

STABILITY CERTIFICATE

For,

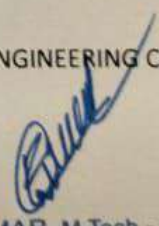
This is to certify THE PROPOSED CONSTRUCTION OF BASEMENT FLOOR (MECHANICAL AUTOMATED CAR PARKING – 2 LEVELS) + GROUND FLOOR & FIRST FLOOR (SHOP) AND SECOND FLOOR (OFFICE) (COMMERCIAL BUILDING) AT PLOT – 663, OLD DOOR NO – 103, NEW DOOR NO – 85, D – BLOCK, SECOND AVENUE & FIRST AVENUE ROAD, ANNA NAGAR 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 CONSULTANTS

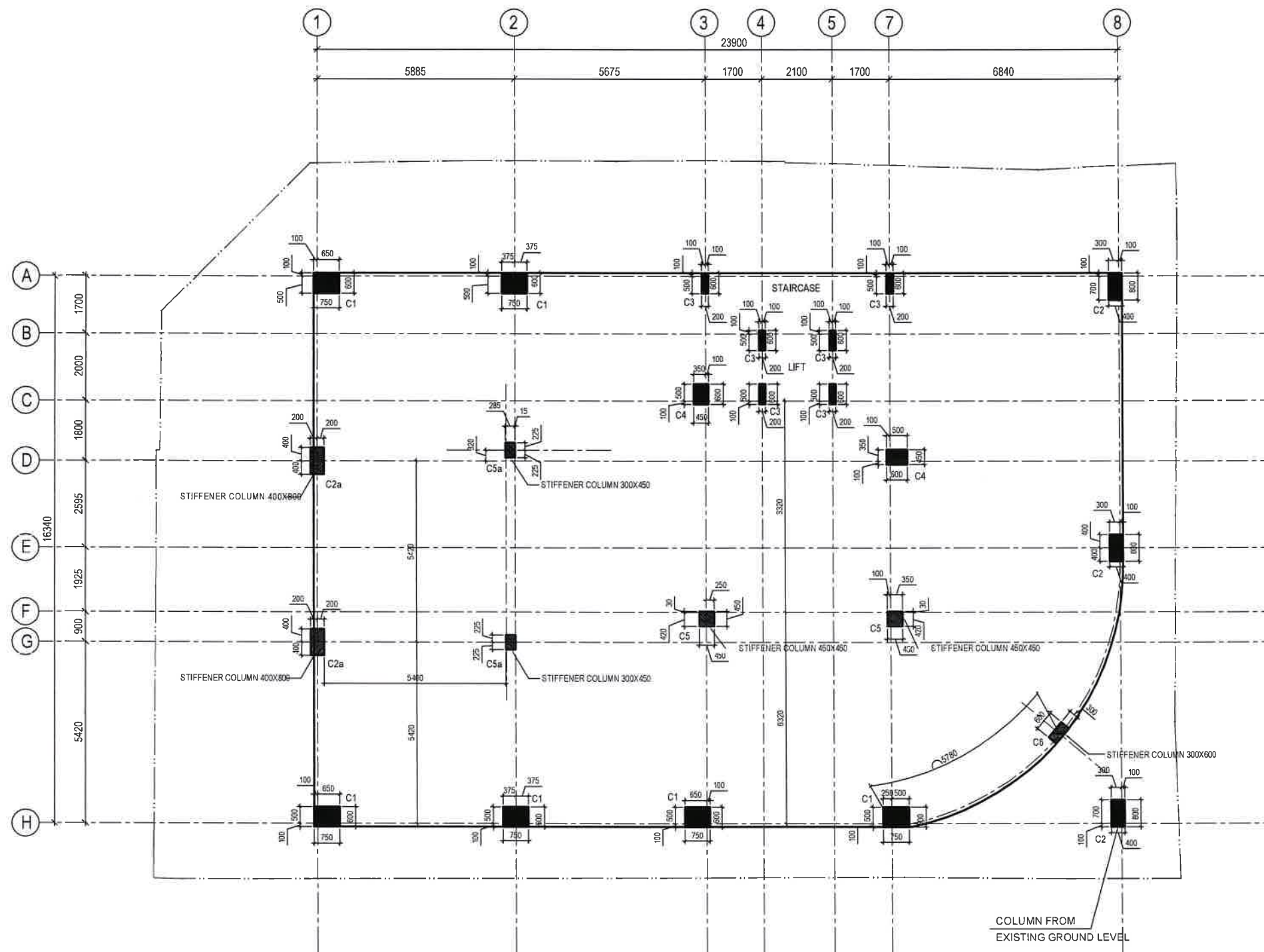

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer - Grade-1,
Reg. No. SE/Gr-1/2024/02/007
97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamalaiyur
Chengalpattu - 603 003
Cell: 98 40 44 86 13

Date: 08-10-2025

Place: Chennai

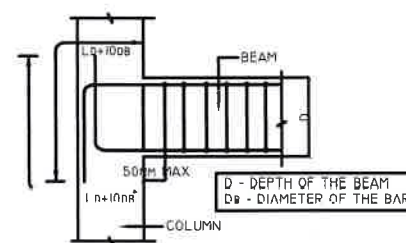
E.S. Kumar., M.Tec

Principal Structural Consultant

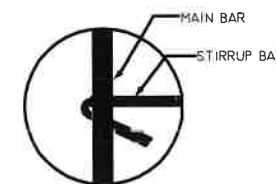


COLUMN AXIS LAYOUT

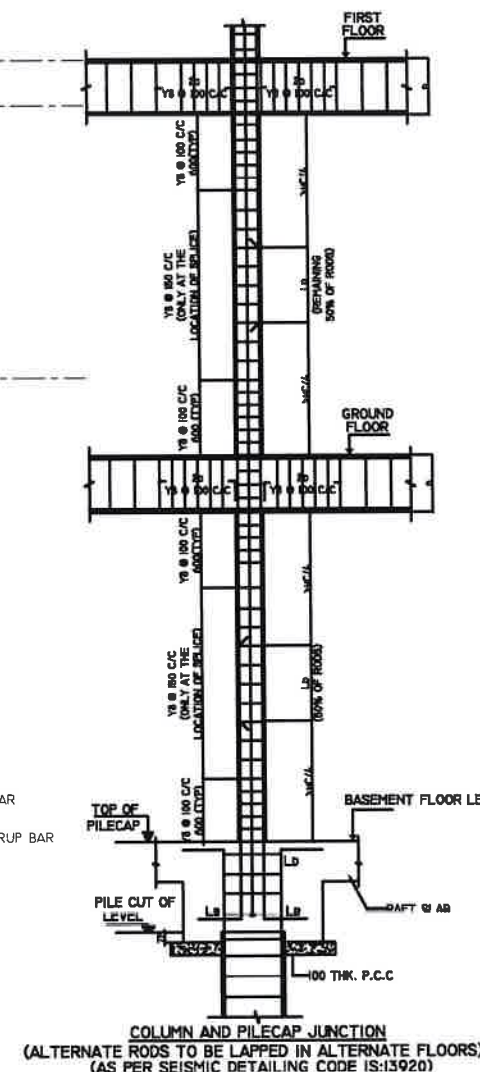
ES. KUMAR
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 CMDA Structural Engineer - Grade - 1,
 Reg. No. SE/Gr-1/2024/02/007
 # 97-B/1, Dr. Abdul Kalam Street,
 Ramakrishna Nagar, Melamaiyur
 Chengalpattu - 603 003
 Cell: 98 40 44 86 13



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 IS:456 : 2000

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND 8 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 (L_d) SHALL BE AS PER SP:34 - 1999 AND GIVEN IN THE TABLE BELOW.

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	125	150	200	250	125	150	200	250
8mm	400	400	370	330	370	370	330	280
10mm	450	450	400	350	400	400	350	300
12mm	500	500	450	400	450	450	400	350
16mm	600	600	550	500	550	550	500	450
20mm	700	700	650	600	650	650	600	550
25mm	850	850	800	750	800	800	750	700
32mm	1100	1100	1050	1000	1050	1050	1000	950
40mm	1400	1400	1350	1300	1350	1350	1300	1250

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 CONFORM 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 BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION)
 5. GRADE OF CONCRETE SHALL CONFORM 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 CONFORMING 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 _____ 50mm

	SAFE LOAD IN COMPRESSION (T)
400 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	05.08.21

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

CLIENT :
 EMPEE GROUP

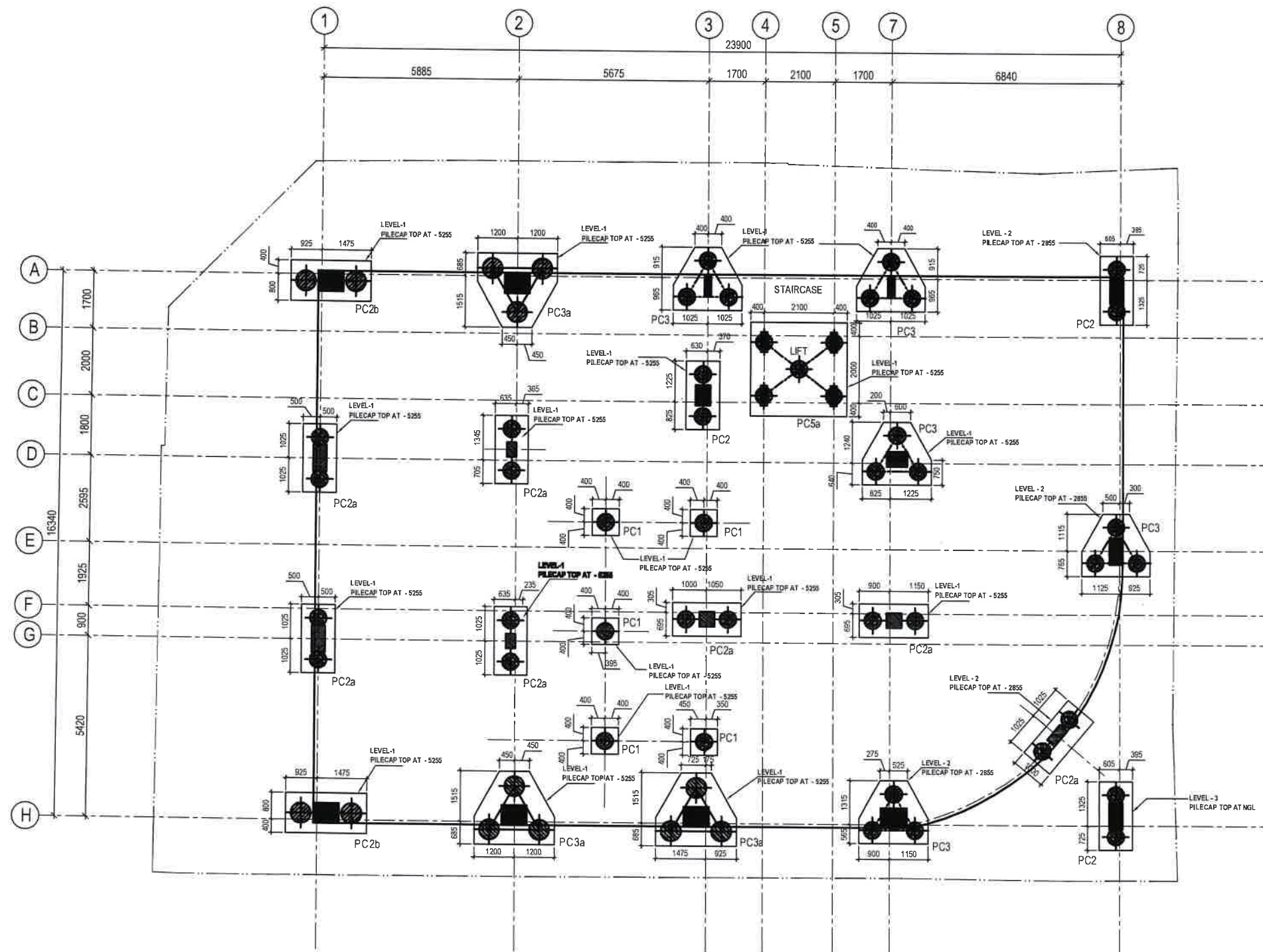
DRAWING TITLE :
 COLUMN AXIS LAYOUT

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

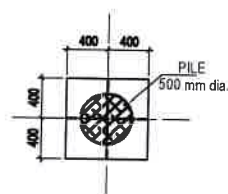
STRUCTURAL CONSULTANTS:
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
 PLOT NO-1233, FIRST FLOOR,
 FLAT NO-1B, PRIYA AJAY ARCADE,
 18th MAIN ROAD, ANNA NAGAR, (WEST)
 Chennai - 600 040. Mob: 98404 48613

ARCHITECTS:
 Harismaa Consultant Ltd
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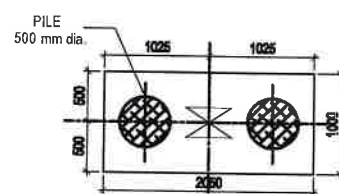
Drawn M.D.	Designed DANE	Checked ES KUMAR



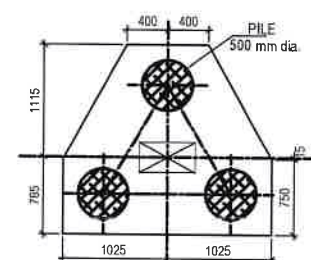
PILE CAP LAYOUT



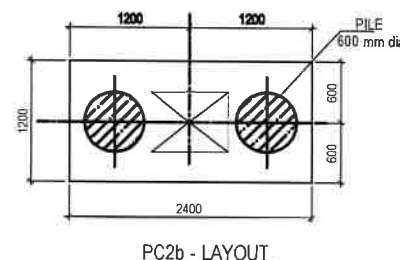
PC1 - LAYOUT



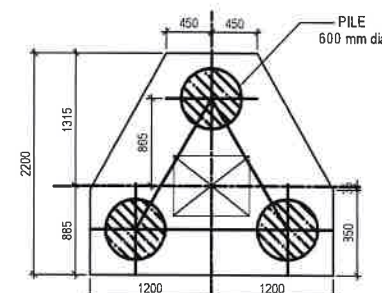
PC2 - LAYOUT



PC3 - LAYOUT



PC2b - LAYOUT



PC3a - LAYOUT

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 IS 456: 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 (L_{DE}) SHALL BE AS PER SP-34 - 1999 AND GIVEN IN THE TABLE BELOW.

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	125	150	175	200	125	150	175	200
8mm	400	400	370	320	370	320	290	250
10mm	450	450	400	350	400	350	320	280
12mm	500	500	450	400	450	400	370	330
14mm	550	550	500	450	500	450	420	380
16mm	600	600	550	500	550	500	470	430
18mm	650	650	600	550	600	550	520	480
20mm	700	700	650	600	650	600	570	530
22mm	750	750	700	650	700	650	620	580
25mm	800	800	750	700	750	700	670	630
28mm	850	850	800	750	800	750	720	680
32mm	900	900	850	800	850	800	770	730

- 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 CONFORM 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 CONFORM TO IS 456: 2000. GRADE OF CO-10 CONCRETE FOR COLUMN, PILE, PILE CAP, RCC WALL AND RAFT SLAB - M30.
6. "Y" DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFORMING 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: 50mm

DIA. OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
600 MM (P1)	137.1
500 MM (P2)	73.1

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

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	06.08.21
REV	DESCRIPTION	DRN	CHK	DATE

PROJECT NAME:

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

CLIENT:

EMPEE GROUP

DRAWING TITLE:

PILE CAP LAYOUT

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

STRUCTURAL CONSULTANTS:



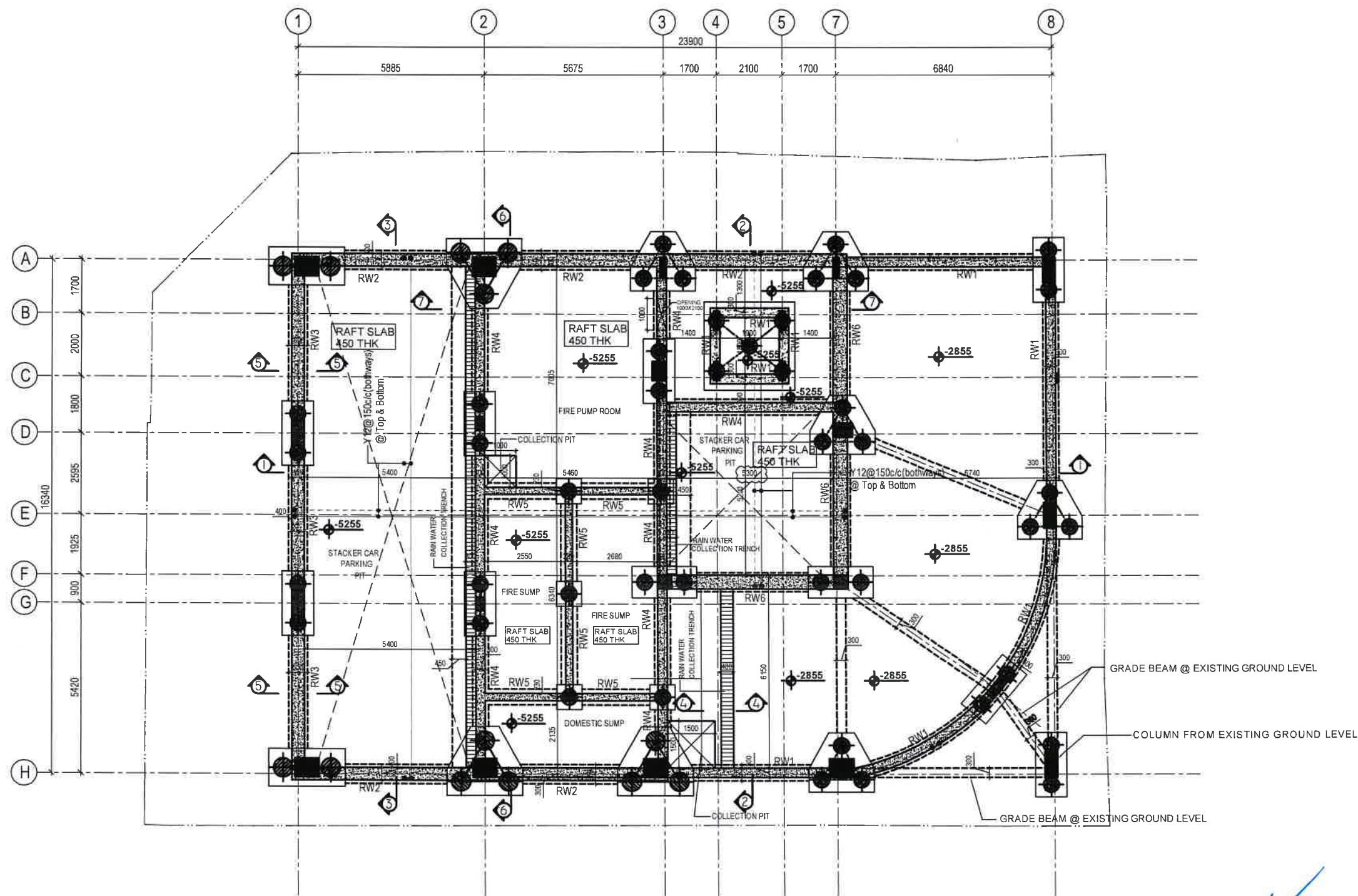
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
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18th MAIN ROAD, ANNA NAGAR (WEST)
Chennai - 600 040. Mob: 98404 48613.

ARCHITECTS:

Harismaa Consultant Ltd
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Tel: 044 - 26223147

Drawn M.D.	Designed DANE	Checked ES KUMAR

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GRADE BEAM, R.C WALL AND RAFT SLAB LAYOUT

TOC - 5255

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Chengalpattu - 603 003
Cell : 98 40 44 86 13

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 IS 456 : 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

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

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	125	125	125	125	125	125	125	125
8mm	450	400	370	320	370	320	290	280
10mm	550	480	450	400	450	380	350	320
12mm	650	580	550	500	550	480	450	420
16mm	850	780	750	680	730	650	620	600
20mm	1150	970	920	850	910	780	730	700
25mm	1450	1220	1140	1000	1140	970	910	880
32mm	1800	1560	1480	1300	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 CONFORM TO SP-34 & IS 13920

GENERAL NOTES:
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 6. 'Y' DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFORMING 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)	13.7 T
500 MM (P2)	7.3 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.
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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. :
 NA - 8-04

DWG. NO. :
 8-04

REV. NO. 04
 Date 11.10.21

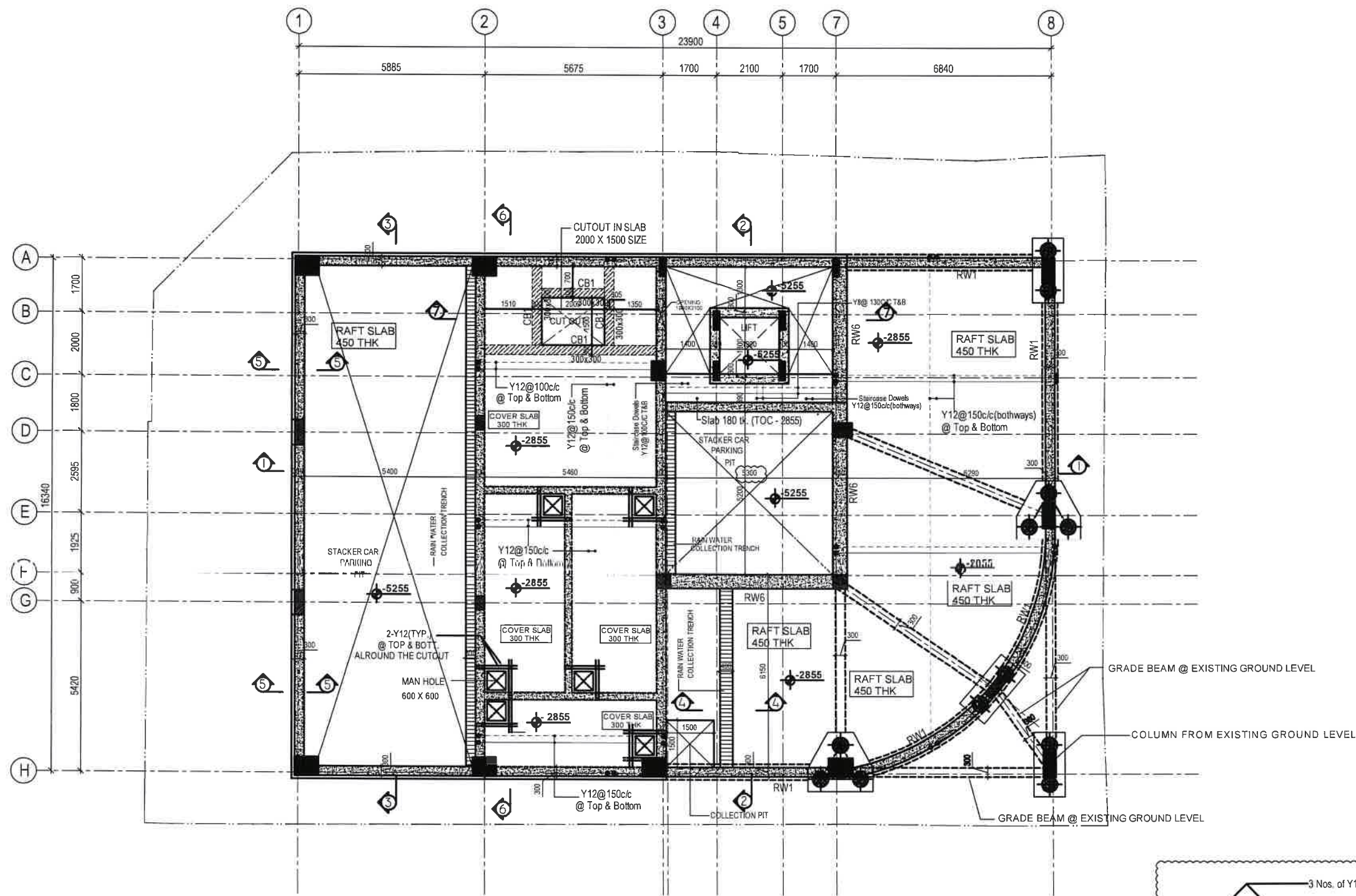
REF. CODE

STRUCTURAL CONSULTANTS:

KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
 PLOT NO:1233, FIRST FLOOR,
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 18th MAIN ROAD, ANNA NAGAR, (WEST)
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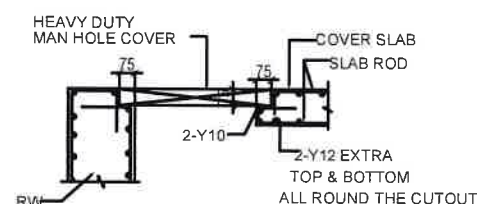
ARCHITECTS:
Harismaa Consultant Ltd
 #109/A46, 1st FLOOR, 3rd AVENUE,
 ANNA NAGAR EAST, CHENNAI - 600102
 Tel: 044- 25203147

Drawn M.D
 Designed DANE
 Checked ES.KUMAR

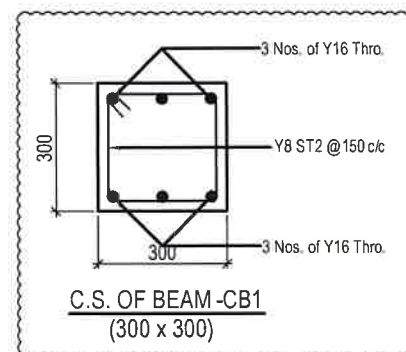


GRADE BEAM, R.C WALL, SUMP COVER SLAB AND RAFT SLAB LAYOUT

TOC - 2855



TYPICAL MANHOLE DETAIL



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

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Chengalpattu - 603 003
Cell: 98 40 44 86 13

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 IS 456: 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			
	125	150	175	200	125	150	175	200
8mm	400	400	370	320	370	320	280	280
10mm	480	480	440	370	440	370	320	320
12mm	560	560	500	400	500	400	350	350
14mm	640	640	570	450	570	450	380	380
16mm	720	720	640	500	640	500	420	420
18mm	800	800	710	560	710	560	470	470
20mm	880	880	780	620	780	620	520	520
22mm	960	960	850	680	850	680	580	580

- 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 CONFORM 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 BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION).
- 5. GRADE OF CONCRETE SHALL CONFORM 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 50C CONFORMING TO IS 1786: 2008
- 7. CLEAR COVER TO THE REINFORCEMENT
 - A. COLUMN _____ 40mm
 - B. PILE _____ 75mm
 - C. PILE CAP BOTTOM - 75mm, TOP & SIDES - 50mm
 - D. RAFT SLAB _____ 50mm

DIAM. OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
400 MM (P1)	137.1
300 MM (P2)	73.1

- 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
02	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

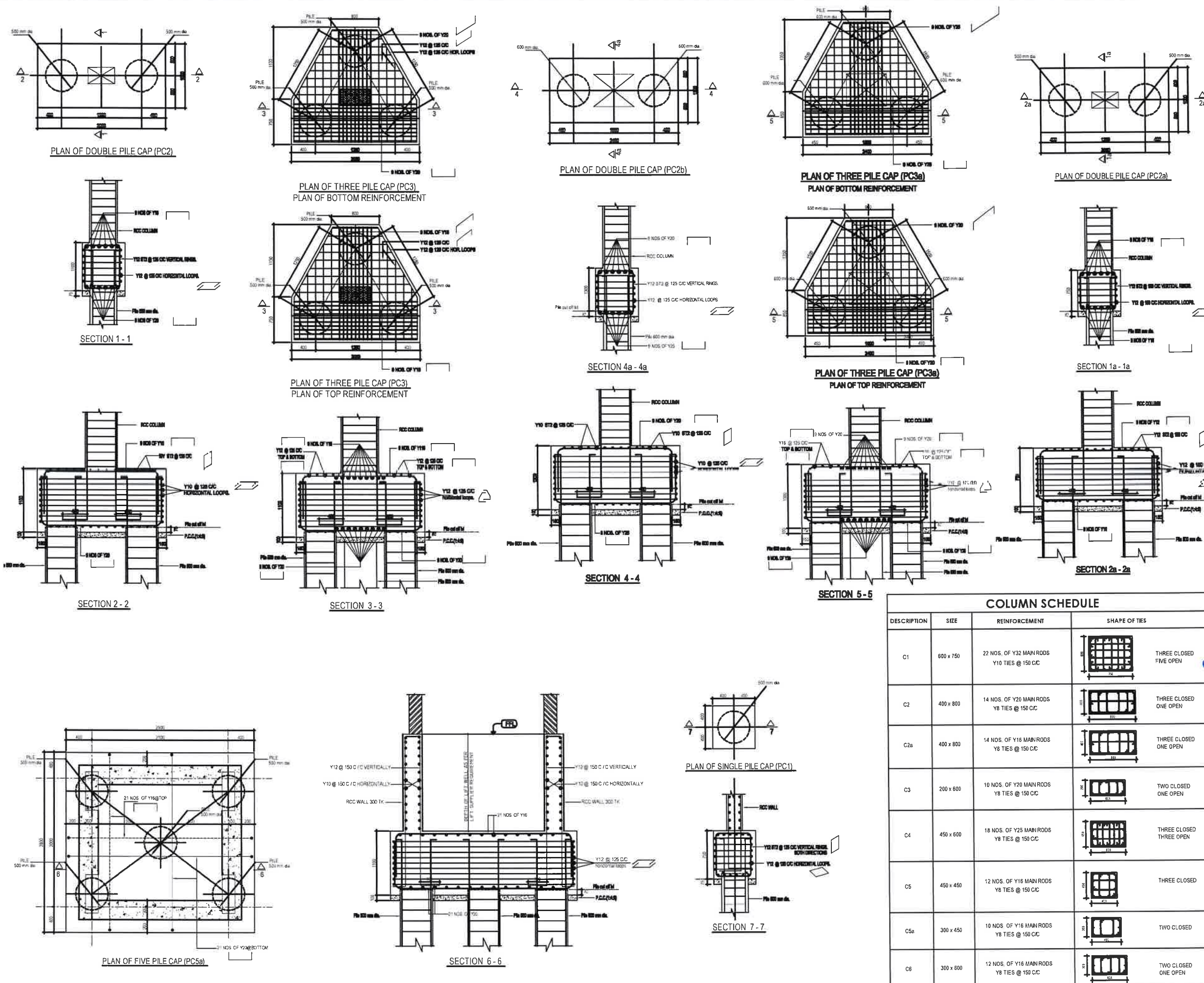
JOB NO.	DWG. NO.	REV. NO. 01	REF. CODE
KA -	S-04A	Date 14.09.21	

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: 70404 40613.

ARCHITECTS: Harismaa Consultant Ltd
#105/A46, 1st FLOOR, 310 AVENUE,
ANNA NAGAR EAST, CHENNAI - 600102
Tel: 044 - 2622147

Drawn M.D. Designed DANE Checked ES. KUMAR



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. THE DWG. SHOULD BE READ IN CONJUNCTION WITH THE ARCHITECTURAL DWG. (LATEST REVISION).

5. GRADE OF CONCRETE SHALL CONFORM TO IS 456: 2000.

6. GRADE OF CONCRETE FOR COLUMN, PILE CAP, RCC WALL AND RAFT SLAB - M30.

7. Y DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFORMING TO IS 1786: 2008.

8. CLEAR COVER TO THE REINFORCEMENT

9. COLUMNS - 40mm

10. PILE - 75mm

11. PILE CAP BOTTOM - 75mm TOP & SIDES - 50mm

12. RAFT SLAB - 50mm

13. DIA. OF PILE (MM) SAFE LOAD IN COMPRESSION (T)

14. 400 MM (P1) 13.7 T

15. 500 MM (P2) 23.1 T

16. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT-GROUND FLOOR + 2 FLOORS ONLY.

17. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS 10139 (PART 1) & IS 10139 (PART 2) IS ONLY TENTATIVE. THE ACTUAL SAFE CAPACITY OF A PILE OF THE CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD. THE LOWER OF THE TWO VALUES SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION.

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

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

GENERAL NOTES

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

2. LAPS IN BEAM AND SLAB SHALL BE PROVIDED AT THE POINT OF CONTRA FLEXURE.

3. 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.

4. DETAILING OF REBAR SHALL CONFORM TO SP-34 & IS 13920.

GENERAL NOTES

1. REMOVAL OF FORM WORK (SCHEDULE)

2. THE FOLLOWING MINIMUM PERIOD (AFTER FINAL POUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS 456: 2000

3. A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND BEAMS - 34 HOURS

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

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

6. D. BOTTOM PROPS TO SLAB

7. i. SPANNING UP TO 4.50M - 7 DAYS

8. ii. SPANNING OVER 4.50M - 14 DAYS

9. E. BOTTOM PROPS TO BEAMS

10. i. SPANNING UP TO 4.50M - 14 DAYS

11. ii. SPANNING OVER 4.50M - 21 DAYS

LAP LENGTH FOR REINFORCEMENT BARS

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

2. 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.

3. DEVELOPMENT LENGTH (L_{DE}) SHALL BE AS PER SP-34 - 1999 AND GIVEN IN THE TABLE BELOW.

REINFORCEMENT BARS

1. Y10 @ 125 C/C VERTICALLY

2. Y12 @ 125 C/C HORIZONTAL LOOPS

3. Y16 @ 125 C/C HORIZONTAL LOOPS

4. Y10 @ 125 C/C VERTICALLY

5. Y12 @ 125 C/C HORIZONTAL LOOPS

6. Y16 @ 125 C/C HORIZONTAL LOOPS

7. Y10 @ 125 C/C VERTICALLY

8. Y12 @ 125 C/C HORIZONTAL LOOPS

9. Y16 @ 125 C/C HORIZONTAL LOOPS

10. Y10 @ 125 C/C VERTICALLY

11. Y12 @ 125 C/C HORIZONTAL LOOPS

12. Y16 @ 125 C/C HORIZONTAL LOOPS

13. Y10 @ 125 C/C VERTICALLY

14. Y12 @ 125 C/C HORIZONTAL LOOPS

15. Y16 @ 125 C/C HORIZONTAL LOOPS

16. Y10 @ 125 C/C VERTICALLY

17. Y12 @ 125 C/C HORIZONTAL LOOPS

18. Y16 @ 125 C/C HORIZONTAL LOOPS

19. Y10 @ 125 C/C VERTICALLY

20. Y12 @ 125 C/C HORIZONTAL LOOPS

21. Y16 @ 125 C/C HORIZONTAL LOOPS

22. Y10 @ 125 C/C VERTICALLY

23. Y12 @ 125 C/C HORIZONTAL LOOPS

24. Y16 @ 125 C/C HORIZONTAL LOOPS

25. Y10 @ 125 C/C VERTICALLY

26. Y12 @ 125 C/C HORIZONTAL LOOPS

27. Y16 @ 125 C/C HORIZONTAL LOOPS

28. Y10 @ 125 C/C VERTICALLY

29. Y12 @ 125 C/C HORIZONTAL LOOPS

30. Y16 @ 125 C/C HORIZONTAL LOOPS

31. Y10 @ 125 C/C VERTICALLY

32. Y12 @ 125 C/C HORIZONTAL LOOPS

33. Y16 @ 125 C/C HORIZONTAL LOOPS

34. Y10 @ 125 C/C VERTICALLY

35. Y12 @ 125 C/C HORIZONTAL LOOPS

36. Y16 @ 125 C/C HORIZONTAL LOOPS

37. Y10 @ 125 C/C VERTICALLY

38. Y12 @ 125 C/C HORIZONTAL LOOPS

39. Y16 @ 125 C/C HORIZONTAL LOOPS

40. Y10 @ 125 C/C VERTICALLY

41. Y12 @ 125 C/C HORIZONTAL LOOPS

42. Y16 @ 125 C/C HORIZONTAL LOOPS

43. Y10 @ 125 C/C VERTICALLY

44. Y12 @ 125 C/C HORIZONTAL LOOPS

45. Y16 @ 125 C/C HORIZONTAL LOOPS

46. Y10 @ 125 C/C VERTICALLY

47. Y12 @ 125 C/C HORIZONTAL LOOPS

48. Y16 @ 125 C/C HORIZONTAL LOOPS

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50. Y12 @ 125 C/C HORIZONTAL LOOPS

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303. Y16 @ 125 C/C HORIZONTAL LOOPS

304. Y10 @ 125 C/C VERTICALLY

305. Y12 @ 125 C/C HORIZONTAL LOOPS

306. Y16 @ 125 C/C HORIZONTAL LOOPS

307. Y10 @ 125 C/C VERTICALLY

308. Y12 @ 125 C/C HORIZONTAL LOOPS

309. Y16 @ 125 C/C HORIZONTAL LOOPS

310. Y10 @ 125 C/C VERTICALLY

311. Y12 @ 125 C/C HORIZONTAL LOOPS

312. Y16 @ 125 C/C HORIZONTAL LOOPS

313. Y10 @ 125 C/C VERTICALLY

314. Y12 @ 125 C/C HORIZONTAL LOOPS

315. Y16 @ 125 C/C HORIZONTAL LOOPS

316. Y10 @ 125 C/C VERTICALLY

317. Y12 @ 125 C/C HORIZONTAL LOOPS

318. Y16 @ 125 C/C HORIZONTAL LOOPS

319. Y10 @ 125 C/C VERTICALLY

320. Y12 @ 125 C/C HORIZONTAL LOOPS

321. Y16 @ 125 C/C HORIZONTAL LOOPS

322. Y10 @ 125 C/C VERTICALLY

323. Y12 @ 125 C/C HORIZONTAL LOOPS

324. Y16 @ 125 C/C HORIZONTAL LOOPS

325. Y10 @ 125 C/C VERTICALLY

326. Y12 @ 125 C/C HORIZONTAL LOOPS

327. Y16 @ 125 C/C HORIZONTAL LOOPS

328. Y10 @ 125 C/C VERTICALLY

329. Y12 @ 125 C/C HORIZONTAL LOOPS

330. Y16 @ 125 C/C HORIZONTAL LOOPS

331. Y10 @ 125 C/C VERTICALLY

332. Y12 @ 125 C/C HORIZONTAL LOOPS

333. Y16 @ 125 C/C HORIZONTAL LOOPS

334. Y10 @ 125 C/C VERTICALLY

335. Y12 @ 125 C/C HORIZONTAL LOOPS

336. Y16 @ 125 C/C HORIZONTAL LOOPS

337. Y10 @ 125 C/C VERTICALLY

338. Y12 @ 125 C/C HORIZONTAL LOOPS

339. Y16 @ 125 C/C HORIZONTAL LOOPS

340. Y10 @ 125 C/C VERTICALLY

341. Y12 @ 125 C/C HORIZONTAL LOOPS

342. Y16 @ 125 C/C HORIZONTAL LOOPS

343. Y10 @ 125 C/C VERTICALLY

344. Y12 @ 125 C/C HORIZONTAL LOOPS

345. Y16 @ 125 C/C HORIZONTAL LOOPS

346. Y10 @ 125 C/C VERTICALLY

347. Y12 @ 125 C/C HORIZONTAL LOOPS

348. Y16 @ 125 C/C HORIZONTAL LOOPS

349. Y10 @ 125 C/C VERTICALLY

350. Y12 @ 125 C/C HORIZONTAL LOOPS

351. Y16 @ 125 C/C HORIZONTAL LOOPS

352. Y10 @ 125 C/C VERTICALLY

353. Y12 @ 125 C/C HORIZONTAL LOOPS

354. Y16 @ 125 C/C HORIZONTAL LOOPS

355. Y10 @ 125 C/C VERTICALLY

356. Y12 @ 125 C/C HORIZONTAL LOOPS

357. Y16 @ 125 C/C HORIZONTAL LOOPS

358. Y10 @ 125 C/C VERTICALLY

359. Y12 @ 125 C/C HORIZONTAL LOOPS

360. Y16 @ 125 C/C HORIZONTAL LOOPS

361. Y10 @ 125 C/C VERTICALLY

362. Y12 @ 125 C/C HORIZONTAL LOOPS

363. Y16 @ 125 C/C HORIZONTAL LOOPS

364. Y10 @ 125 C/C VERTICALLY

365. Y12 @ 125 C/C HORIZONTAL LOOPS

366. Y16 @ 125 C/C HORIZONTAL LOOPS

367. Y10 @ 125 C/C VERTICALLY

368. Y12 @ 125 C/C HORIZONTAL LOOPS

369. Y16 @ 125 C/C HORIZONTAL LOOPS

370. Y10 @ 125 C/C VERTICALLY

371. Y12 @ 125 C/C HORIZONTAL LOOPS

372. Y16 @ 125 C/C HORIZONTAL LOOPS

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374. Y12 @ 125 C/C HORIZONTAL LOOPS

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377. Y12 @ 125 C/C HORIZONTAL LOOPS

378. Y16 @ 125 C/C HORIZONTAL LOOPS

379. Y10 @ 125 C/C VERTICALLY

380. Y12 @ 125 C/C HORIZONTAL LOOPS

381. Y16 @ 125 C/C HORIZONTAL LOOPS

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383. Y12 @ 125 C/C HORIZONTAL LOOPS

