of Concrete is M25
2. The design of the structure is carried out as per the latest code IS-456.
3. The loading is taken as per the requirements of Code of practice for Loading IS-875 (all parts) and Seismic forces as per the latest code IS-1893:2016

The structure is Checked and found to be Safe and Stable taking into consideration structural safety requirements as published in the "National Building Code" published from time to time and also confirms to the material requirements with the minimum standards prescribed by the Indian Bureau of Standards, India.

For KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS

Date: 02-06-2024

Place: Chennai

E.S. Kumar., M.Tech.,

Principal Structural Consultant

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 98404 48613

INDEX

1. INTRODUCTION:

The structure is a "Commercial building" consisting of Stilt Floor + 3 floors only. It is designed as a Reinforced concrete framed structure supported on deep foundations as per the Geo technical engineer recommendations provided in the soil investigation report.

1.a Building Dimensions

Length (X-direction) - 14.14 M

Width (Y -direction) - 24.20 M

1.b Building Height

Story	Clear Height	unit
ROOF LVL	3.50	m
3RD SLAB LVL	3.50	m
2ND SLAB LVL	3.50	m
1ST SLAB LVL	3.50	m
PLINTH LVL	3.50	m
Base	0	N/A

*Foundation system is design with deep foundation _ Pile foundation systems and strap beam for negative moment

**Typical clear height of 3.50m is followed for all upper floors

2. MATERIALS USED:

Following materials are generally used in the design of the various components of the structure.

1. Minimum Design mix of M30 shall be used for Columns & M25 for other elements Reinforced concrete part of the structure.
2. Fe 500 grade reinforcement steel for all diameters.

The self-weight other various elements are computed based on the unit weight of materials as given Below

MATERIALS	UNIT WEIGHT KN/M^3
Steel	78.5
Plain cement concrete	24.00
reinforced cement concrete	25.00
soil fill (landscape)	20.00(Saturated)
Water	10.00
Cement concrete screed	24.00

Nominal weight aggregates shall be considered for arriving at self-weight of all concrete works

3. CODES USED:

The relevant Indian Standard Codes, as given below, shall be followed for structural design:

Code of Practice For design Loads (Other Than Earthquake) For Buildings and Structures– Unit weights of buildings materials and stored material.

S.No.	Code	Description
1	IS-875 (Part 1) - 1987	Code of Practice For design Loads (Other Than Earthquake) For Buildings and Structures – Unit weights of buildings materials and stored material.
2	IS-875(Part 2) - 1987	Code of Practice for Design Loads (Other than earthquake) For buildings and structures – Imposed loads.
3	IS-875 (Part 3) – 1987	Code of Practice for Design Loads (Other than earthquake) For buildings and structures – Wind loads
4	IS-875 (Part 4) - 1987	Code of Practice for Design Loads (Other than earthquake) For buildings and structures – Snow loads.
5	IS-875 (Part 5) – 1987	Code of Practice for Design Loads (Other than earthquake) For buildings and structures – Special loads and load combinations.
6	IS: 456-2000	Code of Practice for Plain and Reinforced Concrete.
7	IS: 1786 – 2008	High Strength Deformed Steel Bars and Wires for Concrete Reinforcement – Specification.
8	IS: 13920 – 1993	Ductile detailing of reinforced concrete structures subjected to seismic forces – Code of Practice
9	IS: 800 – 1984/2007	Code of Practice for General Construction in Steel
10	IS: 1893 – 2016	Criteria for Earthquake resistant design of structures.
11	IS3370-(PART I & II) – 2009	Code of Practice for Concrete structures for the storage of liquids – Reinforced concrete structures
12	IS 2950(Part I) - 1981	Code of Practice for Design and construction of Raft foundation

4. DESIGN DATA:

1.	No. of floors	=	STILT + 3 floors
2.	Height of floors	=	Varying as per section 1.b
3.	Density of Reinforced Concrete	=	25kN/m^3
4.	D.L. due to Partitions		
	(Light Partition as per IS 875)	=	1.50 kN/m^2
5.	D.L. due to Flooring	=	1.50 kN/m^2
6.	Live Load in stairs, lobby & corridors	=	3.0 / 4.00 kN/m^2
7.	Live Load in General rooms	=	2.00 / 3.0 kN/m^2
8.	Live Load in Class rooms	=	3.0 /4.00 kN/m^2
9.	Live Load in Heavy areas	=	4.00 / 5.00 kN/m^2
10.	Live Load in Toilets & Bathrooms	=	2.00 kN/m^2
11.	Live Load in Lift Machine Room,		
	AHU Room, Heavy dining area	=	5.00 kN/m^2
12.	Seismic Zone	=	III
13.	Design Mix	=	M25
14.	SBC as per Soil report	=	Pile foundation (Refer soil report)

5. ANALYSIS OF STRUCTURE:

The Structure is analysed using ETABS 2019 software. All beams & slabs are designed as conventional beam & slabs.

6. DESIGN OF CONCEPT:

Limit State concept of design has been used as per IS:456-2000 & SP – 16. The detailing of reinforcement shall be done satisfying the requirements of IS: 456, SP-16, IS: 13920, IS:4326 and SP-34.

The Slabs and beams are checked for deflection as per IS:456. Cover for Reinforcement is considered as per Table 16A of NBC, for a Fire resistance of 2 hrs.

7. LOAD CASES:

1. Dead Load (DL)
2. Live Load (LL)
3. Earthquake Load (EL)

7.2 Load Combinations:

The following combinations of Loads as per IS 1893 is considered in ETABs Analysis. TABLE: Load Combination Definitions			
Name	**Load Name**	**SF**	**Notes**
UDCon1	Dead	1.5	Dead [Strength]
UDCon2	Dead	1.5	Dead + Live [Strength]
UDCon2	Live	1.5	
UDCon3	Dead	1.2	Dead + Live + Static Earthquake [Strength]
UDCon3	Live	1.2	
UDCon3	EQX	1.2	
UDCon4	Dead	1.2	Dead + Live - Static Earthquake [Strength]
UDCon4	Live	1.2	
UDCon4	EQX	-1.2	
UDCon5	Dead	1.2	Dead + Live + Static Earthquake [Strength]
UDCon5	Live	1.2	
UDCon5	EQY	1.2	
UDCon6	Dead	1.2	Dead + Live - Static Earthquake [Strength]
UDCon6	Live	1.2	
UDCon6	EQY	-1.2	
UDCon7	Dead	1.5	Dead + Static Earthquake [Strength]
UDCon7	EQX	1.5	
UDCon8	Dead	1.5	Dead - Static Earthquake [Strength]
UDCon8	EQX	-1.5	
UDCon9	Dead	1.5	Dead + Static Earthquake [Strength]
UDCon9	EQY	1.5	
UDCon10	Dead	1.5	Dead - Static Earthquake [Strength]
UDCon10	EQY	-1.5	
UDCon11	Dead	0.9	Dead (min) + Static Earthquake [Strength]
UDCon11	EQX	1.5	
UDCon12	Dead	0.9	Dead (min) - Static Earthquake [Strength]
UDCon12	EQX	-1.5	
UDCon13	Dead	0.9	Dead (min) + Static Earthquake [Strength]
UDCon13	EQY	1.5	
UDCon14	Dead	0.9	Dead (min) - Static Earthquake [Strength]
UDCon14	EQY	-1.5	

8. SEISMIC ZONE:

The **Proposed Building (STILT + 3 Floors) at Chennai** falls in **Zone III** as per the IS Code. Response spectrum method shall be used for seismic analysis of R.C.C Frames as recommended by **IS: 1893:2016.**

Seismic Zone	=	III
Zone Factor "Z"	=	0.16
Response Reduction Factor "R" (For OMRF)	=	3.00
Importance Factor "I"	=	1.0

9. STATIC ANALYSIS

As per IS 1893:2002, the total design lateral force or seismic design base shear (V_b) is determined by the following expression.

$$\mathbf{V_b = A_h W}$$

Where $A_h$is Design acceleration spectrum value using fundamental natural period and W is seismic weight of the building.

The Seismic weight is estimated based on full dead load + % of live load

(live load up to 3kN/m^2 75 % reduction, above 3 kN/m^2 50% reduction)

The fundamental natural period of vibration (T_a) in seconds of a RC building is estimated by using the following formula

$\mathbf{T_a = 0.075\ h^{0.75}}$(moment resisting building without brick infill panel)

$\mathbf{T_a = 0.09\ h/\sqrt{d}}$(moment resisting building with brick infill panel)

Where h is the height of building in meters.

"d" is the base dimension of the building at the plinth level in m.

From is 1893:2016 Clause 6.4.2, Design horizontal seismic coefficient

$$A_h = \frac{Z\ I\ Sa}{2\ R\ g}$$

Where, Z for Zone III	=	0.1
I for building	=	1.0
R for MRF	=	3.0

10. COVER FOR REINFORCEMENT:

Cover for Reinforcement is considered as per Table 16A of NBC, for a Fire resistance of 2hrs.

