DETAIL DESIGN REPORT

Bangladesh University of Engineering and Technology

CE 404: Capstone Project

Project Topic

Reconstruction of Green-Road Staff Quarter

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Group 02, Section B1

<u>Name of the Project:</u> Reconstruction of Green Road Staff Quarter

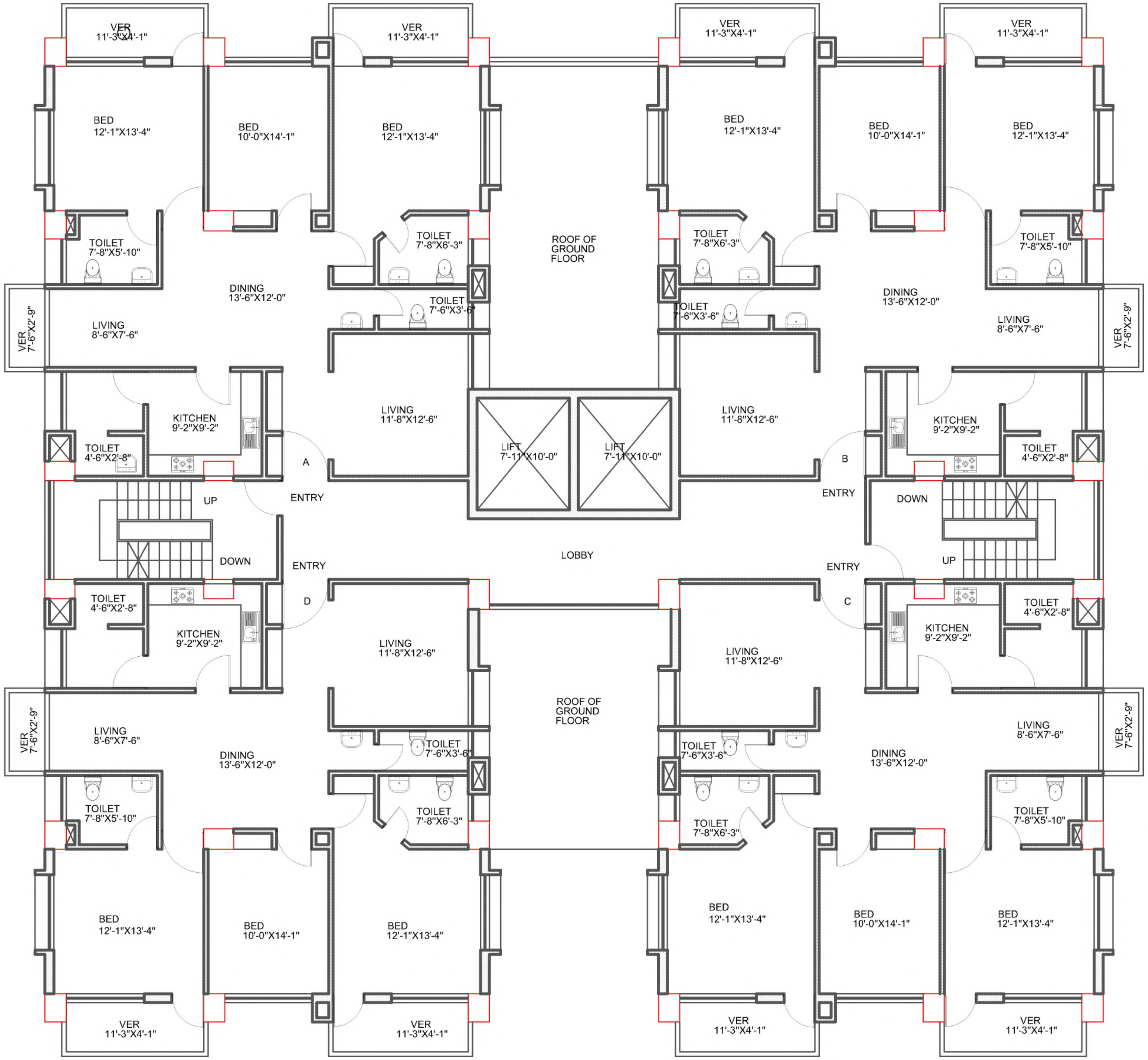
Introduction:

This report aims to provide the findings from the analysis done on the suggested project structure. The structural analysis followed BNBC 2020's instructions. To guarantee the intended service life, the structural system's selection prioritized efficiency, cost-effectiveness, and safe design. The suggested structure was modeled and analyzed using ETABS, and the project's foundation was decided upon as a mat foundation. Considering soil type, strength, and compressibility, a thorough geotechnical study of the site served as the basis for the foundation modeling and analysis. The comprehensive research sought to ensure the structure's stability and safety under anticipated loading scenarios. In conclusion, the purpose of this study is to provide a thorough evaluation of the proposed building's structural integrity.

Design Basis:

Building Type	High Rise Building (B+G+12)
Occupancy Class/Sub-Class	Residential (A3)
Fire Zone	1
Type of Construction	Non-Combustible: Type I-C (2 hours protected)
Code Followed	BNBC (2020)
Exposure Category	A
Site Class	SC
Occupancy Category	2
Seismic Design Category	SDC C
Seismic Zone	2
Seismic Force Resistant System	Dual System: Intermediate Moment Frame with Ordinary Reinforced Concrete Shear Wall
Wind Case	ASCE 7-05
Building Type	Flexible (T>1s)

MASTERPLAN
Mirpur Road
Future Extension
(Preserved)
Only Pedestrian
Only pedestrian
Z-1
Z-2
Z-3
Z-4
G1
G2
G3
N



TYPICAL FLOOR PLAN

TYPICAL 1 UNIT AREA: 1500 sft

COMMON SPACE AREA: 1380 sft

TOTAL AREA: 7380 sft

Checked and Vetted By:

Designed By: Group-2(B1)

Prepared By: Group-2(B1)

Project: Reconstruction of Green Road Staff Quarter

Client:

Submitted By: Group-2(B1)

Summary of Load:

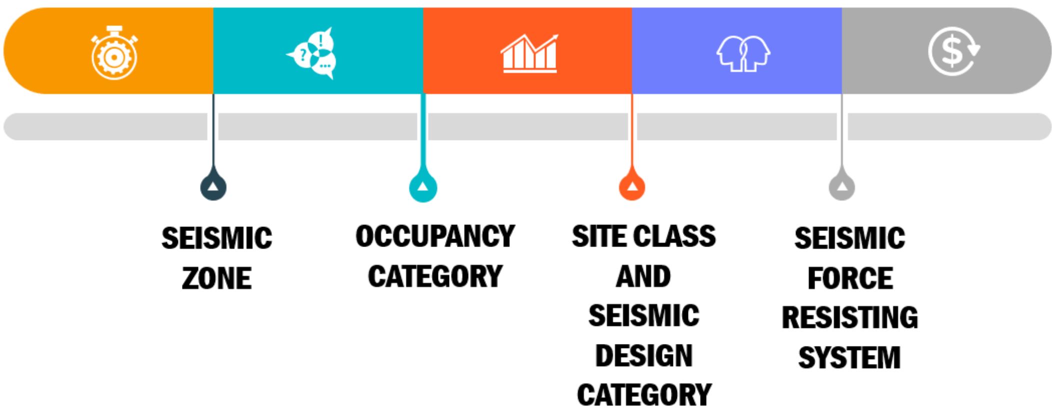
Assigned Position	Type of Load	Value of Load (Psf)
Slab	Live Load	40
Stair Slab	Live Load	100
Floor Finish	Dead Load	25
Base of OHWT	Dead Load	624
Base of OHWT	Dead Load	1860
Basement Wall (Surcharge)	Live Load	200
Basement Wall (Earth-Pressure)	Dead Load	C=-54.56 D=545

Load Calculation for Partition Wall:

Panel No.	Area of Panel (sq.ft)	Length of Partition Wall (ft)	Thickness of Wall (ft)	Height (ft)	Volume of Wall (cft)	Weight of Wall (lb)	Pressure on Slab Area (Psf)
1	240	29	0.4	9	113	16917	70
2	353	65	0.4	9	251	37625	107
3	400	47	0.4	9	183	27417	69
4	588	59	0.4	9	229	34417	59
5	300	33	0.4	9	119	17820	59

Modeling Basis:

For Equivalent Static Load Analysis: The following factors are important: -

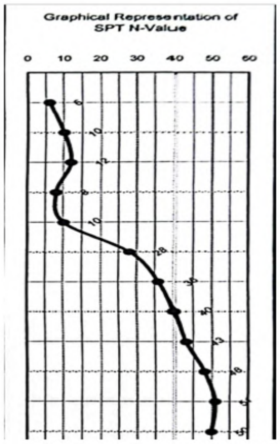


a. Seismic Zone and Occupancy Category:

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Occupancy Category	Importance factor I
I or II	1.0
III	1.25
IV	1.5

b. Site Class: SC



Difference between successive Depths (m)	Avg of Spt N value	AxB	SPT N
1.5	8	12	30.25
1.5	11	16.5	
1.5	10	15	
1.5	9	13.5	
1.5	19	28.5	
1.5	32	48	
1.5	38	57	
1.5	41.5	62.25	
1.5	45.5	68.25	
1.5	49	73.5	
1.5	50	75	
1.5	50	75	
		544.5	

Site classification based on soil properties

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity V_s (m/s)	Standard Penetration Value, N (blows/30cm)	Undrained shear strength, S_u (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70

c. Seismic Design Category: SDC-C

Table 6.2.18: Seismic Design Category of Buildings

Site Class	Occupancy Category I, II and III			
	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D
SB	B	C	D	D
SC	B	C	D	D
SD	C	D	D	D
SE, S_1, S_2	D	D	D	D

d. Seismic Force Resisting System: Dual System (IMRF with ordinary reinforced concrete shear wall)

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
1. Special steel concentrically braced frames	6	2.5	5	NL	NL	11
2. Special reinforced concrete shear walls	6.5	2.5	5	NL	NL	50
3. Ordinary reinforced masonry shear walls	3	3	3	NL	50	NP
4. Ordinary reinforced concrete shear walls	5.5	2.5	4.5	NL	NL	NP

e. Building Type: Flexible (T>1s)

$T=Ct*hn^{m}=.0466*42.7^{.9}=1.37s>1s$

Table 6.2.20: Values for Coefficients to Estimate Approximate Period

Structure type	C_t	m
Concrete moment-resisting frames	0.0466	0.9
Steel moment-resisting frames	0.0724	0.8
Eccentrically braced steel frame	0.0731	0.75
All other structural systems	0.0488	0.75

Note: Consider moment resisting frames as frames which resist 100% of seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting under seismic forces.

Section Property:

Material Property:

fc`=4000psi (Concrete compressive strength)

fy= 60000psi (Rebar yield strength)

Load Combinations Used:

1. 1.4D
2. 1.2D + 1.6L
3. 1.2D + L

4. 1.2D + 0.8Wx
5. 1.2D 0.8Wx
6. 1.2D + 0.8Wy
7. 1.2D-0.8Wy
8. 1.2D+L+ 1.6Wx
9. 1.2D + L-1.6Wx
10. 1.2D+L+ 1.6Wy
11. 1.2D+L-1.6Wy
12. 1.2D + L + Ex + 0.3Ey
13. 1.2D + L + Ex- 0.3Ey
14. 1.2D + L - Ex + 0.3Ey
15. 1.2D + L - Ex- 0.3Ey
16. 1.2D + L + Ey + 0.3Ex
17. 1.2D + L + Ey- 0.3Ex
18. 1.2D + L - Ey + 0.3Ex
19. 1.2D + L - Ey- 0.3Ex
20. 0.9D + 1.6Wx
21. 0.9D - 1.6Wx
22. 0.9D + 1.6Wy
23. 0.9D - 1.6Wy
24. 0.9D + Ex + 0.3Ey
25. 0.9D + Ex- 0.3Ey
26. 0.9D - Ex + 0.3Ey

27. 0.9D - Ex- 0.3Ey

28. 0.9D + Ey + 0.3Ex

29. 0.9D + Ey- 0.3Ex

30. 0.9D - Ey + 0.3Ex

31. 0.9D - Ey- 0.3Ex

Here,

D=dead load

L= Live load

Wx= wind load in X direction

Wy= wind load in Y direction

Ex= Earthquake load in X direction

Ey= Earthquake load in Y direction

Serviceability Check:

Beam Deflection Check:

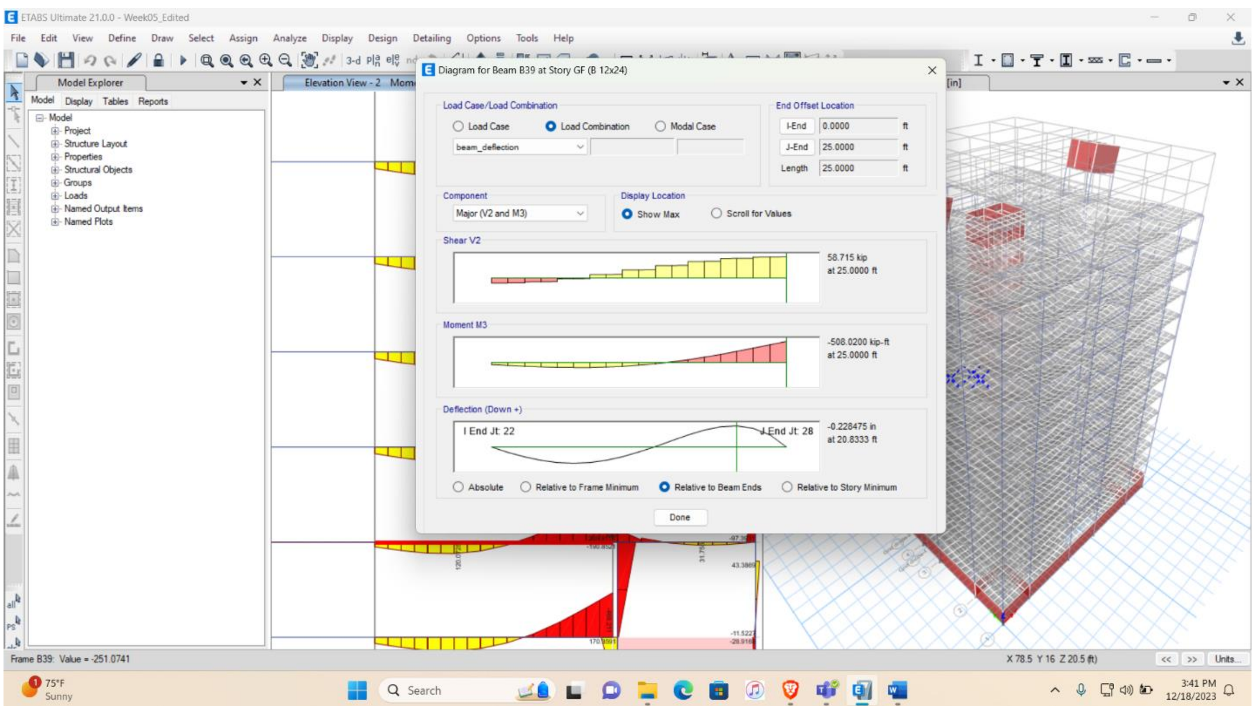
Load Combination = DL+LL

Limit = L/240 = (25*12)/240 = 1.25

From ETABS analysis maximum beam deflection = 0.228 inch < 1.25 (okay)

Long term deflection factor $\lambda = 3$

Long term deflection = 3*0.228 = 0.684 inch < 1.25 inch (okay)

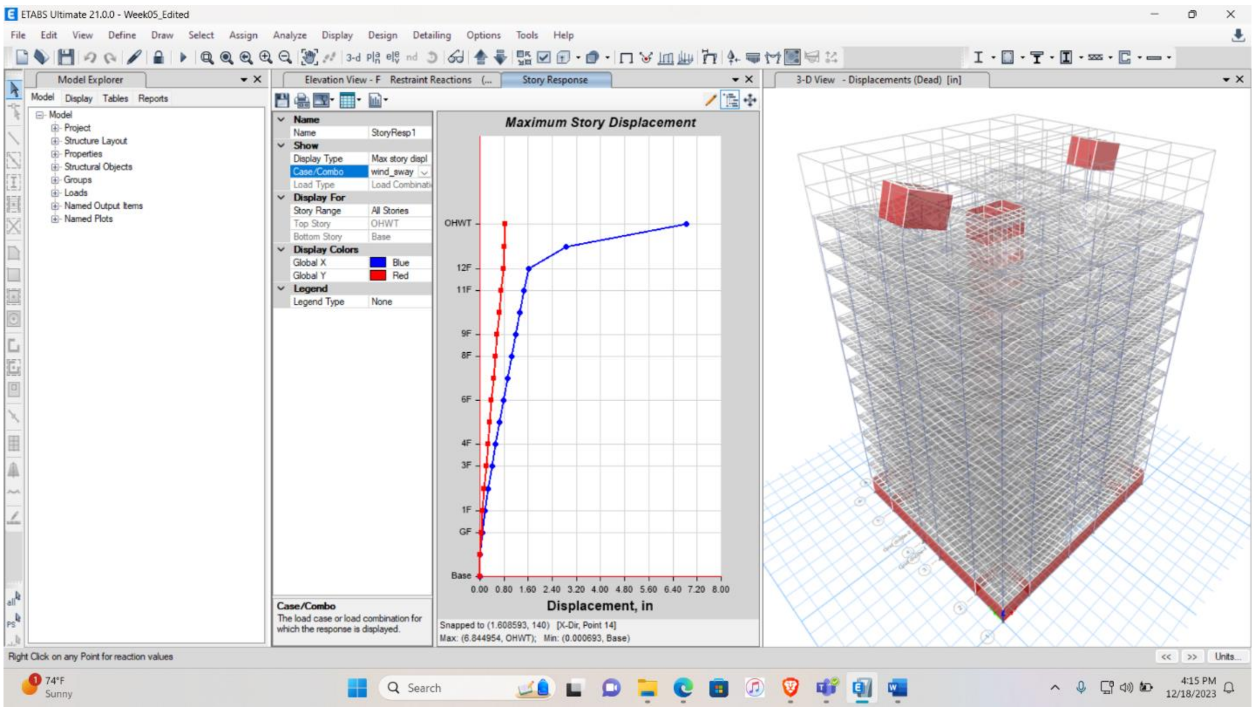


Wind Sway Check:

Load Combination: DL+0.5LL+0.7WL

Limit = H/500 = (140*12)/500 = 3.36 inch

From ETABS analysis maximum wind sway= 1.61 inch < 3.36 inch (okay)

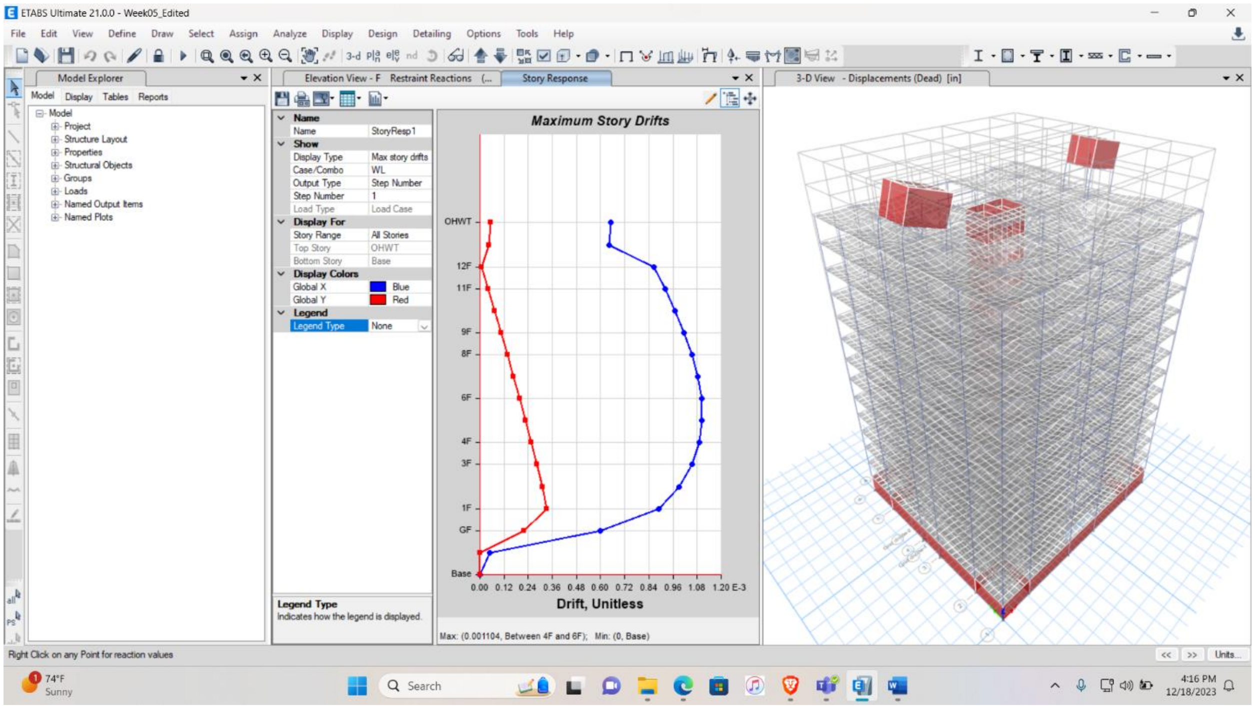


<u>**Wind Drift:**</u>

Load Combination = WL

Limit: Δ / h = 0.004

From ETABS analysis maximum beam deflection = .001104 < .004 (okay)



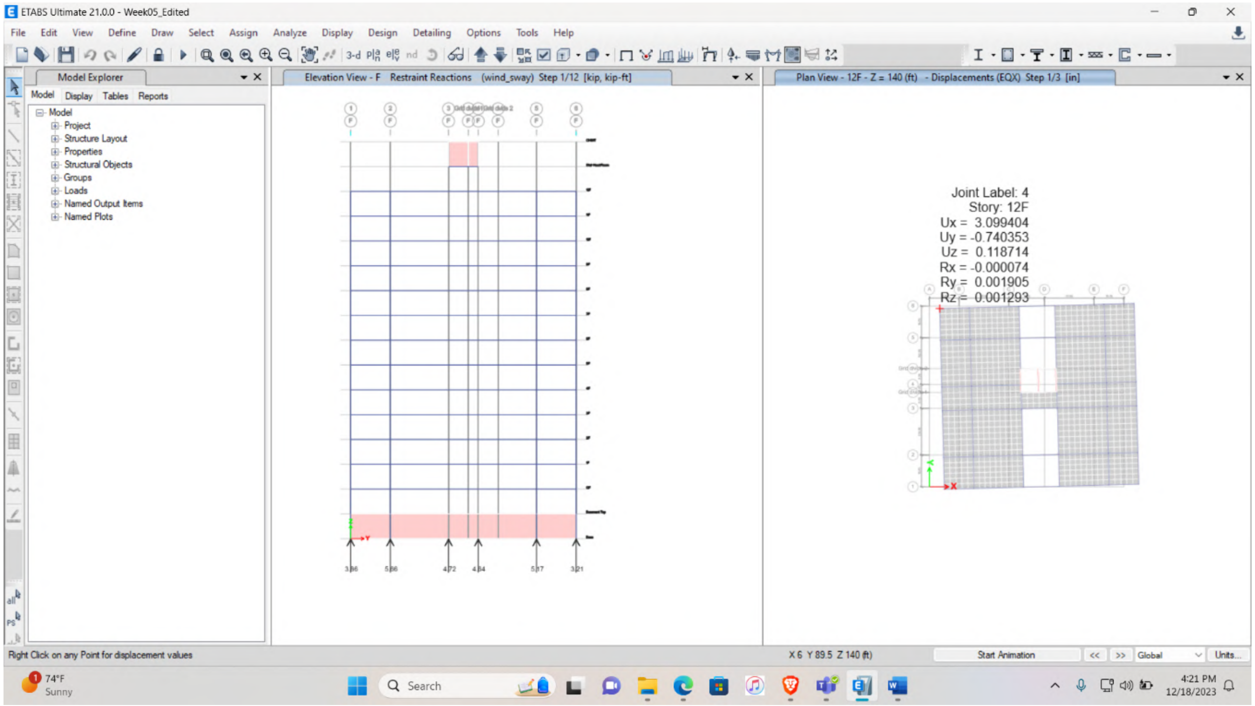
Torsional Irregularity:

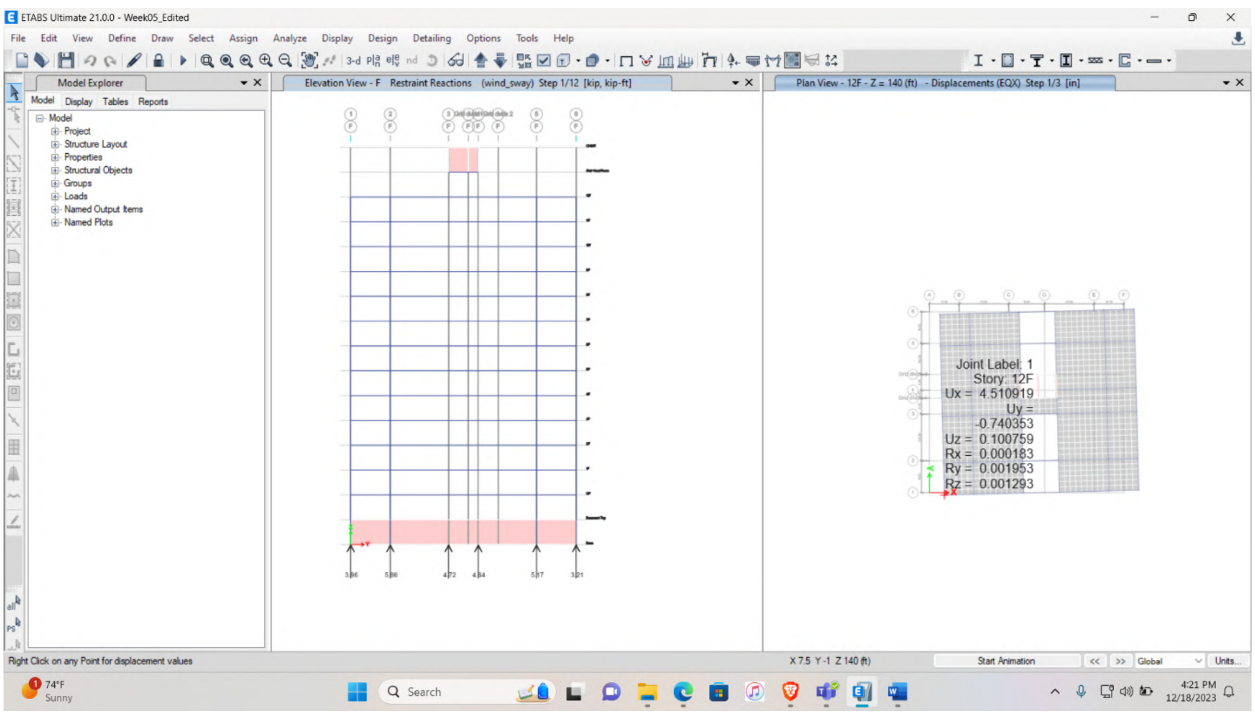
For EQx :

Limit: $\Delta_{max} < 1.2\ \Delta_{avg}$. Where, $\Delta_{avg} = (\Delta_{max} + \Delta_{min})/2$

From ETABS analysis: $\Delta_{max} = 4.511$, $\Delta_{min} = 3.0994$

$\Delta_{avg} = 3.81$; $1.2^* \ \Delta_{avg} = 4.57 > 4.511$ (Okay)


ETABS Ultimate 21.0.0 - Week05_Edited
Model Explorer
Elevation View - F Restraint Reactions (wind_sway) Step 1/12 [kip, kip-ft]
Plan View - 12F - Z = 140 (ft) - Displacements (EQX) Step 1/3 [in]
Joint Label: 4
Story: 12F
Ux = 3.099404
Uy = -0.740353
Uz = 0.118714
Rx = -0.000074
Ry = 0.001905
Rz = 0.001293
Right Click on any Point for displacement values
X 6 Y 89.5 Z 140 (ft)
Start Animation
Global
Units...

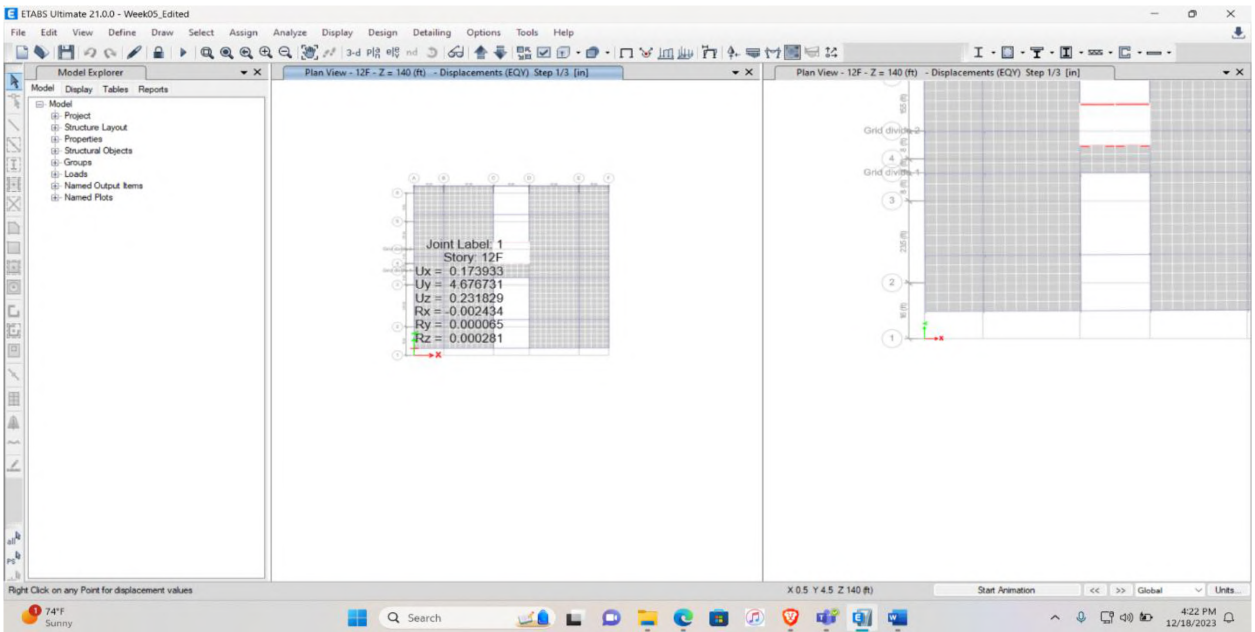

ETABS Ultimate 21.0.0 - Week05_Edited
Model Explorer
Elevation View - F Restraint Reactions (wind_sway) Step 1/12 [kip, kip-ft]
Plan View - 12F - Z = 140 (ft) - Displacements (EQX) Step 1/3 [in]
Joint Label: 1
Story: 12F
Ux = 4.510919
Uy =
-0.740353
Uz = 0.100759
Rx = 0.000183
Ry = 0.001953
Rz = 0.001293
Right Click on any Point for displacement values
X 7.5 Y -1 Z 140 (ft)
Start Animation
Global
Units...

