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RE REGULATORY RECORD

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DIGITALLY VERIFIED RECORD

**KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTA**

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

Mob: 98404 48613 eMail Id: kirtiassociates@gmail

## STABILITY CERTIFICATE

For,

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

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

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

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

For KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANT

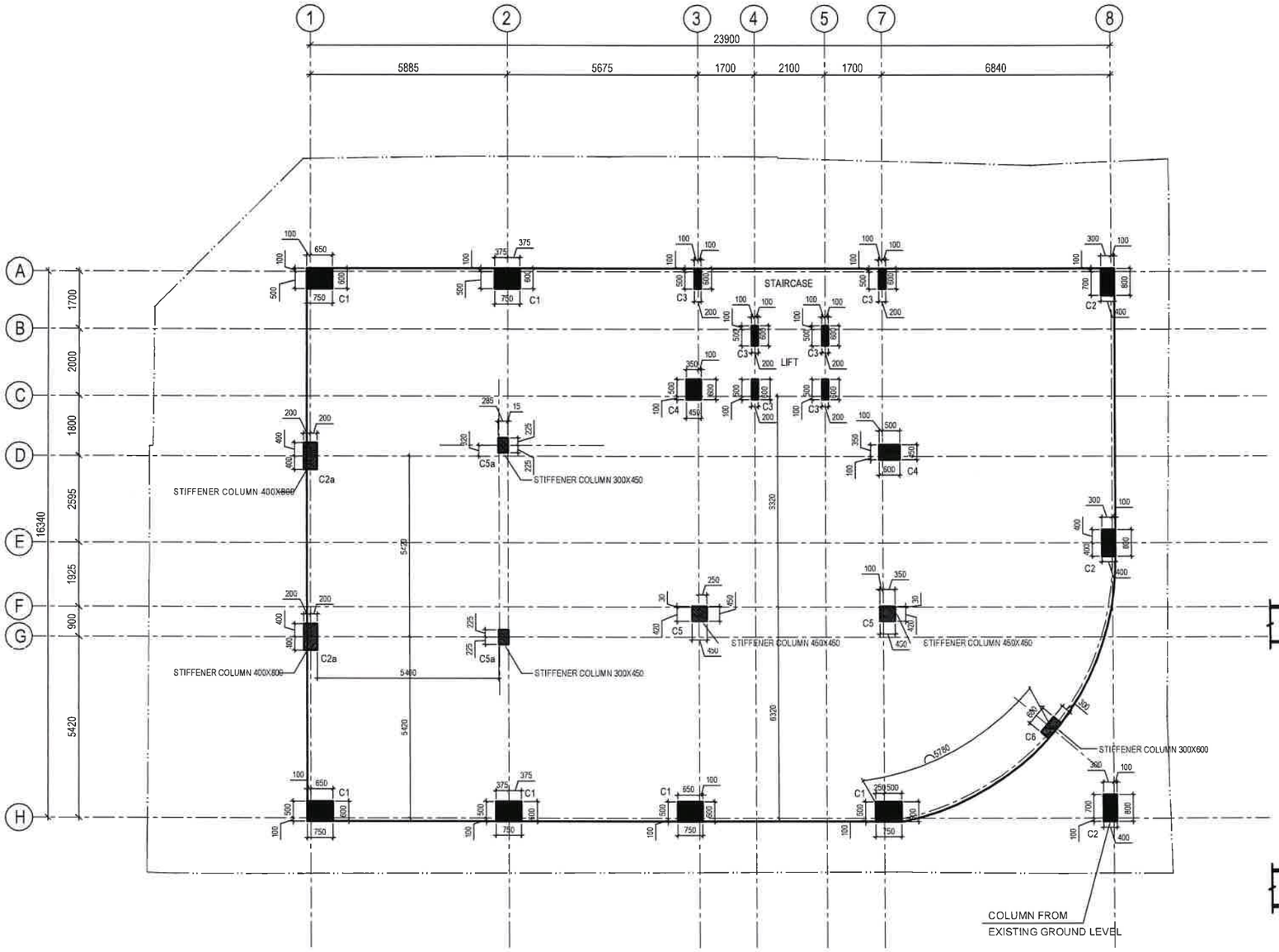
ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
# 97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 9840448613

Date: 08-10-2025

Place: Chennai

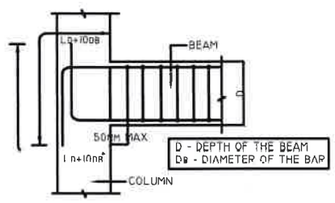
E.S. Kumar., M.Tec

Principal Structural Consulta

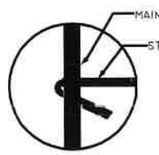



## COLUMN AXIS LAYOUT

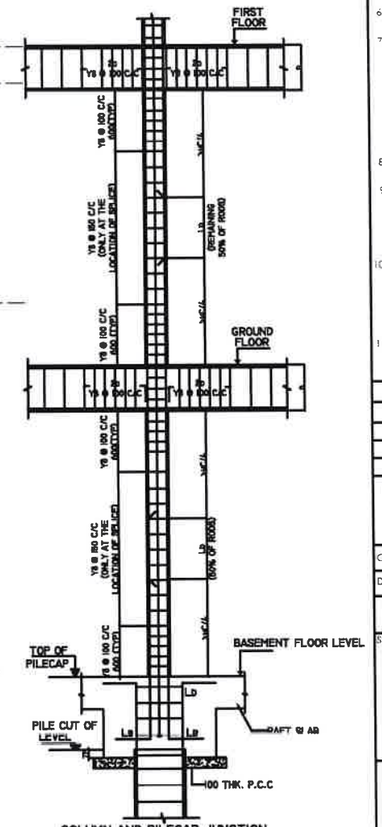
ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No.SE/Gr-I/2024/02/007
#97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell : 9840448613




TYP. EXTERNAL JOINT DETAIL AT BEAM-COLUMN JUNCTION




HOOK DETAIL




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

**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERING):**

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

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

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

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

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

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

**LAP JOINTS FOR REINFORCEMENT BARS:**

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

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

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

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 400 | 400 | 370 | 350 | 370 | 350 | 280 | 280 |
| 10mm | 500 | 480 | 460 | 400 | 480 | 390 | 370 | 320 |
| 12mm | 600 | 580 | 550 | 480 | 580 | 470 | 440 | 380 |
| 16mm | 800 | 780 | 730 | 640 | 730 | 630 | 590 | 520 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1410 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1280 | 1460 | 1250 | 1170 | 1030 |

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

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

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

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

**GENERAL NOTES:**

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

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

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

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

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

CLIENT: EMPEE GROUP

DRAWING TITLE: COLUMN AXIS LAYOUT

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

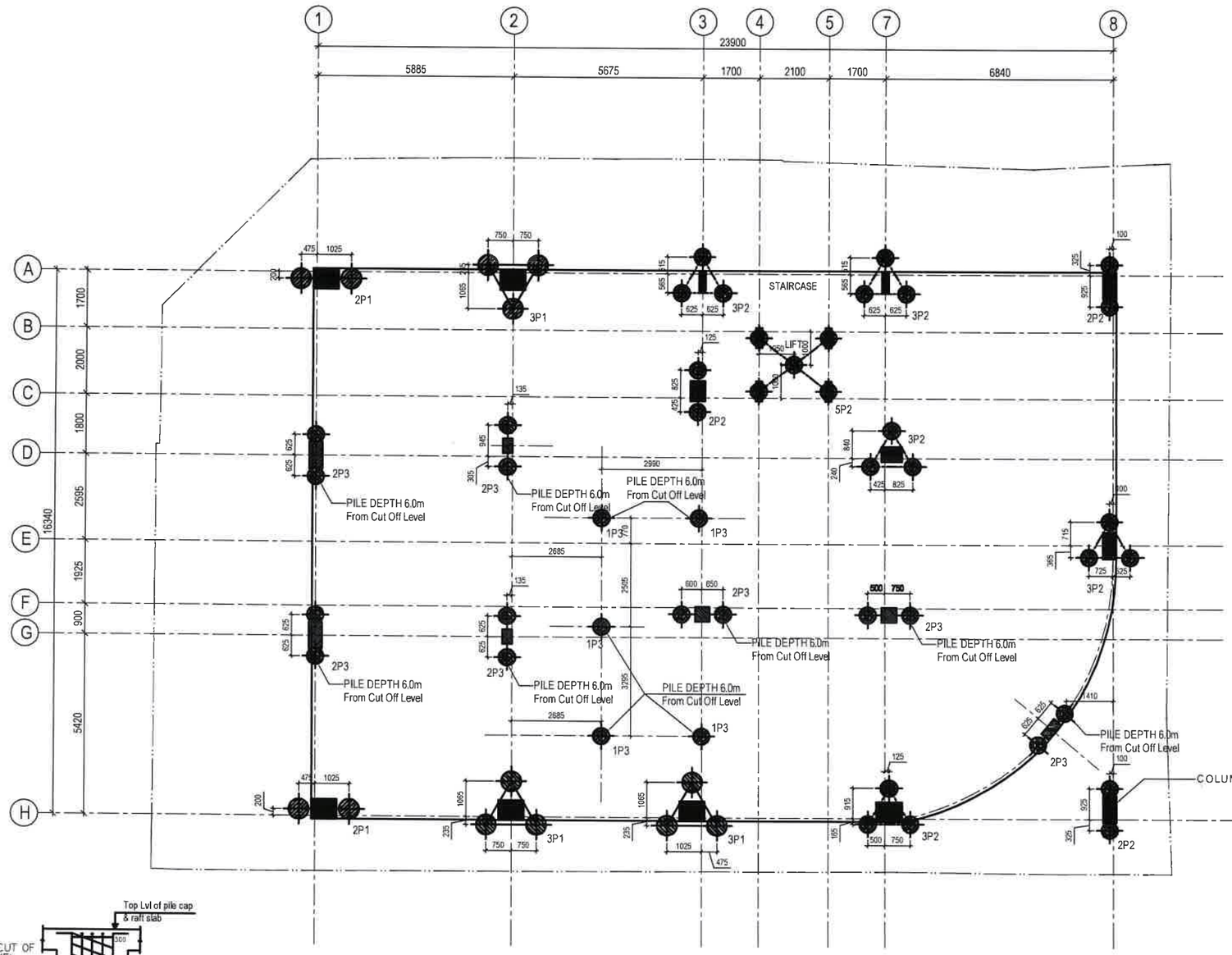
STRUCTURAL CONSULTANTS:

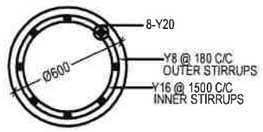
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob: 98404 48613.

ARCHITECTS:

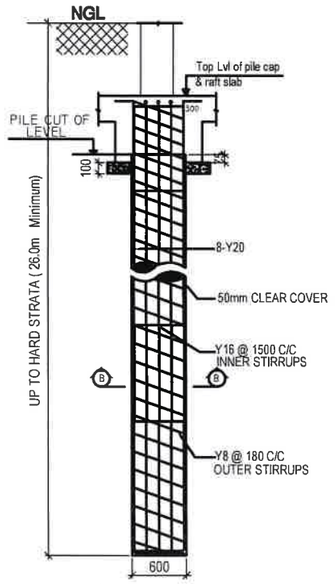
Harismaa Consultant Ltd
#109 A&E, 1st FLOOR 3rd AVENUE
ANNA NAGAR EAST CHENNAI - 600102
Tel: 044-26209147

| Drawn M.D | Designed DANE | Checked ES KUMAR | |
|---|---|---|---|

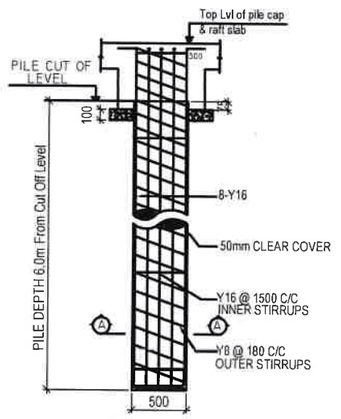



SECTION "B-B"
C.S OF PILE - 600 DIA.

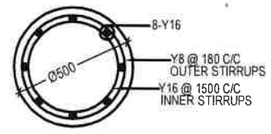



L.S OF 600mm DIA. PILE - P1

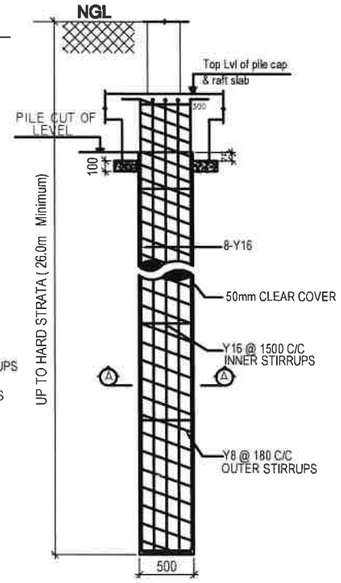



L.S OF 500mm DIA. PILE - P3

PILE GROUP LAYOUT




SECTION "A-A"
C.S OF PILE - 500 DIA.




L.S OF 500mm DIA. PILE - P2

**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERING):**

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

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

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

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

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

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

**LAP JOINTS FOR REINFORCEMENT BARS:**

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

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

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

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 460 | 400 | 370 | 320 | 370 | 320 | 290 | 260 |
| 10mm | 580 | 490 | 460 | 400 | 460 | 390 | 370 | 320 |
| 12mm | 690 | 590 | 550 | 480 | 550 | 470 | 440 | 390 |
| 16mm | 900 | 780 | 730 | 640 | 730 | 630 | 590 | 520 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1419 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1300 | 1460 | 1250 | 1170 | 1030 |

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

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

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

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

**GENERAL NOTES:**

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

| | SAFE LOAD IN COMPRESSION (T) |
|---|---|
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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

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

PROJECT NAME :

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX

AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: 500 DIA. PILE GROUP LAYOUT

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

STRUCTURAL CONSULTANTS:

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

ARCHITECTS:

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

| Drawn M.D | Designed DANE | Checked ES.KUMAR | |
|---|---|---|---|

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
Reg. No. SE/Gr-I/2024/02/007
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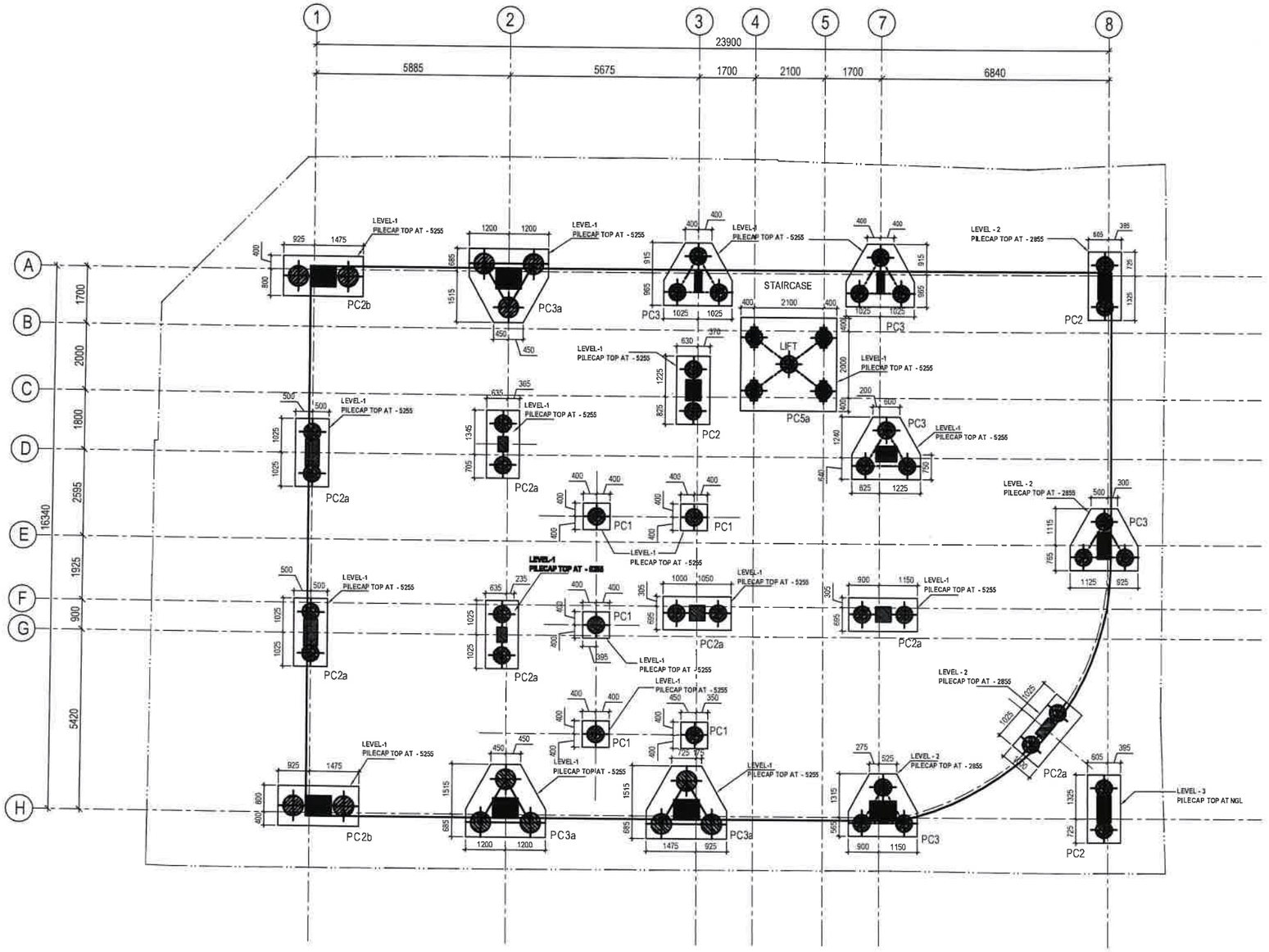
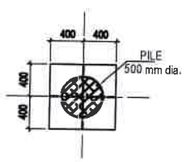
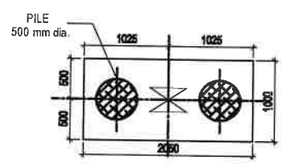


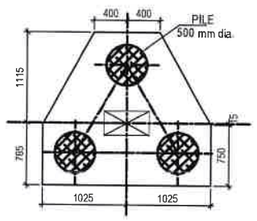

## PILE CAP LAYOUT



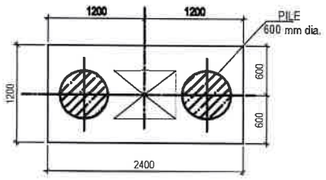

PC1 - LAYOUT



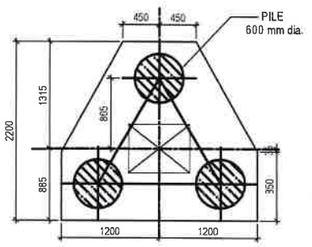

PC2 - LAYOUT




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 15456 2000

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

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

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

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

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

**LAP JOINTS FOR REINFORCEMENT BARS:**

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

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

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

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 440 | 400 | 370 | 350 | 370 | 350 | 290 | 280 |
| 10mm | 550 | 490 | 460 | 430 | 460 | 390 | 370 | 350 |
| 12mm | 660 | 590 | 550 | 490 | 550 | 470 | 440 | 390 |
| 16mm | 900 | 780 | 730 | 640 | 730 | 620 | 580 | 500 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1410 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1300 | 1460 | 1250 | 1170 | 1030 |