1. Foundation

   Isolated /combined foundation / Pile cap

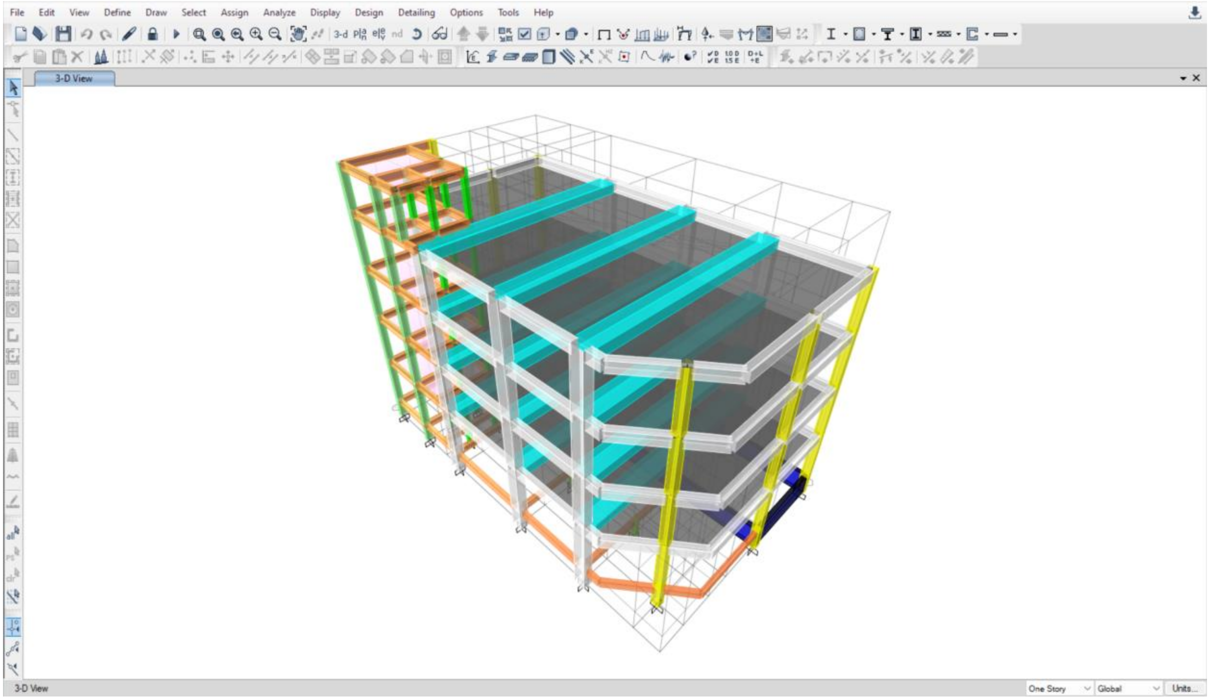
   Foundation bottom = 50mm

   Foundation top & sides = 40mm

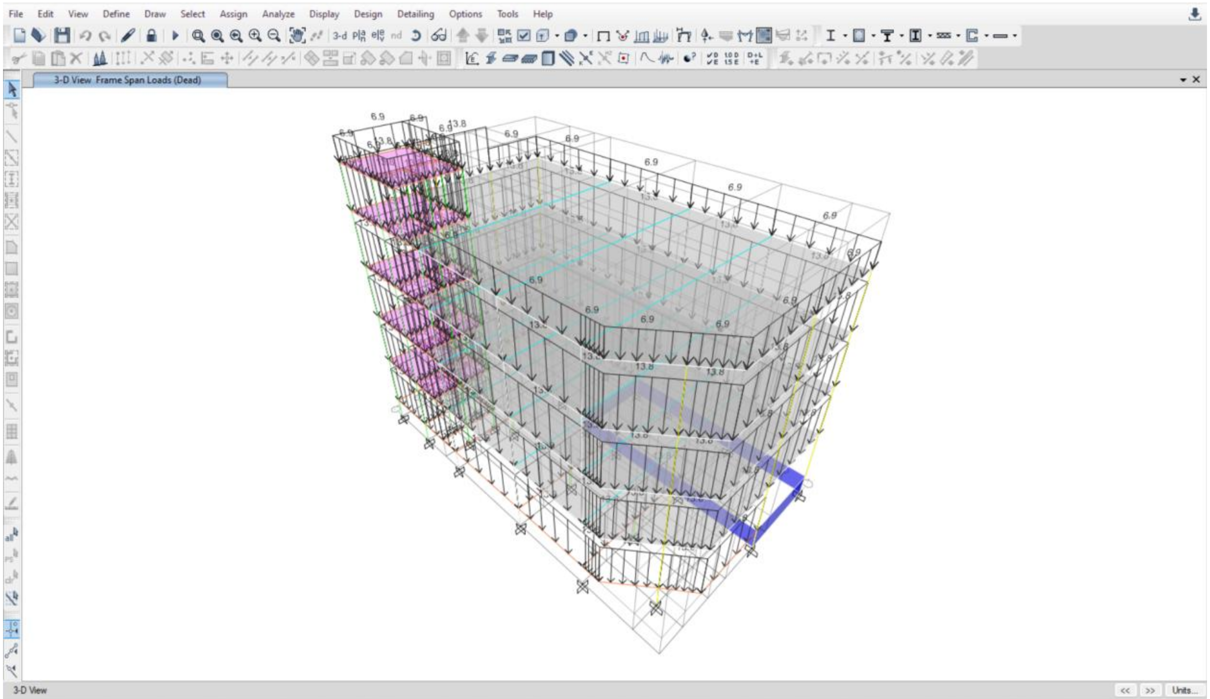
   Plinth beam = 30mm
2. Columns = 40mm
3. Beams = 30mm
4. Slabs = 20mm

