KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTA

Plot No. 1233, First Floor, Flat No. 1B, Priya Ajay Arcade, 18th Main Road, Anna Nagar west, Chennai – 60

Mob: 98404 48613 eMail Id: kirtiassociates@gmail

STABILITY CERTIFICATE

For,

This is to certify THE PROPOSED CONSTRUCTION OF BASEMENT FLOOR (MECHANICAL AUTOMATED CAR PARKING – 2 LEVELS) + GROUND FLOOR & FIRST FLOOR (SHOP) AND SECOND FLOOR (OFFICE) (COMMERCIAL BUILDING) AT PLOT – 663, OLD DOOR NO – 103, NEW DOOR NO – 85, D – BLOCK, SECOND AVENUE & FIRST AVENUE ROAD, ANNA NAGAR EAST, CHENNAI – 600 102, COMPRISED IN OLD S. NO – 2/1B (PART), 7/1 (PART) NEW T.S NO – 3, BLOCK NO – 6, PERIYAKUDAL VILLAGE, AMINJIKARAI TALUK, CHENNAI DISTRICT WITHIN THE LIMIT OF GREATER CHENNAI CORPORATION.ZONE – VIII, DIVISION – 102

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 in to 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 CONSULTANT

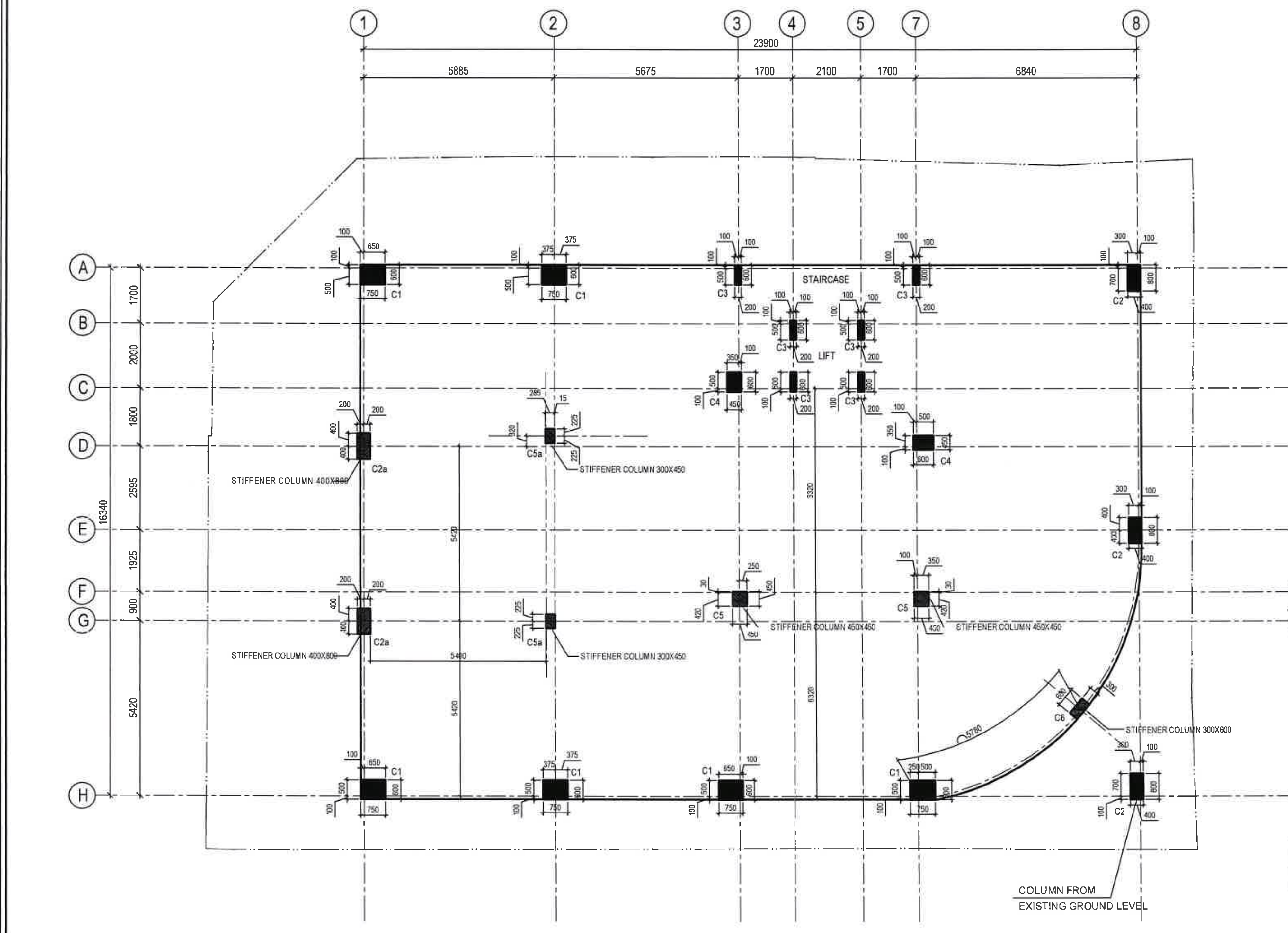
ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer - Grade - 1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 98404 48613

Date: 08-10-2025

Place: Chennai

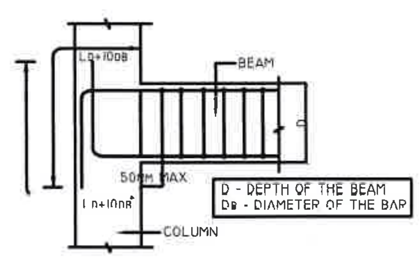
E.S. Kumar., M.Tec

Principal Structural Consult

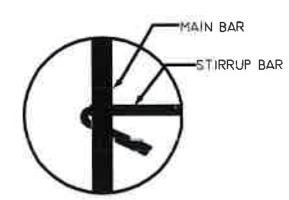


COLUMN AXIS LAYOUT

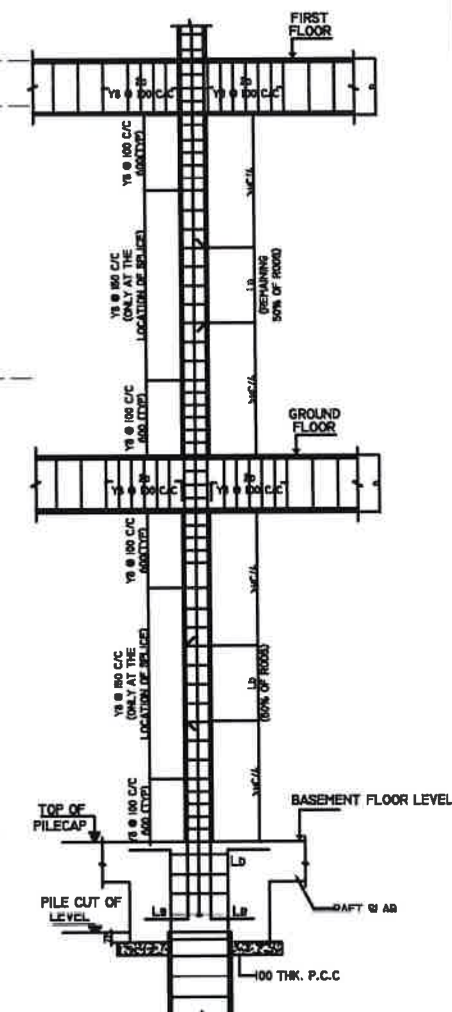
ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer - Grade - 1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 9840448613



TYP. EXTERNAL JOINT DETAIL AT BEAM-COLUMN JUNCTION



HOOK DETAIL



COLUMN AND PILECAP JUNCTION
(ALTERNATE RODS TO BE LAPPED IN ALTERNATE FLOORS)
(AS PER SEISMIC DETAILING CODE IS:13920)

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 : 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS —— 24 HOURS
B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) —— 3 DAYS
C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) —— 7 DAYS
D. BOTTOM PROPS TO SLAB
  i) SPANNING UP TO 4.50M —— 7 DAYS
  ii) SPANNING OVER 4.50M —— 14 DAYS
E. BOTTOM PROPS TO BEAMS
  i) SPANNING UP TO 6.00M —— 14 DAYS
  ii) SPANNING OVER 6.00M —— 21 DAYS

LAP JOINTS FOR REINFORCEMENT BARS:

i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED.
ii) LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS
iii) DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW.

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	450	400	370	330	370	320	290	280
10mm	565	480	460	400	460	390	370	320
12mm	680	580	560	480	550	470	440	390
16mm	900	780	730	640	730	630	590	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1560	1460	1300	1460	1250	1170	1030

iv) LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL.
v) LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE.
vi) WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT.
vii) DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED.
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY.
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION)
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 : 2000 GRADE OF CONCRETE FOR COLUMN,PILE, PILE CAP, RCC WALL AND RAFT SLAB - M30
6. 'Y' DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFIRMING TO IS 1786 : 2008
7. CLEAR COVER TO THE REINFORCEMENT
   A. COLUMNS —— 40mm
   B. PILE —— 75mm
   C. PILE CAP BOTTOM - 75mm, TOP & SIDES - 50mm
   D. RAFT SLAB —— 50 mm

	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137 T
500 MM (P2)	73 T

8. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT+GROUND FLOOR + 2 FLOORS ONLY
9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS 2911 (PART 1/SEC-2) -1999 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE. HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE
10. WORKING PILE : INITIALLY ONE OR MORE PILES WHICH ARE WORKING PILES, MAY BE INSTALLED TO ACCESS LOAD CARRYING CAPACITY OF A PILE. PILE OF THIS CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION.
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

REV.	DESCRIPTION	DRN	CHK.	DATE
03	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
02	COLUMN C5a DETAIL ADDED	MD	ESK	10.09.21
01	ISSUED FOR CONSTRUCTION	MD	ESK	17.08.21
00	ISSUED FOR APPROVAL	MD	ESK	06.08.21

PROJECT NAME :
PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: COLUMN AXIS LAYOUT

JOB NO.	DWG. NO.	REV. NO. 03	REF. CODE
KA -	S-01	Date 14.09.21	

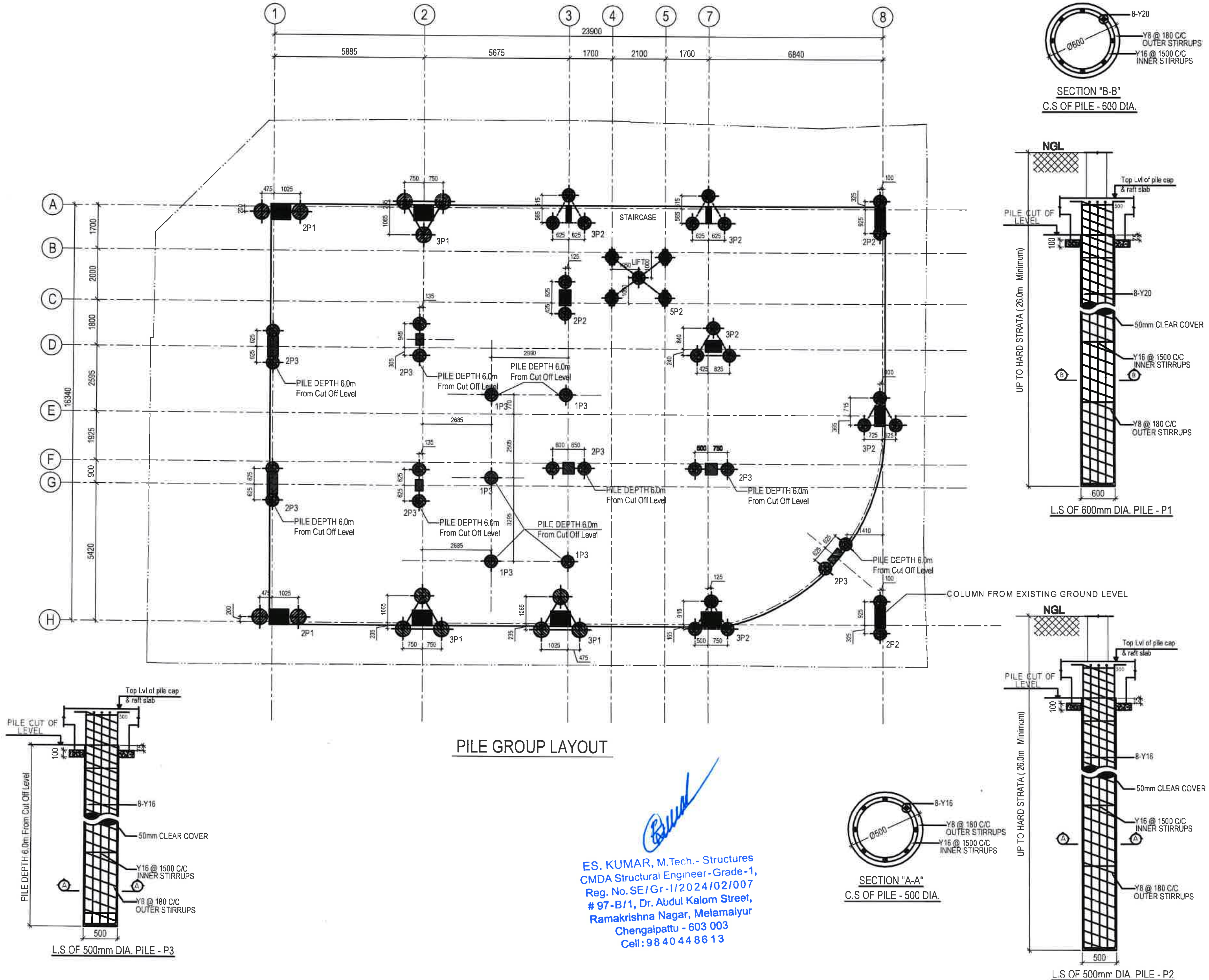
STRUCTURAL CONSULTANTS:

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B, PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob: 98404 48613.

ARCHITECTS:

Harismaa Consultant Ltd
#109 / A46 1 st FLOOR 3 rd AVENUE
ANNA NAGAR EAST CHENNAI - 600102
Tel 044 - 26203147

Drawn M.D	Designed DANE	Checked ES KUMAR



PILE GROUP LAYOUT

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

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 : 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS WALLS AND & BEAMS ——— 24 HOURS

B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) ——— 3 DAYS

C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) ——— 7 DAYS

D. BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M ——— 7 DAYS
ii) SPANNING OVER 4.50M ——— 14 DAYS

E. BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M ——— 14 DAYS
ii) SPANNING OVER 6.00M ——— 21DAYS

LAP JOINTS FOR REINFORCEMENT BARS:

i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED.

ii) LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS.

iii) DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	450	400	370	320	370	320	290	280
10mm	565	480	460	400	460	390	370	320
12mm	680	580	550	480	550	470	440	380
16mm	900	780	730	640	730	630	590	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1560	1460	1300	1460	1250	1170	1030

iv) LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL

v) LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE

vi) WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT

vii) DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY
4. THIS DWG SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION)
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 2000 GRADE OF CONCRETE FOR COLUMN,PILE, PILE CAP, RCC WALL AND RAFT SLAB - M30
6. 'Y' DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFIRMING TO IS 1786 2008
7. CLEAR COVER TO THE REINFORCEMENT
   A. COLUMNS ——— 40mm
   B. PILE ——— 75mm
   C. PILE CAP BOTTOM - 75mm TOP & SIDES - 50mm
   D. RAFT SLAB ——— 50 mm

	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137 T
500 MM (P2)	73 T

8. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT+GROUND FLOOR + 2 FLOORS ONLY
9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS 2911 (PART 1/SEC- 2) -1999 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE. HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE
10. WORKING PILE INITIALLY ONE OR MORE PILES WHICH ARE WORKING PILES, MAY BE INSTALLED TO ACCESS LOAD CARRYING CAPACITY OF A PILE. PILE OF THIS CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

REV.	DESCRIPTION	DRN.	CHK.	DATE
04	600 DIA PILE ADDED	MD	ESK	11.10.21
03	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
02	SINGLE PILE LOCATION ADDED	MD	ESK	10.09.21
01	ISSUED FOR CONSTRUCTION	MD	ESK	17.08.21
00	ISSUED FOR APPROVAL	MD	ESK	06.08.21

PROJECT NAME :

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX
AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: 500 DIA. PILE GROUP LAYOUT

JOB NO.	DWG. NO.	REV. NO. 04	REF. CODE
KA -	S-02	Date 11.10.21	

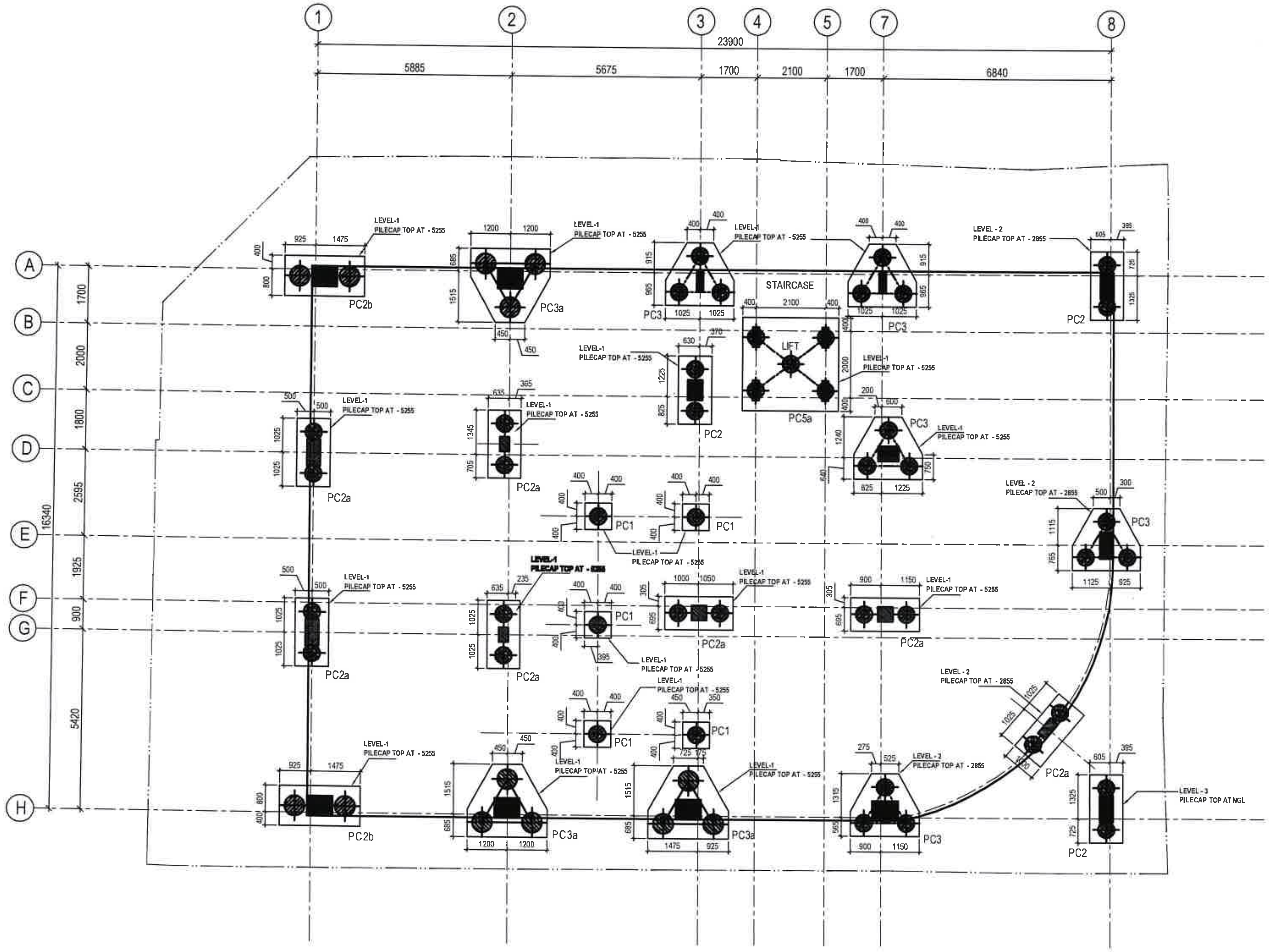
STRUCTURAL CONSULTANTS:

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob. 98404 40613.

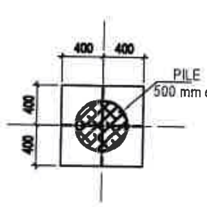
ARCHITECTS:

Harismaa Consultant Ltd
#109 / A46 1 st FLOOR 3 rd AVENUE
ANNA NAGAR EAST CHENNAI - 600102
Tel 044 - 26203147

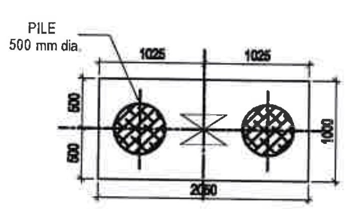
Drawn M.D	Designed DANE	Checked ES.KUMAR



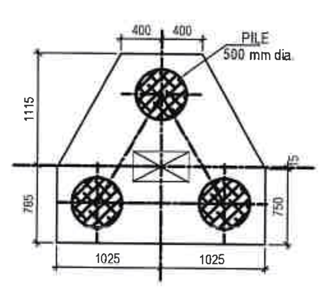
PILE CAP LAYOUT



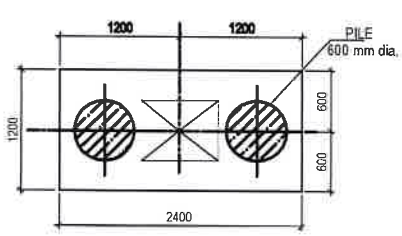
PC1 - LAYOUT



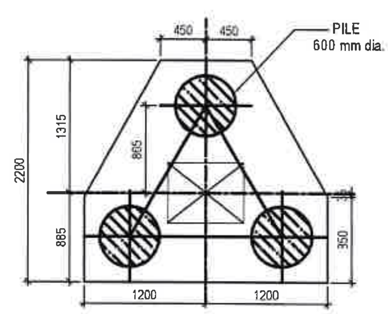
PC2 - LAYOUT



PC3 - LAYOUT



PC2b - LAYOUT



PC3a - LAYOUT

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

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS ———— 24 HOURS

B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) ———— 3 DAYS

C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) ———— 7 DAYS

D. BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M ———— 7 DAYS
ii) SPANNING OVER 4.50M ———— 14 DAYS

E. BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M ———— 14 DAYS
ii) SPANNING OVER 6.00M ———— 21DAYS

LAP JOINTS FOR REINFORCEMENT BARS:

i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED.

ii) LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS.

iii) DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW.