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

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

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

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

**GENERAL NOTES:**

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

| DIA OF PILE ( MM) | SAFE LOAD IN COMPRESSION (T) |
|---|---|
| 60C MM (P1) | 132T |
| 50C MM (P2) | 73T |

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

| REV. | DESCRIPTION | DRN. | CHK. | DATE |
|---|---|---|---|---|
| 04 | 600 DIA PILE CAP ADDED | MD | ESK | 11.10.21 |
| 03 | GOOD FOR CONSTRUCTION | MD | ESK | 14.09.21 |
| 02 | PC1 ADDED FOR SUMP WALL SUPPORT | MD | ESK | 10.09.21 |
| 01 | ISSUED FOR CONSTRUCTION | MD | ESK | 17.08.21 |
| 00 | ISSUED FOR APPROVAL | MD | ESK | 06.08.21 |

PROJECT NAME :

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX

AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: PILE CAP LAYOUT

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


STRUCTURAL CONSULTANTS:

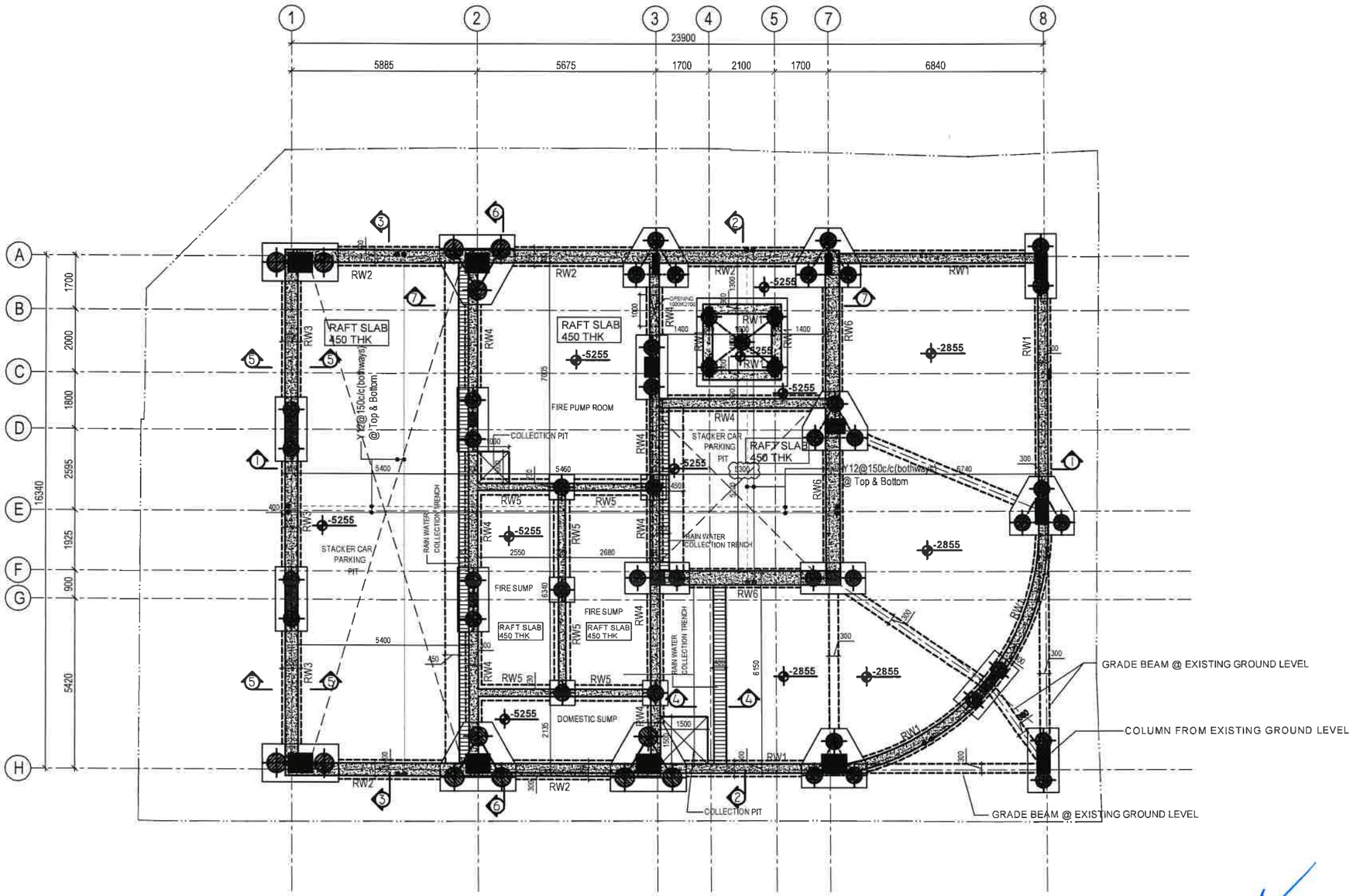
**KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,**
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai - 600 040. Mob: 98404 48613.

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

ARCHITECTS:

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


| Drawn M.D | Designed DANE | Checked ES.KUMAR | |
|---|---|---|---|
| | | | |




# GRADE BEAM, R.C WALL AND RAFT SLAB LAYOUT

**TOC - 5255**

**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERING):**

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

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

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

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

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

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

**LAP JOINTS FOR REINFORCEMENT BARS:**

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

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

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

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 450 | 400 | 370 | 320 | 370 | 320 | 290 | 280 |
| 10mm | 560 | 490 | 460 | 400 | 460 | 390 | 370 | 350 |
| 12mm | 680 | 590 | 550 | 480 | 550 | 470 | 440 | 360 |
| 16mm | 900 | 780 | 730 | 640 | 730 | 630 | 580 | 520 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1410 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1300 | 1460 | 1250 | 1170 | 1050 |

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

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

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

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

**GENERAL NOTES:**

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

| DIA. OF PILE ( MM) | SAFE LOAD IN COMPRESSION (T) |
|---|---|
| 60C MM (P1) | 137T |
| 50C MM (P2) | 73T |

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

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

PROJECT NAME :

PROPOSED COMMERCIAL BUILDING - ARUNA COMPLEX

AT ANNA NAGAR EAST, CHENNAI 600102.

CLIENT: EMPEE GROUP

DRAWING TITLE: RAFT SLAB AND R.C WALL LAYOUT

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


STRUCTURAL CONSULTANTS:

**KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,**
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
Chennai 600 040. Mob: 98401 18613.

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

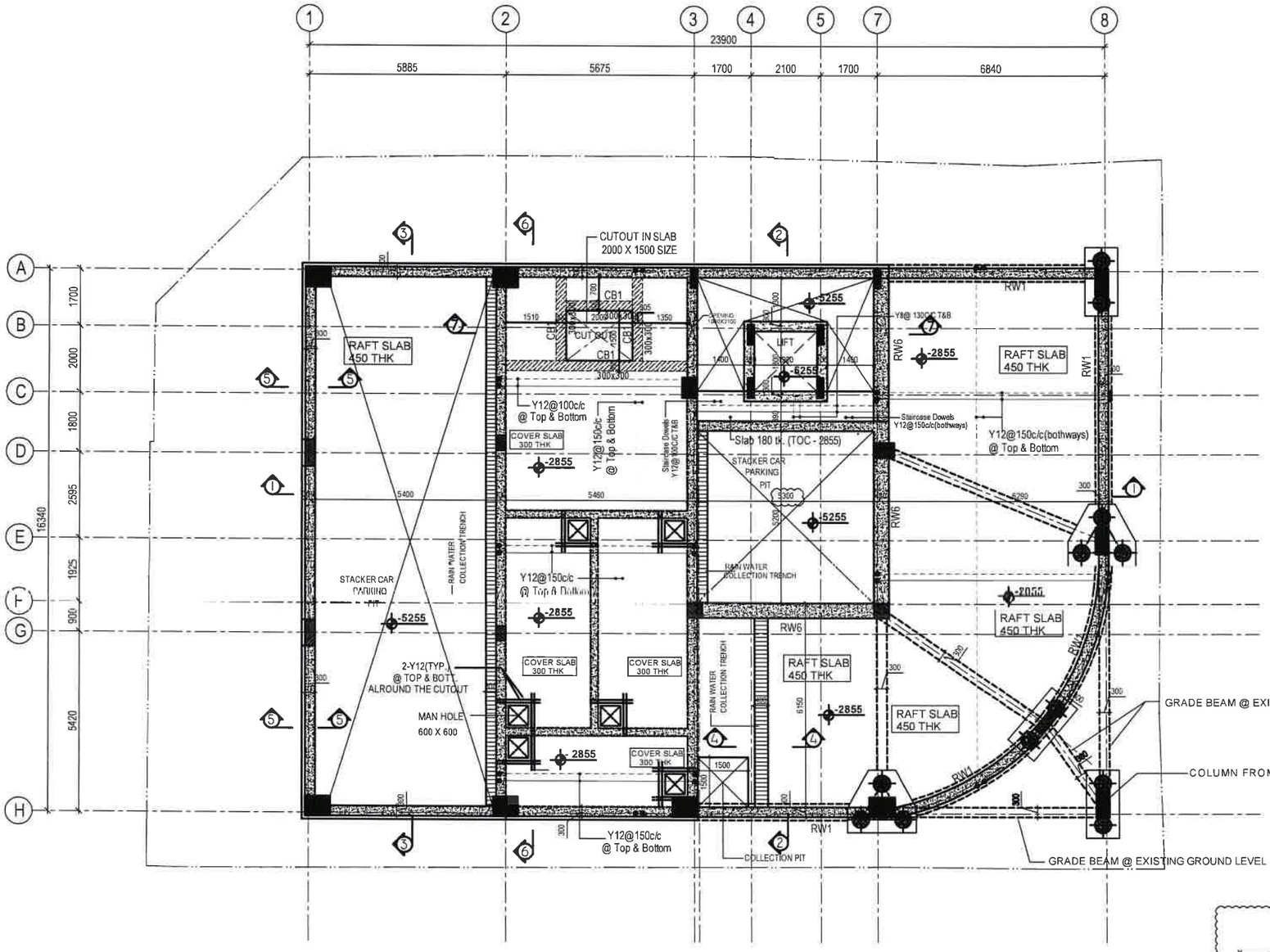
**ARCHITECTS:**

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 | |
|---|---|---|---|

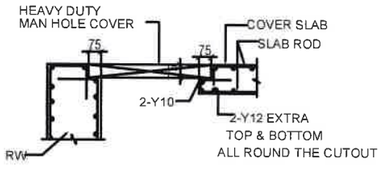
**IMPORTANT NOTE:**

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



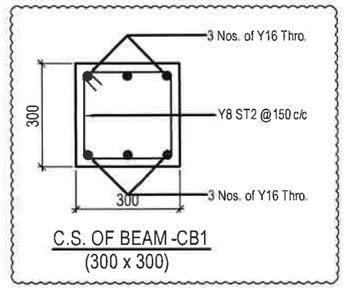

## GRADE BEAM, R.C WALL, SUMP COVER SLAB AND RAFT SLAB LAYOUT

**TOC - 2855**




TYPICAL MANHOLE DETAIL

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




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

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

**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERING):**

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

A. VERTICAL SIDE OF SHUTTERING OF COLUMNS, WALLS AND B. 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 4.5DM. ——— 7 DAYS.
ii) SPANNING OVER 4.5DM ——— 14 DAYS.

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

**LAP JOINTS FOR REINFORCEMENT BARS:**

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

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

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

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 400 | 400 | 370 | 350 | 370 | 320 | 290 | 280 |
| 10mm | 500 | 490 | 460 | 430 | 460 | 390 | 370 | 350 |
| 12mm | 590 | 580 | 550 | 490 | 550 | 470 | 440 | 390 |
| 16mm | 800 | 780 | 730 | 640 | 730 | 630 | 580 | 520 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1410 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1300 | 1460 | 1250 | 1170 | 1030 |

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

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

**GENERAL NOTES:**

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

| DIA. OF PILE ( MM) | SAFE LOAD IN COMPRESSION (T) |
|---|---|
| 600 MM (P1) | 137.1 |
| 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

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

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

CLIENT: EMPEE GROUP

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

| JOB NO. | 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:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR (WEST)
Chennai - 600 040. Mob: 90404 40610.

**ARCHITECTS:**
Harismaa Consultant Ltd
#105 / A46, 1st FLOOR, 3rd AVENUE,
ANNA NAGAR EAST, CHENNAI - 600102
Tel: 044 - 26203747

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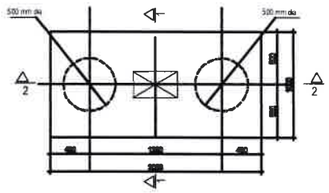
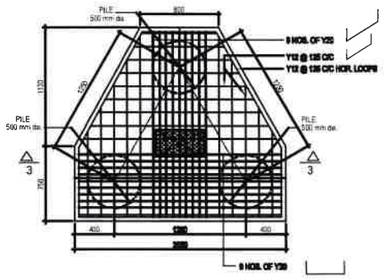
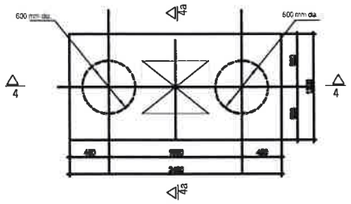


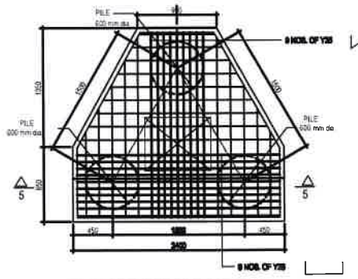

PLAN OF DOUBLE PILE CAP (PC2)



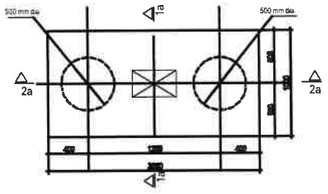

PLAN OF THREE PILE CAP (PC3)
PLAN OF BOTTOM REINFORCEMENT



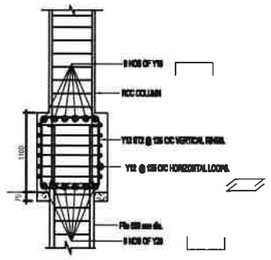

PLAN OF DOUBLE PILE CAP (PC2b)



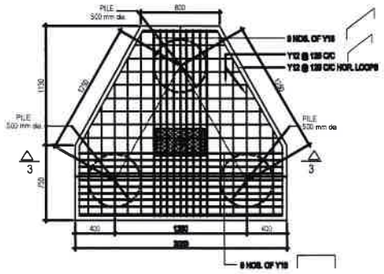

PLAN OF THREE PILE CAP (PC3a)
PLAN OF BOTTOM REINFORCEMENT



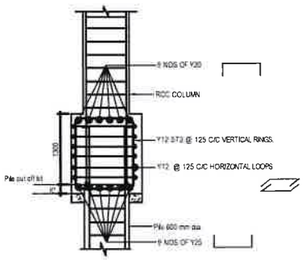

PLAN OF DOUBLE PILE CAP (PC2a)



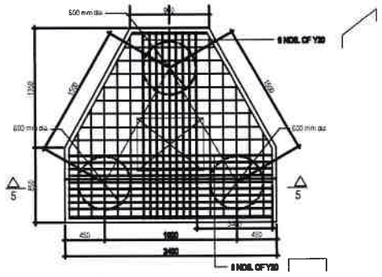

SECTION 1 - 1



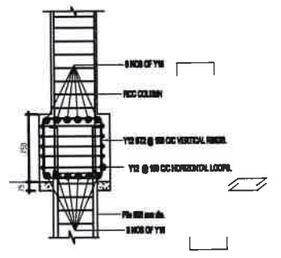

PLAN OF THREE PILE CAP (PC3)
PLAN OF TOP REINFORCEMENT




SECTION 4a - 4a




PLAN OF THREE PILE CAP (PC3a)
PLAN OF TOP REINFORCEMENT

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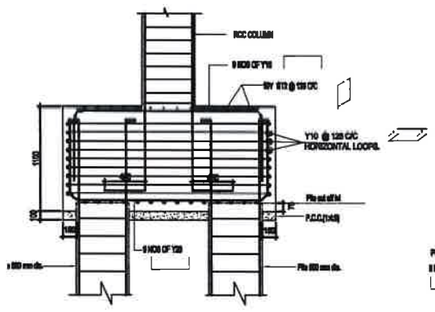


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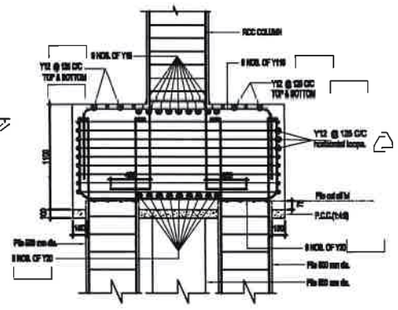


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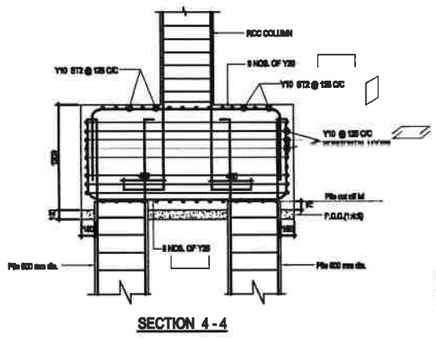


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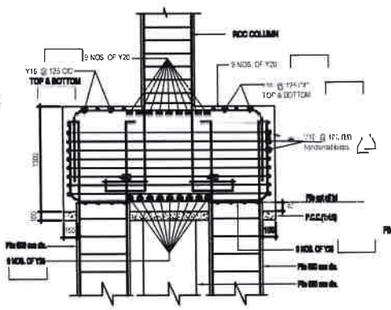
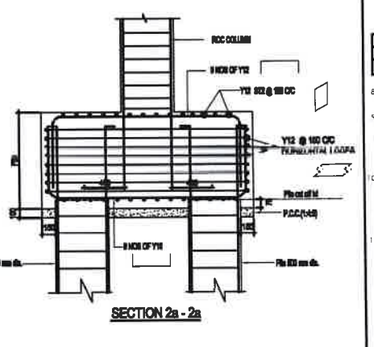
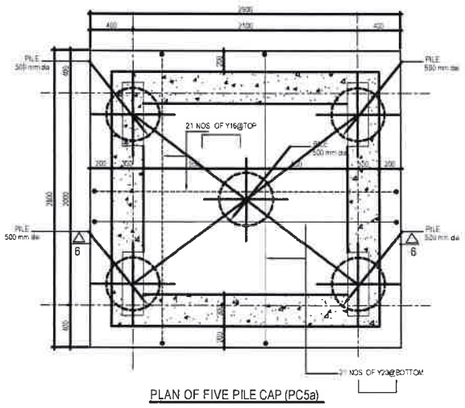


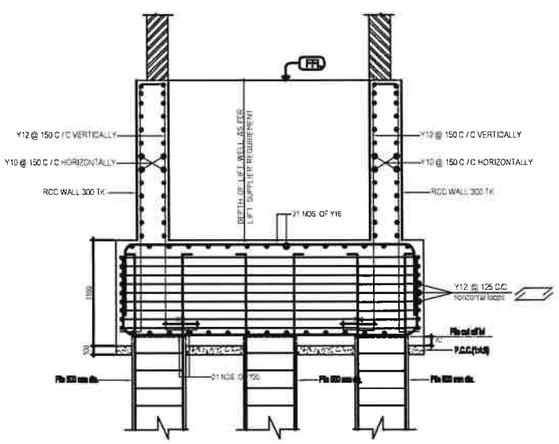

SECTION 5 - 5



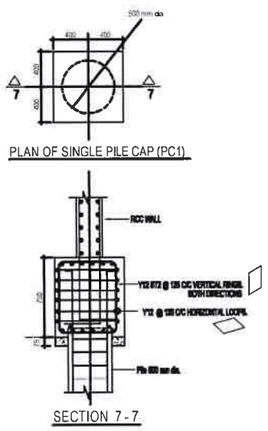

SECTION 2a - 2a




PLAN OF FIVE PILE CAP (PC5a)