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 98404 48613

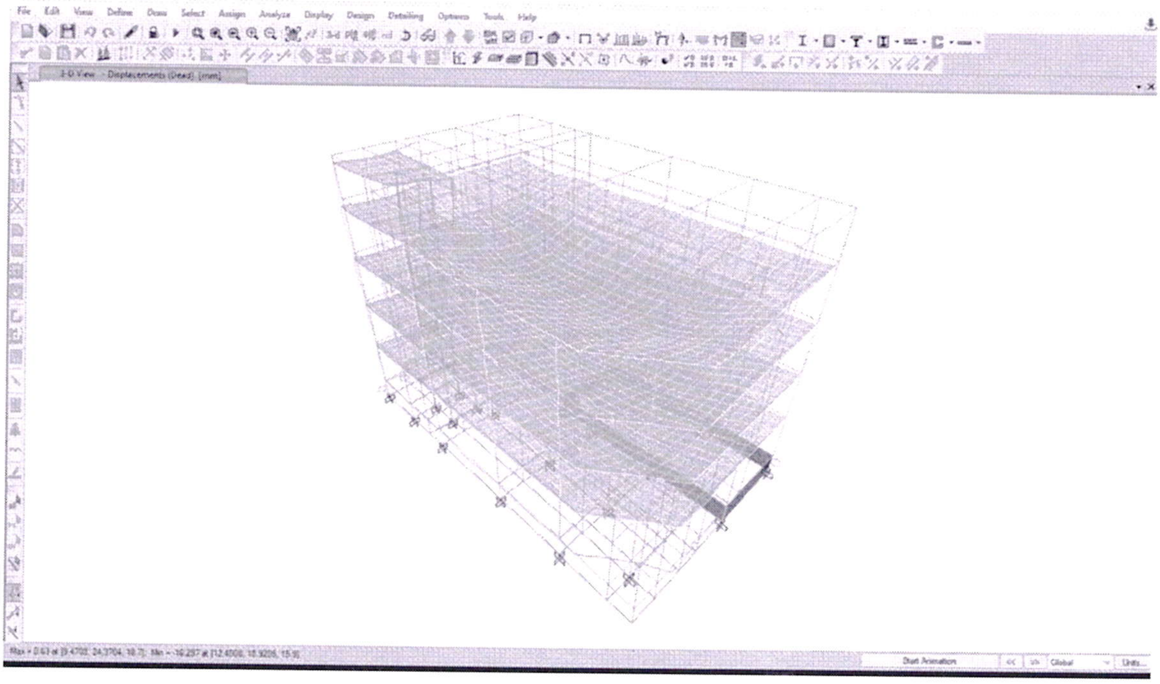
DESIGN OF STRUCTURE



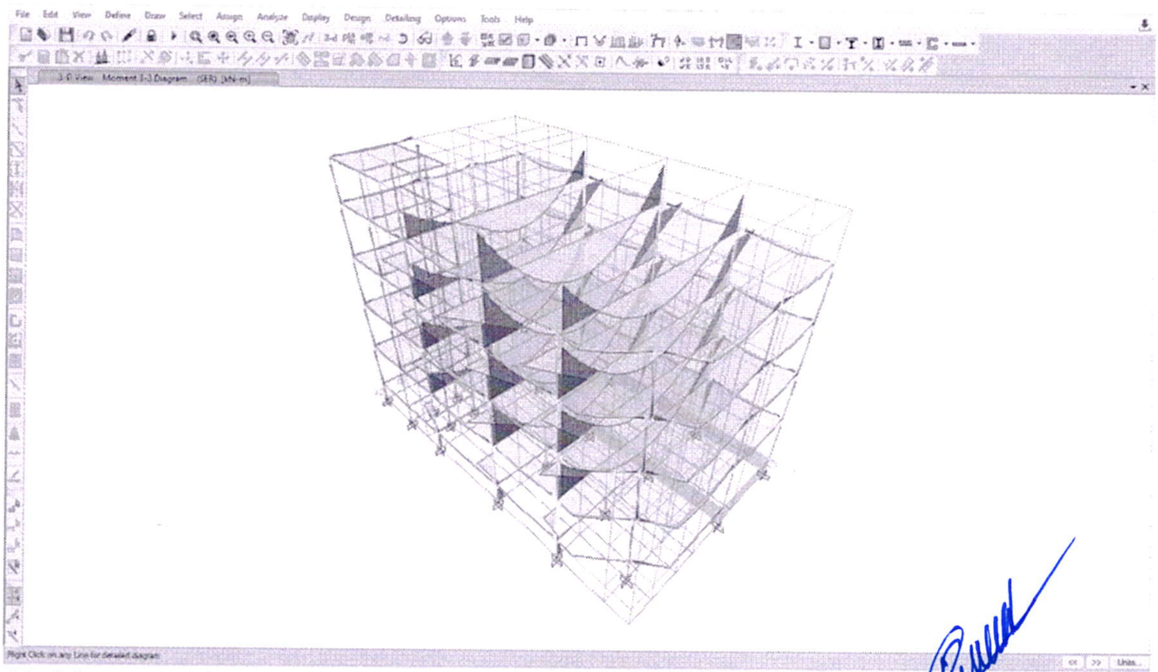
3d view – rendered image



Wall load – dead load



Model displacement



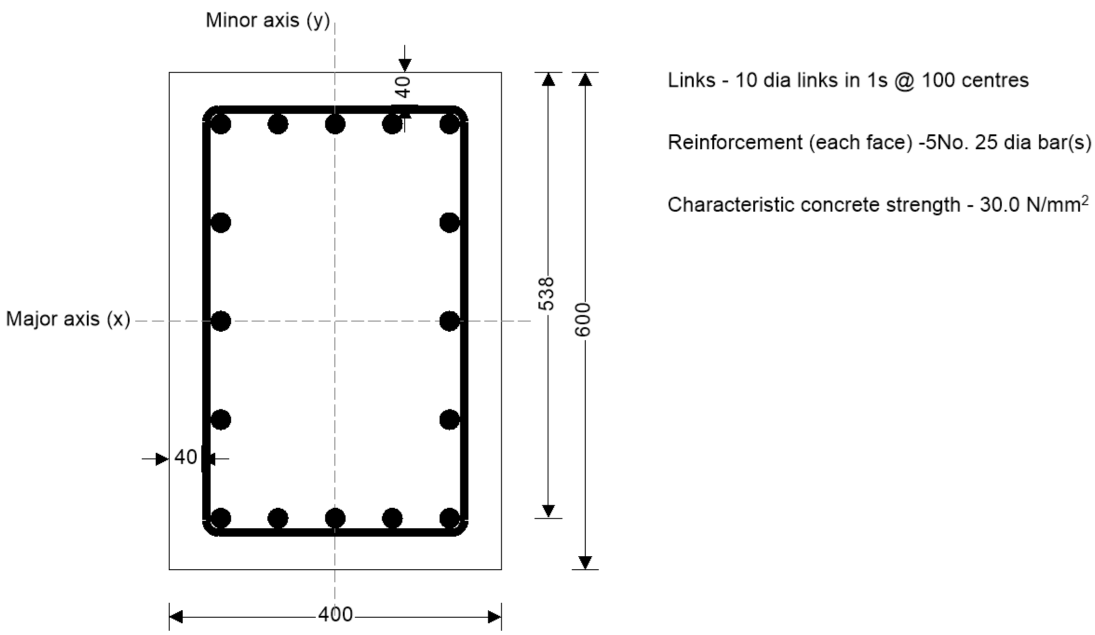
Model moment check

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 9840448613

RC COLUMN DESIGN

Column definition

Column depth (larger column dim);	h = **600** mm
Nominal cover to all reinforcement (longer dim);	c_h = **40** mm
Depth to tension steel;	$h' = h - c_h – L_{dia} – D_{col}/2$ = **538** mm
Column width (smaller column dim);	b = **400** mm
Nominal cover to all reinforcement (shorter dim);	c_b = **40** mm
Depth to tension steel;	$b' = b - c_b - L_{dia} – D_{col}/2$ = **338** mm
Characteristic strength of reinforcement;	f_y = **500** N/mm^2
Characteristic strength of concrete;	f_{cu} = **30** N/mm^2



Unbraced Column Design

Check on overall column dimensions

Column OK - h < 4b

Unbraced column slenderness check

Unbraced column clear height;	l_o = **3500** mm
Slenderness limit;	$l_{limit} = 60 \times b$ = **24000** mm
Slenderness limit;	$l_{limit1} = 100 \times b^2 / h$ = **26667** mm

Column slenderness limit OK

If column is unrestrained, slenderness limit OK

Short column check for unbraced columns

Column clear height;	l_o = **3500** mm
Effect height factor for unbraced columns - maj axis;	β_x = **1.20**
Effective height (major axis);	$l_{ex} = \beta_x \times l_o$ = **4.200** m
Slenderness check;	l_{ex}/h = **7.00**

The unbraced column is short (major axis)

Effect height factor for unbraced columns - min axis; β_y = **1.20**

Effective height (minor axis); $l_{ey} = \beta_y \times l_o$ = **4.200** m

Slenderness check; l_{ey}/b = **10.50**

The unbraced column is slender (minor axis)

Unbraced slender column (minor axis) - bi-axial bending

Define column reinforcement

Main reinforcement in column

Assumed diameter of main reinforcement; D_{col} = **25** mm

Assumed no. of bars in one face (assumed sym); L_{ncol} = **5**

Area of "tension" steel; $A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4$ = **2454** mm^2

Area of compression steel; $A_{sc} = A_{st}$ = **2454** mm^2

Total area of steel ; $A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2))$ = **7854.0** mm^2

Percentage of steel; $A_{scol} / (b \times h)$ = **3.3** %

Design ultimate loading

Design ultimate axial load; N = **2000** kN

Initial end moment (major axis); M_{x1} = **180** kNm

Initial end moment (minor axis); M_{y1} = **150** kNm

Initial moment approx (major axis); $M_{ix} = abs(M_{x1})$ = **180.0** kNm

Initial moment approx (minor axis); $M_{iy} = abs(M_{y1})$ = **150.0** kNm

Additional moment

$\beta_{ay} = l_{ey}^2 / (2000 \times b^2)$ = **0.06**

Reduction factor to correct deflection for axial load

$N_{uz} = 0.45 \times f_{cu} \times h \times b + 1/\gamma_{ms} \times f_y \times A_{scol}$ = **6654.8** kN

$N_{bal} = 0.25 \times f_{cu} \times h \times b'$ = **1518.8** kN

$K = min((N_{uz} - N)/(N_{uz} - N_{bal}), 1.0)$ = **0.91**

$a_{uy} = \beta_{ay} \times K \times b$ = **20.0** mm

Additional moment; $M_{addy} = N \times a_{uy}$ = **40.0** kNm

Minimum design moments

Min design moment (Major axis); $M_{xmin} = min(0.05 \times h, 20 \text{ mm}) \times N$ = **40.0** kNm

Min design moment (Minor axis); $M_{ymin} = min(0.05 \times b, 20 \text{ mm}) \times N$ = **40.0** kNm

Design moments

Design moment (major axis); $M_{xdes} = max (M_{ix}, M_{xmin})$ = **180.0** kNm

Design moment (minor axis); $M_{ydes} = max (M_{iy} + M_{addy}, M_{ymin})$ = **190.0** kNm

Simplified method for dealing with bi-axial bending:-

h' = **538** mm; b' = **338** mm

Approx uniaxial design moment

$\beta = 1 - 1.165 \times min(0.6, N/(b \times h \times f_{cu}))$ = **0.68**

Design moment;

$M_{design} = if(M_{xdes}/h' < M_{ydes}/b', M_{ydes} + \beta \times b'/h' \times M_{xdes}, M_{xdes} + \beta \times h'/b' \times M_{ydes})$ = **266.4** kNm

Set up section dimensions for design:-

Section depth; $D = if(M_{xdes}/h' < M_{ydes}/b', b, h)$ = **400.0** mm

Depth to "tension" steel; $d = if(M_{xdes}/h' < M_{ydes}/b', b', h')$ = **337.5** mm

Section width; $B = if(M_{xdes}/h' < M_{ydes}/b', h, b)$ = **600.0** mm

Check of design forces - symmetrically reinforced section

NOTE

Note:- the section dimensions used in the following calculation are:-

Section width (parallel to axis of bending); B = **600** mm

Section depth perpendicular to axis of bending); D = **400** mm

Depth to "tension" steel (symmetrical); d = **338** mm

Compression steel yields (0.9x<D)