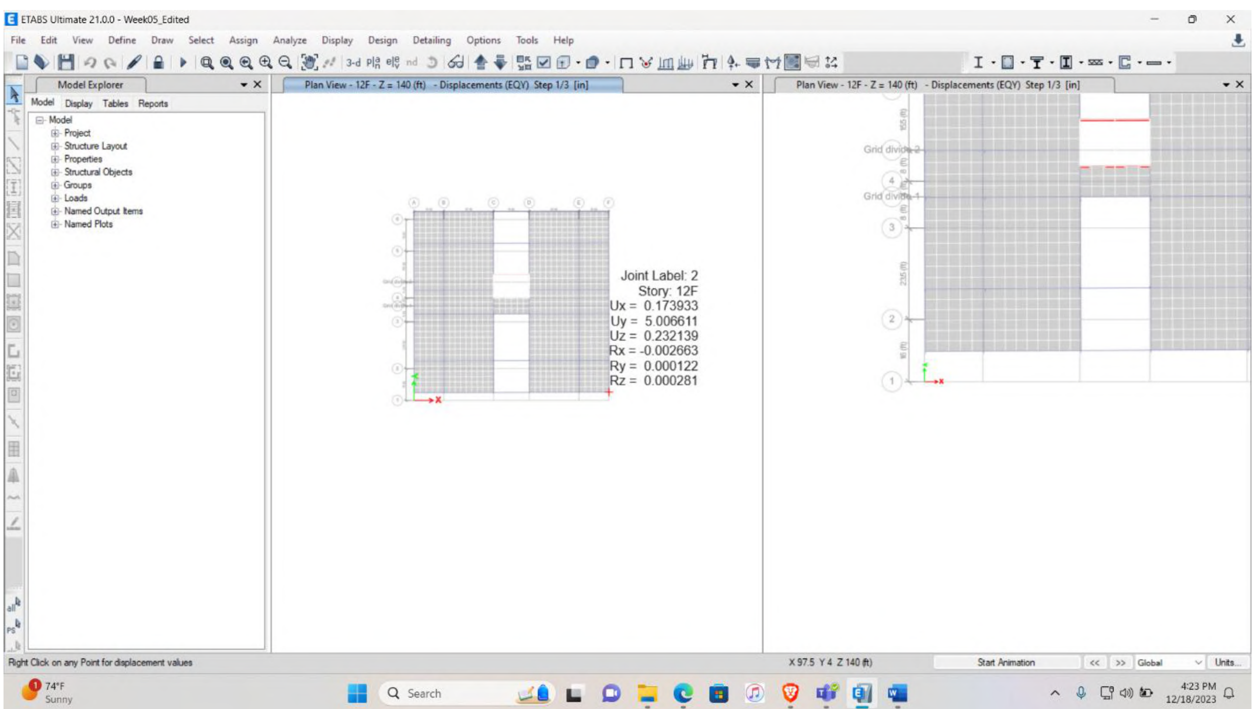
For EQy =

Limit : $\Delta_{max} < 1.2\ \Delta_{avg}$. Where, $\Delta_{avg} = (\Delta_{max}+\Delta_{min})/2$

From ETABS analysis : $\Delta_{max} = 5.006$, $\Delta_{min} = 4.68$

$\Delta_{avg} = 4.843$; $1.2*\ \Delta_{avg} = 5.8116 > 5.006$ (Okay)





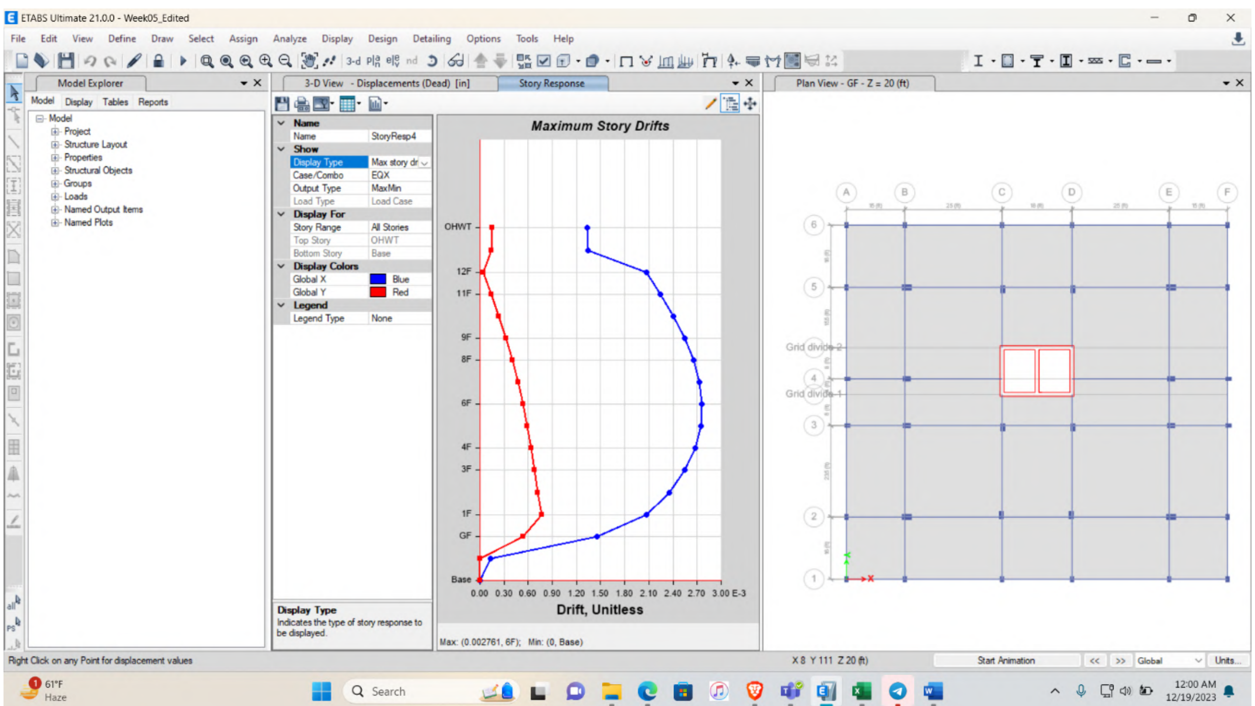
<u>Earthquake Drift:</u>

<u>EQx:</u>

Max = 0.002761

$C_d = 4.5$

$\Delta/h = 0.002761 * 4.5 = 0.01242 < 0.02$ (okay)

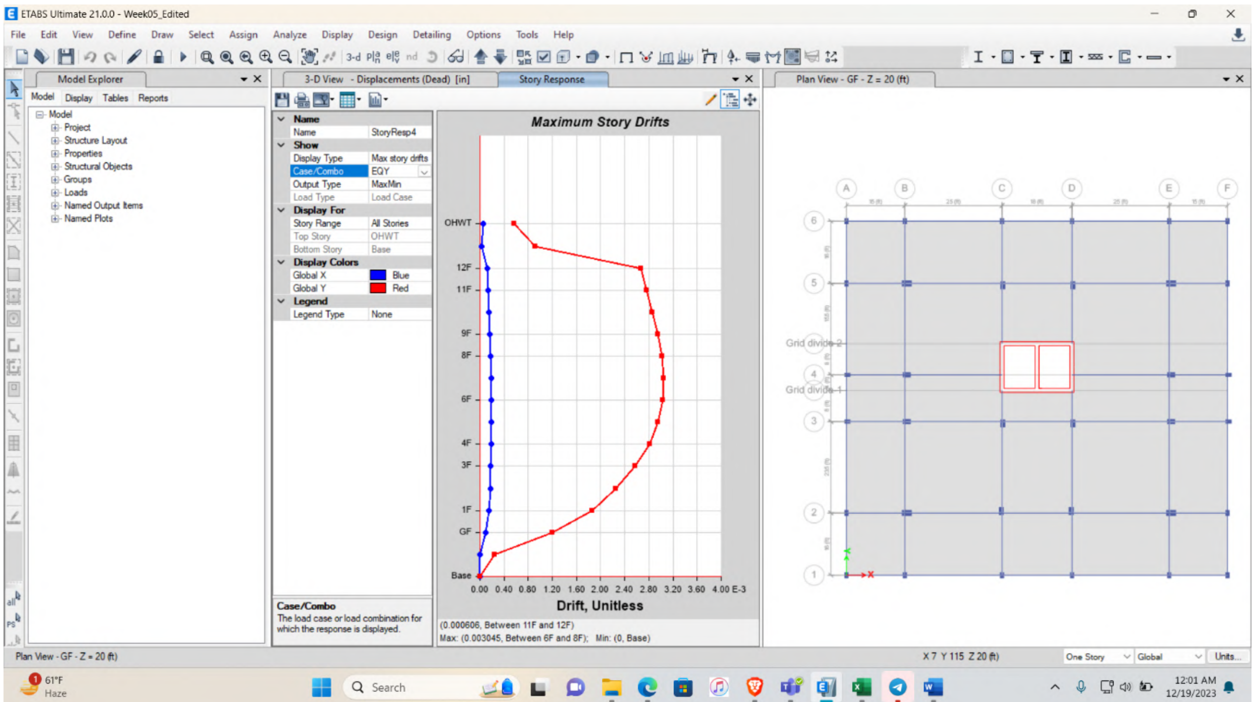


EQy:

Max = 0.00305

$C_d = 4.5$

$\Delta/h = 0.00305 * 4.5 = 0.013725 < 0.02$ (okay)



Slab Deflection:

Stress on slab = 439psi

Modulus of Rupture =7.5*$\sqrt{f_c}$ =474 psi> stress on slab

Therefore, the slab is uncracked.

So, Stiffness modifier = 1

(Deflection) for dead load=0.39”

Deflection for live load = 0.11”

Now long term deflection will be considered for dead load only where the factor is 2.5

So total deflection =0.39*2.5+.11=1.085”

Limit = L/240

=23.5*12/240 in

=1.18 in>1.085”

(ok)

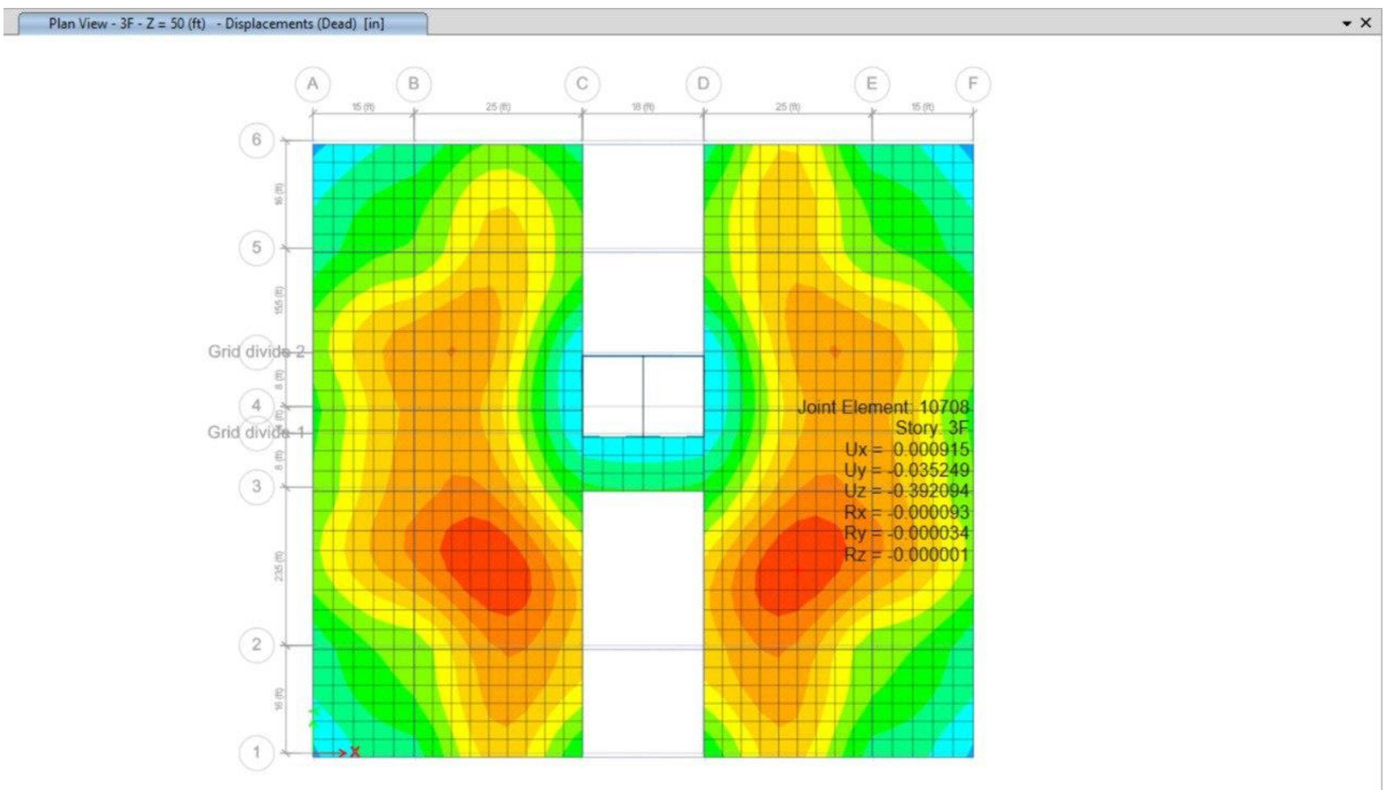


Figure: Deflection due to dead load

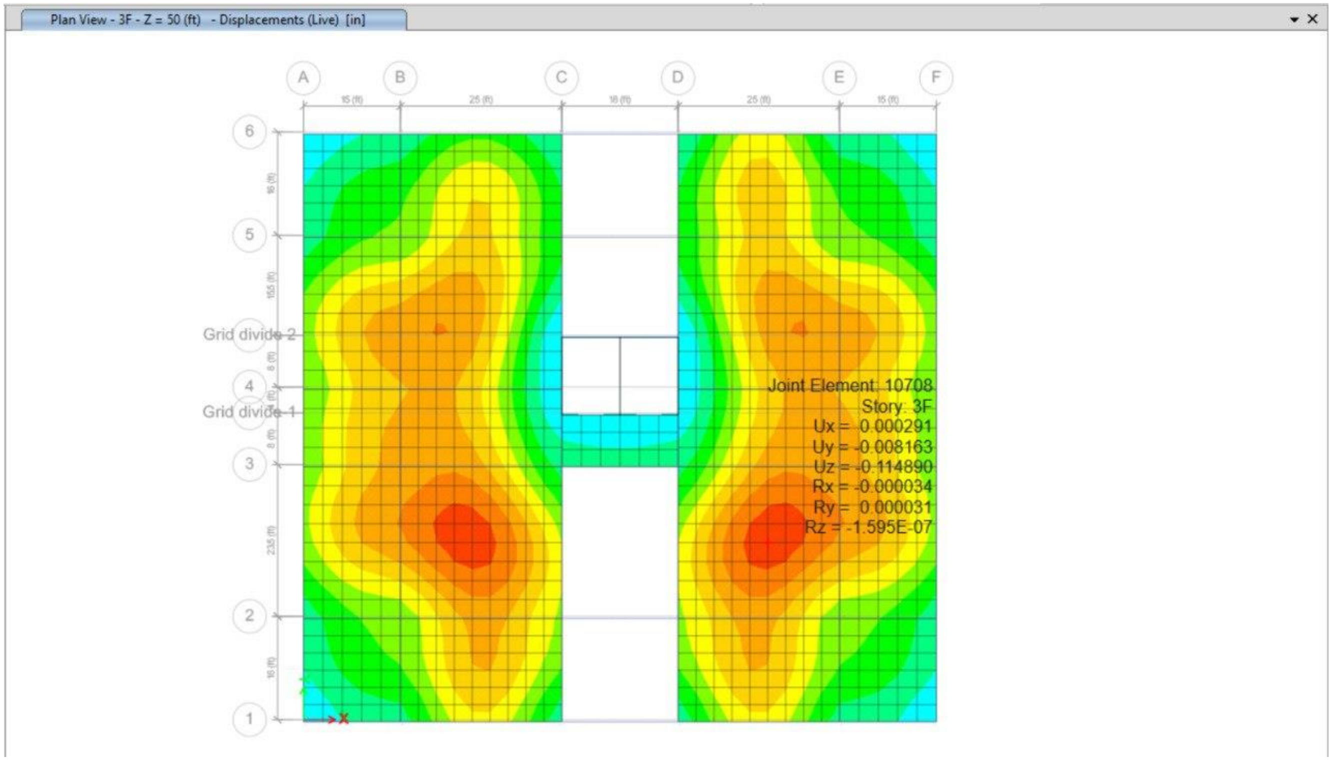


Figure: Deflection due to live load

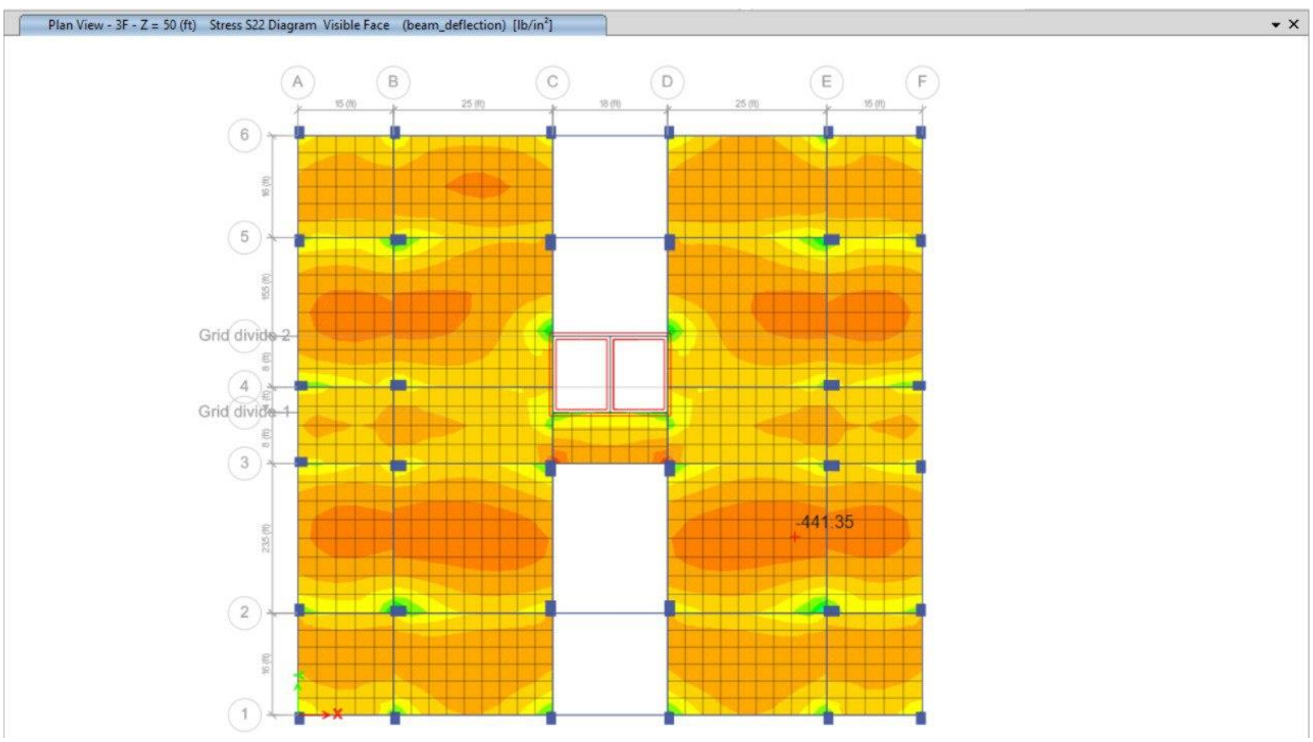


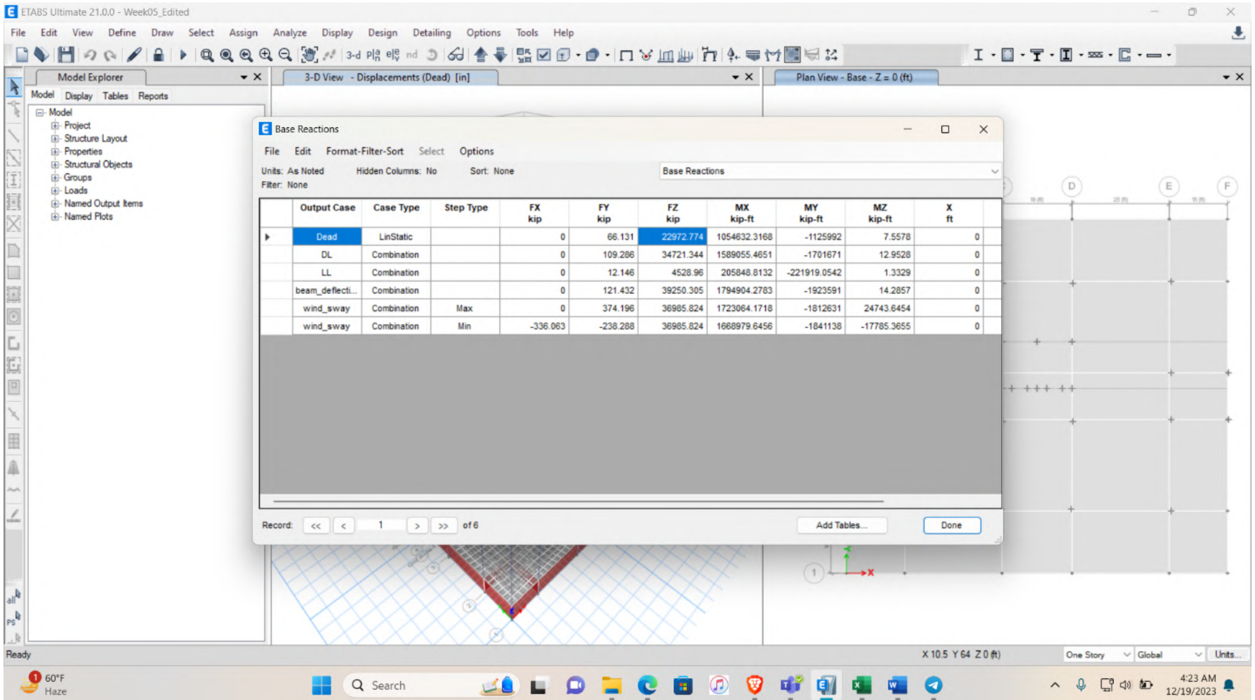
Figure: Stress on Slab

Load Validation

Dead Load Validation							
Component	Length (ft)	Depth (ft)	Width (ft)	Concrete Unit Weight (lb/ft^3)	Numbers	Story	Weight(Kip)
Beam	1092	2	1	150	1	14	4586.4
C18*12	10	1.5	1	150	20	14	630
C30*18	10	2.5	1.5	150	4	14	315
C24*15	10	2	1.25	150	10	14	525
	Area (ft^2)						
Slab 8	3094	0.666666667		150	13		4022.2
Slab 8 GF	3796	0.666666667		150	1		379.6
Slab 7	3970	0.583333333		150	13		4515.875
Slab 7 GF	4546	0.583333333		150	1		397.775
Stair Slab	360	0.833333333		150	14		630
Shear Wall	10	1	72	150	14		1512
Basement Wall	10	1.25	378	150	1		708.75
UGWT	31	10	0.75	150	4		139.5
Mat Foundation	91	98	3.416666667	150	1		4570.475
						Total	22932.58
C18*12	10	1.5	1	150	4		9
C24*15	10	1.25	2	150	4		15
OHWT Wall	54	0.75	0.833333333	150	2		10.125
OHWT Slab	180	0.333333333	1	150	2		18
						Total	52.125
						Total	22984.7

From ETABS,

Base Reaction for DL = 22973 Kip



% Error = (22984-22973)/22984*100%

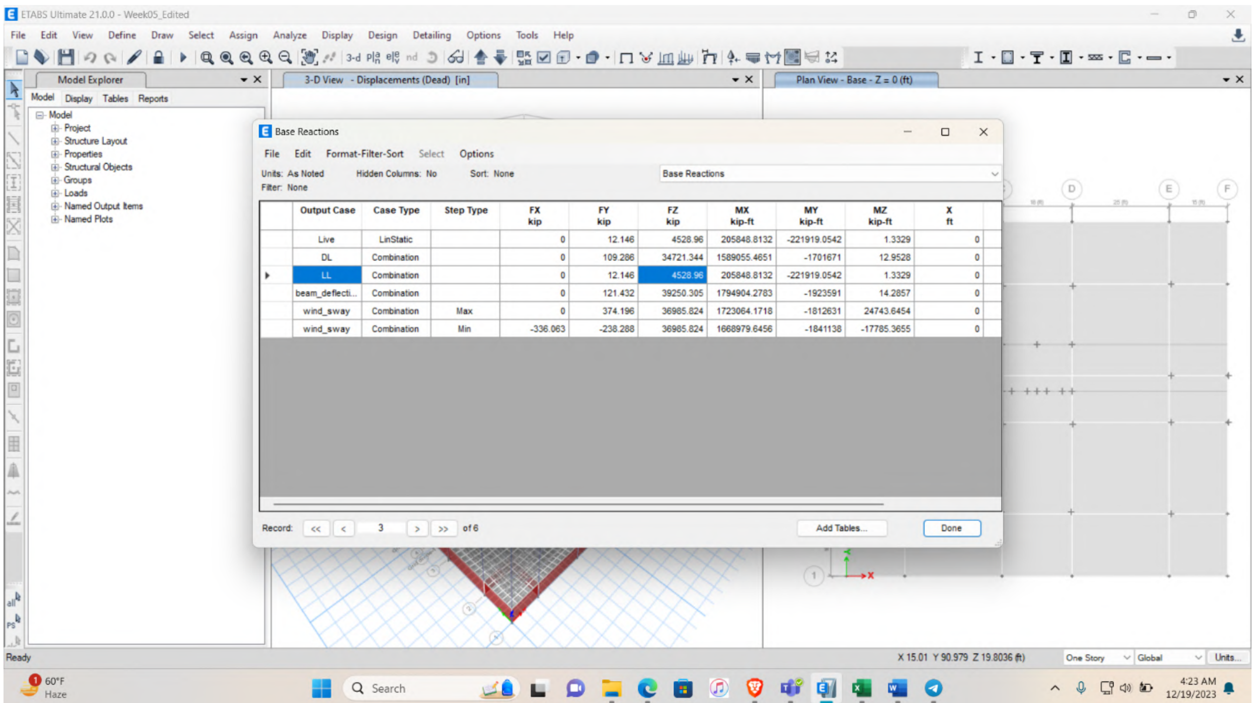
= 0.0478% (OK)

Live Load, Partition Wall, Floor Finish Validation					
Type of Load	**Load (psf)**	**Area (ft^2)**	**Number**	**Stories**	**Total Load (kip)**
LL	40	7280		13	3785.6
LL GF	40	8342		1	333.68
LL OHWT	10	360		1	3.6
LL Stair	100	360		14	504
LL Stair Top	40	360		1	14.4
Total Live Load					**4641.28**
PW1	70	640	4	14	2508.8
PW2	107	352.5	4	14	2112.18
PW3	60	587.5	4	14	1974
PW4	60	300	2	14	504
Total Partition Wall Load					**7098.98**
FF1	25	7640		13	2483
FF2	25	8702		1	217.55
FF3	25	360		1	9
Total Load Floor Finish Load					**2709.55**

LL:

From ETABS,

Base Reaction for LL = 4529 Kip



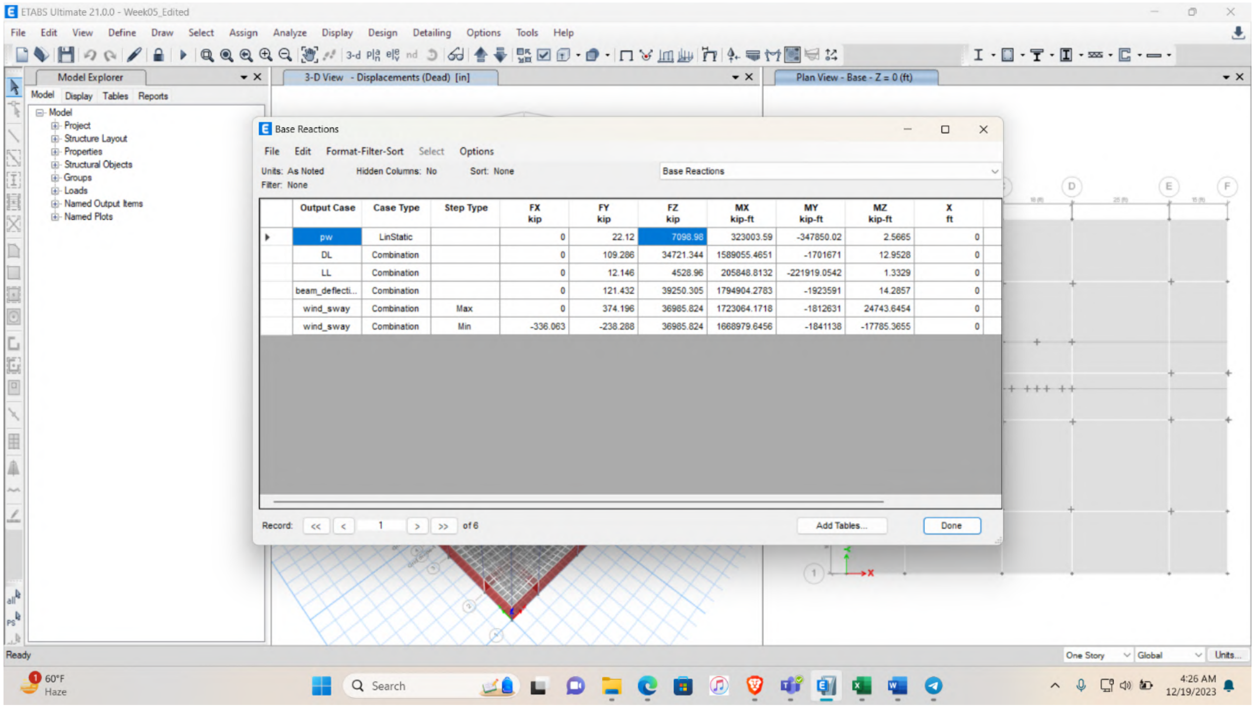
% Error = (4641-4529)/4641*100%

= 2.42% (OK)

PW:

From ETABS,

Base Reaction for PW = 7099 Kip



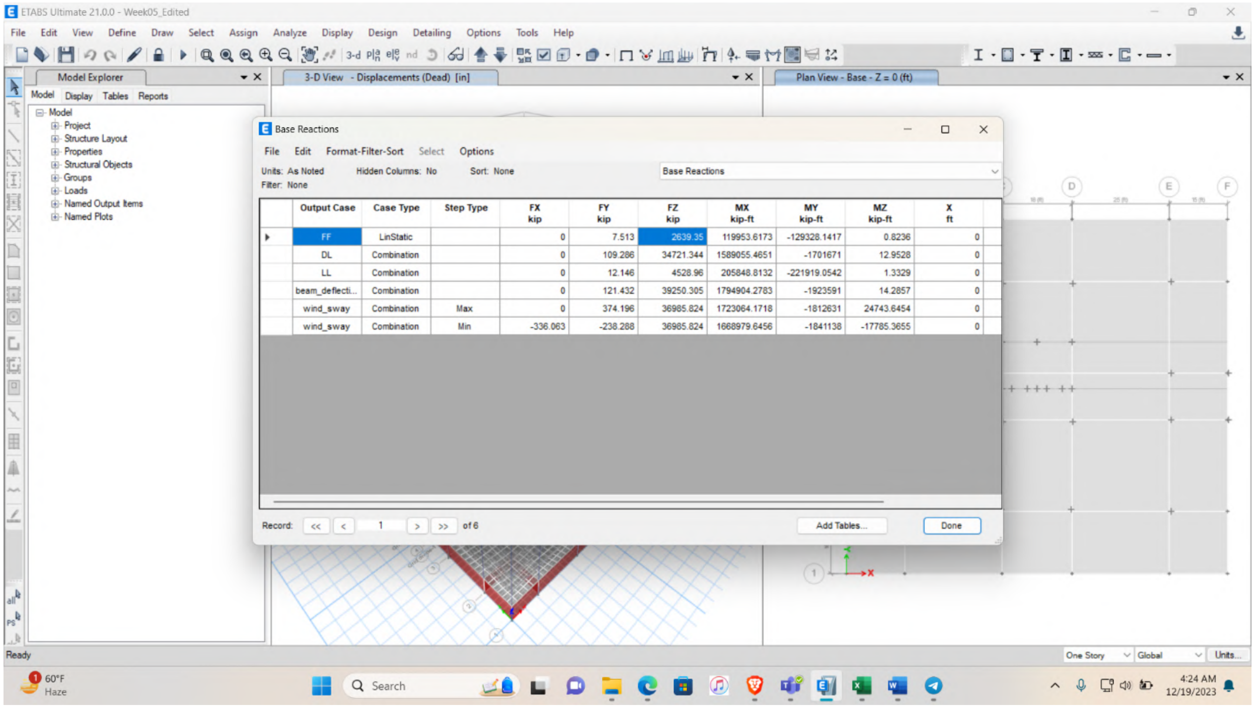
% Error = (7099-7099)/7099*100%

= 0% (OK)

FF:

From ETABS,

Base Reaction for FF= 2639 Kip



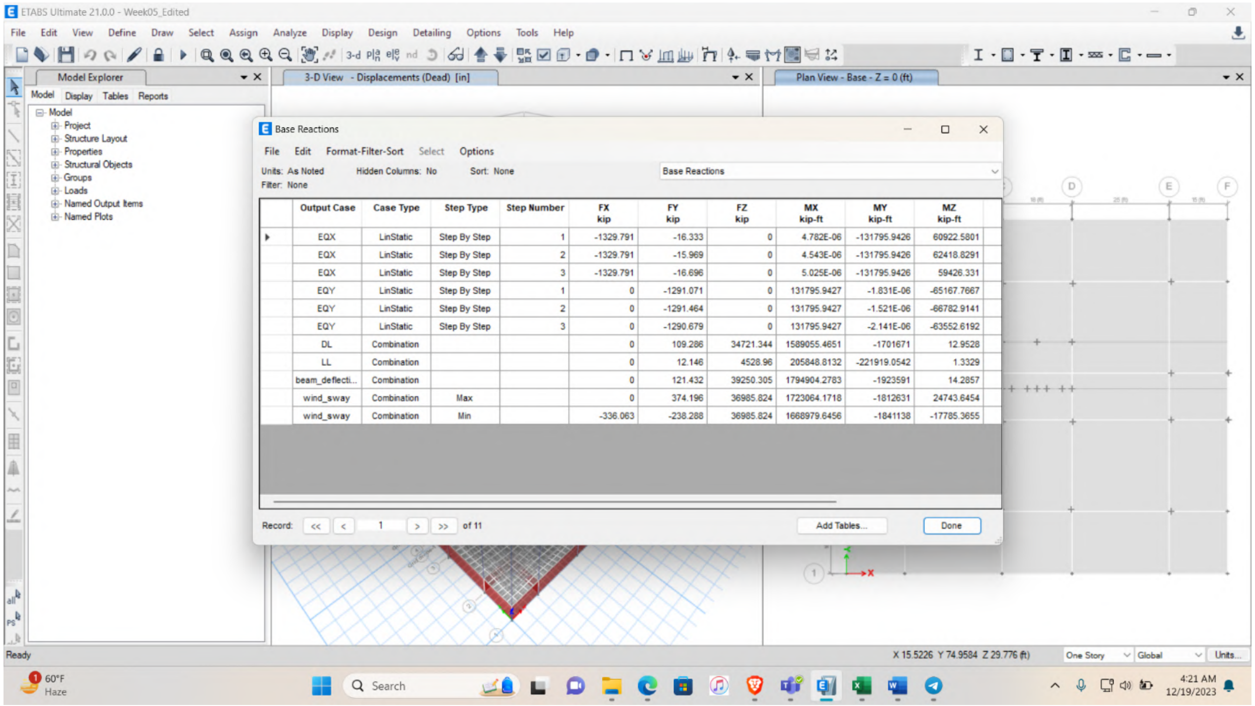
% Error = (2709.5-2639)/2709.5*100%

= 2.6 % (OK)

EQ Load Validation						
Type of Load	**Load (kip)**	**Factor**	**Factored Load**	**Total Load (W)**	**Sa**	**Sa* W (kip)**
DL	27313.38	1	27313.38	28469.2	0.0469	1335.20548
LL	4623.28	0.25	1155.82			

From ETABS,

Base Reaction for EQx= 1330 Kip



Base Reactions

Units: As Noted   Hidden Columns: No   Sort: None
Filter: None

Output Case	Case Type	Step Type	Step Number	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
EQX	LinStatic	Step By Step	1	-1329.791	-16.333	0	4.782E-06	-131795.9426	60922.5801
EQX	LinStatic	Step By Step	2	-1329.791	-15.969	0	4.543E-06	-131795.9426	62418.8291
EQX	LinStatic	Step By Step	3	-1329.791	-16.696	0	5.025E-06	-131795.9426	59426.331
EQY	LinStatic	Step By Step	1	0	-1291.071	0	131795.9427	-1.831E-06	-65167.7667
EQY	LinStatic	Step By Step	2	0	-1291.464	0	131795.9427	-1.521E-06	-66782.9141
EQY	LinStatic	Step By Step	3	0	-1290.679	0	131795.9427	-2.141E-06	-63552.6192
DL	Combination			0	109.286	34721.344	1589055.4651	-1701671	12.9528
LL	Combination			0	12.146	4528.96	205848.8132	-221919.0542	1.3329
beam_deflecti...	Combination			0	121.432	39250.305	1794904.2783	-1923591	14.2857
wind_sway	Combination	Max		0	374.196	36985.824	1723064.1718	-1812631	24743.6454
wind_sway	Combination	Min		-336.063	-238.288	36985.824	1668979.6456	-1841138	-17785.3655

% Error = (1335-1330)/1335*100%

= 0.37 % (OK)

Wind Load Validation: From Design assistant BNBC 2020:

∑ Fx (Total) = 2369 kN	∑ Fy (Windward) = 1232 kN	∑ Fy (Leeward) = 943 kN	∑ Fy (Total) = 2174 kN

Fx=2369*.2248 =532.6 kip

Fy=2174*.2248 =489 kip

From ETABS,

Fx = 535.5 kip

Fy=572 kip

For Fx,

% Error = (535.5-532.6)/535.5*100%

= 0.54% (OK)

WL	LinStatic	Step By Step	1	-535.466
WL	LinStatic	Step By Step	2	0

For Fy,

% Error = (505-489)/505*100%

= 3.16%

WL	LinStatic	Step By Step	2	0	-505.21
WL	LinStatic	Step By Step	3	-360.067	-4.883

DESIGN ANALYSIS

Sample calculation for beam

Beam A1-B1:
Beam size 15’’x24”
Main bar:

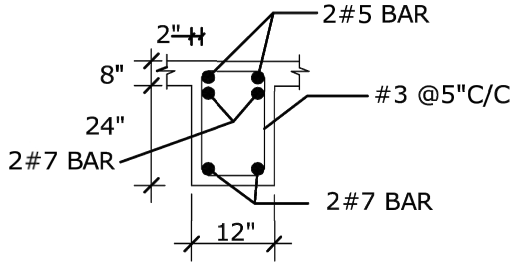
From ETABS,
Top reinforcement, at middle = 0.45 in^2
at corner (left)= 1.18 in^2
at corner (right) = 0.77 in^2
Bottom reinforcement, at middle = 0.57 in^2
at corner (left)= 0.78 in^2
at corner (right) = 0.85 in^2

Reinforcement provided,

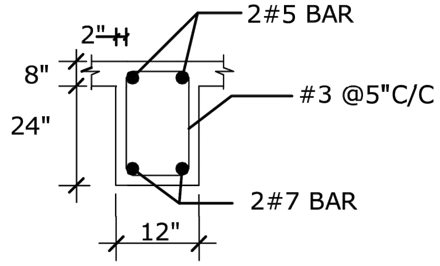
Top reinforcement, at middle = 2#5 bar =0.62 in^2 > 0.45 in^2
at corner (left)= 2#5 bar & 2#6 bar = 1.5 in^2 > 1.18 in^2
at corner (right) = 2#5 bar & 2#6 bar = 1.5 in^2 > 0.77 in^2
Here, L/3 = 15’/3 = 5’, L/4 = 15’/4’ =3.75’

Bottom reinforcement, at middle = 2#7 bar = 1.2 in^2 >0.57 in^2
at corner (left)= 2#7 bar = 1.2 in^2 >0.78 in^2
at corner (right) = 2#7 bar = 1.2 in^2 >0.85 in^2
Here, L/8 = 15’/8 =1.875’

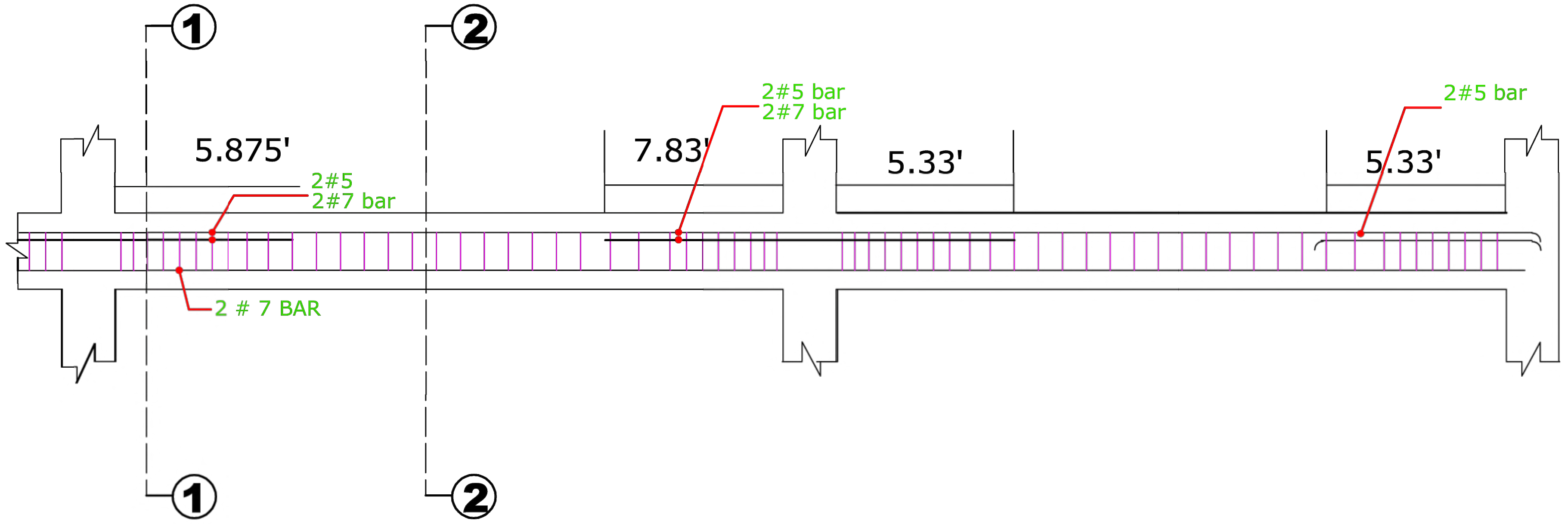
Stirrup:
From ETABS, Reinforcement = 0.33 in^2
Provided, #4 bar = 2* 0.2 in^2 >0.33 in^2
Spacing
d/4 = (24-1.5)/4 = 5.5”
S < 8 d_b (Smallest of long bar) = 8 x 5 / 8 = 5” (Govern)
24 d_b (Stirrup bar) = 24 x 4/ 8 = 9”
12”
So, #4 @ 5”c/c provided.
From all beam-column joint to 2h = 2*24” =44” distance provide #4@5”c/c. rest provide D-#4@10”c/c



Cross Section:1-1



Cross Section:2-2



Beam Detailing For RB-01

Checked and Vetted By:

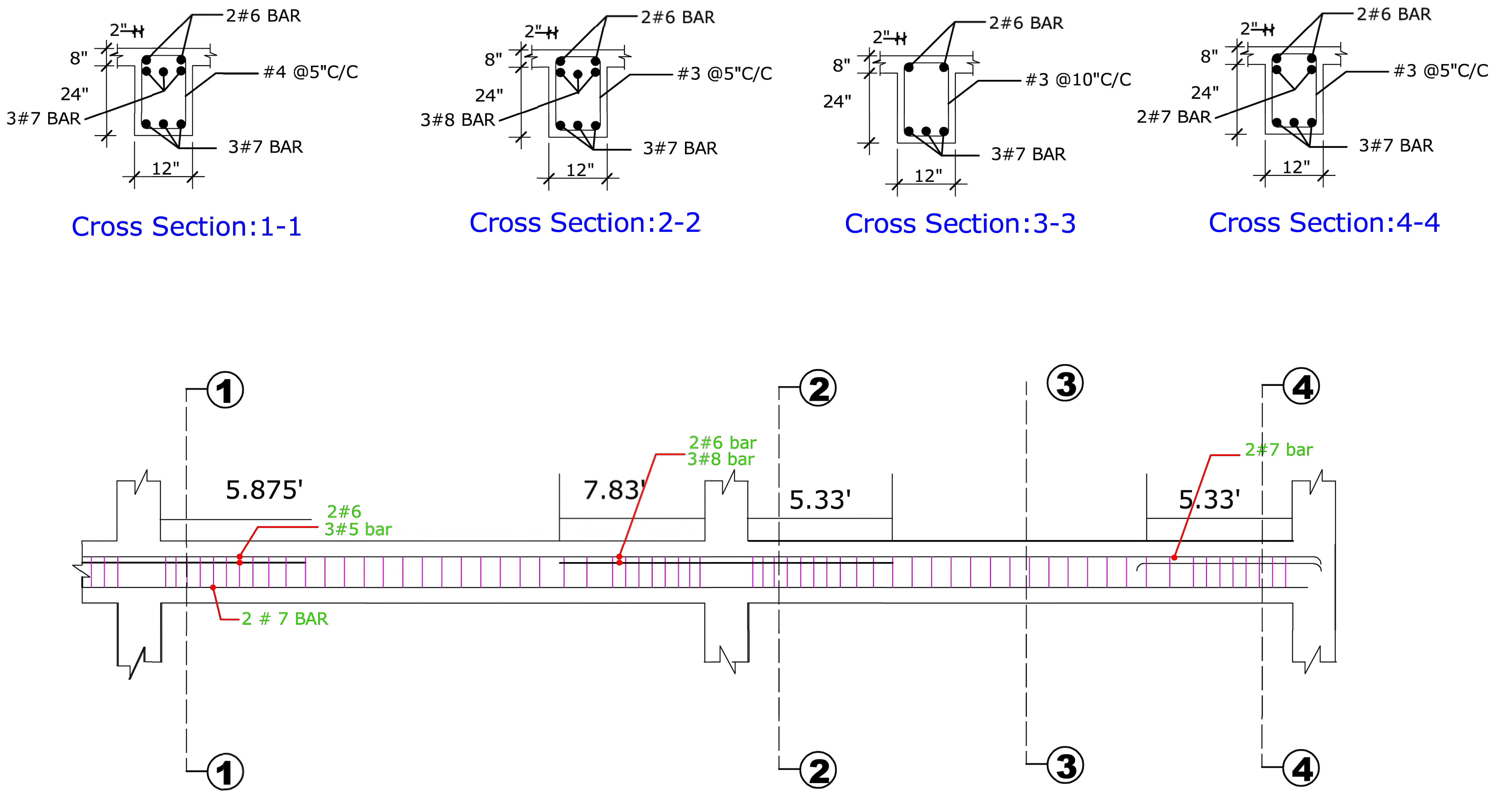
Designed By: Group-2(B1)

Preparedd By: Group-2(B1)

Project: Reconstruction of Green Road Staff Quarter

Client:

Submitted By: Group-2(B1)



Typical Long Sec. Of Beam Details

Beam Detailing For RB-02

Checked and Vetted By:

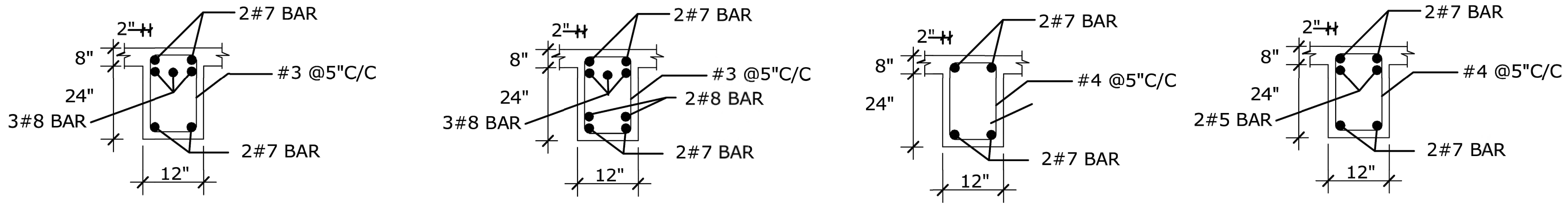
Designed By: Group-2(B1)

Prepared By: Group-2(B1)

Project: Reconstruction of Green Road Staff Quarter

Client:

Submitted By: Group-2(B1)

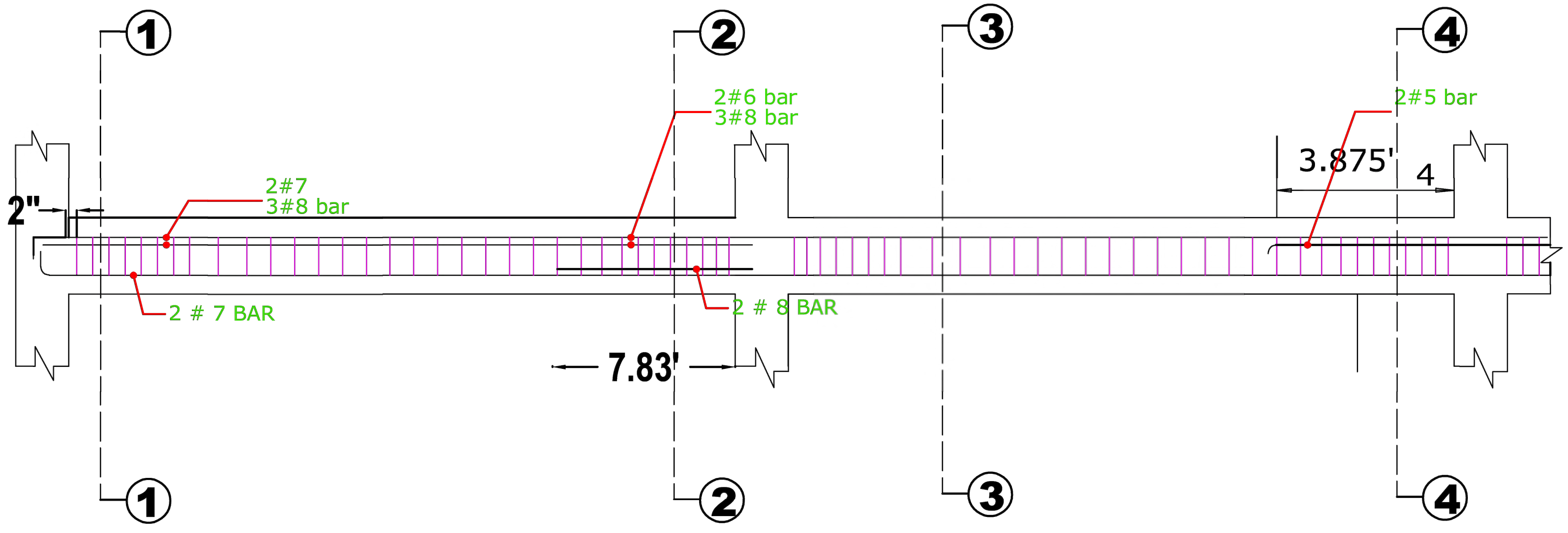


Cross Section:1-1

Cross Section:2-2

Cross Section:3-3

Cross Section:4-4



Typical Long Sec. Of Beam Details

Beam Detailing For RB-03

Checked and Vetted By:

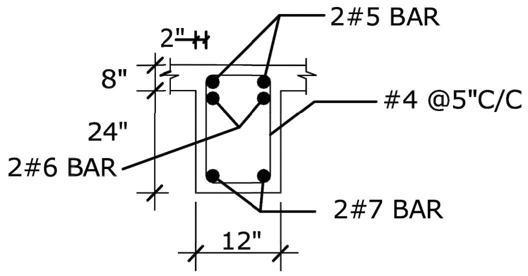
Designed By: Group-2(B1)

Preparedd By: Group-2(B1)

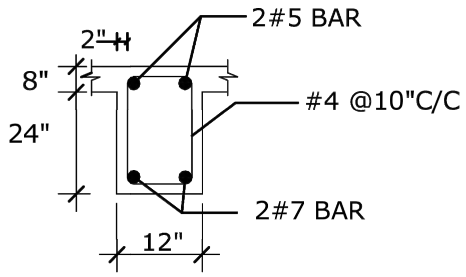
Project: Reconstruction of Green Road Staff Quarter

Client:

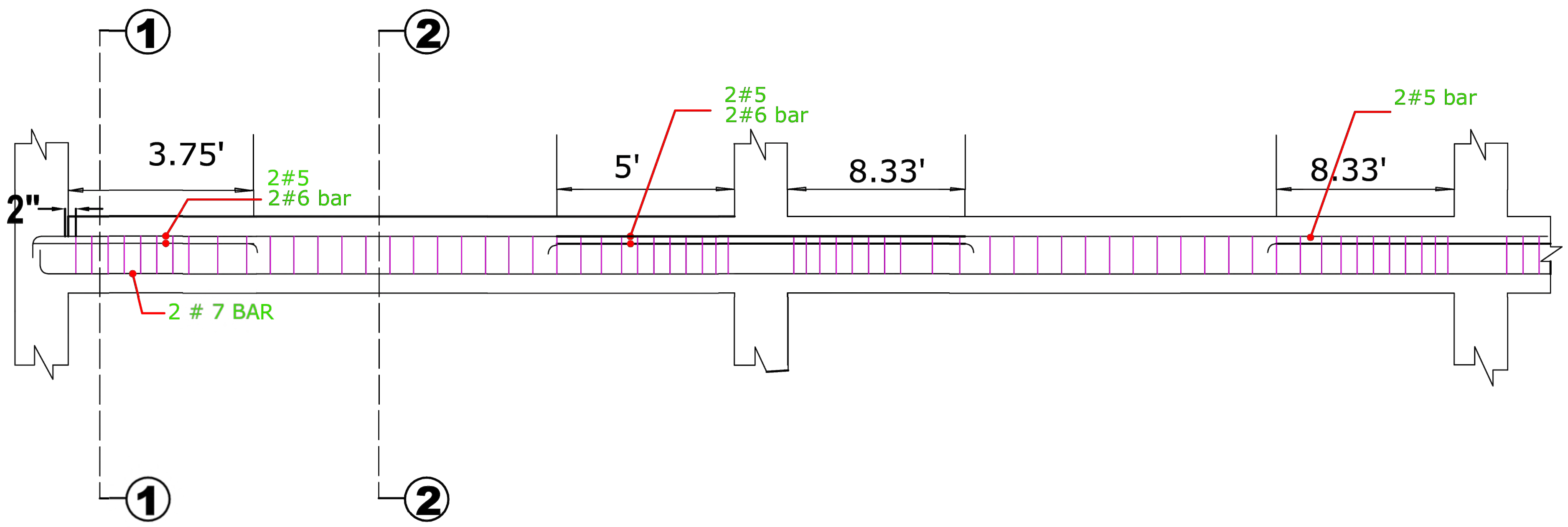
Submitted By: Group-2(B1)



Cross Section:1-1



Cross Section:2-2



Typical Long Sec. Of Beam Details

Beam Detailing For RB-04

Checked and Vetted By:

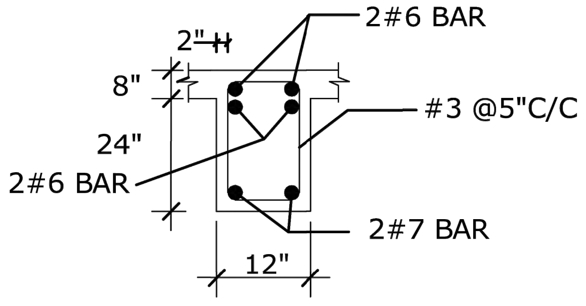
Designed By: Group-2(B1)

Prepared By: Group-2(B1)

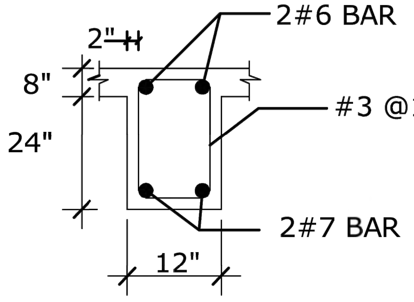
Project: Reconstruction of Green Road Staff Quarter

Client:

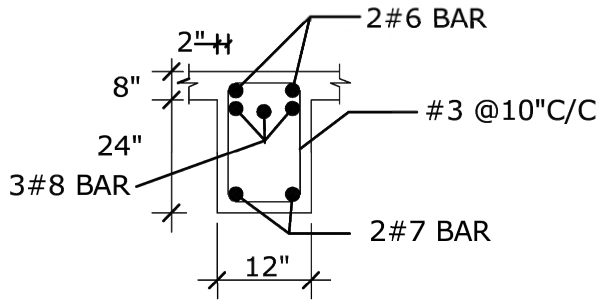
Submitted By: Group-2(B1)



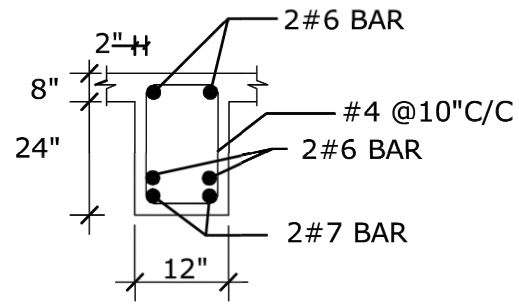
Cross Section:1-1



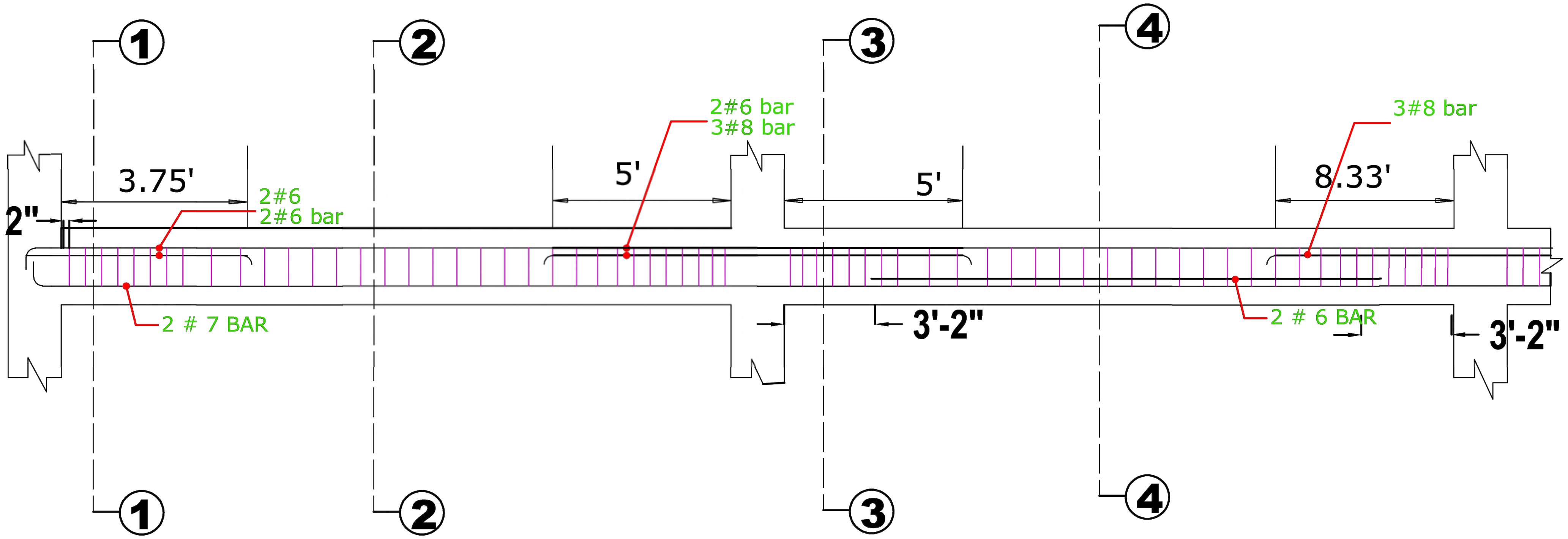
Cross Section:2-2



Cross Section:3-3



Cross Section:4-4



Typical Long Sec. Of Beam Details

Beam Detailing For RB-05

Checked and Vetted By:

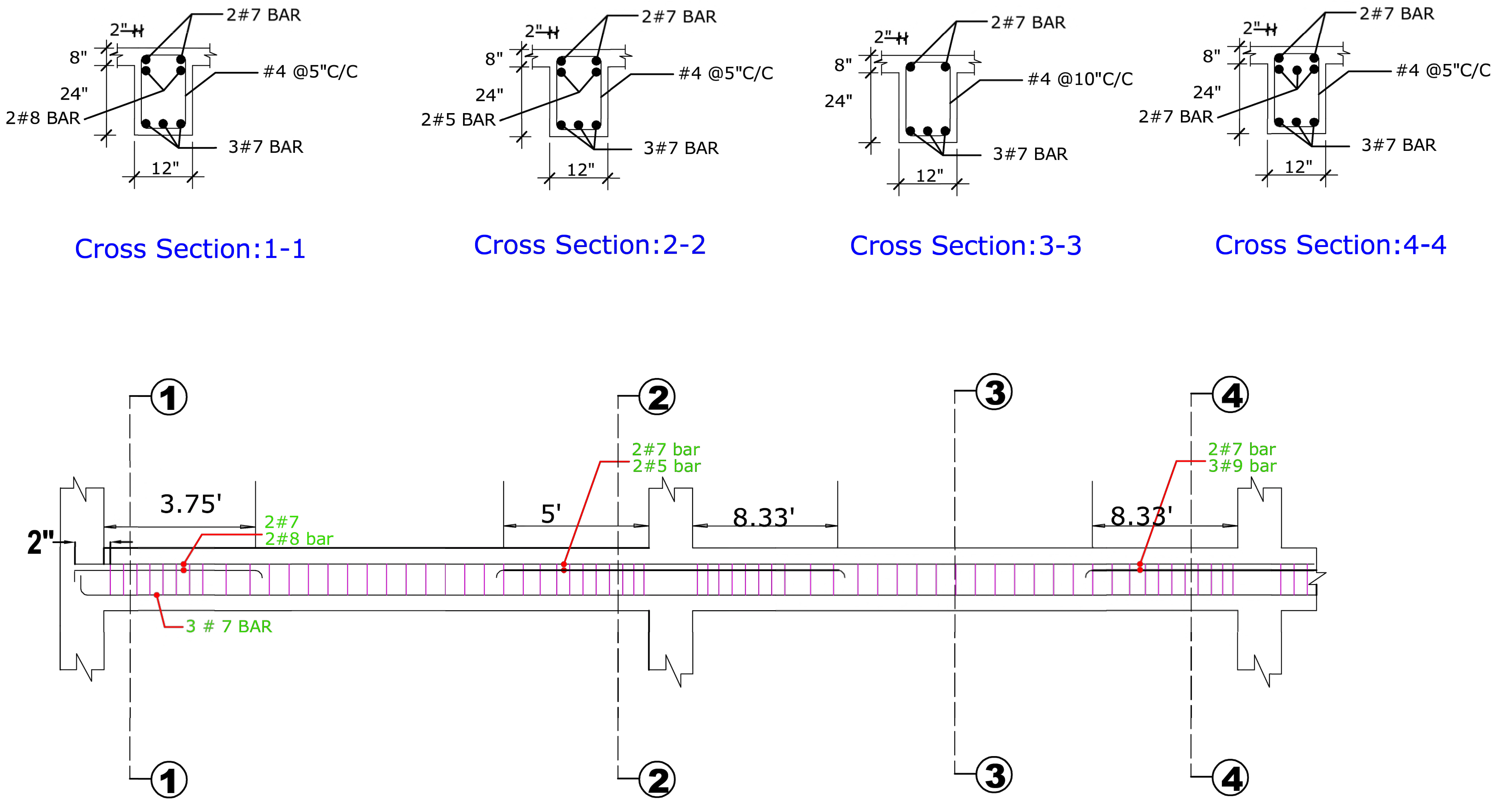
Designed By: Group-2(B1)

Prepared By: Group-2(B1)

Project: Reconstruction of Green Road Staff Quarter

Client:

Submitted By: Group-2(B1)



Typical Long Sec. Of Beam Details

Beam Detailing For RB-06

Checked and Vetted By:	
Designed By:	Group-2(B1)
Preparedd By:	Group-2(B1)
Project:	Reconstruction of Green Road Staff Quarter
Client:	
Submitted By:	Group-2(B1)

Sample calculation for column

Sample Calculation for Exterior Column:

Base-2F:

Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 18 \times 24 = 4.32$ in^2

From ETABS required reinforcement Area $A_{s,\,req} = 12.9$ in^2

Reinforcement Provided--(12 #10 bar),

$A_{s,\,provided} = (1.27 \times 12)$ in^2 = 15.24 in^2

$\rho = A_s/A_g = \frac{15.24}{432} = 3.52\%$ [ok]

Tie Reinforcement:

- Using #3 bar as tie when the long. bar up to #10.

Tie Spacing:

16 d_{long}= 20”

48 d_{tie} = 18”

Least Dimension = 18”

∴ Provided Tie Reinforcement #3 @ 18” c/c

Spacing between two longitudinal Column = $\frac{18-2*3/8-2*1.5-\frac{10}{8}*4}{3}$ =3.1” [ok]

<u>IMRF Requirement for Ties:</u>

l_0
- larger of c1 and c2 = 24”
- clear span / 6 = 20”
- or, 18”

s_0
- 8 x smallest longitudinal bar diameter = 10”
- 24 x tie bar diameter = 9”
- 0.5 x smaller of c1 or c2 = 9”
- 12”

According to IMRF, least spacing between ties can be provided = 9”

2F-11F:

Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 18 \times 24 = 4.32$ in^2

From ETABS required reinforcement Area $A_{s,\,req} = 4.32$ in^2

Reinforcement Provided--(8 #8 bar),

$A_{s,\,provided} = (0.79 \times 8)$ in^2 = 6.32 in^2

$\rho = A_s/A_g = \frac{6.32}{432} = 1.5\%$ [ok]

Tie Reinforcement:

- Using #3 bar as tie when the long. bar up to #10.