384. Y16 @ 125 C/C HORIZONTAL LOOPS

385. Y10 @ 125 C/C VERTICALLY

386. Y12 @ 125 C/C HORIZONTAL LOOPS

387. Y16 @ 125 C/C HORIZONTAL LOOPS

388. Y10 @ 125 C/C VERTICALLY

389. Y12 @ 125 C/C HORIZONTAL LOOPS

390. Y16 @ 125 C/C HORIZONTAL LOOPS

391. Y10 @ 125 C/C VERTICALLY

392. Y12 @ 125 C/C HORIZONTAL LOOPS

393. Y16 @ 125 C/C HORIZONTAL LOOPS

394. Y10 @ 125 C/C VERTICALLY

395. Y12 @ 125 C/C HORIZONTAL LOOPS

396. Y16 @ 125 C/C HORIZONTAL LOOPS

397. Y10 @ 125 C/C VERTICALLY

398. Y12 @ 125 C/C HORIZONTAL LOOPS

399. Y16 @ 125 C/C HORIZONTAL LOOPS

400. Y10 @ 125 C/C VERTICALLY

401. Y12 @ 125 C/C HORIZONTAL LOOPS

402. Y16 @ 125 C/C HORIZONTAL LOOPS

403. Y10 @ 125 C/C VERTICALLY

404. Y12 @ 125 C/C HORIZONTAL LOOPS

405. Y16 @ 125 C/C HORIZONTAL LOOPS

406. Y10 @ 125 C/C VERTICALLY

407. Y12 @ 125 C/C HORIZONTAL LOOPS

408. Y16 @ 125 C/C HORIZONTAL LOOPS

409. Y10 @ 125 C/C VERTICALLY

410. Y12 @ 125 C/C HORIZONTAL LOOPS

411. Y16 @ 125 C/C HORIZONTAL LOOPS

412. Y10 @ 125 C/C VERTICALLY

413. Y12 @ 125 C/C HORIZONTAL LOOPS

414. Y16 @ 125 C/C HORIZONTAL LOOPS

415. Y10 @ 125 C/C VERTICALLY

416. Y12 @ 125 C/C HORIZONTAL LOOPS

417. Y16 @ 125 C/C HORIZONTAL LOOPS

418. Y10 @ 125 C/C VERTICALLY

419. Y12 @ 125 C/C HORIZONTAL LOOPS

420. Y16 @ 125 C/C HORIZONTAL LOOPS

421. Y10 @ 125 C/C VERTICALLY

422. Y12 @ 125 C/C HORIZONTAL LOOPS

423. Y16 @ 125 C/C HORIZONTAL LOOPS

424. Y10 @ 125 C/C VERTICALLY

425. Y12 @ 125 C/C HORIZONTAL LOOPS

426. Y16 @ 125 C/C HORIZONTAL LOOPS

427. Y10 @ 125 C/C VERTICALLY

428. Y12 @ 125 C/C HORIZONTAL LOOPS

429. Y16 @ 125 C/C HORIZONTAL LOOPS

430. Y10 @ 125 C/C VERTICALLY

431. Y12 @ 125 C/C HORIZONTAL LOOPS

432. Y16 @ 125 C/C HORIZONTAL LOOPS

433. Y10 @ 125 C/C VERTICALLY

434. Y12 @ 125 C/C HORIZONTAL LOOPS

435. Y16 @ 125 C/C HORIZONTAL LOOPS

436. Y10 @ 125 C/C VERTICALLY

437. Y12 @ 125 C/C HORIZONTAL LOOPS

438. Y16 @ 125 C/C HORIZONTAL LOOPS

439. Y10 @ 125 C/C VERTICALLY

440. Y12 @ 125 C/C HORIZONTAL LOOPS

441. Y16 @ 125 C/C HORIZONTAL LOOPS

442. Y10 @ 125 C/C VERTICALLY

443. Y12 @ 125 C/C HORIZONTAL LOOPS

444. Y16 @ 125 C/C HORIZONTAL LOOPS

445. Y10 @ 125 C/C VERTICALLY

446. Y12 @ 125 C/C HORIZONTAL LOOPS

447. Y16 @ 125 C/C HORIZONTAL LOOPS

448. Y10 @ 125 C/C VERTICALLY

449. Y12 @ 125 C/C HORIZONTAL LOOPS

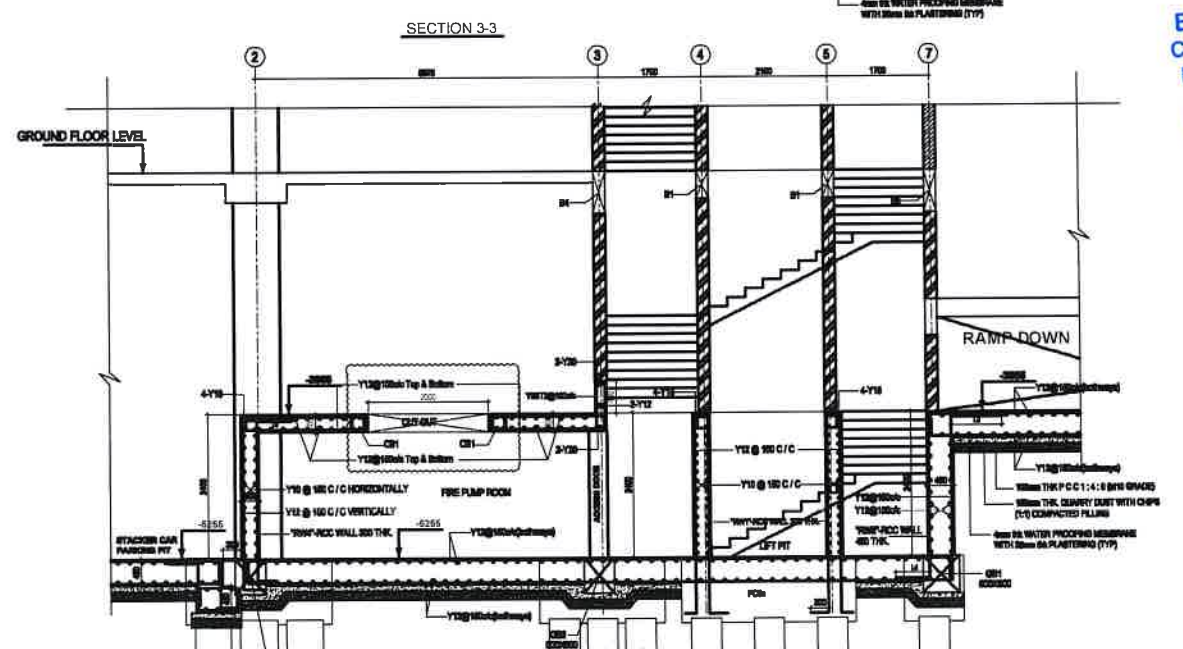
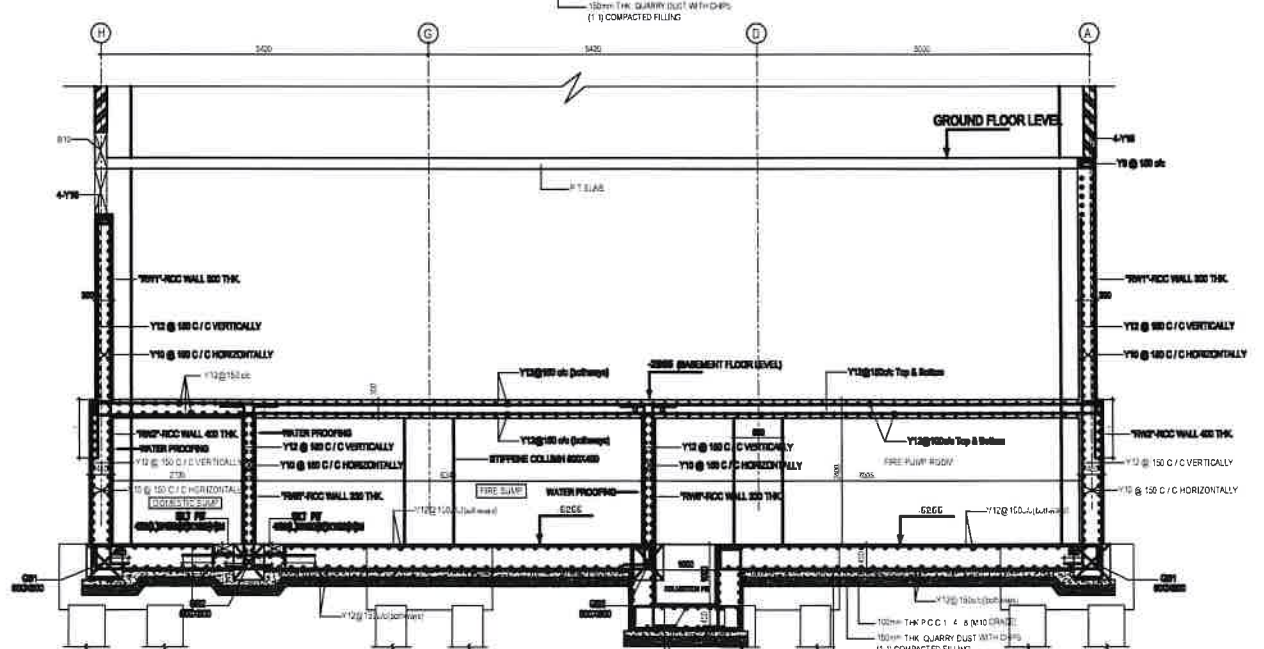
450. Y16 @ 125 C/C HORIZONTAL LOOPS

451. Y10 @ 125 C/C VERTICALLY

452. Y12 @ 125 C/C HORIZONTAL LOOPS

453. Y16 @ 125 C/C HORIZONTAL LOOPS

454. Y10 @ 125 C/C VERTICALLY

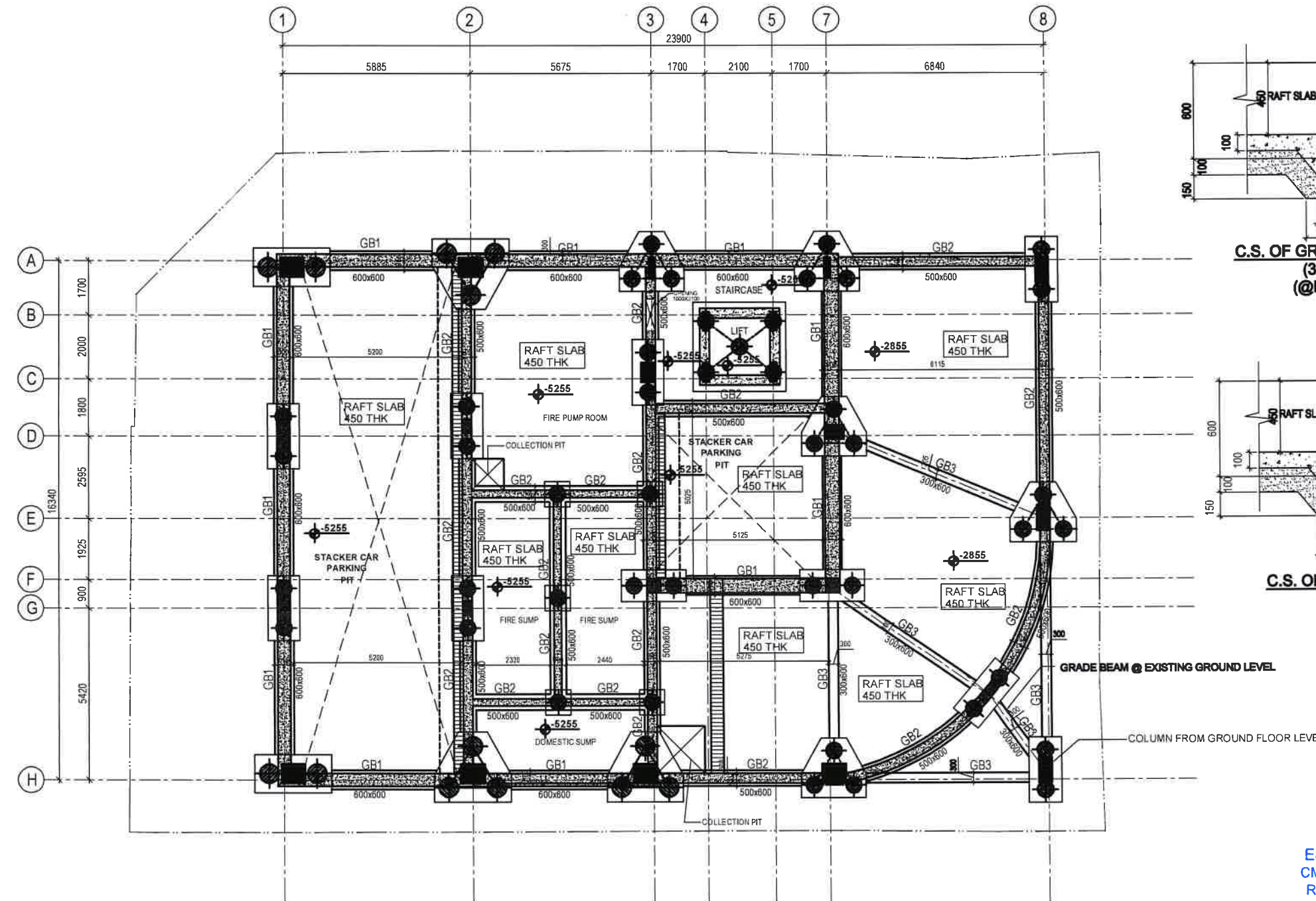


GENERAL NOTES:
MINIMUM OF 20% NOT IN SLIGHT RISK.
 THE FOLLOWING MINIMUM PERIOD (ATB FINAL FOUR) SHALL BE ALLOWED BEFORE REMOVAL OF SHUTTERING AS PER CLAUSE 11.3.1 OF IS:456 - 2000

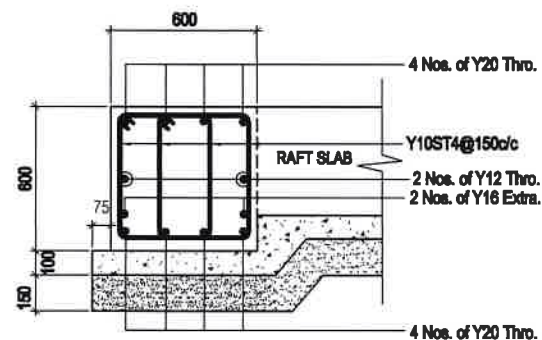
A. VERTICAL, SIDE OF SHUTTERING OF COLUMNS, WALLS AND BEAMS 24 HOURS
 B. BOTTOM SHUTTERING OF SLABS (KEEP THE PROPS UNDER) 30 DAYS
 C. BOTTOM SHUTTERING OF BEAMS (KEEP THE PROPS UNDER) 7 DAYS
 D. BOTTOM PROPS TO SLABS
 (i) SPANNING UP TO 4.0M 7 DAYS
 (ii) SPANNING OVER 4.0M 14 DAYS
 E. BOTTOM PROPS TO BEAMS
 (i) SPANNING UP TO 4.0M 14 DAYS
 (ii) SPANNING OVER 4.0M 21 DAYS

LAP JUNCTIONS FOR REINFORCEMENT BARS:
 (i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPED
 (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 SPECIAL REQUIREMENTS
 (iii) DEVELOPMENT LENGTH (L_{DE}) SHALL BE AS PER SP-34: 1999 AND GIVEN IN THE TABLE BELOW.

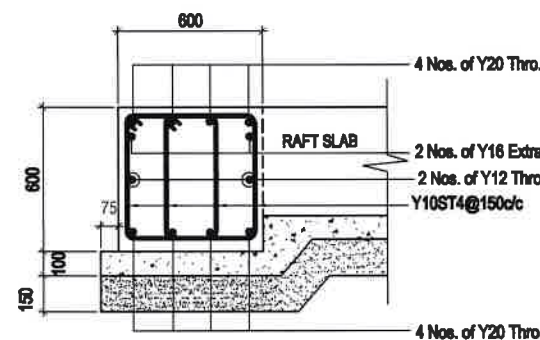
DIAMETER OF BARS	TENSION BARS	COMPRESSION BARS
10	30d	25d
12	35d	30d
16	45d	40d
20	55d	50d
25	65d	60d
32	80d	75d
40	100d	90d
50	125d	110d
63	160d	145d
80	200d	180d
100	250d	225d
125	315d	280d
160	400d	360d
200	500d	450d
250	625d	560d
315	787d	700d
400	1000d	900d
500	1250d	1125d
630	1600d	1400d
800	2000d	1800d
1000	2500d	2250d
1250	3150d	2800d
1600	4000d	3600d
2000	5000d	4500d
2500	6250d	5600d
3150	7875d	7000d
4000	10000d	9000d
5000	12500d	11250d
6300	16000d	14000d
8000	20000d	18000d
10000	25000d	22500d
12500	31500d	28000d
16000	40000d	36000d
20000	50000d	45000d
25000	62500d	56000d
31500	78750d	70000d
40000	100000d	90000d
50000	125000d	112500d
63000	160000d	140000d
80000	200000d	180000d
100000	250000d	225000d
125000	315000d	280000d
160000	400000d	360000d
200000	500000d	450000d
250000	625000d	560000d
315000	787500d	700000d
400000	1000000d	900000d
500000	1250000d	1125000d
630000	1600000d	1400000d
800000	2000000d	1800000d
1000000	2500000d	2250000d
1250000	3150000d	2800000d
1600000	4000000d	3600000d
2000000	5000000d	4500000d
2500000	6250000d	5600000d
3150000	7875000d	7000000d
4000000	10000000d	9000000d
5000000	12500000d	11250000d
6300000	16000000d	14000000d
8000000	20000000d	18000000d
10000000	25000000d	22500000d
12500000	31500000d	28000000d
16000000	40000000d	36000000d
20000000	50000000d	45000000d
25000000	62500000d	56000000d
31500000	78750000d	70000000d
40000000	100000000d	90000000d
50000000	125000000d	112500000d
63000000	160000000d	140000000d
80000000	200000000d	180000000d
100000000	250000000d	225000000d
125000000	315000000d	280000000d
160000000	400000000d	360000000d
200000000	500000000d	450000000d
250000000	625000000d	560000000d
315000000	787500000d	700000000d
400000000	1000000000d	900000000d
500000000	1250000000d	1125000000d
630000000	1600000000d	1400000000d
800000000	2000000000d	1800000000d
1000000000	2500000000d	2250000000d
1250000000	3150000000d	2800000000d
1600000000	4000000000d	3600000000d
2000000000	5000000000d	4500000000d
2500000000	6250000000d	5600000000d
3150000000	7875000000d	7000000000d
4000000000	10000000000d	9000000000d
5000000000	12500000000d	11250000000d
6300000000	16000000000d	14000000000



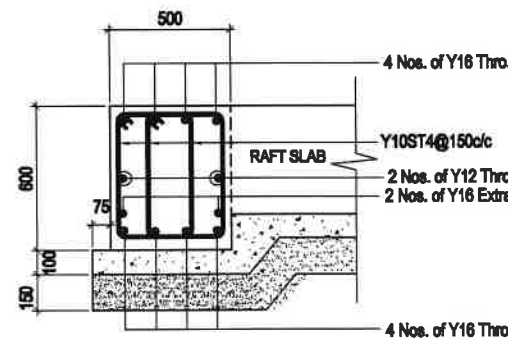
**GRADE BEAM LAYOUT
FOR T.O.C LEVELS - 2855 & - 5255**



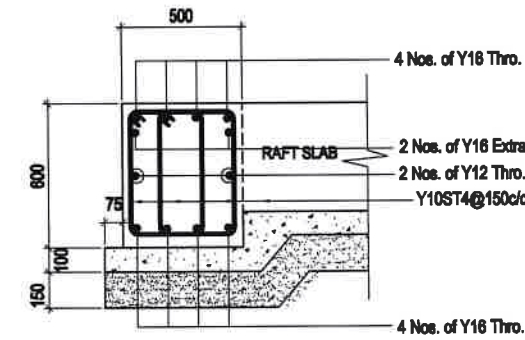
**C.S. OF GRADE BEAM - GB1
(600 x 600)
(@MID-SPAN)**



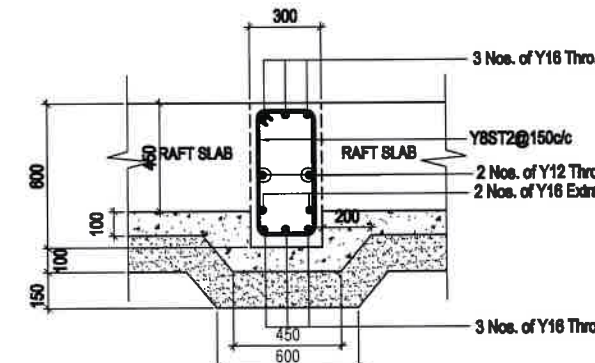
**C.S. OF GRADE BEAM - GB1
(600 x 600)
(@SUPPORT)**



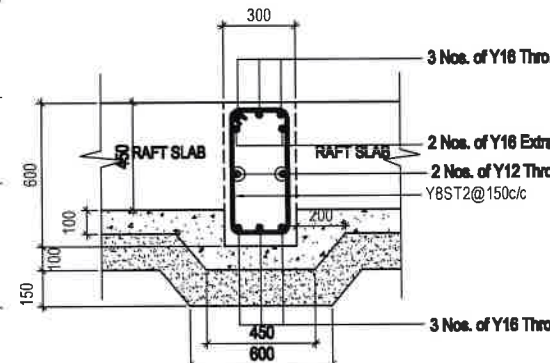
**C.S. OF GRADE BEAM - GB2
(500 x 600)
(@MID-SPAN)**



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



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



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

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

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 IS 456 : 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 (L_d) SHALL BE AS PER SP-34 : 1999 AND GIVEN IN THE TABLE BELOW

DIAMETER OF BARS	TENSION BARS				COMPRESSION BARS			
	M25	M30	M35	M40	M25	M30	M35	M40
8mm	450	400	370	320	370	320	290	250
10mm	550	480	450	400	450	380	350	300
12mm	650	560	530	480	530	470	440	380
16mm	850	750	720	640	720	630	600	520
20mm	1150	970	910	800	910	780	750	640
25mm	1410	1220	1140	1000	1140	970	910	800
32mm	1800	1550	1480	1300	1480	1250	1170	1020

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 3 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 CONFORM TO SP-34 & IS 1920.

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN mm.

2. ONLY WRITTEN DIMENSION IS TO BE FOLLOWED

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

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

5. GRADE OF CONCRETE SHALL CONFORM TO IS 456 : 2000

6. GRADE OF CONCRETE FOR COLUMN, PILE, PILE CAP, RCC WALL, RAFT SLAB AND GRADE BEAM - M30

7. 'Y' DENOTES HIGH STRENGTH DEFORMED BARS OF GRADE FE 500 CONFORMING TO IS 1786 : 2008

8. CLEAR COVER TO THE REINFORCEMENT

A. COLUMNS _____ 40mm

B. PILE _____ 75mm

C. PILE CAP BOTTOM - 75mm, TOP & SIDES - 50mm

D. RAFT SLAB _____ 50mm

E. GRADE BEAM _____ 40mm

DIAM. OF PILE (MM)	SAFE LOAD IN COMPRESSION (T)
450 MM (P1)	137 T
300 MM (P2)	73 T

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

10. 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

11. 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

12. 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	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.	REF. CODE
KA-	S-07	04	

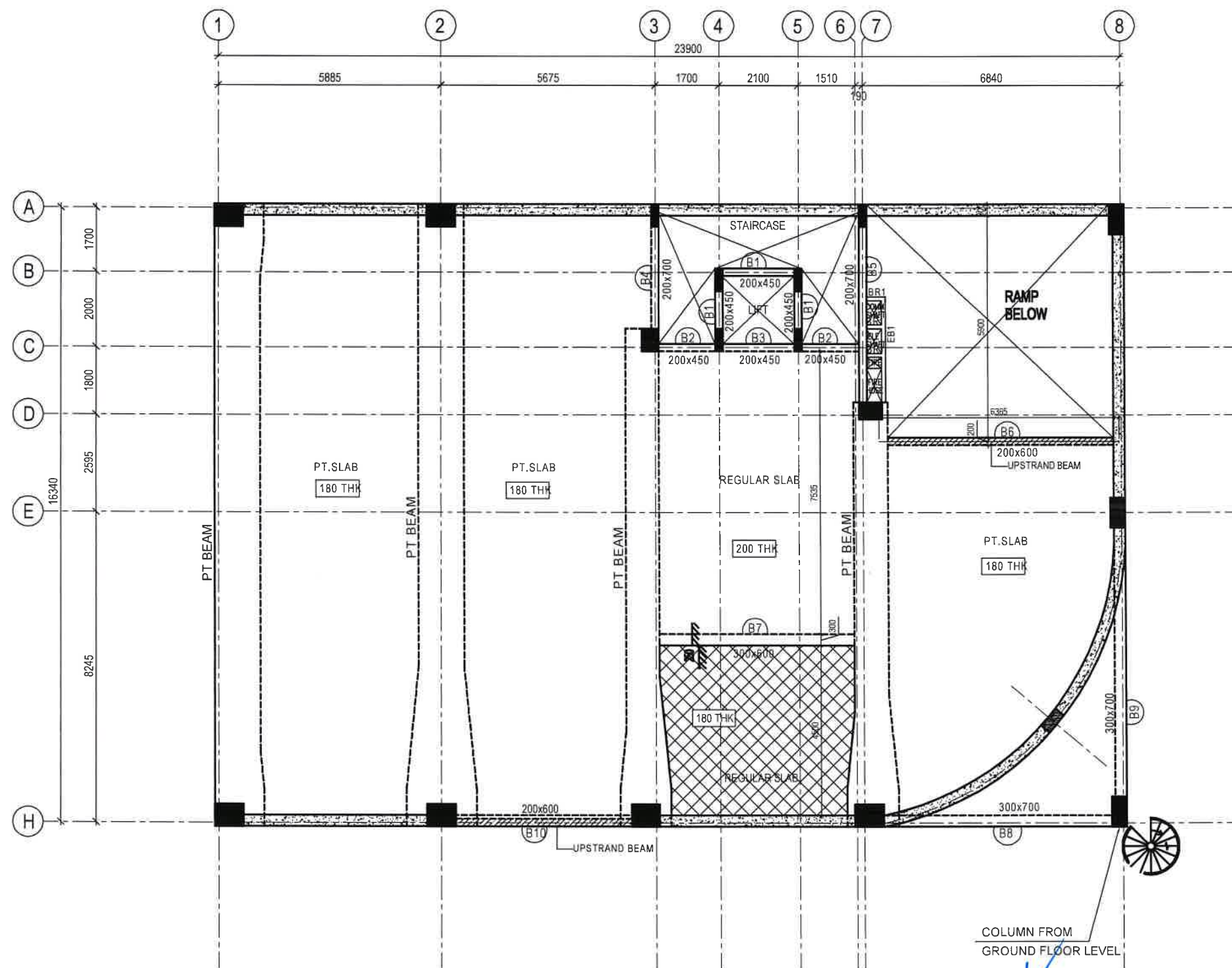
STRUCTURAL CONSULTANTS:

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/A/46, 1st FLOOR, 3rd AVENUE,
ANNA NAGAR EAST, CHENNAI - 600002
Tel: 044-26202147