Determine correct moment of resistance

N_R =ceiling(if(x_{calc}<D/0.9, N_{R1} , N_{R2}),0.001kN) = **2000.0** kN

M_R = ceiling(if(x_{calc}<D/0.9, M_{R1} , M_{R2}) ,0.001kNm) = **411.6** kNm

Applied axial load; N = **2000.0** kN

Check for moment; M_{design} = **266.4** kNm

Moment check satisfied

Check min and max areas of steel

Total area of concrete; $A_{conc} = b \times h$ = **240000** mm^2

Area of steel (symmetrical); A_{scol} = **7854** mm^2

Minimum percentage of compression reinforcement; k_c = **0.80** %

Minimum steel area; $A_{sc\_min} = k_c \times A_{conc}$ = **1920** mm^2

Maximum steel area; A_{s_max} = 6 % $\times A_{conc}$ = **14400** mm^2

Area of compression steel provided OK

Major axis Shear Resistance of Concrete Columns

Column width; b = **400** mm

Column depth; h = **600** mm

Effective depth to steel; h' = **538** mm

Area of concrete; $A_{conc} = b \times h$ = **240000** mm^2

Design ultimate shear force (major axis); V_x = **75** kN

Characteristic strength of concrete; f_{cu} = **30** N/mm^2

Is a check required? (3.8.4.6)

Axial load; N = **2000.0** kN

Major axis moment; M_x = **266.4** kNm

Eccentricity; $e = M_x / N$ = **133.2** mm

Limit to eccentricity; $e_{limit} = 0.6 \times h$ = **360.0** mm

Actual shear stress; $v_x = V_x / (b \times h')$ = **0.3** N/mm^2

Allowable stress; $v_{allowable}$ = min ((0.8 N$^{1/2}$/mm) $\times \sqrt{(f_{cu})}$, 5 N/mm^2) = **4.382** N/mm^2

No shear check required

Define Containment Links Provided

Link spacing; s_v = **100** mm; Link diameter; L_{dia} = **10** mm; No of links in each group; L_n = **1**

Minimum Containment Steel

Shear steel

Link spacing; s_v = **100** mm

Link diameter; L_{dia} = **10** mm

Column steel

Diameter; D_{col} = **25** mm

Min diameter; L_{limit} = max((6 mm), D_{col}/4) = **6.3** mm

Link diameter OK

Max spacing; $s_{limit} = 12 \times D_{col}$ = **300.0** mm

Link spacing OK

Crack Control in columns

Column design ultimate axial load; N = **2000.0** kN

Column dimensions

Column depth (larger column dimension); h = **600** mm

Column width (smaller column dimension); b = **400** mm

Column area; $A_c = h \times b$ = **240000** mm^2

Limit for crack check; $N_{limit} = 0.2 \times f_{cu} \times A_c$ = **1440.0** kN

No crack checks required

Serviceability Limit State - Cracking in Columns

Bent about the major axis

The following calculations ignore the presence of compression steel and axial load.

Design serviceability moment about the major axis; M_{X_SLS} = **50** kNm

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension); h = **600** mm

Depth to steel; h' = **538** mm

Column width (smaller column dimension); b = **400** mm

Characteristic strength of concrete; f_{cu} = **30** N/mm^2

Characteristic strength of reinforcement; f_y = **500** N/mm^2

Diameter of links; L_{dia} = **10** mm

Diameter of tension reinforcement; D_{col} = **25** mm

Number of tension reinforcement bars; L_{ncol} = **5**

Area of tension reinforcement; $A_{st} = \pi \times D_{col}^2 /4 \times L_{ncol}$ = **2454** mm^2

Nominal cover to reinforcement; $c_{nom} = h - h' - D_{col}/2 - L_{dia}$ = **40** mm

Cover to tension reinforcement; $c_{ten} = c_{nom} + L_{dia}$ = **50.0** mm

Effective depth to tension reinforcement; h' = **537.5** mm

Tension bar centres; $bar_{crs} = (b - 2\times(c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1)$ = **68.8** mm

Modular Ratio

Modulus of elasticity for reinforcement; E_s = 200 kN/mm^2

Modulus of elast for conc (half the instanteneous); E_c = ((20 kN/mm^2) + 200×f_{cu}) / 2 = **13** kN/mm^2

Modular ratio; $m = E_s / E_c$ = **15.385**

Neutral Axis position

For equilibrium; F_{st} equates F_c

Therefore: $m \times A_{st} \times [f_c\times(h'-x)/x]$ equates to $0.5 \times f_c \times b \times x$

Solving for x gives the position of the neutral axis in the section:-

$$x = h' \times [-1\times E_s\times A_{st}/(E_c\times b\times h') + \sqrt{(E_s\times A_{st}/(E_c\times b\times h') \times (2+E_s\times A_{st}/(E_c\times b\times h')))}] = \mathbf{237.9}\text{ mm}$$

Depth of concrete in compression; x = **237.9** mm

Concrete and Steel stresses

The serviceability limit state moment; M_{X_SLS} = **50** kNm

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is $0.5 \times f_c \times b \times x \times (h' - x/3)$

Solving for concrete stress f_c gives; $f_c = 2 \times M_{X\_SLS} / (b \times x \times (h' - x/3))$ = **2.29** N/mm^2

Allowable stress; $0.45 \times f_{cu}$ = **13.50** N/mm^2

Concrete stress OK

Taking moments about the centre of action of the concrete force:-

Moment of resistance of steel is $f_{st} \times A_s \times (h' - x/3)$

Solving for steel stress f_{st} gives; $f_{st} = M_{X\_SLS} / (A_{st} \times (h' - x/3))$ = **44.46** N/mm^2

Concrete and Steel strains

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s$ = **222.3×10^{-6}**

Allowable steel strain; $0.8 \times f_y / E_s$ = **2.000×10^{-3}**

Steel strain OK

Strain in the concrete at the level at which crack width is required

Level of crack; $a' = h$ = **600** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x)$ = **268.7×10^{-6}**

Strain in the concrete at the level at which crack width is required adjusted for stiffening of the concrete tension zone

Allowable crack width; $Crack_{Allowable}$ = **0.2** mm

Factor for stiffening based on limitiing crack width; factor = **1.0** N/mm^2

Breadth of tension face; $b_t = b$ = **400** mm

$\varepsilon_m = \min(\varepsilon_1 ,\max(0, \varepsilon_1 - [factor \times b_t \times (h - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (h' - x))]))$ = **149.8×10^{-6}**

Distance from tension bar to crack in tension face between tension bars

$a_{cr1} = \sqrt{ (bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2} - D_{col}/2$ = **58.8** mm

Distance from tension bar to crack in tension face at corner of column

$a_{cr2} = \sqrt{2} \times (c_{nom} + L_{dia} + D_{col}/2) - D_{col}/2$ = **75.9** mm

Critical distance from tension bar; $a_{cr} = \max(a_{cr1}, a_{cr2})$ = **75.9** mm

Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2\times(a_{cr} - c_{ten})/(h - x))$ = **0.030** mm

Max allowable crack width; $Crack_{Allowable}$ = **0.20** mm

Serviceability Limit State - Cracking in Columns

Bent about the minor axis

The following calculations ignore the presence of compression steel and axial load.

Design serviceability moment about the minor axis; M_{Y_SLS} = **50** kNm

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension); h = **600** mm

Column width (smaller column dimension); b = **400** mm

Depth to steel; b' = **338** mm

Characteristic strength of concrete; f_{cu} = **30** N/mm^2

Characteristic strength of reinforcement; f_y = **500** N/mm^2

Diameter of links; L_{dia} = **10** mm

Diameter of tension reinforcement; D_{col} = **25** mm

Number of tension reinforcement bars; L_{ncol} = **5**

Area of tension reinforcement; $A_{st} = \pi \times D_{col}^2 /4 \times L_{ncol}$ = **2454** mm^2
Nominal cover to reinforcement; $c_{nom} = b - b' - D_{col}/2 - L_{dia}$ = **40** mm
Cover to tension reinforcement; $c_{ten} = c_{nom} + L_{dia}$ = **50.0** mm
Effective depth to tension reinforcement; b' = **337.5** mm
Tension bar centres; $bar_{crs} = (h - 2\times(c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1)$ = **118.7** mm

Modular Ratio

Modulus of elasticity for reinforcement; E_s = 200 kN/mm^2
Modulus of elasticity for conc (half instanteneous); $E_c = ((20\ kN/mm^2) + 200\times f_{cu}) / 2$ = **13** kN/mm^2
Modular ratio; $m = E_s / E_c$ = **15.385**

Neutral Axis position

For equilibrium; F_{st} equates F_c
Therefore: $m \times A_{st} \times [f_c\times(b'-x)/x]$ equates to $0.5 \times f_c \times h \times x$
Solving for x gives the position of the neutral axis in the section:-

$x = b' \times [-1\times E_s\times A_{st}/(E_c\times h\times b') + \sqrt{(E_s\times A_{st}/(E_c\times h\times b') \times (2+E_s\times A_{st}/(E_c\times h\times b')))}]$ = **152.6** mm