Tie Spacing:

16 d_{long}= 16”

48 d_{tie} = 18”

Least Dimension = 18”

∴ Provided Tie Reinforcement #3 @ 16” c/c

Spacing between two longitudinal Column = $\frac{24-2*3/8-2*1.5-\frac{8}{8}*4}{3}$ =5.41” [ok]

<u>IMRF Requirement for Ties:</u>

larger of c1 and c2 = 24”

l_0 clear span / 6 = 20”

or, 18”

8 x smallest longitudinal bar diameter = 8”

s_0 24 x tie bar diameter = 9”

0.5 x smaller of c1 or c2 = 9”

12”

According to IMRF, least spacing between ties can be provided = 8”

11F-12F:

Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 18 \times 24 = 4.32$ in^2

From ETABS required reinforcement Area $A_{s,\,req}$ = 7.2 in^2

Reinforcement Provided--(8 #9 bar),

$A_{s,\,provided}$ = (1 ×8) in^2 = 8 in^2

$\rho = A_s/A_g = \frac{8}{432}$ = 1.85% [ok]

Tie Reinforcement:

- Using #3 bar as tie when the long. bar up to #10.

Tie Spacing:

16 d_{long}= 18”

48 d_{tie} = 18”

Least Dimension = 18”

∴ Provided Tie Reinforcement #3 @ 18” c/c

Spacing between two longitudinal Column = $\frac{24-2*3/8-2*1.5-\frac{9}{8}*4}{3}$ =5.25” [ok]

<u>IMRF Requirement for Ties:</u>

larger of c1 and c2 = 24”

l_0 clear span / 6 = 20”

or,   18”

8 x smallest longitudinal bar diameter   = 9”
s_0 24 x tie bar diameter   = 9”
0.5 x smaller of c1 or c2   = 9”
12”

According to IMRF, least spacing between ties can be provided = 9”

Sample Calculation for Column 2 Interior Column Except C3, D3:
Base-2F:
Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 20 \times 30 = 6$ in^2

From ETABS required reinforcement Area $A_{s, req} = 22.46$ in^2

Reinforcement Provided--(10 #14 bar),

$A_{s, provided} = (2.250 \times 10)$ in^2 = 22.5 in^2

$\rho = A_s/A_g = \frac{22.5}{600} = 3.75\%$ [ok]

Tie Reinforcement:

- Using #4 bar as tie when the long. Bar > #10.

Tie Spacing:

16 d_{long}= 28”
48 d_{tie} = 24”
Least Dimension = 20”

∴ Provided Tie Reinforcement #4 @ 20” c/c

Spacing between two longitudinal Column (Smaller direction) = $\frac{20-2*3/8-2*1.5-\frac{14}{8}*3}{2}$ =5.5” [ok]

Spacing between two longitudinal Column (Longer direction) = $\frac{30-2*3/8-2*1.5-\frac{14}{8}*4}{3}$ =6.4” [not ok]

<u>IMRF Requirement for Ties:</u>

larger of c1 and c2 = 30”
l_0 clear span / 6   = 20”
or,   18”

8 x smallest longitudinal bar diameter   = 14”
s_0 24 x tie bar diameter   = 12”
0.5 x smaller of c1 or c2   = 10”
12”

According to IMRF, least spacing between ties can be provided = 10”

2F-11F

Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 20 \times 30 = 6.0$ in^2

From ETABS required reinforcement Area $A_{s,\,req} = 6.0$ in^2

Reinforcement Provided--(8 #8 bar),

$A_{s,\,provided} = (0.79 \times 8)$ in^2 = 6.32 in^2

$\rho = A_s/A_g = \frac{6.32}{600} = 1.1\%$ [ok]

Tie Reinforcement:

- Using #3 bar as tie when the long. bar up to #10.

Tie Spacing:

16 d_{long}= 16”

48 d_{tie} = 18”

Least Dimension = 20”

∴ Provided Tie Reinforcement #3 @ 16” c/cot

Spacing between two longitudinal Column = $\frac{30-2*3/8-2*1.5-\frac{8}{8}*4}{3}$ =7.4” [Not ok]

IMRF Requirement for Ties:

l_0 — larger of c1 and c2 = 30”; clear span / 6 = 20”; or, 18”

s_0 — 8 x smallest longitudinal bar diameter = 8”; 24 x tie bar diameter = 9”; 0.5 x smaller of c1 or c2 = 9”; 12”

According to IMRF, least spacing between ties can be provided = 8”

11F-12F

Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 20 \times 30 = 6$ in^2

From ETABS required reinforcement Area $A_{s,\,req} = 9.88$ in^2

Reinforcement Provided--(8 #10 bar),

$A_{s,\,provided} = (1.27 \times 8)$ in^2 = 10.16 in^2

$\rho = A_s/A_g = \frac{10.16}{600} = 1.69\%$ [ok]

Tie Reinforcement:

- Using #3 bar as tie when the long. bar up to #10.

Tie Spacing:

16 d_{long}= 20”

48 d_{tie} = 18”

Least Dimension = 20”

∴ Provided Tie Reinforcement #3 @ 18” c/c

Spacing between two longitudinal Column = $\frac{30-2*3/8-2*1.5-\frac{10}{8}*4}{3}$ =7.1” [not ok]

<u>IMRF Requirement for Ties:</u>

	larger of c1 and c2	= 30”
l_0	clear span / 6	= 20”
	or,	18”

	8 x smallest longitudinal bar diameter	= 10”
s_0	24 x tie bar diameter	= 9”
	0.5 x smaller of c1 or c2	= 10”
		12”

According to IMRF, least spacing between ties can be provided = 9”

Sample Calculation for Column C3, D3

Base-5F

Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 20 \times 30 = 6\ in^2$

From ETABS required reinforcement Area $A_{s,\ req} = 12.45\ in^2$

Reinforcement Provided--(10 #10 bar),

$A_{s,\ provided} = (1.27 \times 10)\ in^2 = 12.7\ in^2$

$\rho = A_s/A_g = \frac{12.7}{600} = 2.1\%$ [ok]

Tie Reinforcement:

- Using #3 bar as tie when the long. Bar > #10.

Tie Spacing:

16 d_{long}= 20”

48 d_{tie} = 18”

Least Dimension = 20”

∴ Provided Tie Reinforcement #3 @ 18” c/c

Spacing between two longitudinal Column (Smaller direction) = $\frac{20-2*3/8-2*1.5-\frac{10}{8}*3}{2}$ 6.25” [not ok]

Spacing between two longitudinal Column (Longer direction) = $\frac{30-2*3/8-2*1.5-\frac{10}{8}*4}{3}$ =7.1” [not ok]

IMRF Requirement for Ties:

larger of c1 and c2 = 30"

l_0 clear span / 6 = 20"

or, 18"

8 x smallest longitudinal bar diameter = 10"

s_0 24 x tie bar diameter = 9"

0.5 x smaller of c1 or c2 = 10"

12"

According to IMRF, least spacing between ties can be provided = 9"

5f-11F:

Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 20 \times 30 = 6.0$ in^2

From ETABS required reinforcement Area $A_{s,\,req} = 8.33$ in^2

Reinforcement Provided--(10 #9 bar),

$A_{s,\,provided} = (1 \times 9)$ in^2 = 9.0 in^2

$\rho = A_s/A_g = \frac{9.0}{600} = 1.5\%$ [ok]

Tie Reinforcement:

- Using #3 bar as tie when the long. bar up to #10.

Tie Spacing:

$16\ d_{long} = 18"$

$48\ d_{tie} = 18"$

Least Dimension = 20"

∴ Provided Tie Reinforcement #3 @ 18" c/c

Spacing between two longitudinal Column in longer dimension = $\frac{30-2*3/8-2*1.5-\frac{9}{8}*4}{3}$ =7.25" [Not ok]

Spacing between two longitudinal Column in smaller dimension = $\frac{30-2*3/8-2*1.5-\frac{9}{8}*4}{3}$ =6.4" [Not ok]

IMRF Requirement for Ties:

larger of c1 and c2 = 30"

l_0 clear span / 6 = 20"

or, 18"

8 x smallest longitudinal bar diameter = 9"

s_0 24 x tie bar diameter = 9"

0.5 x smaller of c1 or c2 = 9”
12”

According to IMRF, least spacing between ties can be provided = 9”

11F-12F:

Longitudinal Reinforcement:

$A_{s,min} = \rho \times A_g = 0.01 \times 20 \times 30 = 6\ in^2$

From ETABS required reinforcement Area $A_{s,\ req} = 12.3\ in^2$

Reinforcement Provided--(10 #10 bar),

$A_{s,\ provided} = (1.27 \times 10)\ in^2 = 12.7\ in^2$

$\rho = A_s/A_g = \frac{12.7}{600} = 2.1\%$ [ok]

Tie Reinforcement:

- Using #3 bar as tie when the long. Bar > #10.

Tie Spacing:

$16\ d_{long} = 20”$

$48\ d_{tie} = 18”$

Least Dimension = 20”

∴ Provided Tie Reinforcement #3 @ 18” c/c

Spacing between two longitudinal Column (Smaller direction) = $\frac{20-2*3/8-2*1.5-\frac{10}{8}*3}{2}$ 6.25” [not ok]

Spacing between two longitudinal Column (Longer direction) = $\frac{30-2*3/8-2*1.5-\frac{10}{8}*4}{3}$ =7.1” [not ok]

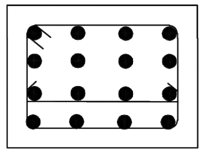
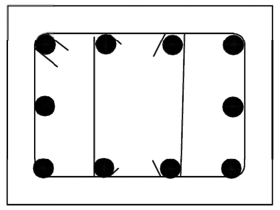
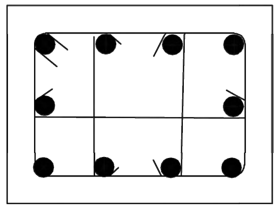
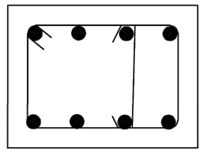
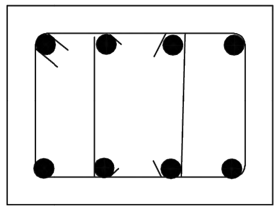
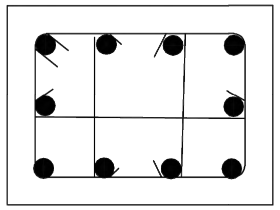
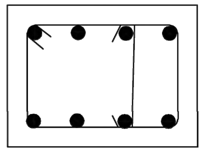
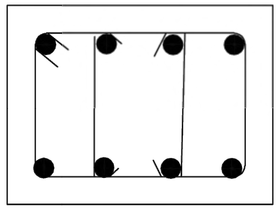
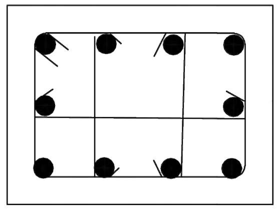
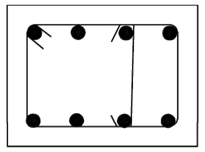
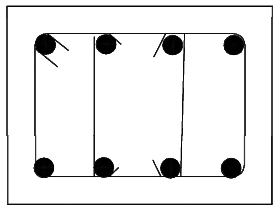
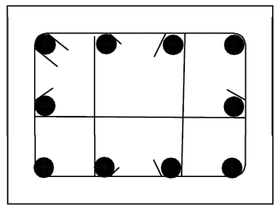
<u>IMRF Requirement for Ties:</u>

larger of c1 and c2 = 30”
l_0 clear span / 6 = 20”
or, 18”

8 x smallest longitudinal bar diameter = 10”
s_0 24 x tie bar diameter = 9”
0.5 x smaller of c1 or c2 = 10”
12”

According to IMRF, least spacing between ties can be provided = 9”

COLUMN CHART:-

FLOOR	C_1	C_2	C_3
	18"x24"	20"x30"	20"x30"
BASE TO 2F	 12#10 BAR tie bar:#3 @ 18" C/C	 10#10 BAR tie bar:#3 @ 18" C/C	 10#10 BAR tie bar:#3 @ 18" C/C
3F to 5F	 8#8 BAR tie bar:#3 @ 16" C/C	 8#8 BAR tie bar:#3 @ 16" C/C	 10#10 BAR tie bar:#3 @ 18" C/C
6F to 10F	 8#8 BAR tie bar:#3 @ 16" C/C	 8#8 BAR tie bar:#3 @ 16" C/C	 10#9 BAR tie bar:#3 @ 18" C/C
11F to 12F	 8#9 BAR tie bar:#3 @ 18" C/C	 8#10 BAR tie bar:#3 @ 18" C/C	 10#10BAR tie bar:#3 @ 18" C/C
CLEAR COVER	1.5''	1.5''	1.5''
HOOK	3"	3"	3"

Checked and Vetted By:	
Designed By:	Group-2(B1)
Preparedd By:	Group-2(B1)
Project:	Reconstruction of Green Road Staff Quarter
Client:	
Submitted By:	Group-2(B1)

Sample calculation for Stair

Clear Span of the stair, L_e= (15''/2) + 7.5' + 5'
= 13.75'

Average thickness of the stair slab, t_{avg} = [8/ {cos($\tan^{-1}$ 6/12)}]+ 6/2
= 12"

So, Self-weight of the slab = t_{avg} * Υ_{rcc} = 12/12 * 150
= 150 pcf/ foot width

Here,

F_c' = 4ksi

F_y = 60 ksi

Live Load, LL = 100 psf

Covering Material Self Weight = 25 psf

Dead Load on Stair, DL= 150+25 = 175 psf/ foot width

Factored Load, Wu = 1.2DL + 1.6LL
=1.2*175+1.6*100
= 370 psf

Maximum Moment, M_u= (370*13.75^2)/9 kip-in
=7.8 kip-ft

Temperature and Shrinkage Reinforcement:

$A_{s(min)}$ = 0.002bt
=0.002*12*8
=0.192 in^2/ft width

Maximum Spacing, S_{max} = Min (5t_{Slab} or 18'')
= Min (40'' or 18'')
=18''

Using #3 bars as temperature and shrinkage reinforcement,

S_{req} = 12*.11/.192
= 6.5'' < S_{max}

So, $A_{s\,(temperature\ \&\ Shrinkage)}$ = #3@ 6.5'' c/c

Main Reinforcement:

$$A_{s(main)} = \frac{7.8*12}{0.9*60*(7-\frac{a}{2})} = 0.25'' \text{ when } a = \frac{As(main)*60}{0.85*4*12} = 0.37''$$

$A_{s(main)} > A_{s(Temperature\ \&\ Shrinkage)}$

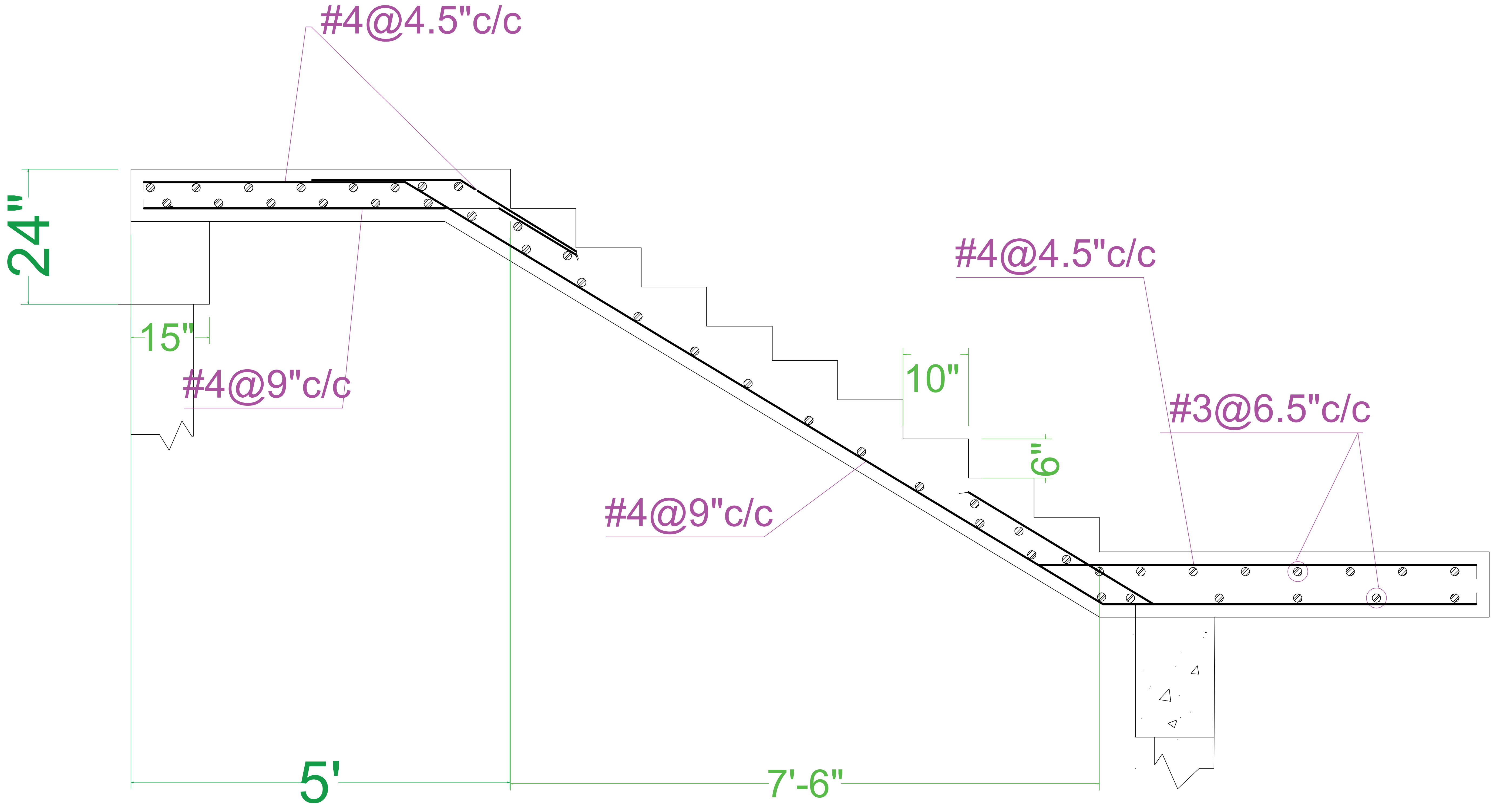
S_{max} = Min (3t_{slab} or 18'')
=Min (24'' or 18'')
=18''

Using #4 bar as main reinforcement

$S_{req} = 12*\frac{12*0.2}{0.25}$

$= 9'' \text{ c/c}$

So, Main Bar would be #4 @ 9''c/c



Submitted By:	Group-2(B1)
Client:	
Project:	Reconstruction of Green Road Staff Quarter
Preparedd By:	Group-2(B1)
Designed By:	Group-2(B1)
Checked and Vetted By:	

Slab design calculation:

Minimum reinforcement requirement:

As_{min} = 0.0018bt

= 0.0018*12*7

= 0.1512 in^2/ft

½ of As_{min} = 0.0756 in^2/ft

s_{min} = 3”

s_{max} = (min^m of 2h or 18”)

= (min^m of 2*7=14” or 18”)

= 14”

For #5 bar, required spacing, s = (12*0.31)/0.0756

= 49” > s_{max} (not ok)

So, taking spacing = s_{max} = 14”

For every 14” spacing, reinforcement requirement = 0.31 in^2/ft

So, for every 12” spacing, reinforcement requirement = (0.31*12)/14

= 0.27 in^2/ft

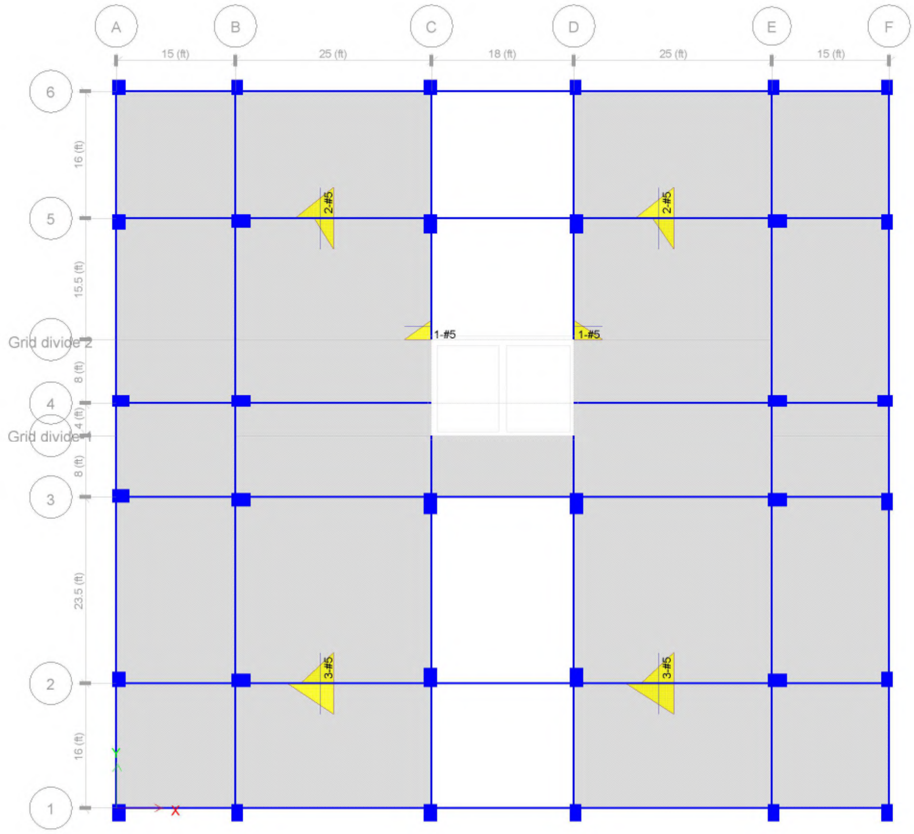
Other requirements of reinforcement are obtained from ETABS analysis.

Slab (x-direction) =>

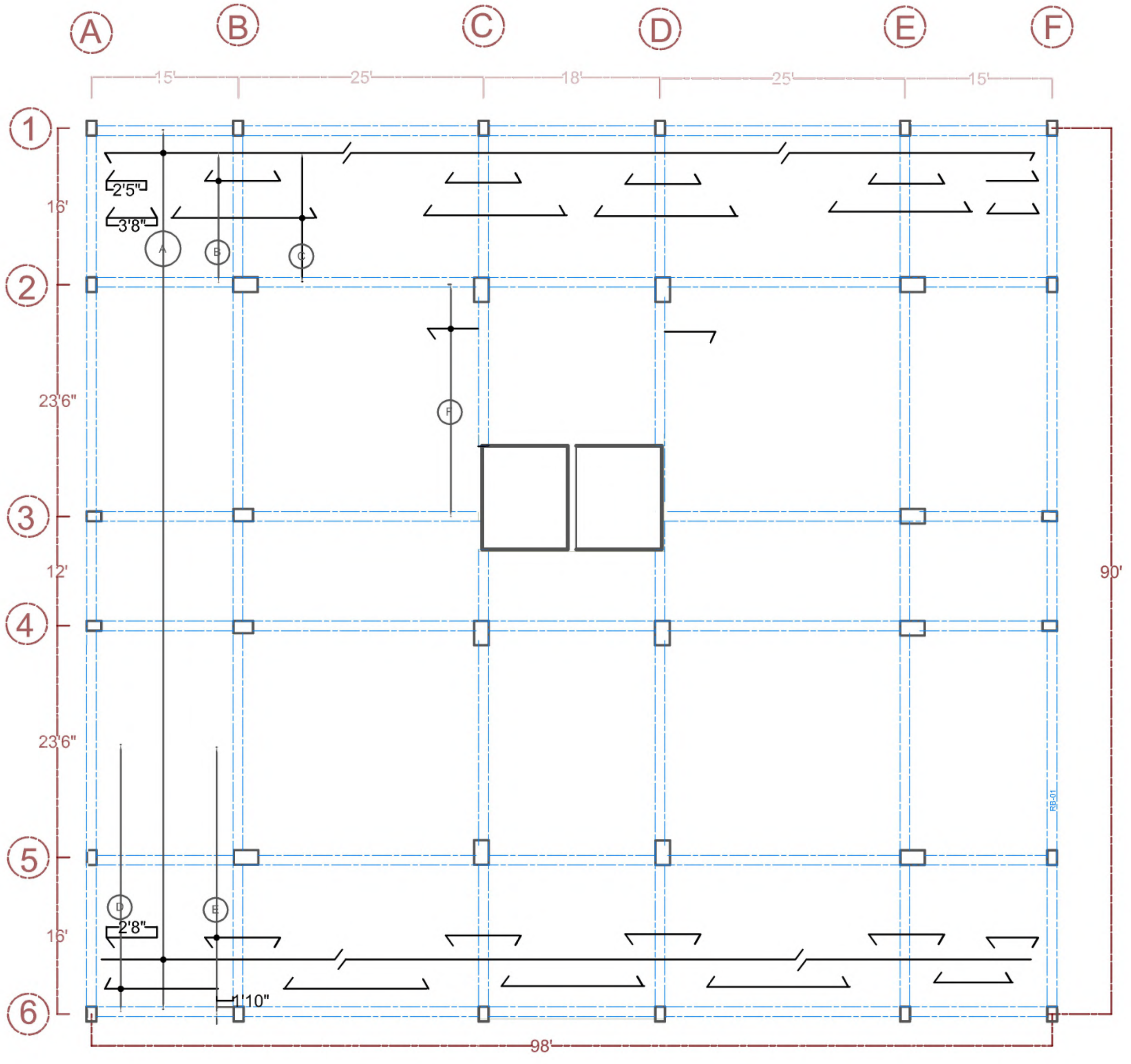
(A) 1#5 per feet bottom bars for both CS and MS
(B) 1#5 per feet top bars for CS
(C) 1#5 per feet top bars for CS
(D) 1#5 per feet bottom bars for MS
(E) 1#5 per feet top bars for MS
(F) 1#5 top bar for MS

Slab (y-direction) =>

(A) 1#5 per feet bottom bars for both CS and MS
(B) 1#5 per feet top bars for CS
(C) 1#5 per feet top bars for CS
(D) 1#5 per feet top bars for MS
(E) 1#5 per feet bottom bars for MS
(F) 2#5 top bar for MS
(G) 3#5 top bar for MS



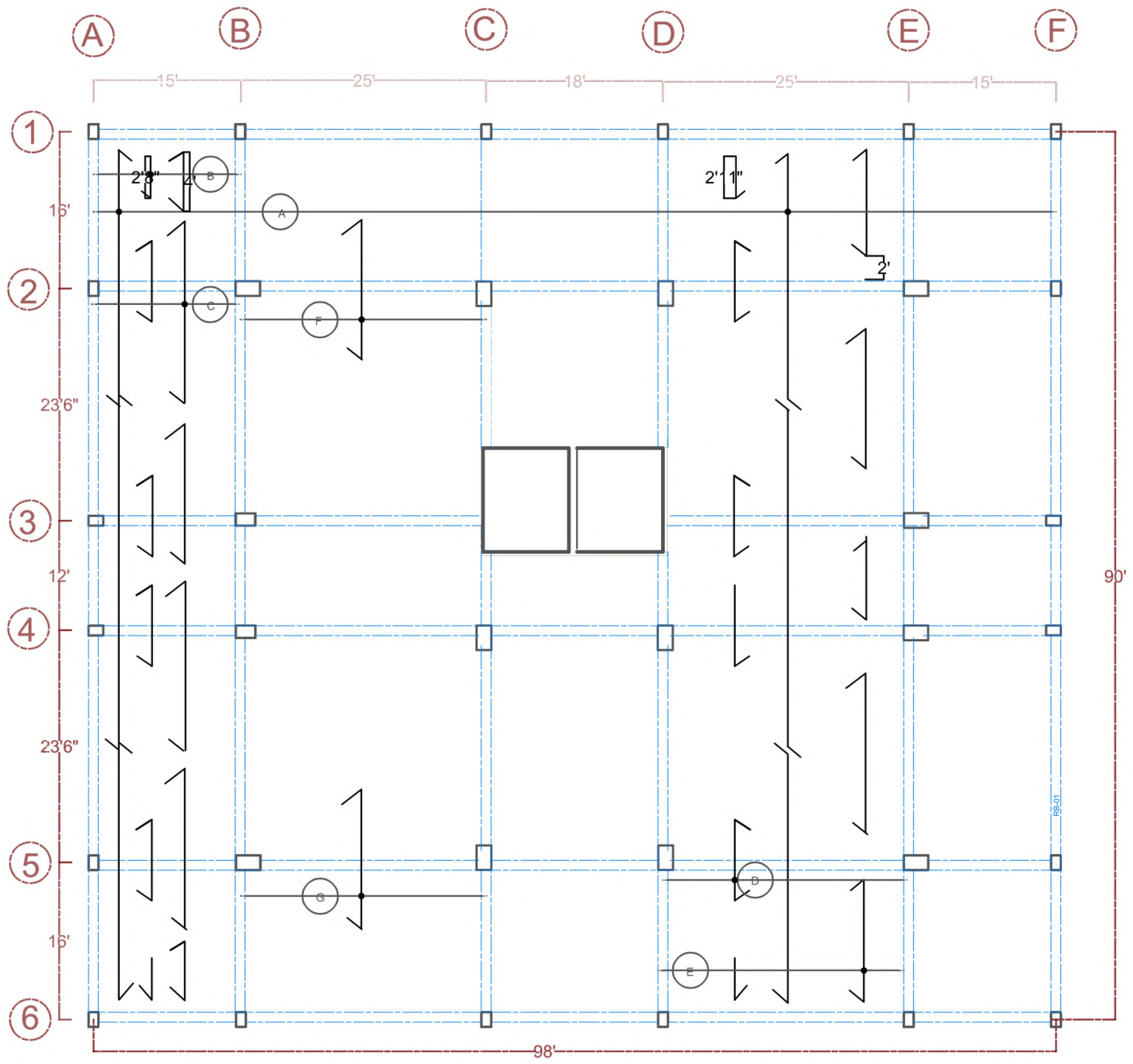
Strip Design - Level 07A_B - Top and Bottom Reinforcement (Enveloping Flexural) - Additional to #5



Slab reinforcement X Direction

(A)	#4 straight bar @6" c/c for both CS and MS
(B)	#4 extra top @6" c/c for CS
(C)	#4 extra top @6" c/c for CS
(D)	#4 extra bottom @6" c/c for MS
(E)	#4 extra top @6" c/c for MS

Checked and Vetted By:	
Designed By:	Group-2(B1)
Prepared By:	Group-2(B1)
Project:	Reconstruction of Green Road Staff Quarter
Client:	
Submitted By:	Group-2(B1)



Slab reinforcement Y Direction

(A)	#4 straight bar @6" c/c for both CS and MS
(B)	#4 extra top @6" c/c for CS
(C)	#4 extra top @6" c/c for CS
(D)	#4 extra bottom @6" c/c for MS
(E)	#4 extra top @6" c/c for MS

Checked and Vetted By:	
Designed By:	Group-2(B1)
Prepared By:	Group-2(B1)
Project:	Reconstruction of Green Road Staff Quarter
Client:	
Submitted By:	Group-2(B1)

Mat Foundation Design:

The thickness of the mat foundation=41”

>As the mat foundation behaves like an inverted flat panel, positive reinforcement will be required at the support position and negative reinforcement at mid-span.

>Min reinforcement required=0.0018*b*t

=.0018*12*41 in^2/ft

=.89 in^2/ft

In X-Direction:

Providing # 8 bar @ 8” c/c

Now in some panels, the reinforcement requirement is not completely fulfilled. So, we will provide extra top and extra bottom bars as required.

Panel AB43:

1. #10 bar @8”c/c-extra top bar is provided
2. #10 bar @10”c/c-extra bottom bar is provided

Panel EF43:

1. #10 bar @8”c/c-extra top bar is provided
2. #10 bar @10”c/c-extra bottom bar is provided

Along Y-Direction:

Providing # 8 bar @ 8” c/c

Now in some panels, the reinforcement requirement is not completely fulfilled. So, we will provide extra top and extra bottom bars as required.