Drawn	Designed	Checked
M.D	DANE	ES. KUMAR



BASEMENT ROOF BEAM LAYOUT

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

LEGEND

= SUNKEN BY 250

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

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 IS 456 : 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			
	M25	M35	M40	M50	M25	M35	M40	M50
6mm	450	450	450	450	350	350	350	350
8mm	480	480	480	480	380	380	380	380
10mm	500	500	500	500	400	400	400	400
12mm	520	520	520	520	420	420	420	420
14mm	540	540	540	540	440	440	440	440
16mm	560	560	560	560	460	460	460	460
18mm	580	580	580	580	480	480	480	480
20mm	600	600	600	600	500	500	500	500
22mm	620	620	620	620	520	520	520	520
25mm	650	650	650	650	550	550	550	550
30mm	700	700	700	700	600	600	600	600

- 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: BASEMENT ROOF BEAM LAYOUT

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

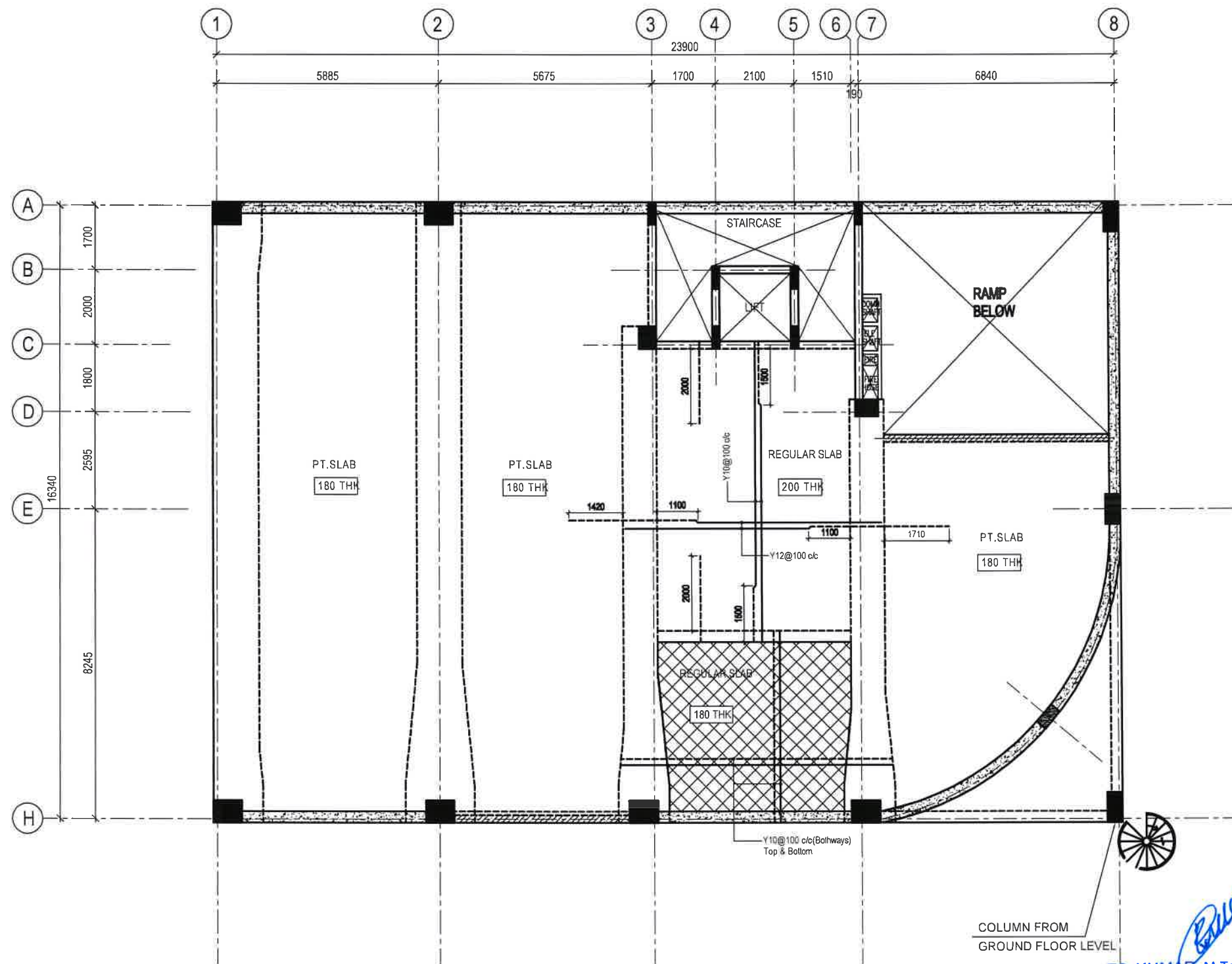
STRUCTURAL CONSULTANTS:

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/A-4B - 1st FLOOR, 3rd AVENUE
ANNA NAGAR EAST, CHENNAI - 600102.
Tel: 044 - 26233147.

Drawn M.D.	Designed ES	Checked ES KUMAR
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BASEMENT ROOF SLAB LAYOUT

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

LEGEND

[Hatched Box] = 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 _____ 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			
	M25	M30	M35	M40	M25	M30	M35	M40
6mm	450	420	375	320	370	320	280	280
8mm	585	495	420	360	460	380	320	320
10mm	680	560	465	390	530	440	360	360
12mm	800	660	540	450	630	520	420	420
16mm	1000	840	690	570	800	660	530	530
20mm	1130	970	810	660	910	760	630	630
25mm	1410	1220	1000	840	1140	970	810	810
32mm	1800	1560	1280	1060	1480	1260	1060	1060

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.
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REV.	DESCRIPTION	DRN	CHK	DATE
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CLIENT: EMPEE GROUP

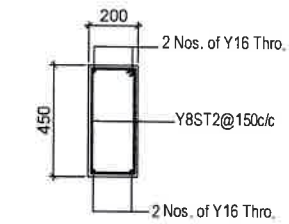
DRAWING TITLE: BASEMENT ROOF SLAB LAYOUT

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

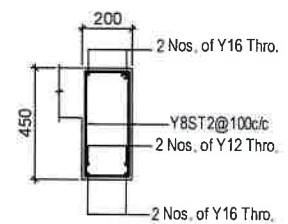
STRUCTURAL CONSULTANTS:
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
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Chennai - 600 040. Mob: 98404 48613.

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

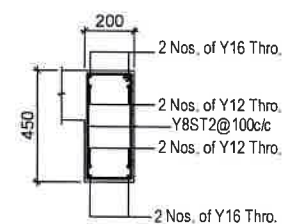
Drawn	Designed	Checked
MD	ES	ES KUMAR



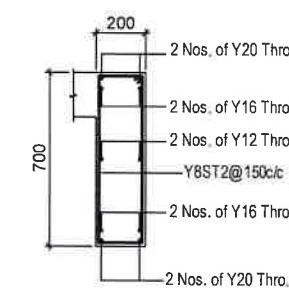
C.S. OF BEAM -"B1"
(200 X 450)



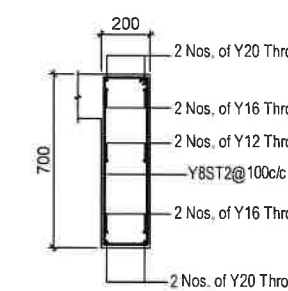
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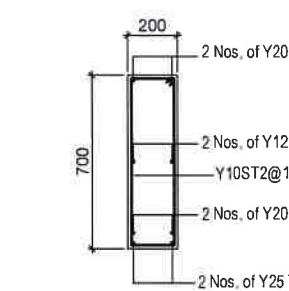
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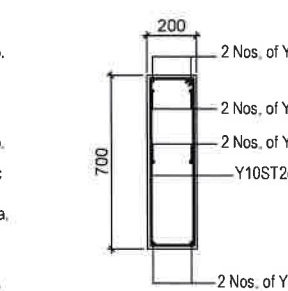
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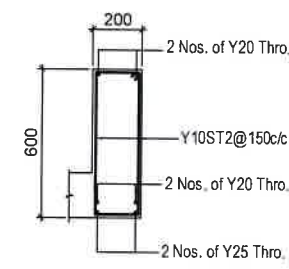
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@SUPPORT



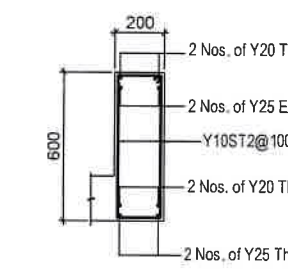
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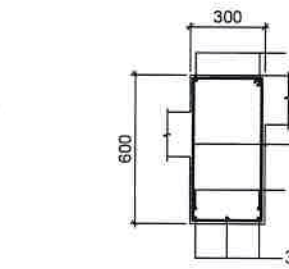
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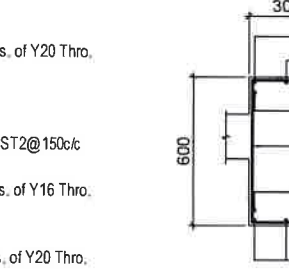
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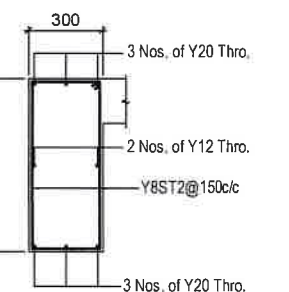
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(200 X 600)
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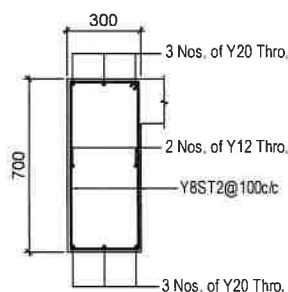
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@MID-SPAN



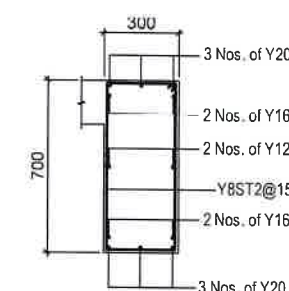
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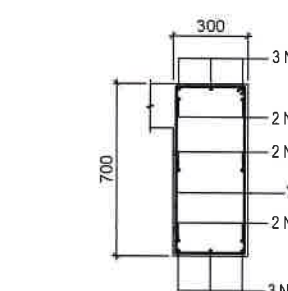
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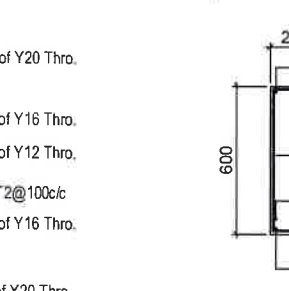
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(300 X 700)
@SUPPORT



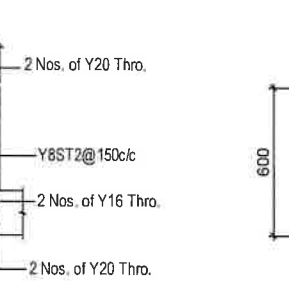
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(300 X 700)
@MID-SPAN



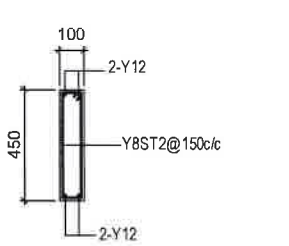
C.S. OF BEAM -"B9"
(300 X 700)
@SUPPORT



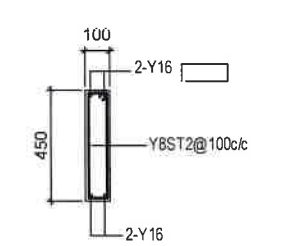
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(200 X 600)
@MID-SPAN



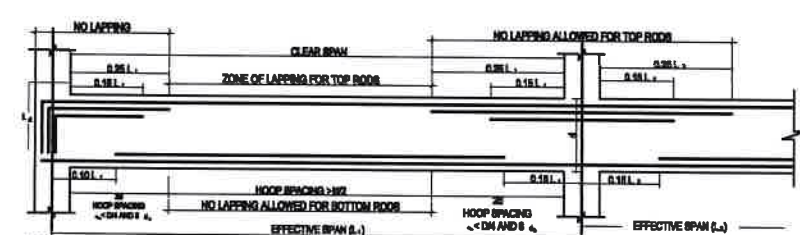
C.S. OF BEAM -"B10"
(200 X 600)
@MID-SPAN



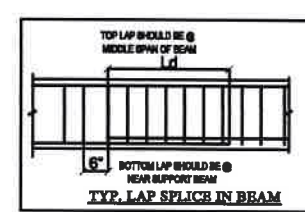
DUCT EDGE BEAM
EB1 (100 X 450)



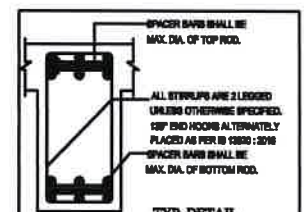
DUCT BRACKET BEAM
BR1 (100 X 450)



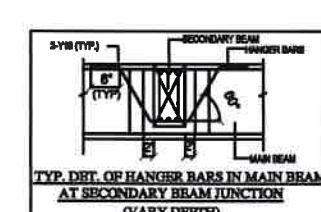
NOTE:
0.15 L_e SHOULD NOT BE LESS THAN
DEVELOPING LENGTH () = 50 L_d d_b
d_b = DIAMETER OF LONGITUDINAL BAR



TYP. LAP SPLICING 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):
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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
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i). AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED.
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iii). DEVELOPMENT LENGTH (L_d) 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	450	370	320	370	320	280	280
10mm	550	450	400	350	400	350	300	320
12mm	650	550	500	450	500	450	400	420
16mm	800	700	650	600	650	600	550	580
20mm	1100	970	910	850	910	850	780	840
25mm	1410	1220	1140	1050	1140	1050	970	1050
32mm	1800	1580	1480	1380	1480	1380	1280	1380

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C. ROOF SLAB 25mm
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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

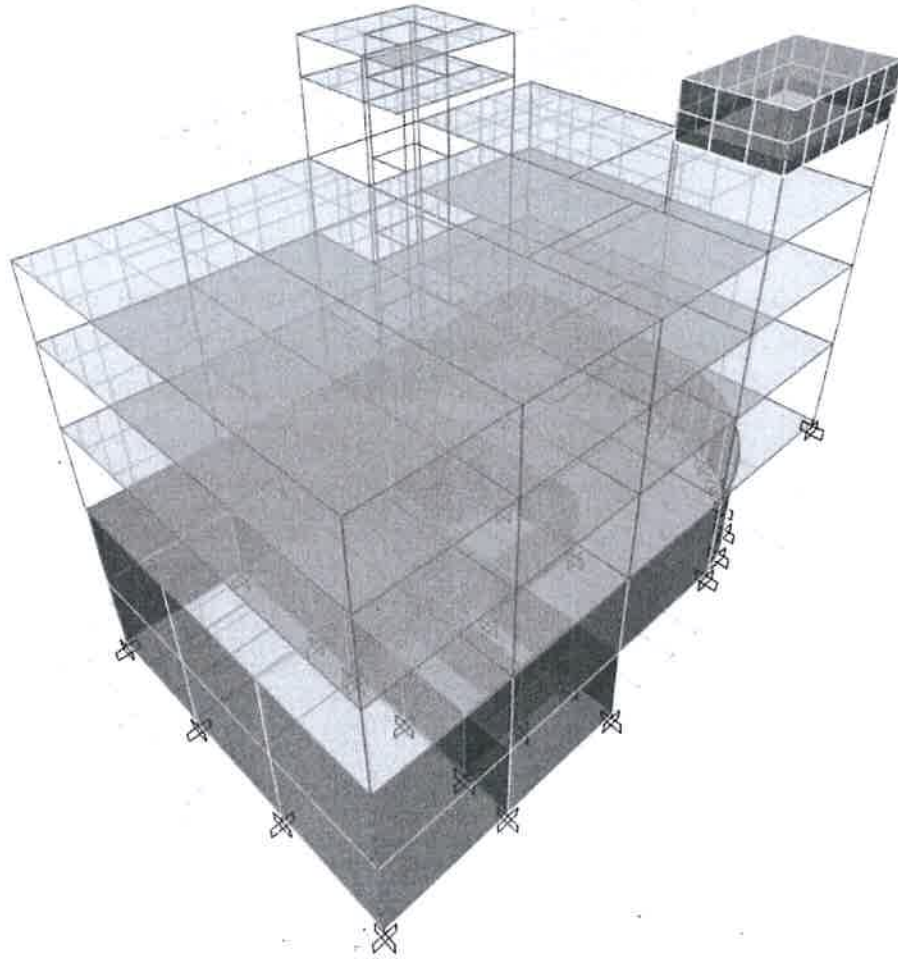
STRUCTURAL CONSULTANTS:
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO. 1233, FIRST FLOOR,
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ARCHITECTS:
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8109 / A-8, 1st FLOOR, 3rd AVENUE,
ANNA NAGAR EAST, CHENNAI - 600102.
Tel: 944 - 26203147

Drawn M.D. Designed ES Checked ES KUMAR

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

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



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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
2 ND SLAB LVL	3.70	m
1 ST 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



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Mob: 98404 48613

eMail Id: kirtiassociates@gmail.com

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



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Mob: 98404 48613

eMail Id: kirtiassociates@gmail.com

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



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Mob: 98404 48613

eMail Id: kirtiassociates@gmail.com

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.



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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	



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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/m² 75 % reduction, above 3 kN/m² 50% reduction)

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

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

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

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

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

Where, Z for Zone III	=	0.16
I for Commercial building	=	1.0
R for MRF	=	3.0



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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

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* 23 47 13 * **DESIGN OF STRUCTURE**

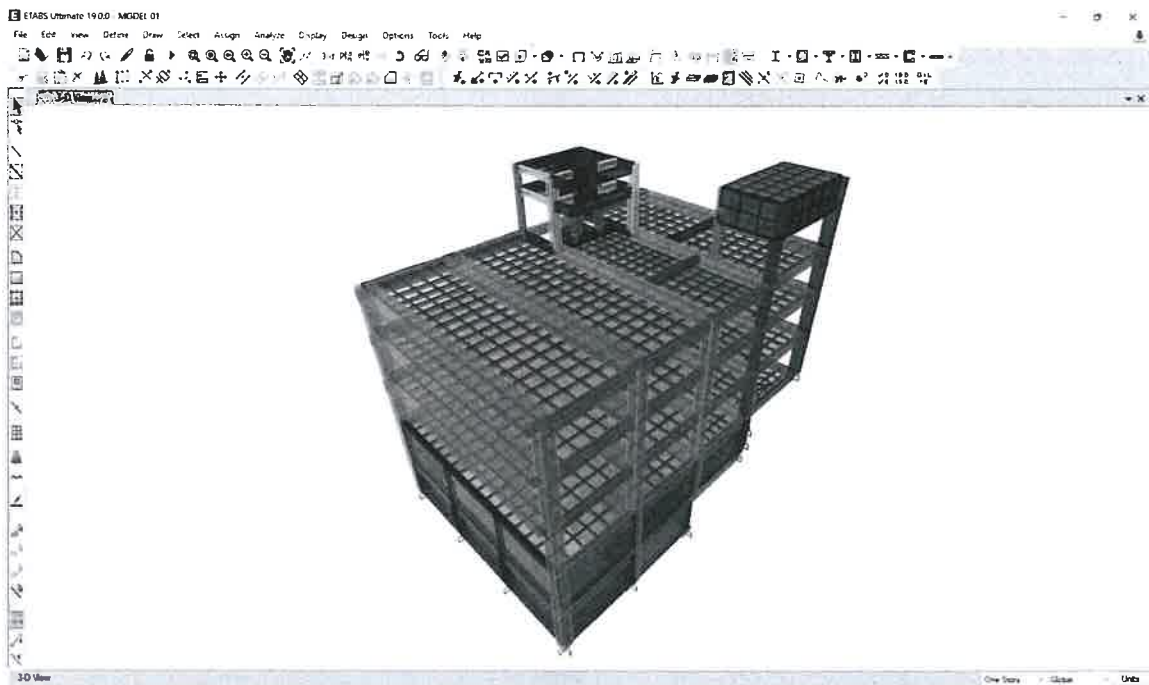


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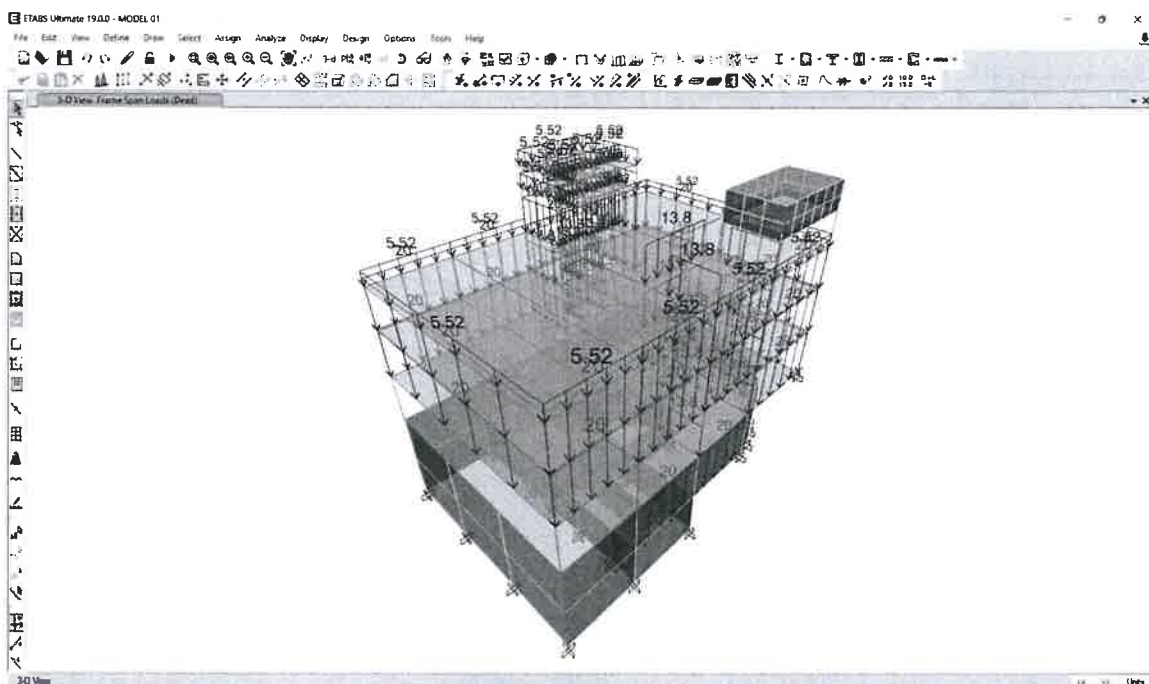
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3d view – rendered image