Depth of concrete in compression; x = **152.6** mm

Concrete and Steel stresses

The serviceability limit state moment; M_{Y_SLS} = **50** kNm
Taking moments about the centreline of the reinforcement:-
Moment of resistance of concrete is $0.5 \times f_c \times h \times x \times (b' - x/3)$
Solving for concrete stress f_c gives; $f_c = 2 \times M_{Y\_SLS} / (h \times x \times (b' - x/3))$ = **3.81** N/mm^2
Allowable stress; $0.45 \times f_{cu}$ = **13.50** N/mm^2

Concrete stress OK

Taking moments about the centre of action of the concrete force:-
Moment of resistance of steel; $f_{st} \times A_s \times (b' - x/3)$
Solving for steel stress f_{st} gives; $f_{st} = M_{Y\_SLS} / (A_{st} \times (b' - x/3))$ = **71.07** N/mm^2

Concrete and Steel strains

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s$ = **355.4×10^{-6}**
Allowable steel strain; $0.8 \times f_y / E_s$ = **2.000×10^{-3}**

Steel strain OK

Strain in the concrete at the level at which crack width is required
Level of crack; a' = b = **400** mm
$\varepsilon_1 = \varepsilon_s \times (a' - x)/(b' - x)$ = **475.4×10^{-6}**
Strain in the concrete at the level at which crack width is required adjusted for stiffening of the concrete tension zone
Allowable crack width; $Crack_{Allowable}$ = **0.2** mm
Factor for stiffening based on limitiing crack width; factor = **1.0** N/mm^2
Breadth of tension face; h_t = h = **600** mm

$\varepsilon_m = \min(\varepsilon_1 , \max(0, \varepsilon_1 - [factor \times h_t\times (b - x) \times (a' - x) / (3\times E_s\times A_{st} \times(b' - x))]))$ = **340.6×10^{-6}**

Distance from tension bar to crack in tension face between tension bars
$a_{cr1} = \sqrt{((bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2$ = **73.7** mm
Distance from tension bar to crack in tension face at corner of column
$a_{cr2} = \sqrt{(2)} \times (c_{nom} + L_{dia} + D_{col}/2) - D_{col}/2$ = **75.9** mm
Critical distance from tension bar; $a_{cr} = \max(a_{cr1}, a_{cr2})$ = **75.9** mm

Design crack width;

mm

Max allowable crack width;

$\text{Crack}_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2\times(a_{cr} - c_{ten})/(b - x))$ = **0.064**

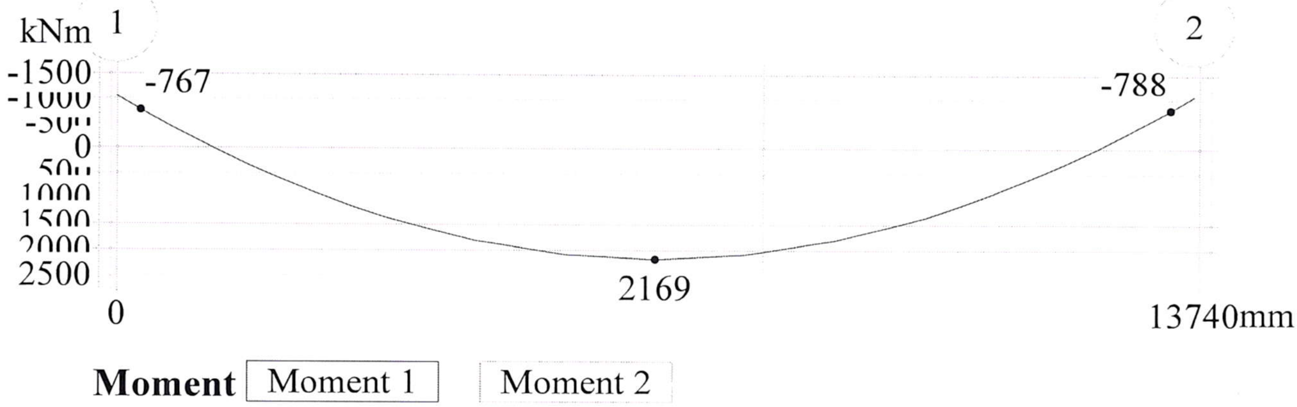
$\text{Crack}_{Allowable}$ = **0.20** mm

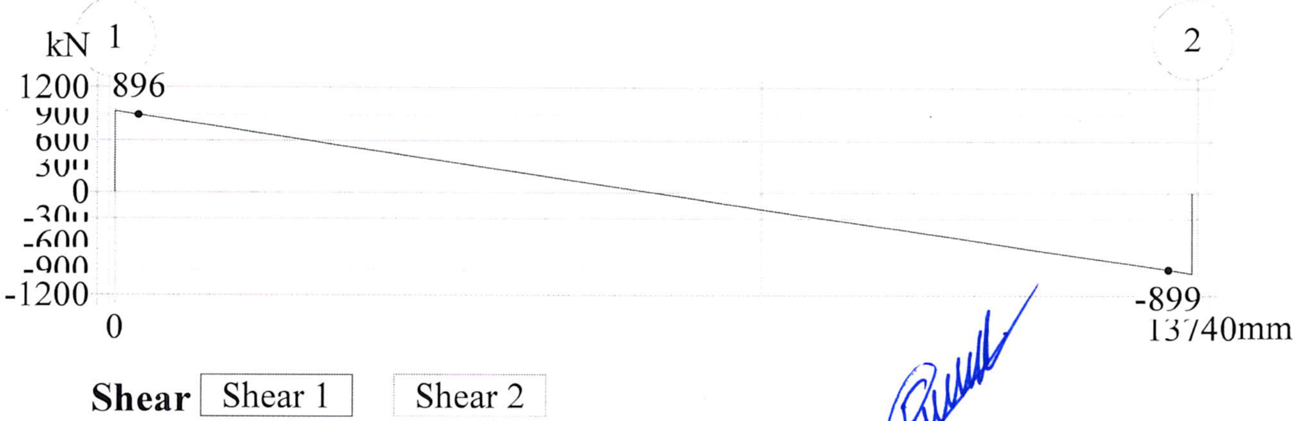
ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 98 40 44 86 13

1PTB-1 to 1PTB-3(1200X600mm):

Load Combination:1.5DL+1.5SDL+1.5LL

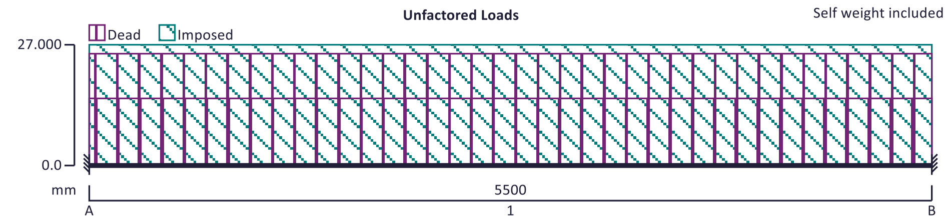
Bending Moments
Load Combinations
Ultimate Flexure

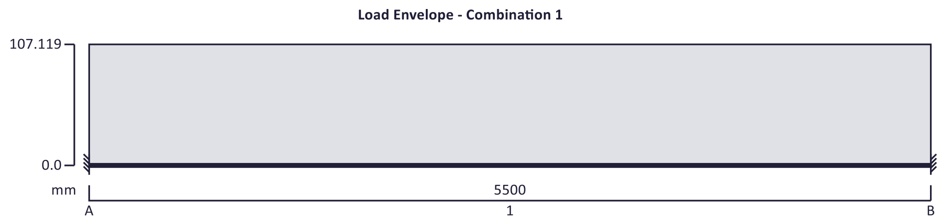


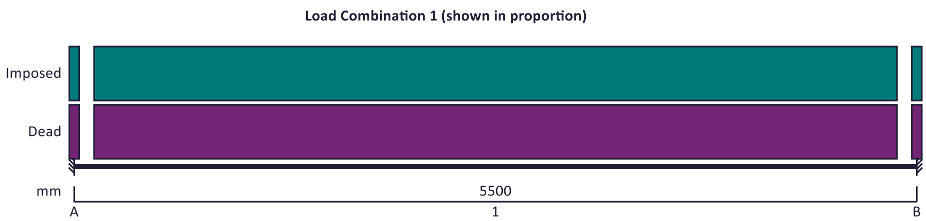


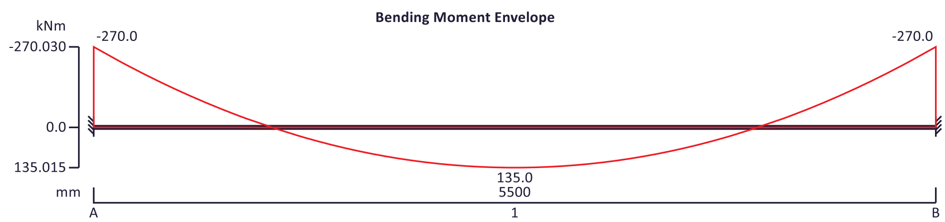
ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 98 40 44 86 13