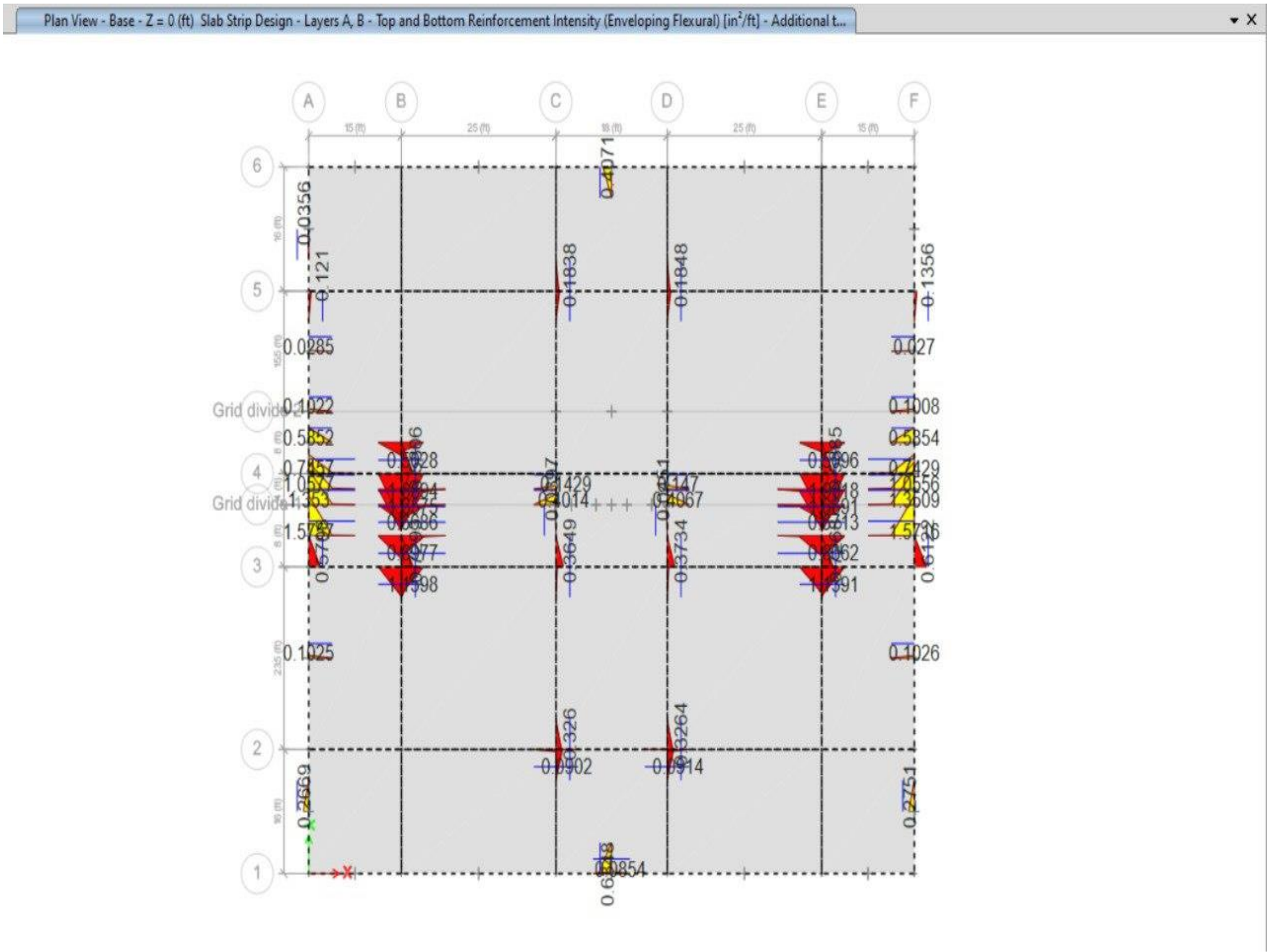
Panel AB23:

1. #7 bar @ 12”c/c-extra bottom bar is provided

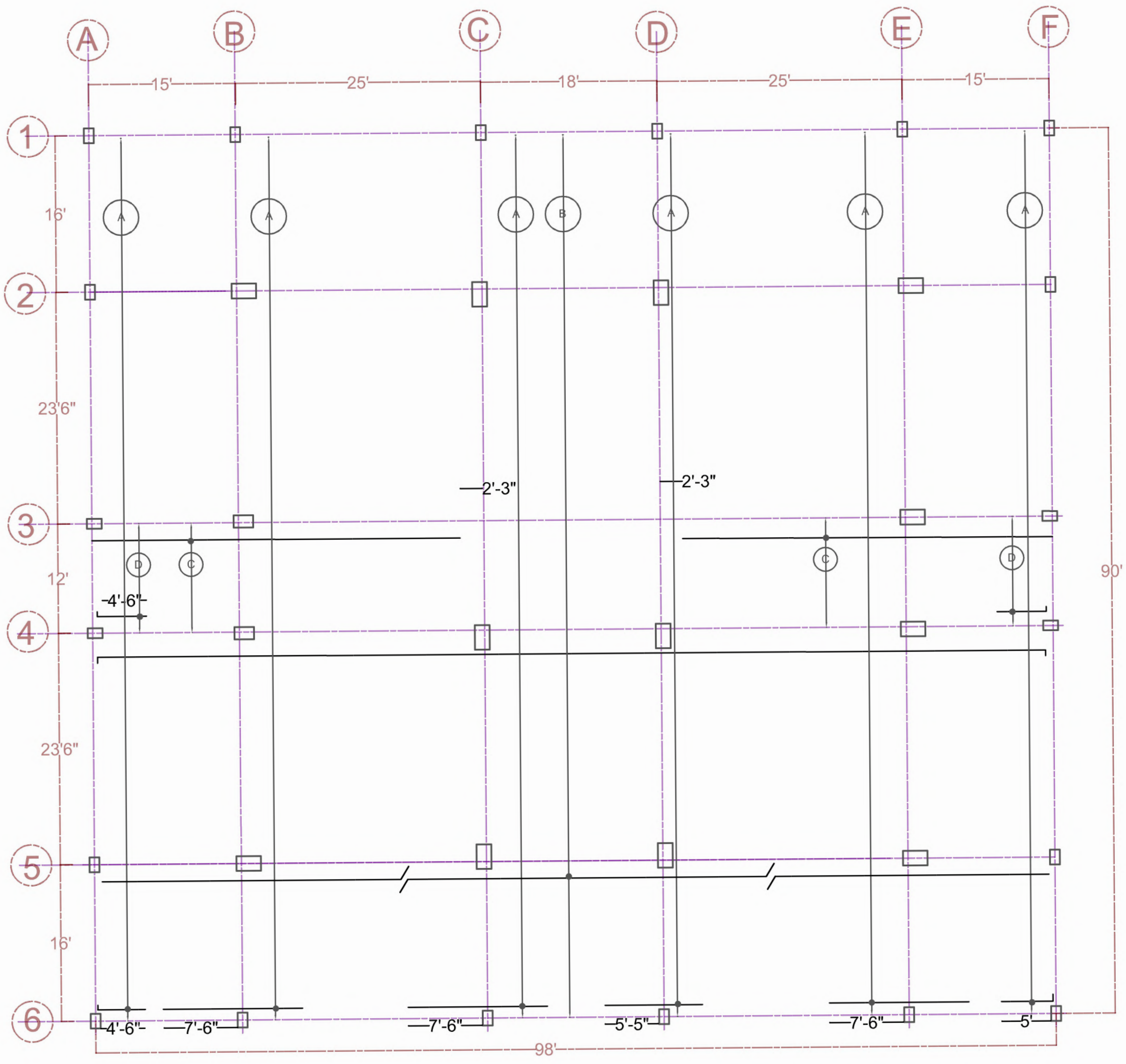
Panel BC23:

1. #7 bar @10”c/c-extra bottom bar is provided

Bar cut-off lengths are shown in the figure.



<u>Figure:</u> Reinforcement amount from ETABS



(A) #8 @ 8” c/c - bottom

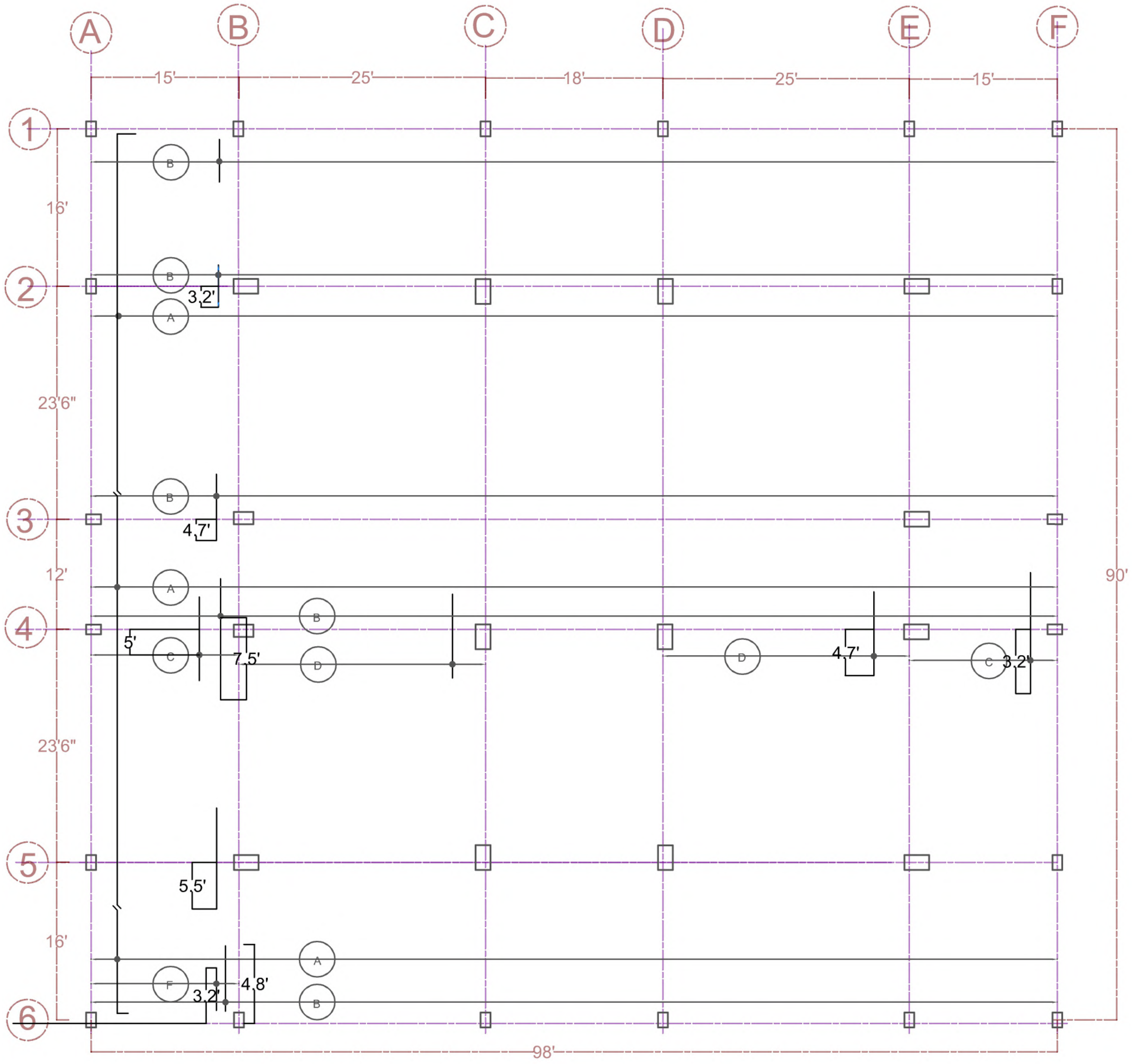
(B) #8 @ 8” c/c - top

(C) #10 @ 8” c/c - extra top

(D) #10 @ 10” c/c - extra bottom

Mat foundation Layout X direction

Checked and Vetted By:	
Designed By:	Group-2(B1)
Prepared By:	Group-2(B1)
Project:	Reconstruction of Green Road Staff Quarter
Client:	
Submitted By:	Group-2(B1)



(A) #8 @ 8" c/c - bottom

(B) #8 @ 8" c/c - top

(C) #7 @ 12" c/c - extra top

(D) #7 @ 10" c/c - extra bottom

Mat foundation Layout Y direction

Checked and Vetted By:

Designed By: Group-2(B1)

Prepared By: Group-2(B1)

Project: Reconstruction of Green Road Staff Quarter

Client:

Submitted By: Group-2(B1)

Shear Wall Design:

At edge, the required bar area=17in^2

>Providing #10 bar at edges

>Number of bar required at edges=17/1.27 =14 bars

>Confinement bar Avh =0.42 in^2 /ft =.035 in^2/in

>Providing 2-legged #3 bar

>Required Spacing, S=(2*.11)/.035=6” c/c

>Bc=edge length-1= 50”

Here,

$$A_{sh} = 0.09 s b_c f_c' / f_{yt}$$

= 1.8 in^2

>Providing #4 bar

>So, the minimum tie bar required =1.8/.2=9

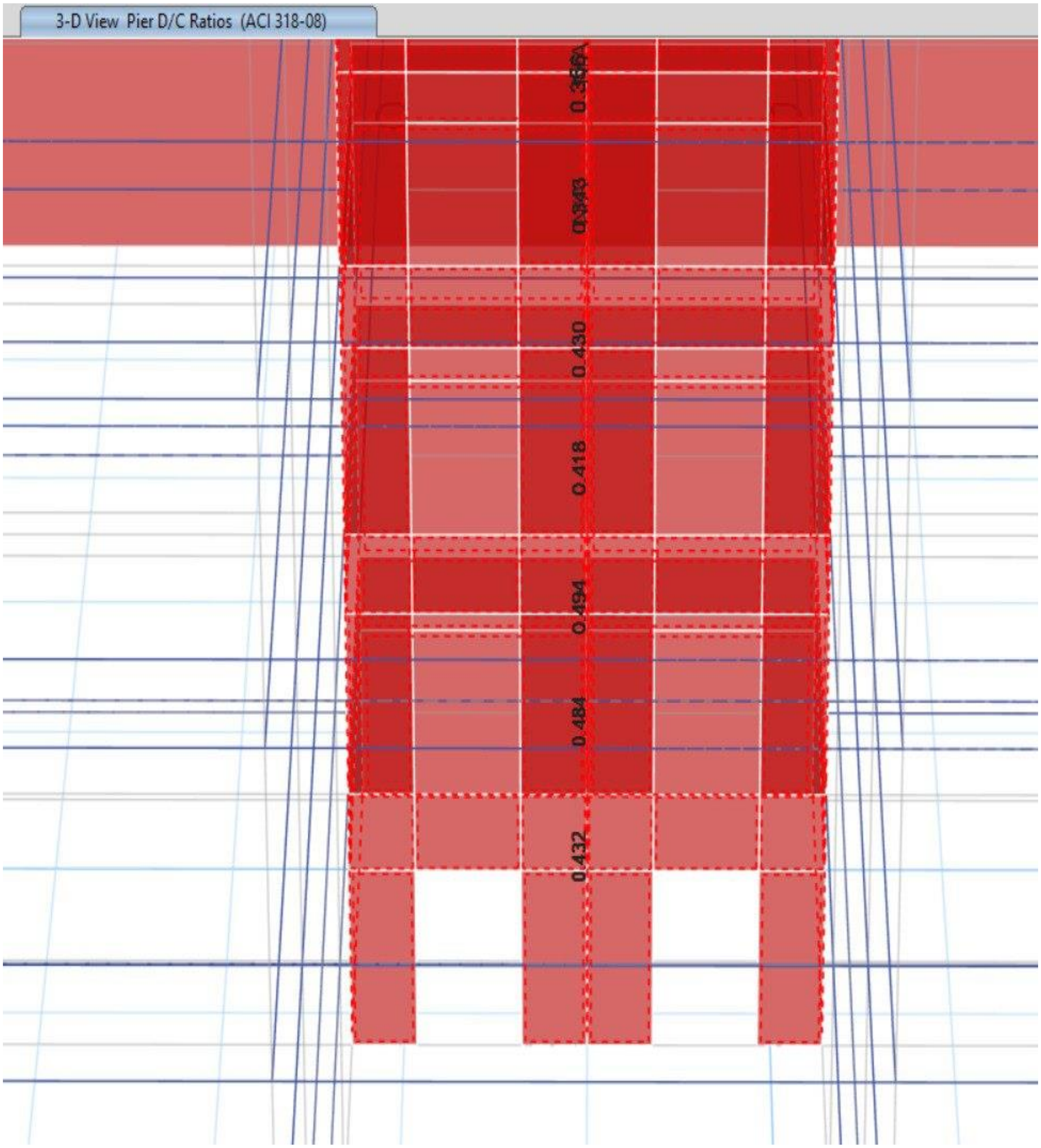
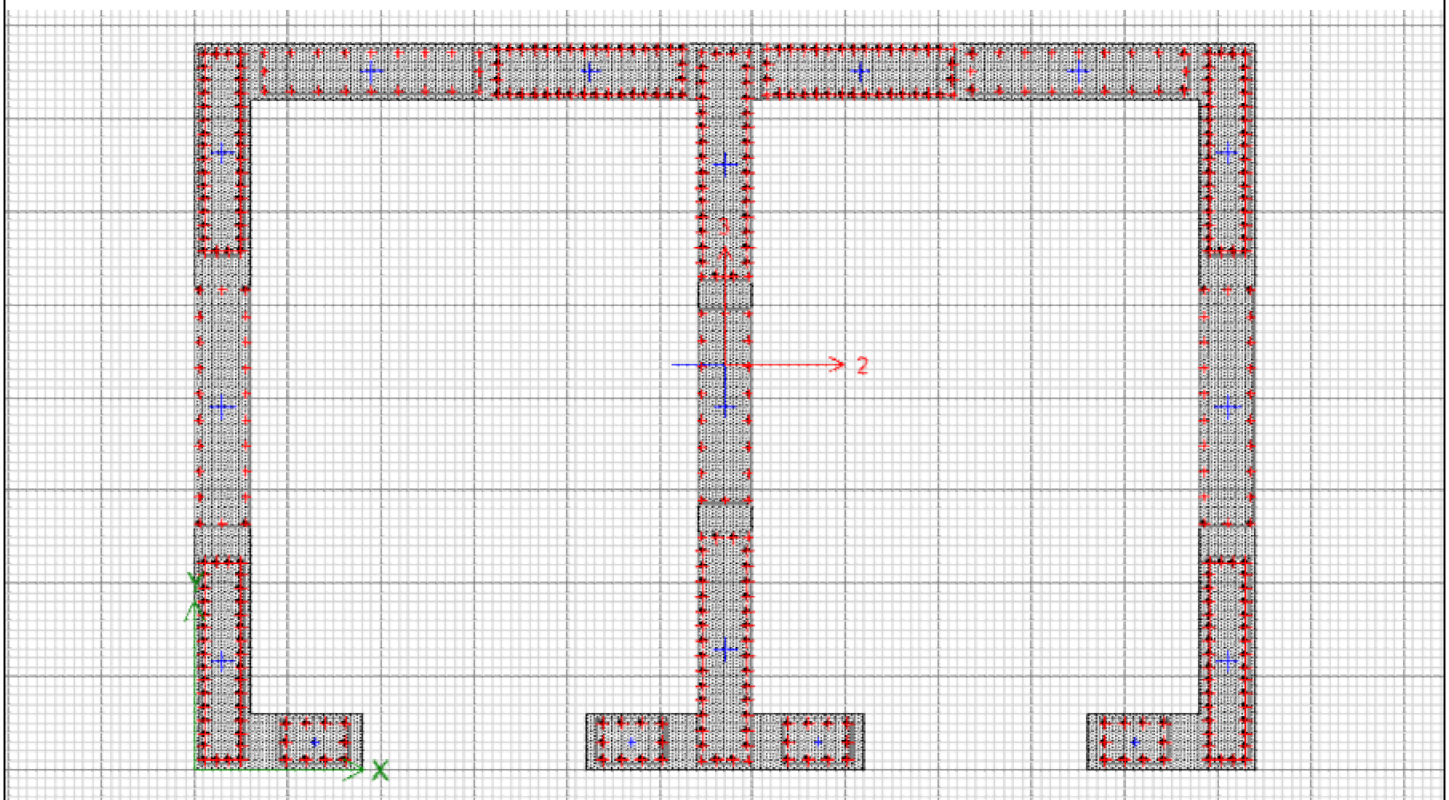


Figure: D/C ratio for shear wall



3
2
Y
X

Basement Wall Design:

>For soil pressure calculation, we need to consider the active earth pressure coefficient and earth pressure coefficient at rest.

>For our soil, the angle of internal friction, φ = 34˚

>So, earth pressure at rest, k_o=1-Sin (φ) = 0.441

>Now by considering the dynamic earthquake effect, the adjusted active earth pressure coefficient is: -

$$C_a = \frac{(1 \pm \alpha_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} \times \left[\frac{1}{1 + \left\{ \frac{\sin(\phi + \delta)\sin(\phi - \iota - \lambda}{\cos(\alpha - \iota)\cos(\delta + \alpha + \lambda)} \right\}^{\frac{1}{2}}} \right]^2$$

(Mononabe-Okabe)

Now horizontal seismic coefficient is:

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

= (.2/2) *(1/1.5) *1 =0.12

Z = Seismic Zone factor which is taken as 0.2 for seismic Zone II

I = Importance factor which is taken as 1.00 for the retaining wall

R = Response reduction factor is taken as 1.50 for unreinforced concrete wall

>The solid retaining wall is almost rigid and no differential displacement shall take place in the wall during seismic acceleration. Hence the wall is taken as zero period structure and the spectral acceleration coefficient of the wall is taken as 1.00.

>The vertical acceleration coefficient, α_v =(2/3) *0.014=0.08

>The angle of wall friction, δ = (3/4) *34=25.5˚

>The angle of the back face of the wall with vertical, α= 0˚

$$\lambda = \tan^{-1} \frac{\alpha_h}{1 \pm \alpha_v}$$

=6.34

>So dynamic active pressure coefficient is, Ca=0.35

>Now Ca< k_o. So, using earth pressure at rest value will be conservative.

Soil pressure, P=Ax+By+Cz+D

=Cz+D

>Now, at z=0,P= $k_0 \times \gamma \times h$=.44*124*10=545.6psf=D

>Now, at z=h, P=0, C=-$k_0 \times \gamma$=-54.56

>Uniform Load due to surcharge=176 psf

>Load is assigned and designed using the basics of shear wall

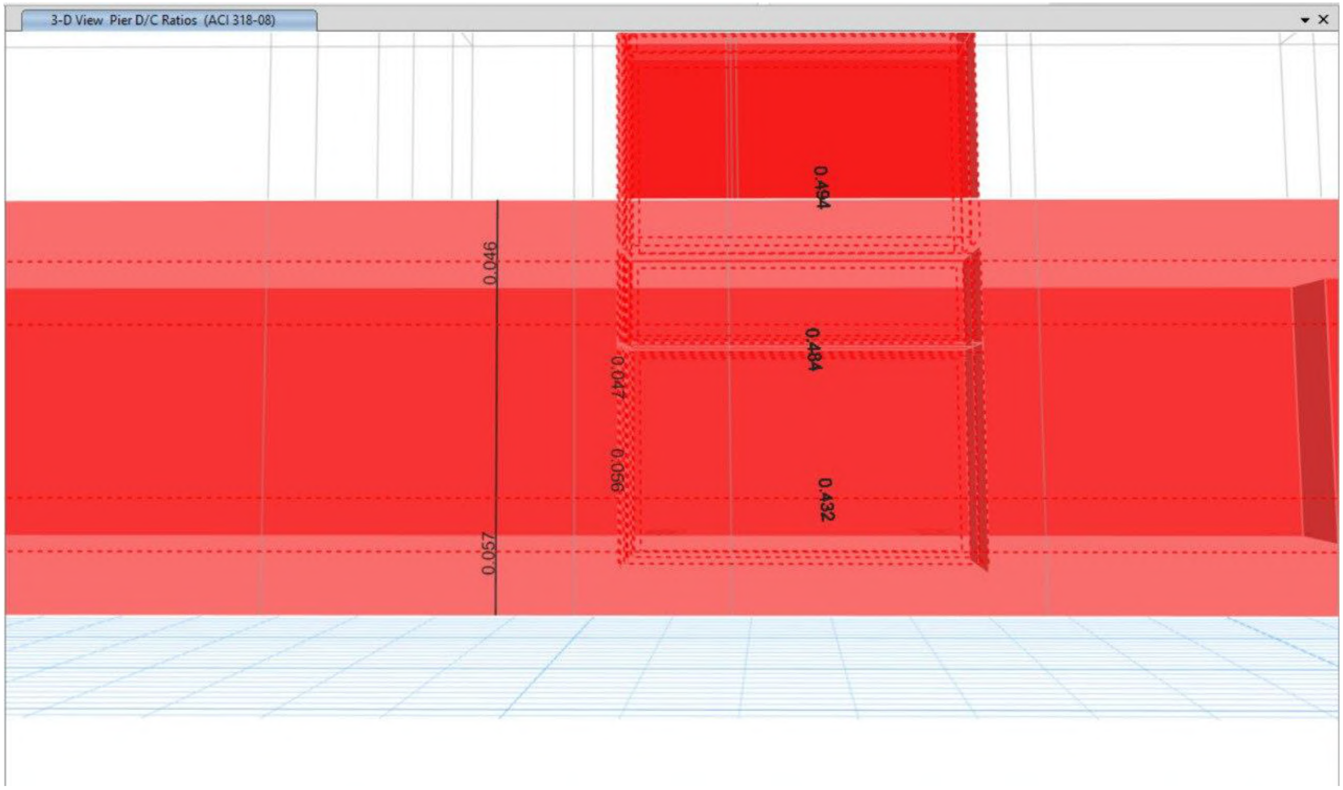
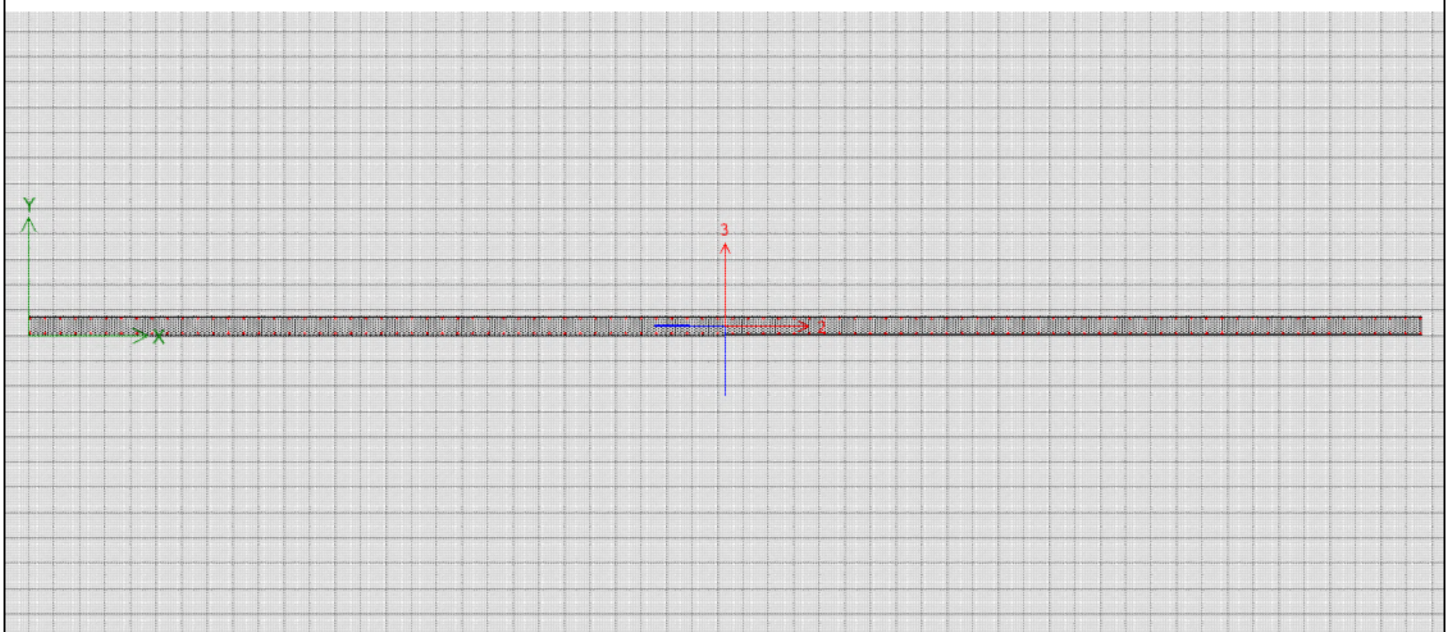


Figure: D/C ratio for basement wall



Y
X
3
2

Water Demand Calculation

As per BNBC 2020

Domestic water demand = 180 lpcd

For mosque = 15 lpcd (per prayer period)

= 15 X 5 =75 lpcd (for 5 prayer period)

For Community center = 90 lpcd

Population Forecast:

Current Population = 1350

$P' = P(1+r/100)^n$

$P' = 1350\ (1+1.03/100)^{20}$

= 1650 (Approx)

So, Future Population = 1650

P' = Future Population

P = Present Population

r = Growth rate (1.03%)

n = Design Year = 20 Years

Domestic Water Consumption:

Water consumption (lpd) = 180 X 1650 lpd = 296.95 Klpd

Water demand (Mlpd) = water consumption x peak factor x time factor

= 296.95*2.5*1 Mlpd

= 0.74Mlpd

Mosque Capacity:

Mosque floor area = 480 m^2 (per story)

Total number of story = 6

Mosque total floor area = (480*6) m^2

= 2880 m^2

Area required per worshipper = 0.8*1.2 m^2

$= 0.96\ m^2$ per person

Congregational capacity of the mosque = 2880/0.96 person

= 3000 person

Ref:

https://hal.science/hal-00810652

Community Center Capacity:

Total area per floor = 635 m^2

Effective area per floor = 500 m^2

Total area = (500*3) m^2 [3 story]

= 1500 m^2

Persons can be accommodated per square meter = 0.7

Total capacity of the community center = 1500*0.7

= 1050 person

Ref:

www.deccanherald.com

www.paramvisions.com

Peak Factor:

Domestic = 2.5

Mosque = 4

Community Center = 4

Time Factor:

Domestic= 24/24 = 1

Mosque = 24/24 = 1

Community Center = 24/(2X6) = 2

For fire-fighting:

Light hazard-I : Occupancy group,
A1, A2, A3, E1

Occupancy group = A3

So, light hazard-I

Standpipe and Hose system

Total volume for fire fighting = 1000*30 liter

= 30,000 liter

For Fire hydrant:

Let, Diameter, d=3"

Area of the pipe, A = (.25*pi*0.25^2) = 0.05 ft^2

Velocity of water, v=3fps

Discharge of each hydrant, Q=A*v =.05*3 = 0.15 ft^3 /s

Volume of water for each hydrant = Q* t

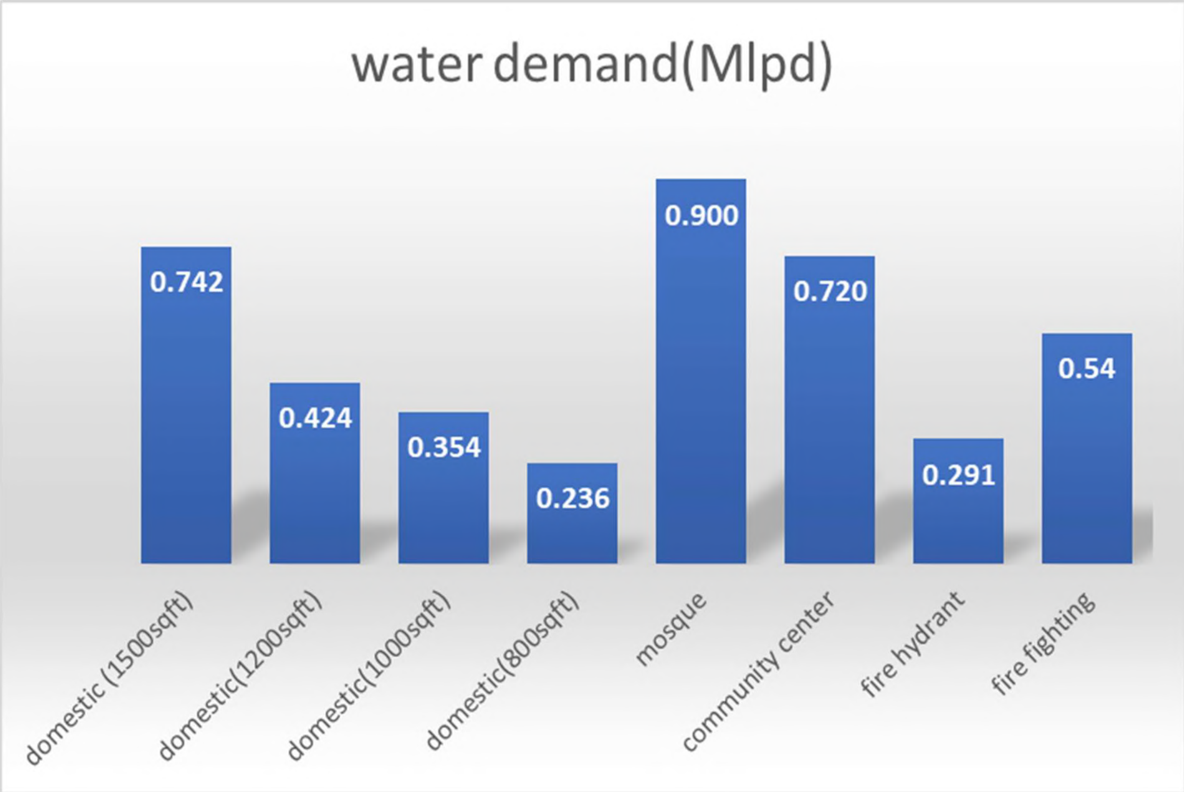
No of hydrant working at a time = 2

Supply water duration, t=30 min=1800 sec

So, firefighting demand in supply line=Q*t*2 = 540 ft^3

= 540*28.32 liter

= 15292 liter


water demand(Mlpd)
0.742
0.424
0.354
0.236
0.900
0.720
0.291
0.54
domestic (1500sqft)
domestic(1200sqft)
domestic(1000sqft)
domestic(800sqft)
mosque
community center
fire hydrant
fire fighting

building size	building no	present population	future Population	facility type	water demand BNBC2020(lpcd)	water comsumption(lpd)	peak factor	time factor	water demand(Mlpd)
1500	4	1344	1649.705	domestic	180	296946.926	2.5	1	0.742
1200	4	1152	1414.033	domestic	120	169683.958	2.5	1	0.424
1000	4	960	1178.361	domestic	120	141403.298	2.5	1	0.354
800	4	960	1178.361	domestic	80	94268.865	2.5	1	0.236
			3000	mosque	75	225000	4	1	0.900
			1000	community center	90	90000	4	2	0.720
				fire hydrant					0.291
				fire fighting					0.54
								Total demand(Mlpd)	4.206