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	460	400	370	320	370	320	290	260
10mm	585	490	460	400	460	390	370	320
12mm	690	680	550	480	550	470	440	380
16mm	900	780	730	640	730	630	580	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1560	1460	1300	1460	1250	1170	1030

iv) LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL

v) LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE

vi) WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT

vii) DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION)
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 2000 GRADE OF CONCRETE FOR COLUMN, PILE, PILE CAP, RCC WALL AND RAFT SLAB - M30
6. "Y" DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFIRMING TO IS 1786 2008
7. CLEAR COVER TO THE REINFORCEMENT
   A. COLUMNS ———— 40mm
   B. PILE ———— 75mm
   C. PILE CAP BOTTOM - 75mm, TOP & SIDES - 50mm
   D. RAFT SLAB ———— 50 mm

DIA OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137 T
500 MM (P2)	73 T

8. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT+GROUND FLOOR + 2 FLOORS ONLY
9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS : 2911 (PART 1/SEC-2) -1999 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE. HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE
10. WORKING PILE : INITIALLY ONE OR MORE PILES WHICH ARE WORKING PILES, MAY BE INSTALLED TO ACCESS LOAD CARRYING CAPACITY OF A PILE. PILE OF THIS CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

REV.	DESCRIPTION	DRN.	CHK.	DATE
04	600 DIA PILE CAP ADDED	MD	ESK	11.10.21
03	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
02	PC1 ADDED FOR SUMP WALL SUPPORT	MD	ESK	10.09.21
01	ISSUED FOR CONSTRUCTION	MD	ESK	17.08.21
00	ISSUED FOR APPROVAL	MD	ESK	05.08.21

PROJECT NAME :
PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX
AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: PILE CAP LAYOUT

JOB NO.	DWG. NO.	REV. NO. 04	REF. CODE
KA -	S-03	Date 11.10.21	

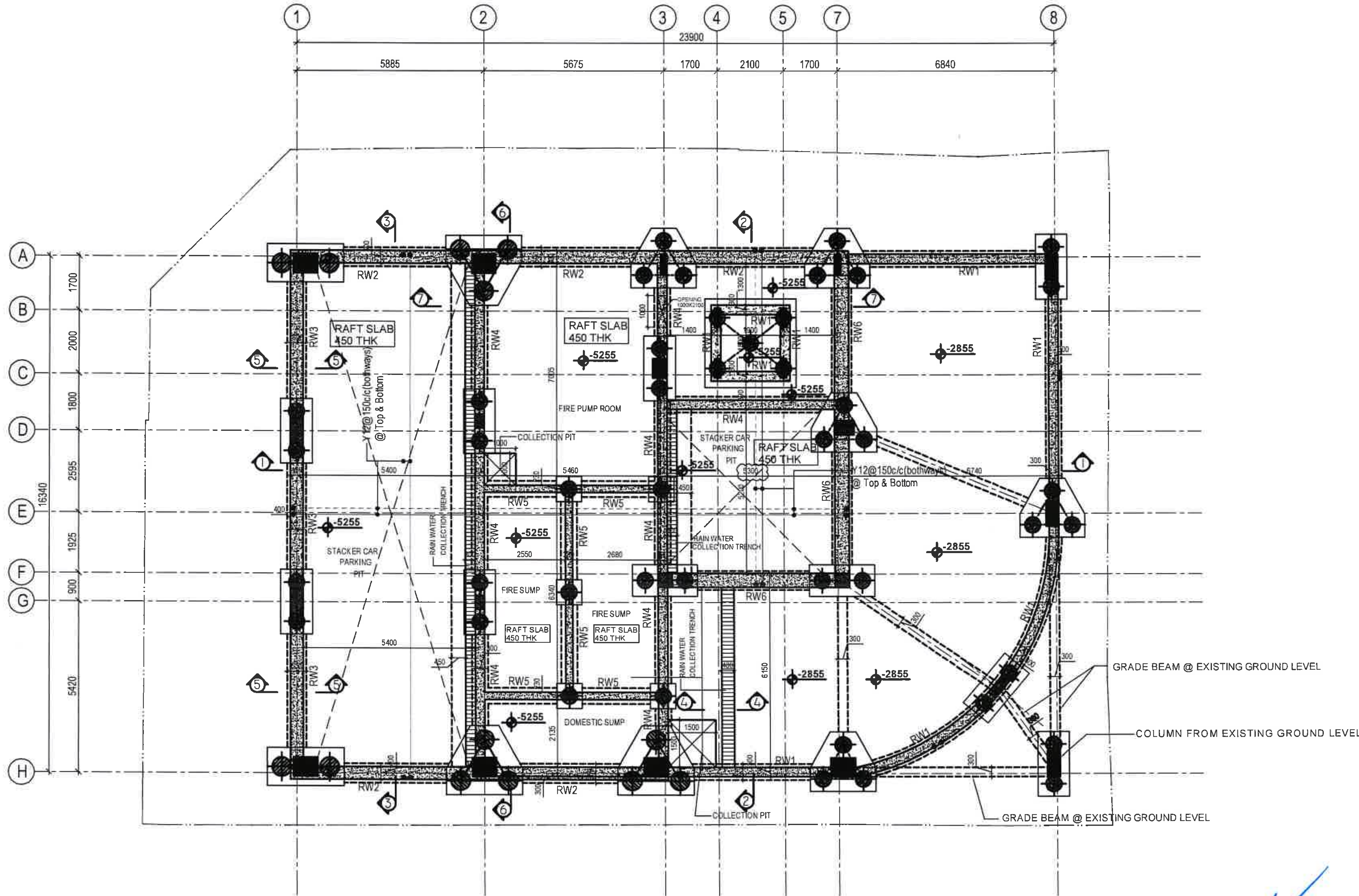
STRUCTURAL CONSULTANTS:

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B, PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob: 98404 48613.

ARCHITECTS:

Harismaa Consultant Ltd
#109 / A46 1 st FLOOR 3 rd AVENUE
ANNA NAGAR EAST CHENNAI - 600102
Tel 044 - 26203147

Drawn M.D	Designed DANE	Checked ES KUMAR



GRADE BEAM, R.C WALL AND RAFT SLAB LAYOUT

TOC - 5255

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 : 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS —— 24 HOURS

B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) —— 3 DAYS

C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) —— 7 DAYS

D. BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M —— 7 DAYS
ii) SPANNING OVER 4.50M —— 14 DAYS

E. BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M —— 14 DAYS
ii) SPANNING OVER 6.00M —— 21DAYS

LAP JOINTS FOR REINFORCEMENT BARS:

i). AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED.

ii). LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS.

iii). DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW.

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	450	400	370	320	370	320	290	280
10mm	565	480	460	400	460	390	370	320
12mm	680	580	550	480	550	470	440	390
16mm	900	780	730	640	730	630	590	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1560	1460	1300	1460	1250	1170	1030

iv). LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL

v). LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE

vi). WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAF OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT

vii). DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED.
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION).
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 : 2000 GRADE OF CONCRETE FOR COLUMN,PILE, PILE CAP, RCC WALL AND RAFT SLAB - M30
6. 'Y' DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFIRMING TO IS 1786 : 2008
7. CLEAR COVER TO THE REINFORCEMENT
   A. COLUMNS —— 40mm
   B. PILE —— 75mm
   C. PILE CAP BOTTOM - 75mm, TOP & SIDES - 50mm
   D. RAFT SLAB —— 50 mm

DIA. OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137 T
500 MM (P2)	73 T

8. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT+GROUND FLOOR + 2 FLOORS ONLY
9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS : 2911 (PART 1/SEC-2) -1999 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE. HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE
10. WORKING PILE : INITIALLY ONE OR MORE PILES WHICH ARE WORKING PILES, MAY BE INSTALLED TO ACCESS LOAD CARRYING CAPACITY OF A PILE. PILE OF THIS CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) : 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

REV.	DESCRIPTION	DRN.	CHK.	DATE
04	600 DIA PILE ADDED	MD	ESK	11.10.21
03	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
02	FIRE PUMP ROOM & FIRE SUMP ADDED	MD	ESK	10.09.21
01	ISSUED FOR CONSTRUCTION	MD	ESK	17.08.21
00	ISSUED FOR APPROVAL	MD	ESK	06.08.21

PROJECT NAME :

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX
AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: RAFT SLAB AND R.C WALL LAYOUT

JOB NO.	DWG. NO.	REV. NO. 04	REF. CODE
KA -	S-04	Date 11.10.21	

STRUCTURAL CONSULTANTS:

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:18,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai 600 040. Mob: 98404 48613.

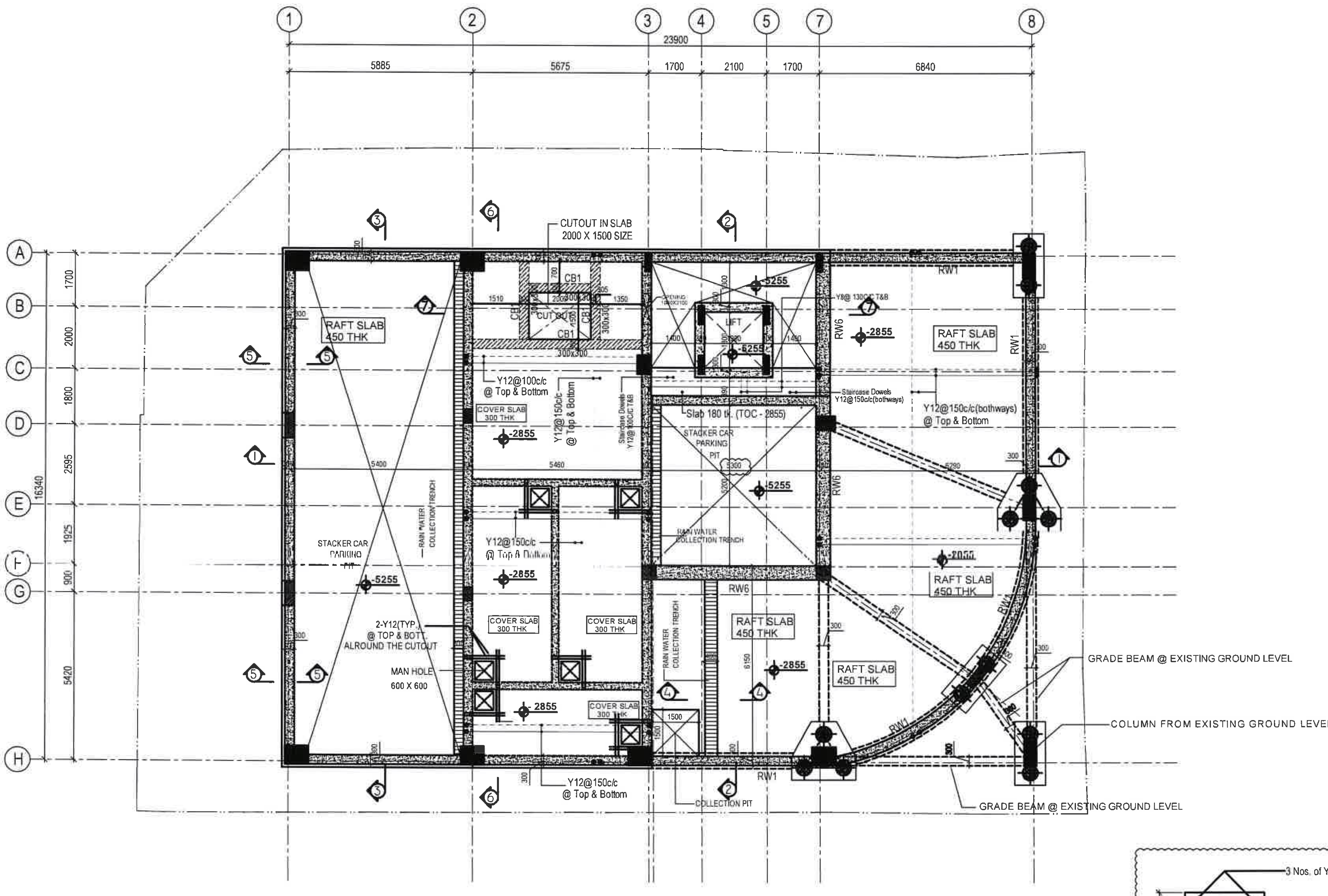
ARCHITECTS:

Harismaa Consultant Ltd
#109 / A46 1 st FLOOR 3 rd AVENUE
ANNA NAGAR EAST, CHENNAI - 600102
Tel 044 - 26203147

Drawn M.D	Designed DANE	Checked ES.KUMAR

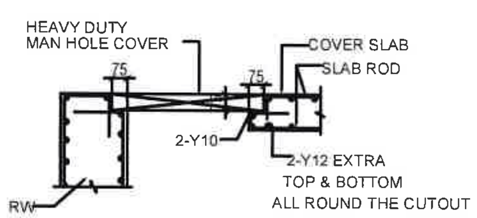
ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer - Grade - 1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell : 9840448613

IMPORTANT NOTE:
ALL REINFORCED CONCRETE MEMBERS BELOW GROUND FLOOR LEVEL SHOULD BE WATER PROOFED WITH INTEGRAL WATER PROOFING ADMIXTURE PENETRON OR EQUIVALENT AND ALL COLD JOINTS SHOULD BE TREATED WITH PRESSURE GROUTING AS PER STANDARD PRACTICE FOR EFFECTIVE WATER TIGHTNESS.

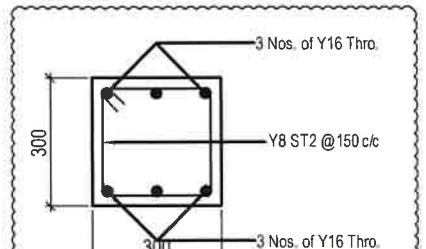


GRADE BEAM, R.C WALL, SUMP COVER SLAB AND RAFT SLAB LAYOUT

TOC - 2855



TYPICAL MANHOLE DETAIL



C.S. OF BEAM -CB1
(300 x 300)

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

IMPORTANT NOTE:
ALL REINFORCED CONCRETE MEMBERS BELOW GROUND FLOOR LEVEL SHOULD BE WATER PROOFED WITH INTEGRAL WATER PROOFING ADMIXTURE PENETRON OR EQUIVALENT AND ALL COLD JOINTS SHOULD BE TREATED WITH PRESSURE GROUTING AS PER STANDARD PRACTICE FOR EFFECTIVE WATER TIGHTNESS

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11 3 1 OF IS456 : 2000

A VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS —— 24 HOURS

B BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) —— 3 DAYS

C BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) —— 7 DAYS

D BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M —— 7 DAYS
ii) SPANNING OVER 4.50M —— 14 DAYS

E BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M —— 14 DAYS
ii) SPANNING OVER 6.00M —— 21DAYS

LAP JOINTS FOR REINFORCEMENT BARS:

i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED

ii) LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS

iii) DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP. 34 - 1999 AND GIVEN IN THE TABLE BELOW

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	450	400	370	320	370	320	290	280
10mm	565	480	460	400	460	380	370	320
12mm	680	580	550	480	550	470	440	390
16mm	900	780	730	640	730	630	590	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1560	1450	1300	1450	1250	1170	1030

iv) LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL

v) LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE

vi) WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT

vii) DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

GENERAL NOTES:

1 ALL DIMENSIONS ARE IN mm

2 ONLY WRITTEN DIMENSIONS TO BE FOLLOWED.

3 ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY

4 THIS DWG SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG (LATEST REVISION)

5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 2000 GRADE OF CONCRETE FOR COLUMN PILE, PILE CAP, RCC WALL AND RAFT SLAB - M30

6. 1 DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFIRMING TO IS 1786 2008

7 CLEAR COVER TO THE REINFORCEMENT
A COLUMNS —— 40mm
B PILE —— 75mm
C PILE CAP BOTTOM - 75mm, TOP & SIDES - 50mm
D RAFT SLAB —— 50 mm

DIA. OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137 T
500 MM (P2)	73 T

8 THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT+GROUND FLOOR + 2 FLOORS ONLY

9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS : 2911 (PART 1/SEC- 2) -1999 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE. HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE

10 WORKING PILE : INITIALLY ONE OR MORE PILES WHICH ARE WORKING PILES. MAY BE INSTALLED TO ACCESS LOAD CARRYING CAPACITY OF A PILE. PILE OF THIS CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION.

11 THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

REV.	DESCRIPTION	DRN	CHK	DATE
01	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
00	ISSUED FOR APPROVAL	MD	ESK	10.09.21

PROJECT NAME :
PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX
AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

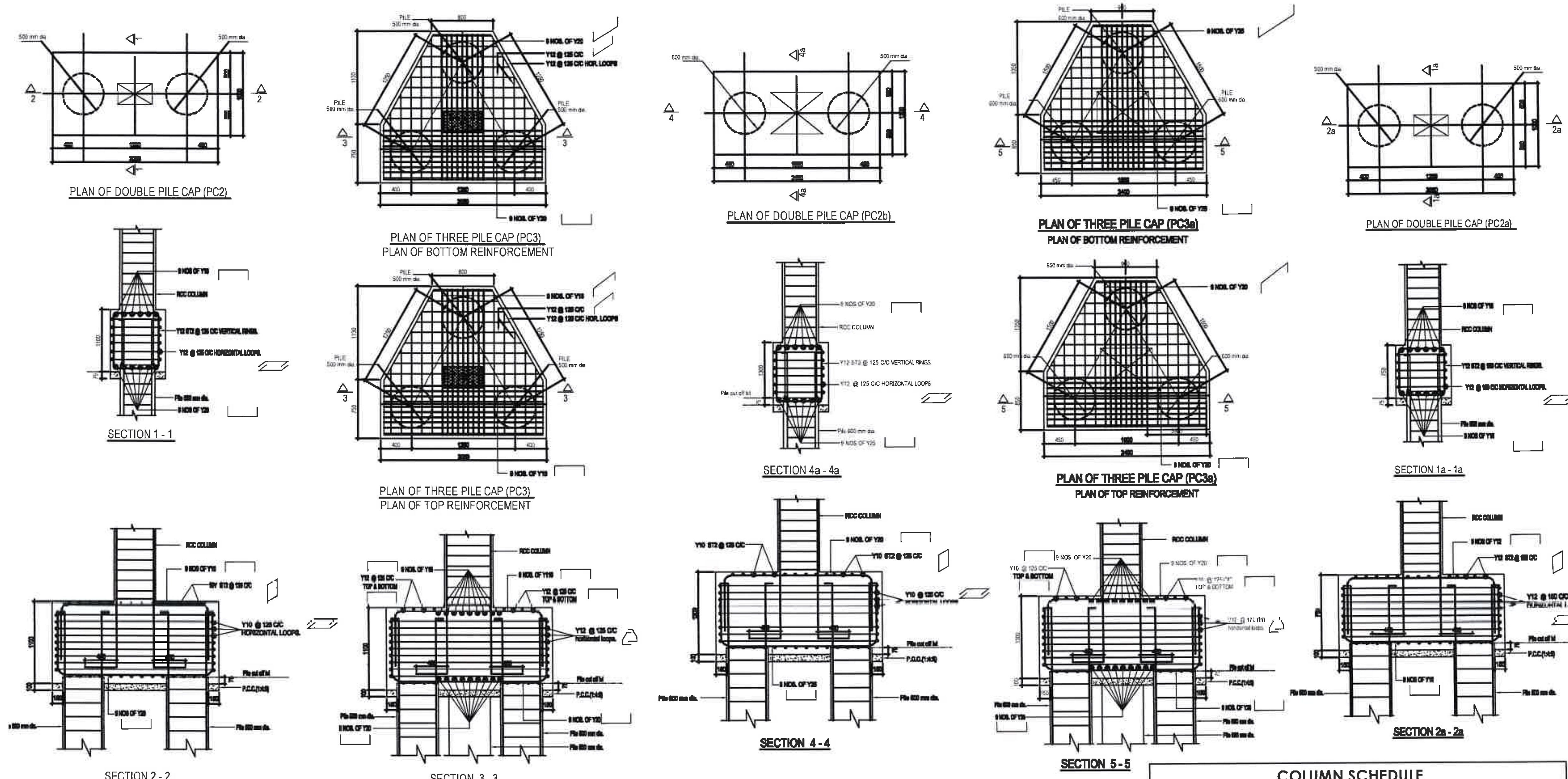
DRAWING TITLE: COVER SLAB, RAFT SLAB AND R.C WALL LAYOUT

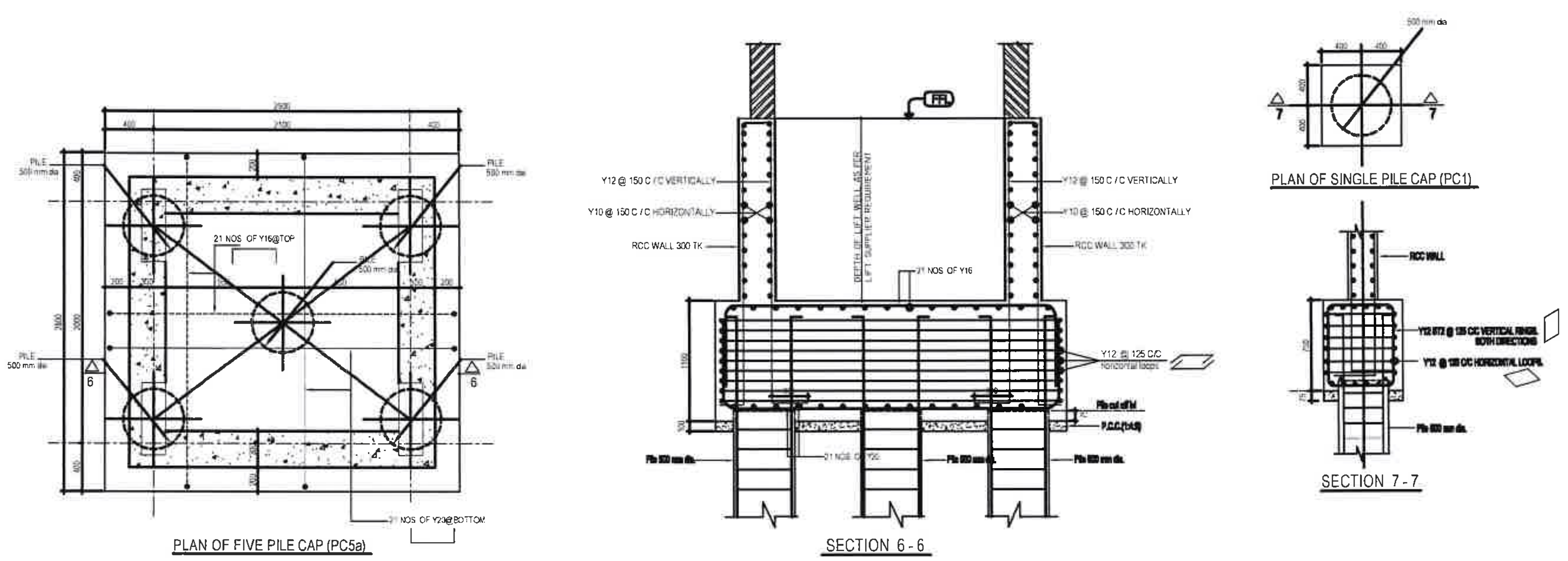
JOB NO. KA - | DWG. NO. S-04A | REV. NO. 01 Date 14.09.21 | REF. CODE

STRUCTURAL CONSULTANTS:
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B, PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR (WEST)
Chennai - 600 040. Mob: 70404 40613.