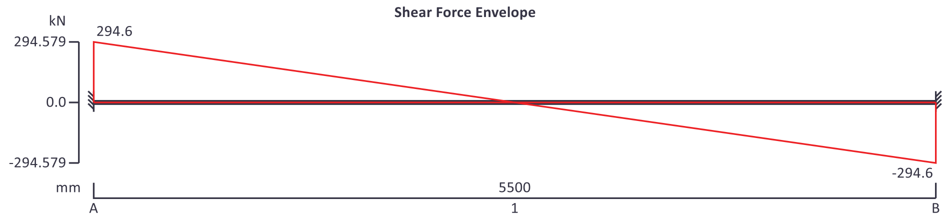
RC BEAM ANALYSIS & DESIGN











Support conditions

Support A	Vertically restrained
	Rotationally restrained
Support B	Vertically restrained
	Rotationally restrained

Applied loading

Imposed full UDL 27 kN/m
Dead full UDL 25 kN/m
Dead full UDL 15 kN/m
Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50
		Imposed × 1.50

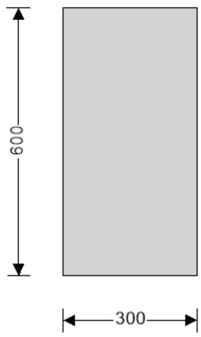
Analysis results

Maximum moment support A;	M_{A_max} = **-270** kNm; 5%	M_{A_red} = **-257** kNm;
Maximum moment span 1 at 2750 mm;	M_{s1_max} = **135** kNm; 10%	M_{s1_red} = **149** kNm;
Maximum moment support B;	M_{B_max} = **-270** kNm; 5%	M_{B_red} = **-257** kNm;
Maximum shear support A;	V_{A_max} = **295** kN;	V_{A_red} = **295** kN
Maximum shear support A span 1 at 542 mm;	$V_{A\_s1\_max}$ = **236** kN;	$V_{A\_s1\_red}$ = **236** kN
Maximum shear support B;	V_{B_max} = **-295** kN;	V_{B_red} = **-295** kN
Maximum shear support B span 1 at 4958 mm;	$V_{B\_s1\_max}$ = **-236** kN;	$V_{B\_s1\_red}$ = **-236** kN
Maximum reaction at support A;	R_A = **295** kN	
Unfactored dead load reaction at support A;	R_{A_Dead} = **122** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **74** kN	
Maximum reaction at support B;	R_B = **295** kN	
Unfactored dead load reaction at support B;	R_{B_Dead} = **122** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **74** kN	

Rectangular section details

Section width; $b = \mathbf{300}$ mm

Section depth; $h = \mathbf{600}$ mm



Concrete details

Concrete strength class; **C20/25**

Characteristic compressive cube strength; $f_{cu} = \mathbf{25}$ N/mm^2

Modulus of elasticity of concrete; $E_c = 20\text{kN/mm}^2 + 200 \times f_{cu} = \mathbf{25000}$ N/mm^2

Maximum aggregate size; $h_{agg} = \mathbf{20}$ mm

Reinforcement details

Characteristic yield strength of reinforcement; $f_y = \mathbf{500}$ N/mm^2

Characteristic yield strength of shear reinforcement; $f_{yv} = \mathbf{500}$ N/mm^2

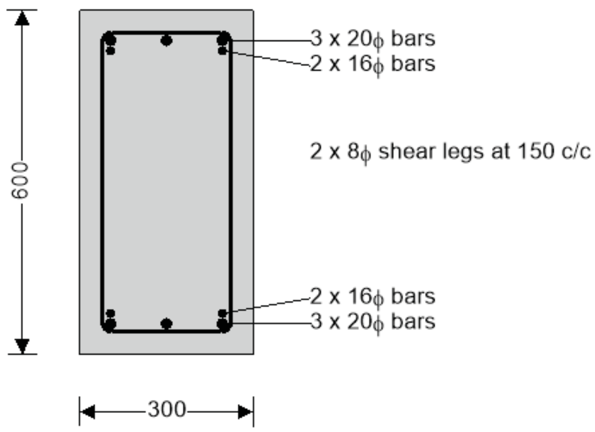
Nominal cover to reinforcement

Nominal cover to top reinforcement; $c_{nom\_t} = \mathbf{35}$ mm

Nominal cover to bottom reinforcement; $c_{nom\_b} = \mathbf{35}$ mm

Nominal cover to side reinforcement; $c_{nom\_s} = \mathbf{35}$ mm

Support A



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1; $3 \times 20\phi$ bars

Area of reinforcement provided - layer 1; A_{s_L1} = **942** mm^2

Depth to layer 1; d_{L1} = **53** mm

Reinforcement provided - layer 2; $2 \times 16\phi$ bars

Area of reinforcement provided - layer 2; A_{s_L2} = **402** mm^2

Depth to layer 2; d_{L2} = **71** mm

Total area of reinforcement; $A_{s2,prov} = A_{s\_L1} + A_{s\_L2}$ = **1345** mm^2

Centroid of reinforcement; $d_{bot} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s2,prov}$ = **58** mm

Multiple layers of top reinforcement

Reinforcement provided - layer 1; $3 \times 20\phi$ bars

Area of reinforcement provided - layer 1; A_{s_L1} = **942** mm^2

Depth to layer 1; d_{L1} = **547** mm

Reinforcement provided - layer 2; $2 \times 16\phi$ bars

Area of reinforcement provided - layer 2; A_{s_L2} = **402** mm^2

Depth to layer 2; d_{L2} = **529** mm

Total area of reinforcement; $A_{s,prov} = A_{s\_L1} + A_{s\_L2}$ = **1345** mm^2

Centroid of reinforcement; $d_{top} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s,prov}$ = **542** mm

Rectangular section in flexure

Design bending moment; $M = abs(M_{A\_red})$ = **257** kNm

Depth to tension reinforcement; $d = d_{top}$ = **542** mm

Redistribution ratio; $\beta_b = min(1 - m_{rA}, 1)$ = **0.950**

$K = M / (b \times d^2 \times f_{cu})$ = **0.117**

K' = 0.156

K' > K - No compression reinforcement is required

Lever arm; $z = min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **459**

mm

Depth of neutral axis; $x = (d - z) / 0.45$ = **184** mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z)$ = **1285** mm^2

Tension reinforcement provided; 3×20 ϕbars + 2×16 ϕbars

Area of tension reinforcement provided; $A_{s,prov}$ = **1345** mm^2

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h$ = **234** mm^2

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h$ = **7200** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear

Design shear force span 1 at 542 mm; $V = max(V_{A\_s1\_max}, V_{A\_s1\_red})$ = **236** kN

Design shear stress; $v = V / (b \times d)$ = **1.450** N/mm^2

Design concrete shear stress; $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **0.593** N/mm^2

Allowable design shear stress; $v_{max} = \min(0.8 N/mm^2 \times (f_{cu}/1 N/mm^2)^{0.5}, 5 N/mm^2)$ = **4.000** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v; $(v_c + 0.4 N/mm^2) < v < v_{max}$

Design shear resistance required; $v_s = \max(v - v_c, 0.4 N/mm^2)$ = **0.857** N/mm^2

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **591** mm^2/m

Shear reinforcement provided; $2 \times 8\phi$ legs at 150 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **670** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d$ = **406** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Spacing of reinforcement

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top,L1}/2)) /(N_{top,L1} - 1) - \phi_{top,L1}$ = **77** mm

Minimum distance between bars in tension

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5$ mm = **25** mm

PASS - Satisfies the minimum spacing criteria

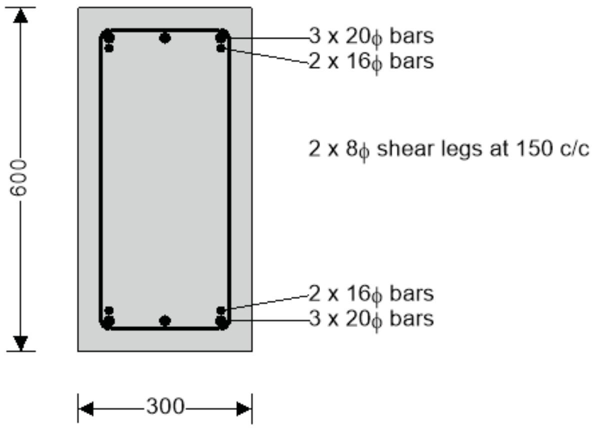
Maximum distance between bars in tension

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **335.4** N/mm^2

Maximum distance between bars in tension; $s_{max} = \min(47000 N/mm / f_s, 300 mm)$ = **140** mm

PASS - Satisfies the maximum spacing criteria

Mid span 1



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1;	3 × 20ϕ bars
Area of reinforcement provided - layer 1;	A_{s_L1} = **942** mm^2
Depth to layer 1;	d_{L1} = **547** mm
Reinforcement provided - layer 2;	2 × 16ϕ bars
Area of reinforcement provided - layer 2;	A_{s_L2} = **402** mm^2
Depth to layer 2;	d_{L2} = **529** mm
Total area of reinforcement;	$A_{s,prov} = A_{s\_L1} + A_{s\_L2}$ = **1345** mm^2
Centroid of reinforcement;	$d_{bot} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s,prov}$ = **542** mm