Design of Septic Tank:

1)Volume required for sedimentation:

$t_h = 1.5 – 0.3\log_{10}(Pq)$

Population = 48*7

= 336 (total 48 units, 7 persons per unit) > 300

Taking P = 168

Water consumption = 180 lpcd

Wastewater flow, q = 70% of daily water consumption

= 0.7*180 lpcd = 126 lpcd

$t_h = 1.5 – 0.3\log_{10}(168*126)$

= 0.2

But t_h should be at least 1day

So, t_h = 1 day

$V_h = 10^{-3}$ P.q.$t_h = 10^{-3}$ *168*126*1

= 21.17 m^3

2)Volume required for Sludge Digestion:

$t_d = 30*(1.035)^{35-T}$

$= 30*(1.035)^{35-15}$

= 60 days

$V_d = 0.5 \times 10^{-3}$ P.t_d

$= 0.5 \times 10^{-3} \times 168 \times 60 = 5.04\ m^3$

3)Volume required for Sludge Storage:

V_{sl} = C*P*N

= 0.04*168*1 = 6.72 m^3

4)Volume required for Scum Storage:

$V_{sc} = 0.4*V_{sl}$

$= 0.4*6.72 = 2.69\ m^3$

Total effective volume of Septic Tank,

$V_{eff} = V_{sc}+V_h+V_d+V_{sl}$

$= 21.17+5.04+6.72+2.69 = 35.62\ m^3$

Selected Cross sectional Area of the septic tank = 3.4*10.4

$= 35.36\ m^2$

Depth of sludge storage zone = 6.72/35.36 = 0.19 m

Depth of sludge digestion zone = 5.04/35.36 = 0.143 m

Depth of scum storage zone = 2.69/35.36 = 0.076 m

Free board = 300mm = 0.3 m

i)Depth of sedimentation zone, h = 21.17/35.36 = 0.599 m

ii)Minimum clear space depth = $0.075 + d_{sc}$

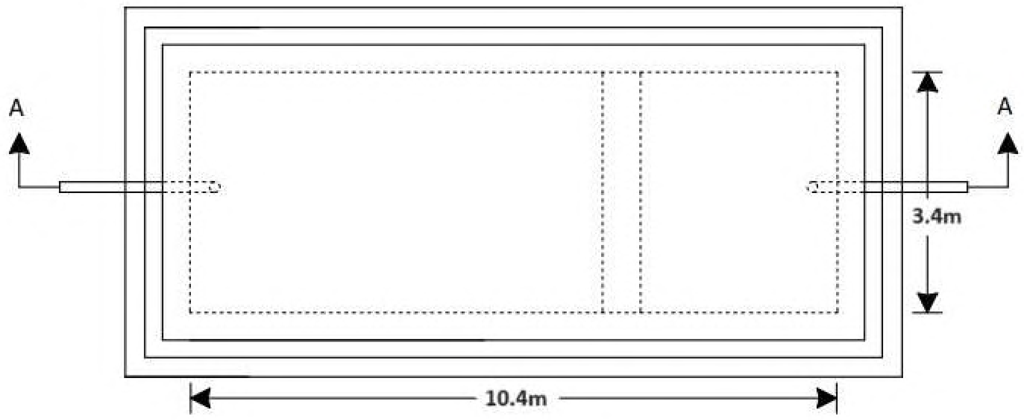
$= 0.075+0.3 = 0.375$ m

Depth of sedimentation zone taken = 0.599 m

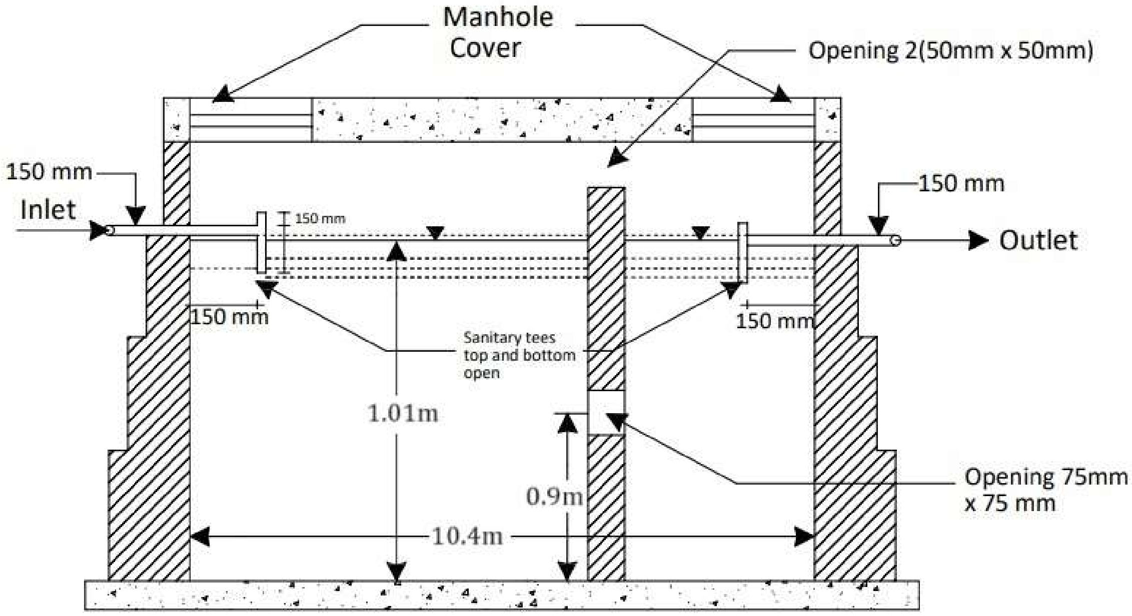
Total depth of the septic tank = 0.19+0.143+0.076+0.599+0.3

= 1.308 m

Dimension of the Tank: 10.4m x 3.4m x 1.31m



PLAN



SECTION A - A

Figure: Two chambered Septic tank

Septic tank reinforcement calculation:

a)Base Slab:

L/B = 10.4/3.4 = 3.06>2

So, one way slab

Minimum thickness = 9”

self weight of base slab, w_{DL} = (9*150)/12 = 112.5 psf/ft width

live load, w_{LL} = 10psf (a minimum amount is taken for cleaning purpose)

live load from water weight, w_{LL} =62.4* 1.01* 3.28 = 206.72 psf

$w_{LL(Total)}$ = 206.72+10 = 216.72 psf

Total load = 1.2*112.5+1.6*216.72 = 481.75 psf

At support, M- = $(0.482*11.15^2)/24$ = 2.5 k-ft

At midspan, M+ = $(0.482*11.15^2)/14$ = 4.28 k-ft

d_{min} = 9-2 = 7”

As(support) = M/(0.9*fy*(d-a/2)) = 0.08 in^2

As(midspan) = 0.138 in^2

As(min) = 0.0018bt = 0.194 in^2

b)Cover slab:

Minimum thickness = 3.5”

self weight of base slab, w_{DL} = (3.5*150)/12 = 43.75 psf/ft width

live load, w_{LL} = 10psf (a minimum amount is taken for cleaning purpose)

Total load = 1.2*43.75+1.6*10 = 68.5 psf

At support, M- = $(0.069*11.15^2)/24$ = 0.36 k-ft

At midspan, M+ = $(0.069*11.15^2)/14$ = 0.61 k-ft

d_{min} = 3.5-1 = 2.5”

As(support) = M/(0.9*fy*(d-a/2)) = 0.032 in^2

As(midspan) = 0.055 in^2

As(min) = 0.0018bt = 0.076 in^2

Geometric dimensions of Underground water Reservoir

water requirements=180 lpcd (litre per capita per day) [as per BNBC 2020]

Population Forecast:

Number of flats = 12x 4=48

Person living per flat= 5

Population = 240

Water consumption (lpd) = 180 X 240

=43200

Water demand (lpd) = water consumption x peak factor

=43200 *2.2 lpd

= 95040 lpd

So, volume of water needed in one day (24 hours)= 95040 litre

For fire-fighting:

Light hazard-I : Occupancy group, A1, A2, A3, E1

Occupancy group = A3

So, light hazard-I, Standpipe and Hose system

Total volume for fire fighting = 1000*30 liter

= 30,000 liter = 1100 cft

We will construct separate tank for fire fighting demand and the dimension is 15’x 8’x 10’

volume of water needed in two days =2 x 95040= 190080 litre =190080 x .0353= 6710 cft

we will construct 2 UGWR

so volume of each tank will be 3350 cft approx

UGWR dimension= 22’ x 15’ x 10’

Geometric dimensions of roof top/overhead water tank:

volume of water needed in one-day=95040 litre =95040 x .0353=3500 cft approx

we will construct 2 OHWT and pump almost 2 times in a day

so volume of each tank will be =V1+V2= 3800/(2x2) + 25% of V1 =1200 cft approx

OHWT dimension= 15’ x 12’ x 10’

Design of Over Head Water Tank (OHWT)

outer dimension= 15’ x 12’ x 10’

design of cover slab:

check,=L/B=15/12=1.25 <2; two-way slab (edge-supported)

t =4 in ; Thickness= perimeter/180= 2 x (15+12) x 12 /180= 3.6 in

self-weight of cover slab, W(DL)=4/12x150pcf=49.5psf/ft width

live load,WLL = 10 psf(a minimum amount is taken for cleaning purpose)

cover slab is simply-supported on all four sides (simply-supported on side-walls) & case I

B/L = 0.8 & Case I

at mid-span,Mua(+) = 1.2x0.056x49.5x12^2+1.6x0.056x10x12^2=+608 1b.-ft/ft width

Mub+ = 1.2x0.023x49.5x15^2+1.6x0.023x10x15^2=+390 lb.-ft/ft width

at free-edge, M(a)=0&M(b)=0(theoretically)

but ACI code suggests that ot free edge,M-ve=1/3 M+ve

Mu(a)(-ve)=-202.67 lb.-ft/ft width and

Mu(b) (-ve)=-130 lb.-ft/ft width

As min = 0.002 b t=0.002x12 inchx4 inch=0.096 inch/ft width

flexural steels (As1):

at mid-span of shorter direction (La=B=12feet) of cover slab

+As1= 0.0225< 0.096 inch/ft width

So +As1=0.096 inch/ft width

at mid-span of longer direction (Lb = L = 15 feet) of cover slab

（+） As1= 0.01056<0.096 inch/ft width

So +As1=0.096 inch/ft width

requirement (provided)	cover slab in shorter direction(La=B=12 feet)		
location	at ends/edge (inch2/ft)	at mid-span/center (inch2/ft)	at ends/edge(inchh2/ft)
at top (upper face)	=0.096	not needed	=0.096
at bottom(lower face)	=0.096	=0.096	=0.096

[*at least 1/3rd of the positive steel continued uninterrupted over the supports]

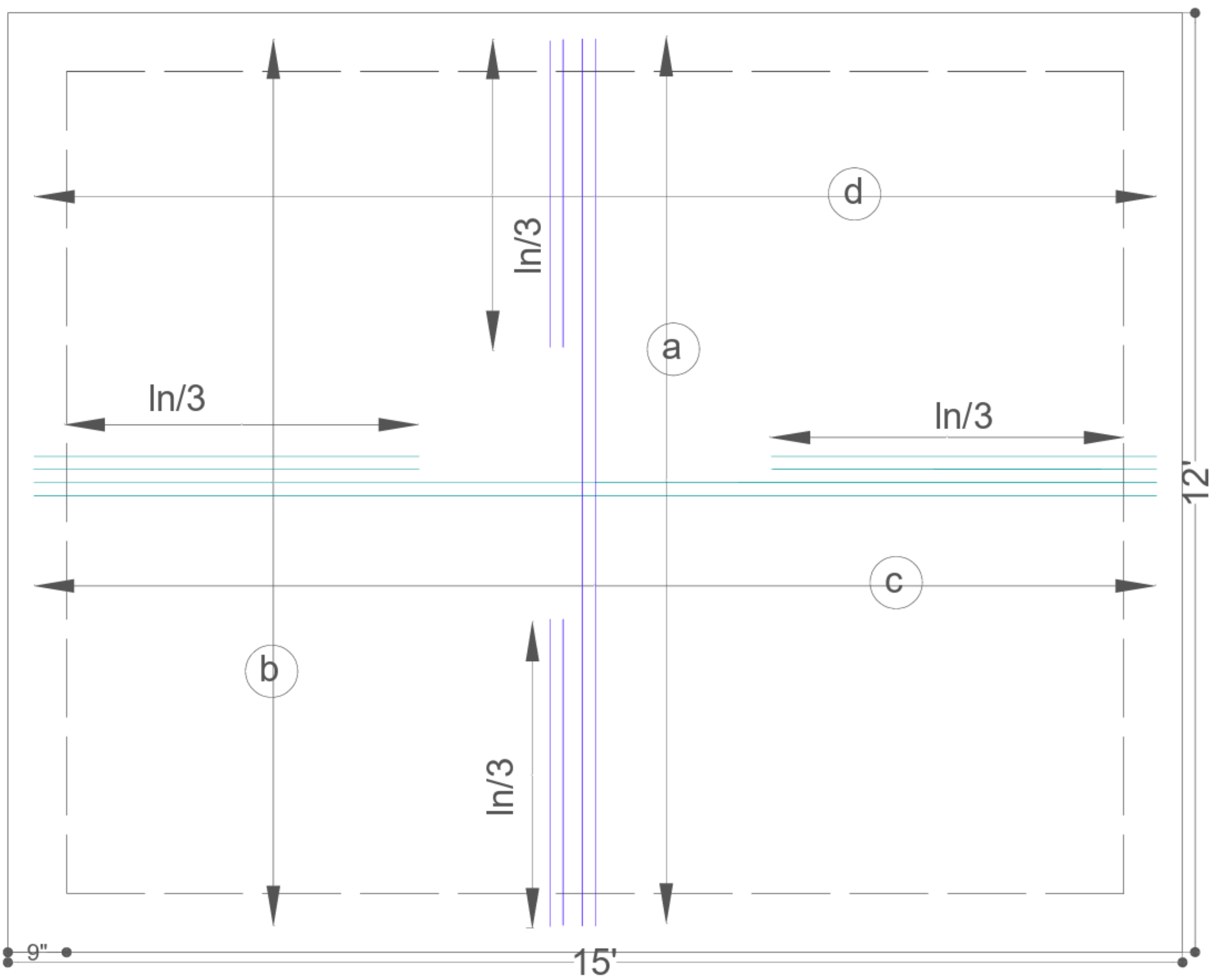
steel spacing provided (using #3 bar)	cover slab in shorter direction(La=B=12 feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at top (upper face)	#3@8”c/c		#3@8”c/c
at bottom (lower face)	#3@8”c/c	#3@8”c/c	#3@8”c/c

Spacing=.11 x 12/0.096 =13.75 > 2t = 8 in

steel requirement (provided)	cover slab in longer direction(L_b=L=15 feet)		
location	at ends/edge (inch2/ft)	At mid-span/center (inch2/ft)	at ends/edge(inchh2/ft)
at top (upper face)	=0.096	not needed	=0.096
at bottom(lower face)	=0.096	=0.096	=0.096

[*at least 1/3rd of the positive steel continued uninterrupted over the supports]

steel spacing provided (using #3 bar)	cover slab in longer direction(La=B=12 feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at top (upper face)	#3@8”c/c		#3@8”c/c
at bottom (lower face)	#3@8”c/c	#3@8”c/c	#3@8”c/c



Reinforcement details of OHWT cover slab

Legends:

(a)#3 @8” c/c at bottom

(b)#3 @8” c/c extra top

(c)#3 @8” c/c at bottom

(d)#3 @8” c/c extra top

Base Slab Design:

self-weight of base slab,W_{DL}.=9/12x150pcf=112.50psf/ft width

live load, WLL = 10 psf (a minimum amount is taken for cleaning purpose)

live load from water weight,W_{LL}=YH=62.40x8.35=522.209 psf

WLL(total)=(10+522.209)=532.2 psf

H=8.35 feet

h = maximum of (H/4 or 3.28083 feet) = maximum of(8.35 feet/4 or 3.28083 feet)

= maximum of(2.0875 feet or 3.28083 feet)=3.28083 feet

(H-h)=(8.35 feet-3.28083 feet)=5.07 feet

M cantilever(service)=1/6 $Y_w Hh^2$=1/6x62.40x8.35x$(3.28083)^2$=934.26 lb.-ft/ft.width

M(factored)=1.6x281.69373b-=1494.82 lb-=M(edge of base slab)

base slab is also simply-supported on all four sides like cover slab (case I) except that base slab has to carry a negative moment at ends resulting from cantilever action of side-walls (Mcantilever)

So, mid-span positive moment will be reduced due this negative end moment.

at ends/edges of base slab,

Mu(a)-ve=Mu(corner/edge of base slab) = M(u)bottom end of side-wall = M(u)cantilever = 934.26 lb.-ft/ft.width

Mu(b)-ve=934.26 lb.-ft/ft.width

at mid-span,Mua(+) = -934.26 +1.2x0.056x112.5x12^2+1.6x0.056x532.2x12^2=7021.04 lb.-ft/ft width

Mub+ = -934.26 +1.2x0.023x112.5x15^2+1.6x0.023x532.2x15^2=+4170.98 lb.-ft/ft width

■ if $\frac{\sigma_t}{f_t}+\frac{\sigma_f}{f_r}\le 1$, no cracking in water tank base slab / side-walls

& $\frac{\sigma_t}{f_t}+\frac{\sigma_f}{f_r}\le 1$, crack will appear in water tank base slab / side-walls → increase base slab / side-wall thickness 't' until uncracked condition (i.e. $\frac{\sigma_t}{f_t}+\frac{\sigma_f}{f_r}\le 1$) is achieved.

T_L= 0.5 γ_wH2=.5x62.40x$(8.35)^2$=2175.34 1b./f.width= T_B

σ_t =2175.34/(12x9)=20.14 psi

M = maximum moment from the moment diagram of base slab=7021.04 1b.ft/ft width

σ_f= Mc/I=M(t/2)/(bt^3/12)= 7021.04x4.5/(12x9^3/12) = 43.34 psi

ft=10% of fc=10% of4000 psi = 400 psi

fr=7.50sqrt(4000)=7.50/3000 psi=474.34 psi

check: (20.14/400)+(43.34/474.34)< 1; ensures no cracking in base slab

As min = 0.002 bt=0.002x12 inchx9 inch=0.216 inch/ft width

direct tension steel:A_{s2}=1.6x2175.34/(.9x60000) = 0.0644 inch/ft width

flexural steels(A_{s1}):at ends/edges of base slab

(-) A_{s1} = 0.0479 inch2/ft width

at mid-span (of L = 15 feet) of base slab

(+) A_{s1} = 0.208 inch2/ft width

at mid-span (of B = 12 feet) of base slab

(+) As1 = 0.214 inch2/ft width

steel requirement (computed)	base-slab in longer direction (L=15 feet)		
location	at ends/edge(inch2/ft)	at mid-span/center (inch2/ft)	at ends/edge(inch2/ft)
at water face	$A_{s1}+A_{s2}/2=$ 0.0479 +0.0322	$A_{s2}/2$ =0.0322	$A_{s1}+A_{s2}/2=$ 0.0479 +0.0322
at free face	$A_{s2}/2=$ 0.0322	$A_{s1}+A_{s2}/2=$ 0.134+0.0322	$A_{s2}/2=$ 0.0322

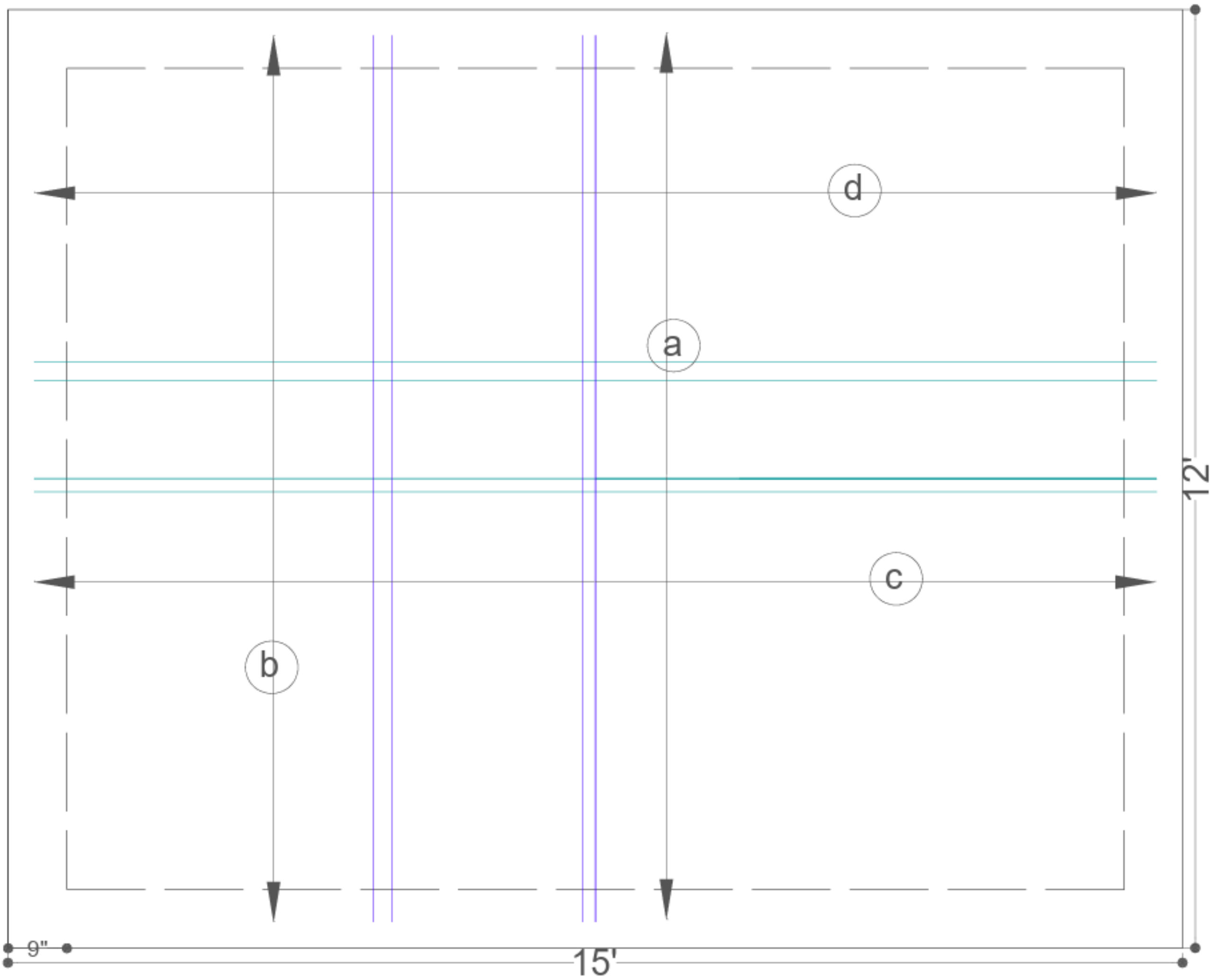
(check: the above computed steel should not be less than As(minimum)=0.002bt=0.002x12 inchx9inch=0.216 inch2/ ft width.

steel spacing provided (using #3 bar)	base-slab in longer direction (L=15feet)		
location	at corner/edge	at mid-span/center	at corner/edge
at water face	#3@6"c/c	#3@6"c/c	#3@6"c/c
at free face	#3@6"c/c	#3@6"c/c	#3@6"c/c

steel requirement (computed)	base-slab in shorter direction (L=12 feet)		
location	at ends/edge(inch2/ft)	at mid-span/center (inch2/ft)	at ends/edge(inch2/ft)
at water face	$A_{s1}+A_{s2}/2=$ 0.0298 +0.0322	$A_{s2}/2$ =0.0322	$A_{s1}+A_{s2}/2=$ 0.0298 +0.0322
at free face	$A_{s2}/2=$ 0.0322	$A_{s1}+A_{s2}/2=$ 0.228 +0.0322	$A_{s2}/2=$ 0.0322

(check: the above computed steel should not be less thanAs(minimum)=0.002bt=0.002x12 inchx9inch=0.216 inch2/ ft width.

steel spacing provided (using #3 bar)	base-slab in shorter direction (L=12 feet)		
location	at corner/edge	at mid-span/center	at corner/edge
at water face	#3@6"c/c	#3@6"c/c	#3@6"c/c
at free face	#3@6"c/c	#3@6"c/c	#3@6"c/c



Reinforcement details of OHWT base slab

Legends:

(a)#3 @6” c/c at bottom

(b)#3 @6” c/c at top

(c)#3 @6” c/c at bottom

(d)#3 @6” c/c top

Design of side-walls

H=8.35 feet

h= 3.28083 feet

(H-h)=(5.50 feet-3.28083 feet)=5.07 feet

P_h=γw(H-h)=62.40 pcfx5.07 feet=322.37 psf/ft. width

M cantilever(service)=1/6 $Y_w Hh^2$=1/6x62.40x8.35x$(3.28083)^2$=934.26 lb.-ft/ft.width

M(factored)=1.6x281.69373b-=1494.82 lb.-ft/ft.width

For Cantilever action (vertical bending)

flexural steels(As1)due to Mcantilever factored):at bottom of side-walls near water-face

As1 = 0.047 inch2/ft width<0.216 inch2/ft width

As1=0.216 inch2/ft width

flexural steels(As1)due to Mcantilever factored):at free face of side-walls

As1=0.216 inch2/ft width

spacing of#3 bar = 3in

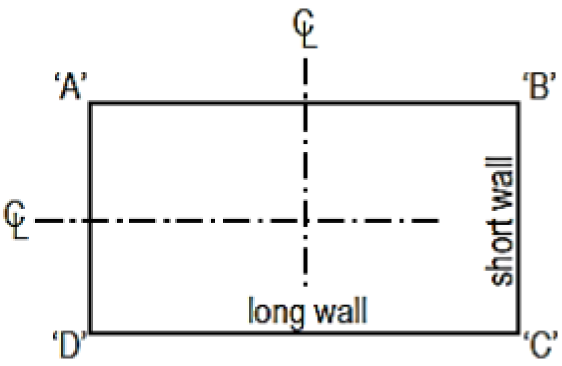
For Slab action (horizontal bending)

FEM(AB)= 322.37/1000x 15^2/12=+5.931.9 kip-ft (+ve sign is given as the moment is clockwise)

FEM AD=322.37/1000x 12^2/12=-3.796 kip-ft (-ve sign is given as the moment is anticlockwise)

$K_{AB=}$ 4EI/L_{AB}=1/15 4EI

K_{AB} : $K_{AD\,=}$ 1/15 : 1/12



DF_{AB} : $DF_{AD=}$ 0.44 : 0.56

unbalanced moment at joint“A'=5.931.9 - 3.796=+2.135k-/

this unbalanced moment will be distributed to member 'AB' & 'AD'in proportion to their stiffness.

distributed moment at end of member 'AB'=(0.44)x(-2.135)=-0.94 k-ft/ft

distributed moment at end of member 'AD' =(0.56) x(-2.135k/)=-1.2 k-ft/ft

A	0.44
0.56	5.931
-3.796	-0.94
-1.2	4.991
-4.991	

M $_{\text{midspan of longside-wall}}$=-M end+$wL^2/2$ = -4.991+ 0.322 x $15^2/8$ = 3.9 k-ft/ft

M $_{\text{midspan of shortside-wall}}$=-M end+$wL^2/2$ = -4.991+ 0.322 x $12^2/8$=0.697 k-ft/ft

$T_L=P_h$ x B/2=322.37x 12/2= 1898.2 1b./ft.width

$T_B=P_h$ x L/2=322.37x 15/2= 2372.78 1b./ft.width

= 0.002bt=0.002x12 inchx9 inch=0.216 inch/ft width

direct tension steels:A_{s2}(L)= 1.6x 1898.2/(0.9x 60000=0.056 inch2/ft width

A_{s2}(B)= 1.6x 2372.78 /(0.9x 60000=0.07 inch2/ft width

flexural steels(As1): at corners of long-wall 'AB' (L=15 feet) & at corners of short-wall 'AD'

(B=12 feet)

(+)As1(L &B)= 0.022 inch2/ft width

at mid-span of long-wall'AB'(L=15 feet)

(-)As1(L)= 0.125 inch2/ft width

at mid-span of short-wall'AD'(L=12 feet)

(-)As1(L)= 0.02218 inch2/ft width

steel requirement (computed)	Long side-wall (L=15 feet)		
location	at ends/edge(inch2/ft)	at mid-span/center	at ends/edge(inch2/ft)
at free face	As2/2= 0.028	As1+As2/2=0.2 +0.028	As2/2=0.028
at water face	As1+As2/2=0.022 +0.028	As2/2=0.028	As1+As2/2=0.022 +0.028

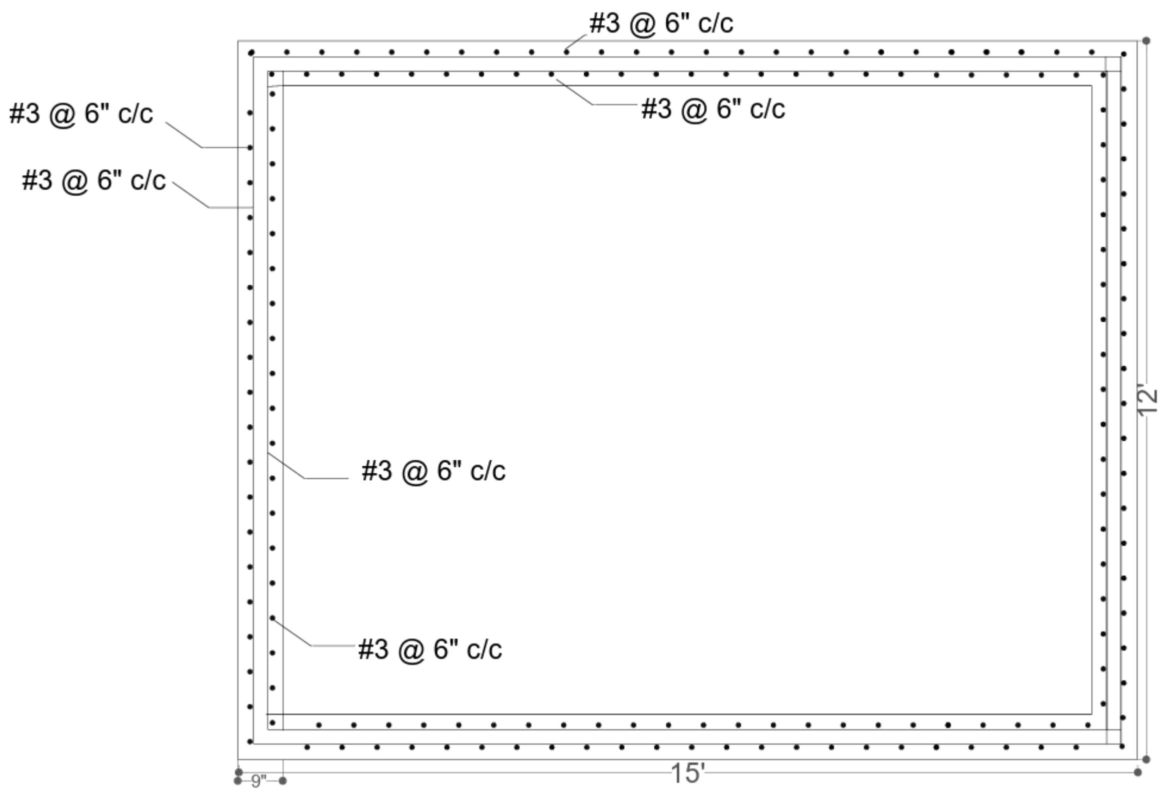
(check:the above computed steel should not be less than A3(mimimm) = 0.002bt =0.002x12 inchx9inch=0.216inch2/ft width.