Wall load – dead load

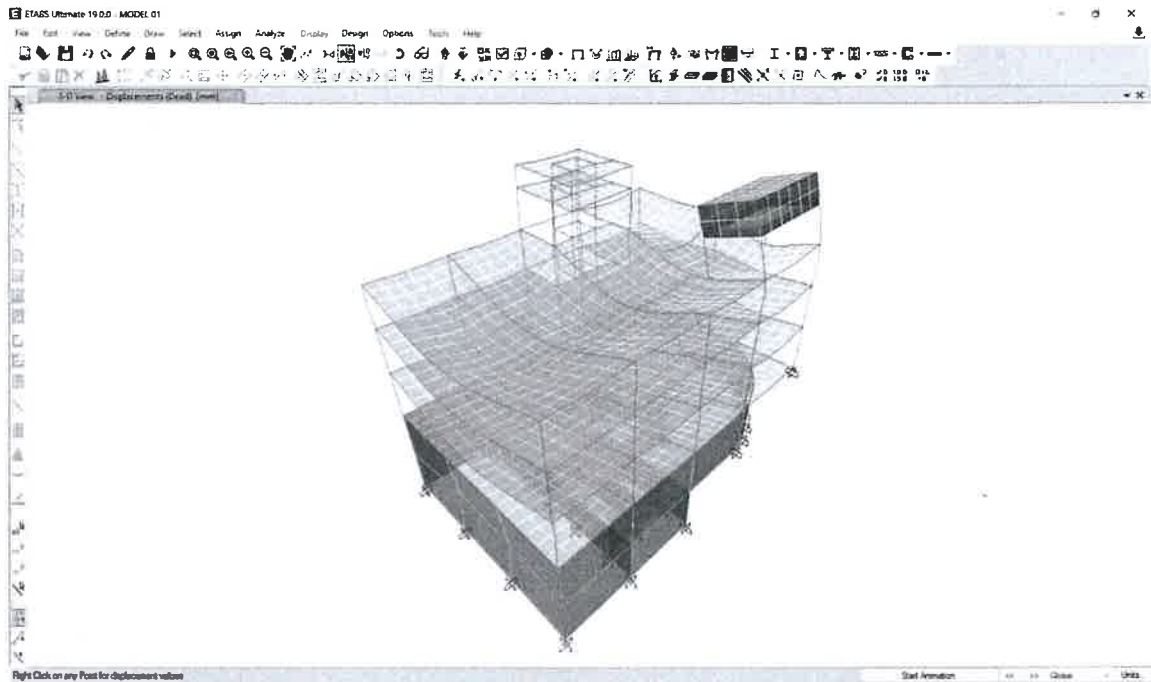


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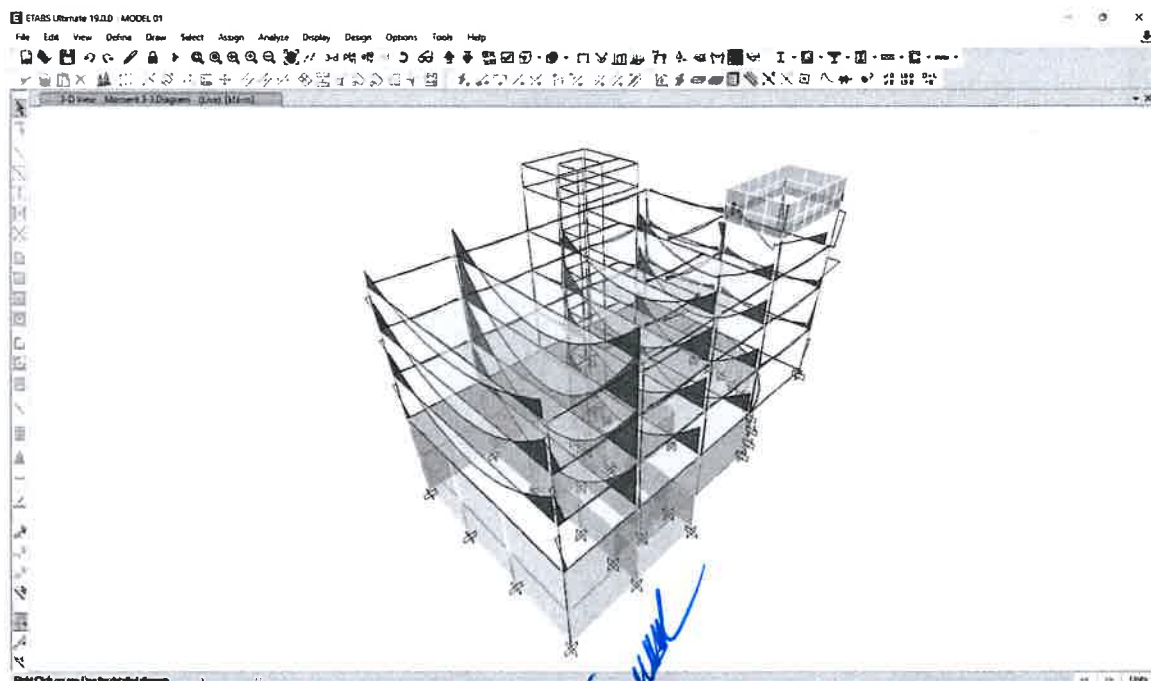
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Model displacement



Model moment check

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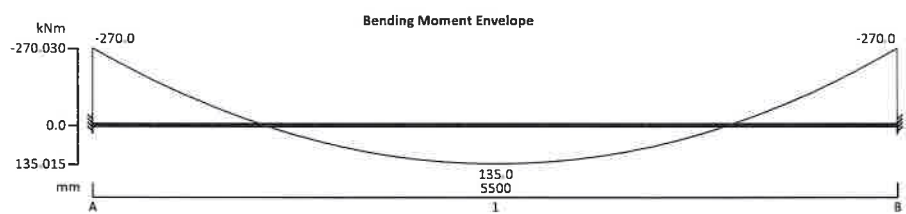
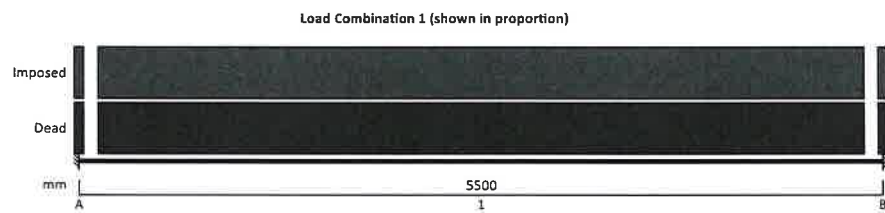
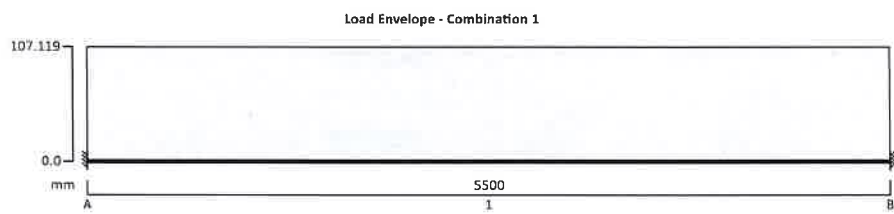
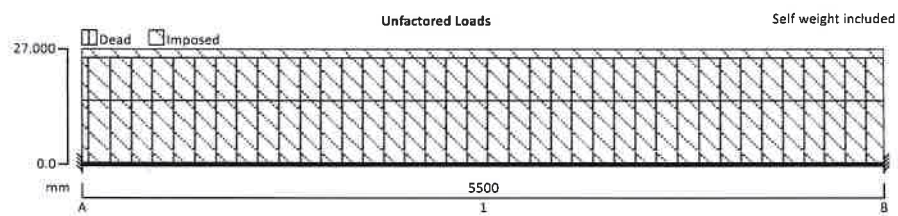
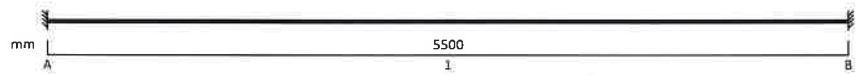
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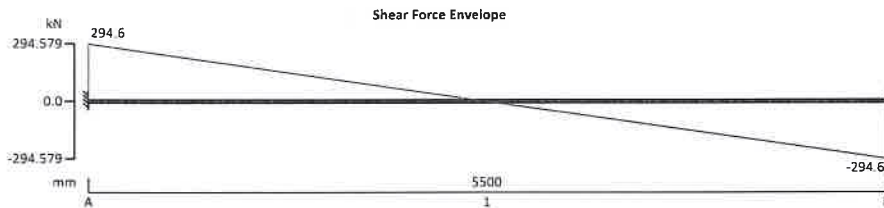
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DESIGN CALCULATION

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 $\times 1$

Load combinations

Load combination 1	Support A	Dead $\times 1.50$ Imposed $\times 1.50$
	Span 1	Dead $\times 1.50$ Imposed $\times 1.50$
	Support B	Dead $\times 1.50$ Imposed $\times 1.50$

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	

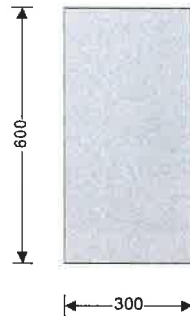
Rectangular section details

Section width;

$b = 300$ mm

Section depth;

$h = 600$ mm



Concrete details

Concrete strength class;

C20/25

Characteristic compressive cube strength;

$f_{cu} = 25$ N/mm²

Modulus of elasticity of concrete;

$E_c = 20 \text{ kN/mm}^2 + 200 \times f_{cu} = 25000$ N/mm²

Maximum aggregate size;

$h_{agg} = 20$ mm

Reinforcement details

Characteristic yield strength of reinforcement;

$f_y = 500$ N/mm²

Characteristic yield strength of shear reinforcement; $f_{yv} = 500$ N/mm²

Nominal cover to reinforcement

Nominal cover to top reinforcement;

$c_{nom_t} = 35$ mm

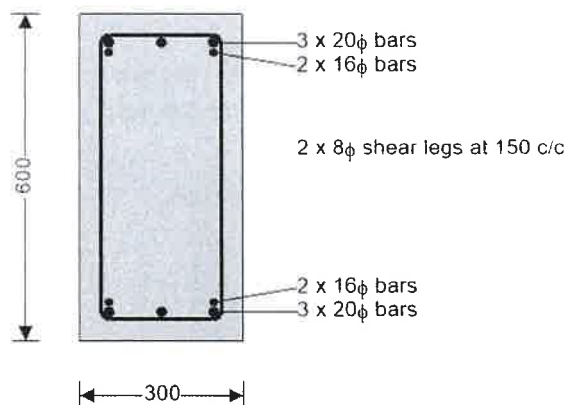
Nominal cover to bottom reinforcement;

$c_{nom_b} = 35$ mm

Nominal cover to side reinforcement;

$c_{nom_s} = 35$ mm

Support A



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1;

3 x 20φ bars

Area of reinforcement provided - layer 1;

$A_{s_L1} = 942 \text{ mm}^2$

Depth to layer 1;

$d_{L1} = 53 \text{ mm}$

Reinforcement provided - layer 2;

2 x 16φ bars

Area of reinforcement provided - layer 2;

$A_{s_L2} = 402 \text{ mm}^2$

Depth to layer 2;

$d_{L2} = 71 \text{ mm}$

Total area of reinforcement;

$A_{s2,prov} = A_{s_L1} + A_{s_L2} = 1345 \text{ mm}^2$

Centroid of reinforcement;

$d_{bot} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s2,prov} = 58 \text{ mm}$

Multiple layers of top reinforcement

Reinforcement provided - layer 1;

3 x 20φ bars

Area of reinforcement provided - layer 1;

$A_{s_L1} = 942 \text{ mm}^2$

Depth to layer 1;

$d_{L1} = 547 \text{ mm}$

Reinforcement provided - layer 2;

2 x 16φ bars

Area of reinforcement provided - layer 2;

$A_{s_L2} = 402 \text{ mm}^2$

Depth to layer 2;

$d_{L2} = 529 \text{ mm}$

Total area of reinforcement;

$A_{s,prov} = A_{s_L1} + A_{s_L2} = 1345 \text{ mm}^2$

Centroid of reinforcement;

$d_{top} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s,prov} = 542 \text{ mm}$

Rectangular section in flexure

Design bending moment;

$M = \text{abs}(M_{A_red}) = 257 \text{ kNm}$

Depth to tension reinforcement;

$d = d_{top} = 542 \text{ mm}$

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;

$x = (d - z) / 0.45 = 184 \text{ mm}$

Area of tension reinforcement required;

$A_{s,req} = M / (0.87 \times f_y \times z) = 1285 \text{ mm}^2$

Tension reinforcement provided;

3 x 20 φbars + 2 x 16 φbars

Area of tension reinforcement provided;

$A_{s,prov} = 1345 \text{ mm}^2$

Minimum area of reinforcement;

$A_{s,min} = 0.0013 \times b \times h = 234 \text{ mm}^2$

Maximum area of reinforcement;

$A_{s,max} = 0.04 \times b \times h = 7200 \text{ mm}^2$

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

Rectangular section in shear

Design shear force span 1 at 542 mm;

$V = \max(V_{A_s1_max}, V_{A_s1_red}) = 236 \text{ kN}$

Design shear stress;

$v = V / (b \times d) = 1.450 \text{ N/mm}^2$

Design concrete shear stress;
 $(400/d)^{1/4} \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$$v_c = 0.79 \times \min(3, [100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max\{1,$$

$$v_c = 0.593 \text{ N/mm}^2$$

Allowable design shear stress;
4.000 N/mm²

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

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;

$$v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) = 0.857 \text{ N/mm}^2$$

Area of shear reinforcement required;

$$A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = 591 \text{ mm}^2/\text{m}$$

Shear reinforcement provided;

$$2 \times 8\phi \text{ legs at } 150 \text{ c/c}$$

Area of shear reinforcement provided;

$$A_{sv,prov} = 670 \text{ mm}^2/\text{m}$$

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing;

$$s_{vl,max} = 0.75 \times d = 406 \text{ mm}$$

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

Spacing of reinforcement

Actual distance between bars in tension;
77 mm

$$s = (b - 2 \times (C_{nom,s} + \phi_v + \phi_{top,L1}/2)) / (N_{top,L1} - 1) - \phi_{top,L1} =$$

Minimum distance between bars in tension

Minimum distance between bars in tension;

$$s_{min} = h_{agg} + 5 \text{ mm} = 25 \text{ mm}$$

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension

Design service stress;

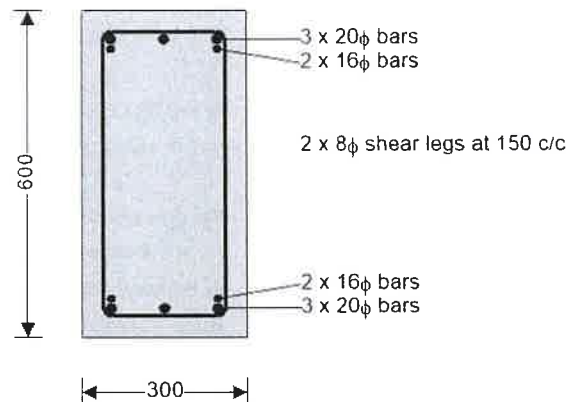
$$f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) = 335.4 \text{ N/mm}^2$$

Maximum distance between bars in tension;

$$s_{max} = \min(47000 \text{ N/mm} / f_s, 300 \text{ mm}) = 140 \text{ mm}$$

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

Multiple layers of top reinforcement

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

Design moment resistance of rectangular section - Positive moment

Design bending moment;	$M = \text{abs}(M_{s1_red}) = 149 \text{ kNm}$
Depth to tension reinforcement;	$d = d_{bot} = 542 \text{ mm}$
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;	$z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = 497 \text{ mm}$
Depth of neutral axis;	$x = (d - z) / 0.45 = 98 \text{ mm}$
Area of tension reinforcement required;	$A_{s,req} = M / (0.87 \times f_y \times z) = 686 \text{ mm}^2$
Tension reinforcement provided;	3 × 20 φbars + 2 × 16 φbars
Area of tension reinforcement provided;	$A_{s,prov} = 1345 \text{ mm}^2$
Minimum area of reinforcement;	$A_{s,min} = 0.0013 \times b \times h = 234 \text{ mm}^2$
Maximum area of reinforcement;	$A_{s,max} = 0.04 \times b \times h = 7200 \text{ mm}^2$

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;	$A_{sv,prov} = 670 \text{ mm}^2/\text{m}$

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

PASS - Area of shear reinforcement provided exceeds minimum required

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

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

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

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

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

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

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; $s = (b - 2 \times (c_{nom,s} + \phi_v + \phi_{bot,L1}/2)) / (N_{bot,L1} - 1) - \phi_{bot,L1} = 77 \text{ mm}$

Minimum distance between bars in tension

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

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension

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

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

PASS - Satisfies the maximum spacing criteria

Span to depth ratio

Basic span to depth ratio; $\text{span_to_depth}_{basic} = 20.0$

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

Modification for tension reinforcement

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

Modification for compression reinforcement

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

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

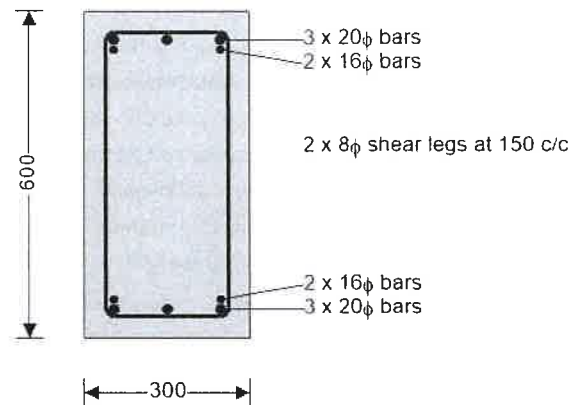
Allowable span to depth ratio; $\text{span_to_depth}_{allow} = \text{span_to_depth}_{basic} \times f_{tens} \times f_{comp} =$

35.9

Actual span to depth ratio; $\text{span_to_depth}_{actual} = L_{s1} / d = 10.2$

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

Support B



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1;

3 x 20φ bars

Area of reinforcement provided - layer 1;

$A_{s_L1} = 942 \text{ mm}^2$

Depth to layer 1;

$d_{L1} = 53 \text{ mm}$

Reinforcement provided - layer 2;

2 x 16φ bars

Area of reinforcement provided - layer 2;

$A_{s_L2} = 402 \text{ mm}^2$

Depth to layer 2;

$d_{L2} = 71 \text{ mm}$

Total area of reinforcement;

$A_{s,prov} = A_{s_L1} + A_{s_L2} = 1345 \text{ mm}^2$

Centroid of reinforcement;

$d_{bot} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s,prov} = 58 \text{ mm}$

Multiple layers of top reinforcement

Reinforcement provided - layer 1;

3 x 20φ bars

Area of reinforcement provided - layer 1;

$A_{s_L1} = 942 \text{ mm}^2$

Depth to layer 1;

$d_{L1} = 547 \text{ mm}$

Reinforcement provided - layer 2;

2 x 16φ bars

Area of reinforcement provided - layer 2;

$A_{s_L2} = 402 \text{ mm}^2$

Depth to layer 2;

$d_{L2} = 529 \text{ mm}$

Total area of reinforcement;

$A_{s,prov} = A_{s_L1} + A_{s_L2} = 1345 \text{ mm}^2$

Centroid of reinforcement;

$d_{top} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s,prov} = 542 \text{ mm}$

Rectangular section in flexure

Design bending moment;

$M = \text{abs}(M_{B_red}) = 257 \text{ kNm}$

Depth to tension reinforcement;

$d = d_{top} = 542 \text{ mm}$

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

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

Lever arm;

mm

Depth of neutral axis;

$x = (d - z) / 0.45 = 184 \text{ mm}$

Area of tension reinforcement required;

$A_{s,req} = M / (0.87 \times f_y \times z) = 1285 \text{ mm}^2$

Tension reinforcement provided;

3 x 20 φbars + 2 x 16 φbars

Area of tension reinforcement provided;

$A_{s,prov} = 1345 \text{ mm}^2$

Minimum area of reinforcement;

$A_{s,min} = 0.0013 \times b \times h = 234 \text{ mm}^2$

Maximum area of reinforcement;

$A_{s,max} = 0.04 \times b \times h = 7200 \text{ mm}^2$

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

Rectangular section in shear

Design shear force span 1 at 4958 mm;

$V = \text{abs}(\min(V_{B_s1_max}, V_{B_s1_red})) = 236 \text{ kN}$

Design shear stress;

$v = V / (b \times d) = 1.450 \text{ N/mm}^2$

Design concrete shear stress;
 $(400/d)^{1/4} \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

Allowable design shear stress;
4.000 N/mm²

$$v_c = 0.79 \times \min(3, [100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1,$$

$$v_c = \mathbf{0.593 \text{ N/mm}^2}$$

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

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;

$$v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) = \mathbf{0.857 \text{ N/mm}^2}$$

Area of shear reinforcement required;

$$A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = \mathbf{591 \text{ mm}^2/m}$$

Shear reinforcement provided;

$$2 \times 8\phi \text{ legs at } 150 \text{ c/c}$$

Area of shear reinforcement provided;

$$A_{sv,prov} = \mathbf{670 \text{ mm}^2/m}$$

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing;

$$s_{vl,max} = 0.75 \times d = \mathbf{406 \text{ mm}}$$

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

Spacing of reinforcement

Actual distance between bars in tension;

$$s = (b - 2 \times (C_{nom,s} + \phi_v + \phi_{top,L1}/2)) / (N_{top,L1} - 1) - \phi_{top,L1} =$$

77 mm

Minimum distance between bars in tension

Minimum distance between bars in tension;

$$s_{min} = h_{agg} + 5 \text{ mm} = \mathbf{25 \text{ mm}}$$

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension

Design service stress;

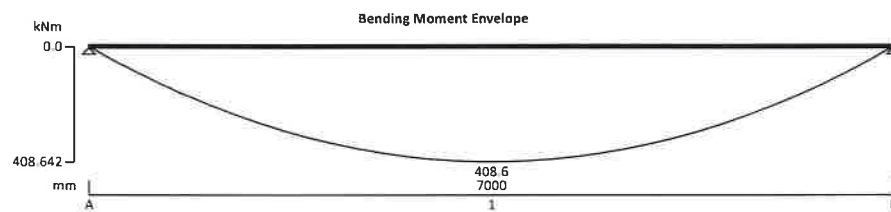
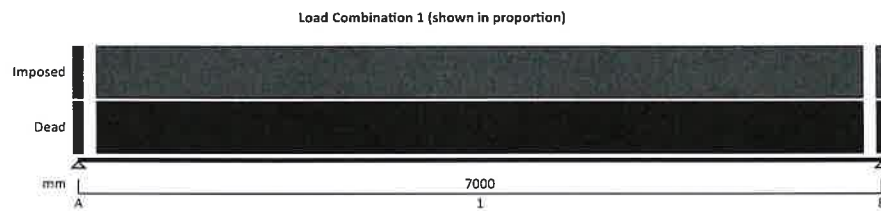
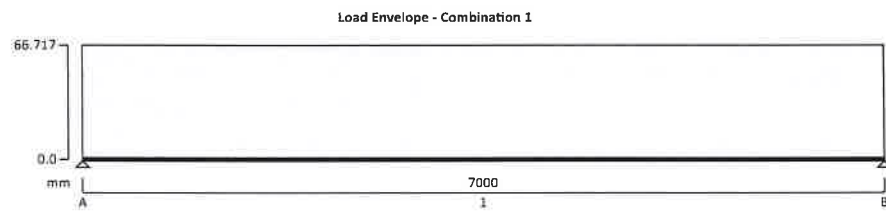
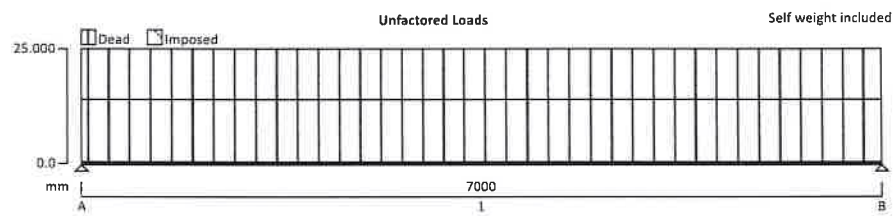
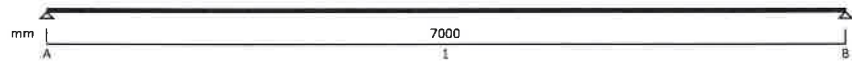
$$f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) = \mathbf{335.4 \text{ N/mm}^2}$$