SECTION 6 - 6




PLAN OF SINGLE PILE CAP (PC1)

SECTION 7 - 7

## COLUMN SCHEDULE

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

**GENERAL NOTES**

**REMOVAL OF FORM WORK (SHUTTERING)**

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

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

**LAP JOINTS FOR REINFORCEMENT BARS**

i) AT ANY CROSS SECTION OF A MEMBER NOT MORE THAN 50% OF THE BARS SHALL BE LAPPED
ii) LAPS SHALL BE STAGGERED WITH A MINIMUM CENTRE TO CENTRE DISTANCE OF 1.2 TIMES THE LAP LENGTH OF THE BAR FOR TENSION AND COMPRESSION MEMBERS
iii) DEVELOPMENT LENGTH (Ld) SHALL BE AS PER SP: 34 - 1999 AND GIVEN IN THE TABLE BELOW

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 40 | 40 | 35 | 35 | 30 | 30 | 25 | 25 |
| 10mm | 50 | 50 | 45 | 40 | 40 | 40 | 35 | 35 |
| 12mm | 60 | 60 | 50 | 50 | 45 | 45 | 40 | 40 |
| 16mm | 80 | 80 | 70 | 65 | 60 | 60 | 55 | 50 |
| 20mm | 100 | 100 | 90 | 80 | 75 | 75 | 70 | 65 |
| 25mm | 125 | 125 | 110 | 100 | 95 | 95 | 85 | 80 |
| 32mm | 160 | 160 | 140 | 130 | 120 | 120 | 110 | 100 |

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 CONTRA FLEXURE
vi) WELDING OF BARS SHALL BE DONE AS A STITCH WELD FOR 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT
vii) DETAILING OF REBARS SHALL CONFIRM TO SP-34 & IS 13920

**GENERAL NOTES.**

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

| DIA. OF PILE ( MM ) | SAFE LOAD IN COMPRESSION (T) |
|---|---|
| 400 MM (P1) | 137 |
| 500 MM (P2) | 737 |

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) 1998 GIVEN IN TABLE 8 ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM INITIAL PILE TESTS CONDUCTED TO PROVEN THE 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 THE CATEGORY IS TESTED EITHER TO ITS ULTIMATE LOAD CAPACITY OR TO 1.5 TIMES THE ESTIMATED SAFE LOAD THE LOWEST OF VALUE OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) : 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES


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


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

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

CLIENT: EMPEE GROUP

DRAWING TITLE: FOUNDATION PILE CAP DETAIL

| JOB NO | DWG NO | REV NO OR Date | REF. CODE |
|---|---|---|---|
| P.A. | S-05 | 11.10.21 | |

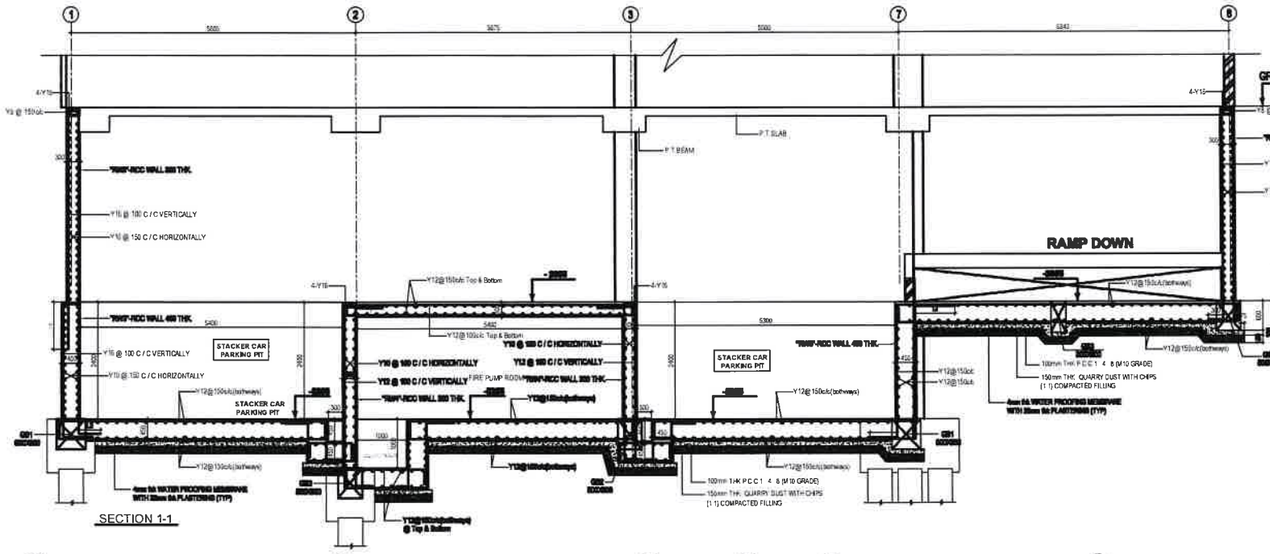
STRUCTURAL CONSULTANTS


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

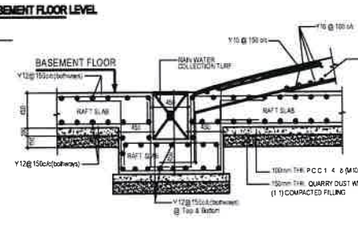

ARCHITECTS:

Harismaa Consultant Ltd

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

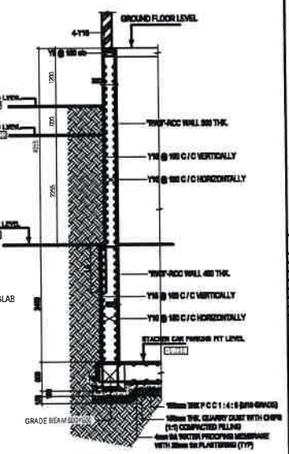



SECTION 1-1

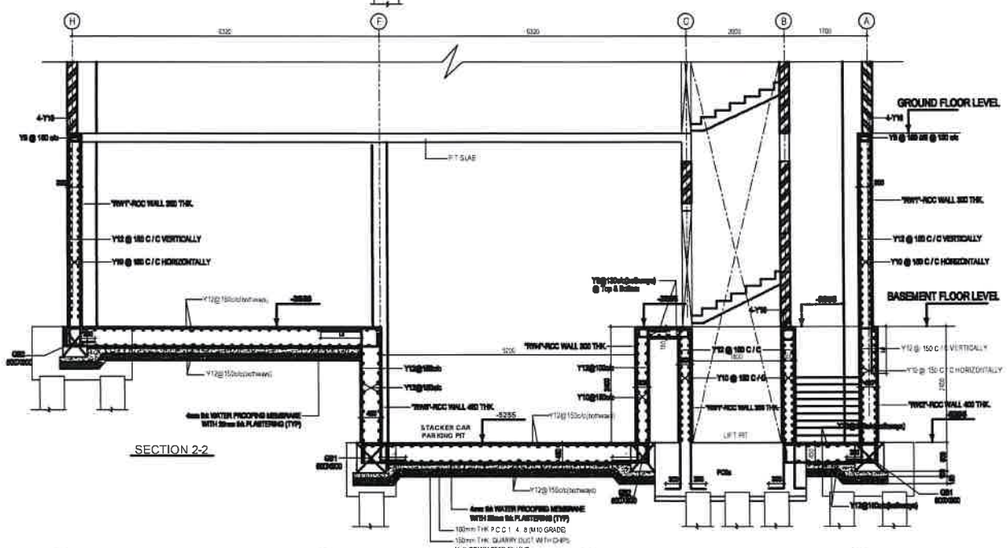



SECTION 4-4

TYP DETAIL OF RAMP SLAB DOWELS




SECTION 5-5

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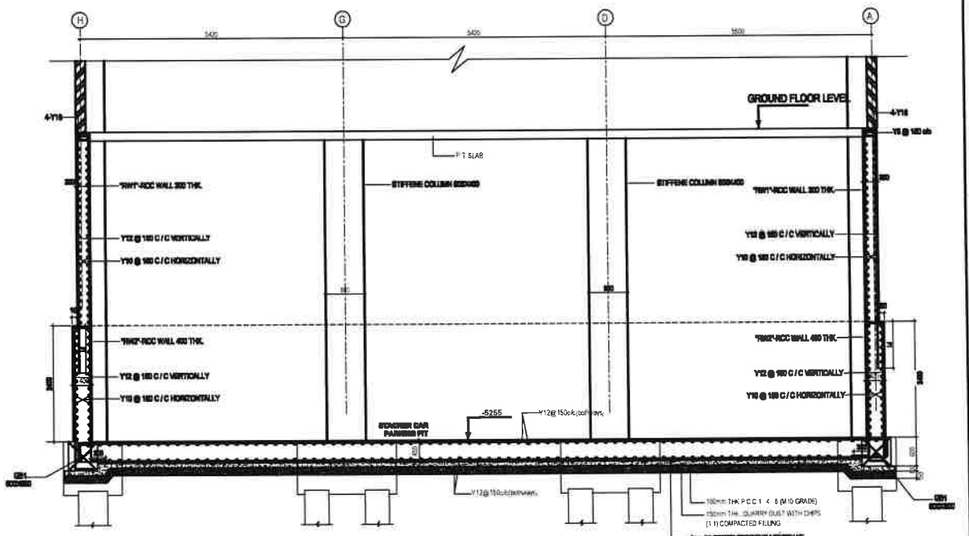


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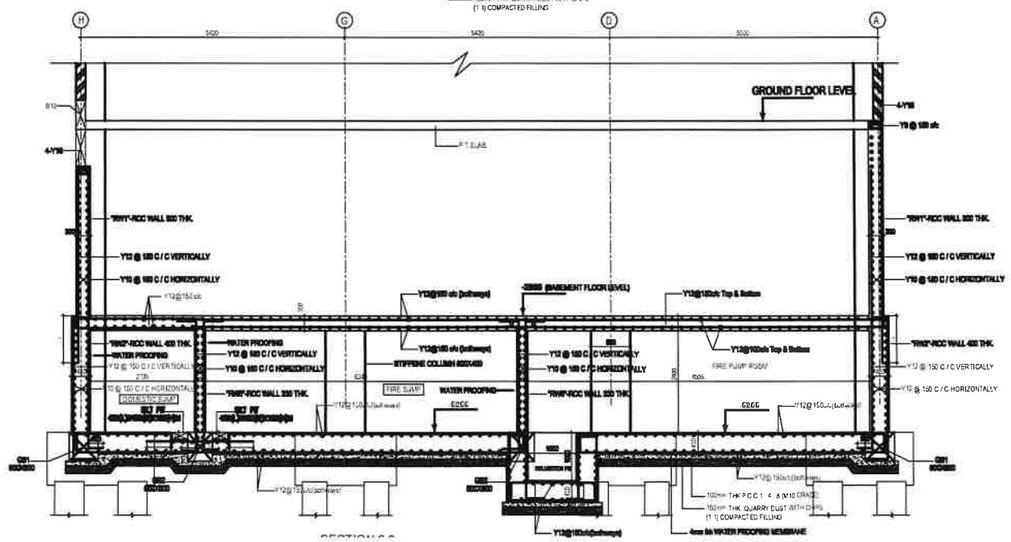
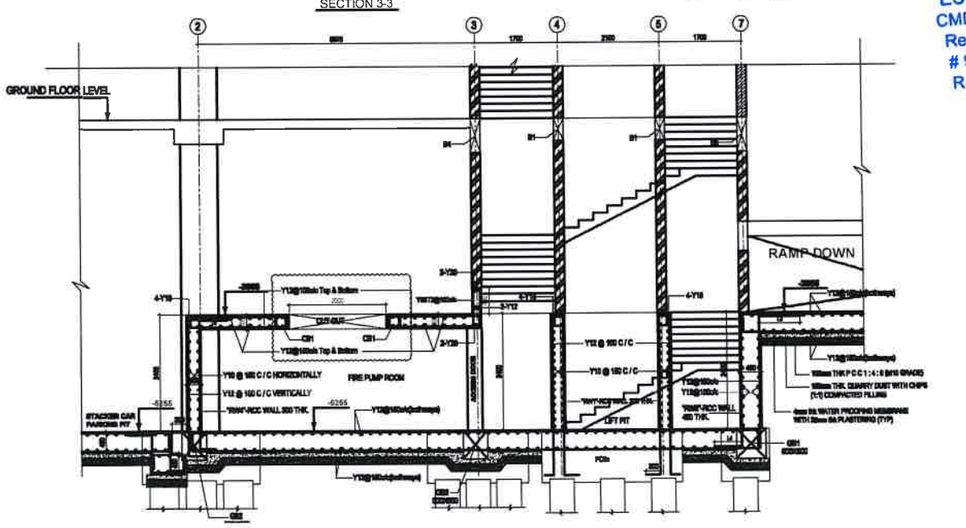



SECTION C-C




SECTION 6-6

**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERINGS)**

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 | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 32 | 32 | 32 | 32 | 24 | 24 | 24 | 24 |
| 10mm | 40 | 40 | 40 | 40 | 30 | 30 | 30 | 30 |
| 12mm | 48 | 48 | 48 | 48 | 36 | 36 | 36 | 36 |
| 16mm | 64 | 64 | 64 | 64 | 48 | 48 | 48 | 48 |
| 20mm | 80 | 80 | 80 | 80 | 60 | 60 | 60 | 60 |
| 25mm | 100 | 100 | 100 | 100 | 75 | 75 | 75 | 75 |
| 32mm | 128 | 128 | 128 | 128 | 96 | 96 | 96 | 96 |
| 36mm | 144 | 144 | 144 | 144 | 108 | 108 | 108 | 108 |
| 40mm | 160 | 160 | 160 | 160 | 120 | 120 | 120 | 120 |

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

| DIA. OF PILE ( MM ) | SAFE LOAD IN COMPRESSION ( T ) |
|---|---|
| 600 MM (P1) | 137.1 |
| 500 MM (P2) | 78.1 |

8. THE BUILDING FOUNDATION HAS BEEN DESIGNED FOR BASEMENT + GROUND FLOOR + 7 FLOORS ONLY
9. THE SAFE LOAD CARRYING CAPACITY OF THE PILE DERIVED FROM THE SOIL PARAMETERS AS PER IS : 2911 (PART 1/SEC-2) -1979 GIVEN IN TABLE IS ONLY TENTATIVE. THE ACTUAL SAFE LOAD CARRYING CAPACITY OF PILE HAS TO BE DERIVED FROM TRIAL PILE HENCE IT IS SUGGESTED TO PROVIDE TRIAL PILE IN THE PROPOSED SITE
10. WORKING PILE : INITIALLY ONE OF 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 VARYING THE LOWEST OF VALUES OF SAFE LOAD SHALL BE CONSIDERED FOR THE DESIGN OF FOUNDATION
11. THE BUILDING HAS BEEN DESIGNED FOR EARTHQUAKE RESISTANCE AS PER IS 1893 (PART 1) : 2002 - CODE FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

ES. KUMAR, M.Tech.- Structures
CMDA Structural Engineer-Grade-1,
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# 97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell : 9840448613

| REV | DESCRIPTION | DRN | CHK | DATE |
|---|---|---|---|---|
| 03 | ISSUED FOR CONSTRUCTION | MD | ESK | 14.09.21 |
| 02 | FIRE PUMP ROOM & FIRE SUMP ADDED | MD | ESK | 16.09.21 |
| 01 | ISSUED FOR CONSTRUCTION | MD | ESK | 17.09.21 |
| 00 | ISSUED FOR APPROVAL | MD | ESK | 06.09.21 |

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

CLIENT: EMPEE GROUP

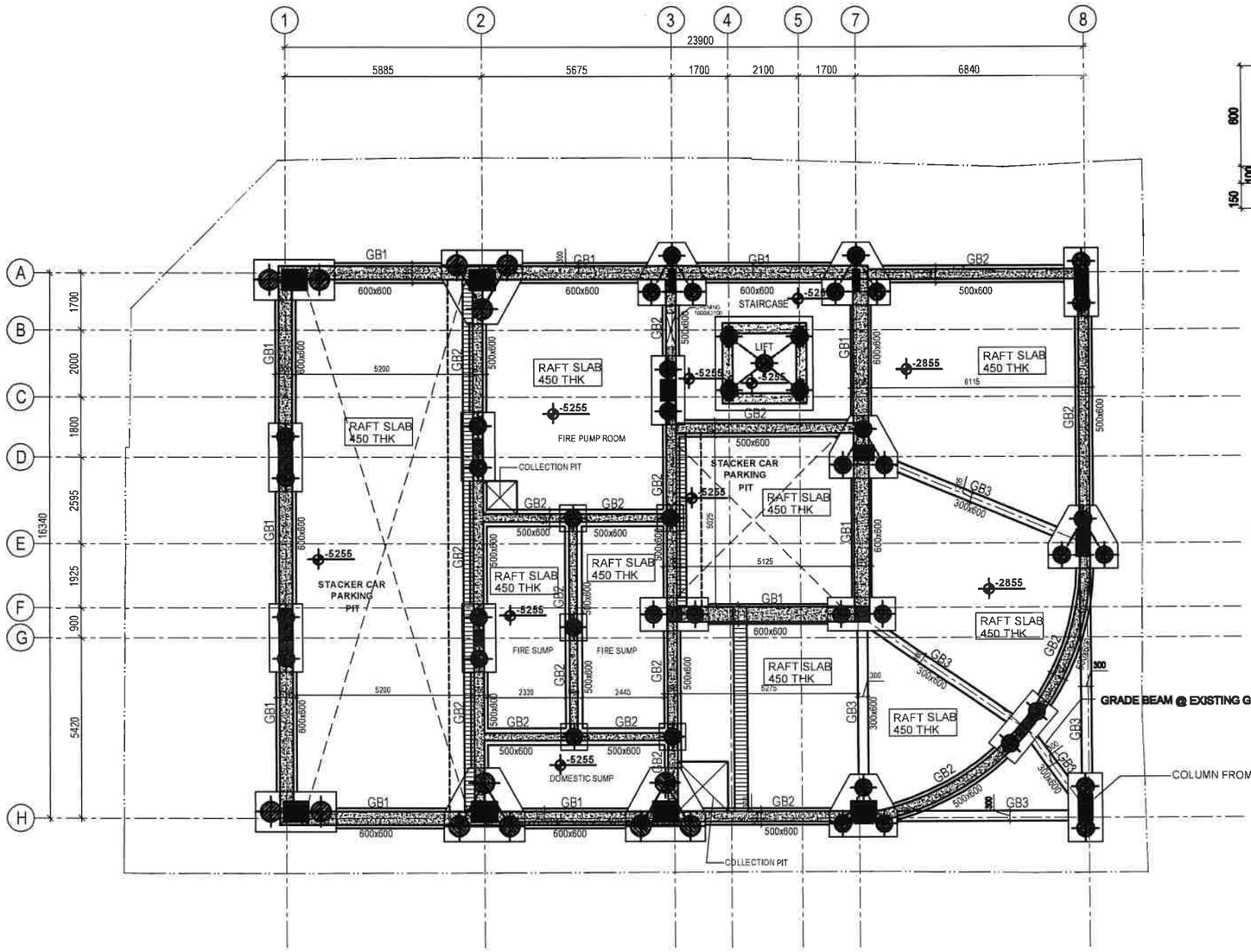
DRAWING TITLE: SECTION DETAILS

JOB NO: KA-
DWG NO: S-06
REV NO: 03
Date: 17.09.21
PEF CODE

STRUCTURAL CONSULTANTS:
KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,
PLOT NO 1233, FIRST FLOOR
FLAT NO 16, PRIYA AJAY ARCADE
18th MAIN ROAD, ANNA NAGAR (WEST)
Chennai - 600 040. Mob: 98404 48613