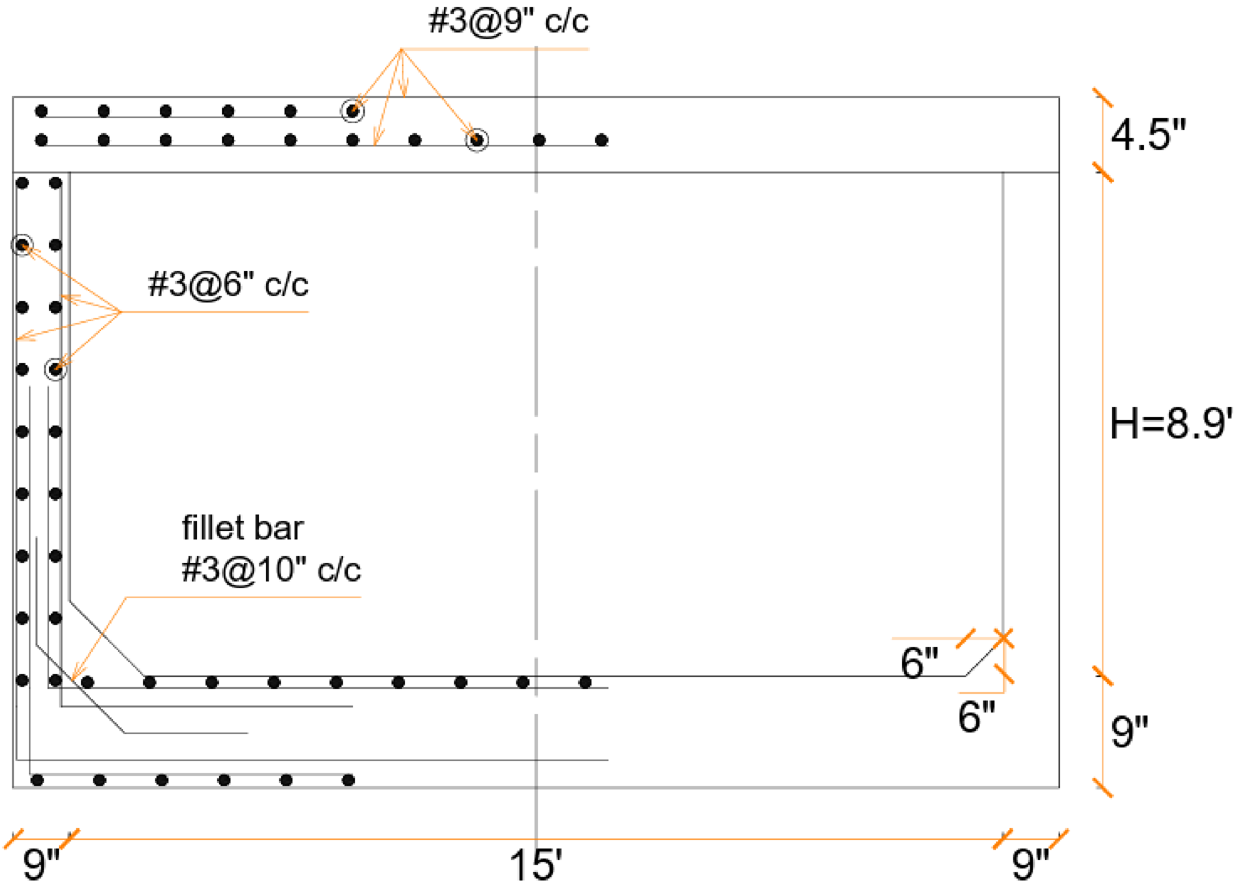
steel spacing provided (using #3 bar)	long sidee-wall'AB'(L=15 feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at free face	#3@6"c/c	#3@6"c/c	#3@6"c/c
at water face	#3@6"c/c	#3@6"c/c	#3@6"c/

steel requirement (computed)	short side-wall (B=12 feet)		
location	at ends/edge(inch2/ft)	at mid-span/center	at ends/edge(inch2/ft)
at free face	As2/2= 0.035	As1+As2/2=0.036 +0.035	As2/2=0.035
at water face	As1+As2/2=0.022 +0.035	As2/2=0.035	As1+As2/2=0.022 +0.035

(check:the above computed steel should not be less than As(minimum)=0.002bt=0.002x12 inchx9inch=0.216 inch2/ft width.

steel spacing provided (using #3 bar)	short sidee-wall'AD'(B=12feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at free face	#3@6"c/c	#3@6"c/c	#3@6"c/c
at water face	#3@6"c/c	#3@6"c/c	#3@6"c/





Reinforcement details of Side wall

Design of Underground Water Reservoir (UGWR)

Cover Slab Design:

L=22 feet

design of cover slab: check,=L/B=22/15=1.207 <2; two-way slab (edge-supported)

t =4.5 in thickness= perimeter/180= 2 x (22+15) x 12/ 180= 4.13 in

self-weight of cover slab, W(DL)=4.5/12x150pcf=56.25psf/ft width

live load,WLL = 10 psf (a minimum amount is taken for cleaning purpose)

cover slab is simply-supported on all four sides (simply-supported on side-walls) & case I

B/L = 0.94 & Case I

at mid-span,Mua(+) = 1.2x0.041x49.5x15^2+1.6x0.041x10x15^2=+695.56lb.-ft/ft width

Mub+ = 1.2x0.032x56.25x22^2+1.6x0.032x10x22^2=+617.67 lb.-ft/ft width

at free-edge,M.((a)=0&M(b)=0(theoretically)

but ACI code suggests that ot free edge, M-ve=1/3 M+ve

Mu(a)(-ve)=-231.85 lb.-ft/ft width and

Mu(b) (-ve)=-205.89 lb.-ft/ft width

As min = 0.002 b t=0.002x12 inchx4.5 inch=0.108 inch/ft width

flexural steels (As1):

at mid-span of shorter direction (La=B=15 feet) of cover slab

+As1= 0.095< 0.108 inch/ft width

So +As1=0.108 inch/ft width

at mid-span of longer direction (Lb = L = 22 feet) of cover slab

（+） As1= 0.0727<0.108 inch/ft width

So +As1=0.108 inch/ft width

steel requirement (provided)	cover slab in shorter direction(La=B=15 feet)		
location	at ends/edge (inch2/ft)	at mid-span/center (inch2/ft)	at ends/edge(inch2/ft)
at top (upper face)	=0.108	not needed	=0.108
at bottom(lower face)	=0.108	=0.108	=0.108

[*at least 1/3rd of the positive steel continued uninterrupted over the supports]

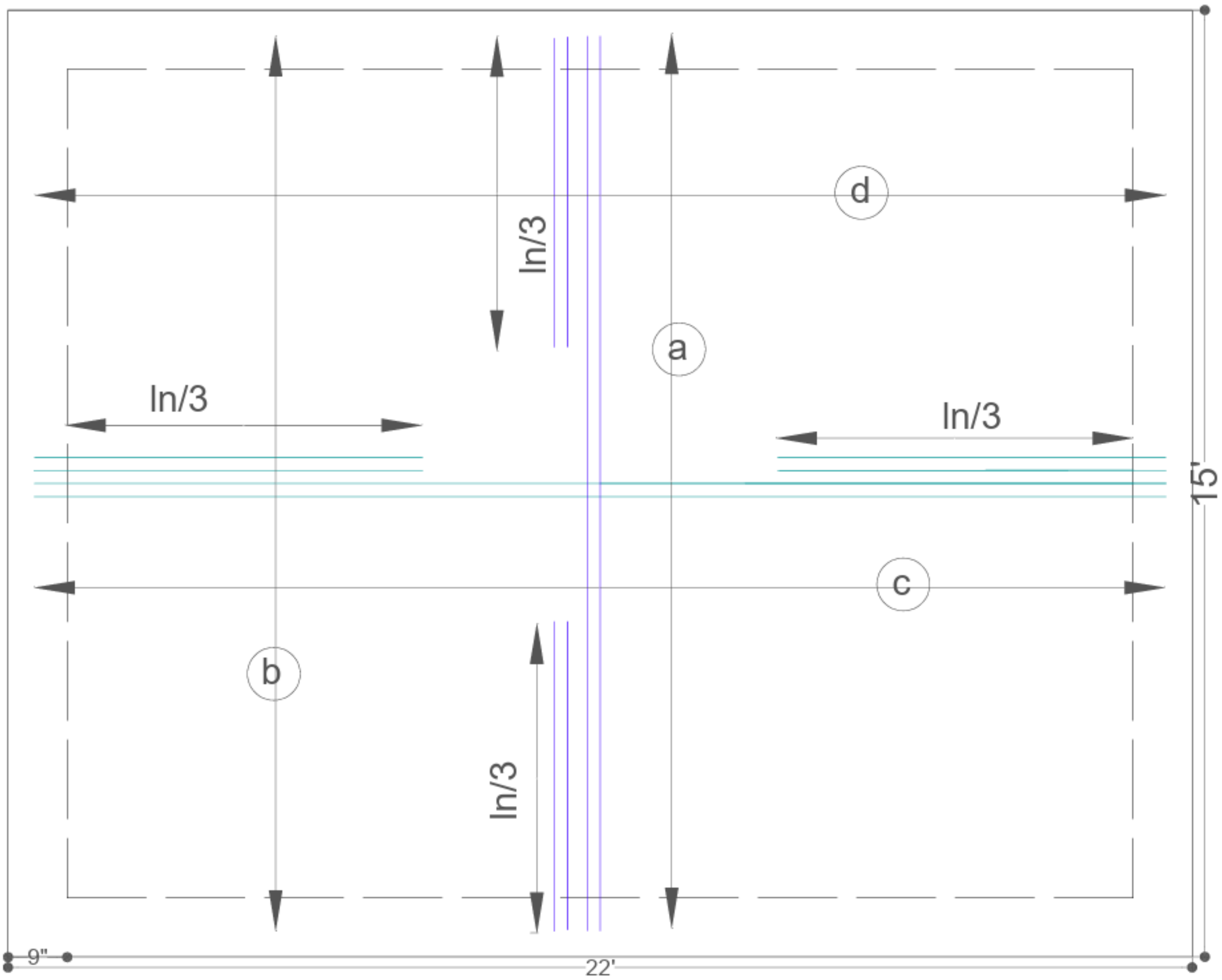
Spacing= .11 x 12/0.108 =12 in > 2t = 9 in

steel spacing provided (using #3 bar)	cover slab in shorter direction(La=B=15 feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at top (upper face)	#3@9”c/c		#3@9”c/c
at bottom (lower face)	#3@9”c/c	#3@9”c/c	#3@9”c/c

steel requirement (provided)	cover slab in longer direction(L_b=L=22 feet)		
location	at ends/edge (inch2/ft)	At mid-span/center (inch2/ft)	at ends/edge(inch2/ft)
at top (upper face)	=0.108	not needed	=0.108
at bottom(lower face)	=0.108	=0.108	=0.108

[*at least 1/3rd of the positive steel continued uninterrupted over the supports]

steel spacing provided (using #3 bar)	cover slab in longer direction(La=B=22 feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at top (upper face)	#3@9"c/c		#3@9"c/c
at bottom (lower face)	#3@9"c/c	#3@9"c/c	#3@9"c/c



Reinforcement details of UGWR cover slab

Legends:

(a)#3 @9” c/c at bottom

(b)#3 @9” c/c extra top

(c)#3 @9” c/c at bottom

(d)#3 @9” c/c extra top

Base Slab Design:

design of base slab

self-weight of base slab,WDL.=9/12x150pcf=112.50psf/ft width

live load, WLL = 10 psf (a minimum amount is taken for cleaning purpose)

live load from water weight,WLL=YH=62.40x9.25=577.2 psf

WLL(total)=(10+577.2)=587.2 psf

H=7.75 ft + 14 in + 4 in= 9.25 ft

h = maximum of (H/4 or 3.28083 feet) = maximum of(9.25 feet/4 or 3.28083 feet)

= maximum of(9.31 feet or 3.28083 feet)=3.28083 feet

(H-h)=(9.31 feet-3.28083 feet)=6.03 feet

$k_{a=}$ (1-sin34)/(1+sin34)= 0.2827

M cantilever(service)=1/6($k_a Y_{eff}$+ $Y_{w)}$$Hh^2$=1/6x(.2827x (125-62.4+62.40)x9.25x$(3.28083)^2$=1328.54 lb.-ft/ft.width

M(factored)=1.6x1328.54 =2125.55 lb-ft=M(edge of base slab)

I):base slab is also simply-supported on all four sides like cover slab (case I) except that base slab has to carry a positive moment at ends resulting from cantilever action of side-walls.

Mcantilever) in addition to an upward water pressure [Y(H+$t_{baselsab}$)=62.40x(9.25+0.75)=624 psf/ft. width] & a downward tank (tank is emply) self-weight of 112.50 psf/ ft. width].

So, mid-span negative moment will be reduced due this positive end moment.

at ends/edges of base slab,

Mu(a)-ve=Mu(corner/edge of base slab) = M(u)bottom end of side-wall = M(u)cantilever= 2125.55 lb.-ft/ft.width

Mu(b)-ve=2125.55 lb.-ft/ft.width

at mid-span,Mua(+) = +2125.55 +1.2x0.041x112.5x15^2-1.6x0.041x577.2x15^2=

-5148.547 lb.-ft/ft width

Mub+ = +2125.55 +1.2x0.032x112.5x22^2-1.6x0.032x577.2x22^2=-4334.00584 lb.-ft/ft width

flexural steels(As1):at ends/edges of base slab

(-) As1 = 0.074 inch2/ft width

at mid-span (of B = 15 feet) of base slab

(+) $As1 = 0.2302$ inch2/ft width

at mid-span (of L = 22 feet) of base slab

(+) $As1 = 0.177$ inch2/ft width

As min= 0.002bt= 0.216 inch2/ft width

steel requirement (provided)	base slab in shorter direction(La=B=15 feet)		
location	at ends/edge (inch2/ft)	at mid-span/center (inch2/ft)	at ends/edge(inch2/ft)
at top (upper face)	=0.216	Not needed	=0.216
at bottom(lower face)	=0. 0.216	= 0.2302	=0.216

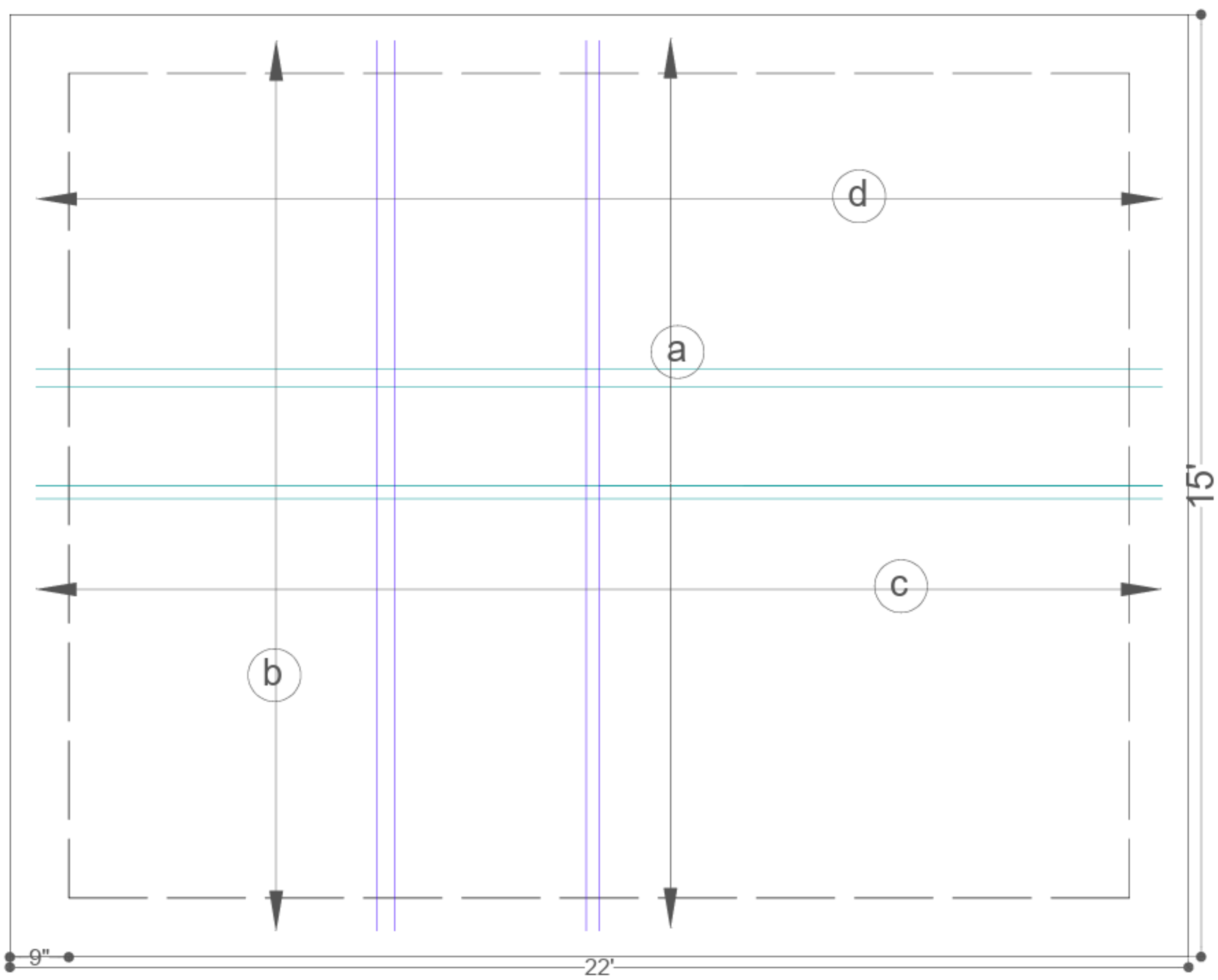
[*at least 1/3rd of the positive steel continued uninterrupted over the supports]

steel spacing provided (using #3 bar)	Base slab in shorter direction(La=B=15 feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at top (upper face)	#3@6"c/c		#3@6"c/c
at bottom (lower face)	#3@6"c/c	#3@5.5"c/c	#3@6"c/c

steel requirement (provided)	base slab in longer direction(Lb=L=22 feet)		
location	at ends/edge (inch2/ft)	At mid-span/center (inch2/ft)	at ends/edge(inch2/ft)
at top (upper face)	=0.216	not needed	=0.216
at bottom(lower face)	=0.216	=0.216	=0.216

[*at least 1/3rd of the positive steel continued uninterrupted over the support

steel spacing provided (using #3 bar)	base slab in longer direction(La=L=22 feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at top (upper face)	#3@6"c/c		#3@6"c/c
at bottom (lower face)	#3@6"c/c	#3@6"c/c	#3@6"c/c



Reinforcement details of UGWR base slab

Legends:

(a)#3 @6” c/c at bottom

(b)#3 @6” c/c top

(c)#3 @6” c/c at bottom

(d)#3 @6” c/c top

Design of side walls

H= 9.25 ft

H= 3.28 feet

(H-h)= 5.97

P_h=($k_a Y_{eff}$+ $Y_{w)}$ x (H-h)=(0.2827x 62.6+62.4)x (5.97)= 478.18 psf/ft width

flexural steels(As1)due to Mcantilever （factored):at bottom of side-walls near water-face

As1 = 0.0298 inch2/ft width<0.216 inch2/ft width

As1=0.216 inch2/ft width

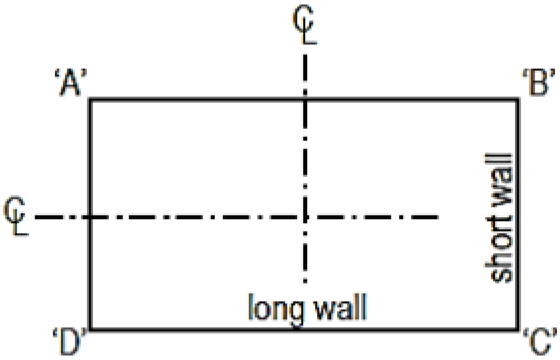
spacing of#3 bar)= 6in

FEM(AB)= P_h x $(L_{AB)}^2$ =3 /12 =322.37/1000x 216/12=-6.74 kip-ft (-ve sign is given as the moment is anti clockwise)

FEM AD=322.37/1000x 15^2/12=+5.925 kip-ft (+ve sign is given as the moment is clockwise)

$K_{AB=}$ 4EI/L_{AB}=1/22 4EI

K_{AB} : $K_{AD\,=}$ 1/22 : 1/15



DF_{AB} : $DF_{AD=}$ 0.48 : 0.52

unbalanced moment at joint“A'= 5.925-6.74 k-ft/ft =-0.815 k-ft/ft this unbalanced moment will be distributed to member 'AB' & 'AD'in proportion to their stiffness.

distributed moment at end of member 'AB'=(0.48)x(-0.815)=-0.39 k-ft/ft

distributed moment at end of member \'AD\' =(0.52) x(-0.815)=-0.42 k-ft/ft

A	0.48
0.52	-6.74
+5.925	+0.39
-0.42	-6.35
+6.345	

M midspan of longside-wall=-M end+$wL^2/2$ = -6.345+ 0.322 x 216/8 = 3.767 k-ft/ft

M midspan of shortside-wall=-M end+$wL^2/2$ = -6.345+ 0.322 x $15^2/8$=2.54 k-ft/ft

flexural steels(A,1): at corners of long-wall 'AB' (L=22 feet) & at corners of short-wall 'AD' （B=15 feet)

(+)As1(L &B)= 0.206 inch2/ft width

at mid-span of long-wall\'AB\'(L=22 feet)

(-)As1)= 0.1211 inch2/ft width

at mid-span of short-wall'AD\'(L=15 feet)

(-)As1= 0.0813 inch2/ft width

steel requirement (computed)	long side-wall'AB'(L=22 feet)		
location	at ends/edge(inch2/ft)	at mid-span/center	at ends/edge(inch2/ft)
at free face	0.216	0.216	0.216
at water face	0.216	0.216	0.216

(check :the above computed steel should not be less than As(mimimm)

=0.002bt=0.002x12 inchx9inch=0.216inch2/ft width.

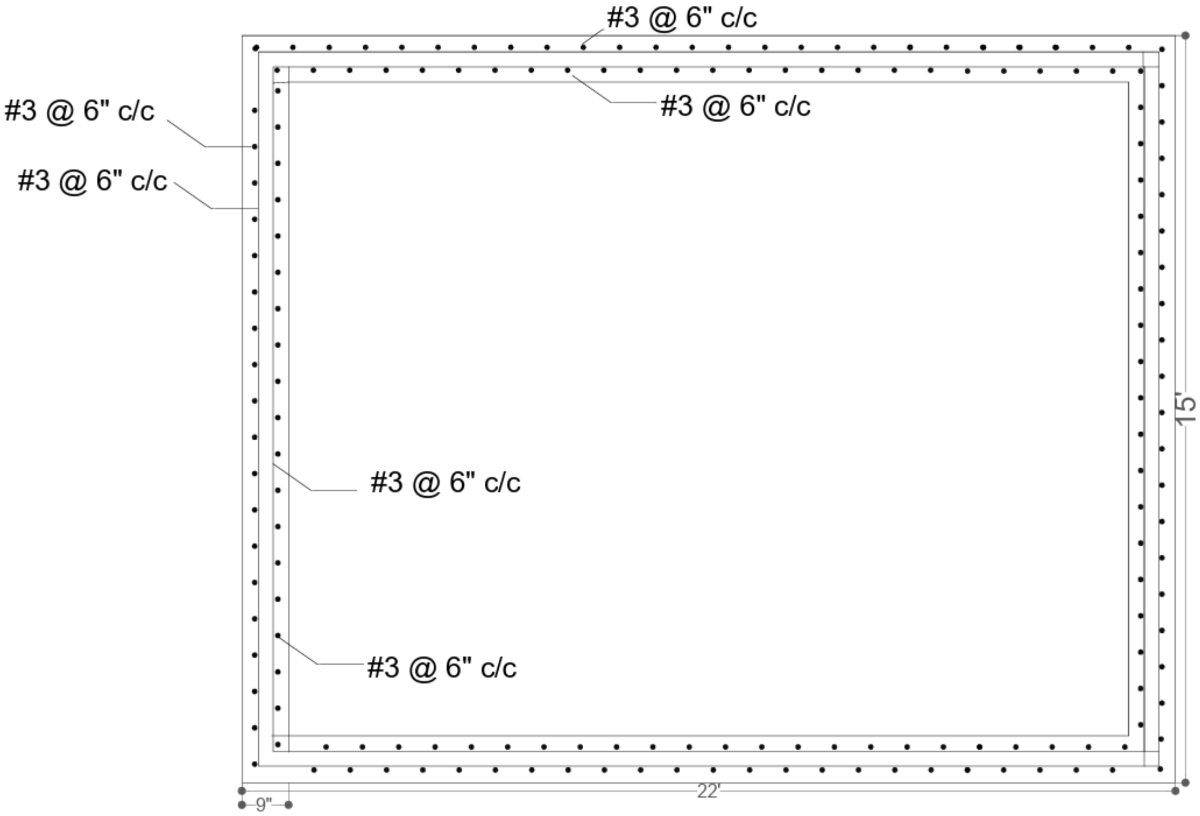
steel spacing provided (using #3 bar)	long side-wall'AB'(L=22 feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at free face	#3@6"c/c	#3@6"c/c	#3@6"c/c
at water face	#3@6"c/c	#3@6"c/c	#3@6"c/

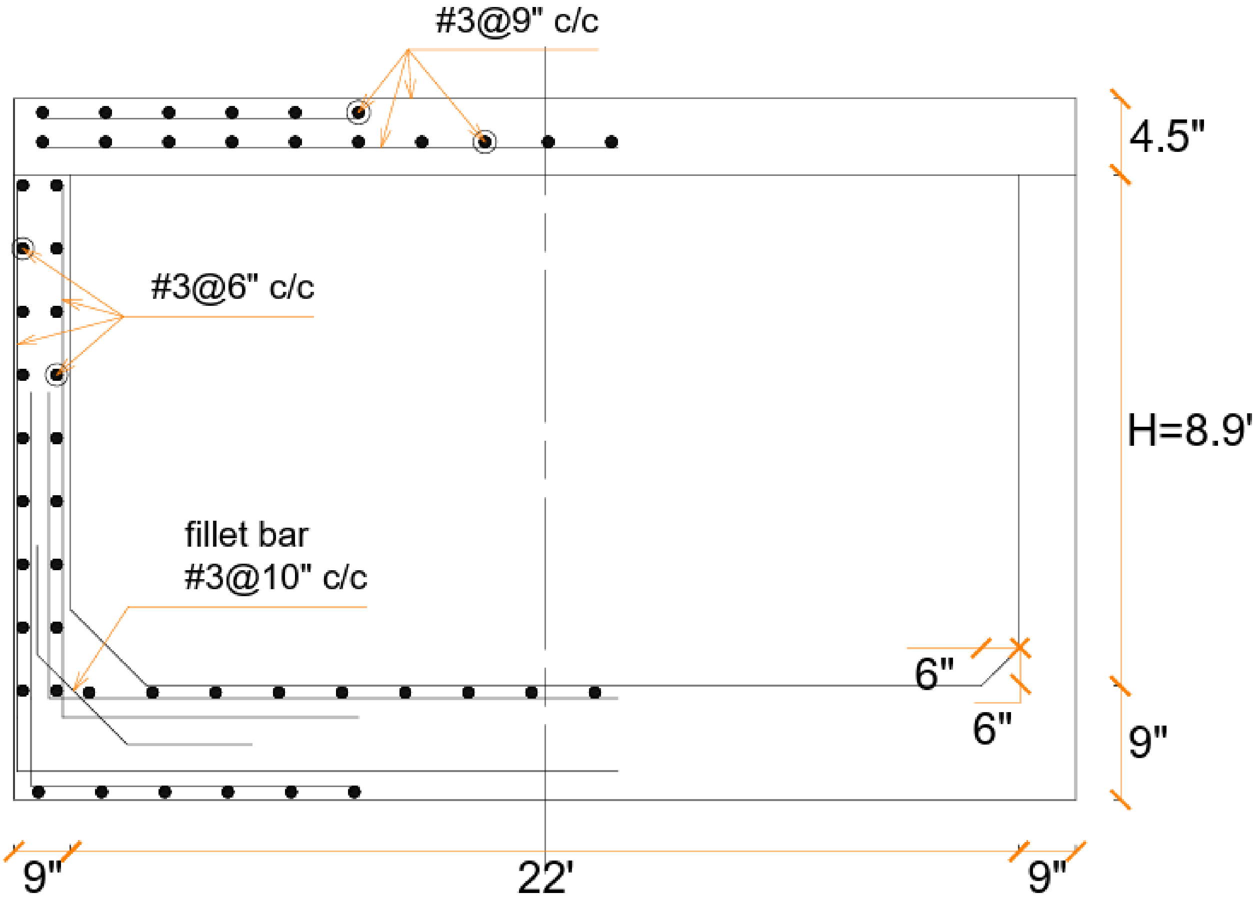
steel requirement (computed)	short side-wall'AB'(L=15 feet)		
location	at ends/edge(inch2/ft)	at mid-span/center	at ends/edge(inch2/ft)
at free face	0.216	0.216	0.216
at water face	0.216	0.216	0.216

(check:the above computed steel should not be less than

As(minimum)=0.002bt=0.002x12 inchx9inch=0.216 inch2/ft width.

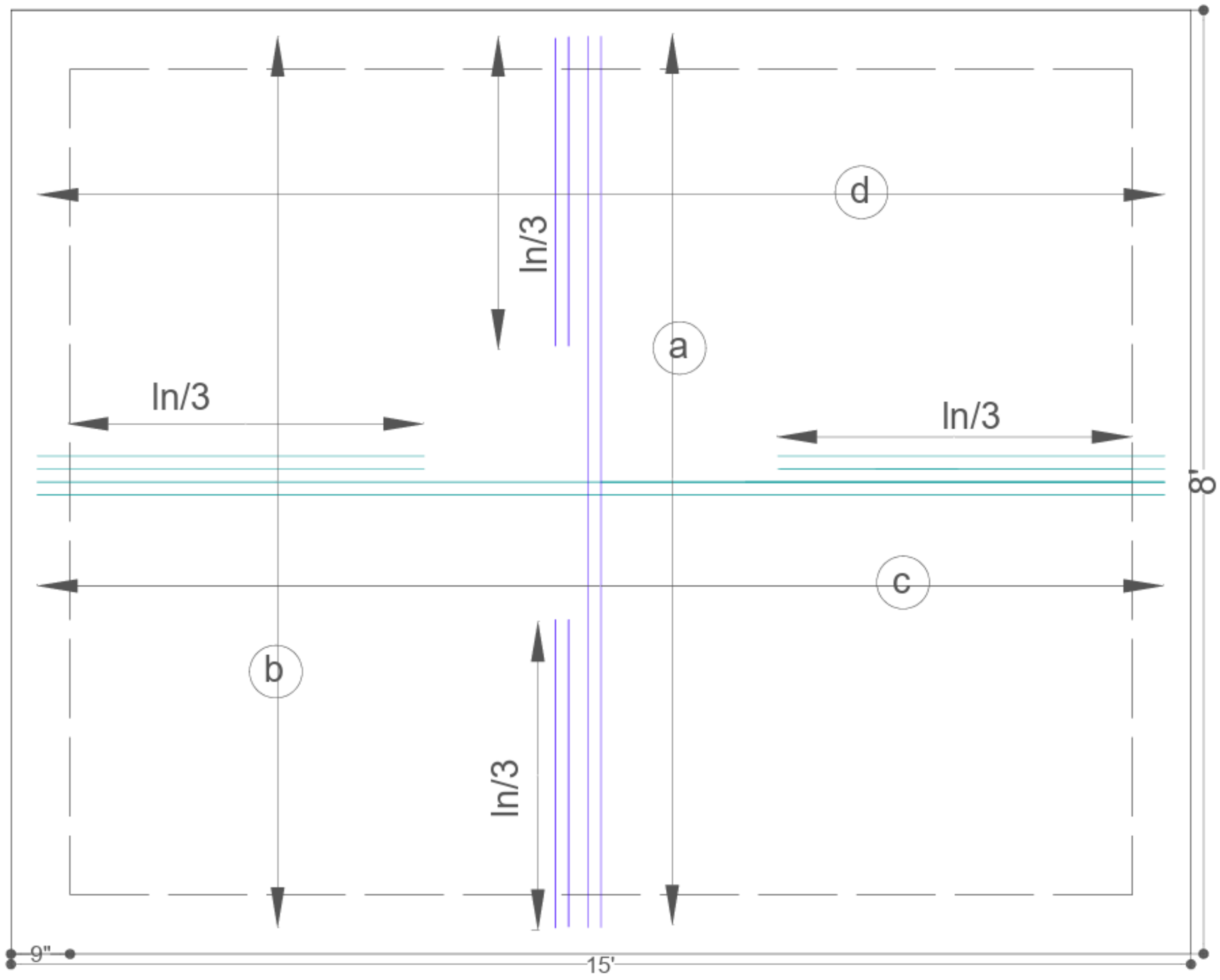
steel spacing provided (using #3 bar)	short side-wall'AD'(B=15feet)		
location	at ends/edge	at mid-span/center	at ends/edge
at free face	#3@6"c/c	#3@6"c/c	#3@6"c/c
at water face	#3@6"c/c	#3@6"c/c	#3@6"c/


#3 @ 6" c/c
#3 @ 6" c/c
#3 @ 6" c/c
#3 @ 6" c/c
#3 @ 6" c/c
#3 @ 6" c/c
15'
22'
9"



Reinforcement details of Side wall

For fire fighting water reservoir minimun reinforcement is satisfied.