ARCHITECTS:
Harismaa Consultant Ltd
#109 / A46 1 st FLOOR 3 rd AVENUE
ANNA NAGAR EAST CHENNAI - 600102
Tel 044 - 20203147

Drawn M.D | Designed DANE | Checked ES KUMAR





COLUMN SCHEDULE

DESCRIPTION	SIZE	REINFORCEMENT	SHAPE OF TIES
C1	600 x 750	22 NOS. OF Y32 MAIN RODS Y10 TIES @ 150 C/C	THREE CLOSED FIVE OPEN
C2	400 x 800	14 NOS. OF Y20 MAIN RODS Y8 TIES @ 150 C/C	THREE CLOSED ONE OPEN
C2a	400 x 800	14 NOS. OF Y16 MAIN RODS Y8 TIES @ 150 C/C	THREE CLOSED ONE OPEN
C3	200 x 600	10 NOS. OF Y20 MAIN RODS Y8 TIES @ 150 C/C	TWO CLOSED ONE OPEN
C4	450 x 600	18 NOS. OF Y25 MAIN RODS Y8 TIES @ 150 C/C	THREE CLOSED THREE OPEN
C5	450 x 450	12 NOS. OF Y16 MAIN RODS Y8 TIES @ 150 C/C	THREE CLOSED
C5a	300 x 450	10 NOS. OF Y16 MAIN RODS Y8 TIES @ 150 C/C	TWO CLOSED
C6	300 x 600	12 NOS. OF Y16 MAIN RODS Y8 TIES @ 150 C/C	TWO CLOSED ONE OPEN

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING)

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456: 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS WALLS AND & BEAMS ———— 24 HOURS

B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) ———— 3 DAYS

C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) ———— 7 DAYS

D. BOTTOM PROPS TO SLAB
- i) SPANNING UP TO 4.50M ———— 7 DAYS
- ii) SPANNING OVER 4.50M ———— 14 DAYS

E. BOTTOM PROPS TO BEAMS
- i) SPANNING UP TO 6.00M ———— 14 DAYS
- ii) SPANNING OVER 6.00M ———— 21 DAYS

LAP JOINTS FOR REINFORCEMENT BARS:

i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED

ii) LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS

iii) DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP 34 - 1999 AND GIVEN IN THE TABLE BELOW

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	1120	[illegible]	910	[illegible]	910	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

iv) LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL

v) LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE

vi) WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT

vii) DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION)
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 : 2000 GRADE OF CONCRETE FOR COLUMN PILE PILE CAP RCC WALL AND RAFT SLAB - M30
6. 'Y' DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFIRMING TO IS 1786 : 2008
7. CLEAR COVER TO THE REINFORCEMENT
   - A. COLUMNS ———— 40mm
   - B. PILE ———— 75mm
   - C. PILE CAP BOTTOM - 75mm. TOP & SIDES - 50mm
   - D. RAFT SLAB ———— 50 mm

DIA. OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137T
500 MM (P2)	73T

8. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT+GROUND FLOOR + 2 FLOORS ONLY
9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS 2911 (PART 1/SEC- 2) -1979 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE. HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE
10. WORKING PILE : INITIALLY ONE OR MORE PILES WHICH ARE WORKING PILES MAY BE INSTALLED TO ACCESS LOAD CARRYING CAPACITY OF A PILE. PILE OF THIS CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) : 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

E.S. 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

REV	DESCRIPTION	DRN	CHK	DATE
04	PILE CAP PC2b AND PC3a ADDED	MD	ESK	11.10.21
03	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
02	COLUMN C5a DETAIL ADDED	MD	ESK	10.09.21
01	ISSUED FOR CONSTRUCTION	MD	ESK	17.08.21
00	ISSUED FOR APPROVAL	MD	ESK	06.08.21

PROJECT NAME

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX AT ANNA NAGAR EAST, CHENNAI 600102

CLIENT: EMPEE GROUP

DRAWING TITLE: FOUNDATION PILE CAP DETAIL

JOB NO	DWG. NO	REV. NO	REF. CODE
KA-	S-05	04 Date: 11.10.21	

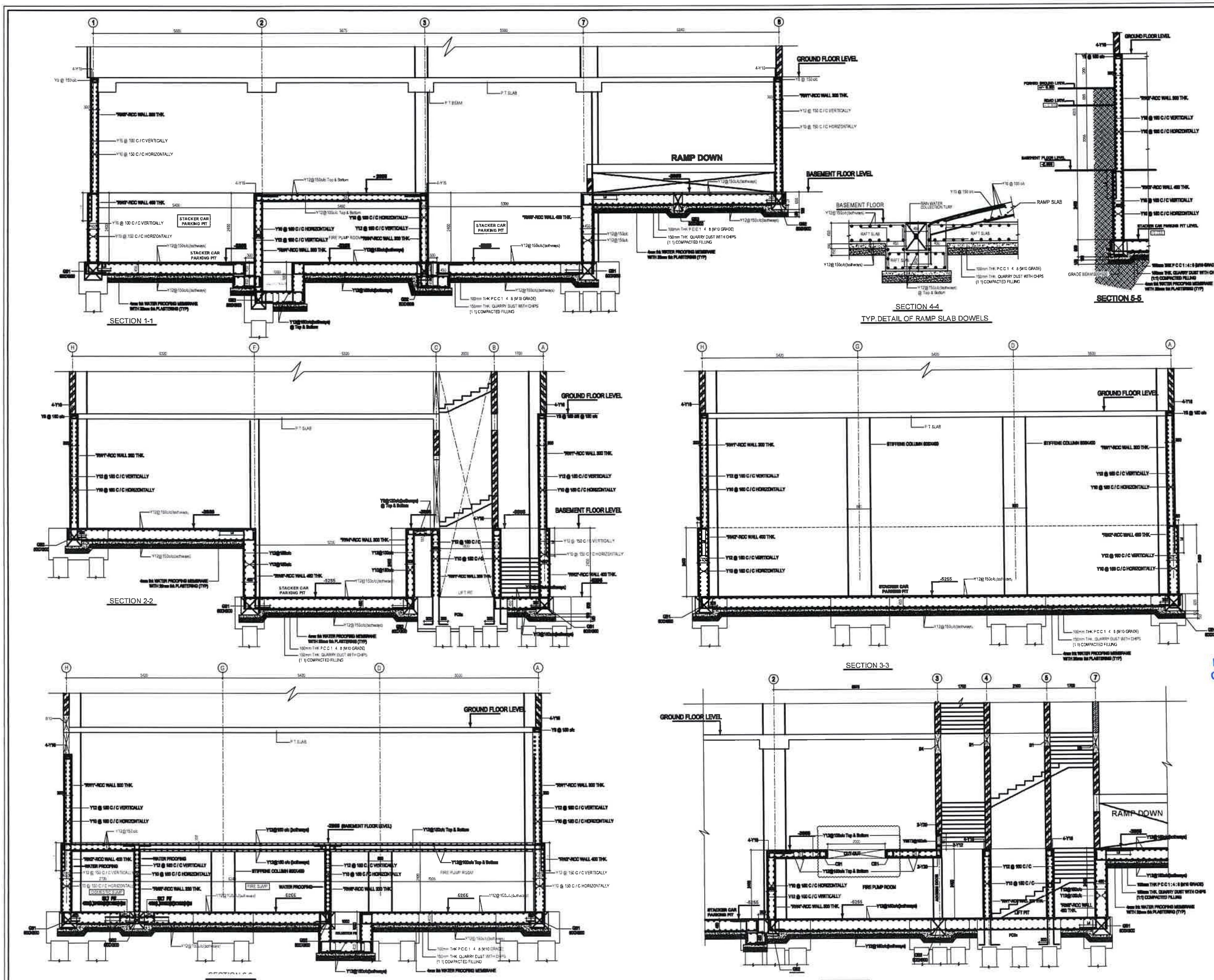
STRUCTURAL CONSULTANTS

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR
FLAT NO 1B, PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR (WEST)
Chennai - 600 040 Mob: 98404 48613

ARCHITECTS:

Harismaa Consultant Ltd
#109/A46, 1st FLOOR, 3rd AVENUE
ANNA NAGAR EAST, CHENNAI 600102
Tel: 044 - 26223147

Drawn: M.D.	Designed: DANE	Checked: ES KUMAR



GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS ———— 24 HOURS

B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) ———— 3 DAYS

C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) ———— 7 DAYS

D. BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M ———— 7 DAYS
ii) SPANNING OVER 4.50M ———— 14 DAYS

E. BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M ———— 14 DAYS
ii) SPANNING OVER 6.00M ———— 21 DAYS

LAP JOINTS FOR REINFORCEMENT BARS:

i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED

ii) LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS

iii) DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP 34 - 1999 AND GIVEN IN THE TABLE BELOW

DIAMETER OF BARS	TENSION BARS	COMPRESSION BARS
[illegible]	[illegible]	[illegible]

iv) LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL

v) LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE

vi) WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT

vii) DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm AND LEVELS ARE IN METERS
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION)
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 2000. GRADE OF CONCRETE FOR COLUMN, PILE, PILE CAP, RCC WALL AND RAFT SLAB = M30
6. 'Y' DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFIRMING TO IS 1786 2008
7. CLEAR COVER TO THE REINFORCEMENT
   A. COLUMNS ———— 40mm
   B. PILE ———— 75mm
   C. PILE CAP BOTTOM - 75mm, TOP & SIDES - 50mm
   D. RAFT SLAB ———— 50 mm

DIA. OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137 T
500 MM (P2)	73 T

8. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT+GROUND FLOOR + 2 FLOORS ONLY
9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS 2911 (PART 1/SEC-2) -1999 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE. HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE
10. WORKING PILE: INITIALLY ONE OR MORE PILES WHICH ARE WORKING PILES MAY BE INSTALLED TO ACCESS LOAD CARRYING CAPACITY OF A PILE. PILE OF THIS CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD. THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

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

REV	DESCRIPTION	DRN	CHK	DATE
03	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
02	FIRE PUMP ROOM & FIRE SUMP ADDED	MD	ESK	10.09.21
01	ISSUED FOR CONSTRUCTION	MD	ESK	17.08.21
00	ISSUED FOR APPROVAL	MD	ESK	05.08.21

PROJECT NAME

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX AT ANNA NAGAR EAST, CHENNAI 600102

CLIENT: EMPEE GROUP

DRAWING TITLE: SECTION DETAILS

JOB NO	DWG. NO	REV. NO. 00	REF. CODE
KA..	S-06	Date 14.09.21	

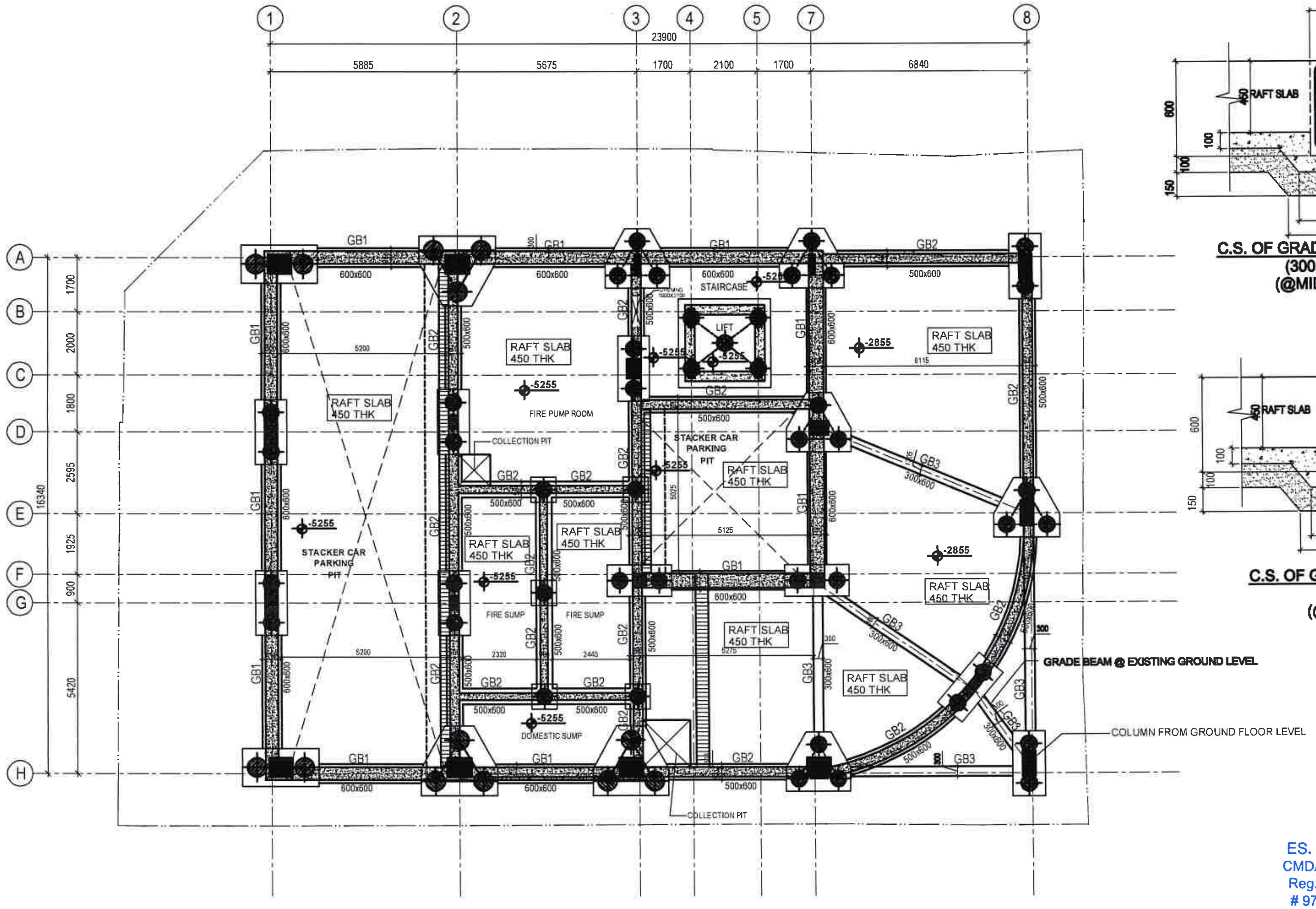
STRUCTURAL CONSULTANTS

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO. 1233, FIRST FLOOR
FLAT NO 1B PRIYA AJAY ARCADE
18th MAIN ROAD, ANNA NAGAR (WEST)
Chennai - 600 040. Mob. 98404 48613

ARCHITECTS:

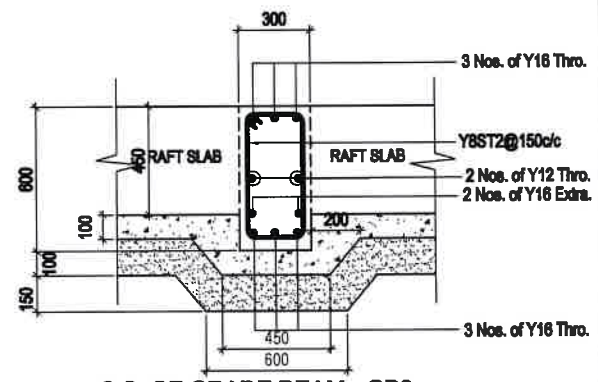
Harismaa Consultant Ltd

Drawn: MD | Designed: DANE | Checked: ES KUMAR

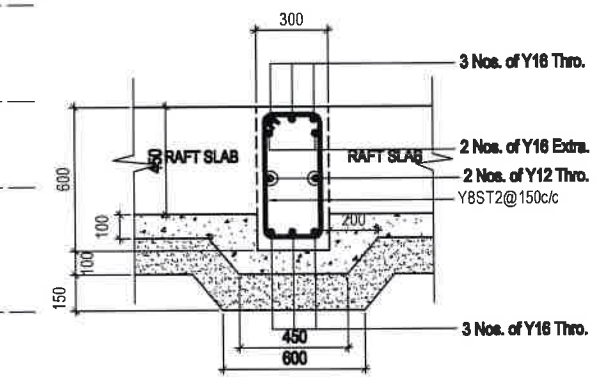


GRADE BEAM LAYOUT

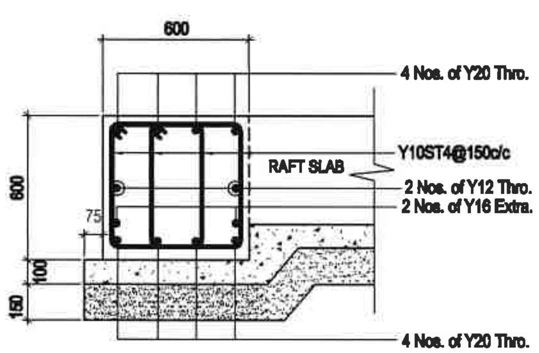
FOR T.O.C LEVELS - 2855 & - 5255



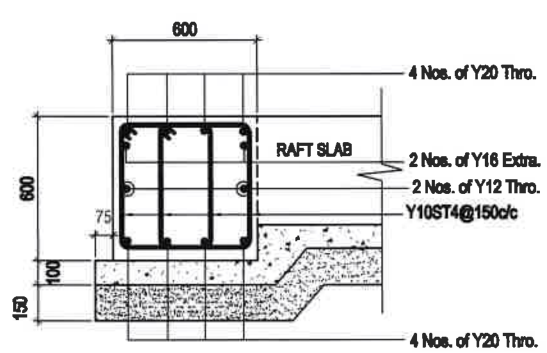
C.S. OF GRADE BEAM - GB3
(300 x 600)
(@MID-SPAN)



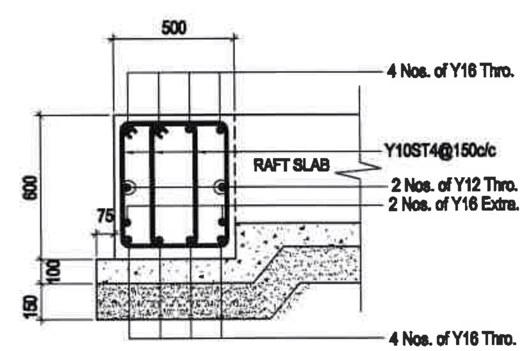
C.S. OF GRADE BEAM - GB3
(300 x 600)
(@SUPPORT)