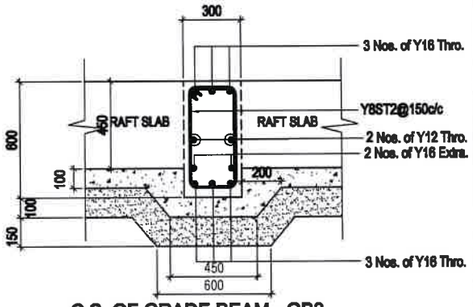
ARCHITECTS:
Harismaa Consultant Ltd
KODAMBAKKAM HIGH ROAD, NUNGAMBAKKAM, CHENNAI - 600 034

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

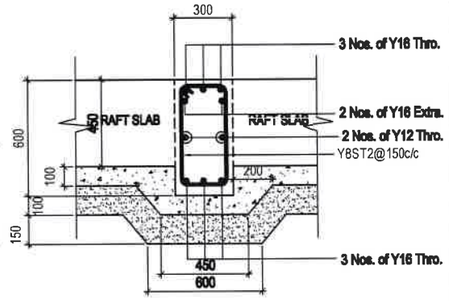



## GRADE BEAM LAYOUT

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




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




**C.S. OF GRADE BEAM - GB3**
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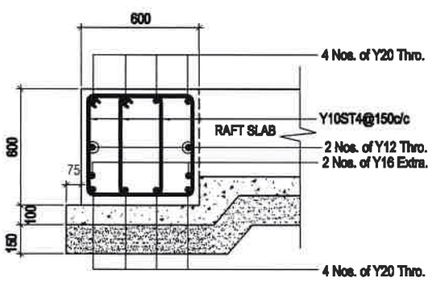


**C.S. OF GRADE BEAM - GB1**
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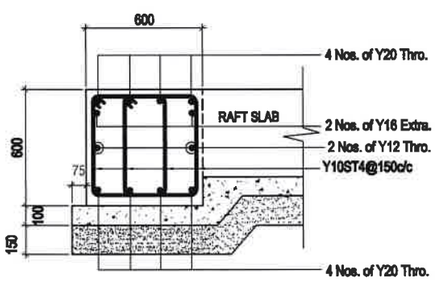
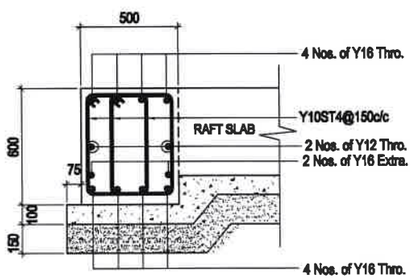
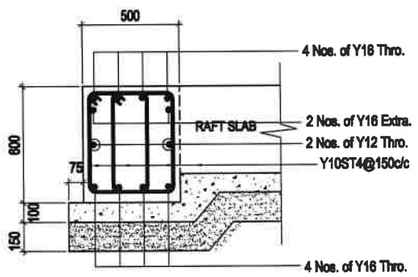



**C.S. OF GRADE BEAM - GB1**
**(600 x 600)**
**(@SUPPORT)**




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




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

**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERING):**

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

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

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

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

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

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

**LAP JOINTS FOR REINFORCEMENT BARS:**

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

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

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

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 450 | 400 | 370 | 350 | 370 | 320 | 290 | 280 |
| 10mm | 565 | 490 | 460 | 430 | 460 | 390 | 370 | 350 |
| 12mm | 680 | 590 | 550 | 480 | 550 | 470 | 440 | 390 |
| 16mm | 900 | 780 | 730 | 640 | 730 | 630 | 580 | 520 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1410 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1280 | 1460 | 1250 | 1170 | 1050 |

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

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

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

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

**GENERAL NOTES:**

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

| DIA. OF PILE ( MM ) | SAFE LOAD IN COMPRESSION (T) |
|---|---|
| 600 MM (P1) | 137 T |
| 300 MM (P2) | 75 T |

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

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

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

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

CLIENT: EMPEE GROUP

DRAWING TITLE: GRADE BEAM DETAIL

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

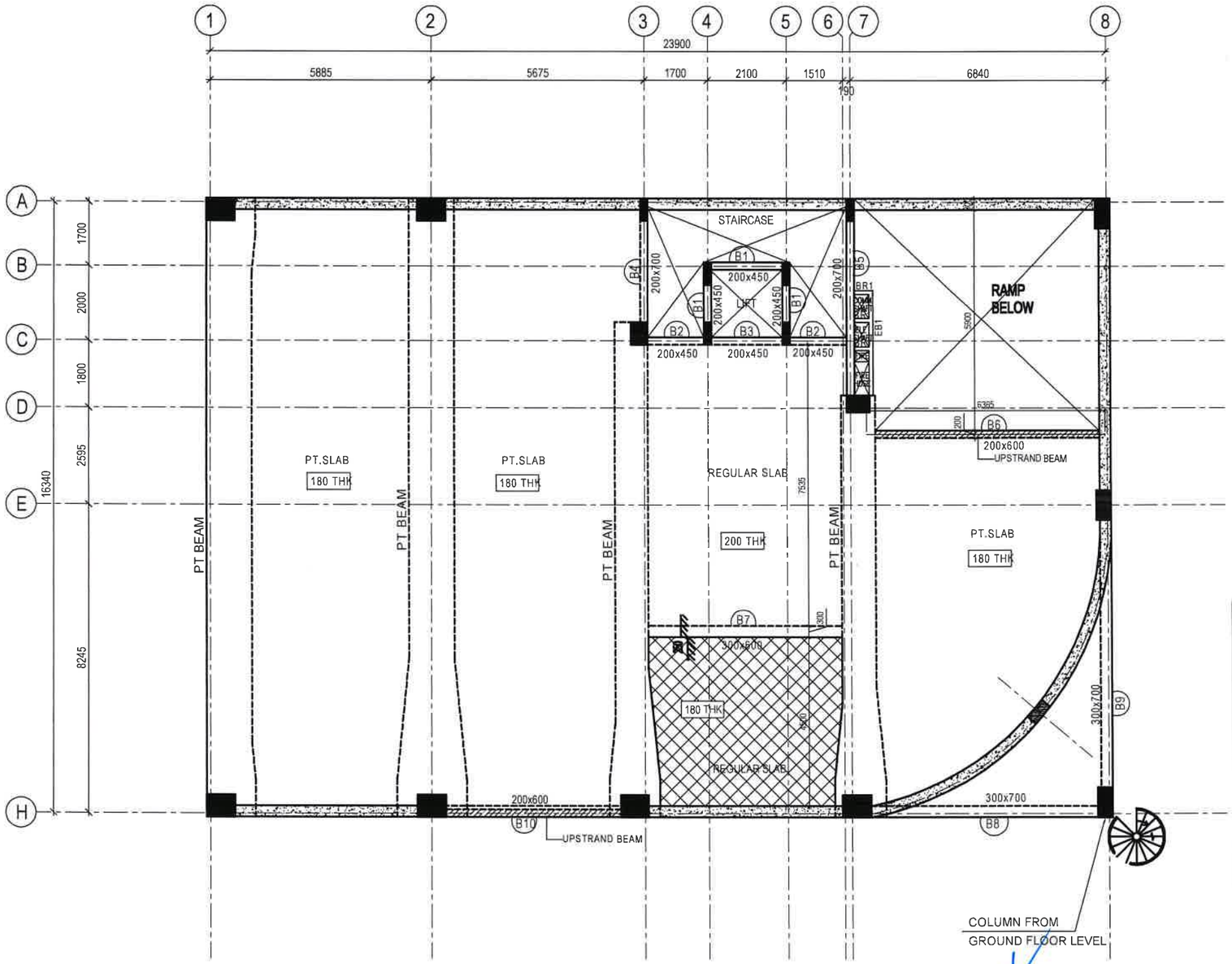
STRUCTURAL CONSULTANTS:

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

ARCHITECTS:

Harismaa Consultant Ltd
#109/A46, 1st FLOOR, 3rd AVENUE
ANNA NAGAR EAST, CHENNAI - 600102
Tel 044 - 26205147

| Drawn M.D | Designed DANE | Checked ES.KUMAR | |
|---|---|---|---|
| | | | |




BASEMENT ROOF BEAM LAYOUT

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

LEGEND

▨ = SUNKEN BY 250


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


**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERING):**

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

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

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

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

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

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

**LAP JOINTS FOR REINFORCEMENT BARS:**

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

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

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

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M25 | M30 | M35 | M35 | M25 | M30 | M35 | M35 |
| 8mm | 450 | 400 | 370 | 320 | 370 | 320 | 290 | 280 |
| 10mm | 565 | 490 | 460 | 400 | 460 | 390 | 370 | 320 |
| 12mm | 680 | 590 | 550 | 480 | 550 | 470 | 440 | 390 |
| 16mm | 900 | 780 | 730 | 640 | 730 | 630 | 580 | 520 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1410 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1280 | 1460 | 1250 | 1170 | 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 5 TIMES THE DIAMETER OF BAR AND A GAP OF 5 TIMES OF BAR AND AGAIN WELDING FOR ANOTHER 5 TIMES DIAMETER OF BAR USING ARC WELDING AND SPECIAL ELECTRODES AFTER GETTING PRIOR WRITTEN APPROVAL FROM THE STRUCTURAL CONSULTANT.

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

**GENERAL NOTES:**

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

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

PROJECT NAME :

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

CLIENT: EMPEE GROUP

DRAWING TITLE: BASEMENT ROOF BEAM LAYOUT

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

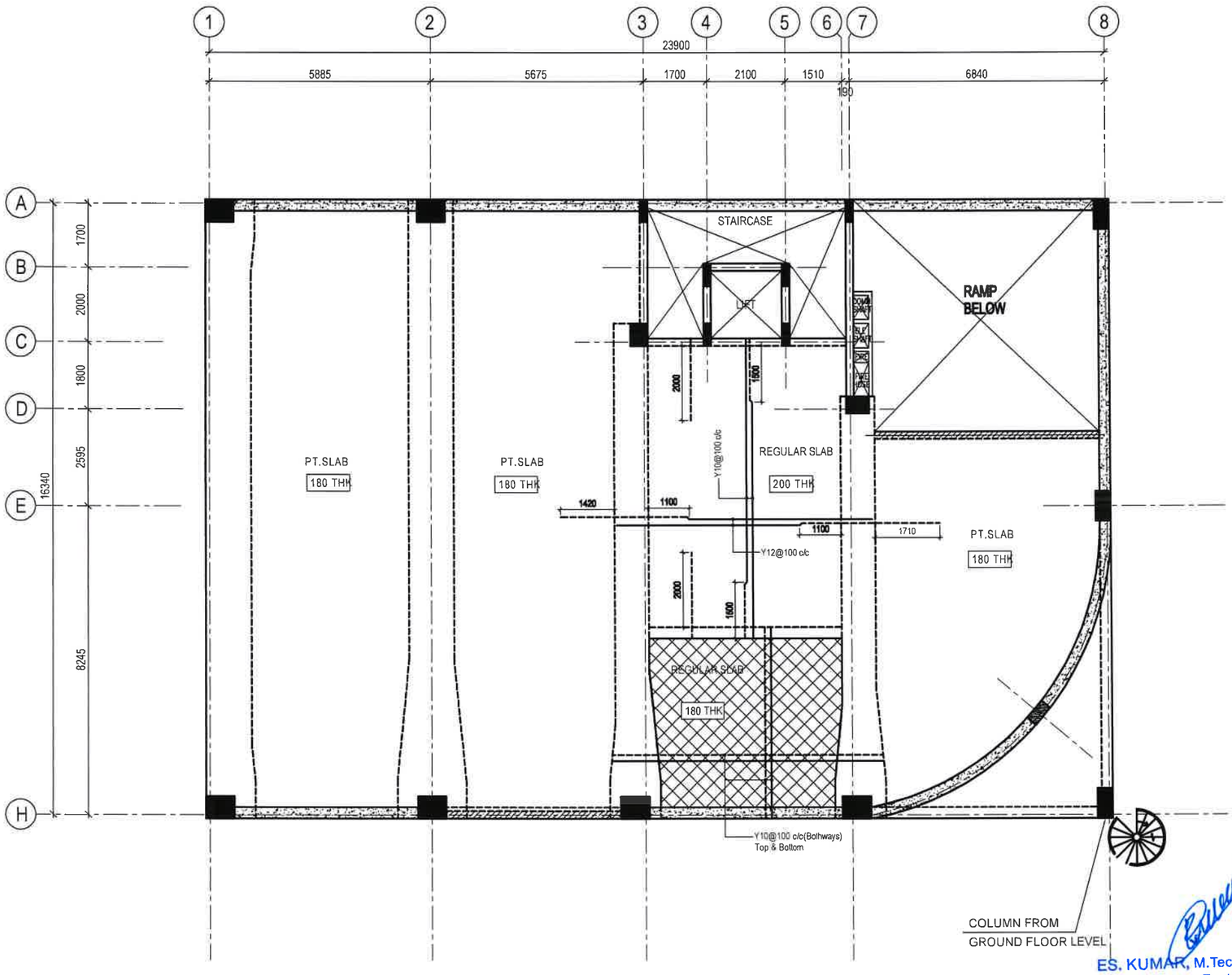
STRUCTURAL CONSULTANTS:


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
#100 / A48, 1 st FLOOR, 3 rd AVENUE
ANNA NAGAR EAST, CHENNAI - 600102.
Tel: 044 - 26203147.


| Drawn M.D | Designed ES | Checked ES.KUMAR |
|---|---|---|




# BASEMENT ROOF SLAB LAYOUT

## LEGEND

= SUNKEN BY 250

**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERING):**

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

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

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

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

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

E. BOTTOM PROPS TO BEAMS
i) SPANNING UP TO 6.00M ———— 14 DAYS.
ii) SPANNING OVER 6.00M ———— 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 |
| 8mm | 400 | 400 | 370 | 320 | 370 | 320 | 280 | 280 |
| 10mm | 585 | 480 | 480 | 400 | 480 | 390 | 370 | 320 |
| 12mm | 690 | 580 | 580 | 480 | 580 | 470 | 440 | 390 |
| 16mm | 900 | 780 | 730 | 640 | 730 | 630 | 580 | 520 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1410 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1300 | 1460 | 1250 | 1170 | 1050 |

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

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

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

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

**GENERAL NOTES:**

1. ALL DIMENSIONS ARE IN mm.
2. ONLY WRITTEN DIMENSIONS TO BE FOLLOWED.
3. ANY DISCREPENCIES NOTICED IN THE DWG. SHALL BE BROUGHT TO OUR KNOWLEDGE IMMEDIATELY.
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7. CLEAR COVER TO THE REINFORCEMENT
   A. COLUMNS ---------- 40mm
   B. ROOF BEAM ---------- 30mm
   C. ROOF SLAB ---------- 25mm
8. DEPTH OF BEAM IS INCLUSIVE OF SLAB THICKNESS

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

PROJECT NAME :

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

CLIENT: EMPEE GROUP

DRAWING TITLE: BASEMENT ROOF SLAB LAYOUT

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


STRUCTURAL CONSULTANTS:

**KIRTE ASSOCIATES, CIVIL & STRUCTURAL ENGINEERING CONSULTANTS,**
PLOT NO:1233, FIRST FLOOR,
FLAT NO:1B,PRIYA AJAY ARCADE,
18th MAIN ROAD, ANNA NAGAR, (WEST)
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ES. KUMAR, M.Tech.- Structures
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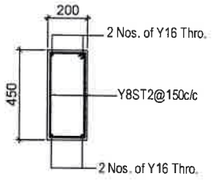
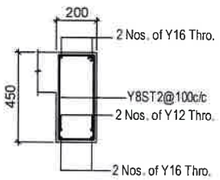
# 97-B/1, Dr. Abdul Kalam Street,
Ramakrishna Nagar, Melamaiyur
Chengalpattu - 603 003
Cell: 98404 48613

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


| Drawn MD | Designed ES | Checked ES.KUMAR | |
|---|---|---|---|




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




C.S. OF BEAM -"B2"
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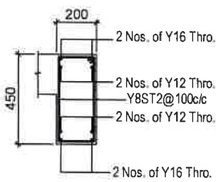


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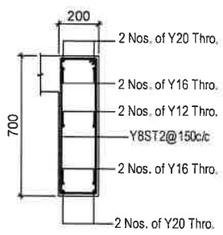
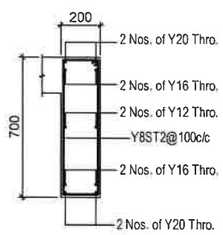



C.S. OF BEAM -"B4"
(200 X 700)
@MID-SPAN




C.S. OF BEAM -"B4"
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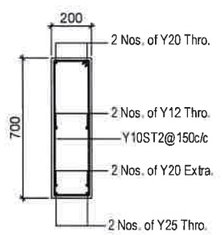


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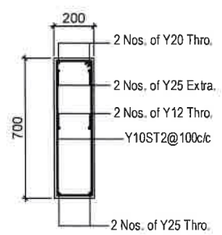


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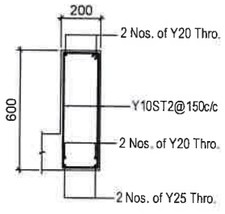


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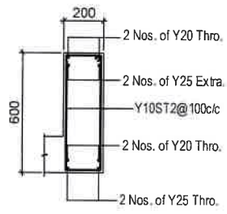


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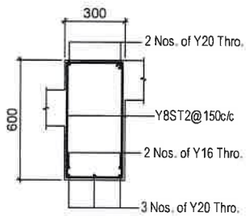


C.S. OF BEAM -"B7"
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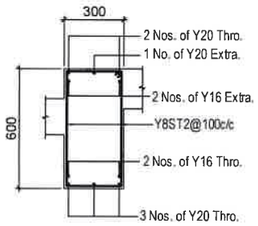


C.S. OF BEAM -"B7"
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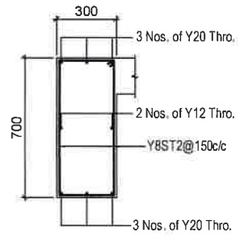


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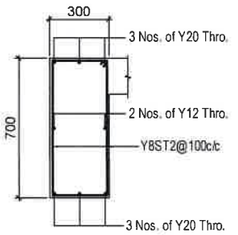


C.S. OF BEAM -"B8"
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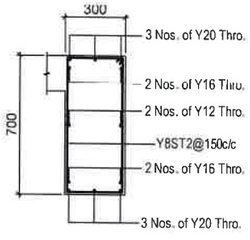


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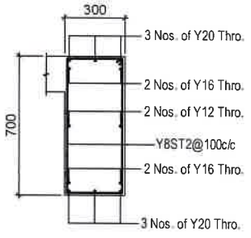


C.S. OF BEAM -"B9"
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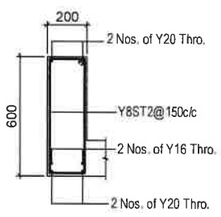
@SUPPORT




C.S. OF BEAM -"B10"
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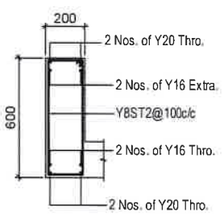
@MID-SPAN