Multiple layers of top reinforcement

Reinforcement provided - layer 1;	3 × 20ϕ bars
Area of reinforcement provided - layer 1;	A_{s_L1} = **942** mm^2
Depth to layer 1;	d_{L1} = **53** mm
Reinforcement provided - layer 2;	2 × 16ϕ bars
Area of reinforcement provided - layer 2;	A_{s_L2} = **402** mm^2
Depth to layer 2;	d_{L2} = **71** mm
Total area of reinforcement;	$A_{s2,prov} = A_{s\_L1} + A_{s\_L2}$ = **1345** mm^2
Centroid of reinforcement;	$d_{top} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s2,prov}$ = **58** mm

Design moment resistance of rectangular section - Positive moment

Design bending moment;	$M = abs(M_{s1\_red})$ = **149** kNm
Depth to tension reinforcement;	$d = d_{bot}$ = **542** mm
Redistribution ratio;	$\beta_b = \min(1 - m_{rs1}, 1)$ = **0.900**
	$K = M / (b \times d^2 \times f_{cu})$ = **0.068**
	$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm;	$z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **497**
mm	
Depth of neutral axis;	$x = (d - z) / 0.45$ = **98** mm
Area of tension reinforcement required;	$A_{s,req} = M / (0.87 \times f_y \times z)$ = **686** mm^2
Tension reinforcement provided;	3 × 20 ϕbars + 2 × 16 ϕbars
Area of tension reinforcement provided;	$A_{s,prov}$ = **1345** mm^2
Minimum area of reinforcement;	$A_{s,min} = 0.0013 \times b \times h$ = **234** mm^2
Maximum area of reinforcement;	$A_{s,max} = 0.04 \times b \times h$ = **7200** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear

Shear reinforcement provided;	2 × 8ϕ legs at 150 c/c
Area of shear reinforcement provided;	$A_{sv,prov}$ = **670** mm^2/m

Minimum area of shear reinforcement; $A_{sv,min} = 0.4N/mm^2 \times b / (0.87 \times f_{yv})$ = **276** mm²/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d$ = **406** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design concrete shear stress; $v_c = 0.79N/mm^2 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400mm /d)^{1/4}) \times (\min(f_{cu}, 40N/mm^2) / 25N/mm^2)^{1/3} / \gamma_m$ = **0.593** N/mm²

Design shear resistance provided; $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b$ = **0.972** N/mm²

Design shear stress provided; $v_{prov} = v_{s,prov} + v_c$ = **1.565** N/mm²

Design shear resistance; $V_{prov} = v_{prov} \times (b \times d)$ = **254.3** kN

Shear links provided valid between 400 mm and 5100 mm with tension reinforcement of 1345 mm²

Spacing of reinforcement

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot,L1}/2)) /(N_{bot,L1} - 1) - \phi_{bot,L1}$ = **77** mm

Minimum distance between bars in tension

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5$ mm = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **189.1** N/mm²

Maximum distance between bars in tension; $s_{max} = \min(47000 N/mm / f_s, 300 mm)$ = **249** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio

Basic span to depth ratio; $span\_to\_depth_{basic}$ = **20.0**

Design service stress in tension reinforcement; $f_s = (2 \times f_y \times A_{s,req})/ (3 \times A_{s,prov} \times \beta_b)$ = **189.1** N/mm²

Modification for tension reinforcement

$f_{tens} = \min(2.0, 0.55 + (477N/mm^2 - f_s) / (120 \times (0.9N/mm^2 + (M / (b \times d^2)))))$ = **1.477**

Modification for compression reinforcement

$f_{comp} = \min(1.5, 1 + (100 \times A_{s2,prov} / (b \times d)) / (3 + (100 \times A_{s2,prov} / (b \times d))))$ = **1.216**

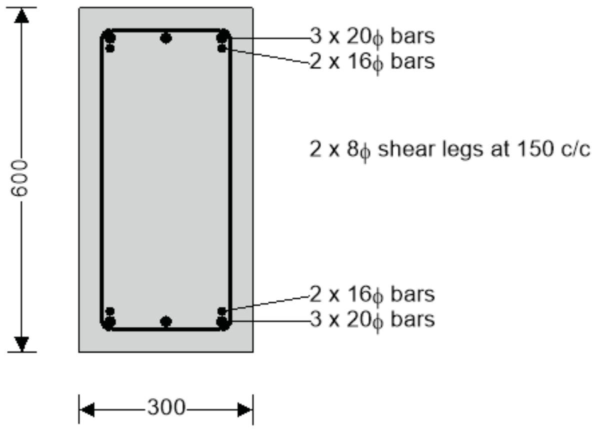
Modification for span length; f_{long} = 1.000

Allowable span to depth ratio; $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp}$ = **35.9**

Actual span to depth ratio; $span\_to\_depth_{actual} = L_{s1} / d$ = **10.2**

PASS - Actual span to depth ratio is within the allowable limit

Support B



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1;	$3 \times 20\phi$ bars
Area of reinforcement provided - layer 1;	A_{s_L1} = **942** mm^2
Depth to layer 1;	d_{L1} = **53** mm
Reinforcement provided - layer 2;	$2 \times 16\phi$ bars
Area of reinforcement provided - layer 2;	A_{s_L2} = **402** mm^2
Depth to layer 2;	d_{L2} = **71** mm
Total area of reinforcement;	$A_{s2,prov} = A_{s\_L1} + A_{s\_L2}$ = **1345** mm^2
Centroid of reinforcement;	$d_{bot} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s2,prov}$ = **58** mm

Multiple layers of top reinforcement

Reinforcement provided - layer 1;	$3 \times 20\phi$ bars
Area of reinforcement provided - layer 1;	A_{s_L1} = **942** mm^2
Depth to layer 1;	d_{L1} = **547** mm
Reinforcement provided - layer 2;	$2 \times 16\phi$ bars
Area of reinforcement provided - layer 2;	A_{s_L2} = **402** mm^2
Depth to layer 2;	d_{L2} = **529** mm
Total area of reinforcement;	$A_{s,prov} = A_{s\_L1} + A_{s\_L2}$ = **1345** mm^2
Centroid of reinforcement;	$d_{top} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s,prov}$ = **542** mm

Rectangular section in flexure

Design bending moment;	$M = abs(M_{B\_red})$ = **257** kNm
Depth to tension reinforcement;	$d = d_{top}$ = **542** mm
Redistribution ratio;	$\beta_b = min(1 - m_{rB}, 1)$ = **0.950**
	$K = M / (b \times d^2 \times f_{cu})$ = **0.117**
	K' = 0.156

K' > K - No compression reinforcement is required

Lever arm;	$z = min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **459**
mm	
Depth of neutral axis;	$x = (d - z) / 0.45$ = **184** mm
Area of tension reinforcement required;	$A_{s,req} = M / (0.87 \times f_y \times z)$ = **1285** mm^2
Tension reinforcement provided;	3×20 ϕbars + 2×16 ϕbars
Area of tension reinforcement provided;	$A_{s,prov}$ = **1345** mm^2
Minimum area of reinforcement;	$A_{s,min} = 0.0013 \times b \times h$ = **234** mm^2
Maximum area of reinforcement;	$A_{s,max} = 0.04 \times b \times h$ = **7200** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear

Design shear force span 1 at 4958 mm;	$V = abs(min(V_{B\_s1\_max}, V_{B\_s1\_red}))$ = **236** kN
Design shear stress;	$v = V / (b \times d)$ = **1.450** N/mm^2

Design concrete shear stress; $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$v_c =$ **0.593** N/mm²

Allowable design shear stress; $v_{max} = \min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2) =$ **4.000** N/mm²

PASS - Design shear stress is less than maximum allowable

Value of v; $(v_c + 0.4 \text{ N/mm}^2) < v < v_{max}$

Design shear resistance required; $v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) =$ **0.857** N/mm²

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) =$ **591** mm²/m

Shear reinforcement provided; $2 \times 8\phi$ legs at 150 c/c

Area of shear reinforcement provided; $A_{sv,prov} =$ **670** mm²/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d =$ **406** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Spacing of reinforcement

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top,L1}/2)) /(N_{top,L1} - 1) - \phi_{top,L1} =$ **77** mm

Minimum distance between bars in tension

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5 \text{ mm} =$ **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) =$ **335.4** N/mm²

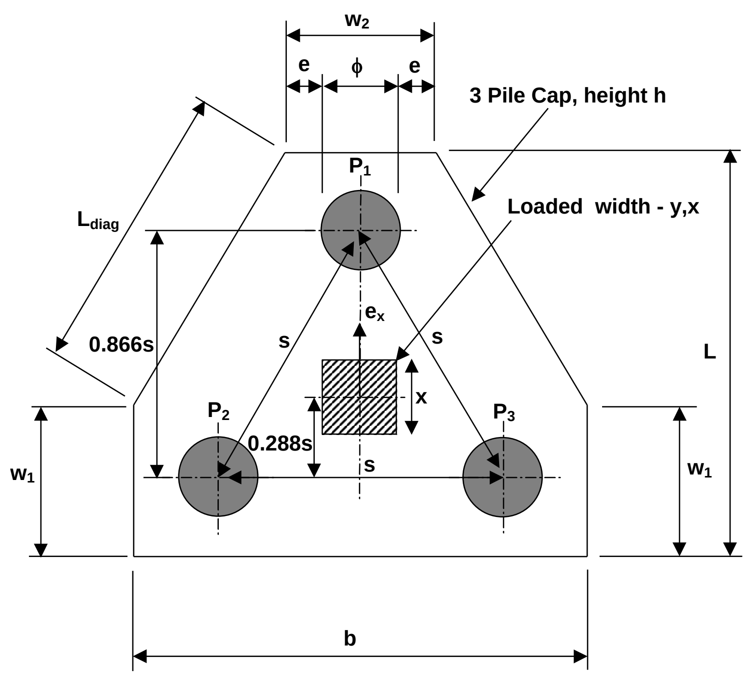
Maximum distance between bars in tension; $s_{max} = \min(47000 \text{ N/mm} / f_s, 300 \text{ mm}) =$ **140** mm