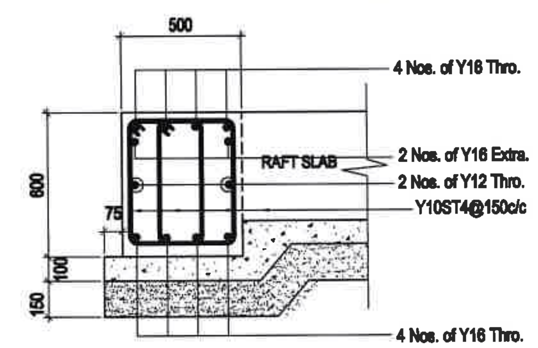
C.S. OF GRADE BEAM - GB1
(600 x 600)
(@MID-SPAN)



C.S. OF GRADE BEAM - GB1
(600 x 600)
(@SUPPORT)



C.S. OF GRADE BEAM - GB2
(500 x 600)
(@MID-SPAN)



C.S. OF GRADE BEAM - GB2
(500 x 600)
(@SUPPORT)

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 : 2000

A VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS — 24 HOURS

B BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) — 3 DAYS

C BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) — 7 DAYS

D BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M — 7 DAYS
ii) SPANNING OVER 4.50M — 14 DAYS

E BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M — 14 DAYS
ii) SPANNING OVER 6.00M — 21DAYS

LAP JOINTS FOR REINFORCEMENT BARS:

i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED

ii) LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS

iii) DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	450	400	370	320	370	320	290	280
10mm	565	480	460	400	480	390	370	320
12mm	680	580	550	480	550	470	440	380
16mm	900	780	730	640	730	630	590	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1560	1460	1300	1460	1250	1170	1030

iv) LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL

v) LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE

vi) WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT

vii) DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION)
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 : 2000 GRADE OF CONCRETE FOR COLUMN PILE, PILE CAP, RCC WALL,RAFT SLAB AND GRADE BEAM - M30
6. 'Y' DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFIRMING TO IS 1786 : 2008
7. CLEAR COVER TO THE REINFORCEMENT
   A COLUMNS — 40mm
   B PILE — 75mm
   C PILE CAP BOTTOM - 75mm, TOP & SIDES - 50mm
   D RAFT SLAB — 50 mm
   E GRADE BEAM — 40 mm

DIA. OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137 T
500 MM (P2)	73 T

8. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT+GROUND FLOOR + 2 FLOORS ONLY
9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS : 2911 (PART 1/SEC- 2) -1999 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE. HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE
10. WORKING PILE : INITIALLY ONE OR MORE PILES WHICH ARE WORKING PILES, MAY BE INSTALLED TO ACCESS LOAD CARRYING CAPACITY OF A PILE. PILE OF THIS CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) : 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

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

REV.	DESCRIPTION	DRN.	CHK.	DATE
04	PILECAP LAYOUT REVISED	MD	ESK	11.10.21
03	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
02	FIRE PUMP ROOM & FIRE SUMP ADDED	MD	ESK	10.09.21
01	ISSUED FOR CONSTRUCTION	MD	ESK	17.08.21
00	ISSUED FOR APPROVAL	MD	ESK	06.08.21

PROJECT NAME :

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX
AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

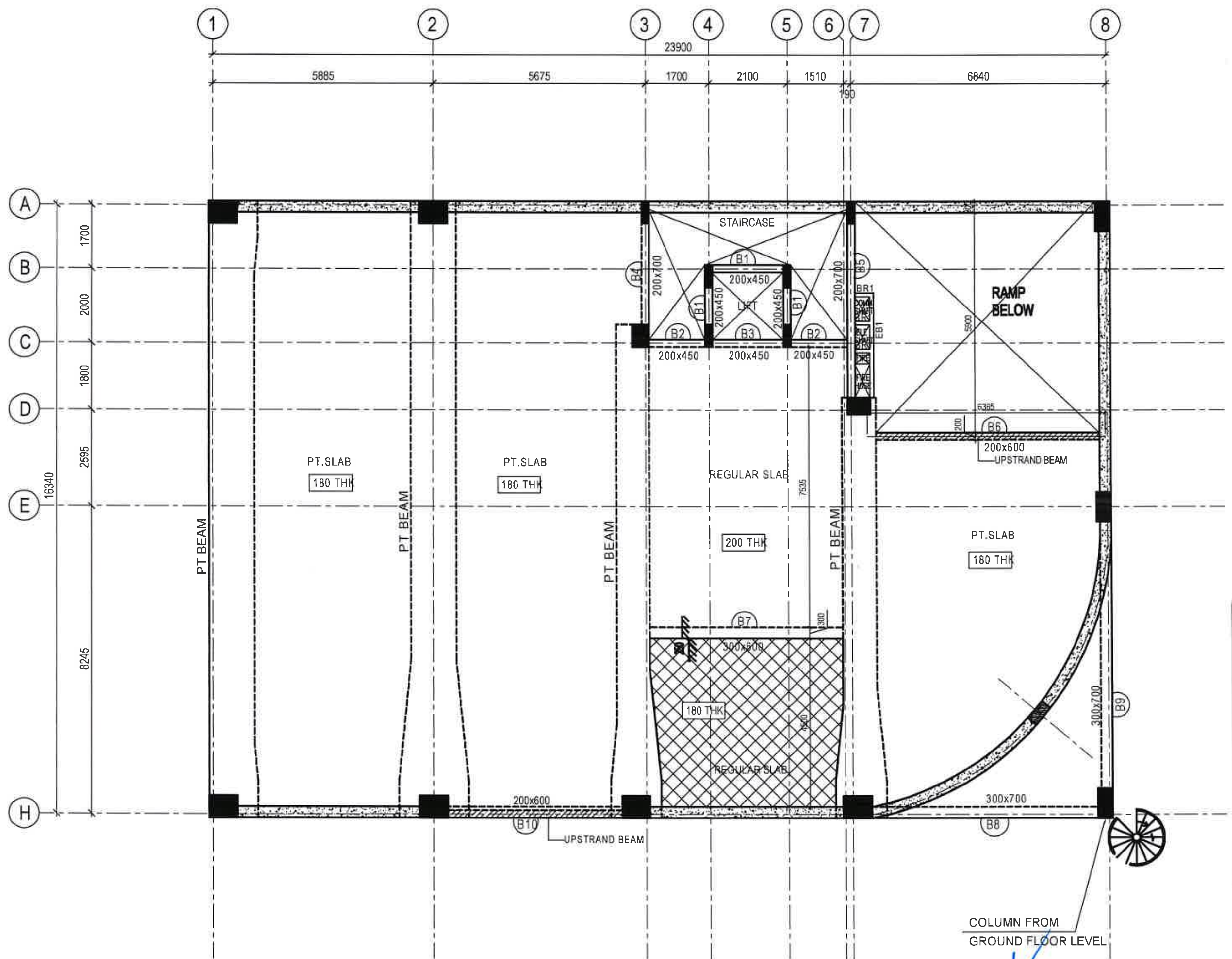
DRAWING TITLE: GRADE BEAM DETAIL

JOB NO.	DWG. NO.	REV. NO. 04	REF. CODE
KA-	S-07	Date 11.10.21	

STRUCTURAL CONSULTANTS:
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob: 98404 48613

ARCHITECTS:
Harismaa Consultant Ltd
#109/A46, 1st FLOOR, 3rd AVENUE
ANNA NAGAR EAST, CHENNAI - 600102
Tel 044 - 26203147

Drawn M.D	Designed DANE	Checked ES.KUMAR



BASEMENT ROOF BEAM LAYOUT

BEAM SCHEDULE		
SL.NO	NAME	SIZE
01	B1	200 X 450
02	B2	200 X 450
03	B3	200 X 450
04	B4	200 X 700
05	B5	200 X 700
06	B6	200 X 600
07	B7	300 X 600
08	B8	300 X 700
09	B9	300 X 700
10	B10	200 X 600
11	EB1	100 X 450
12	BR1	100 X 450

LEGEND

= SUNKEN BY 250

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

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 : 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS ———— 24 HOURS.

B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) ———— 3 DAYS.

C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) ———— 7 DAYS.

D. BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M ———— 7 DAYS.
ii) SPANNING OVER 4.50M ———— 14 DAYS.

E. BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M ———— 14 DAYS.
ii) SPANNING OVER 6.00M ———— 21DAYS.

LAP JOINTS FOR REINFORCEMENT BARS:

i). AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED.

ii). LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS.

iii). DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW.

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	450	400	370	320	370	320	290	260
10mm	565	490	460	400	460	390	370	320
12mm	680	590	550	480	550	470	440	390
16mm	900	780	730	640	730	630	590	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1560	1460	1280	1460	1250	1170	1030

iv). LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL.

v). LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE.

vi). WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT.

vii). DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920.

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED.
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY.
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION).
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 : 2000 GRADE OF CONCRETE FOR COLUMN - M30, SLAB & BEAM - M35
6. "Y" DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500D CONFIRMING TO IS 1786 : 2008
7. CLEAR COVER TO THE REINFORCEMENT
   A. COLUMNS ---------- 40mm
   B. ROOF BEAM ---------- 30mm
   C. ROOF SLAB ---------- 25mm
8. DEPTH OF BEAM IS INCLUSIVE OF SLAB THICKNESS

01	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
00	ISSUED FOR APPROVAL	MD	ESK	27.08.21
REV.	DESCRIPTION	DRN.	CHK.	DATE

PROJECT NAME :

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: BASEMENT ROOF BEAM LAYOUT

JOB NO.	DWG. NO.	REV. NO. 01	REF. CODE
KA -	S-09	Date 14.09.21	

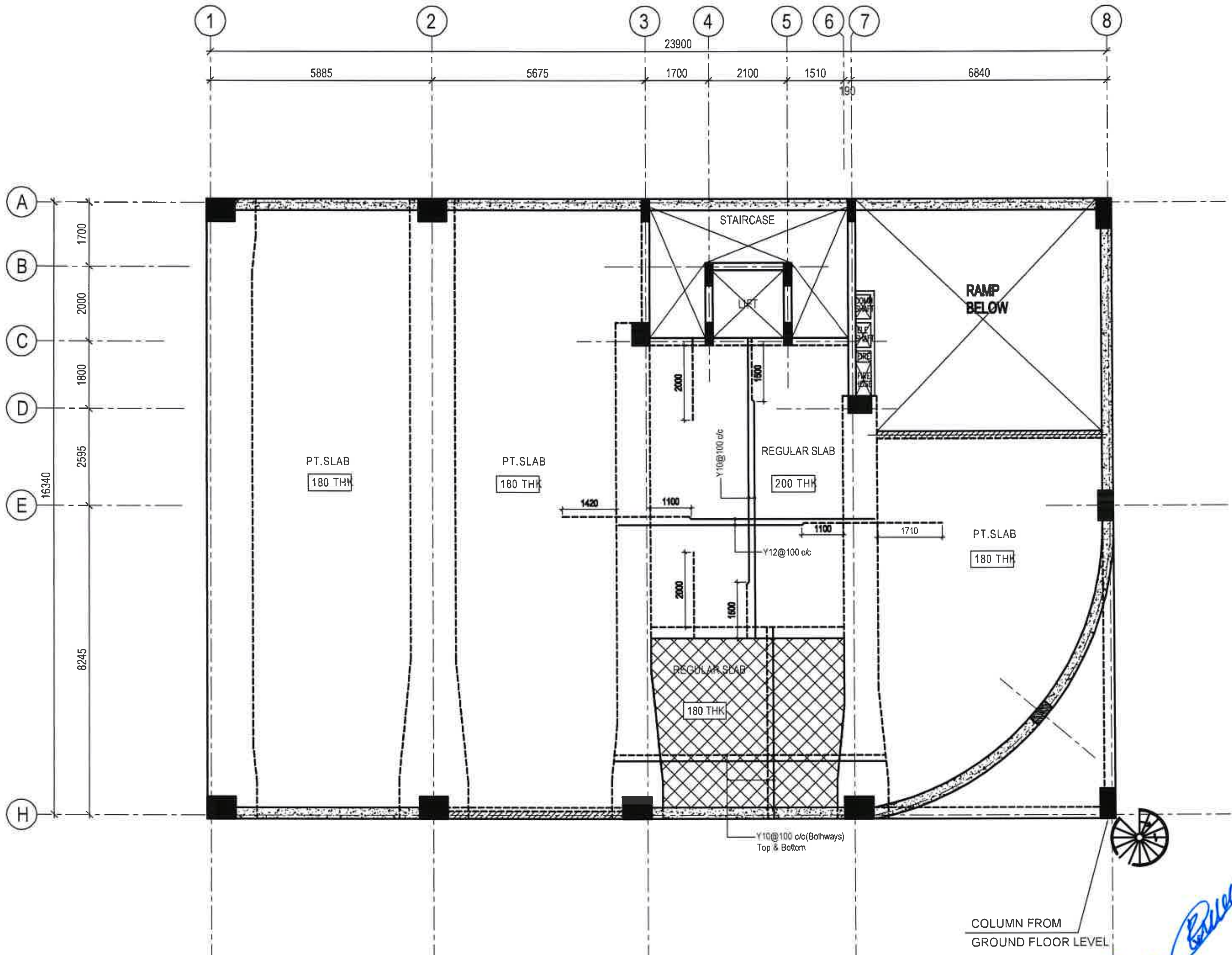
STRUCTURAL CONSULTANTS:

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob: 98404 48613.

ARCHITECTS:

Harismaa Consultant Ltd
#109 / A46 , 1 st FLOOR, 3 rd AVENUE
ANNA NAGAR EAST CHENNAI - 600102
Tel: 044 - 26203147

Drawn M.D	Designed: ES	Checked ES.KUMAR



BASEMENT ROOF SLAB LAYOUT

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

LEGEND

= SUNKEN BY 250

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 : 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS ———— 24 HOURS.

B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) ———— 3 DAYS.

C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) ———— 7 DAYS.

D. BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M ———— 7 DAYS.
ii) SPANNING OVER 4.50M ———— 14 DAYS.

E. BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M ———— 14 DAYS.
ii) SPANNING OVER 6.00M ———— 21DAYS.

LAP JOINTS FOR REINFORCEMENT BARS:

i). AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED.

ii). LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS.

iii). DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW.

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	480	400	370	320	370	320	280	280
10mm	585	490	460	400	460	390	370	320
12mm	680	590	550	480	550	470	440	390
16mm	900	780	730	640	730	630	590	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1550	1460	1300	1450	1250	1170	1030

iv). LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL.

v). LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE.

vi). WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT.

vii). DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920.

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED.
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY.
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION).
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 : 2000
GRADE OF CONCRETE FOR COLUMN - M30, SLAB & BEAM - M35
6. "Y" DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500D CONFIRMING TO IS 1786 : 2008
7. CLEAR COVER TO THE REINFORCEMENT
A. COLUMNS ———— 40mm
B. ROOF BEAM ———— 30mm
C. ROOF SLAB ———— 25mm
8. DEPTH OF BEAM IS INCLUSIVE OF SLAB THICKNESS

REV.	DESCRIPTION	DRN.	CHK	DATE
01	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
00	ISSUED FOR APPROVAL	MD	ESK	27.08.21

PROJECT NAME :
PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: BASEMENT ROOF SLAB LAYOUT

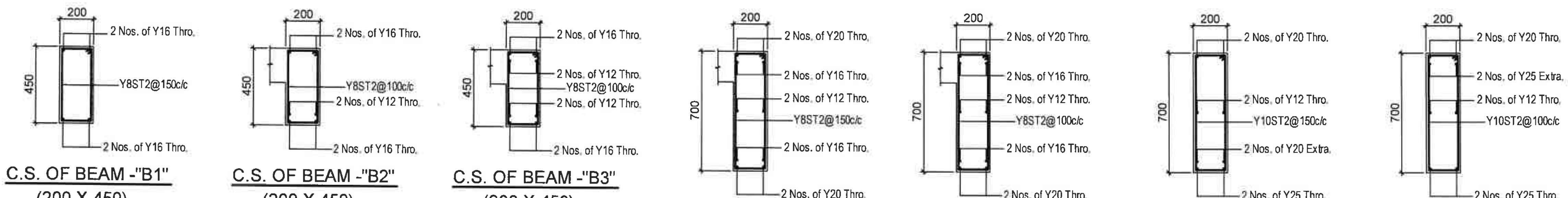
JOB NO.	DWG. NO.	REV. NO. 01	REF. CODE
KA -	S-10	Date 14.09.21	

STRUCTURAL CONSULTANTS:

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob: 98404 48613.

ARCHITECTS: Harismaa Consultant Ltd
#109/A46, 1st FLOOR 3rd AVENUE
ANNA NAGAR EAST CHENNAI - 600102
Tel 044 - 26203147

Drawn MD	Designed ES	Checked ES.KUMAR



C.S. OF BEAM -"B1"
(200 X 450)

C.S. OF BEAM -"B2"
(200 X 450)

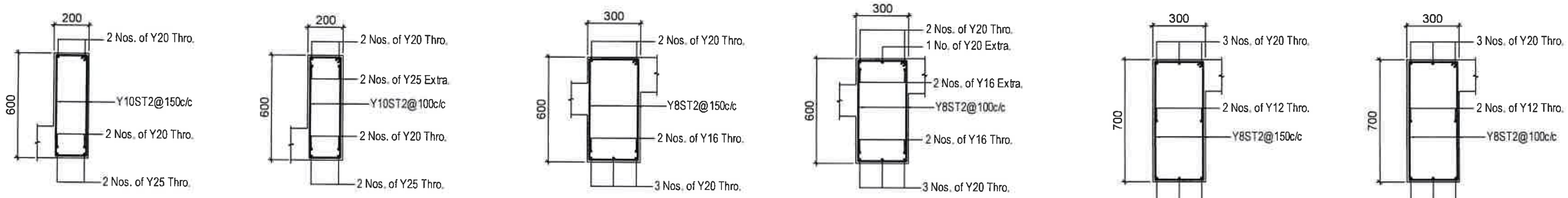
C.S. OF BEAM -"B3"
(200 X 450)

C.S. OF BEAM -"B4"
(200 X 700)
@MID-SPAN

C.S. OF BEAM -"B4"
(200 X 700)
@SUPPORT

C.S. OF BEAM -"B5"
(200 X 700)
@MID-SPAN

C.S. OF BEAM -"B5"
(200 X 700)
@SUPPORT



C.S. OF BEAM -"B6"
(200 X 600)
@MID-SPAN

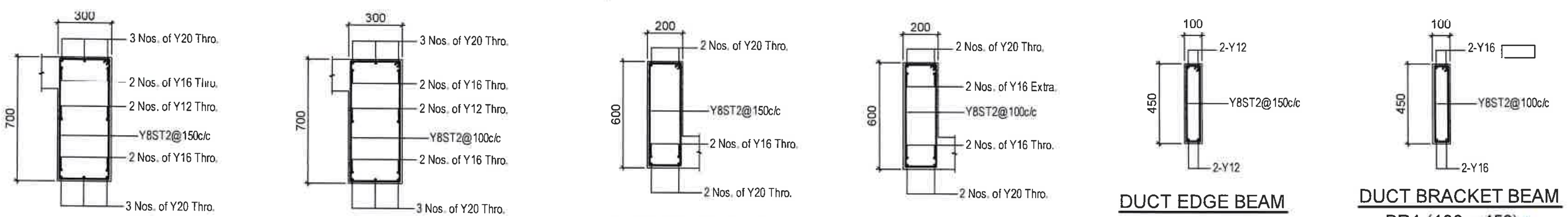
C.S. OF BEAM -"B6"
(200 X 600)
@SUPPORT

C.S. OF BEAM -"B7"
(300 X 600)
@MID-SPAN

C.S. OF BEAM -"B7"
(300 X 600)
@SUPPORT

C.S. OF BEAM -"B8"
(300 X 700)
@MID-SPAN

C.S. OF BEAM -"B8"
(300 X 700)
@SUPPORT



C.S. OF BEAM -"B9"
(300 X 700)
@MID-SPAN

C.S. OF BEAM -"B9"
(300 X 700)
@SUPPORT

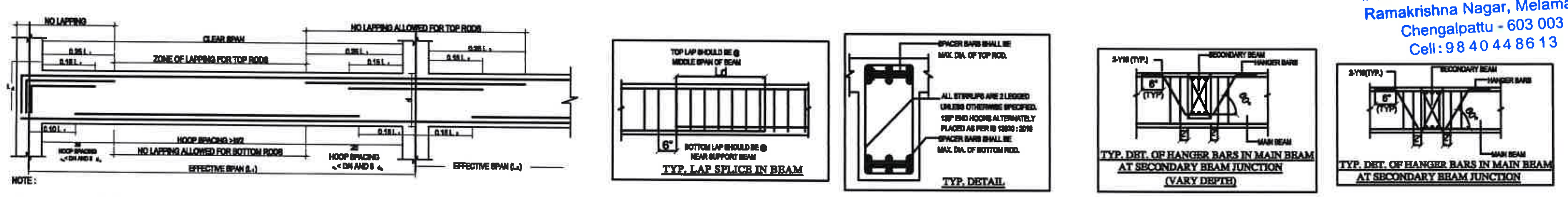
C.S. OF BEAM -"B10"
(200 X 600)
@MID-SPAN

C.S. OF BEAM -"B10"
(200 X 600)
@MID-SPAN

DUCT EDGE BEAM
EB1 (100 X 450)

DUCT BRACKET BEAM
BR1 (100 x 450)

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



NOTE:
0.15 L_1 SHOULD NOT BE LESS THAN L_d
DEVELOPING LENGTH ( ) = 50 L_d d_b
d_b = DIAMETER OF LONGITUDINAL BAR

CURTAILMENT DETAILS FOR BEAMS

TYP. LAP SPLICE IN BEAM

TYP. DETAIL

TYP. DET. OF HANGER BARS IN MAIN BEAM AT SECONDARY BEAM JUNCTION (VARY DEPTH)

TYP. DET. OF HANGER BARS IN MAIN BEAM AT SECONDARY BEAM JUNCTION

GENERAL NOTES:

REMOVAL OF FORM WORK (SHUTTERING):

THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS456 : 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND & BEAMS ---- 24 HOURS.