C.S. OF BEAM -"B10"
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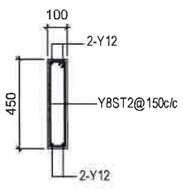
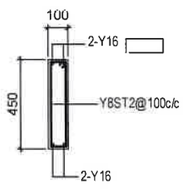
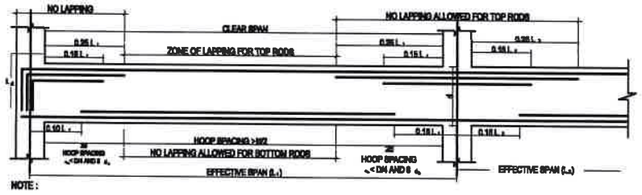


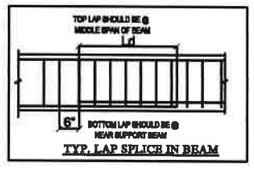

DUCT EDGE BEAM
EB1 (100 X 450)



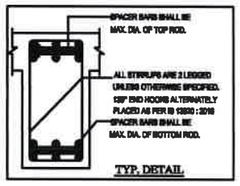

DUCT BRACKET BEAM
BR1 (100 x 450)



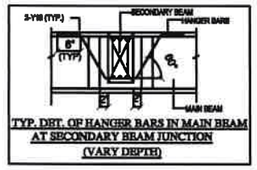

CURTAILMENT DETAILS FOR BEAMS



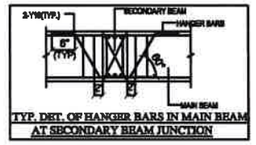

TYP. LAP SPLICE IN BEAM




TYP. DETAIL




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




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

**GENERAL NOTES:**

**REMOVAL OF FORM WORK (SHUTTERING):**

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

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

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

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

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

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

**LAP JOINTS FOR REINFORCEMENT BARS:**

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

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

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

| DIAMETER OF BARS | TENSION BARS | | | | COMPRESSION BARS | | | |
|---|---|---|---|---|---|---|---|---|
| | M20 | M25 | M30 | M35 | M20 | M25 | M30 | M35 |
| 8mm | 450 | 400 | 370 | 350 | 370 | 320 | 280 | 260 |
| 10mm | 565 | 490 | 460 | 400 | 460 | 390 | 370 | 320 |
| 12mm | 680 | 590 | 550 | 480 | 550 | 470 | 440 | 380 |
| 16mm | 900 | 780 | 730 | 640 | 730 | 630 | 580 | 500 |
| 20mm | 1130 | 970 | 910 | 800 | 910 | 780 | 730 | 640 |
| 25mm | 1410 | 1220 | 1140 | 1000 | 1140 | 970 | 910 | 800 |
| 32mm | 1800 | 1560 | 1460 | 1280 | 1460 | 1250 | 1170 | 1030 |

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

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

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

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

**GENERAL NOTES:**

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

| 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 | |

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ARCHITECTS:

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Drawn M.D | Designed ES | Checked ES.KUMAR

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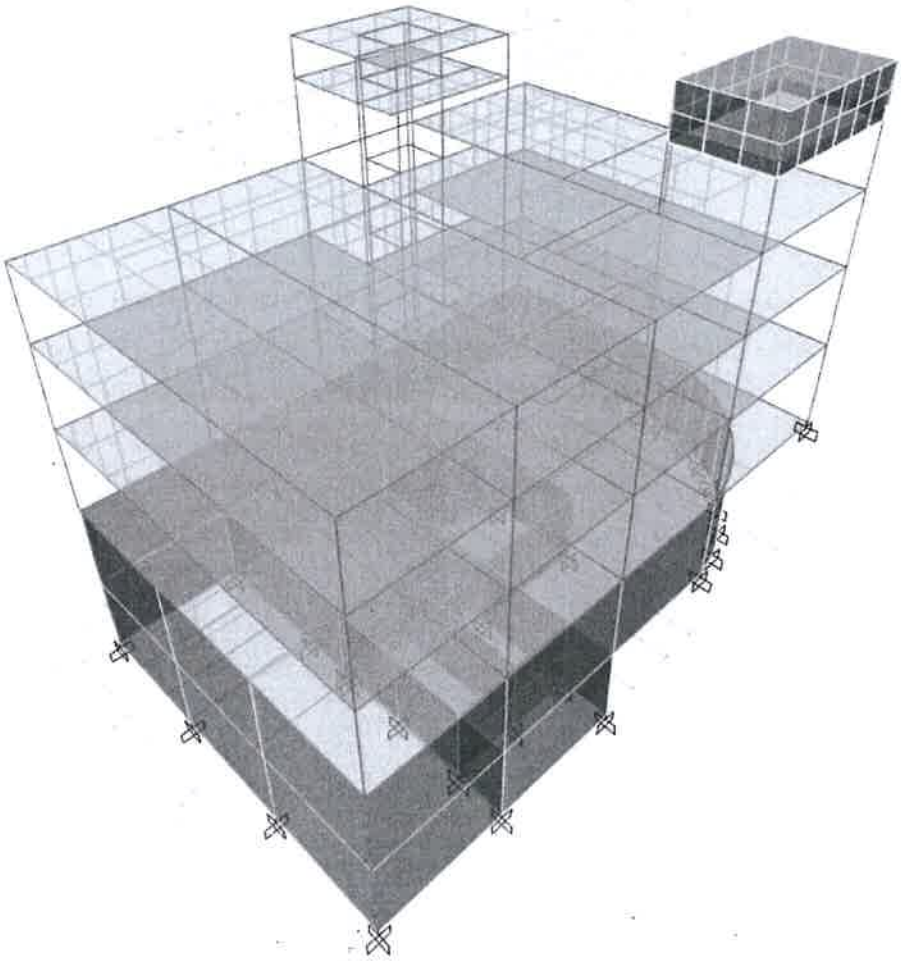
ETABS®



**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

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## 1. INTRODUCTION:

The structure is a "Commercial building" consisting of Basement floor + Ground Floor + 2 floors only. It is designed as a Reinforced concrete framed structure supported on deep foundations as per the Geo technical engineer recommendations provided in the soil investigation report.

### 1.a Building Dimensions

| | | |
|---|---|---|
| Length (X-direction) | - | 23.90 M |
| Width (Y -direction) | - | 16.34 M |

### 1.b Building Height

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

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

**Typical clear height of 3.70m is followed for all upper floors

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## 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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## 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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## 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/$m^2$ 75 % reduction, above 3 kN/$m^2$ 50% reduction)

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

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

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

Where h is the height of building in meters.

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

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

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

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

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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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# DESIGN OF STRUCTURE

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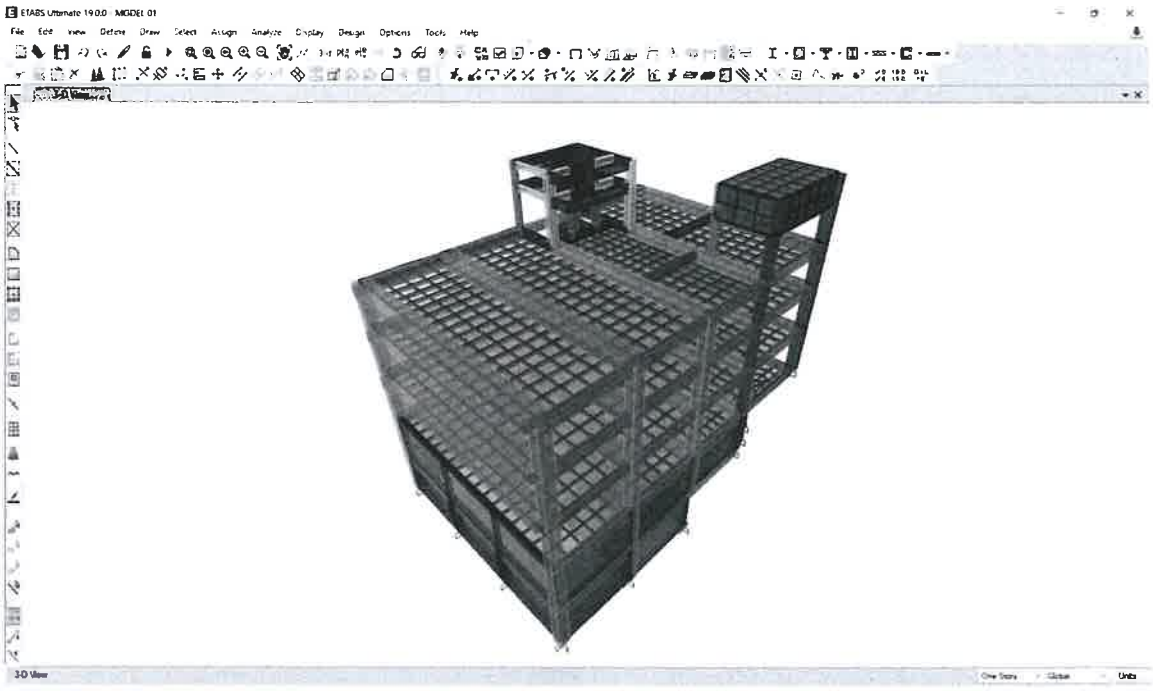

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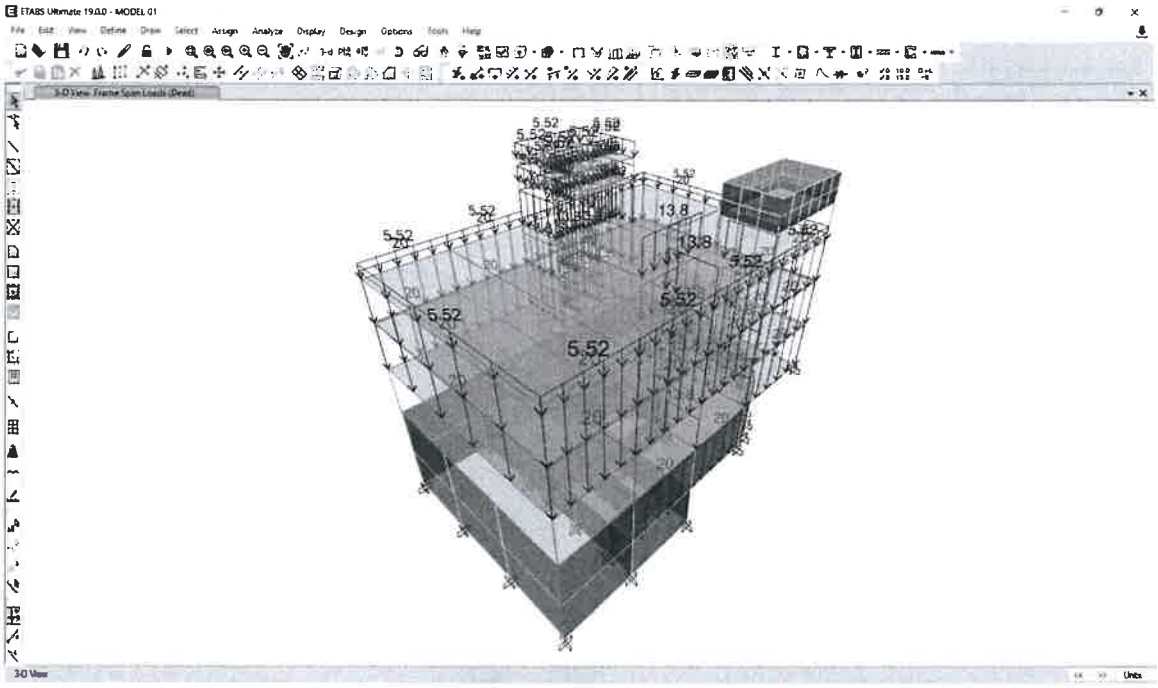


Wall load – dead load

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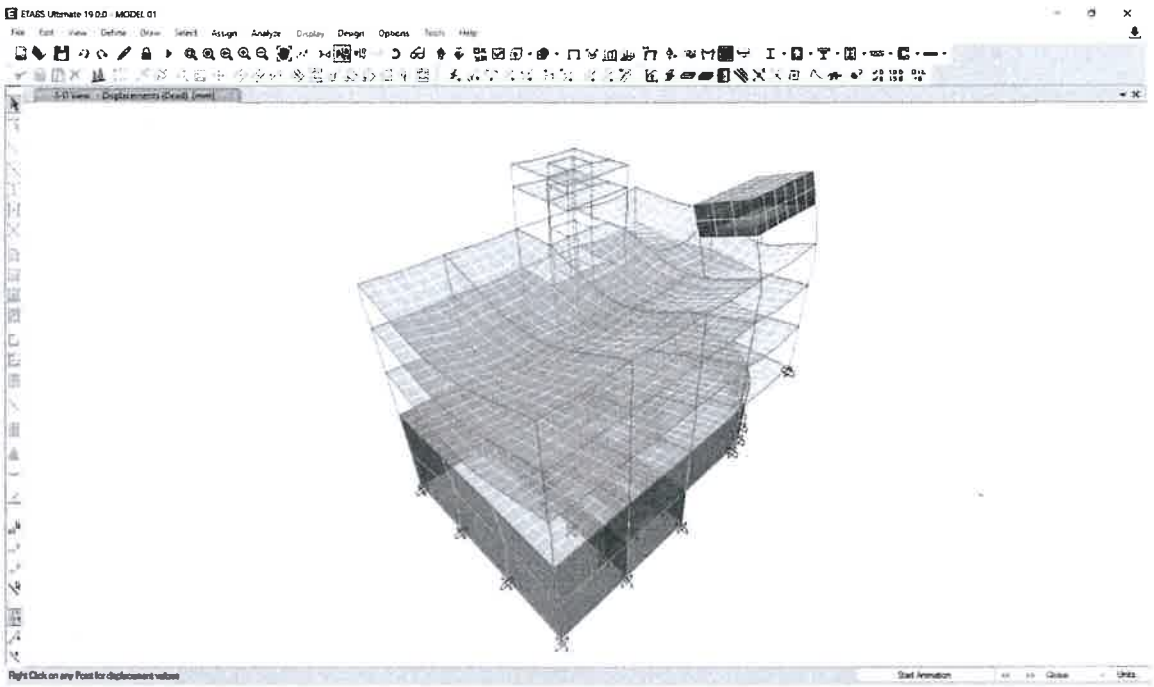
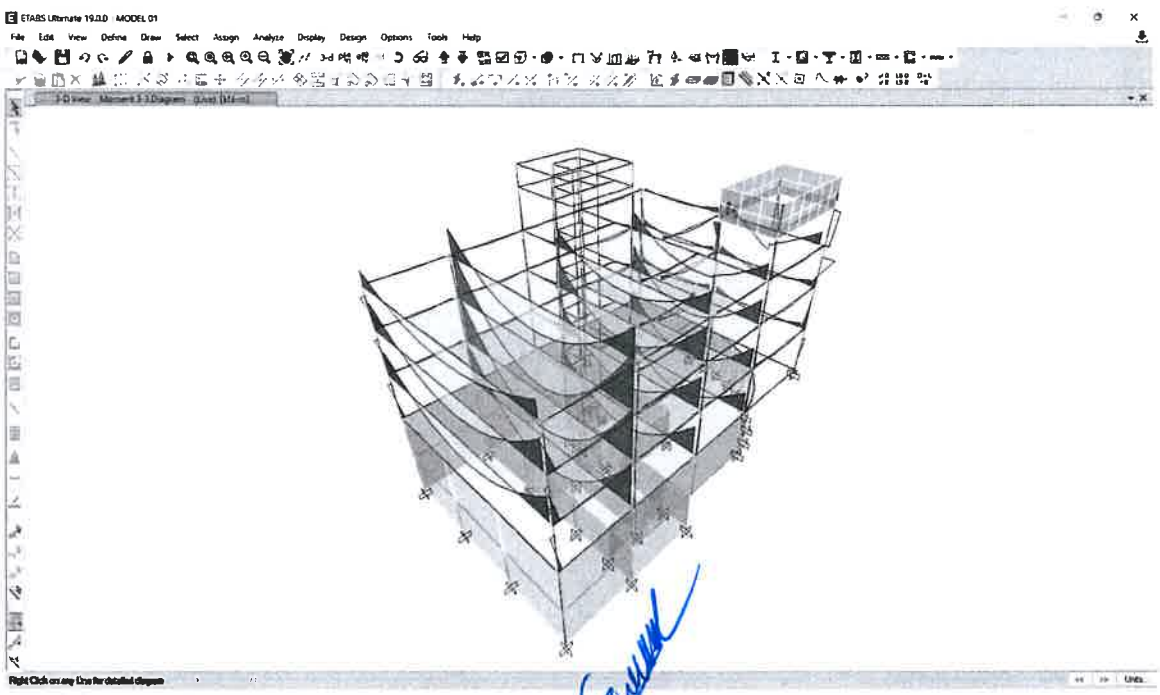


Model displacement



Model moment check

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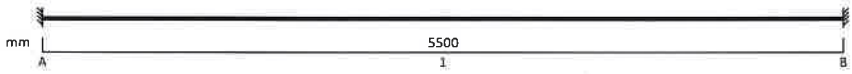
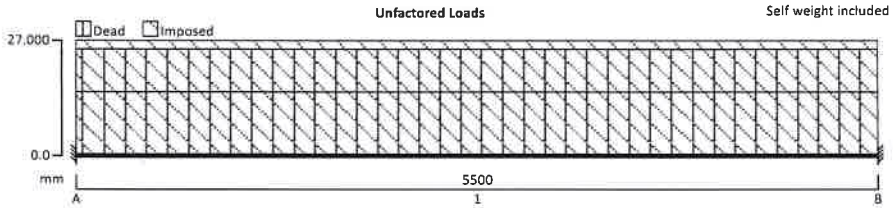
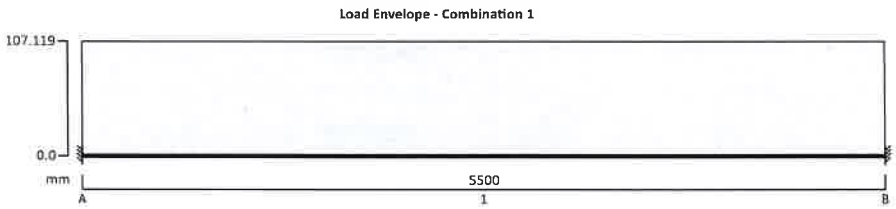
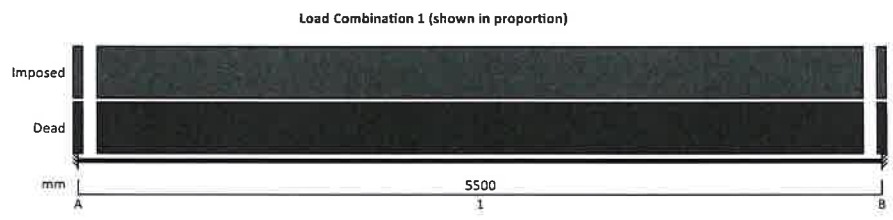
# DESIGN CALCULATION