PASS - Satisfies the maximum spacing criteria

;

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No.SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 9840448613

RC PILE CAP DESIGN



Pile Cap Design – Truss Method

Design Input - 3 Piles - No Eccentricity

Number of piles;	N = 3
ULS axial load;	F_{uls} = **1500.0** kN
The ultimate load per pile is;	$F_{uls\_pile} = F_{uls}/3$ = **500.0** kN
Characteristic axial load;	F_{char} = **1000.0** kN
The characteristic load per pile is;	$F_{char\_pile} = F_{char}/3$ = **333.3** kN
Pile diameter;	ϕ = **600** mm
Pile spacing;	s = **1200** mm
Pile cap overhang;	e = **250** mm
Overall length of pile cap;	$L = \sin(60) \times s + \phi + 2 \times e$ = **2139** mm
Overall width of pile cap;	$b = s + \phi + 2 \times e$ = **2300** mm
Width at pile parallel to length, L;	$w_1 = \phi + 2 \times e$ = **1100** mm
Width at pile parallel to overall width, b;	$w_2 = \phi + 2 \times e$ = **1100** mm
Overall height of pile cap;	h = **1000** mm
Diagonal length of sides;	$L_{side\_diag} = \sqrt{((L-w_1)^2 + ((b-w_2)/2)^2)}$ = **1200** mm
Dimensions of loaded area;	x = **300** mm
	y = **300** mm

Cover

Concrete grade;	f_{cu} = **25.0** N/mm^2
Nominal cover;	c_{nom} = **40** mm
Tension bar diameter;	D_t = **16** mm
Link bar diameter;	L_{dia} = **12** mm
Depth to tension steel;	$d = h - c_{nom} - L_{dia} - D_t/2$ = **940** mm

Pile Cap Forces

Compression within pile cap; $F_c = F_{uls}/(3 \times \sin(\theta))$ = **621.1** kN

Tension within pile cap; $F_t = F_c \times \cos(\theta) / (2 \times \cos(30))$ = **212.8** kN

Compression In Pile Cap - Suggested Additional Check

Check compression diagonal as an unreinforced column, using a core equivalent to pile diameter

Compressive force in pile cap; $P_c = 0.4 \times f_{cu} \times \pi \times \phi^2/4$ = **2827.4** kN

PASS Compression

Tension In One Truss Member

Characteristic strength of reinforcement; f_y = **500** N/mm^2

Partial safety factor for strength of steel; γ_{ms} = **1.15**

Req'd area of reinf''t (in one truss member); $A_{s\_req} = F_t /(1/\gamma_{ms} \times f_y)$ =**489** mm^2

Provided area of reinf't (in one truss member); $A_{s\_prov} = A_{st}$ = **2011** mm^2

Tension capacity of truss member; $P_t = (1/\gamma_{ms} \times f_y) \times A_{s\_prov}$ = **874.2** kN

PASS Tension

Max / Min Areas of Reinforcement - Considering A Strip Of Cap

Minimum required area of steel; $A_{st\_min} = k_t \times A_c$ = **650** mm^2

Maximum allowable area of steel; $A_{st\_max} = 4\ \% \times A_c$ = **20000** mm^2

Area of tension steel provided OK

Beam Shear

Check shear stress on the sections at distance ϕ / 5 inside face of piles.

Applied shear stress

Applied shear force; $V = F_{uls} / 3$ = **500.0** kN

Min effective width of pile cap along plane of shear; $b_v = \min(b_{v1} , b_{v2} , 3 \times \phi)$ = **1780** mm

Design shear stress; $v = V / (b_v \times d)$ = **0.30** N/mm^2

Allowable shear stress; $v_{allowable} = \min ((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2)$ = **4.00** N/mm^2

Shear stress - OK

Design concrete shear strength

$v_{c\_25} = 0.79 \times r^{1/3} \times \max(0.67, (400\ mm/d)^{1/4}) \times 1.0\ N/mm^2 / 1.25$ = **0.32** N/mm^2

Shear enhancement; $v_c = v_{c\_25} \times (\min(f_{cu}, 40\ N/mm^2) / 25\ N/mm^2)^{1/3}$ = **0.32** N/mm^2

$a_v = \min(2 \times d, \max((s / \mathrm{sqrt}(3) - \phi / 2 + \phi / 5 - x / 2), 0.1mm))$ = **363** mm

Enhanced shear stress; $v_{c\_enh} = \min(v_{allowable} , 2 \times d \times v_c / a_v)$ = **1.64** N/mm^2

Concrete shear strength - OK, no links reqd.

Note: If no links are provided, the bond strengths for **PLAIN** bars must be used in calculations for anchorage and lap lengths.

Local Shear At Concentrated Loads

Total length of inner perim. at edge of loaded area; $u_0 = 2 \times (x + y)$ =**1200** mm

Assumed average depth to tension steel; $d_{av} = d - D_t$ = **924** mm

Max shear effective across perimeter; $V_p = F_{uls}$ = **1500.0** kN

Stress around loaded area; $v_{max} = V_p / (u_0 \times d_{av})$ = **1.35** N/mm^2

Allowable shear stress; $v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) = \mathbf{4.00}$ N/mm²

Shear stress - OK

Clear Distance Between Bars In Tension

Maximum / Minimum allowable clear distances between tension bars considering a strip of cap

Actual bar spacing;

$spacing_{bars} = \max(0mm, (b_{ccs} - n_{surfaces} \times (c_{adopt} + L_{dia}) - D_t)/(L_{nt} - 1) - D_t) = \mathbf{32}$ mm

Maximum allowable spacing of bars; $spacing_{max} = \min((47000\ N/mm)/f_s, 300\ mm) = \mathbf{300}$ mm

Minimum required spacing of bars; $spacing_{min} = h_{agg} + 5\ mm = \mathbf{25}$ mm

Bar spacing OK

Clear Distance Between Face Of Beam And Tension Bars

Distance to face of beam; $Dist_{edge} = c_{adopt} + L_{dia} + D_t/2 = \mathbf{60}$ mm

Design service stress in reinforcement; $f_s = 2 \times f_y \times As_{\_req} /(3 \times As_{\_prov} \times \beta_b) = \mathbf{81.1}$ N/mm²

Max allowable clear spacing; $Spacing_{max} = \min((47000\ N/mm)/f_s, 300\ mm) = \mathbf{300}$ mm

Max distance to face of beam; $Dist_{max} = Spacing_{max}/2 = \mathbf{150}$ mm

Max distance to beam edge check - OK

Anchorage Of Tension Steel

Anchorage factor; $\phi_{factor} = \mathbf{44}$

Type of lap length; lap_type =**"tens_lap"**

Type of reinforcement; reft_type = **"def2_fy500"**

Minimum radius; $r_{bar} = \mathbf{32}$ mm

Minimum end projection; $P_{bar} = \mathbf{130}$ mm

Minimum anchorage length or lap length req'd; $= \phi_{factor} \times D_t = \mathbf{704}$ mm

Check anchorage length $= 12 \times D_t + d/2 = \mathbf{662}$ mm

Required minimum effective anchorage length; $L_a = = \mathbf{704}$ mm

Check minimum radius required on bend

Note that the bars must extend at least 4D past the bend

Force per bar at bend; $F_{bt} = F_t / L_{nt} = \mathbf{21.3}$ kN

Edge bar centres; $s_{ext} = c_{adopt} + D_t = \mathbf{56}$ mm

Edge maximum allowable bearing stress; $f_{bt\_max\_ext} = 2 \times f_{cu} / (1 + 2\times(D_t / s_{ext})) = \mathbf{31.82}$ N/mm²

Internal bar centres; $s_{int} = spacing_{bars} + D_t = \mathbf{48}$ mm

Internal maximum allowable bearing stress; $f_{bt\_max\_int} = 2 \times f_{cu} / (1 + 2\times(D_t / s_{int})) = \mathbf{30.00}$ N/mm²

Design max allowable bearing stress; $f_{bt\_max} = \min(f_{bt\_max\_ext}, f_{bt\_max\_int}) = \mathbf{30.00}$ N/mm²

Minimum radius required; $r_{min} = \max(r_{bar}, F_{bt} /(f_{bt\_max} \times D_t)) = \mathbf{44.3}$ mm

Minimum radius of bend required = 44 mm

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 9840448613