B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) ---- 3 DAYS.

C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) ---- 7 DAYS.

D. BOTTOM PROPS TO SLAB
i) SPANNING UP TO 4.50M ---- 7 DAYS.
ii) SPANNING OVER 4.50M ---- 14 DAYS.

E. BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M ---- 14 DAYS.
ii) SPANNING OVER 6.00M ---- 21DAYS.

LAP JOINTS FOR REINFORCEMENT BARS:

i). AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED.

ii). LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.3 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS.

iii). DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW.

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M20	M25	M30	M35	M20	M25	M30	M35
8mm	460	400	370	320	370	320	290	260
10mm	585	490	460	400	460	390	370	320
12mm	690	620	550	480	550	470	440	390
16mm	900	780	730	640	730	630	590	520
20mm	1130	970	910	800	910	780	730	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1560	1460	1280	1460	1250	1170	1090

iv). LAPS IN COLUMNS SHALL BE PROVIDED AT MID HEIGHT OF FLOOR AND NOT AT SLAB LEVEL.

v). LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRAFLEXURE.

vi). WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT.

vii). DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920.

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED.
3. ANY DISCREPANCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY.
4. THIS DWG. SHOULD TO BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION).
5. GRADE OF CONCRETE SHALL CONFIRM TO IS 456 : 2000
GRADE OF CONCRETE FOR COLUMN - M30, SLAB & BEAM - M35
6. "Y" DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500D CONFIRMING TO IS 1786 : 2008
7. CLEAR COVER TO THE REINFORCEMENT
A. COLUMNS ---- 40mm
B. ROOF BEAM ---- 30mm
C. ROOF SLAB ---- 25mm
8. DEPTH OF BEAM IS INCLUSIVE OF SLAB THICKNESS

REV	DESCRIPTION	DRN.	CHK.	DATE
01	GOOD FOR CONSTRUCTION	MD	ESK	14.09.21
00	ISSUED FOR APPROVAL	MD	ESK	27.08.21

PROJECT NAME :
PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: R.C BEAM DETAILS OF BASEMENT ROOF

JOB NO.	DWG. NO.	REV. NO. 01	REF. CODE
KA -	S-11	Date 14.09.21	

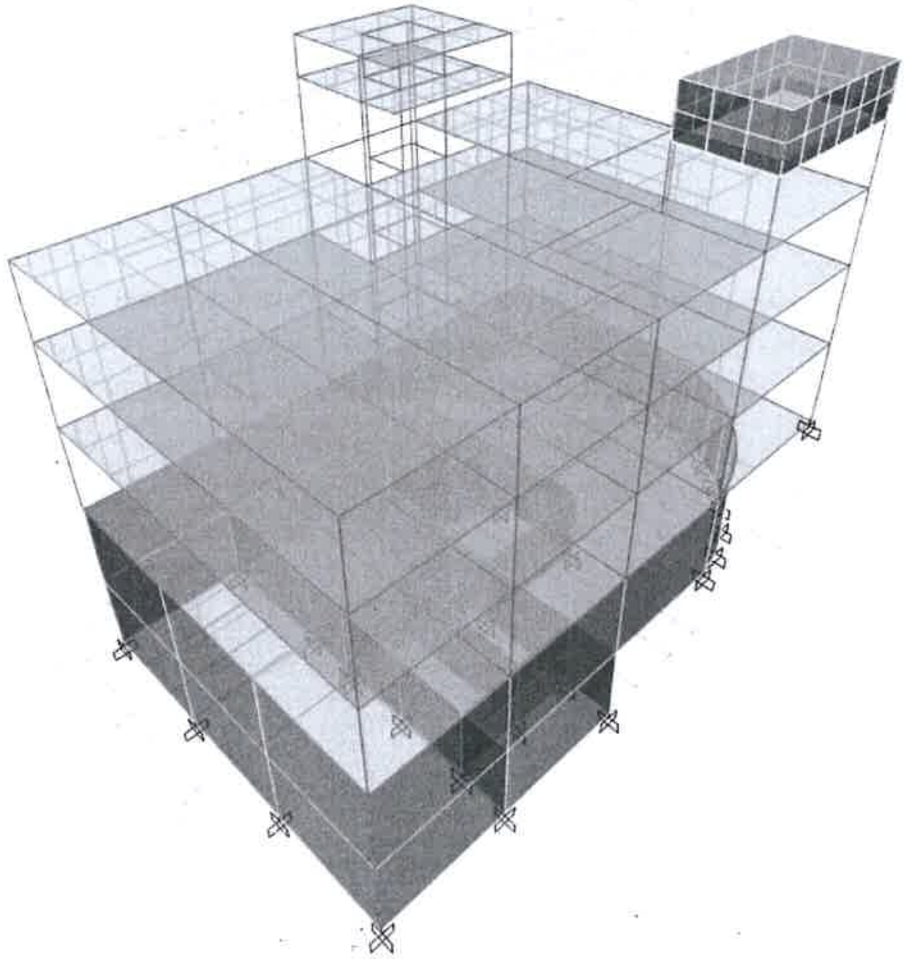
STRUCTURAL CONSULTANTS:

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO.1233, FIRST FLOOR,
FLAT NO:1B, PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob: 98404 48613

ARCHITECTS:
Harismaa Consultant Ltd

Drawn M.D	Designed ES	Checked ES.KUMAR

ETABS®



PROJECT TITLE: THE PROPOSED CONSTRUCTION OF BASEMENT FLOOR (MECHANICAL AUTOMATED CAR PARKING – 2 LEVELS) + GROUND FLOOR & FIRST FLOOR (SHOP) AND SECOND FLOOR (OFFICE) (COMMERCIAL BUILDING) AT PLOT – 663, OLD DOOR NO – 103, NEW DOOR NO – 85, D – BLOCK, SECOND AVENUE & FIRST AVENUE ROAD, ANNA NAGAR EAST, CHENNAI – 600 102, COMPRISED IN OLD S. NO – 2/1B (PART), 7/1 (PART) NEW T.S NO – 3, BLOCK NO – 6, PERIYAKUDAL VILLAGE, AMINJIKARAI TALUK, CHENNAI DISTRICT WITHIN THE LIMIT OF GREATER CHENNAI CORPORATION.ZONE – VIII, DIVISION – 102

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

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

1. INTRODUCTION:

The structure is a "Commercial building" consisting of Basement floor + Ground Floor + 2 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) - 23.90 M

Width (Y -direction) - 16.34 M

1.b Building Height

Story	Clear Height	unit
TOP ROOF LVL	2.50	m
ROOF LVL	3.40	m
2ND SLAB LVL	3.70	m
1ST SLAB LVL	3.70	m
PLINTH LVL	6.63	m
Base	0	N/A

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

**Typical clear height of 3.70m 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. Design mix of M35 shall be used for Columns, PT beams and slab & M30 for pile, pile caps, grade and strap beam and 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	=	B+G+ 2 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	=	M35
14.	SBC as per Soil report	=	Pile 500 & 600 Dia _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 Commercial Building (Basement + Ground + 2 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.

$$V_b = A_h W$$

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

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

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	=	0.16
I for Commercial 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

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

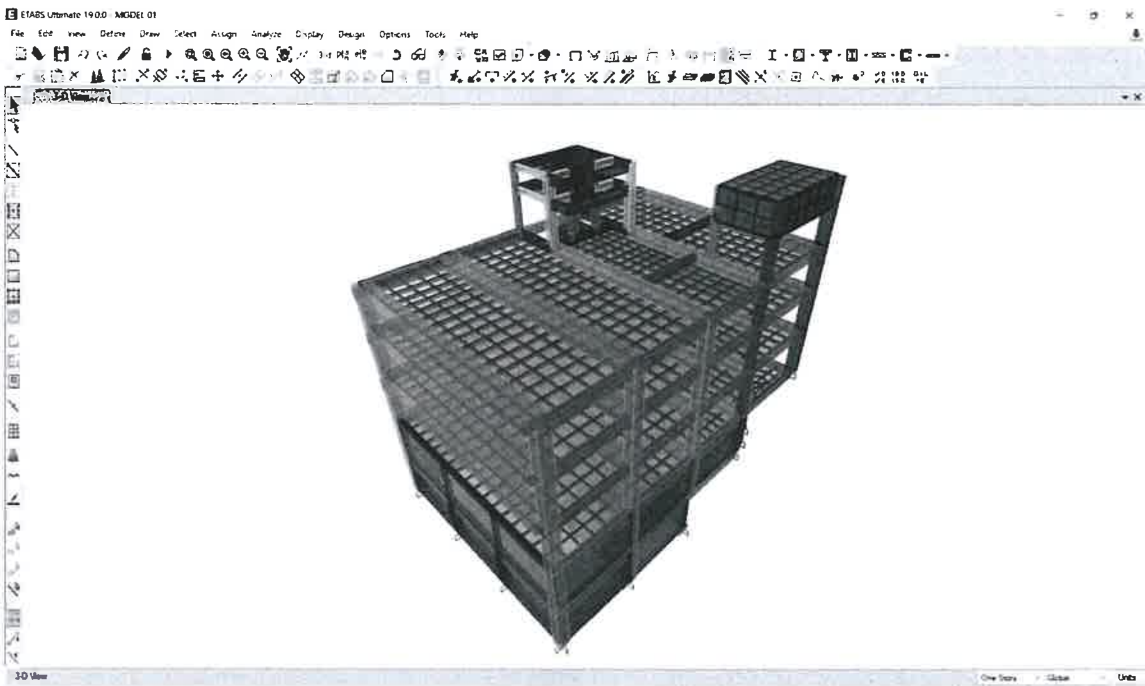
Mob: 98404 48613 eMail Id: kirtiassociates@gmail.com

DESIGN OF STRUCTURE

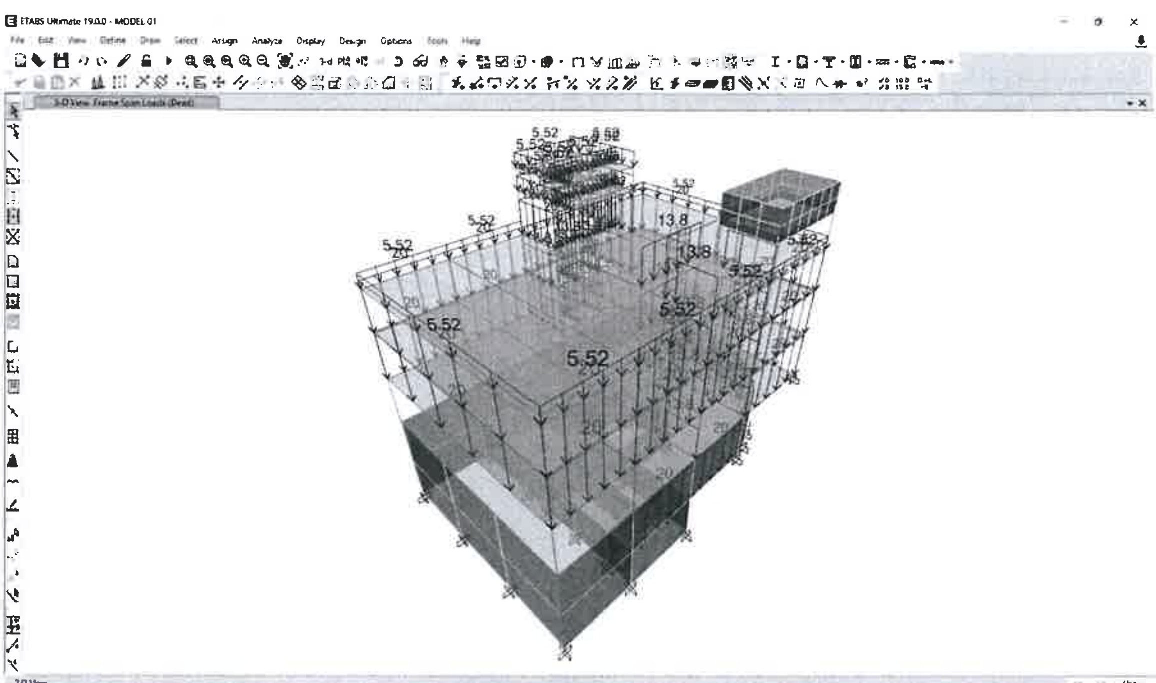
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS

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



3d view – rendered image

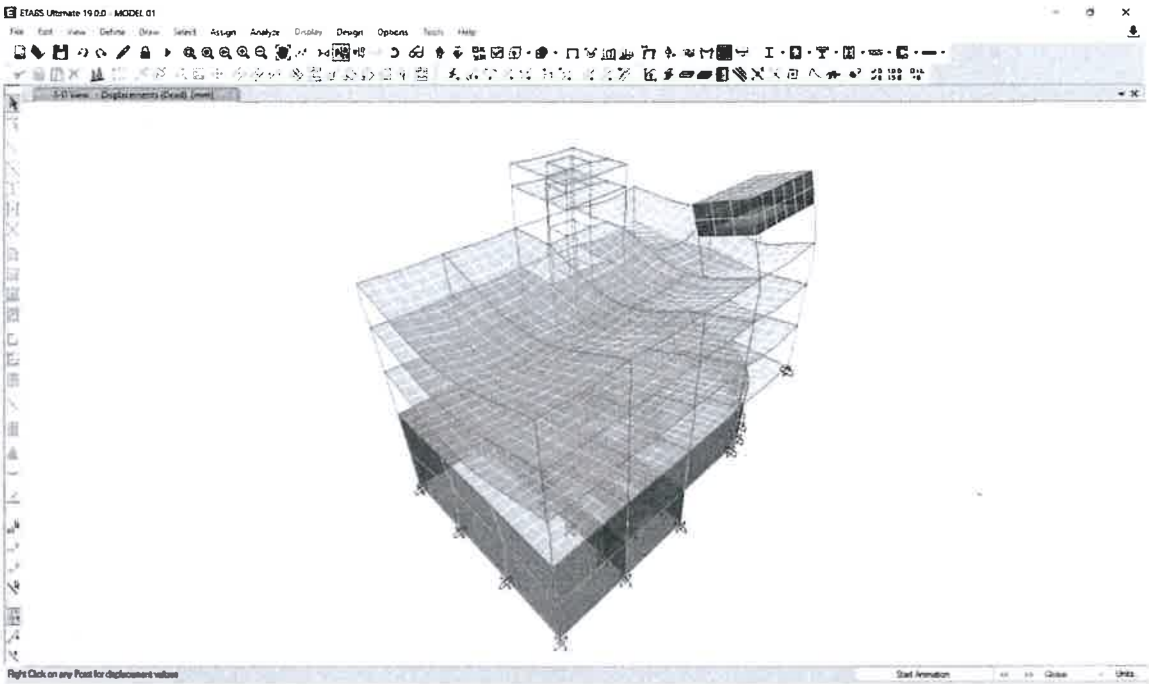


Wall load – dead load

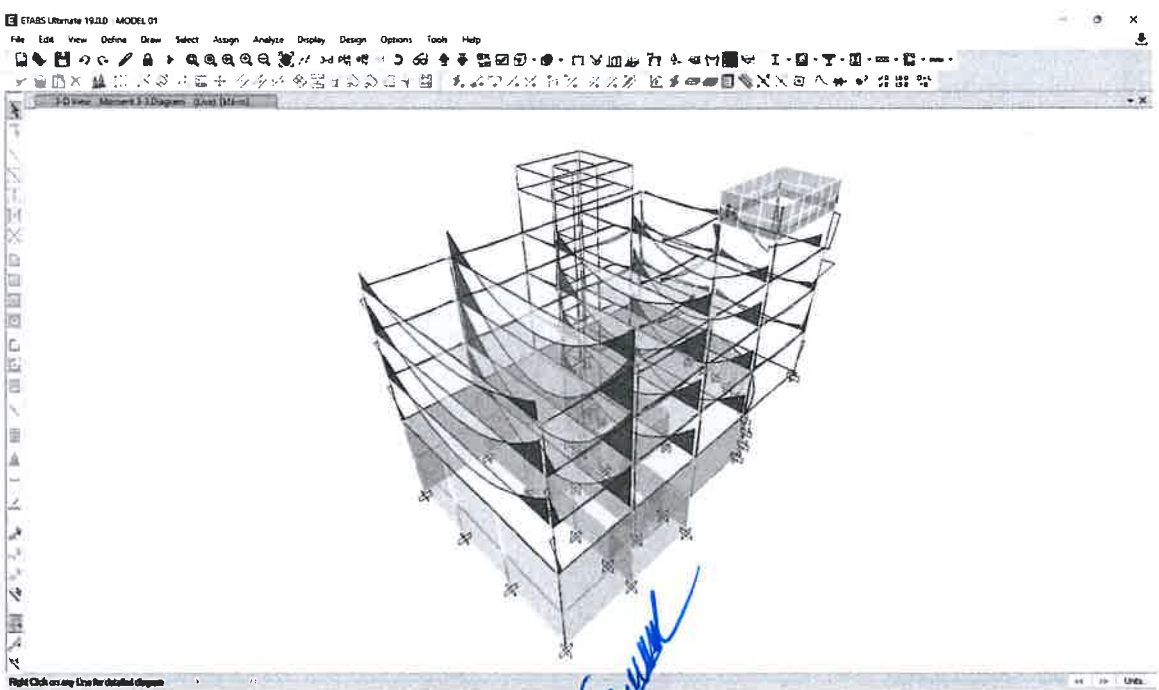
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS

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



Model displacement

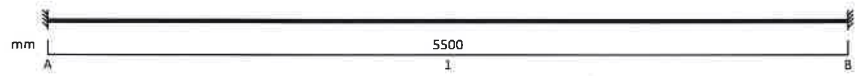


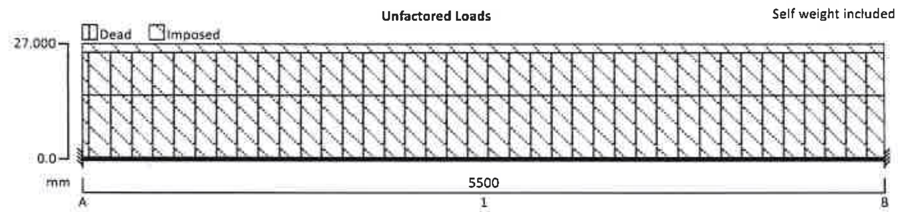
Model moment check

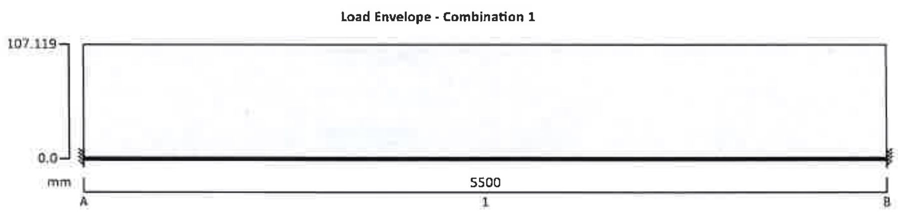
ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 9840448613

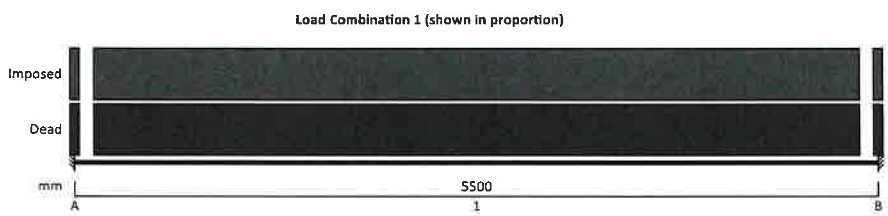
DESIGN CALCULATION

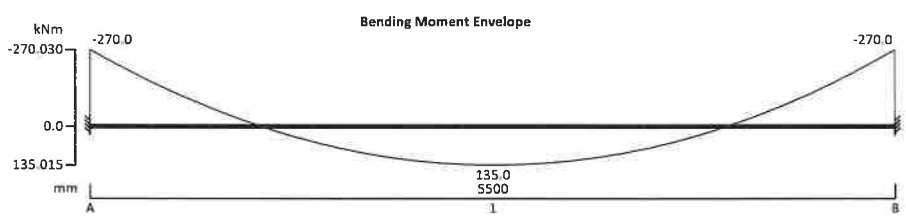
RC BEAM ANALYSIS & DESIGN

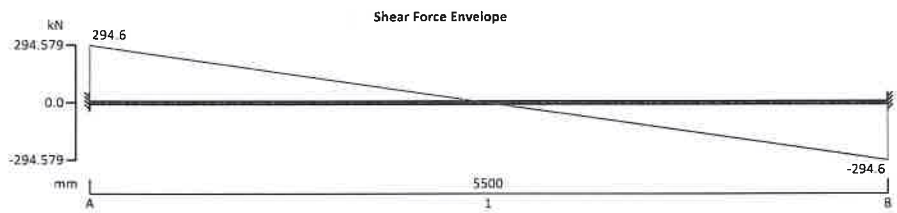












Support conditions

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

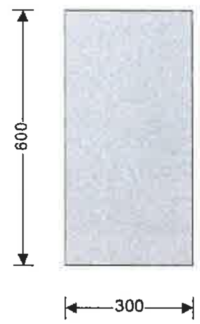
Analysis results

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

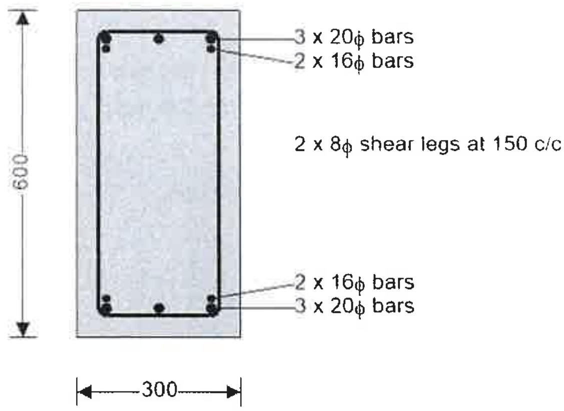
Nominal cover to reinforcement

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

Multiple layers of top reinforcement

Reinforcement provided - layer 1;	3 × 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;	$d_{top} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s,prov}$ = **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; $v_{max} = \min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2)$ = **4.000** N/mm^2