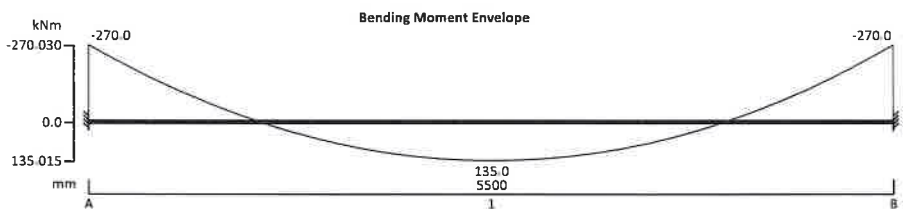
## RC BEAM ANALYSIS & DESIGN

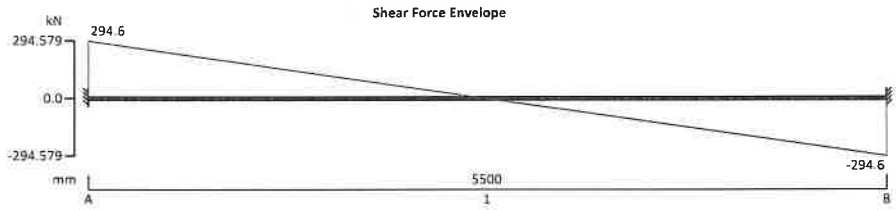





**Support conditions**

| | |
|---|---|
| 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 × 1 |

**Load combinations**

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

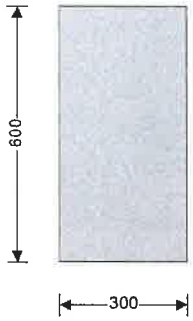
**Analysis results**

| | | |
|---|---|---|
| Maximum moment support A; | $M_{A_max}$ = **-270** kNm; 5% | $M_{A_red}$ = **-257** kNm; |
| Maximum moment span 1 at 2750 mm; | $M_{s1_max}$ = **135** kNm; 10% | $M_{s1_red}$ = **149** kNm; |
| Maximum moment support B; | $M_{B_max}$ = **-270** kNm; 5% | $M_{B_red}$ = **-257** kNm; |
| Maximum shear support A; | $V_{A_max}$ = **295** kN; | $V_{A_red}$ = **295** kN |
| Maximum shear support A span 1 at 542 mm; | $V_{A_s1_max}$ = **236** kN; | $V_{A_s1_red}$ = **236** kN |
| Maximum shear support B; | $V_{B_max}$ = **-295** kN; | $V_{B_red}$ = **-295** kN |
| Maximum shear support B span 1 at 4958 mm; | $V_{B_s1_max}$ = **-236** kN; | $V_{B_s1_red}$ = **-236** kN |
| Maximum reaction at support A; | $R_A$ = **295** kN | |
| Unfactored dead load reaction at support A; | $R_{A_Dead}$ = **122** kN | |
| Unfactored imposed load reaction at support A; | $R_{A_Imposed}$ = **74** kN | |
| Maximum reaction at support B; | $R_B$ = **295** kN | |
| Unfactored dead load reaction at support B; | $R_{B_Dead}$ = **122** kN | |
| Unfactored imposed load reaction at support B; | $R_{B_Imposed}$ = **74** kN | |

**Rectangular section details**

Section width; b = **300** mm

Section depth; h = **600** mm




**Concrete details**

Concrete strength class; **C20/25**

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

Modulus of elasticity of concrete; $E_c$ = 20kN/mm$^2$ + 200 × $f_{cu}$ = **25000** N/mm$^2$

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

**Reinforcement details**

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

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

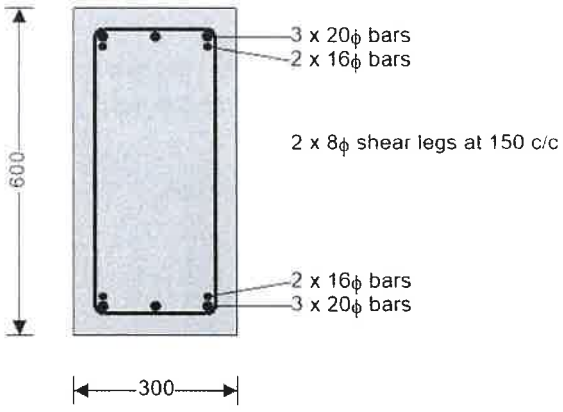
**Nominal cover to reinforcement**

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

**Multiple layers of top reinforcement**

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

**Rectangular section in flexure**

| | |
|---|---|
| Design bending moment; | M = abs($M_{A_red}$) = **257** kNm |
| Depth to tension reinforcement; | d = $d_{top}$ = **542** mm |
| Redistribution ratio; | $\beta_b = \min(1 - m_{rA}, 1)$ = **0.950** |
| | K = M / (b × d$^2$ × $f_{cu}$) = **0.117** |
| | K' = 0.156 |
| | ***K' > K - No compression reinforcement is required*** |
| Lever arm; | z = min(d × (0.5 + (0.25 - K / 0.9)$^{0.5}$), 0.95 × d) = **459** |
| mm | |
| Depth of neutral axis; | x = (d - z) / 0.45 = **184** mm |
| Area of tension reinforcement required; | $A_{s,req}$ = M / (0.87 × $f_y$ × z) = **1285** mm$^2$ |
| Tension reinforcement provided; | 3 × 20 ϕbars + 2 × 16 ϕbars |
| Area of tension reinforcement provided; | $A_{s,prov}$ = **1345** mm$^2$ |
| Minimum area of reinforcement; | $A_{s,min}$ = 0.0013 × b × h = **234** mm$^2$ |
| Maximum area of reinforcement; | $A_{s,max}$ = 0.04 × b × h = **7200** mm$^2$ |

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

**Rectangular section in shear**

| | |
|---|---|
| Design shear force span 1 at 542 mm; | V = max($V_{A_s1_max}$, $V_{A_s1_red}$) = **236** kN |
| Design shear stress; | v = V / (b × d) = **1.450** N/mm$^2$ |

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

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

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

***PASS - Design shear stress is less than maximum allowable***

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

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

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

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

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

***PASS - Area of shear reinforcement provided exceeds minimum required***

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

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

**Spacing of reinforcement**

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

**Minimum distance between bars in tension**

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

***PASS - Satisfies the minimum spacing criteria***

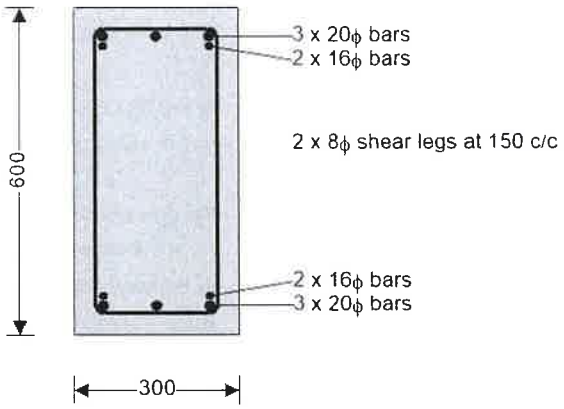
**Maximum distance between bars in tension**

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

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

***PASS - Satisfies the maximum spacing criteria***

**Mid span 1**




**Multiple layers of bottom reinforcement**

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

**Multiple layers of top reinforcement**

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

**Design moment resistance of rectangular section - Positive moment**

| | |
|---|---|
| Design bending moment; | M = abs($M_{s1_red}$) = **149** kNm |
| Depth to tension reinforcement; | d = $d_{bot}$ = **542** mm |
| Redistribution ratio; | $\beta_b = \min(1 - m_{rs1}, 1)$ = **0.900** |
| | $K = M / (b \times d^2 \times f_{cu})$ = **0.068** |
| | K' = 0.156 |
| | ***K' > K - No compression reinforcement is required*** |
| Lever arm; | $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **497** |
| mm | |
| Depth of neutral axis; | x = (d - z) / 0.45 = **98** mm |
| Area of tension reinforcement required; | $A_{s,req} = M / (0.87 \times f_y \times z)$ = **686** mm$^2$ |
| Tension reinforcement provided; | 3 × 20 ϕbars + 2 × 16 ϕbars |
| Area of tension reinforcement provided; | $A_{s,prov}$ = **1345** mm$^2$ |
| Minimum area of reinforcement; | $A_{s,min} = 0.0013 \times b \times h$ = **234** mm$^2$ |
| Maximum area of reinforcement; | $A_{s,max} = 0.04 \times b \times h$ = **7200** mm$^2$ |

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

**Rectangular section in shear**

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

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

***PASS - Area of shear reinforcement provided exceeds minimum required***

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

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

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

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

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

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

***Shear links provided valid between 400 mm and 5100 mm with tension reinforcement of 1345 $mm^2$***

**Spacing of reinforcement**

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

**Minimum distance between bars in tension**

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

***PASS - Satisfies the minimum spacing criteria***

**Maximum distance between bars in tension**

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

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

***PASS - Satisfies the maximum spacing criteria***

**Span to depth ratio**

Basic span to depth ratio; $span_to_depth_{basic}$ = **20.0**

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

Modification for tension reinforcement

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

Modification for compression reinforcement

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

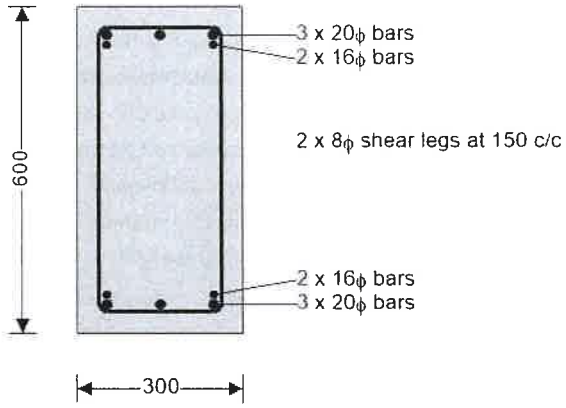
Modification for span length; $f_{long}$ = 1.000

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

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

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

**Support B**




**Multiple layers of bottom reinforcement**

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

**Multiple layers of top reinforcement**

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

**Rectangular section in flexure**

| | |
|---|---|
| Design bending moment; | M = abs($M_{B_red}$) = **257** kNm |
| Depth to tension reinforcement; | d = $d_{top}$ = **542** mm |
| Redistribution ratio; | $\beta_b = \min(1 - m_{rB}, 1)$ = **0.950** |
| | K = M / (b × d$^2$ × $f_{cu}$) = **0.117** |
| | K' = 0.156 |
| | ***K' > K - No compression reinforcement is required*** |
| Lever arm; | z = min(d × (0.5 + (0.25 - K / 0.9)$^{0.5}$), 0.95 × d) = **459** |
| mm | |
| Depth of neutral axis; | x = (d - z) / 0.45 = **184** mm |
| Area of tension reinforcement required; | $A_{s,req}$ = M / (0.87 × $f_y$ × z) = **1285** mm$^2$ |
| Tension reinforcement provided; | 3 × 20 ϕbars + 2 × 16 ϕbars |
| Area of tension reinforcement provided; | $A_{s,prov}$ = **1345** mm$^2$ |
| Minimum area of reinforcement; | $A_{s,min}$ = 0.0013 × b × h = **234** mm$^2$ |
| Maximum area of reinforcement; | $A_{s,max}$ = 0.04 × b × h = **7200** mm$^2$ |

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

**Rectangular section in shear**

| | |
|---|---|
| Design shear force span 1 at 4958 mm; | V = abs(min($V_{B_s1_max}$, $V_{B_s1_red}$)) = **236** kN |
| Design shear stress; | v = V / (b × d) = **1.450** N/mm$^2$ |

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

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

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

***PASS - Design shear stress is less than maximum allowable***

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

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

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

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

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

***PASS - Area of shear reinforcement provided exceeds minimum required***

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

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

**Spacing of reinforcement**

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

**Minimum distance between bars in tension**

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

***PASS - Satisfies the minimum spacing criteria***

**Maximum distance between bars in tension**

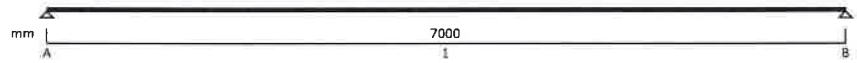
Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **335.4** N/mm$^2$

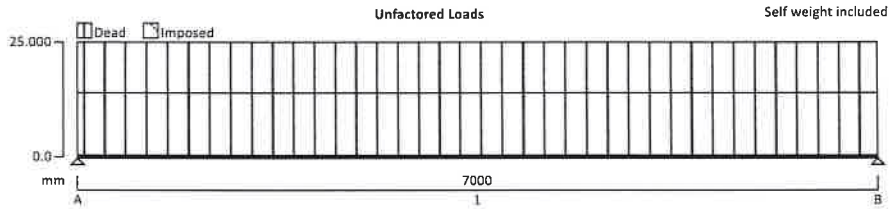
Maximum distance between bars in tension; $s_{max} = \min(47000\ N/mm / f_s, 300\ mm)$ = **140** mm

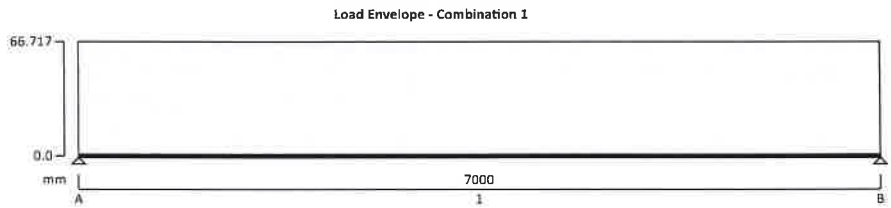
***PASS - Satisfies the maximum spacing criteria***

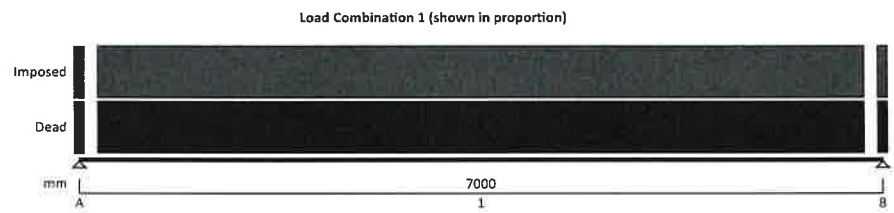
;

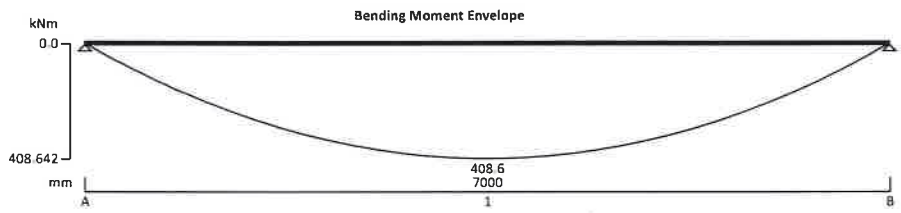
## RC BEAM ANALYSIS & DESIGN



**Support conditions**

| | |
|---|---|
| Support A | Vertically restrained |
| | Rotationally free |
| Support B | Vertically restrained |
| | Rotationally free |

**Applied loading**

Dead full UDL 25 kN/m
Dead full UDL 14 kN/m
Dead self weight of beam × 1

**Load combinations**

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

**Analysis results**

| | | |
|---|---|---|
| Maximum moment support A; | $M_{A_max}$ = **0** kNm; | $M_{A_red}$ = **0** kNm; 15% |
| Maximum moment span 1 at 3500 mm; | $M_{s1_max}$ = **409** kNm; | $M_{s1_red}$ = **409** kNm; |
| Maximum moment support B; | $M_{B_max}$ = **0** kNm; | $M_{B_red}$ = **0** kNm; 15% |
| Maximum shear support A; | $V_{A_max}$ = **234** kN; | $V_{A_red}$ = **234** kN |
| Maximum shear support A span 1 at 548 mm; | $V_{A_s1_max}$ = **197** kN; | $V_{A_s1_red}$ = **197** kN |
| Maximum shear support B; | $V_{B_max}$ = **-234** kN; | $V_{B_red}$ = **-234** kN |
| Maximum shear support B span 1 at 6452 mm; | $V_{B_s1_max}$ = **-197** kN; | $V_{B_s1_red}$ = **-197** kN |
| Maximum reaction at support A; | $R_A$ = **234** kN | |
| Unfactored dead load reaction at support A; | $R_{A_Dead}$ = **156** kN | |
| Maximum reaction at support B; | $R_B$ = **234** kN | |
| Unfactored dead load reaction at support B; | $R_{B_Dead}$ = **156** kN | |

**Rectangular section details**

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

**Concrete details**

| | |
|---|---|
| Concrete strength class; | **C28/35** |
| Characteristic compressive cube strength; | $f_{cu}$ = **35** N/mm$^2$ |
| Modulus of elasticity of concrete; | $E_c$ = 20kN/mm$^2$ + 200 × $f_{cu}$ = **27000** N/mm$^2$ |
| Maximum aggregate size; | $h_{agg}$ = **20** mm |

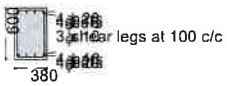
**Reinforcement details**

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

**Nominal cover to reinforcement**

| | |
|---|---|
| Nominal cover to top reinforcement; | $c_{nom_t}$ = **25** mm |
| Nominal cover to bottom reinforcement; | $c_{nom_b}$ = **25** mm |
| Nominal cover to side reinforcement; | $c_{nom_s}$ = **25** mm |

**Support A**




**Multiple layers of bottom reinforcement**

| | |
|---|---|
| Reinforcement provided - layer 1; | 4 × 20ϕ bars |
| Area of reinforcement provided - layer 1; | $A_{s_L1}$ = **1257** mm$^2$ |
| Depth to layer 1; | $d_{L1}$ = **555** mm |
| Reinforcement provided - layer 2; | 4 × 16ϕ bars |
| Area of reinforcement provided - layer 2; | $A_{s_L2}$ = **804** mm$^2$ |
| Depth to layer 2; | $d_{L2}$ = **537** mm |
| Total area of reinforcement; | $A_{s,prov}$ = $A_{s_L1}$ + $A_{s_L2}$ = **2061** mm$^2$ |
| Centroid of reinforcement; | $d_{bot}$ = ($A_{s_L1}$ × $d_{L1}$ + $A_{s_L2}$ × $d_{L2}$) / $A_{s,prov}$ = **548** mm |

**Multiple layers of top reinforcement**

| | |
|---|---|
| Reinforcement provided - layer 1; | 4 × 20ϕ bars |
| Area of reinforcement provided - layer 1; | $A_{s_L1}$ = **1257** mm$^2$ |
| Depth to layer 1; | $d_{L1}$ = **45** mm |
| Reinforcement provided - layer 2; | 4 × 16ϕ bars |

| | |
|---|---|
| Area of reinforcement provided - layer 2; | $A_{s_L2}$ = **804** mm$^2$ |
| Depth to layer 2; | $d_{L2}$ = **63** mm |
| Total area of reinforcement; | $A_{s2,prov} = A_{s_L1} + A_{s_L2}$ = **2061** mm$^2$ |
| Centroid of reinforcement; | $d_{top} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s2,prov}$ = **52** mm |

**Rectangular section in shear**

| | |
|---|---|
| Design shear force span 1 at 548 mm; | $V = \max(V_{A_s1_max}, V_{A_s1_red})$ = **197** kN |
| Design shear stress; | $v = V / (b \times d)$ = **0.945** N/mm$^2$ |
| Design concrete shear stress; | $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1,$ |
| $(400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$ | |
| | $v_c$ = **0.705** N/mm$^2$ |
| Allowable design shear stress; | $v_{max} = \min(0.8$ N/mm$^2 \times (f_{cu}/1$ N/mm$^2)^{0.5}$, 5 N/mm$^2$) = |
| **4.733** N/mm$^2$ | |