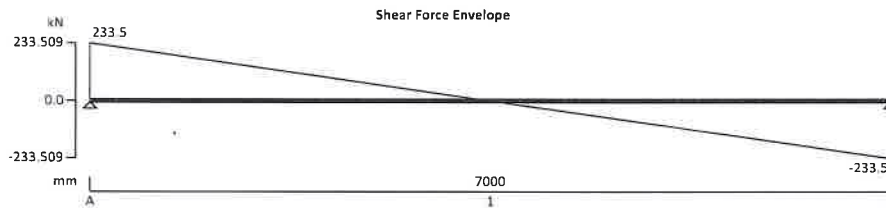
Maximum distance between bars in tension;

$$s_{max} = \min(47000 \text{ N/mm} / f_s, 300 \text{ mm}) = \mathbf{140 \text{ mm}}$$

PASS - Satisfies the maximum spacing criteria

RC BEAM ANALYSIS & DESIGN





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 $\times 1$

Load combinations

Load combination 1	Support A	Dead $\times 1.50$ Imposed $\times 1.50$
	Span 1	Dead $\times 1.50$ Imposed $\times 1.50$
	Support B	Dead $\times 1.50$ Imposed $\times 1.50$

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;

$f_{cu} = 35 \text{ N/mm}^2$

Modulus of elasticity of concrete;

$E_c = 20 \text{ kN/mm}^2 + 200 \times f_{cu} = 27000 \text{ N/mm}^2$

Maximum aggregate size;

$h_{agg} = 20 \text{ mm}$

Reinforcement details

Characteristic yield strength of reinforcement;

$f_y = 500 \text{ N/mm}^2$

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

Nominal cover to reinforcement

Nominal cover to top reinforcement;

$C_{nom_t} = 25 \text{ mm}$

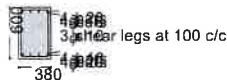
Nominal cover to bottom reinforcement;

$C_{nom_b} = 25 \text{ mm}$

Nominal cover to side reinforcement;

$C_{nom_s} = 25 \text{ mm}$

Support A



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1;

$4 \times 20\phi$ bars

Area of reinforcement provided - layer 1;

$A_{s_L1} = 1257 \text{ mm}^2$

Depth to layer 1;

$d_{L1} = 555 \text{ mm}$

Reinforcement provided - layer 2;

$4 \times 16\phi$ bars

Area of reinforcement provided - layer 2;

$A_{s_L2} = 804 \text{ mm}^2$

Depth to layer 2;

$d_{L2} = 537 \text{ mm}$

Total area of reinforcement;

$A_{s,prov} = A_{s_L1} + A_{s_L2} = 2061 \text{ mm}^2$

Centroid of reinforcement;

$d_{bot} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s,prov} = 548 \text{ mm}$

Multiple layers of top reinforcement

Reinforcement provided - layer 1;

$4 \times 20\phi$ bars

Area of reinforcement provided - layer 1;

$A_{s_L1} = 1257 \text{ mm}^2$

Depth to layer 1;

$d_{L1} = 45 \text{ mm}$

Reinforcement provided - layer 2;

$4 \times 16\phi$ bars

Area of reinforcement provided - layer 2;
 Depth to layer 2;
 Total area of reinforcement;
 Centroid of reinforcement;

$$A_{s_L2} = 804 \text{ mm}^2$$

$$d_{L2} = 63 \text{ mm}$$

$$A_{s2,prov} = A_{s_L1} + A_{s_L2} = 2061 \text{ mm}^2$$

$$d_{top} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s2,prov} = 52 \text{ mm}$$

Rectangular section in shear

Design shear force span 1 at 548 mm;
 Design shear stress;
 Design concrete shear stress;
 $(400/d)^{1/4} \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$$V = \max(V_{A_s1_max}, V_{A_s1_red}) = 197 \text{ kN}$$

$$v = V / (b \times d) = 0.945 \text{ N/mm}^2$$

$$v_c = 0.79 \times \min(3, [100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max\{1,$$

Allowable design shear stress;
4.733 N/mm²

$$v_c = 0.705 \text{ N/mm}^2$$

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

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7;
 Design shear resistance required;
 Area of shear reinforcement required;
 Shear reinforcement provided;
 Area of shear reinforcement provided;

$$0.5 \times v_c < v < (v_c + 0.4 \text{ N/mm}^2)$$

$$v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) = 0.400 \text{ N/mm}^2$$

$$A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = 349 \text{ mm}^2/\text{m}$$

$$3 \times 10\phi \text{ legs at } 100 \text{ c/c}$$

$$A_{sv,prov} = 2356 \text{ mm}^2/\text{m}$$

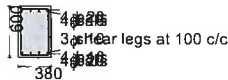
PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing;

$$s_{vl,max} = 0.75 \times d = 411 \text{ mm}$$

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

Mid span 1



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1;
 Area of reinforcement provided - layer 1;
 Depth to layer 1;
 Reinforcement provided - layer 2;
 Area of reinforcement provided - layer 2;
 Depth to layer 2;
 Total area of reinforcement;
 Centroid of reinforcement;

$$4 \times 20\phi \text{ bars}$$

$$A_{s_L1} = 1257 \text{ mm}^2$$

$$d_{L1} = 555 \text{ mm}$$

$$4 \times 16\phi \text{ bars}$$

$$A_{s_L2} = 804 \text{ mm}^2$$

$$d_{L2} = 537 \text{ mm}$$

$$A_{s,prov} = A_{s_L1} + A_{s_L2} = 2061 \text{ mm}^2$$

$$d_{bot} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s,prov} = 548 \text{ mm}$$

Multiple layers of top reinforcement

Reinforcement provided - layer 1;
 Area of reinforcement provided - layer 1;
 Depth to layer 1;
 Reinforcement provided - layer 2;
 Area of reinforcement provided - layer 2;
 Depth to layer 2;

$$4 \times 20\phi \text{ bars}$$

$$A_{s_L1} = 1257 \text{ mm}^2$$

$$d_{L1} = 45 \text{ mm}$$

$$4 \times 16\phi \text{ bars}$$

$$A_{s_L2} = 804 \text{ mm}^2$$

$$d_{L2} = 63 \text{ mm}$$

Total area of reinforcement; $A_{s2,prov} = A_{s_L1} + A_{s_L2} = 2061 \text{ mm}^2$
Centroid of reinforcement; $d_{top} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s2,prov} = 52 \text{ mm}$

Design moment resistance of rectangular section - Positive moment

Design bending moment; $M = \text{abs}(M_{s1_red}) = 409 \text{ kNm}$
Depth to tension reinforcement; $d = d_{bot} = 548 \text{ mm}$
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; $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = 476 \text{ mm}$

Depth of neutral axis; $x = (d - z) / 0.45 = 159 \text{ mm}$
Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z) = 1972 \text{ mm}^2$
Tension reinforcement provided; $4 \times 20 \phi \text{ bars} + 4 \times 16 \phi \text{ bars}$
Area of tension reinforcement provided; $A_{s,prov} = 2061 \text{ mm}^2$
Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h = 296 \text{ mm}^2$
Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h = 9120 \text{ mm}^2$

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

Rectangular section in shear

Shear reinforcement provided; $3 \times 10 \phi \text{ legs at } 100 \text{ c/c}$
Area of shear reinforcement provided; $A_{sv,prov} = 2356 \text{ mm}^2/\text{m}$
Minimum area of shear reinforcement (Table 3.7); $A_{sv,min} = 0.4 \text{ N/mm}^2 \times b / (0.87 \times f_{yv}) = 349 \text{ mm}^2/\text{m}$

PASS - Area of shear reinforcement provided exceeds minimum required

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

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

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

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

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

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

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; $s = (b - 2 \times (C_{nom_s} + \phi_v + \phi_{bot,L1}/2)) / (N_{bot,L1} - 1) - \phi_{bot,L1} = 77 \text{ mm}$

Minimum distance between bars in tension

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

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension

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

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

PASS - Satisfies the maximum spacing criteria

Span to depth ratio

Basic span to depth ratio (Table 3.9); $\text{span_to_depth}_{basic} = 20.0$

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

Modification for tension reinforcement

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

Modification for compression reinforcement

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

Modification for span length;

$$f_{long} = 1.000$$

Allowable span to depth ratio;

$$span_to_depth_{allow} = span_to_depth_{basic} \times f_{tens} \times f_{comp} =$$

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

Support B



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1;

$$4 \times 20\phi \text{ bars}$$

Area of reinforcement provided - layer 1;

$$A_{s_L1} = 1257 \text{ mm}^2$$

Depth to layer 1;

$$d_{L1} = 555 \text{ mm}$$

Reinforcement provided - layer 2;

$$4 \times 16\phi \text{ bars}$$

Area of reinforcement provided - layer 2;

$$A_{s_L2} = 804 \text{ mm}^2$$

Depth to layer 2;

$$d_{L2} = 537 \text{ mm}$$

Total area of reinforcement;

$$A_{s,prov} = A_{s_L1} + A_{s_L2} = 2061 \text{ mm}^2$$

Centroid of reinforcement;

$$d_{bot} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s,prov} = 548 \text{ mm}$$

Multiple layers of top reinforcement

Reinforcement provided - layer 1;

$$4 \times 16\phi \text{ bars}$$

Area of reinforcement provided - layer 1;

$$A_{s_L1} = 804 \text{ mm}^2$$

Depth to layer 1;

$$d_{L1} = 43 \text{ mm}$$

Reinforcement provided - layer 2;

$$4 \times 16\phi \text{ bars}$$

Area of reinforcement provided - layer 2;

$$A_{s_L2} = 804 \text{ mm}^2$$

Depth to layer 2;

$$d_{L2} = 59 \text{ mm}$$

Total area of reinforcement;

$$A_{s2,prov} = A_{s_L1} + A_{s_L2} = 1608 \text{ mm}^2$$

Centroid of reinforcement;

$$d_{top} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s2,prov} = 51 \text{ mm}$$

Rectangular section in shear

Design shear force span 1 at 6452 mm;

$$V = \text{abs}(\min(V_{B_s1_max}, V_{B_s1_red})) = 197 \text{ kN}$$

Design shear stress;

$$v = V / (b \times d) = 0.945 \text{ N/mm}^2$$

Design concrete shear stress;

$$v_c = 0.79 \times \min(3, [100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1,$$

$$(400 / d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$$

$$v_c = 0.705 \text{ N/mm}^2$$

Allowable design shear stress;

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

$$4.733 \text{ N/mm}^2$$

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;

$$v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) = 0.400 \text{ N/mm}^2$$

Area of shear reinforcement required;

$$A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = 349 \text{ mm}^2/\text{m}$$

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

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

PASS - Area of shear reinforcement provided exceeds minimum required

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

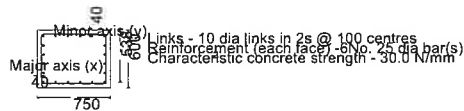
PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

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RC COLUMN DESIGN

Column definition

Column depth (larger column dim);	$h = 600 \text{ mm}$
Nominal cover to all reinforcement (longer dim);	$c_h = 40 \text{ mm}$
Depth to tension steel;	$h' = h - c_h - L_{dia} - D_{col}/2 = 538 \text{ mm}$
Column width (smaller column dim);	$b = 750 \text{ mm}$
Nominal cover to all reinforcement (shorter dim);	$c_b = 40 \text{ mm}$
Depth to tension steel;	$b' = b - c_b - L_{dia} - D_{col}/2 = 688 \text{ mm}$
Characteristic strength of reinforcement;	$f_y = 500 \text{ N/mm}^2$
Characteristic strength of concrete;	$f_{cu} = 30 \text{ N/mm}^2$



Unbraced Column Design

Check on overall column dimensions

Column OK - $h < 4b$

Unbraced column slenderness check

Unbraced column clear height;	$l_o = 3500 \text{ mm}$
Slenderness limit;	$l_{limit} = 60 \times b = 45000 \text{ mm}$
Slenderness limit;	$l_{limit1} = 100 \times b^2 / h = 93750 \text{ mm}$

Column slenderness limit OK

If column is unrestrained, slenderness limit OK

Short column check for unbraced columns

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

The unbraced column is short (major axis)

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

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

$$D_{col} = 25 \text{ mm}$$

Assumed no. of bars in one face (assumed sym);

$$L_{ncol} = 6$$

Area of "tension" steel;

$$A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4 = 2945 \text{ mm}^2$$

Area of compression steel;

$$A_{sc} = A_{st} = 2945 \text{ mm}^2$$

Total area of steel ;

$$A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2)) = 9817.5 \text{ mm}^2$$

Percentage of steel;

$$A_{scol} / (b \times h) = 2.2 \%$$

Design ultimate loading

Design ultimate axial load;

$$N = 2880 \text{ kN}$$

Design ultimate moment (major axis);

$$M_x = 600 \text{ kNm}$$

Design ultimate moment (minor axis);

$$M_y = 600 \text{ kNm}$$

Minimum design moments

Min design moment (major axis);

$$M_{xmin} = \min(0.05 \times h, 20 \text{ mm}) \times N = 57.6 \text{ kNm}$$

Min design moment (minor axis);

$$M_{ymin} = \min(0.05 \times b, 20 \text{ mm}) \times N = 57.6 \text{ kNm}$$

Design moments

Design moment (major axis);

$$M_{xdes} = \max(\text{abs}(M_x), M_{xmin}) = 600.0 \text{ kNm}$$

Design moment (minor axis);

$$M_{ydes} = \max(\text{abs}(M_y), M_{ymin}) = 600.0 \text{ kNm}$$

Simplified method for dealing with bi-axial bending:-

$$h' = 538 \text{ mm}; b' = 688 \text{ mm}$$

Approx uniaxial design moment

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

Design moment;

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

Set up section dimensions for design:-

Section depth;

$$D = \text{if}(M_{xdes}/h' < M_{ydes}/b', b, h) = 600.0 \text{ mm}$$

Depth to "tension" steel;

$$d = \text{if}(M_{xdes}/h' < M_{ydes}/b', b', h') = 537.5 \text{ mm}$$

Section width;

$$B = \text{if}(M_{xdes}/h' < M_{ydes}/b', h, b) = 750.0 \text{ mm}$$

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);

$$B = 750 \text{ mm}$$

Section depth perpendicular to axis of bending);

$$D = 600 \text{ mm}$$

Depth to "tension" steel (symmetrical);

$$d = 538 \text{ mm}$$

Tension and compression steel yield

Determine correct moment of resistance

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

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

Applied axial load;

$$N = 2880.0 \text{ kN}$$

Check for moment;

$$M_{design} = 952.5 \text{ kNm}$$

Moment check satisfied Moment capacity with no axial load

Check min and max areas of steel

Total area of concrete;	$A_{conc} = b \times h = 450000 \text{ mm}^2$
Area of steel (symmetrical);	$A_{scol} = 9817 \text{ mm}^2$
Minimum percentage of compression reinforcement; $k_c = 0.80 \%$	
Minimum steel area;	$A_{sc_min} = k_c \times A_{conc} = 3600 \text{ mm}^2$
Maximum steel area;	$A_{s_max} = 6 \% \times A_{conc} = 27000 \text{ mm}^2$

Area of compression steel provided OK**Major axis Shear Resistance of Concrete Columns**

Column width;	$b = 750 \text{ mm}$
Column depth;	$h = 600 \text{ mm}$
Effective depth to steel;	$h' = 538 \text{ mm}$
Area of concrete;	$A_{conc} = b \times h = 450000 \text{ mm}^2$
Design ultimate shear force (major axis);	$V_x = 75 \text{ kN}$
Characteristic strength of concrete;	$f_{cu} = 30 \text{ N/mm}^2$

Is a check required? (3.8.4.6)

Axial load;	$N = 2880.0 \text{ kN}$
Major axis moment;	$M_x = 600.0 \text{ kNm}$
Eccentricity;	$e = M_x / N = 208.3 \text{ mm}$
Limit to eccentricity;	$e_{limit} = 0.6 \times h = 360.0 \text{ mm}$
Actual shear stress;	$v_x = V_x / (b \times h') = 0.2 \text{ N/mm}^2$
Allowable stress; N/mm^2	$V_{allowable} = \min((0.8 \text{ N}^{1/2}/\text{mm}) \times \sqrt{f_{cu}}, 5 \text{ N/mm}^2) = 4.382$

No shear check required**Define Containment Links Provided**Link spacing; $s_v = 100 \text{ mm}$; Link diameter; $L_{dia} = 10 \text{ mm}$; No of links in each group; $L_n = 2$ **Minimum Containment Steel****Shear steel**

Link spacing;	$s_v = 100 \text{ mm}$
Link diameter;	$L_{dia} = 10 \text{ mm}$

Column steel

Diameter;	$D_{col} = 25 \text{ mm}$
Min diameter;	$L_{limit} = \max(6 \text{ mm}, D_{col}/4) = 6.3 \text{ mm}$

Link diameter OK

Max spacing;	$s_{limit} = 12 \times D_{col} = 300.0 \text{ mm}$
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Link spacing OK**Crack Control in columns**

Column design ultimate axial load;	$N = 2880.0 \text{ kN}$
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Column dimensions

Column depth (larger column dimension);	$h = 600 \text{ mm}$
Column width (smaller column dimension);	$b = 750 \text{ mm}$
Column area;	$A_c = h \times b = 450000 \text{ mm}^2$
Limit for crack check;	$N_{limit} = 0.2 \times f_{cu} \times A_c = 2700.0 \text{ kN}$

No crack checks required

Serviceability Limit State - Cracking in Columns

Bent about the major axis

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

Design serviceability moment about the major axis; $M_{X_SLS} = 50 \text{ kNm}$

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension);	$h = 600 \text{ mm}$
Depth to steel;	$h' = 538 \text{ mm}$
Column width (smaller column dimension);	$b = 750 \text{ mm}$
Characteristic strength of concrete;	$f_{cu} = 30 \text{ N/mm}^2$
Characteristic strength of reinforcement;	$f_y = 500 \text{ N/mm}^2$
Diameter of links;	$L_{dia} = 10 \text{ mm}$
Diameter of tension reinforcement;	$D_{col} = 25 \text{ mm}$
Number of tension reinforcement bars;	$L_{ncol} = 6$
Area of tension reinforcement;	$A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol} = 2945 \text{ mm}^2$
Nominal cover to reinforcement;	$c_{nom} = h - h' - D_{col}/2 - L_{dia} = 40 \text{ mm}$
Cover to tension reinforcement;	$c_{ten} = c_{nom} + L_{dia} = 50.0 \text{ mm}$
Effective depth to tension reinforcement;	$h' = 537.5 \text{ mm}$
Tension bar centres;	$bar_{crs} = (b - 2 \times (c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) = 125.0 \text{ mm}$

Modular Ratio

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

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

Depth of concrete in compression; $x = 201.5 \text{ mm}$

Concrete and Steel stresses

The serviceability limit state moment; $M_{X_SLS} = 50 \text{ kNm}$

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; $f_c = 2 \times M_{X_SLS} / (b \times x \times (h' - x/3)) = 1.41 \text{ N/mm}^2$

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

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; $f_{st} = M_{X_SLS} / (A_{st} \times (h' - x/3)) = 36.09 \text{ N/mm}^2$

Concrete and Steel strains

Strain in the reinforcement; $\epsilon_s = f_{st} / E_s = 180.5 \times 10^{-6}$

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

Steel strain OK

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

Level of crack;	$a' = h = 600 \text{ mm}$
	$\varepsilon_1 = \varepsilon_s \times (a' - x) / (h' - x) = 214.0 \times 10^{-6}$
Strain in the concrete at the level at which crack width is required adjusted for stiffening of the concrete tension zone	
Allowable crack width;	$\text{Crack}_{\text{Allowable}} = 0.2 \text{ mm}$
Factor for stiffening based on limiting crack width;	$\text{factor} = 1.0 \text{ N/mm}^2$
Breadth of tension face;	$b_t = b = 750 \text{ mm}$
	$\varepsilon_m = \min(\varepsilon_1, \max(0, \varepsilon_1 - [\text{factor} \times b_t \times (h - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (h' - x))])) = 13.45 \times 10^{-6}$
Distance from tension bar to crack in tension face between tension bars	$a_{cr1} = \sqrt{(\text{bar}_{crs}/2)^2 + (C_{nom} + L_{dia} + D_{col}/2)^2} - D_{col}/2 = 75.9$
mm	
Distance from tension bar to crack in tension face at corner of column	$a_{cr2} = \sqrt{2} \times (C_{nom} + L_{dia} + D_{col}/2) - D_{col}/2 = 75.9 \text{ mm}$
Critical distance from tension bar;	$a_{cr} = \max(a_{cr1}, a_{cr2}) = 75.9 \text{ mm}$
Design crack width;	$\text{Crack}_{\text{design}} = 3 \times a_{cr} \times \varepsilon_m / (1 + 2 \times (a_{cr} - C_{ten}) / (h - x)) = 0.003$
mm	
Max allowable crack width;	$\text{Crack}_{\text{Allowable}} = 0.20 \text{ mm}$

Serviceability Limit State - Cracking in Columns

Bent about the minor axis

Design serviceability moment about the minor axis; $M_{Y_SLS} = 50 \text{ kNm}$

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension);	$h = 600 \text{ mm}$
Column width (smaller column dimension);	$b = 750 \text{ mm}$
Depth to steel;	$b' = 688 \text{ mm}$
Characteristic strength of concrete;	$f_{cu} = 30 \text{ N/mm}^2$
Characteristic strength of reinforcement;	$f_y = 500 \text{ N/mm}^2$
Diameter of links;	$L_{dia} = 10 \text{ mm}$
Diameter of tension reinforcement;	$D_{col} = 25 \text{ mm}$
Number of tension reinforcement bars;	$L_{ncol} = 6$
Area of tension reinforcement;	$A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol} = 2945 \text{ mm}^2$
Nominal cover to reinforcement;	$C_{nom} = b - b' - D_{col}/2 - L_{dia} = 40 \text{ mm}$
Cover to tension reinforcement;	$C_{ten} = C_{nom} + L_{dia} = 50.0 \text{ mm}$
Effective depth to tension reinforcement;	$b' = 687.5 \text{ mm}$
Tension bar centres;	$\text{bar}_{crs} = (h - 2 \times (C_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) = 95.0 \text{ mm}$