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/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

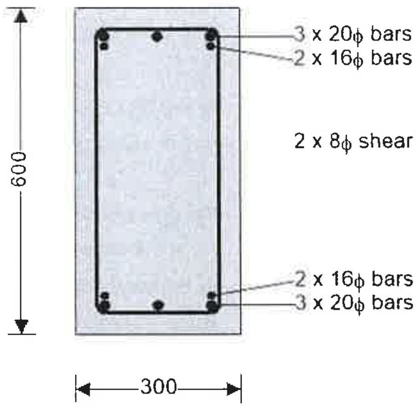
Maximum distance between bars in tension

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; $s_{max} = \min(47000 \text{ N/mm} / f_s, 300 \text{ mm})$ = **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; $A_{sv,min} = 0.4N/mm^2 \times b / (0.87 \times f_{yv}) = \mathbf{276}\ mm^2/m$

PASS - Area of shear reinforcement provided exceeds minimum required

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

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

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

Design shear resistance provided; $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b = \mathbf{0.972}\ N/mm^2$

Design shear stress provided; $v_{prov} = v_{s,prov} + v_c = \mathbf{1.565}\ N/mm^2$

Design shear resistance; $V_{prov} = v_{prov} \times (b \times d) = \mathbf{254.3}$ 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; $s_{min} = h_{agg} + 5\ mm = \mathbf{25}$ mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension

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

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

PASS - Satisfies the maximum spacing criteria

Span to depth ratio

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

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

Modification for tension reinforcement

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

Modification for compression reinforcement

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

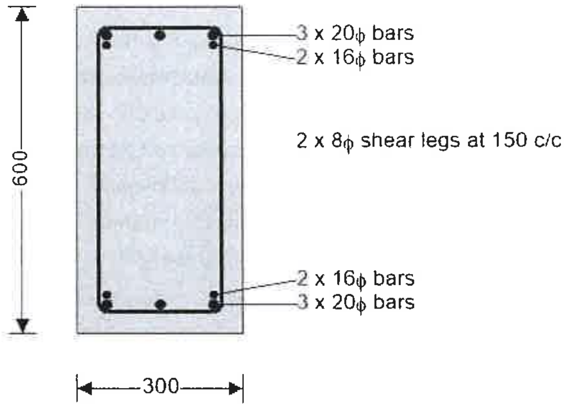
Modification for span length; $f_{long} = 1.000$

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; $span\_to\_depth_{actual} = L_{s1} / d = \mathbf{10.2}$

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

Support B



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1;	3 × 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;	$d_{bot} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s2,prov}$ = **58** mm

Multiple layers of top reinforcement

Reinforcement provided - layer 1;	3 × 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;	$d_{top} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s,prov}$ = **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/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

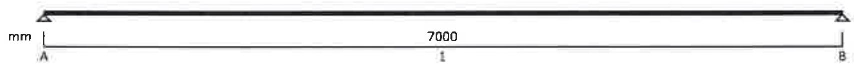
Maximum distance between bars in tension

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

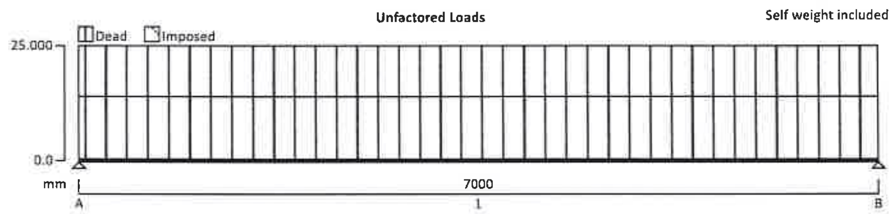
Maximum distance between bars in tension; $s_{max} = \min(47000\ N/mm / f_s, 300\ mm)$ = **140** mm

PASS - Satisfies the maximum spacing criteria

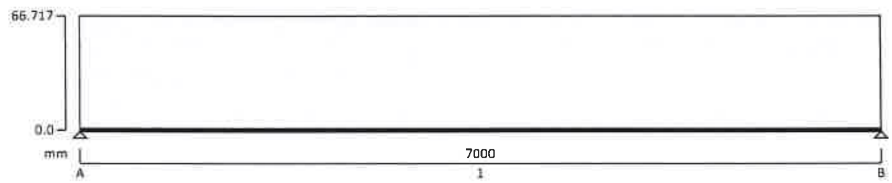
RC BEAM ANALYSIS & DESIGN



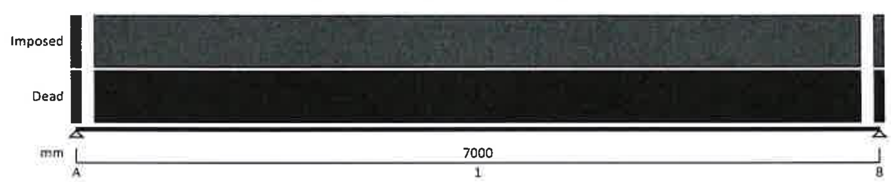
Unfactored Loads



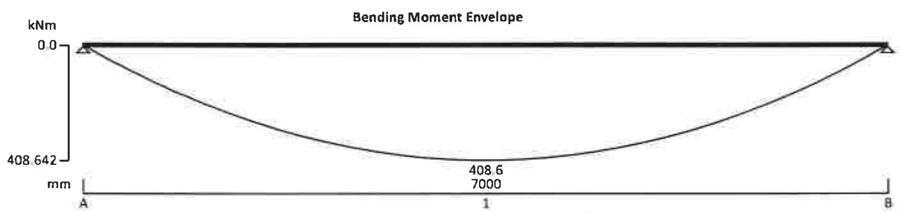
Load Envelope - Combination 1

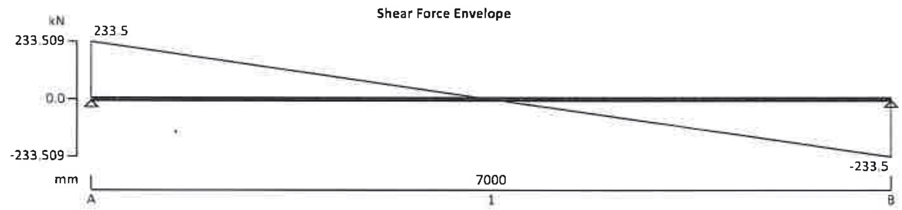


Load Combination 1 (shown in proportion)



Bending Moment Envelope





Support conditions

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

Applied loading

Dead full UDL 25 kN/m

Dead full UDL 14 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} = **0** kNm;	M_{A_red} = **0** kNm; 15%
Maximum moment span 1 at 3500 mm;	M_{s1_max} = **409** kNm;	M_{s1_red} = **409** kNm;
Maximum moment support B;	M_{B_max} = **0** kNm;	M_{B_red} = **0** kNm; 15%
Maximum shear support A;	V_{A_max} = **234** kN;	V_{A_red} = **234** kN
Maximum shear support A span 1 at 548 mm;	$V_{A\_s1\_max}$ = **197** kN;	$V_{A\_s1\_red}$ = **197** kN
Maximum shear support B;	V_{B_max} = **-234** kN;	V_{B_red} = **-234** kN
Maximum shear support B span 1 at 6452 mm;	$V_{B\_s1\_max}$ = **-197** kN;	$V_{B\_s1\_red}$ = **-197** kN
Maximum reaction at support A;	R_A = **234** kN	
Unfactored dead load reaction at support A;	R_{A_Dead} = **156** kN	
Maximum reaction at support B;	R_B = **234** kN	
Unfactored dead load reaction at support B;	R_{B_Dead} = **156** kN	

Rectangular section details

Section width;	b = **380** mm
Section depth;	h = **600** mm

Concrete details

Concrete strength class; **C28/35**

Characteristic compressive cube strength; f_{cu} = **35** N/mm^2

Modulus of elasticity of concrete; E_c = 20kN/mm^2 + 200 × f_{cu} = **27000** N/mm^2

Maximum aggregate size; h_{agg} = **20** mm

Reinforcement details

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

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

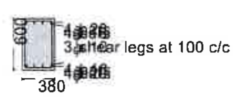
Nominal cover to reinforcement

Nominal cover to top reinforcement; c_{nom_t} = **25** mm

Nominal cover to bottom reinforcement; c_{nom_b} = **25** mm

Nominal cover to side reinforcement; c_{nom_s} = **25** mm

<u>**Support A**</u>



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1; 4 × 20ϕ bars

Area of reinforcement provided - layer 1; A_{s_L1} = **1257** mm^2

Depth to layer 1; d_{L1} = **555** mm

Reinforcement provided - layer 2; 4 × 16ϕ bars

Area of reinforcement provided - layer 2; A_{s_L2} = **804** mm^2

Depth to layer 2; d_{L2} = **537** mm

Total area of reinforcement; $A_{s,prov} = A_{s\_L1} + A_{s\_L2}$ = **2061** mm^2

Centroid of reinforcement; $d_{bot} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s,prov}$ = **548** mm

Multiple layers of top reinforcement

Reinforcement provided - layer 1; 4 × 20ϕ bars

Area of reinforcement provided - layer 1; A_{s_L1} = **1257** mm^2

Depth to layer 1; d_{L1} = **45** mm

Reinforcement provided - layer 2; 4 × 16ϕ bars

Area of reinforcement provided - layer 2; A_{s_L2} = **804** mm^2

Depth to layer 2; d_{L2} = **63** mm

Total area of reinforcement; $A_{s2,prov} = A_{s\_L1} + A_{s\_L2}$ = **2061** mm^2

Centroid of reinforcement; $d_{top} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s2,prov}$ = **52** mm

Rectangular section in shear

Design shear force span 1 at 548 mm; $V = \max(V_{A\_s1\_max}, V_{A\_s1\_red})$ = **197** kN

Design shear stress; $v = V / (b \times d)$ = **0.945** 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 = **0.705** N/mm^2

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

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $0.5 \times v_c < v < (v_c + 0.4 \text{ N/mm}^2)$

Design shear resistance required; $v_s = \max(v - v_c, 0.4 \text{ N/mm}^2)$ = **0.400** N/mm^2

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **349** mm^2/m

Shear reinforcement provided; 3 × 10ϕ legs at 100 c/c

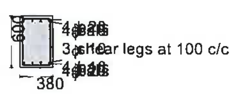
Area of shear reinforcement provided; $A_{sv,prov}$ = **2356** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d$ = **411** mm

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

<u>Mid span 1</u>



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1; 4 × 20ϕ bars

Area of reinforcement provided - layer 1; A_{s_L1} = **1257** mm^2

Depth to layer 1; d_{L1} = **555** mm

Reinforcement provided - layer 2; 4 × 16ϕ bars

Area of reinforcement provided - layer 2; A_{s_L2} = **804** mm^2

Depth to layer 2; d_{L2} = **537** mm

Total area of reinforcement; $A_{s,prov} = A_{s\_L1} + A_{s\_L2}$ = **2061** mm^2

Centroid of reinforcement; $d_{bot} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s,prov}$ = **548** mm

Multiple layers of top reinforcement

Reinforcement provided - layer 1; 4 × 20ϕ bars

Area of reinforcement provided - layer 1; A_{s_L1} = **1257** mm^2

Depth to layer 1; d_{L1} = **45** mm

Reinforcement provided - layer 2; 4 × 16ϕ bars

Area of reinforcement provided - layer 2; A_{s_L2} = **804** mm^2

Depth to layer 2; d_{L2} = **63** mm

Total area of reinforcement; $A_{s2,prov} = A_{s\_L1} + A_{s\_L2} =$ **2061** mm^2

Centroid of reinforcement; $d_{top} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s2,prov} =$ **52** mm

Design moment resistance of rectangular section - Positive moment

Design bending moment; $M = abs(M_{s1\_red}) =$ **409** kNm

Depth to tension reinforcement; $d = d_{bot} =$ **548** mm

Redistribution ratio; $\beta_b = min(1 - m_{rs1}, 1) =$ **1.000**

$K = M / (b \times d^2 \times f_{cu}) =$ **0.102**

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm; $z = min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) =$ **476**

mm

Depth of neutral axis; $x = (d - z) / 0.45 =$ **159** mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z) =$ **1972** mm^2

Tension reinforcement provided; 4×20 ϕbars + 4×16 ϕbars

Area of tension reinforcement provided; $A_{s,prov} =$ **2061** mm^2

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h =$ **296** mm^2

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h =$ **9120** mm^2

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

Rectangular section in shear

Shear reinforcement provided; $3 \times 10\phi$ legs at 100 c/c

Area of shear reinforcement provided; $A_{sv,prov} =$ **2356** mm^2/m

Minimum area of shear reinforcement (Table 3.7); $A_{sv,min} = 0.4N/mm^2 \times b / (0.87 \times f_{yv}) =$ **349** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing $s_{vl,max} = 0.75 \times d =$ **411** mm

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

Design concrete shear stress; $v_c = 0.79N/mm^2 \times min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times max(1, (400mm /d)^{1/4}) \times (min(f_{cu}, 40N/mm^2) / 25N/mm^2)^{1/3} / \gamma_m =$ **0.705** N/mm^2

Design shear resistance provided; $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b =$ **2.697** N/mm^2

Design shear stress provided; $v_{prov} = v_{s,prov} + v_c =$ **3.402** N/mm^2

Design shear resistance; $V_{prov} = v_{prov} \times (b \times d) =$ **708.4** kN

Shear links provided valid between 0 mm and 7000 mm with tension reinforcement of 2061 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; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) =$ **319.0** N/mm^2

Maximum distance between bars in tension; $s_{max} = min(47000\ N/mm / f_s, 300\ mm) =$ **147** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio

Basic span to depth ratio (Table 3.9); $span\_to\_depth_{basic} =$ **20.0**

Design service stress in tension reinforcement; $f_s = (2 \times f_y \times A_{s,req})/ (3 \times A_{s,prov} \times \beta_b) =$ **319.0** N/mm^2

Modification for tension reinforcement

$f_{tens} = min(2.0, 0.55 + (477N/mm^2 - f_s) / (120 \times (0.9N/mm^2 + (M / (b \times d^2))))) =$ **0.844**

Modification for compression reinforcement

f_{comp} = min(1.5, 1 + (100 × $A_{s2,prov}$ / (b × d)) / (3 + (100 × $A_{s2,prov}$ / (b × d)))) = **1.248**

Modification for span length; f_{long} = 1.000

Allowable span to depth ratio; $span\_to\_depth_{allow}$ = $span\_to\_depth_{basic}$ × f_{tens} × f_{comp} = **21.1**

Actual span to depth ratio; $span\_to\_depth_{actual}$ = L_{s1} / d = **12.8**

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

<u>Support B</u>

Multiple layers of bottom reinforcement

Reinforcement provided - layer 1; 4 × 20ϕ bars

Area of reinforcement provided - layer 1; A_{s_L1} = **1257** mm^2

Depth to layer 1; d_{L1} = **555** mm

Reinforcement provided - layer 2; 4 × 16ϕ bars

Area of reinforcement provided - layer 2; A_{s_L2} = **804** mm^2

Depth to layer 2; d_{L2} = **537** mm

Total area of reinforcement; $A_{s,prov}$ = A_{s_L1} + A_{s_L2} = **2061** mm^2

Centroid of reinforcement; d_{bot} = (A_{s_L1} × d_{L1} + A_{s_L2} × d_{L2}) / $A_{s,prov}$ = **548** mm

Multiple layers of top reinforcement

Reinforcement provided - layer 1; 4 × 16ϕ bars

Area of reinforcement provided - layer 1; A_{s_L1} = **804** mm^2

Depth to layer 1; d_{L1} = **43** mm

Reinforcement provided - layer 2; 4 × 16ϕ bars

Area of reinforcement provided - layer 2; A_{s_L2} = **804** mm^2

Depth to layer 2; d_{L2} = **59** mm

Total area of reinforcement; $A_{s2,prov}$ = A_{s_L1} + A_{s_L2} = **1608** mm^2

Centroid of reinforcement; d_{top} = (A_{s_L1} × d_{L1} + A_{s_L2} × d_{L2}) / $A_{s2,prov}$ = **51** mm

Rectangular section in shear

Design shear force span 1 at 6452 mm; V = abs(min($V_{B\_s1\_max}$, $V_{B\_s1\_red}$)) = **197** kN

Design shear stress; v = V / (b × d) = **0.945** N/mm^2

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

v_c = **0.705** N/mm^2

Allowable design shear stress; v_{max} = min(0.8 N/mm^2 × (f_{cu}/1 N/mm^2)$^{0.5}$, 5 N/mm^2) = **4.733** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; 0.5 × v_c < v < (v_c + 0.4 N/mm^2)

Design shear resistance required; v_s = max(v - v_c, 0.4 N/mm^2) = **0.400** N/mm^2

Area of shear reinforcement required; $A_{sv,req}$ = v_s × b / (0.87 × f_{yv}) = **349** mm^2/m

Shear reinforcement provided; $3 \times 10\phi$ legs at 100 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **2356** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d$ = **411** mm

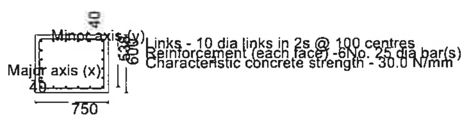
PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

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 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 = **750** mm
Nominal cover to all reinforcement (shorter dim);	c_b = **40** mm
Depth to tension steel;	b' = b - c_b - L_{dia} – $D_{col}/2$ = **688** 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 × b = **45000** mm
Slenderness limit;	l_{limit1} = 100 × b^2 / h = **93750** 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} = β_x × 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} = β_y × l_o = **4.200** m
Slenderness check;	l_{ey}/b = **5.60**

The unbraced column is short (minor axis)

Short column - 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} = **6**

Area of "tension" steel; $A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4$ = **2945** mm^2

Area of compression steel; $A_{sc} = A_{st}$ = **2945** mm^2

Total area of steel ; $A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2))$ = **9817.5** mm^2

Percentage of steel; $A_{scol} / (b \times h)$ = **2.2** %

Design ultimate loading

Design ultimate axial load; N = **2880** kN

Design ultimate moment (major axis); M_x = **600** kNm

Design ultimate moment (minor axis); M_y = **600** kNm

Minimum design moments

Min design moment (major axis); M_{xmin} = min(0.05 × h, 20 mm) × N = **57.6** kNm

Min design moment (minor axis); M_{ymin} = min(0.05 × b, 20 mm) × N = **57.6** kNm

Design moments

Design moment (major axis); M_{xdes} = max (abs(M_x), M_{xmin}) = **600.0** kNm

Design moment (minor axis); M_{ydes} = max (abs(M_y) , M_{ymin}) = **600.0** kNm

Simplified method for dealing with bi-axial bending:-

h' = **538** mm; b' = **688** mm

Approx uniaxial design moment

$\beta = 1 - 1.165 \times \min(0.6, N/(b \times h \times f_{cu}))$ = **0.75**

Design moment;

M_{design} = if(M_{xdes}/h' < M_{ydes}/b' , $M_{ydes} + \beta \times$ b'/h' × M_{xdes} , $M_{xdes} + \beta \times$ h'/b' × M_{ydes}) = **952.5** kNm

Set up section dimensions for design:-

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

Depth to "tension" steel; d = if(M_{xdes}/h' < M_{ydes}/b' , b', h') = **537.5** mm

Section width; B = if(M_{xdes}/h' < M_{ydes}/b' , h, b) = **750.0** mm

Library item - Calcs – short col N+Mmaj+Mmin

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 = **750** mm

Section depth perpendicular to axis of bending); D = **600** mm

Depth to "tension" steel (symmetrical); d = **538** mm

Tension and compression steel yield

Determine correct moment of resistance

N_R =ceiling(if(x_{calc}<D/0.9, N_{R1} , N_{R2}),0.001kN) = **2880.0** kN

M_R = ceiling(if(x_{calc}<D/0.9, M_{R1} , M_{R2}) ,0.001kNm) = **1059.6** kNm

Applied axial load; N = **2880.0** kN

Check for moment; M_{design} = **952.5** kNm

Moment check satisfiedMoment capacity with no axial load

Check min and max areas of steel

Total area of concrete; $A_{conc} = b \times h =$ **450000** mm^2

Area of steel (symmetrical); $A_{scol} =$ **9817** mm^2

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

Minimum steel area; $A_{sc\_min} = k_c \times A_{conc} =$ **3600** mm^2

Maximum steel area; $A_{s\_max} = 6\ \% \times A_{conc} =$ **27000** mm^2

Area of compression steel provided OK

Major axis Shear Resistance of Concrete Columns

Column width; b = **750** mm

Column depth; h = **600** mm

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

Area of concrete; $A_{conc} = b \times h =$ **450000** 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 = **2880.0** kN