***PASS - Design shear stress is less than maximum allowable***

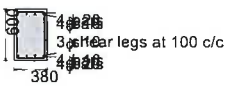
| | |
|---|---|
| Value of v from Table 3.7; | $0.5 \times v_c < v < (v_c + 0.4$ N/mm$^2$) |
| Design shear resistance required; | $v_s = \max(v - v_c, 0.4$ N/mm$^2$) = **0.400** N/mm$^2$ |
| Area of shear reinforcement required; | $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **349** mm$^2$/m |
| Shear reinforcement provided; | 3 × 10ϕ legs at 100 c/c |
| Area of shear reinforcement provided; | $A_{sv,prov}$ = **2356** mm$^2$/m |

***PASS - Area of shear reinforcement provided exceeds minimum required***

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

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

**Mid span 1**




**Multiple layers of bottom reinforcement**

| | |
|---|---|
| Reinforcement provided - layer 1; | 4 × 20ϕ bars |
| Area of reinforcement provided - layer 1; | $A_{s_L1}$ = **1257** mm$^2$ |
| Depth to layer 1; | $d_{L1}$ = **555** mm |
| Reinforcement provided - layer 2; | 4 × 16ϕ bars |
| Area of reinforcement provided - layer 2; | $A_{s_L2}$ = **804** mm$^2$ |
| Depth to layer 2; | $d_{L2}$ = **537** mm |
| Total area of reinforcement; | $A_{s,prov} = A_{s_L1} + A_{s_L2}$ = **2061** mm$^2$ |
| Centroid of reinforcement; | $d_{bot} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s,prov}$ = **548** mm |

**Multiple layers of top reinforcement**

| | |
|---|---|
| Reinforcement provided - layer 1; | 4 × 20ϕ bars |
| Area of reinforcement provided - layer 1; | $A_{s_L1}$ = **1257** mm$^2$ |
| Depth to layer 1; | $d_{L1}$ = **45** mm |
| Reinforcement provided - layer 2; | 4 × 16ϕ bars |
| Area of reinforcement provided - layer 2; | $A_{s_L2}$ = **804** mm$^2$ |
| Depth to layer 2; | $d_{L2}$ = **63** mm |

Total area of reinforcement; $A_{s2,prov} = A_{s_L1} + A_{s_L2}$ = **2061** mm$^2$

Centroid of reinforcement; $d_{top} = (A_{s_L1} \times d_{L1} + A_{s_L2} \times d_{L2}) / A_{s2,prov}$ = **52** mm

**Design moment resistance of rectangular section - Positive moment**

Design bending moment; $M = abs(M_{s1_red})$ = **409** kNm

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

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

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

K' = 0.156

***K' > K - No compression reinforcement is required***

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

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

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

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

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

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

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

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

**Rectangular section in shear**

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

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

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

***PASS - Area of shear reinforcement provided exceeds minimum required***

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

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

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

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

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

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

***Shear links provided valid between 0 mm and 7000 mm with tension reinforcement of 2061 mm$^2$***

**Spacing of reinforcement**

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

**Minimum distance between bars in tension**

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

***PASS - Satisfies the minimum spacing criteria***

**Maximum distance between bars in tension**

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

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

***PASS - Satisfies the maximum spacing criteria***

**Span to depth ratio**

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

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

Modification for tension reinforcement

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

Modification for compression reinforcement

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

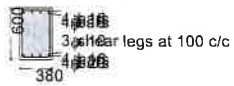
Modification for span length; $f_{long} = 1.000$

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 × 20ϕ bars

Area of reinforcement provided - layer 1; $A_{s_L1} =$ **1257** mm²

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

Reinforcement provided - layer 2; 4 × 16ϕ bars

Area of reinforcement provided - layer 2; $A_{s_L2} =$ **804** mm²

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

Total area of reinforcement; $A_{s,prov} = A_{s_L1} + A_{s_L2} =$ **2061** mm²

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

**Multiple layers of top reinforcement**

Reinforcement provided - layer 1; 4 × 16ϕ bars

Area of reinforcement provided - layer 1; $A_{s_L1} =$ **804** mm²

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

Reinforcement provided - layer 2; 4 × 16ϕ bars

Area of reinforcement provided - layer 2; $A_{s_L2} =$ **804** mm²

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

Total area of reinforcement; $A_{s2,prov} = A_{s_L1} + A_{s_L2} =$ **1608** mm²

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

**Rectangular section in shear**

Design shear force span 1 at 6452 mm; $V = abs(\min(V_{B_s1_max}, V_{B_s1_red})) =$ **197** kN

Design shear stress; $v = V / (b \times d) =$ **0.945** N/mm²

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

$v_c =$ **0.705** N/mm²

Allowable design shear stress; $v_{max} = \min(0.8\ N/mm^2 \times (f_{cu}/1\ N/mm^2)^{0.5}, 5\ N/mm^2) =$ **4.733** N/mm²

***PASS - Design shear stress is less than maximum allowable***

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

Design shear resistance required; $v_s = \max(v - v_c, 0.4\ N/mm^2) =$ **0.400** N/mm²

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) =$ **349** mm²/m

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

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

***PASS - Area of shear reinforcement provided exceeds minimum required***

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

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

;

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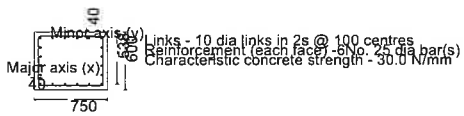
## RC COLUMN DESIGN

**Column definition**

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




**Unbraced Column Design**

**Check on overall column dimensions**

***Column OK - h < 4b***

**Unbraced column slenderness check**

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

***Column slenderness limit OK***

***If column is unrestrained, slenderness limit OK***

**Short column check for unbraced columns**

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

***The unbraced column is short (major axis)***

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

| | |
|---|---|
| Effective height (minor axis); | $l_{ey} = \beta_y \times l_o$ = **4.200** m |
| Slenderness check; | $l_{ey}/b$ = **5.60** |

***The unbraced column is short (minor axis)***

**Short column - bi-axial bending**

**Define column reinforcement**

Main reinforcement in column

| | |
|---|---|
| Assumed diameter of main reinforcement; | $D_{col}$ = **25** mm |
| Assumed no. of bars in one face (assumed sym); | $L_{ncol}$ = **6** |
| Area of "tension" steel; | $A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4$ = **2945** $mm^2$ |
| Area of compression steel; | $A_{sc} = A_{st}$ = **2945** $mm^2$ |
| Total area of steel ; | $A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2))$ = **9817.5** $mm^2$ |
| Percentage of steel; | $A_{scol} / (b \times h)$ = **2.2** % |

**Design ultimate loading**

| | |
|---|---|
| Design ultimate axial load; | N = **2880** kN |
| Design ultimate moment (major axis); | $M_x$ = **600** kNm |
| Design ultimate moment (minor axis); | $M_y$ = **600** kNm |

**Minimum design moments**

| | |
|---|---|
| Min design moment (major axis); | $M_{xmin} = \min(0.05 \times h, 20\ mm) \times N$ = **57.6** kNm |
| Min design moment (minor axis); | $M_{ymin} = \min(0.05 \times b, 20\ mm) \times N$ = **57.6** kNm |

**Design moments**

| | |
|---|---|
| Design moment (major axis); | $M_{xdes} = \max(\text{abs}(M_x), M_{xmin})$ = **600.0** kNm |
| Design moment (minor axis); | $M_{ydes} = \max(\text{abs}(M_y), M_{ymin})$ = **600.0** kNm |

**Simplified method for dealing with bi-axial bending:-**

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

Approx uniaxial design moment

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

Design moment;

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

**Set up section dimensions for design:-**

| | |
|---|---|
| Section depth; | D = if($M_{xdes}/h' < M_{ydes}/b'$, b, h) = **600.0** mm |
| Depth to "tension" steel; | d = if($M_{xdes}/h' < M_{ydes}/b'$, b', h') = **537.5** mm |
| Section width; | B = if($M_{xdes}/h' < M_{ydes}/b'$, h, b) = **750.0** mm |

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

**Check of design forces - symmetrically reinforced section**

**NOTE**

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

| | |
|---|---|
| Section width (parallel to axis of bending); | B = **750** mm |
| Section depth perpendicular to axis of bending); | D = **600** mm |
| Depth to "tension" steel (symmetrical); | d = **538** mm |

***Tension and compression steel yield***

**Determine correct moment of resistance**

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

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

| | |
|---|---|
| Applied axial load; | N = **2880.0** kN |
| Check for moment; | $M_{design}$ = **952.5** kNm |

***Moment check satisfiedMoment capacity with no axial load***

**Check min and max areas of steel**

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

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

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

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

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

***Area of compression steel provided OK***

**Major axis Shear Resistance of Concrete Columns**

Column width; $b =$ **750** mm

Column depth; $h =$ **600** mm

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

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

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

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

**Is a check required? (3.8.4.6)**

Axial load; $N =$ **2880.0** kN

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

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

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

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

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

***No shear check required***

**Define Containment Links Provided**

Link spacing; $s_v =$ **100** mm; Link diameter; $L_{dia} =$ **10** mm; No of links in each group; $L_n =$ **2**

**Minimum Containment Stee**

**Shear steel**

Link spacing; $s_v =$ **100** mm

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

**Column steel**

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

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

***Link diameter OK***

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

***Link spacing OK***

**Crack Control in columns**

Column design ultimate axial load; $N =$ **2880.0** kN

**Column dimensions**

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

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

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

Limit for crack check; $N_{limit} = 0.2 \times f_{cu} \times A_c =$ **2700.0** kN

***No crack checks required***

**Serviceability Limit State - Cracking in Columns**

**Bent about the major axis**

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

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

**Column dimensions, depth to steel (assumed symmetrical)**

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

**Modular Ratio**

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

**Neutral Axis position**

For equilibrium; $F_{st}$ equates $F_c$

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

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

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

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

**Concrete and Steel stresses**

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

Taking moments about the centreline of the reinforcement:-

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

| | |
|---|---|
| Solving for concrete stress $f_c$ gives; | $f_c = 2 \times M_{X_SLS} / ( b \times x \times (h' - x/3))$ = **1.41** N/mm$^2$ |
| Allowable stress; | $0.45 \times f_{cu}$ = **13.50** N/mm$^2$ |

***Concrete stress OK***

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

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

Solving for steel stress $f_{st}$ gives; $f_{st} = M_{X_SLS} / ( A_{st} \times (h' - x/3))$ = **36.09** N/mm$^2$

**Concrete and Steel strains**

| | |
|---|---|
| Strain in the reinforcement; | $\varepsilon_s = f_{st} / E_s$ = **180.5×10$^{-6}$** |
| Allowable steel strain; | $0.8 \times f_y / E_s$ = **2.000×10$^{-3}$** |

***Steel strain OK***

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

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

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

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

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

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

Breadth of tension face; $b_t = b =$ **750** mm

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

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

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

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

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

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

**Serviceability Limit State - Cracking in Columns**

**Bent about the minor axis**

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

**Column dimensions, depth to steel (assumed symmetrical)**

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

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

Depth to steel; $b' =$ **688** mm

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

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

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

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

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

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

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

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

Effective depth to tension reinforcement; $b' =$ **687.5** mm

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

**Modular Ratio**

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

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

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

**Neutral Axis position**

For equilibrium; $F_{st}$ equates $F_c$

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

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

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

Depth of concrete in compression; $x =$ **255.5** mm

**Concrete and Steel stresses**

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

Taking moments about the centreline of the reinforcement:-

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

Solving for concrete stress $f_c$ gives; $f_c = 2 \times M_{Y_SLS} / ( h \times x \times (b' - x/3)) =$ **1.08** N/mm$^2$

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

***Concrete stress OK***

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

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

Solving for steel stress $f_{st}$ gives; $f_{st} = M_{Y_SLS} / ( A_{st} \times (b' - x/3)) =$ **28.18** N/mm$^2$

**Concrete and Steel strains**

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s =$ **$140.9 \times 10^{-6}$**

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; $a' = b =$ **750** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(b' - x) =$ **$161.3 \times 10^{-6}$**

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

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

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

Breadth of tension face; $h_t = h =$ **600** mm

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

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

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

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

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

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

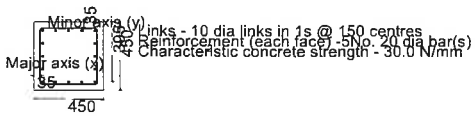
Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2 \times (a_{cr} - c_{ten})/(b - x)) =$ **0.000** mm

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

## RC COLUMN DESIGN

**Column definition**

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




**Braced Column Design**

**Check on overall column dimensions**

***Column OK - h < 4b***

**Braced column slenderness check**

| | |
|---|---|
| Column clear height; | $l_o$ = **3500** mm |
| Slenderness limit; | $l_{limit}$ = 60 × b = **27000** mm |

***Column slenderness limit OK***

**Short column check for braced columns**

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

***The braced column is short (major axis)***

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

***The braced column is short (minor axis)***

**Short column - bi-axial bending**

**Define column reinforcement**

Main reinforcement in column

| | |
|---|---|
| Assumed diameter of main reinforcement; | $D_{col}$ = **20** mm |
| Assumed no. of bars in one face (assumed sym); | $L_{ncol}$ = **5** |
| Area of "tension" steel; | $A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4$ = **1571** $mm^2$ |
| Area of compression steel; | $A_{sc} = A_{st}$ = **1571** $mm^2$ |
| Total area of steel ; | $A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2))$ = **5026.5** $mm^2$ |
| Percentage of steel; | $A_{scol} / (b \times h)$ = **2.5** % |

**Design ultimate loading**

| | |
|---|---|
| Design ultimate axial load; | N = **850** kN |
| Design ultimate moment (major axis); | $M_x$ = **11** kNm |
| Design ultimate moment (minor axis); | $M_y$ = **34** kNm |

**Minimum design moments**

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

**Design moments**

| | |
|---|---|
| Design moment (major axis); | $M_{xdes} = \max(abs(M_x), M_{xmin})$ = **17.0** kNm |
| Design moment (minor axis); | $M_{ydes} = \max(abs(M_y), M_{ymin})$ = **34.0** kNm |

**Simplified method for dealing with bi-axial bending:-**

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

Approx uniaxial design moment (Cl 3.8.4.5)

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

Design moment;

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

**Set up section dimensions for design:-**

| | |
|---|---|
| Section depth; | $D = if(M_{xdes}/h' < M_{ydes}/b' , b, h)$ = **450.0** mm |
| Depth to "tension" steel; | $d = if(M_{xdes}/h' < M_{ydes}/b' , b', h')$ = **395.0** mm |
| Section width; | $B = if(M_{xdes}/h' < M_{ydes}/b' , h, b)$ = **450.0** mm |

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

**Check of design forces - symmetrically reinforced section**

**NOTE**

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

| | |
|---|---|
| Section width (parallel to axis of bending); | B = **450** mm |
| Section depth perpendicular to axis of bending); | D = **450** mm |
| Depth to "tension" steel (symmetrical); | d = **395** mm |

***Tension and compression steel yield***

**Determine correct moment of resistance**

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

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

| | |
|---|---|
| Applied axial load; | N = **850.0** kN |
| Check for moment; | $M_{design}$ = **48.2** kNm |

***Moment check satisfied***

**Check min and max areas of steel**

| | |
|---|---|
| Total area of concrete; | $A_{conc} = b \times h =$ **202500** $mm^2$ |
| Area of steel (symmetrical); | $A_{scol} =$ **5027** $mm^2$ |
| Minimum percentage of compression reinforcement; | $k_c =$ **0.80** % |
| Minimum steel area; | $A_{sc_min} = k_c \times A_{conc} =$ **1620** $mm^2$ |
| Maximum steel area; | $A_{s_max} = 6\ \% \times A_{conc} =$ **12150** $mm^2$ |

***Area of compression steel provided OK***

**Major axis Shear Resistance of Concrete Columns**

| | |
|---|---|
| Column width; | b = **450** mm |
| Column depth; | h = **450** mm |
| Effective depth to steel; | h' = **395** mm |
| Area of concrete; | $A_{conc} = b \times h =$ **202500** $mm^2$ |
| Design ultimate shear force (major axis); | $V_x =$ **55** kN |
| Characteristic strength of concrete; | $f_{cu} =$ **30** $N/mm^2$ |
| Axial load; | N = **850.0** kN |
| Major axis moment; | $M_x =$ **11.0** kNm |
| Eccentricity; | $e = M_x / N =$ **12.9** mm |
| Limit to eccentricity; | $e_{limit} = 0.6 \times h =$ **270.0** mm |
| Actual shear stress; | $v_x = V_x / (b \times h') =$ **0.3** $N/mm^2$ |
| Allowable stress; | $v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) =$ **4.382** $N/mm^2$ |

***No shear check required***

**Define Containment Links Provided**

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

**Minimum Containment Steel**

**Shear steel**

| | |
|---|---|
| Link spacing; | $s_v =$ **150** mm |
| Link diameter; | $L_{dia} =$ **10** mm |

**Column steel**

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

***Link diameter OK***

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

***Link spacing OK***

**Crack Control in columns**

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

**Column dimensions**

| | |
|---|---|
| Column depth (larger column dimension); | h = **450** mm |
| Column width (smaller column dimension); | b = **450** mm |
| Column area; | $A_c = h \times b =$ **202500** $mm^2$ |
| Limit for crack check; | $N_{limit} = 0.2 \times f_{cu} \times A_c =$ **1215.0** kN |

***Column should be checked as a beam for cracking***

**Serviceability Limit State - Cracking in Columns**

**Bent about the major axis**

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

Design serviceability moment about the major axis; $M_{x_SLS}$ = **23** kNm

**Column dimensions, depth to steel (assumed symmetrical)**

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

**Modular Ratio**

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

**Neutral Axis position**

For equilibrium; $F_{st}$ equates $F_c$

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

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

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

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

**Concrete and Steel stresses**

The serviceability limit state moment; $M_{x_SLS}$ = **23** kNm

Taking moments about the centreline of the reinforcement:-

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

Solving for concrete stress $f_c$ gives; $f_c = 2 \times M_{x_SLS} / (\ b \times x \times (h' - x/3))$ = **1.88** N/mm$^2$

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

***Concrete stress OK***

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

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

Solving for steel stress $f_{st}$ gives; $f_{st} = M_{x_SLS} / (\ A_{st} \times (h' - x/3))$ = **42.82** N/mm$^2$

**Concrete and Steel strains**

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s$ = **214.1**$\times$**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** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x) =$ **$264.0\times10^{-6}$**

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

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

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

Breadth of tension face; $b_t = b =$ **450** mm

$\varepsilon_m = \min( \varepsilon_1 ,\max(0, \varepsilon_1 - [ \text{factor} \times b_t \times (h - x) \times (a' - x) / (3\times E_s \times A_{st} \times (h' - x))])) =$ **$92.78\times10^{-6}$**

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

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

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

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

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

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

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

**Serviceability Limit State - Cracking in Columns**

**Bent about the minor axis**

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

Design serviceability moment about the minor axis; $M_{Y_SLS} =$ **21** kNm

**Column dimensions, depth to steel (assumed symmetrical)**

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

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

Depth to steel; b' = **395** mm

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

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

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

Diameter of tension reinforcement; $D_{col} =$ **20** mm

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

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

Nominal cover to reinforcement; $c_{nom} = b - b' - D_{col}/2 - L_{dia} =$ **35** mm

Cover to tension reinforcement; $c_{ten} = c_{nom} + L_{dia} =$ **45.0** mm

Effective depth to tension reinforcement; b' = **395.0** mm

Tension bar centres; $bar_{crs} = (h - 2\times(c_{nom} + L_{dia}) - D_{col} ) / (L_{ncol} - 1) =$ **85.0** mm