Modular Ratio

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

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

Depth of concrete in compression; $x = 255.5 \text{ mm}$

Concrete and Steel stresses

The serviceability limit state moment;

$$M_{Y_SLS} = 50 \text{ kNm}$$

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;

$$f_c = 2 \times M_{Y_SLS} / (h \times x \times (b' - x/3)) = 1.08 \text{ N/mm}^2$$

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

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;

$$f_{st} = M_{Y_SLS} / (A_s \times (b' - x/3)) = 28.18 \text{ N/mm}^2$$

Concrete and Steel strains

Strain in the reinforcement;

$$\epsilon_s = f_{st} / E_s = 140.9 \times 10^{-6}$$

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

Steel strain OK

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

Level of crack;

$$a' = b = 750 \text{ mm}$$

$$\epsilon_1 = \epsilon_s \times (a' - x) / (b' - x) = 161.3 \times 10^{-6}$$

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

Allowable crack width;

$$\text{Crack}_{\text{Allowable}} = 0.2 \text{ mm}$$

Factor for stiffening based on limiting crack width;

$$\text{factor} = 1.0 \text{ N/mm}^2$$

Breadth of tension face;

$$h_t = h = 600 \text{ mm}$$

$$\epsilon_m = \min(\epsilon_1, \max(0, \epsilon_1 - [\text{factor} \times h_t \times (b - x) \times (a' - x) / (3 \times E_s \times A_s \times (b' - x))])) = 0.0000$$

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

$$a_{cr1} = \sqrt{((\text{bar}_{\text{crs}}/2)^2 + (C_{\text{nom}} + L_{\text{dia}} + D_{\text{col}}/2)^2)} - D_{\text{col}}/2 = 66.0$$

mm

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

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

Critical distance from tension bar;

$$a_{cr} = \max(a_{cr1}, a_{cr2}) = 75.9 \text{ mm}$$

Design crack width;

$$\text{Crack}_{\text{design}} = 3 \times a_{cr} \times \epsilon_m / (1 + 2 \times (a_{cr} - C_{\text{ten}}) / (b - x)) = 0.000$$

mm

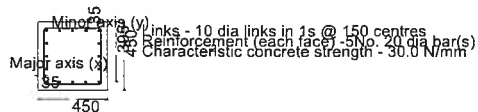
Max allowable crack width;

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

RC COLUMN DESIGN

Column definition

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



Braced Column Design

Check on overall column dimensions

Column OK - $h < 4b$

Braced column slenderness check

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

Column slenderness limit OK

Short column check for braced columns

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

The braced column is short (major axis)

Effect height factor for braced columns - minor axis;	$\beta_y = 0.75$
Effective height – minor axis;	$l_{ey} = \beta_y \times l_o = 2.625 \text{ m}$
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;

$$D_{col} = 20 \text{ mm}$$

Assumed no. of bars in one face (assumed sym);

$$L_{ncol} = 5$$

Area of "tension" steel;

$$A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4 = 1571 \text{ mm}^2$$

Area of compression steel;

$$A_{sc} = A_{st} = 1571 \text{ mm}^2$$

Total area of steel ;

$$A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2)) = 5026.5 \text{ mm}^2$$

Percentage of steel;

$$A_{scol} / (b \times h) = 2.5 \%$$

Design ultimate loading

Design ultimate axial load;

$$N = 850 \text{ kN}$$

Design ultimate moment (major axis);

$$M_x = 11 \text{ kNm}$$

Design ultimate moment (minor axis);

$$M_y = 34 \text{ kNm}$$

Minimum design moments

Min design moment (major axis);

$$M_{xmin} = \min(0.05 \times h, 20 \text{ mm}) \times N = 17.0 \text{ kNm}$$

Min design moment (minor axis);

$$M_{ymin} = \min(0.05 \times b, 20 \text{ mm}) \times N = 17.0 \text{ kNm}$$

Design moments

Design moment (major axis);

$$M_{xdes} = \max(\text{abs}(M_x), M_{xmin}) = 17.0 \text{ kNm}$$

Design moment (minor axis);

$$M_{ydes} = \max(\text{abs}(M_y), M_{ymin}) = 34.0 \text{ kNm}$$

Simplified method for dealing with bi-axial bending:-

$$h' = 395 \text{ mm}; b' = 395 \text{ mm}$$

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;

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

Set up section dimensions for design:-

Section depth;

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

Depth to "tension" steel;

$$d = \text{if}(M_{xdes}/h' < M_{ydes}/b', b', h') = 395.0 \text{ mm}$$

Section width;

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

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);

$$B = 450 \text{ mm}$$

Section depth perpendicular to axis of bending);

$$D = 450 \text{ mm}$$

Depth to "tension" steel (symmetrical);

$$d = 395 \text{ mm}$$

Tension and compression steel yield

Determine correct moment of resistance

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

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

Applied axial load;

$$N = 850.0 \text{ kN}$$

Check for moment;

$$M_{design} = 48.2 \text{ kNm}$$

Moment check satisfied

Check min and max areas of steel

Total area of concrete;

$$A_{conc} = b \times h = 202500 \text{ mm}^2$$

Area of steel (symmetrical);

$$A_{scol} = 5027 \text{ mm}^2$$

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

Minimum steel area;

$$A_{sc_min} = k_c \times A_{conc} = 1620 \text{ mm}^2$$

Maximum steel area;

$$A_{s_max} = 6 \% \times A_{conc} = 12150 \text{ mm}^2$$

Area of compression steel provided OK

Major axis Shear Resistance of Concrete Columns

Column width;

$$b = 450 \text{ mm}$$

Column depth;

$$h = 450 \text{ mm}$$

Effective depth to steel;

$$h' = 395 \text{ mm}$$

Area of concrete;

$$A_{conc} = b \times h = 202500 \text{ mm}^2$$

Design ultimate shear force (major axis);

$$V_x = 55 \text{ kN}$$

Characteristic strength of concrete;

$$f_{cu} = 30 \text{ N/mm}^2$$

Axial load;

$$N = 850.0 \text{ kN}$$

Major axis moment;

$$M_x = 11.0 \text{ kNm}$$

Eccentricity;

$$e = M_x / N = 12.9 \text{ mm}$$

Limit to eccentricity;

$$e_{limit} = 0.6 \times h = 270.0 \text{ mm}$$

Actual shear stress;

$$v_x = V_x / (b \times h') = 0.3 \text{ N/mm}^2$$

Allowable stress;

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

N/mm²

No shear check required

Define Containment Links Provided

Link spacing; $s_v = 150 \text{ mm}$; Link diameter; $L_{dia} = 10 \text{ mm}$; No of links in each group; $L_n = 1$

Minimum Containment Steel

Shear steel

Link spacing;

$$s_v = 150 \text{ mm}$$

Link diameter;

$$L_{dia} = 10 \text{ mm}$$

Column steel

Diameter;

$$D_{col} = 20 \text{ mm}$$

Min diameter;

$$L_{limit} = \max(6 \text{ mm}, D_{col}/4) = 6.0 \text{ mm}$$

Link diameter OK

Max spacing;

$$s_{limit} = 12 \times D_{col} = 240.0 \text{ mm}$$

Link spacing OK

Crack Control in columns

Column design ultimate axial load;

$$N = 850.0 \text{ kN}$$

Column dimensions

Column depth (larger column dimension);

$$h = 450 \text{ mm}$$

Column width (smaller column dimension);

$$b = 450 \text{ mm}$$

Column area;

$$A_c = h \times b = 202500 \text{ mm}^2$$

Limit for crack check;

$$N_{limit} = 0.2 \times f_{cu} \times A_c = 1215.0 \text{ kN}$$

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; $M_{X_SLS} = 23 \text{ kNm}$

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension);	$h = 450 \text{ mm}$
Depth to steel;	$h' = 395 \text{ mm}$
Column width (smaller column dimension);	$b = 450 \text{ mm}$
Characteristic strength of concrete;	$f_{cu} = 30 \text{ N/mm}^2$
Characteristic strength of reinforcement;	$f_y = 500 \text{ N/mm}^2$
Diameter of links;	$L_{dia} = 10 \text{ mm}$
Diameter of tension reinforcement;	$D_{col} = 20 \text{ mm}$
Number of tension reinforcement bars;	$L_{ncol} = 5$
Area of tension reinforcement;	$A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol} = 1571 \text{ mm}^2$
Nominal cover to reinforcement;	$C_{nom} = h - h' - D_{col}/2 - L_{dia} = 35 \text{ mm}$
Cover to tension reinforcement;	$C_{ten} = C_{nom} + L_{dia} = 45.0 \text{ mm}$
Effective depth to tension reinforcement;	$h' = 395.0 \text{ mm}$
Tension bar centres;	$bar_{crs} = (b - 2 \times (C_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) = 85.0 \text{ mm}$

Modular Ratio

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

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

Depth of concrete in compression; $x = 159.2 \text{ mm}$

Concrete and Steel stresses

The serviceability limit state moment; $M_{X_SLS} = 23 \text{ kNm}$

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; $f_c = 2 \times M_{X_SLS} / (b \times x \times (h' - x/3)) = 1.88 \text{ N/mm}^2$

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

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; $f_{st} = M_{X_SLS} / (A_{st} \times (h' - x/3)) = 42.82 \text{ N/mm}^2$

Concrete and Steel strains

Strain in the reinforcement; $\epsilon_s = f_{st} / E_s = 214.1 \times 10^{-6}$

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

Steel strain OK

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

Level of crack; $a' = h = 450 \text{ mm}$

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

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 limiting crack width; factor = 1.0 N/mm²

Breadth of tension face; b_t = b = 450 mm

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

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

$$a_{cr1} = \sqrt{((\text{bar}_{crs}/2)^2 + (C_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2 = 59.5$$

mm

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

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

Critical distance from tension bar;

$$a_{cr} = \max(a_{cr1}, a_{cr2}) = 67.8 \text{ mm}$$

Design crack width;

$$\text{Crack}_{design} = 3 \times a_{cr} \times \varepsilon_m / (1 + 2 \times (a_{cr} - C_{ten}) / (h - x)) = 0.016$$

mm

Max allowable crack width;

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

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/mm²

Characteristic strength of reinforcement; f_y = 500 N/mm²

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} = π × D_{col}² / 4 × L_{ncol} = 1571 mm²

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 × (C_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) = 85.0 mm

Modular Ratio

Modulus of elasticity for reinforcement; E_s = 200 kN/mm²

Modulus of elasticity for conc (half instantaneous); E_c = ((20 kN/mm²) + 200 × f_{cu}) / 2 = 13 kN/mm²

Modular ratio; m = E_s / E_c = 15.385

Neutral Axis position

For equilibrium; F_{st} equates F_c

Therefore:

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

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

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

Depth of concrete in compression;

$$x = 159.2 \text{ mm}$$

Concrete and Steel stresses

The serviceability limit state moment;

$$M_{Y_SLS} = 21 \text{ kNm}$$

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;

$$f_c = 2 \times M_{Y_SLS} / (h \times x \times (b' - x/3)) = 1.71 \text{ N/mm}^2$$

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

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;

$$f_{st} = M_{Y_SLS} / (A_s \times (b' - x/3)) = 39.10 \text{ N/mm}^2$$

Concrete and Steel strains

Strain in the reinforcement;

$$\epsilon_s = f_{st} / E_s = 195.5 \times 10^{-6}$$

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

Steel strain OK

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

Level of crack;

$$a' = b = 450 \text{ mm}$$

$$\epsilon_1 = \epsilon_s \times (a' - x) / (b' - x) = 241.1 \times 10^{-6}$$

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

Allowable crack width;

$$\text{Crack}_{\text{Allowable}} = 0.2 \text{ mm}$$

Factor for stiffening based on limiting crack width;

$$\text{factor} = 1.0 \text{ N/mm}^2$$

Breadth of tension face;

$$h_t = h = 450 \text{ mm}$$

$$\epsilon_m = \min(\epsilon_1, \max(0, \epsilon_1 - [\text{factor} \times h_t \times (b - x) \times (a' - x) / (3 \times E_s \times A_s \times (b' - x))])) = 69.82 \times 10^{-6}$$

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

$$a_{cr1} = \sqrt{((\text{bar}_{crs}/2)^2 + (C_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2 = 59.5$$

mm

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

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

Critical distance from tension bar;

$$a_{cr} = \max(a_{cr1}, a_{cr2}) = 67.8 \text{ mm}$$

Design crack width;

$$\text{Crack}_{\text{design}} = 3 \times a_{cr} \times \epsilon_m / (1 + 2 \times (a_{cr} - C_{ten}) / (b - x)) = 0.012$$

mm

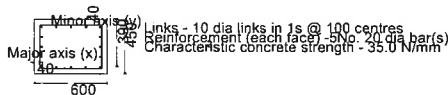
Max allowable crack width;

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

RC COLUMN DESIGN

Column definition

Column depth (larger column dim);	$h = 450$ mm
Nominal cover to all reinforcement (longer dim);	$c_h = 40$ mm
Depth to tension steel;	$h' = h - c_h - L_{dia} - D_{col}/2 = 390$ mm
Column width (smaller column dim);	$b = 600$ mm
Nominal cover to all reinforcement (shorter dim);	$c_b = 40$ mm
Depth to tension steel;	$b' = b - c_b - L_{dia} - D_{col}/2 = 540$ mm
Characteristic strength of reinforcement;	$f_y = 500$ N/mm ²
Characteristic strength of concrete;	$f_{cu} = 35$ N/mm ²



Unbraced Column Design

Check on overall column dimensions

Column OK - $h < 4b$

Unbraced column slenderness check

Unbraced column clear height;	$l_o = 3500$ mm
Slenderness limit;	$l_{limit} = 60 \times b = 36000$ mm
Slenderness limit;	$l_{limit} = 100 \times b^2 / h = 80000$ 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	$\beta_x = 1.20$
Effective height (major axis);	$l_{ex} = \beta_x \times l_o = 4.200$ m
Slenderness check;	$l_{ex}/h = 9.33$

The unbraced column is short (major axis)

Effect height factor for unbraced columns - min axis; β_y	$\beta_y = 1.20$
Effective height (minor axis);	$l_{ey} = \beta_y \times l_o = 4.200$ m
Slenderness check;	$l_{ey}/b = 7.00$

The unbraced column is short (minor axis)

Short column - bi-axial bending

Define column reinforcement

Main reinforcement in column

Assumed diameter of main reinforcement;	$D_{col} = 20 \text{ mm}$
Assumed no. of bars in one face (assumed sym);	$L_{ncol} = 5$
Area of "tension" steel;	$A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4 = 1571 \text{ mm}^2$
Area of compression steel;	$A_{sc} = A_{st} = 1571 \text{ mm}^2$
Total area of steel ;	$A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2)) = 5026.5 \text{ mm}^2$
Percentage of steel;	$A_{scol} / (b \times h) = 1.9 \%$

Design ultimate loading

Design ultimate axial load;	$N = 2000 \text{ kN}$
Design ultimate moment (major axis);	$M_x = 275 \text{ kNm}$
Design ultimate moment (minor axis);	$M_y = 220 \text{ kNm}$

Minimum design moments

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

Design moments

Design moment (major axis);	$M_{xdes} = \max(\text{abs}(M_x), M_{xmin}) = 275.0 \text{ kNm}$
Design moment (minor axis);	$M_{ydes} = \max(\text{abs}(M_y), M_{ymin}) = 220.0 \text{ kNm}$

Simplified method for dealing with bi-axial bending:-

$h' = 390 \text{ mm}$; $b' = 540 \text{ mm}$
 Approx uniaxial design moment
 $\beta = 1 - 1.165 \times \min(0.6, N/(b \times h \times f_{cu})) = 0.75$
 Design moment;

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

Set up section dimensions for design:-

Section depth;	$D = \text{if}(M_{xdes}/h' < M_{ydes}/b', b, h) = 450.0 \text{ mm}$
Depth to "tension" steel;	$d = \text{if}(M_{xdes}/h' < M_{ydes}/b', b', h') = 390.0 \text{ mm}$
Section width;	$B = \text{if}(M_{xdes}/h' < M_{ydes}/b', h, b) = 600.0 \text{ mm}$

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);	$B = 600 \text{ mm}$
Section depth perpendicular to axis of bending);	$D = 450 \text{ mm}$
Depth to "tension" steel (symmetrical);	$d = 390 \text{ mm}$

Tension and compression steel yield

Determine correct moment of resistance

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

$$M_R = \text{ceiling}(\text{if}(x_{calc} < D/0.9, M_{R1}, M_{R2}), 0.001 \text{ kNm}) = 462.2 \text{ kNm}$$

Applied axial load;	$N = 2000.0 \text{ kN}$
Check for moment;	$M_{design} = 394.7 \text{ kNm}$

Moment check satisfied

Check min and max areas of steel

Total area of concrete;	$A_{conc} = b \times h = 270000 \text{ mm}^2$
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Area of steel (symmetrical); $A_{scol} = 5027 \text{ mm}^2$
 Minimum percentage of compression reinforcement; $k_c = 0.80 \%$
 Minimum steel area; $A_{sc_min} = k_c \times A_{conc} = 2160 \text{ mm}^2$
 Maximum steel area; $A_{s_max} = 6 \% \times A_{conc} = 16200 \text{ mm}^2$

Area of compression steel provided OK

Major axis Shear Resistance of Concrete Columns

Column width; $b = 600 \text{ mm}$
 Column depth; $h = 450 \text{ mm}$
 Effective depth to steel; $h' = 390 \text{ mm}$
 Area of concrete; $A_{conc} = b \times h = 270000 \text{ mm}^2$
 Design ultimate shear force (major axis); $V_x = 75 \text{ kN}$
 Characteristic strength of concrete; $f_{cu} = 35 \text{ N/mm}^2$

Is a check required? (3.8.4.6)

Axial load; $N = 2000.0 \text{ kN}$
 Major axis moment; $M_x = 275.0 \text{ kNm}$
 Eccentricity; $e = M_x / N = 137.5 \text{ mm}$
 Limit to eccentricity; $e_{limit} = 0.6 \times h = 270.0 \text{ mm}$
 Actual shear stress; $v_x = V_x / (b \times h') = 0.3 \text{ N/mm}^2$
 Allowable stress; $v_{allowable} = \min((0.8 \text{ N}^{1/2}/\text{mm}) \times \sqrt{f_{cu}}, 5 \text{ N/mm}^2) = 4.733$
 N/mm^2

No shear check required

Define Containment Links Provided

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

Minimum Containment Steel

Shear steel

Link spacing; $s_v = 100 \text{ mm}$
 Link diameter; $L_{dia} = 10 \text{ mm}$

Column steel

Diameter; $D_{col} = 20 \text{ mm}$
 Min diameter; $L_{limit} = \max(6 \text{ mm}, D_{col}/4) = 6.0 \text{ mm}$

Link diameter OK

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

Link spacing OK

Crack Control in columns

Column design ultimate axial load; $N = 2000.0 \text{ kN}$

Column dimensions

Column depth (larger column dimension); $h = 450 \text{ mm}$
 Column width (smaller column dimension); $b = 600 \text{ mm}$
 Column area; $A_c = h \times b = 270000 \text{ mm}^2$
 Limit for crack check; $N_{limit} = 0.2 \times f_{cu} \times A_c = 1890.0 \text{ kN}$

No crack checks required

Serviceability Limit State - Cracking in Columns

Bent about the major axis

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

Design serviceability moment about the major axis; $M_{X_SLS} = 50 \text{ kNm}$

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension);	$h = 450 \text{ mm}$
Depth to steel;	$h' = 390 \text{ mm}$
Column width (smaller column dimension);	$b = 600 \text{ mm}$
Characteristic strength of concrete;	$f_{cu} = 35 \text{ N/mm}^2$
Characteristic strength of reinforcement;	$f_y = 500 \text{ N/mm}^2$
Diameter of links;	$L_{dia} = 10 \text{ mm}$
Diameter of tension reinforcement;	$D_{col} = 20 \text{ mm}$
Number of tension reinforcement bars;	$L_{ncol} = 5$
Area of tension reinforcement;	$A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol} = 1571 \text{ mm}^2$
Nominal cover to reinforcement;	$C_{nom} = h - h' - D_{col}/2 - L_{dia} = 40 \text{ mm}$
Cover to tension reinforcement;	$C_{ten} = C_{nom} + L_{dia} = 50.0 \text{ mm}$
Effective depth to tension reinforcement;	$h' = 390.0 \text{ mm}$
Tension bar centres;	$bar_{crs} = (b - 2 \times (C_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) = 120.0 \text{ mm}$

Modular Ratio

Modulus of elasticity for reinforcement;	$E_s = 200 \text{ kN/mm}^2$
Modulus of elast for conc (half the instantaneous);	$E_c = ((20 \text{ kN/mm}^2) + 200 \times f_{cu}) / 2 = 14 \text{ kN/mm}^2$
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:-

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

Depth of concrete in compression; $x = 139.4 \text{ mm}$

Concrete and Steel stresses

The serviceability limit state moment; $M_{X_SLS} = 50 \text{ kNm}$

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; $f_c = 2 \times M_{X_SLS} / (b \times x \times (h' - x/3)) = 3.48 \text{ N/mm}^2$

Allowable stress; $0.45 \times f_{cu} = 15.75 \text{ N/mm}^2$

Concrete stress OK

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

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

Solving for steel stress f_{st} gives; $f_{st} = M_{X_SLS} / (A_{st} \times (h' - x/3)) = 92.66 \text{ N/mm}^2$

Concrete and Steel strains

Strain in the reinforcement; $\epsilon_s = f_{st} / E_s = 463.3 \times 10^{-6}$

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

Steel strain OK

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

Level of crack; $a' = h = 450 \text{ mm}$

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

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 limiting crack width; factor = 1.0 N/mm²

Breadth of tension face; b_t = b = 600 mm

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

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

$$a_{cr1} = \sqrt{((\text{bar}_{crs}/2)^2 + (C_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2 = 74.9$$

mm

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

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

Critical distance from tension bar;

$$a_{cr} = \max(a_{cr1}, a_{cr2}) = 74.9 \text{ mm}$$

Design crack width;

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

mm

Max allowable crack width;

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

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 = 460 mm

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

Depth to steel; b' = 540 mm

Characteristic strength of concrete; f_{cu} = 35 N/mm²

Characteristic strength of reinforcement; f_y = 500 N/mm²

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} = π × D_{col}² / 4 × L_{ncol} = 1571 mm²

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 × (C_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) = 82.5 mm

Modular Ratio

Modulus of elasticity for reinforcement; E_s = 200 kN/mm²

Modulus of elasticity for conc (half instantaneous); E_c = ((20 kN/mm²) + 200 × f_{cu}) / 2 = 14 kN/mm²