Major axis moment; $M_x =$ **600.0** kNm

Eccentricity; $e = M_x / N =$ **208.3** mm

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

Actual shear stress; $v_x = V_x / (b \times h') =$ **0.2** 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; $L_n =$ **2**

Minimum Containment Stee

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 = **2880.0** kN

Column dimensions

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

Column width (smaller column dimension); b = **750** mm

Column area; $A_c = h \times b =$ **450000** mm^2

Limit for crack check; $N_{limit} = 0.2 \times f_{cu} \times A_c =$ **2700.0** 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; $Mx_{\_SLS}$ = **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 = **750** 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} = **6**
Area of tension reinforcement;	$A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol}$ = **2945** 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;	$bar_{crs} = (b - 2\times(c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1)$ = **125.0** mm

Modular Ratio

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

Neutral Axis position

For equilibrium; F_{st} equates F_c

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

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

$x = h' \times [-1\times E_s\times A_{st}/(E_c\times b\times h') + \sqrt{(E_s\times A_{st}/(E_c\times b\times h') \times (2+E_s\times A_{st}/(E_c\times b\times h')))}]$ = **201.5** mm

Depth of concrete in compression; x = **201.5** mm

Concrete and Steel stresses

The serviceability limit state moment; $Mx_{\_SLS}$ = **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;	$f_c = 2 \times Mx_{\_SLS} / (b \times x \times (h' - x/3))$ = **1.41** 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; $f_{st} = Mx_{\_SLS} / (A_{st} \times (h' - x/3))$ = **36.09** N/mm^2

Concrete and Steel strains

Strain in the reinforcement;	$\varepsilon_s = f_{st} / E_s$ = **180.5×10^{-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; $a' = h = \mathbf{600}$ mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x) = \mathbf{214.0\times10^{-6}}$

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

Allowable crack width; $Crack_{Allowable} = \mathbf{0.2}$ mm

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

Breadth of tension face; $b_t = b = \mathbf{750}$ mm

$$\varepsilon_m = \min(\varepsilon_1, \max(0, \varepsilon_1 - [\text{factor} \times b_t \times (h - x) \times (a' - x) / (3\times E_s\times A_{st} \times(h' - x))])) = \mathbf{13.45\times10^{-6}}$$

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

$a_{cr1} = \sqrt{((bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2 = \mathbf{75.9}$ mm

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

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

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

Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2\times(a_{cr} - c_{ten})/(h - x)) = \mathbf{0.003}$ mm

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

Serviceability Limit State - Cracking in Columns

Bent about the minor axis

Design serviceability moment about the minor axis; $M_{Y\_SLS} = \mathbf{50}$ kNm

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension); $h = \mathbf{600}$ mm

Column width (smaller column dimension); $b = \mathbf{750}$ mm

Depth to steel; $b' = \mathbf{688}$ mm

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

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

Diameter of links; $L_{dia} = \mathbf{10}$ mm

Diameter of tension reinforcement; $D_{col} = \mathbf{25}$ mm

Number of tension reinforcement bars; $L_{ncol} = \mathbf{6}$

Area of tension reinforcement; $A_{st} = \pi \times D_{col}^2 /4 \times L_{ncol} = \mathbf{2945}$ mm^2

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

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

Effective depth to tension reinforcement; $b' = \mathbf{687.5}$ mm

Tension bar centres; $bar_{crs} = (h - 2\times(c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) = \mathbf{95.0}$ mm

Modular Ratio

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

Modulus of elasticity for conc (half instanteneous); $E_c = ((20 \text{ kN/mm}^2) + 200\times f_{cu}) / 2 = \mathbf{13}$ kN/mm^2

Modular ratio; $m = E_s / E_c = \mathbf{15.385}$

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:-

$$x = b' \times [-1\times E_s\times A_{st}/(E_c\times h\times b') + \sqrt{(E_s\times A_{st}/(E_c\times h\times b') \times (2+E_s\times A_{st}/(E_c\times h\times b')))}] = \mathbf{255.5} \text{ mm}$$

Depth of concrete in compression; $x = \mathbf{255.5}$ 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; $f_c = 2 \times M_{Y\_SLS} / (h \times x \times (b' - x/3))$ = **1.08** 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; $f_{st} = M_{Y\_SLS} / (A_{st} \times (b' - x/3))$ = **28.18** N/mm^2

Concrete and Steel strains

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s$ = **140.9×10^{-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 = **750** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(b' - x)$ = **161.3×10^{-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

$\varepsilon_m = \min(\varepsilon_1 , \max(0, \varepsilon_1 - [factor \times h_t \times (b - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (b' - x))]))$ = **0.0000**

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

$a_{cr1} = \sqrt{ ((bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2$ = **66.0** 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

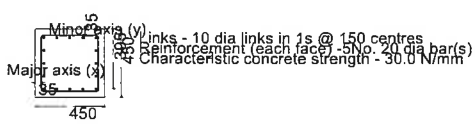
Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2 \times (a_{cr} - c_{ten})/(b - x))$ = **0.000** mm

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

RC COLUMN DESIGN

Column definition

Column depth (larger column dim);	h = **450** mm
Nominal cover to all reinforcement (longer dim);	c_h = **35** mm
Depth to tension steel;	$h' = h - c_h - L_{dia} - D_{col}/2$ = **395** mm
Column width (smaller column dim);	b = **450** mm
Nominal cover to all reinforcement (shorter dim);	c_b = **35** mm
Depth to tension steel;	$b' = b - c_b - L_{dia} - D_{col}/2$ = **395** mm
Characteristic strength of reinforcement;	f_y = **500** N/mm^2
Characteristic strength of concrete;	f_{cu} = **30** N/mm^2



Braced Column Design

Check on overall column dimensions

Column OK - h < 4b

Braced column slenderness check

Column clear height;	l_o = **3500** mm
Slenderness limit;	$l_{limit} = 60 \times b$ = **27000** mm

Column slenderness limit OK

Short column check for braced columns

Column clear height;	l_o = **3500** mm
Effect. height factor for braced columns - maj axis;	β_x = **0.75**
Effective height – major axis;	$l_{ex} = \beta_x \times l_o$ = **2.625** m;
Slenderness check;	l_{ex}/h = **5.83**

The braced column is short (major axis)

Effect height factor for braced columns - minor axis;	β_y = **0.75**
Effective height – minor axis;	$l_{ey} = \beta_y \times l_o$ = **2.625** m
Slenderness check;	l_{ey}/b = **5.83**

The braced column is short (minor axis)

Short column - bi-axial bending

Define column reinforcement

Main reinforcement in column

Assumed diameter of main reinforcement; D_{col} = **20** mm

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

Area of "tension" steel; $A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4$ = **1571** mm^2

Area of compression steel; $A_{sc} = A_{st}$ = **1571** mm^2

Total area of steel ; $A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2))$ = **5026.5** mm^2

Percentage of steel; $A_{scol} / (b \times h)$ = **2.5** %

Design ultimate loading

Design ultimate axial load; N = **850** kN

Design ultimate moment (major axis); M_x = **11** kNm

Design ultimate moment (minor axis); M_y = **34** kNm

Minimum design moments

Min design moment (major axis); $M_{xmin} = \min(0.05 \times h, 20\text{ mm}) \times N$ = **17.0** kNm

Min design moment (minor axis); $M_{ymin} = \min(0.05 \times b, 20\text{ mm}) \times N$ = **17.0** kNm

Design moments

Design moment (major axis); $M_{xdes} = \max(\text{abs}(M_x), M_{xmin})$ = **17.0** kNm

Design moment (minor axis); $M_{ydes} = \max(\text{abs}(M_y), M_{ymin})$ = **34.0** kNm

Simplified method for dealing with bi-axial bending:-

h' = **395** mm; b' = **395** mm

Approx uniaxial design moment (Cl 3.8.4.5)

$\beta = 1 - 1.165 \times \min(0.6, N/(b \times h \times f_{cu}))$ = **0.84**

Design moment;

$M_{design} = \text{if}(M_{xdes}/h' < M_{ydes}/b', M_{ydes} + \beta \times b'/h' \times M_{xdes}, M_{xdes} + \beta \times h'/b' \times M_{ydes})$ = **48.2** kNm

Set up section dimensions for design:-

Section depth; $D = \text{if}(M_{xdes}/h' < M_{ydes}/b', b, h)$ = **450.0** mm

Depth to "tension" steel; $d = \text{if}(M_{xdes}/h' < M_{ydes}/b', b', h')$ = **395.0** mm

Section width; $B = \text{if}(M_{xdes}/h' < M_{ydes}/b', h, b)$ = **450.0** mm

Library item - Calcs – short col N+Mmaj+Mmin

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 = **450** mm

Section depth perpendicular to axis of bending); D = **450** mm

Depth to "tension" steel (symmetrical); d = **395** mm

Tension and compression steel yield

Determine correct moment of resistance

$N_R = \text{ceiling}(\text{if}(x_{calc} < D/0.9, N_{R1}, N_{R2}), 0.001\text{kN})$ = **850.0** kN

$M_R = \text{ceiling}(\text{if}(x_{calc} < D/0.9, M_{R1}, M_{R2}), 0.001\text{kNm})$ = **363.5** kNm

Applied axial load; N = **850.0** kN

Check for moment; M_{design} = **48.2** kNm

Moment check satisfied

Check min and max areas of steel

Total area of concrete; $A_{conc} = b \times h =$ **202500** mm^2

Area of steel (symmetrical); $A_{scol} =$ **5027** mm^2

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

Minimum steel area; $A_{sc\_min} = k_c \times A_{conc} =$ **1620** mm^2

Maximum steel area; $A_{s\_max} = 6\ \% \times A_{conc} =$ **12150** mm^2

Area of compression steel provided OK

Major axis Shear Resistance of Concrete Columns

Column width; b = **450** mm

Column depth; h = **450** mm

Effective depth to steel; h' = **395** mm

Area of concrete; $A_{conc} = b \times h =$ **202500** mm^2

Design ultimate shear force (major axis); $V_x =$ **55** kN

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

Axial load; N = **850.0** kN

Major axis moment; $M_x =$ **11.0** kNm

Eccentricity; $e = M_x / N =$ **12.9** mm

Limit to eccentricity; $e_{limit} = 0.6 \times h =$ **270.0** 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; $s_v =$ **150** mm; Link diameter; $L_{dia} =$ **10** mm; No of links in each group; $L_n =$ **1**

Minimum Containment Steel

Shear steel

Link spacing; $s_v =$ **150** mm

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

Column steel

Diameter; $D_{col} =$ **20** mm

Min diameter; $L_{limit} = \max(\ (6\ mm), D_{col}/4) =$ **6.0** mm

Link diameter OK

Max spacing; $s_{limit} = 12 \times D_{col} =$ **240.0** mm

Link spacing OK

Crack Control in columns

Column design ultimate axial load; N = **850.0** kN

Column dimensions

Column depth (larger column dimension); h = **450** mm

Column width (smaller column dimension); b = **450** mm

Column area; $A_c = h \times b =$ **202500** mm^2

Limit for crack check; $N_{limit} = 0.2 \times f_{cu} \times A_c =$ **1215.0** kN

Column should be checked as a beam for cracking

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} = **23** kNm

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension);	h = **450** mm
Depth to steel;	h' = **395** mm
Column width (smaller column dimension);	b = **450** 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} = **20** mm
Number of tension reinforcement bars;	L_{ncol} = **5**
Area of tension reinforcement;	$A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol}$ = **1571** mm^2
Nominal cover to reinforcement;	$c_{nom} = h - h' - D_{col}/2 - L_{dia}$ = **35** mm
Cover to tension reinforcement;	$c_{ten} = c_{nom} + L_{dia}$ = **45.0** mm
Effective depth to tension reinforcement;	h' = **395.0** mm
Tension bar centres;	$bar_{crs} = (b - 2\times(c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1)$ = **85.0** mm

Modular Ratio

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

Neutral Axis position

For equilibrium; F_{st} equates F_c

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

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

$x = h' \times [-1\times E_s\times A_{st}/(E_c\times b\times h') + \sqrt{(E_s\times A_{st}/(E_c\times b\times h') \times (2+E_s\times A_{st}/(E_c\times b\times h')))}]$ = **159.2** mm

Depth of concrete in compression; x = **159.2** mm

Concrete and Steel stresses

The serviceability limit state moment; M_{X_SLS} = **23** 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; $f_c = 2 \times M_{X\_SLS} / (b \times x \times (h' - x/3))$ = **1.88** 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; $f_{st} = M_{X\_SLS} / (A_{st} \times (h' - x/3))$ = **42.82** N/mm^2

Concrete and Steel strains

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s$ = **214.1×10^{-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 = **450** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x) = \mathbf{264.0 \times 10^{-6}}$

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; $b_t = b$ = **450** mm

$\varepsilon_m = \min(\varepsilon_1, \max(0, \varepsilon_1 - [\text{factor} \times b_t \times (h - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (h' - x))])) = \mathbf{92.78 \times 10^{-6}}$

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

$a_{cr1} = \sqrt{((bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2$ = **59.5** mm

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

$a_{cr2} = \sqrt{(2)} \times (c_{nom} + L_{dia} + D_{col}/2) - D_{col}/2$ = **67.8** mm

Critical distance from tension bar; $a_{cr} = \max(a_{cr1}, a_{cr2})$ = **67.8** mm

Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m / (1 + 2 \times (a_{cr} - c_{ten})/(h - x))$ = **0.016** 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} = **21** kNm

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension);	h = **450** mm
Column width (smaller column dimension);	b = **450** mm
Depth to steel;	b' = **395** 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} = **20** mm
Number of tension reinforcement bars;	L_{ncol} = **5**
Area of tension reinforcement;	$A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol}$ = **1571** mm^2
Nominal cover to reinforcement;	$c_{nom} = b - b' - D_{col}/2 - L_{dia}$ = **35** mm
Cover to tension reinforcement;	$c_{ten} = c_{nom} + L_{dia}$ = **45.0** mm
Effective depth to tension reinforcement;	b' = **395.0** mm
Tension bar centres;	$bar_{crs} = (h - 2 \times (c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1)$ = **85.0** mm

Modular Ratio

Modulus of elasticity for reinforcement;	E_s = 200 kN/mm^2
Modulus of elasticity for conc (half instanteneous);	$E_c = ((20 \text{ kN/mm}^2) + 200 \times f_{cu}) / 2$ = **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:-

$x = b' \times [-1 \times E_s \times A_{st}/(E_c \times h \times b') + \sqrt{(E_s \times A_{st}/(E_c \times h \times b') \times (2 + E_s \times A_{st}/(E_c \times h \times b')))}]$ = **159.2** mm

Depth of concrete in compression; x = **159.2** mm

Concrete and Steel stresses

The serviceability limit state moment; M_{Y_SLS} = **21** 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; $f_c = 2 \times M_{Y\_SLS} / (h \times x \times (b' - x/3))$ = **1.71** 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; $f_{st} = M_{Y\_SLS} / (A_{st} \times (b' - x/3))$ = **39.10** N/mm^2

Concrete and Steel strains

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s$ = **195.5×10^{-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; $a' = b$ = **450** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(b' - x)$ = **241.1×10^{-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; $h_t = h$ = **450** mm

$\varepsilon_m = \min(\varepsilon_1, \max(0, \varepsilon_1 - [\text{factor} \times h_t \times (b - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (b' - x))]))$ = **69.82×10^{-6}**

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

$a_{cr1} = \sqrt{(bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2} - D_{col}/2$ = **59.5**

mm

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

$a_{cr2} = \sqrt{2} \times (c_{nom} + L_{dia} + D_{col}/2) - D_{col}/2$ = **67.8** mm

Critical distance from tension bar; $a_{cr} = \max(a_{cr1}, a_{cr2})$ = **67.8** mm

Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m / (1 + 2 \times (a_{cr} - c_{ten})/(b - x))$ = **0.012**

mm

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

RC COLUMN DESIGN

Column definition

Column depth (larger column dim); $h = \mathbf{450}$ mm

Nominal cover to all reinforcement (longer dim); $c_h = \mathbf{40}$ mm

Depth to tension steel; $h' = h - c_h - L_{dia} - D_{col}/2 = \mathbf{390}$ mm

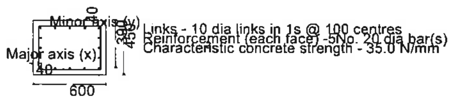
Column width (smaller column dim); $b = \mathbf{600}$ mm

Nominal cover to all reinforcement (shorter dim); $c_b = \mathbf{40}$ mm

Depth to tension steel; $b' = b - c_b - L_{dia} - D_{col}/2 = \mathbf{540}$ mm

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

Characteristic strength of concrete; $f_{cu} = \mathbf{35}$ N/mm^2



Unbraced Column Design

Check on overall column dimensions

Column OK - h < 4b

Unbraced column slenderness check

Unbraced column clear height; $l_o = \mathbf{3500}$ mm

Slenderness limit; $l_{limit} = 60 \times b = \mathbf{36000}$ mm

Slenderness limit; $l_{limit1} = 100 \times b^2 / h = \mathbf{80000}$ mm

Column slenderness limit OK

If column is unrestrained, slenderness limit OK

Short column check for unbraced columns

Column clear height; $l_o = \mathbf{3500}$ mm

Effect height factor for unbraced columns - maj axis; $\beta_x = \mathbf{1.20}$

Effective height (major axis); $l_{ex} = \beta_x \times l_o = \mathbf{4.200}$ m

Slenderness check; $l_{ex}/h = \mathbf{9.33}$

The unbraced column is short (major axis)

Effect height factor for unbraced columns - min axis; $\beta_y = \mathbf{1.20}$

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

Slenderness check; $l_{ey}/b = \mathbf{7.00}$

The unbraced column is short (minor axis)

Short column - bi-axial bending

Define column reinforcement

Main reinforcement in column

Assumed diameter of main reinforcement; D_{col} = **20** mm

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

Area of "tension" steel; $A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4$ = **1571** mm^2

Area of compression steel; $A_{sc} = A_{st}$ = **1571** mm^2

Total area of steel ; $A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2))$ = **5026.5** mm^2

Percentage of steel; $A_{scol} / (b \times h)$ = **1.9** %

Design ultimate loading

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

Design ultimate moment (major axis); M_x = **275** kNm

Design ultimate moment (minor axis); M_y = **220** 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); $M_{xdes} = \max(\text{abs}(M_x), M_{xmin})$ = **275.0** kNm

Design moment (minor axis); $M_{ydes} = \max(\text{abs}(M_y), M_{ymin})$ = **220.0** kNm

Simplified method for dealing with bi-axial bending:-

h' = **390** mm; b' = **540** mm

Approx uniaxial design moment

$\beta = 1 - 1.165 \times \min(0.6, N/(b \times h \times f_{cu}))$ = **0.75**

Design moment;

$M_{design} = \text{if}(M_{xdes}/h' < M_{ydes}/b', M_{ydes} + \beta \times b'/h' \times M_{xdes}, M_{xdes} + \beta \times h'/b' \times M_{ydes})$ = **394.7** kNm

Set up section dimensions for design:-

Section depth; D = if(M_{xdes}/h' < M_{ydes}/b' , b, h) = **450.0** mm

Depth to "tension" steel; d = if(M_{xdes}/h' < M_{ydes}/b' , b', h') = **390.0** mm

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

Library item - Calcs – short col N+Mmaj+Mmin

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 = **450** mm

Depth to "tension" steel (symmetrical); d = **390** mm

Tension and compression steel yield

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) = **462.2** kNm

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

Check for moment; M_{design} = **394.7** kNm

Moment check satisfied

Check min and max areas of steel

Total area of concrete; $A_{conc} = b \times h$ = **270000** mm^2

Area of steel (symmetrical); A_{scol} = **5027** mm^2

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

Minimum steel area; $A_{sc\_min} = k_c \times A_{conc}$ = **2160** mm^2

Maximum steel area; A_{s_max} = 6 % × A_{conc} = **16200** mm^2

Area of compression steel provided OK

Major axis Shear Resistance of Concrete Columns

Column width; b = **600** mm

Column depth; h = **450** mm

Effective depth to steel; h' = **390** mm

Area of concrete; A_{conc} = b × h = **270000** mm^2

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

Characteristic strength of concrete; f_{cu} = **35** N/mm^2

Is a check required? (3.8.4.6)

Axial load; N = **2000.0** kN

Major axis moment; M_x = **275.0** kNm

Eccentricity; e = M_x / N = **137.5** mm

Limit to eccentricity; e_{limit} = 0.6 × h = **270.0** 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) × $\sqrt{(f_{cu})}$, 5 N/mm^2) = **4.733** 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} = **20** mm

Min diameter; L_{limit} = max((6 mm), D_{col}/4) = **6.0** mm

Link diameter OK

Max spacing; s_{limit} = 12 × D_{col} = **240.0** 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 = **450** mm

Column width (smaller column dimension); b = **600** mm

Column area; A_c = h × b = **270000** mm^2

Limit for crack check; N_{limit} = 0.2 × f_{cu} × A_c = **1890.0** 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; Mx_{SLS} = **50** kNm