**Modular Ratio**

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

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

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

**Neutral Axis position**

For equilibrium; $F_{st}$ equates $F_c$

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

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

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

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

**Concrete and Steel stresses**

The serviceability limit state moment; $M_{Y_SLS}$ = **21** kNm

Taking moments about the centreline of the reinforcement:-

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

Solving for concrete stress $f_c$ gives; $f_c = 2 \times M_{Y_SLS} / ( h \times x \times (b' - x/3)) =$ **1.71** N/mm$^2$

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

***Concrete stress OK***

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

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

Solving for steel stress $f_{st}$ gives; $f_{st} = M_{Y_SLS} / ( A_{st} \times (b' - x/3)) =$ **39.10** N/mm$^2$

**Concrete and Steel strains**

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s =$ **195.5×10$^{-6}$**

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

***Steel strain OK***

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

Level of crack; $a' = b =$ **450** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(b' - x) =$ **241.1×10$^{-6}$**

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

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

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

Breadth of tension face; $h_t = h =$ **450** mm

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

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

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

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

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

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

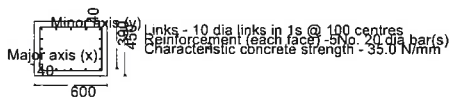
Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2 \times (a_{cr} - c_{ten})/(b - x)) =$ **0.012** mm

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

## RC COLUMN DESIGN

**Column definition**

| | |
|---|---|
| Column depth (larger column dim); | $h$ = **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; | $l_{limit1} = 100 \times b^2 / h$ = **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; | $\beta_x$ = **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; | $\beta_y$ = **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** mm

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

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

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

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

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

**Design ultimate loading**

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

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

Design ultimate moment (minor axis); $M_y$ = **220** kNm

**Minimum design moments**

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

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

**Design moments**

Design moment (major axis); $M_{xdes} = \max(abs(M_x), M_{xmin})$ = **275.0** kNm

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

**Simplified method for dealing with bi-axial bending:-**

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

Approx uniaxial design moment

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

Design moment;

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

**Set up section dimensions for design:-**

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

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

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

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

**Check of design forces - symmetrically reinforced section**

**NOTE**

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

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

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

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

***Tension and compression steel yield***

**Determine correct moment of resistance**

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

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

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

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

***Moment check satisfied***

**Check min and max areas of steel**

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

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

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

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

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

***Area of compression steel provided OK***

**Major axis Shear Resistance of Concrete Columns**

Column width; b = **600** mm

Column depth; h = **450** mm

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

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

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

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

**Is a check required? (3.8.4.6)**

Axial load; N = **2000.0** kN

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

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

Limit to eccentricity; $e_{limit} = 0.6 \times h$ = **270.0** mm

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

Allowable stress; $v_{allowable}$ = min ((0.8 N$^{1/2}$/mm) × $\sqrt{(f_{cu})}$, 5 N/mm$^2$ ) = **4.733** N/mm$^2$

***No shear check required***

**Define Containment Links Provided**

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

**Minimum Containment Steel**

**Shear steel**

Link spacing; $s_v$ = **100** mm

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

**Column steel**

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

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

***Link diameter OK***

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

***Link spacing OK***

**Crack Control in columns**

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

**Column dimensions**

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

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

Column area; $A_c = h \times b$ = **270000** mm$^2$

Limit for crack check; $N_{limit} = 0.2 \times f_{cu} \times A_c$ = **1890.0** kN

***No crack checks required***

**Serviceability Limit State - Cracking in Columns**

**Bent about the major axis**

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

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

**Column dimensions, depth to steel (assumed symmetrical)**

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

**Modular Ratio**

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

**Neutral Axis position**

For equilibrium; $F_{st}$ equates $F_c$

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

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

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

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

**Concrete and Steel stresses**

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

Taking moments about the centreline of the reinforcement:-

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

| | |
|---|---|
| Solving for concrete stress $f_c$ gives; | $f_c = 2 \times M_{x_SLS} / (\ b \times x \times (h' - x/3))$ = **3.48** N/mm$^2$ |
| Allowable stress; | $0.45 \times f_{cu}$ = **15.75** N/mm$^2$ |

***Concrete stress OK***

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

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

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

**Concrete and Steel strains**

| | |
|---|---|
| Strain in the reinforcement; | $\varepsilon_s = f_{st} / E_s$ = **463.3×10$^{-6}$** |
| Allowable steel strain; | $0.8 \times f_y / E_s$ = **2.000×10$^{-3}$** |

***Steel strain OK***

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

Level of crack; a' = h = **450** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x) =$ **$574.2\times10^{-6}$**

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

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

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

Breadth of tension face; $b_t = b =$ **600** mm

$\varepsilon_m = \min( \varepsilon_1 ,\max(0, \varepsilon_1 - [ \text{factor} \times b_t \times (h - x) \times (a' - x) / (3\times E_s \times A_{st} \times (h' - x))])) =$ **$329.2\times10^{-6}$**

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

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

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

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

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

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

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

**Serviceability Limit State - Cracking in Columns**

**Bent about the minor axis**

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

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

**Column dimensions, depth to steel (assumed symmetrical)**

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

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

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

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

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

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

Diameter of tension reinforcement; $D_{col} =$ **20** mm

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

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

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

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

Effective depth to tension reinforcement; b' = **540.0** mm

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

**Modular Ratio**

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

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

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

**Neutral Axis position**

For equilibrium; $F_{st}$ equates $F_c$

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

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

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

Depth of concrete in compression; x = **190.2** mm

**Concrete and Steel stresses**

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

Taking moments about the centreline of the reinforcement:-

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

Solving for concrete stress $f_c$ gives; $f_c = 2 \times M_{Y_SLS} / ( h \times x \times (b' - x/3)) =$ **2.45** N/mm$^2$

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

***Concrete stress OK***

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

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

Solving for steel stress $f_{st}$ gives; $f_{st} = M_{Y_SLS} / ( A_{st} \times (b' - x/3)) =$ **66.79** N/mm$^2$

**Concrete and Steel strains**

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s =$ **333.9×10$^{-6}$**

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

***Steel strain OK***

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

Level of crack; a' = b = **600** mm

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(b' - x) =$ **391.2×10$^{-6}$**

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

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

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

Breadth of tension face; $h_t = h$ = **450** mm

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

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

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

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

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

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

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

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

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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** mm

Inner sagging steel

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

Depth to inner tension steel (resisting sagging)

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

Outer hogging steel

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

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

Depth to outer tension steel (resisting hogging)

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

Inner hogging steel

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

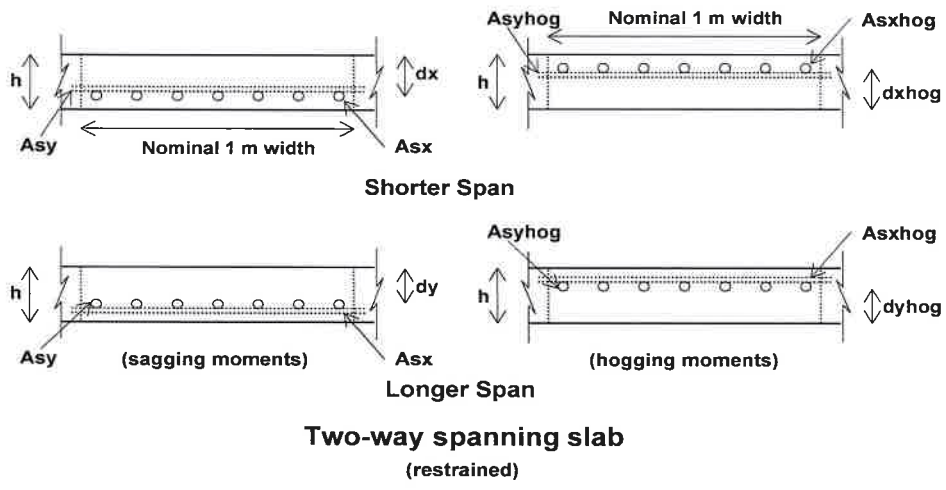
Depth to inner tension steel (resisting hogging)

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

Materials

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

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




**Two-way spanning slab**

**(restrained)**

## RESTRAINED – 2 WAY SPANNING

### MAXIMUM DESIGN MOMENTS

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

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

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

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

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

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

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

Number of discontinuous edges

$N_d$ = **2**

Moment coefficients

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

$\beta_1$ = if($Edge_1$ == "C", $4/3\times\beta_{sy}$,0) = **0.045**

$\beta_2$ = if($Edge_2$ == "C", $4/3\times\beta_{sy}$,0) = **0.000**

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

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

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

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

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

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

Maximum span moments per unit width - restrained slabs

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

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

Maximum support moments per unit width - restrained slabs

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

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

**CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL**

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

Moment Redistribution Factor; $\beta_{bx}$ = **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** mm

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

Area of tension steel required

$A_{sx_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x)$ = **283** mm²/m

**Tension steel**

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

$A_{sx_prov} = A_{sx}$ = **524** mm²/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** mm

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

Area of tension steel required

$A_{sy_req} = abs(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y)$ = **180** mm²/m

**Tension steel**

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

$A_{sy_prov} = A_{sy}$ = **524** mm²/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** mm

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

Area of tension steel required

$A_{sxhog_req} = abs(m_{sxhog}) / (1/\gamma_{ms} \times f_y \times z_{xhog})$ = **382** mm²/m

**Tension steel**

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

$A_{sxhog_prov} = A_{sxhog}$ = **524** mm²/m

***Area of outer tension steel provided sufficient to resist hogging***

**Tension In One Truss Member**

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

Partial safety factor for strength of steel; $\gamma_{ms}$ = **1.15**

Req'd area of reinf't (in one truss member); $A_{s_req} = F_t / (1/\gamma_{ms} \times f_y)$ =**588** mm$^2$

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

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

***PASS Tension***

**Max / Min Areas Of Reinforcement**

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

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

***Area of tension steel provided OK***

**Shear**

**Applied shear stress**

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

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

Design shear stress; $v = V / (b_v \times d)$ = **0.47** N/mm$^2$

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

***Shear stress - OK***

**Design concrete shear strength**

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

Shear stress

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

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

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

Enhanced shear stress; $v_{c_enh} = \min(v_{allowable}, 2 \times d \times v_c / a_v)$ = **2.21** N/mm$^2$

***Concrete shear strength - OK, no links reqd.***

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

**Clear Distance Between Bars In Tension**

Bar spacing;

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

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

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

***Bar spacing OK***

**Clear Distance Between Face Of Beam And Tension Bars**

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

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

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

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

***Max distance to beam edge check - OK***

**Anchorage Of Tension Steel**

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

**Deflection Check**

| | |
|---|---|
| Redistribution ratio; | $\beta_b = 1.0$ |
| Design service stress in tension reinforcement; | $f_s = 2 \times f_y \times A_{s_req} / (3 \times A_{s_prov} \times \beta_b)$ = **97.5** N/mm$^2$ |
| Modification for tension reinforcement; | |

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

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

***PASS - Deflection***

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

$v_{c3} = v_{c2} = v_{c_252} \times ( \min(f_{cu}, 40\ N/mm^2 )/25\ N/mm^2 )^{1/3} = \mathbf{0.32}$ N/mm²

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

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

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

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

**Local Shear At Concentrated Loads**

| | |
|---|---|
| Total length of inner perim. at edge of loaded area; | $u_0 = 2 \times ( x + y ) =$**2000** mm |
| Assumed average depth to tension steel; | $d_{av} = d - D_t =$ **1424** mm |
| Max shear effective across perimeter; | $V_p = F_{uls} =$ **1400.0** kN |
| Stress around loaded area; | $v_{max} = V_p / (u_0 \times d_{av}) =$ **0.49** N/mm² |
| Allowable shear stress; | $v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2 ) =$ **4.00** N/mm² |

***Shear stress - OK***

**Clear Distance Between Bars In Tension**

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

Actual bar spacing;

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

| | |
|---|---|
| Maximum allowable spacing of bars; | $spacing_{max} = \min((17000\ N/mm)/f_s, 300\ mm) =$ **300** mm |
| Minimum required spacing of bars; | $spacing_{min} = h_{agg} + 5\ mm =$ **25** mm |

***Bar spacing OK***

**Clear Distance Between Face Of Beam And Tension Bars**

| | |
|---|---|
| Distance to face of beam; | $Dist_{edge} = c_{adopt} + L_{dia} + D_t/2 =$ **60** mm |
| Design service stress in reinforcement; | $f_s = 2 \times f_y \times A_{s_req} /(3 \times A_{s_prov} \times \beta_b) =$ **34.3** N/mm² |
| Max allowable clear spacing; | $Spacing_{max} = \min((47000\ N/mm)/f_s, 300\ mm) =$ **300** mm |
| Max distance to face of beam; | $Dist_{max} = Spacing_{max}/2 =$ **150** mm |

***Max distance to beam edge check - OK***

**Anchorage Of Tension Steel**

| | |
|---|---|
| Anchorage factor; | $\phi_{factor} =$**44** |
| Type of lap length; | lap_type =**"tens_lap"** |
| Type of reinforcement; | reft_type = **"def2_fy500"** |
| Minimum radius; | $r_{bar} =$ **32** mm |
| Minimum end projection; | $P_{bar} =$ **130** mm |
| Minimum anchorage length or lap length req'd; | $L_{table\ 3.27} = \phi_{factor} \times D_t =$ **704** mm |
| Check anchorage length | $L_{cl.\ 3.12.9.4} = 12 \times D_t + d/2 =$ **912** mm |
| Required minimum effective anchorage length; | $L_a = \max(L_{table\ 3.27}, L_{cl.\ 3.12.9.4}) =$ **912** mm |

**Check bearing stress on minimum radius bend**

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

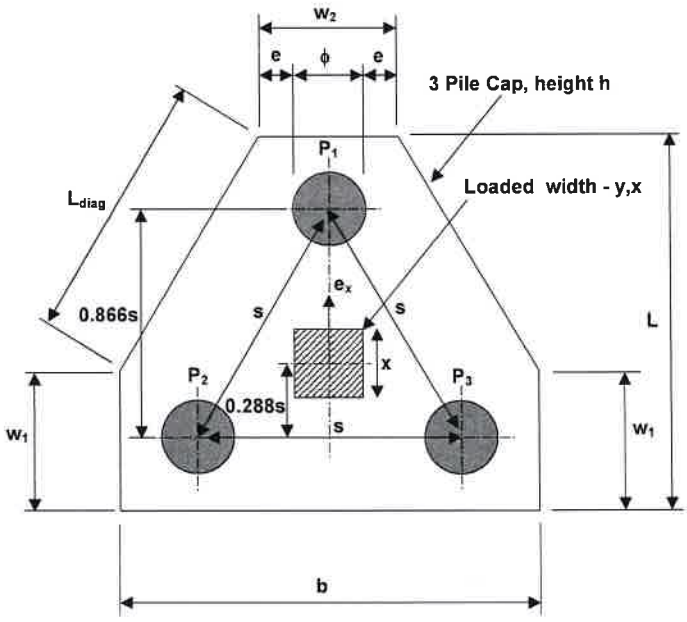
| | |
|---|---|
| Force per bar at bend; | $F_{bt} = F_t / L_{nt} =$ **9.0** kN |
| Bearing stress; | $f_{bt} = F_{bt} / (r_{bar} \times D_t) =$ **17.58** N/mm² |
| Edge bar centres; | $s_{ext} = c_{adopt} + D_t =$ **56** mm |
| Edge maximum allowable bearing stress; | $f_{bt_max_ext} = 2 \times f_{cu} / ( 1 + 2 \times (D_t / s_{ext} )) =$ **31.82** N/mm² |

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

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

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

## RC PILE CAP DESIGN




**Pile Cap Design – Truss Method**

**Design Input - 3 Piles - No Eccentricity**

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

**Cover**

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

**Pile Cap Forces**

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

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

**Compression In Pile Cap - Suggested Additional Check**

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

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

***PASS Compression***

**Tension In One Truss Member**

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

Partial safety factor for strength of steel; $\gamma_{ms}$ = **1.15**

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

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

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

***PASS Tension***

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

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

Maximum allowable area of steel; $A_{st_max}$ = 4 % × $A_c$ = **20000** 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** kN

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

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

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

***Shear stress - OK***

**Design concrete shear strength**

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

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

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

***Concrete shear strength - OK, no links reqd.***

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

**Local Shear At Concentrated Loads**

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

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

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

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

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

***Shear stress - OK***

**Clear Distance Between Bars In Tension**

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

Actual bar spacing;

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

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

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

***Bar spacing OK***

**Clear Distance Between Face Of Beam And Tension Bars**

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

Design service stress in reinforcement; $f_s = 2 \times f_y \times A_{s_req}/(3 \times A_{s_prov} \times \beta_b) = \mathbf{81.1}\ N/mm^2$

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

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

***Max distance to beam edge check - OK***

**Anchorage Of Tension Steel**

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

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

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

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

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

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

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

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

**Check minimum radius required on bend**

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

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

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

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

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

Internal maximum allowable bearing stress; $f_{bt_max_int} = 2 \times f_{cu} / (1 + 2 \times (D_t / s_{int})) = \mathbf{30.00}\ N/mm^2$

Design max allowable bearing stress; $f_{bt_max} = \min(f_{bt_max_ext}, f_{bt_max_int}) = \mathbf{30.00}\ N/mm^2$

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

***Minimum radius of bend required = 44 mm***

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# STRUCTURAL DRAWINGS

THE LANDLENS SUITE

# Eight ways to know a piece of land before you pay for it.

This report covers one part of the picture. These cover the rest, drawn from the same government records, for any property in Tamil Nadu.

### LandLens One
Ask anything about a property and get an answer grounded in live government records, with the sources shown.
**/landlens/one**

### Verify My Land
Every record check run on one property and returned as a single report.
**/landlens/verify-my-land**

### Legal Opinion
An advocate's written opinion on the title, built on the records we pull for it.
**/landlens**

### One Map
Every layer on one map: the parcel, its boundary, zoning, water bodies and what sits around it.
**/landlens/one/map**

### PriceMap
What land actually changed hands for around this plot, taken from the registered deeds.
**/landlens/pricemap**

### Build Scope
What you are allowed to build here: floor space, height, coverage and setbacks.
**/landlens/build-scope**

### Property Tracker
Watch a property and hear about it when something new is registered against it.
**/landlens/property-tracker**

### Deal Room
Run the transaction in one place: documents, stamping, signatures and the parties.
**/landlens/deal-room**

FREE TOOLS · NO ACCOUNT NEEDED

# Twenty-four checks you can run yourself, at no cost.

## Records and verification

The registers themselves, read for you.

- Encumbrance Certificate
- Patta and Chitta
- Field Measurement Sketch
- EC Analyzer
- Bank charge check
- Court case search
- Seller background check
- Approval check by survey

## Value and duty

What it is worth and what it costs to register.

- Guideline value by street
- Guideline value by survey
- Stamp duty and registration fees
- Capital gains
- Floor space index
- Area unit converter

## Zoning and land use

What the plan says can happen on this land.

- Land use zone finder
- CMDA land use check
- Coastal regulation zone
- Master plan check
- Water bodies finder
- Airport height clearance

## Registers and directories

Who holds it, and which office keeps the record.

- Find your registering office
- Temple property check
- Waqf property check
- RERA register

**Every free tool**
/tools

**Services we run for you**
/services

**The LandLens suite**
/landlens/one

Verified RealEstate™

# Every claim in this document traces back to a government record.

- Legal opinions, due diligence and title verification
- Building plan, layout and sub-division approvals
- Completion and occupancy certificates, regularisation
- Escrow, negotiation and NRI property management
- Land survey, soil testing, architecture and construction

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Prepared from the registers named on the page each finding sits on. Where a register could not be read, this document says so in its place rather than leaving the gap to be read as a clean result.