Modular ratio; m = E_s / E_c = 14.815

Neutral Axis position

For equilibrium; F_{st} equates F_c

Therefore; m × A_{st} × [f_c × (b' - x) / x] equates to 0.5 × f_c × h × x

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

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

Depth of concrete in compression; x = 190.2 mm

Concrete and Steel stresses

The serviceability limit state moment;

$$M_{Y_SLS} = 50 \text{ kNm}$$

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;

$$f_c = 2 \times M_{Y_SLS} / (h \times x \times (b' - x/3)) = 2.45 \text{ N/mm}^2$$

$$\text{Allowable stress; } 0.45 \times f_{cu} = 15.75 \text{ N/mm}^2$$

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;

$$f_{st} = M_{Y_SLS} / (A_s \times (b' - x/3)) = 66.79 \text{ N/mm}^2$$

Concrete and Steel strains

Strain in the reinforcement;

$$\epsilon_s = f_{st} / E_s = 333.9 \times 10^{-6}$$

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

Steel strain OK

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

Level of crack;

$$a' = b = 600 \text{ mm}$$

$$\epsilon_1 = \epsilon_s \times (a' - x) / (b' - x) = 391.2 \times 10^{-6}$$

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

Allowable crack width;

$$\text{Crack}_{\text{Allowable}} = 0.2 \text{ mm}$$

Factor for stiffening based on limiting crack width; factor = 1.0 N/mm²

Breadth of tension face;

$$h_t = h = 450 \text{ mm}$$

$$\epsilon_m = \min(\epsilon_1, \max(0, \epsilon_1 - [\text{factor} \times h_t \times (b - x) \times (a' - x) / (3 \times E_s \times A_s \times (b' - x))])) = 162.0 \times 10^{-6}$$

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

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

mm

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

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

Critical distance from tension bar;

$$a_{cr} = \max(a_{cr1}, a_{cr2}) = 74.9 \text{ mm}$$

Design crack width;

$$\text{Crack}_{\text{design}} = 3 \times a_{cr} \times \epsilon_m / (1 + 2 \times (a_{cr} - c_{ten}) / (b - x)) = 0.032$$

mm

Max allowable crack width;

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

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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)

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

Inner sagging steel

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

Depth to inner tension steel (resisting sagging)

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

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)

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

Inner hogging steel

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

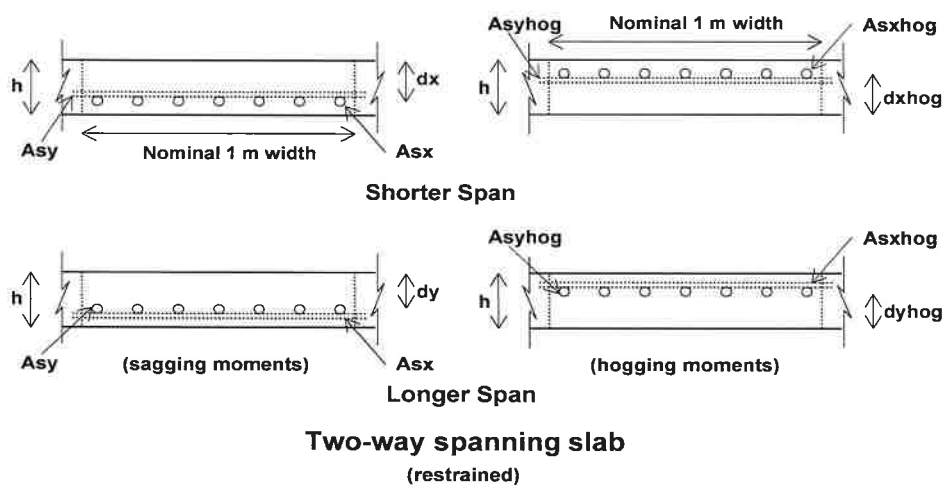
Depth to inner tension steel (resisting hogging)

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

Materials

Characteristic strength of reinforcement; $f_y = 500$ N/mm²

Characteristic strength of concrete; $f_{cu} = 25$ N/mm²



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/m²

Edge condition shorter side (1); Edge₁ = "C"

Edge condition other shorter side (2); Edge₂ = "D"

Edge condition longer side (3); Edge₃ = "C"

Edge condition other longer side (4); Edge₄ = "D"

Number of discontinuous edges

$N_d = 2$

Moment coefficients

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

$$\beta_1 = \text{if}(\text{Edge}_1 == \text{"C"}, 4/3 \times \beta_{sy}, 0) = 0.045$$

$$\beta_2 = \text{if}(\text{Edge}_2 == \text{"C"}, 4/3 \times \beta_{sy}, 0) = 0.000$$

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

$$\beta_{3x} = \text{if}(\text{Edge}_3 == \text{"C"}, 4/3, 0) = 1.333$$

$$\beta_{4x} = \text{if}(\text{Edge}_4 == \text{"C"}, 4/3, 0) = 0.000$$

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

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

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

Maximum span moments per unit width - restrained slabs

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

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

Maximum support moments per unit width - restrained slabs

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

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

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; $\beta_{bx} = 1.0$

Slab requiring outer tension steel only - bars (sagging)

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

$$\text{Neutral axis depth; } x_x = (d_x - z_x) / 0.45 = 13 \text{ mm}$$

Area of tension steel required

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

Tension steel

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

$$A_{sx_prov} = A_{sx} = 524 \text{ mm}^2/\text{m}$$

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; $\beta_{by} = 1.0$

Slab requiring inner tension steel only - bars (sagging)

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

$$\text{Neutral axis depth; } x_y = (d_y - z_y) / 0.45 = 12 \text{ mm}$$

Area of tension steel required

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

Tension steel

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

$$A_{sy_prov} = A_{sy} = 524 \text{ mm}^2/\text{m}$$

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; $\beta_{bx} = 1.0$

Slab requiring outer tension steel only - bars (hogging)

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

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

Area of tension steel required

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

Tension steel

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

$$A_{sxhog_prov} = A_{sxhog} = 524 \text{ mm}^2/\text{m}$$

Area of outer tension steel provided sufficient to resist hogging

Tension In One Truss Member

Characteristic strength of reinforcement;

$$f_y = 500 \text{ N/mm}^2$$

Partial safety factor for strength of steel;

$$\gamma_{ms} = 1.15$$

Req'd area of reinf't (in one truss member);

$$A_{s_req} = F_t / (1/\gamma_{ms} \times f_y) = 588 \text{ mm}^2$$

Provided area of reinf't (in one truss member);

$$A_{s_prov} = A_{st} = 2011 \text{ mm}^2$$

Tension capacity of truss member;

$$P_t = (1/\gamma_{ms} \times f_y) \times A_{s_prov} = 874.2 \text{ kN}$$

PASS Tension

Max / Min Areas Of Reinforcement

Minimum area of tension steel;

$$A_{st_min} = k_t \times A_c = 1170 \text{ mm}^2$$

Maximum area of tension steel;

$$A_{st_max} = 4 \% \times A_c = 36000 \text{ mm}^2$$

Area of tension steel provided OK

Shear

Applied shear stress

Applied Shear stress;

$$V = F_{uls} / 2 = 405.0 \text{ kN}$$

Width of pile cap shear plane;

$$b_v = \min(b, 3 \times \phi) = 900 \text{ mm}$$

Design shear stress;

$$v = V / (b_v \times d) = 0.47 \text{ N/mm}^2$$

Allowable shear stress;

$$V_{allowable} = \min((0.8 \text{ N}^{1/2}/\text{mm}) \times \sqrt{f_{cu}}, 5 \text{ N/mm}^2) = 4.00$$

N/mm²

Shear stress - OK

Design concrete shear strength

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

Shear stress

$$v_{c_25} = 0.79 \times r^{1/3} \times \max(0.67, (400 \text{ mm}/d)^{1/4}) \times 1.0 \text{ N/mm}^2 / 1.25 = 0.31 \text{ N/mm}^2$$

Shear stress -

$$v_c = v_{c_25} \times (\min(f_{cu}, 40 \text{ N/mm}^2) / 25 \text{ N/mm}^2)^{1/3} = 0.31$$

N/mm²

Spacing;

$$a_v = \min(2 \times d, \max(s / 2 - \phi / 2 + \phi / 5 - x / 2, 0.1 \text{ mm}))$$

= 270 mm

Enhanced shear stress;

$$v_{c_enh} = \min(V_{allowable}, 2 \times d \times v_c / a_v) = 2.21 \text{ N/mm}^2$$

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;

$$\text{spacing}_{bars} = \max(0 \text{ mm}, (b - 2 \times (C_{adopt} + L_{dia}) - D_t) / (L_{nt} - 1) - D_t) = 73 \text{ mm}$$

Maximum allowable spacing;

$$\text{spacing}_{max} = \min((47000 \text{ N/mm}) / f_s, 300 \text{ mm}) = 300 \text{ mm}$$

Minimum allowable spacing;

$$\text{spacing}_{min} = h_{agg} + 5 \text{ mm} = 25 \text{ mm}$$

Bar spacing OK

Clear Distance Between Face Of Beam And Tension Bars

Distance to face of beam;

$$\text{Dist}_{edge} = C_{adopt} + L_{dia} + D/2 = 50 \text{ mm}$$

Design service stress in reinforcement;

$$f_s = 2 \times f_y \times A_{s_req} / (3 \times A_{s_prov} \times \beta_b) = 97.5 \text{ N/mm}^2$$

Max allowable clear spacing;

$$\text{Spacing}_{max} = \min((47000 \text{ N/mm}) / f_s, 300 \text{ mm}) = 300 \text{ mm}$$

Max distance to face of beam;

$$\text{Dist}_{max} = \text{Spacing}_{max} / 2 = 150 \text{ mm}$$

Max distance to beam edge check - OK

Anchorage Of Tension Steel

Anchorage factor;

$$\phi_{\text{factor}} = 44$$

Type of lap length;

lap_type = "tens_lap"

Type of reinforcement;

reft_type = "def2_fy500"

Minimum radius;

$$r_{\text{bar}} = 32 \text{ mm}$$

Minimum end projection;

$$P_{\text{bar}} = 130 \text{ mm}$$

Minimum anchorage length or lap length req'd;

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

Check anchorage length to cl. 3.12.9.4 (b);

$$L_{\text{cl. 3.12.9.4}} = 12 \times D_t + d/2 = 667 \text{ mm}$$

Required minimum effective anchorage length;

$$L_a = \max(L_{\text{table 3.27}}, L_{\text{cl. 3.12.9.4}}) = 704 \text{ mm}$$

Deflection Check

Redistribution ratio;

$$\beta_b = 1.0$$

Design service stress in tension reinforcement;

$$f_s = 2 \times f_y \times A_{s_req} / (3 \times A_{s_prov} \times \beta_b) = 97.5 \text{ N/mm}^2$$

Modification for tension reinforcement;

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

Modified span to depth ratio;

$$\text{modfspan_depth} = \text{factor}_{\text{tens}} \times \text{basicspan_depth} = 40.0$$

Span of pile cap for deflection check;

$$L_s = 1000 \text{ mm}$$

Actual span to depth ratio;

$$\text{actualspan_depth} = L_s / d = 1.05$$

PASS - Deflection

$$v_{c_252} = 0.79 \times r_2^{1/3} \times \max(0.67, (400 \text{ mm}/d)^{1/4}) \times 1.0 \text{ N/mm}^2 / 1.25 = \mathbf{0.32 \text{ N/mm}^2}$$

$$v_{c3} = v_{c2} = v_{c_252} \times (\min(f_{cu}, 40 \text{ N/mm}^2) / 25 \text{ N/mm}^2)^{1/3} = \mathbf{0.32}$$

N/mm²

$$a_{v_2} = \min(2 \times d, \max((s/2 / \cos(\theta_{23}) - \phi/2 + \phi/5 - y/2), 0.1 \text{ mm})) = \mathbf{108 \text{ mm}}$$

$$v_{c_enh_3} = v_{c_enh_2} = 2 \times d \times v_{c2} / a_{v_2} = \mathbf{8.50 \text{ N/mm}^2}$$

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

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

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

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

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

N/mm²

Shear stress - OK

Clear Distance Between Bars In Tension

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

Actual bar spacing;

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

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

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

Bar spacing OK

Clear Distance Between Face Of Beam And Tension Bars

Distance to face of beam; $\text{Dist}_{edge} = C_{adopt} + L_{dia} + D_t / 2 = \mathbf{60 \text{ mm}}$

Design service stress in reinforcement; $f_s = 2 \times f_y \times A_{s_req} / (3 \times A_{s_prov} \times \beta_b) = \mathbf{34.3 \text{ N/mm}^2}$

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

Max distance to face of beam; $\text{Dist}_{max} = \text{Spacing}_{max} / 2 = \mathbf{150 \text{ mm}}$

Max distance to beam edge check - OK

Anchorage Of Tension Steel

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

Type of lap length; $\text{lap_type} = \mathbf{"tens_lap"}$

Type of reinforcement; $\text{reft_type} = \mathbf{"def2_fy500"}$

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

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

Minimum anchorage length or lap length req'd; $L_{table\ 3.27} = \phi_{factor} \times D_t = \mathbf{704 \text{ mm}}$

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

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

Check bearing stress on minimum radius bend

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

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

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

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

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

Internal bar centres;

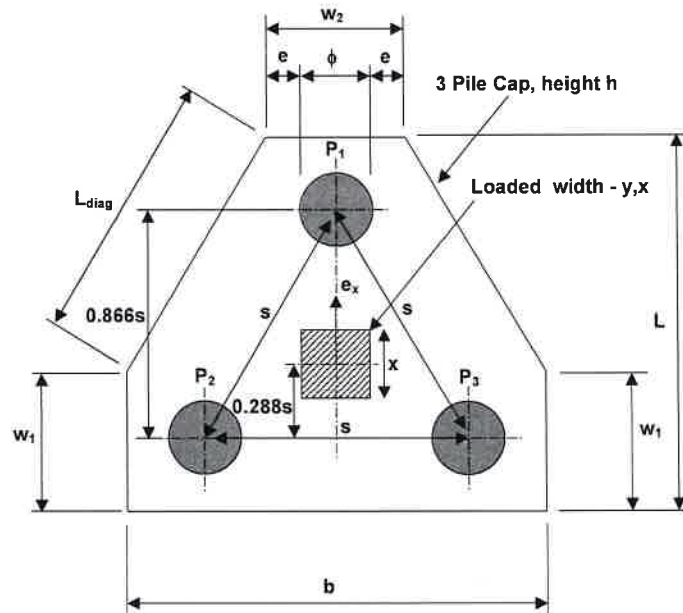
$$s_{int} = \text{spacing}_{bars} + D_t = 54 \text{ mm}$$

Internal maximum allowable bearing stress;

$$f_{bt_max_int} = 2 \times f_{cu} / (1 + 2 \times (D_t / s_{int})) = 31.40 \text{ N/mm}^2$$

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;

$$F_{uls} = 1500.0 \text{ kN}$$

The ultimate load per pile is;

$$F_{uls_pile} = F_{uls}/3 = 500.0 \text{ kN}$$

Characteristic axial load;

$$F_{char} = 1000.0 \text{ kN}$$

The characteristic load per pile is;

$$F_{char_pile} = F_{char}/3 = 333.3 \text{ kN}$$

Pile diameter;

$$\phi = 600 \text{ mm}$$

Pile spacing;

$$s = 1200 \text{ mm}$$

Pile cap overhang;

$$e = 250 \text{ mm}$$

Overall length of pile cap;

$$L = \sin(60) \times s + \phi + 2 \times e = 2139 \text{ mm}$$

Overall width of pile cap;

$$b = s + \phi + 2 \times e = 2300 \text{ mm}$$

Width at pile parallel to length, L;

$$w_1 = \phi + 2 \times e = 1100 \text{ mm}$$

Width at pile parallel to overall width, b;

$$w_2 = \phi + 2 \times e = 1100 \text{ mm}$$

Overall height of pile cap;

$$h = 1000 \text{ mm}$$

Diagonal length of sides;

$$L_{side_diag} = \sqrt{(L-w_1)^2 + ((b-w_2)/2)^2} = 1200 \text{ mm}$$

Dimensions of loaded area;

$$x = 300 \text{ mm}$$

$$y = 300 \text{ mm}$$

Cover

Concrete grade;

$$f_{cu} = 25.0 \text{ N/mm}^2$$

Nominal cover;

$$c_{nom} = 40 \text{ mm}$$

Tension bar diameter;

$$D_t = 16 \text{ mm}$$

Link bar diameter;

$$L_{dia} = 12 \text{ mm}$$

Depth to tension steel;

$$d = h - c_{nom} - L_{dia} - D_t/2 = 940 \text{ mm}$$

Pile Cap Forces

Compression within pile cap;

$$F_c = F_{uls} / (3 \times \sin(\theta)) = 621.1 \text{ kN}$$

Tension within pile cap;

$$F_t = F_c \times \cos(\theta) / (2 \times \cos(30)) = 212.8 \text{ kN}$$

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;

$$P_c = 0.4 \times f_{cu} \times \pi \times \phi^2 / 4 = 2827.4 \text{ kN}$$

PASS Compression

Tension In One Truss Member

Characteristic strength of reinforcement;

$$f_y = 500 \text{ N/mm}^2$$

Partial safety factor for strength of steel;

$$\gamma_{ms} = 1.15$$

Req'd area of reinf't (in one truss member);

$$A_{s_req} = F_t / (1 / \gamma_{ms} \times f_y) = 489 \text{ mm}^2$$

Provided area of reinf't (in one truss member);

$$A_{s_prov} = A_{st} = 2011 \text{ mm}^2$$

Tension capacity of truss member;

$$P_t = (1 / \gamma_{ms} \times f_y) \times A_{s_prov} = 874.2 \text{ kN}$$

PASS Tension

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

Minimum required area of steel;

$$A_{st_min} = k_t \times A_c = 650 \text{ mm}^2$$

Maximum allowable area of steel;

$$A_{st_max} = 4 \% \times A_c = 20000 \text{ mm}^2$$

Area of tension steel provided OK

Beam Shear

Check shear stress on the sections at distance $\phi / 5$ inside face of piles.

Applied shear stress

Applied shear force;

$$V = F_{uls} / 3 = 500.0 \text{ kN}$$

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

Design shear stress;

$$v = V / (b_v \times d) = 0.30 \text{ N/mm}^2$$

Allowable shear stress;

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

Shear stress - OK

Design concrete shear strength

$$v_{c_25} = 0.79 \times r^{1/3} \times \max(0.67, (400 \text{ mm}/d)^{1/4}) \times 1.0 \text{ N/mm}^2 / 1.25 = 0.32 \text{ N/mm}^2$$

Shear enhancement;

$$v_c = v_{c_25} \times (\min(f_{cu}, 40 \text{ N/mm}^2) / 25 \text{ N/mm}^2)^{1/3} = 0.32 \text{ N/mm}^2$$

N/mm²

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

= 363 mm

Enhanced shear stress;

$$v_{c_enh} = \min(v_{allowable}, 2 \times d \times v_c / a_v) = 1.64 \text{ N/mm}^2$$

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; $u_0 = 2 \times (x + y) = 1200 \text{ mm}$

Assumed average depth to tension steel;

$$d_{av} = d - D_t = 924 \text{ mm}$$

Max shear effective across perimeter;

$$V_p = F_{uls} = 1500.0 \text{ kN}$$

Stress around loaded area;

$$v_{max} = V_p / (u_0 \times d_{av}) = 1.35 \text{ N/mm}^2$$

Allowable shear stress;
N/mm²

$$V_{\text{allowable}} = \min((0.8 \text{ N}^{1/2}/\text{mm}) \times \sqrt{f_{cu}}, 5 \text{ N/mm}^2) = 4.00$$

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;

$$\text{spacing}_{\text{bars}} = \max(0 \text{ mm}, (b_{\text{ccs}} - n_{\text{surfaces}} \times (C_{\text{adopt}} + L_{\text{dia}}) - D_t) / (L_{\text{nt}} - 1) - D_t) = 32 \text{ mm}$$

Maximum allowable spacing of bars;

$$\text{spacing}_{\text{max}} = \min((47000 \text{ N/mm}) / f_s, 300 \text{ mm}) = 300 \text{ mm}$$

Minimum required spacing of bars;

$$\text{spacing}_{\text{min}} = h_{\text{agg}} + 5 \text{ mm} = 25 \text{ mm}$$

Bar spacing OK

Clear Distance Between Face Of Beam And Tension Bars

Distance to face of beam;

$$\text{Dist}_{\text{edge}} = C_{\text{adopt}} + L_{\text{dia}} + D_t / 2 = 60 \text{ mm}$$

Design service stress in reinforcement;

$$f_s = 2 \times f_y \times A_{s_{\text{req}}} / (3 \times A_{s_{\text{prov}}} \times \beta_b) = 81.1 \text{ N/mm}^2$$

Max allowable clear spacing;

$$\text{Spacing}_{\text{max}} = \min((47000 \text{ N/mm}) / f_s, 300 \text{ mm}) = 300 \text{ mm}$$

Max distance to face of beam;

$$\text{Dist}_{\text{max}} = \text{Spacing}_{\text{max}} / 2 = 150 \text{ mm}$$

Max distance to beam edge check - OK

Anchorage Of Tension Steel

Anchorage factor;

$$\phi_{\text{factor}} = 44$$

Type of lap length;

$$\text{lap_type} = \text{"tens_lap"}$$

Type of reinforcement;

$$\text{reft_type} = \text{"def2_fy500"}$$

Minimum radius;

$$r_{\text{bar}} = 32 \text{ mm}$$

Minimum end projection;

$$P_{\text{bar}} = 130 \text{ mm}$$

Minimum anchorage length or lap length req'd;

$$= \phi_{\text{factor}} \times D_t = 704 \text{ mm}$$

Check anchorage length

$$= 12 \times D_t + d/2 = 662 \text{ mm}$$

Required minimum effective anchorage length;

$$L_a = 704 \text{ mm}$$

Check minimum radius required on bend

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

Force per bar at bend;

$$F_{\text{bt}} = F_t / L_{\text{nt}} = 21.3 \text{ kN}$$

Edge bar centres;

$$S_{\text{ext}} = C_{\text{adopt}} + D_t = 56 \text{ mm}$$

Edge maximum allowable bearing stress;

$$f_{\text{bt_max_ext}} = 2 \times f_{cu} / (1 + 2 \times (D_t / S_{\text{ext}})) = 31.82 \text{ N/mm}^2$$

Internal bar centres;

$$S_{\text{int}} = \text{spacing}_{\text{bars}} + D_t = 48 \text{ mm}$$

Internal maximum allowable bearing stress;

$$f_{\text{bt_max_int}} = 2 \times f_{cu} / (1 + 2 \times (D_t / S_{\text{int}})) = 30.00 \text{ N/mm}^2$$

Design max allowable bearing stress;

$$f_{\text{bt_max}} = \min(f_{\text{bt_max_ext}}, f_{\text{bt_max_int}}) = 30.00 \text{ N/mm}^2$$

Minimum radius required;

$$r_{\text{min}} = \max(r_{\text{bar}}, F_{\text{bt}} / (f_{\text{bt_max}} \times D_t)) = 44.3 \text{ mm}$$

Minimum radius of bend required = 44 mm

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STRUCTURAL DRAWINGS