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension);	h = **450** mm
Depth to steel;	h' = **390** mm
Column width (smaller column dimension);	b = **600** mm
Characteristic strength of concrete;	f_{cu} = **35** 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} = **20** mm
Number of tension reinforcement bars;	L_{ncol} = **5**
Area of tension reinforcement;	$A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol}$ = **1571** 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' = **390.0** mm
Tension bar centres;	$bar_{crs} = (b - 2\times(c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1)$ = **120.0** mm

Modular Ratio

Modulus of elasticity for reinforcement;	E_s = 200 kN/mm^2
Modulus of elast for conc (half the instanteneous);	$E_c = ((20\text{ kN/mm}^2) + 200\times f_{cu}) / 2$ = **14** kN/mm^2
Modular ratio;	$m = E_s / E_c$ = **14.815**

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:-

$$x = h' \times [\, -1\times E_s\times A_{st}/(E_c\times b\times h') + \sqrt{(E_s\times A_{st}/(E_c\times b\times h') \times (2+E_s\times A_{st}/(E_c\times b\times h')))}\,] = \mathbf{139.4}\text{ mm}$$

Depth of concrete in compression; x = **139.4** mm

Concrete and Steel stresses

The serviceability limit state moment; Mx_{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;	$f_c = 2 \times Mx\_{SLS} / (b \times x \times (h' - x/3))$ = **3.48** N/mm^2
Allowable stress;	$0.45 \times f_{cu}$ = **15.75** 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; $f_{st} = Mx\_{SLS} / (A_{st} \times (h' - x/3))$ = **92.66** N/mm^2

Concrete and Steel strains

Strain in the reinforcement;	$\varepsilon_s = f_{st} / E_s$ = **463.3×10^{-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 = **450** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x) = \mathbf{574.2 \times 10^{-6}}$

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; $b_t = b$ = **600** mm

$\varepsilon_m = \min(\varepsilon_1, \max(0, \varepsilon_1 - [\text{ factor} \times b_t \times (h - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (h' - x))])) = \mathbf{329.2 \times 10^{-6}}$

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

$a_{cr1} = \sqrt{((bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2$ = **74.9** mm

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

$a_{cr2} = \sqrt{(2)} \times (c_{nom} + L_{dia} + D_{col}/2) - D_{col}/2$ = **74.9** mm

Critical distance from tension bar; $a_{cr} = \max(a_{cr1}, a_{cr2})$ = **74.9** mm

Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m / (1 + 2 \times (a_{cr} - c_{ten})/(h - x))$ = **0.064** 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 = **450** mm

Column width (smaller column dimension); b = **600** mm

Depth to steel; b' = **540** mm

Characteristic strength of concrete; f_{cu} = **35** 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} = **20** mm

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

Area of tension reinforcement; $A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol}$ = **1571** 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' = **540.0** mm

Tension bar centres; $bar_{crs} = (h - 2 \times (c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1)$ = **82.5** mm

Modular Ratio

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

Modulus of elasticity for conc (half instanteneous); $E_c = ((20 \text{ kN/mm}^2) + 200 \times f_{cu}) / 2$ = **14** kN/mm^2

Modular ratio; $m = E_s / E_c$ = **14.815**

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:-

$x = b' \times [-1 \times E_s \times A_{st}/(E_c \times h \times b') + \sqrt{(E_s \times A_{st}/(E_c \times h \times b') \times (2 + E_s \times A_{st}/(E_c \times h \times b')))}]$ = **190.2** mm

Depth of concrete in compression; x = **190.2** 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; $f_c = 2 \times M_{Y\_SLS} / (h \times x \times (b' - x/3))$ = **2.45** N/mm^2

Allowable stress; $0.45 \times f_{cu}$ = **15.75** 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; $f_{st} = M_{Y\_SLS} / (A_{st} \times (b' - x/3))$ = **66.79** N/mm^2

Concrete and Steel strains

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s$ = **333.9×10^{-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; $a' = b$ = **600** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(b' - x)$ = **391.2×10^{-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; $h_t = h$ = **450** mm

$\varepsilon_m = \min(\varepsilon_1 , \max(0, \varepsilon_1 - [\text{factor} \times h_t \times (b - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (b' - x))]))$ = **162.0×10^{-6}**

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

$a_{cr1} = \sqrt{ ((bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2$ = **62.8** mm

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

$a_{cr2} = \sqrt{(2)} \times (c_{nom} + L_{dia} + D_{col}/2) - D_{col}/2$ = **74.9** mm

Critical distance from tension bar; $a_{cr} = \max(a_{cr1}, a_{cr2})$ = **74.9** mm

Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2 \times (a_{cr} - c_{ten})/(b - x))$ = **0.032** mm

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

RC SLAB DESIGN

TWO WAY SPANNING SLAB DEFINITION - RESTRAINED

Overall depth of slab; h = **150** mm

Outer sagging steel

Cover to outer tension reinforcement resisting sagging; c_{sag} = **25** mm

Trial bar diameter; D_{tryx} = **10** mm

Depth to outer tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2$ = **120** mm

Inner sagging steel

Trial bar diameter; D_{tryy} = **10** mm

Depth to inner tension steel (resisting sagging)

$d_y = h - c_{sag} - D_{tryx} - D_{tryy}/2$ = **110** mm

Outer hogging steel

Cover to outer tension reinforcement resisting hogging; c_{hog} = **25** mm

Trial bar diameter; $D_{tryxhog}$ = **10** mm

Depth to outer tension steel (resisting hogging)

$d_{xhog} = h - c_{hog} - D_{tryxhog}/2$ = **120** mm

Inner hogging steel

Trial bar diameter; $D_{tryyhog}$ = **10** mm

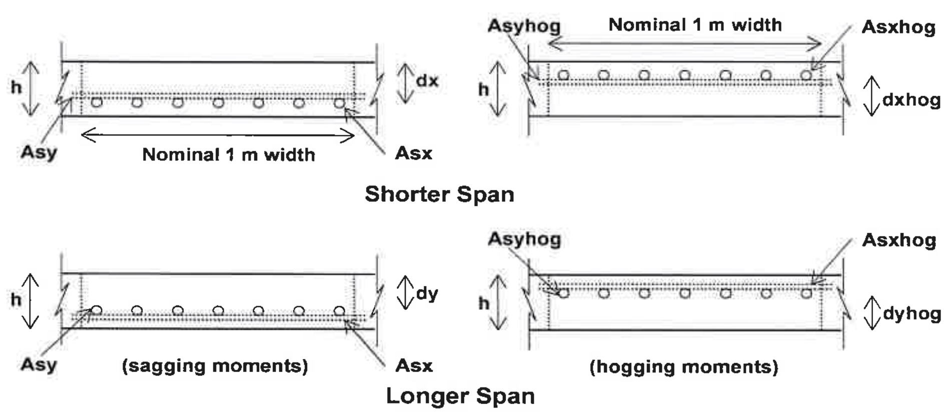
Depth to inner tension steel (resisting hogging)

$d_{yhog} = h - c_{hog} - D_{tryxhog} - D_{tryyhog}/2$ = **110** mm

Materials

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

Characteristic strength of concrete; f_{cu} = **25** N/mm^2



Two-way spanning slab

(restrained)

RESTRAINED – 2 WAY SPANNING

MAXIMUM DESIGN MOMENTS

Length of shorter side of slab; l_x = **4.000** m

Length of longer side of slab; l_y = **6.000** m

Design ultimate load per unit area; n_s = **15.0** kN/m^2

Edge condition shorter side (1); $Edge_1$ = **"C"**

Edge condition other shorter side (2); $Edge_2$ = **"D"**

Edge condition longer side (3); $Edge_3$ = **"C"**

Edge condition other longer side (4); $Edge_4$ = **"D"**

Number of discontinuous edges

N_d = **2**

Moment coefficients

$\beta_{sy} = (24 + 2\times N_d + 1.5\times N_d^2) / 1000 =$ **0.034**

β_1 = if($Edge_1$ == "C", $4/3\times\beta_{sy}$,0) = **0.045**

β_2 = if($Edge_2$ == "C", $4/3\times\beta_{sy}$,0) = **0.000**

$\gamma = 2/9 \times [3 - \sqrt{(18)} \times l_x/l_y \times (\sqrt{(\beta_{sy} + \beta_1)} + \sqrt{(\beta_{sy} + \beta_2)})] =$ **0.374**

β_{3x} = if($Edge_3$ == "C", 4/3,0) = **1.333**

β_{4x} = if($Edge_4$ == "C", 4/3,0) = **0.000**

$\beta_{sx} = \gamma / [(1+\beta_{3x})^{0.5} + (1+\beta_{4x})^{0.5}]^2 =$ **0.059**

$\beta_3 = \beta_{3x} \times \beta_{sx} =$ **0.078**

$\beta_4 = \beta_{4x} \times \beta_{sx} =$ **0.000**

Maximum span moments per unit width - restrained slabs

$m_{sx} = \beta_{sx} \times n_s \times l_x^2 =$ **14.0** kNm/m

$m_{sy} = \beta_{sy} \times n_s \times l_x^2 =$ **8.2** kNm/m

Maximum support moments per unit width - restrained slabs

$m_{sxhog} = \max(\beta_3, \beta_4) \times n_s \times l_x^2 =$ **18.7** kNm/m

$m_{syhog} = \max(\beta_1, \beta_2) \times n_s \times l_x^2 =$ **10.9** kNm/m

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL

Design sagging moment (per m width of slab); m_{sx} = **14.0** kNm/m

Moment Redistribution Factor; β_{bx} = **1.0**

Slab requiring outer tension steel only - bars (sagging)

$z_x = \min((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)})))$ = **114** mm

Neutral axis depth; $x_x = (d_x - z_x) / 0.45$ = **13** mm

Area of tension steel required

$A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x)$ = **283** mm^2/m

Tension steel

;;Provide 10 dia bars @ 150 centres; outer tension steel resisting sagging

$A_{sx\_prov} = A_{sx}$ = **524** mm^2/m

Area of outer tension steel provided sufficient to resist sagging

Concrete Slab Design - Sagging - Inner layer of steel

Design sagging moment (per m width of slab); m_{sy} = **8.2** kNm/m

Moment Redistribution Factor; β_{by} = **1.0**

Slab requiring inner tension steel only - bars (sagging)

$z_y = \min((0.95 \times d_y),(d_y \times (0.5+\sqrt{(0.25-K_y/0.9)})))$ = **105** mm

Neutral axis depth; $x_y = (d_y - z_y) / 0.45$ = **12** mm

Area of tension steel required

$A_{sy\_req} = abs(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y)$ = **180** mm^2/m

Tension steel

Provide 10 dia bars @ 150 centres; inner tension steel resisting sagging

$A_{sy\_prov} = A_{sy}$ = **524** mm^2/m

Area of inner tension steel provided sufficient to resist sagging

CONCRETE SLAB DESIGN – HOGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design hogging moment (per m width of slab); m_{sxhog} = **18.7** kNm/m

Moment Redistribution Factor; β_{bx} = **1.0**

Slab requiring outer tension steel only - bars (hogging)

$z_{xhog} = \min((0.95 \times d_{xhog}),(d_{xhog} \times (0.5+\sqrt{(0.25-K_{xhog}/0.9)})))$ = **113** mm

Neutral axis depth; $x_{xhog} = (d_{xhog} - z_{xhog}) / 0.45$ = **16** mm

Area of tension steel required

$A_{sxhog\_req} = abs(m_{sxhog}) / (1/\gamma_{ms} \times f_y \times z_{xhog})$ = **382** mm^2/m

Tension steel

Provide 10 dia bars @ 150 centres; outer tension steel resisting hogging

$A_{sxhog\_prov} = A_{sxhog}$ = **524** mm^2/m

Area of outer tension steel provided sufficient to resist hogging

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); $A_{s\_req} = F_t /(1/\gamma_{ms} \times f_y)$ =**588** 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

Minimum area of tension steel; $A_{st\_min} = k_t \times A_c$ = **1170** mm^2

Maximum area of tension steel; $A_{st\_max} = 4\ \% \times A_c$ = **36000** mm^2

Area of tension steel provided OK

Shear

Applied shear stress

Aplied Shear stress; $V = F_{uls} / 2$ = **405.0** kN

Width of pile cap shear plane; $b_v = \min(b, 3 \times \phi)$ = **900** mm

Design shear stress; $v = V / (b_v \times d)$ = **0.47** 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

Determine concrete shear strength on the section at distance ϕ / 5 inside face of pile:

Shear stress

$v_{c\_25} = 0.79 \times r^{1/3} \times \max(0.67, (400\ mm/d)^{1/4}) \times 1.0\ N/mm^2 / 1.25$ = **0.31** N/mm^2

Shear stress - $v_c = v_{c\_25} \times (\min(f_{cu}, 40\ N/mm^2) / 25\ N/mm^2)^{1/3}$ = **0.31**
N/mm^2

Spacing; $a_v = \min(2 \times d, \max((s / 2 - \phi / 2 + \phi / 5 - x / 2), 0.1\ mm))$
= **270** mm

Enhanced shear stress; $v_{c\_enh} = \min(v_{allowable} , 2 \times d \times v_c / a_v)$ = **2.21** 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.

Clear Distance Between Bars In Tension

Bar spacing;

$spacing_{bars} = \max(0mm, (b- 2 \times (c_{adopt} + L_{dia}) - D_t)/(L_{nt} - 1) - D_t)$ = **73** mm

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

Minimum allowable spacing; $spacing_{min} = h_{agg} + 5\ mm$ = **25** mm

Bar spacing OK

Clear Distance Between Face Of Beam And Tension Bars

Distance to face of beam; $Dist_{edge} = c_{adopt} + L_{dia} + D_t/2$ = **50** mm

Design service stress in reinforcement; $f_s = 2 \times f_y \times A_{s\_req} /(3 \times A_{s\_prov} \times \beta_b)$ = **97.5** N/mm^2

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

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

Max distance to beam edge check - OK

Anchorage Of Tension Steel

Anchorage factor;	ϕ_{factor} =**44**
Type of lap length;	lap_type =**"tens_lap"**
Type of reinforcement;	reft_type = **"def2_fy500"**
Minimum radius;	r_{bar} = **32** mm
Minimum end projection;	P_{bar} = **130** mm
Minimum anchorage length or lap length req'd;	$L_{table\ 3.27} = \phi_{factor} \times D_t$ = **704** mm
Check anchorage length to cl. 3.12.9.4 (b);	$L_{cl.\ 3.12.9.4} = 12 \times D_t + d/2$ = **667** mm
Required minimum effective anchorage length;	$L_a = \max(L_{table\ 3.27}, L_{cl.\ 3.12.9.4})$ = **704** mm

Deflection Check

Redistribution ratio; $\beta_b = 1.0$

Design service stress in tension reinforcement; $f_s = 2 \times f_y \times A_{s\_req} /(3 \times A_{s\_prov} \times \beta_b)$ = **97.5** N/mm^2

Modification for tension reinforcement;

$$factor_{tens} = \min(2, 0.55 + (477\ N/mm^2 - f_s)/(120 \times (0.9\ N/mm^2 + F_t /(b \times d)))) = \mathbf{2.000}$$

Modified span to depth ratio;	$modf_{span\_depth} = factor_{tens} \times basic_{span\_depth}$ = **40.0**
Span of pile cap for deflection check;	L_s = **1000** mm
Actual span to depth ratio;	$actual_{span\_depth} = L_s /d$ = **1.05**

PASS - Deflection

$v_{c\_252} = 0.79 \times r_2^{1/3} \times \max(0.67, (400\ mm/d)^{1/4}) \times 1.0\ N/mm^2 / 1.25 = \mathbf{0.32}\ N/mm^2$

$v_{c3} = v_{c2} = v_{c\_252} \times (\min(f_{cu}, 40\ N/mm^2)/25\ N/mm^2)^{1/3} = \mathbf{0.32}$

N/mm^2

$a_{v\_2} = \min(2 \times d, \max((s/2 / \cos(\theta_{23}) - \phi/2 + \phi/5 - y/2), 0.1mm)) = \mathbf{108}\ mm$

$v_{c\_enh\_3} = v_{c\_enh\_2} = 2 \times d \times v_{c2}/a_{v\_2} = \mathbf{8.50}\ N/mm^2$

Concrete shear strength - OK, no links reqd. at Piles 2 and 3

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; $u_0 = 2 \times (x + y) = \mathbf{2000}\ mm$

Assumed average depth to tension steel; $d_{av} = d - D_t = \mathbf{1424}\ mm$

Max shear effective across perimeter; $V_p = F_{uls} = \mathbf{1400.0}\ kN$

Stress around loaded area; $v_{max} = V_p / (u_0 \times d_{av}) = \mathbf{0.49}\ N/mm^2$

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

N/mm^2

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;

$spacing_{bars} = \max(0mm, (b_{ccs} - n_{surfaces} \times (c_{adopt} + L_{dia}) - D_t)/(L_{nt} - 1) - D_t) = \mathbf{38}\ mm$

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; $f_s = 2 \times f_y \times A_{s\_req} / (3 \times A_{s\_prov} \times \beta_b) = \mathbf{34.3}\ N/mm^2$

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; $L_{table\ 3.27} = \phi_{factor} \times D_t = \mathbf{704}\ mm$

Check anchorage length $L_{cl.\ 3.12.9.4} = 12 \times D_t + d/2 = \mathbf{912}\ mm$

Required minimum effective anchorage length; $L_a = \max(L_{table\ 3.27}, L_{cl.\ 3.12.9.4}) = \mathbf{912}\ mm$

Check bearing stress on minimum radius bend

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

Force per bar at bend; $F_{bt} = F_t / L_{nt} = \mathbf{9.0}\ kN$

Bearing stress; $f_{bt} = F_{bt} / (r_{bar} \times D_t) = \mathbf{17.58}\ N/mm^2$

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

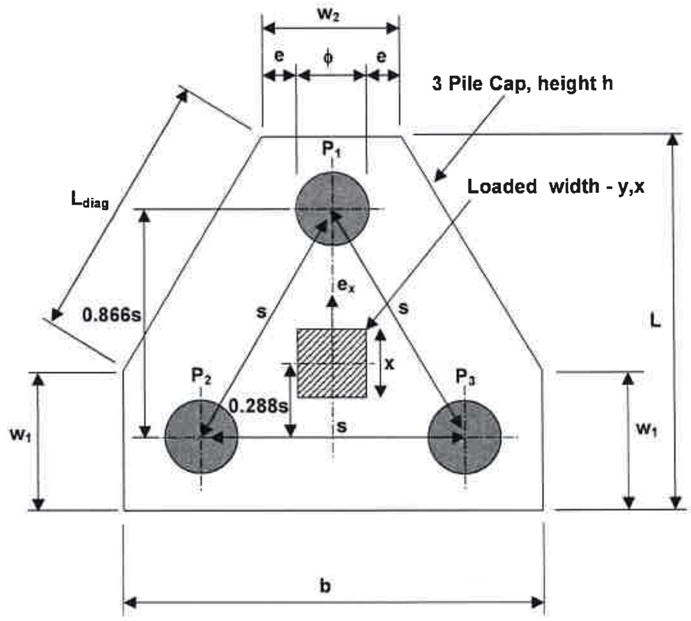
Edge maximum allowable bearing stress; $f_{bt\_max\_ext} = 2 \times f_{cu} / (1 + 2 \times (D_t / s_{ext})) = \mathbf{31.82}\ N/mm^2$

Internal bar centres; s_{int} = spacing$_{bars}$ + D_t = **54** mm

Internal maximum allowable bearing stress; $f_{bt\_max\_int} = 2 \times f_{cu} / (1 + 2\times(D_t / s_{int}))$ = **31.40** N/mm^2

PASS - Bearing stress on minimum radius bend is less than maximum allowable

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; $F_t = F_c \times \cos(\theta) / (2 \times \mathrm{Cos}(30))$ = **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) × √(f_{cu}), 5 N/mm^2) = **4.00** N/mm^2

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} - $n_{surfaces}$ × (c_{adopt} + L_{dia}) - D_t)/(L_{nt} - 1) - D_t) =**32** mm

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

Minimum required spacing of bars; $spacing_{min}$ = h_{agg} + 5 mm = **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 = **60** mm

Design service stress in reinforcement; f_s = 2 × f_y × $As_{\_req}$ /(3 × $As_{\_prov}$ × β_b) = **81.1** N/mm^2

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

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

Max distance to beam edge check - OK

Anchorage Of Tension Steel

Anchorage factor; ϕ_{factor} =**44**

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

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

Minimum radius; r_{bar} = **32** mm

Minimum end projection; P_{bar} = **130** mm

Minimum anchorage length or lap length req'd; = ϕ_{factor} × D_t = **704** mm

Check anchorage length = 12 × D_t + d/2 = **662** mm

Required minimum effective anchorage length; L_a = = **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} = **21.3** kN

Edge bar centres; s_{ext} = c_{adopt} + D_t = **56** mm

Edge maximum allowable bearing stress; $f_{bt\_max\_ext}$ = 2 × f_{cu} / (1 + 2×(D_t / s_{ext})) = **31.82** N/mm^2

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

Internal maximum allowable bearing stress; $f_{bt\_max\_int}$ = 2 × f_{cu} / (1 + 2×(D_t / s_{int})) = **30.00** N/mm^2

Design max allowable bearing stress; f_{bt_max} = min($f_{bt\_max\_ext}$, $f_{bt\_max\_int}$) = **30.00** N/mm^2

Minimum radius required; r_{min} = max(r_{bar}, F_{bt} /(f_{bt_max} × D_t)) = **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: 98 40 44 86 13

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

STRUCTURAL DRAWINGS