Reinforcement details fire fighting water reservoir cover slab

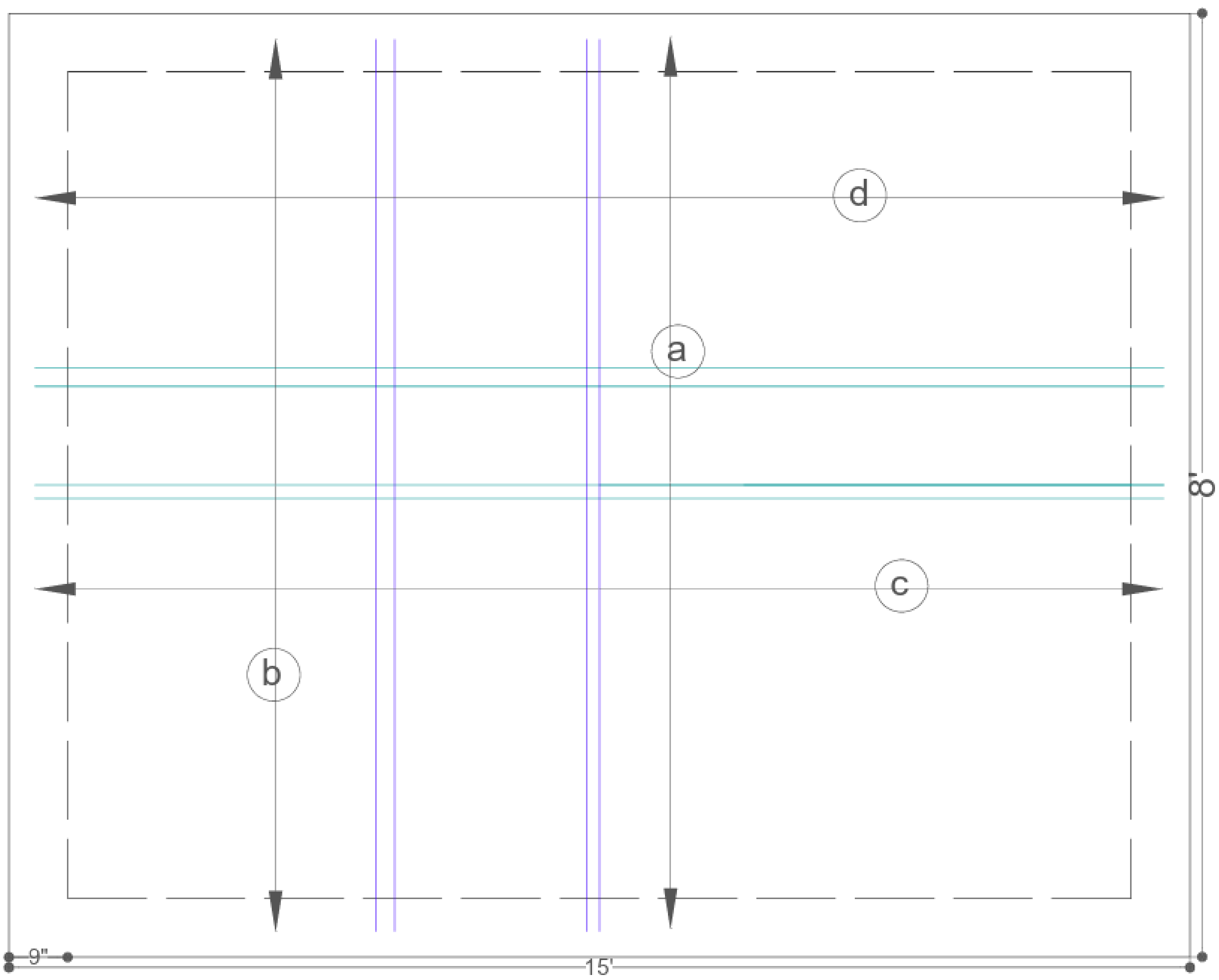
Legends:

(a)#3 @9” c/c at bottom

(b)#3 @9” c/c extra top

(c)#3 @9” c/c at bottom

(d)#3 @9” c/c extra top



Reinforcement details fire fighting water reservoir base slab

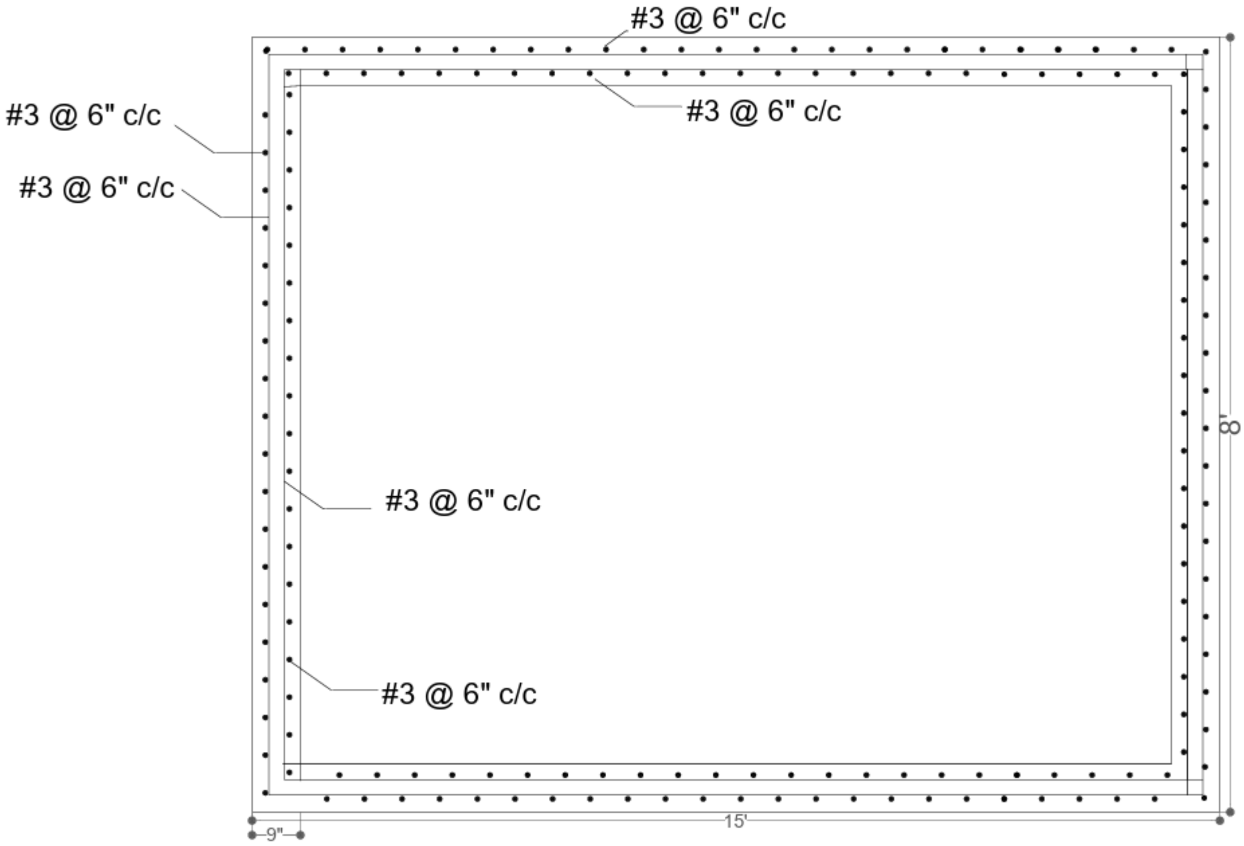
Legends:

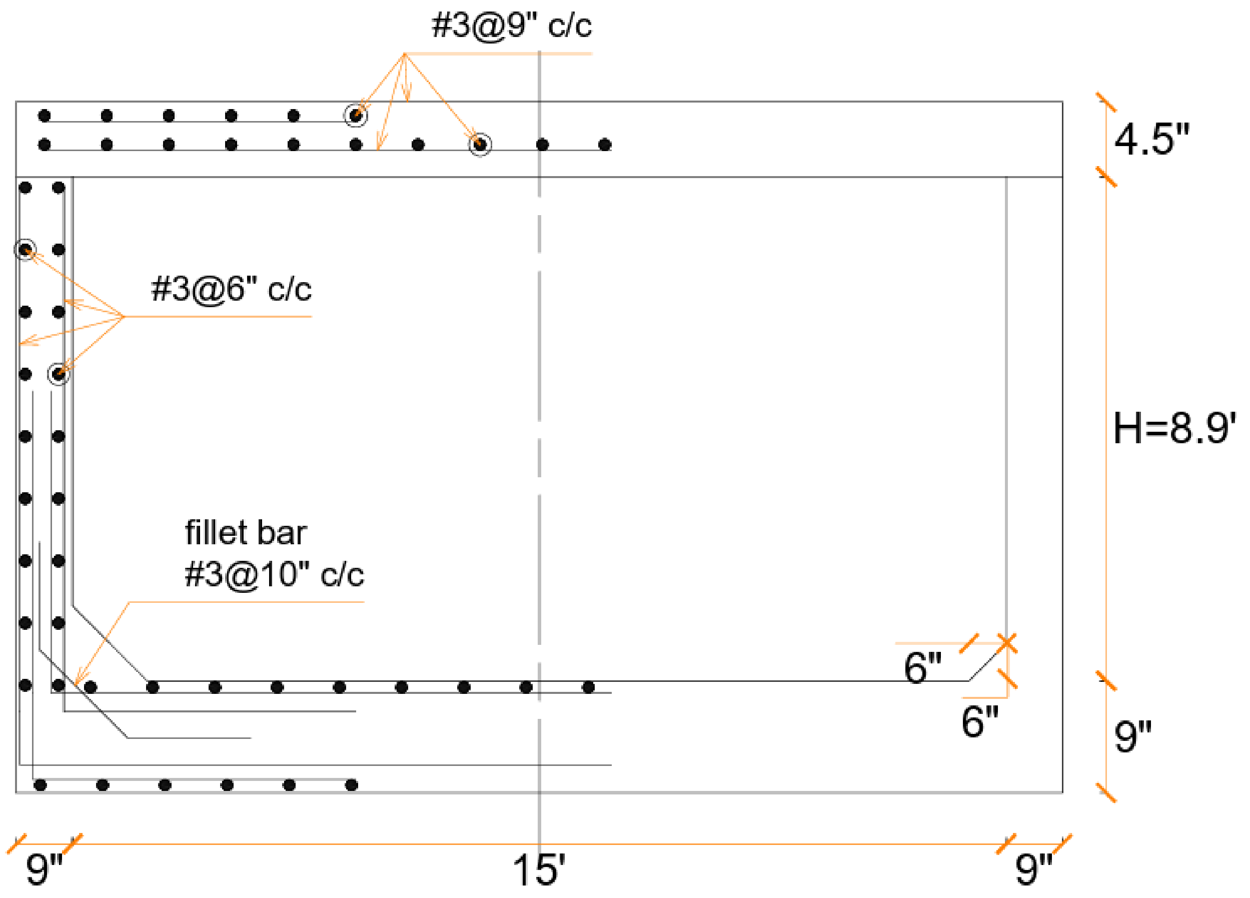
(a)#3 @6” c/c at bottom

(b)#3 @6” c/c at top

(c)#3 @6” c/c at bottom

(d)#3 @6” c/c top


#3 @ 6" c/c
#3 @ 6" c/c
#3 @ 6" c/c
#3 @ 6" c/c
#3 @ 6" c/c
#3 @ 6" c/c
8'
15'
9"



Reinforcement details fire fighting water reservoir side wall

Plumbing and Drainage System:

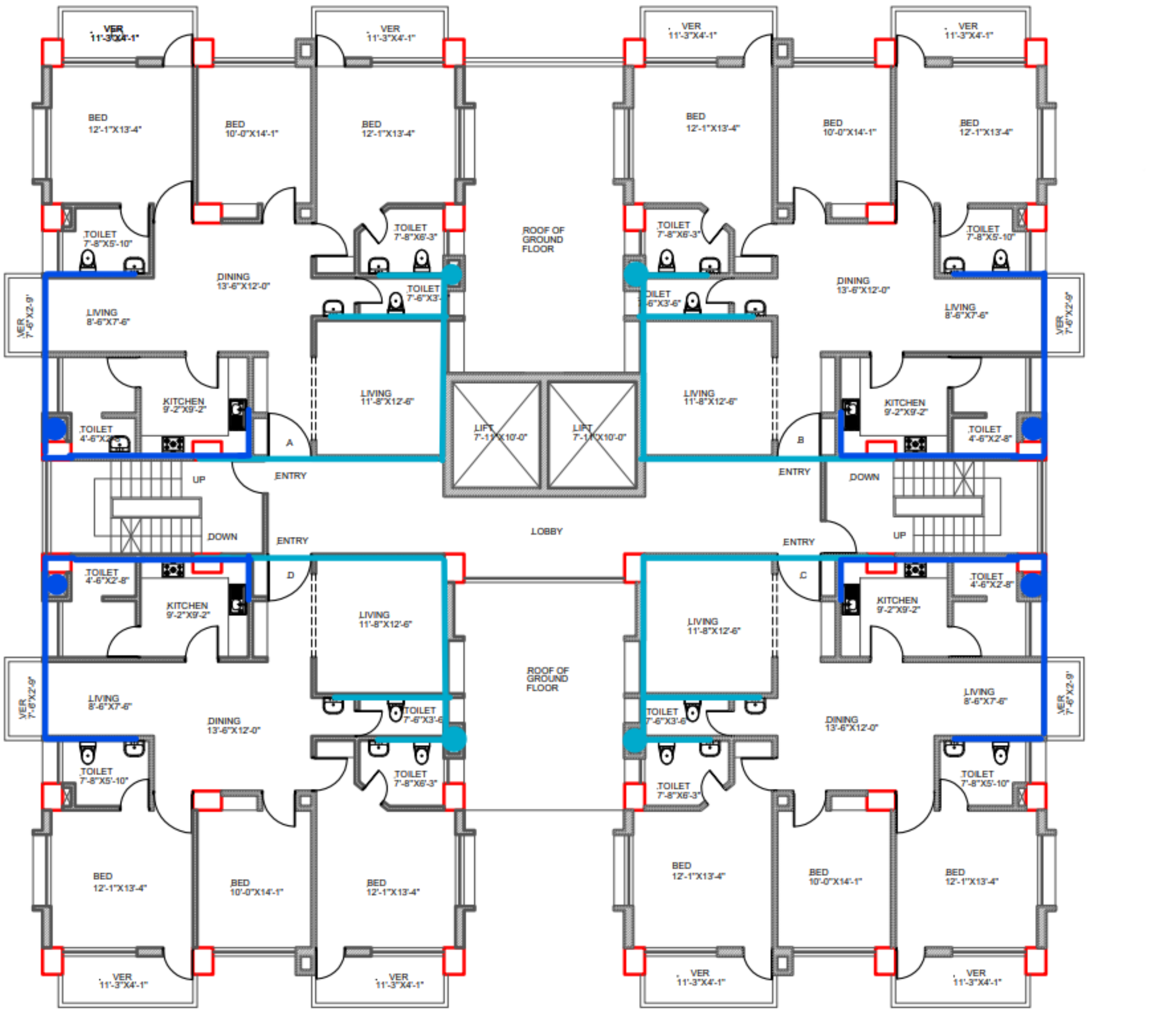


Fig: Branch Layout

WSFU determination:

Room	Fixtures	WSFU
Toilet	• Wash Basin • Shower Head • Water Closet (Flush tank) • Bidet	1 4(Both hot & cold) 5 2
Kitchen	• Kitchen Sink	2
Dinning	• Wash Basin	1

Total WSFU in one apartment =4 toilet+ 1 kitchen+ 1 Dinning= 4*12+2+1=51 WSFU

Our building is 13 storied among which ground floor is used for parking. For cash washing and other purposes, 4 toilet is installed with no shower head.

So total WSFU in ground floor is= 4 toilet= 4*(1+5+2) =32 WSFU
There are 4 units per floor. So total FU of building= 4*12*51+32= 2480 WSFU

For Branch A:

Equivalent length of branches:
Maximum horizontal distance of fixture from distribution pipe= 25 ft
For top floor, vertical height is 10 ft from OH tank and fixtures are placed 5 ft from roof level.
Total pipe length = 25+10+5= 40 ft
Since pipe diameter is not known, we assume head loss at bend is 50% of pipe length.
Total equivalent length for top floor= 1.5*40= 60 ft

Equivalent pipe length : Trial 1				
Floor	Vertical length (ft)	Horizontal length (ft)	Total length(ft)	Equivalent length (ft)
12	10	25	40	60
11	20	25	50	75
10	30	25	60	90
9	40	25	70	105
8	50	25	80	120
7	60	25	90	135
6	70	25	100	150
5	80	25	110	165
4	90	25	120	180
3	100	25	130	195
2	110	25	140	210
1	120	25	150	225
GF	130	4.5	139.5	209.25

Pressure at fixtures:

For downfeed system, P=0 and H is positive. Minimum required pressure, f= 8 psi= 55 KPa
Average available pressure loss (Kpa) Fp= 9.8H(m) -f
Average available pressure loss (psi) Fp= 0.434H(ft) -f
For top floor, available pressure loss, Fp= 9.8*6.096(20ft)-55= 4.7 Kpa = .725 psi= .31 ft

Pressure at fixture				
Floor	Elevation	Allowable head loss (Kpa)	Allowable head loss (psi)	Allowable head loss (ft)
12	20	5	0.725	0.31
11	30	35	5.075	2.2
10	40	65	9.425	4.08
9	50	95	13.775	5.96
8	60	125	18.125	7.85
7	70	155	22.475	9.73
6	80	184	26.68	11.55
5	90	214	31.03	13.44
4	100	244	35.38	15.32
3	110	274	39.73	17.2
2	120	304	44.08	19.09
1	130	334	48.43	20.97
GF	140	364	52.78	22.85

Selection of pipe size:

Selection of pipe size								
zone-1: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
12	60	0.31	5.17	26	52	30	3.5	1.5
11	75	2.2	29.33	26	26	16	2.5	3
zone-2: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
10	90	4.08	45.33	26	130	52	2.5	3.7
9	105	5.96	56.76	26	104	45	2.5	3.9

8	120	7.85	65.42	26	78	37	2.5	4.25
7	135	9.73	72.07	26	52	30	2.5	4.5
6	150	11.55	77	26	26	16	2.5	4.6
4.8								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
5	165	13.44	81.45	26	138	53	2.5	4.8
4	180	15.32	85.11	26	112	47	2.5	4.8
3	195	17.2	88.21	26	86	42	2	4.8
2	210	19.09	90.9	26	60	33	2	4.9
1	225	20.97	93.2	26	34	23	2	4.9
GF	209.25	22.85	109.2	8	8	7	2	5

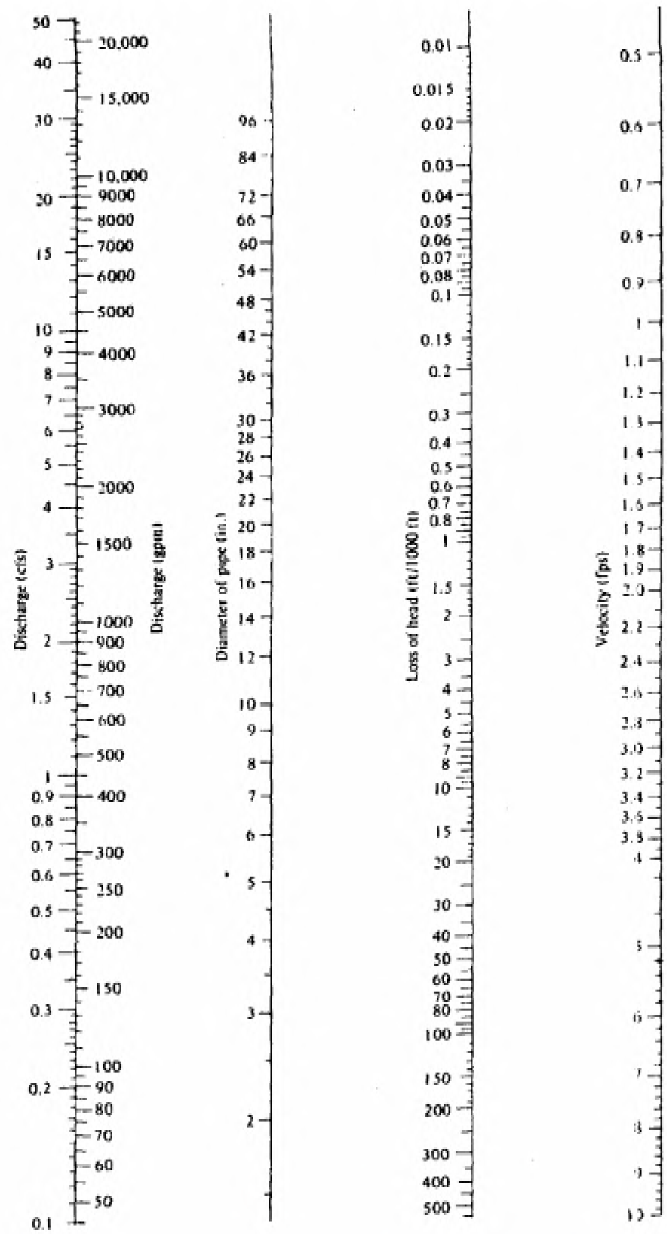


Fig.- P4: Hazen-Williams Nomograph with Roughness Coefficient, C = 100

Trial 2- Actual pipe size determination:

Actual pipe diameter is known, now actual equivalent length of different fittings can be determined.
From plan view, for the longest brunch, we find 3 90 degree bends.
We assume, for each fixture 2 90 degree bend and 1 t joint is required.
Our longest brunch connects with three fixtures (each toilet has a reservoir so individual fixtures are not considered).
So per floor no of fixtures in the longest brunch:
90 degree bend= 3 + 3*2= 9
T joint= 3*1=3
At ground floor, longest branch will connect with one toilet.
Therefore, 90 degree bend= 1+2*1=3
T joint= 1=1

Equivalent length of fitting										
Floor	Pipe dia (in)	Pipe dia (mm)	90 degree long rad fitting loss for one unit	No of 90 degree long rad	Total equivalent length of 90 degree bend(m)	Straight T-joint fitting loss for one unit(m)	No of T joint	Total equivalent length of T joint(m)	Total equivalent length(m)	Total equivalent length(ft)
12	3.5	100	2.04	9	18.36	1.1	3	3.3	21.66	71
11	2.5	75	1.53	9	13.77	0.8	3	2.4	16.17	53
10	2.5	75	1.53	9	13.77	0.8	3	2.4	16.17	53
9	2.5	75	1.53	9	13.77	0.8	3	2.4	16.17	53
8	2.5	75	1.53	9	13.77	0.8	3	2.4	16.17	53
7	2.5	75	1.53	9	13.77	0.8	3	2.4	16.17	53
6	2.5	75	1.53	9	13.77	0.8	3	2.4	16.17	53
5	2.5	75	1.53	9	13.77	0.8	3	2.4	16.17	53
4	2.5	75	1.53	9	13.77	0.8	3	2.4	16.17	53
3	2	75	1.53	9	13.77	0.6	3	1.8	15.57	51.1
2	2	63	1.25	9	11.25	0.6	3	1.8	13.05	42.8
1	2	63	1.25	9	11.25	0.6	3	1.8	13.05	42.8
GF	2	63	1.25	3	3.75	0.6	1	0.6	4.35	14.3

Equivalent Pipe Length					
Floor	Vertical Legth (ft)	Horizontal length(ft)	Total pipe length(ft)	Fitting equivalent length(ft)	Total equivalent length (ft)
12	10	25	40	71	111.0
11	20	25	50	53	103.0
10	30	25	60	53	113.0
9	40	25	70	53	123.0
8	50	25	80	53	133.0
7	60	25	90	53	143.0
6	70	25	100	53	153.0
5	80	25	110	53	163.0
4	90	25	120	53	173.0
3	100	25	130	51.1	181.1
2	110	25	140	42.8	182.8
1	120	25	150	42.8	192.8
GF	130	4.5	139.5	14.3	153.8

Selection of pipe size:

Selection of pipe size								
zone-1: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
12	111	0.31	2.79	26	52	30	4	1.1
11	103	2.2	21.36	26	26	16	3	2.6
zone-2: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
10	113	4.08	36.11	26	130	52	2.5	3.4
9	123	5.96	48.46	26	104	45	2.5	3.6
8	133	7.85	59.02	26	78	37	2.5	4
7	143	9.73	68.04	26	52	30	2.5	4.25
6	153	11.55	75.49	26	26	16	2.5	4.5
zone-3: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
5	163	13.44	82.45	26	138	53	2.5	4.8
4	173	15.32	88.55	26	112	47	2.5	4.8
3	181.1	17.2	94.98	26	86	42	2	4.7
2	182.8	19.09	104.43	26	60	33	2	5
1	192.8	20.97	108.77	26	34	23	2	5.25
GF	153.8	22.85	148.57	8	8	7	2	5.5

For Branch B:

Equivalent length of branches:

Maximum horizontal distance of fixture from distribution pipe= 14.5 ft

For top floor, vertical height is 10 ft from OH tank and fixtures are placed 5 ft from roof level.

At roof tank is 34.5 ft distant from distribution pipe.

Total pipe length = 14.5+34.5+10+5= 64 ft

Since pipe diameter is not known, we assume head loss at bend is 50% of pipe length.

Total equivalent length for top floor= 1.5*64= 96 ft

Floor	Vertical length (ft)	Horizontal length (ft)	Total length(ft)	Equivalent length (ft)
12	10	14.5	64	96
11	20	14.5	74	111
10	30	14.5	84	126
9	40	14.5	94	141
8	50	14.5	104	156
7	60	14.5	114	171
6	70	14.5	124	186
5	80	14.5	134	201
4	90	14.5	144	216
3	100	14.5	154	231
2	110	14.5	164	246
1	120	14.5	174	261
GF	0	0	0	0

Selection of pipe size:

Selection of pipe size								
zone-1: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
12	96	0.31	3.23	25	50	29	4	1.2
11	111	2.2	19.82	25	25	17	3	2.5
zone-2: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
10	126	4.08	32.38	25	125	50	2.5	3.2
9	141	5.96	42.27	25	100	42	2.5	3.4
8	156	7.85	50.32	25	75	37	2.5	3.8
7	171	9.73	56.9	25	50	29	2.5	3.9
6	186	11.55	62.1	25	26	17	2.5	4.25
4.8								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
5	201	13.44	66.87	25	125	50	2.5	4.25
4	216	15.32	70.93	25	100	42	2.5	4.5
3	231	17.2	74.46	25	75	37	2.5	4.5
2	246	19.09	77.6	25	50	29	2.5	4.5
1	261	20.97	80.34	25	25	17	2.5	4.5
GF	0	0	0	0	0	0	0	0

Trial 2- Actual pipe size determination:

Actual pipe diameter is known, now actual equivalent length of different fittings can be determined.
From plan view, for the longest brunch, we find 2 90 degree bends.
We assume, for each fixture 2 90 degree bend and 1 t joint is required.
Our longest brunch connects with three fixtures (each toilet has a reservoir so individual fixtures are not considered).
So per floor no of fixtures in the longest brunch:
90 degree bend= 2 + 3*2= 11
T joint= 3

Equivalent length of fitting										
Floor	Pipe dia (in)	Pipe dia (mm)	90 degree long rad fitting loss for one unit	No of 90 degree long rad	Total equivalent length of 90 degree bend(m)	Straight T-joint fitting loss for one unit(m)	No of T joint	Total equivalent length of T joint(m)	Total equivalent length(m)	Total equivalent length(ft)
12	4	125	2.5	11	27.5	1.2	3	3.6	31.1	102
11	3	88	1.8	11	19.8	0.9	3	2.7	22.5	74
10	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
9	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
8	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
7	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
6	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
5	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
4	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
3	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
2	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
1	2.5	75	1.53	11	16.83	0.8	3	2.4	19.23	63
GF	0	0	0	0	0	0	0	0	0	0

Equivalent Pipe Length					
Floor	Vertical Length (ft)	Horizontal length(ft)	Total pipe length(ft)	Fitting equivalent length(ft)	Total equivalent length (ft)
12	10	14.5	64	102	166.0
11	20	14.5	74	74	148.0
10	30	14.5	84	63	147.0
9	40	14.5	94	63	157.0
8	50	14.5	104	63	167.0
7	60	14.5	114	63	177.0
6	70	14.5	124	63	187.0
5	80	14.5	134	63	197.0
4	90	14.5	144	63	207.0
3	100	14.5	154	63	217.0
2	110	14.5	164	63	227.0
1	120	14.5	174	63	237.0
GF	130	0	0	0	0.0

Selection of pipe size:

Selection of pipe size								
zone-1: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
12	166.0	0.31	1.87	25	50	29	4.5	0.9
11	148.0	2.2	14.86	25	25	17	3	2.2
zone-2: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
10	147.0	4.08	27.76	25	125	50	3	3
9	157.0	5.96	37.96	25	100	42	2.5	3.3
8	167.0	7.85	47.01	25	75	37	2.5	3.5
7	177.0	9.73	54.97	25	50	29	2.5	4
6	187.0	11.55	61.76	25	25	17	2.5	4.25
zone-3: Pipe diameters:								
Floor	Equivalet length (ft)	Allowable head loss (ft)	head loss (ft/1000 ft)	FU	Cumulative FU	Demand (gpm)	Pipe size (inch)	Velocity (fps)
5	197.0	13.44	68.22	25	125	50	2.5	4.5
4	207.0	15.32	74.01	25	100	42	2.5	4.5
3	217.0	17.2	79.26	25	75	37	2.5	4.5
2	227.0	19.09	84.1	25	50	29	2	4.5
1	237.0	20.97	88.48	25	25	17	2	4.75
GF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Design of Riser pipe:

There will be 2 overhead tanks, so 2 riser pipe will be used. Water will be pumped 2 times in a day for one hour.

So waterflow per riser = (Daily water demand/2) liter/hour

= (95040/2) liter/hour

=47520 liter/hour

=12553.5gallon/hour

=209.23 gpm

Assuming velocity 8ft/sec,

From figure p5, for velocity 8ft/sec, corresponding diameter of pipe is 3.5 inches. Corresponding head loss per 100ft is 4.7 lbs/in^2.

Design of pump:

Total length of riser(L)

=130ft (building height) +10ft (basement)+ 20ft (OH tank height from top roof surface)

= 150 ft

Total frictional head loss

=1.5*150/100+ 8 (minimum required pressure at OH tank) +5(minor loss due to bend) psi

=15.25 psi

Frictional head= 15.25*144/62.2= 35.31 ft

Velocity Head= $8^2/(2*32)$= 1 ft

Total head, H = Static Head+ Velocity Head+ Frictional Head

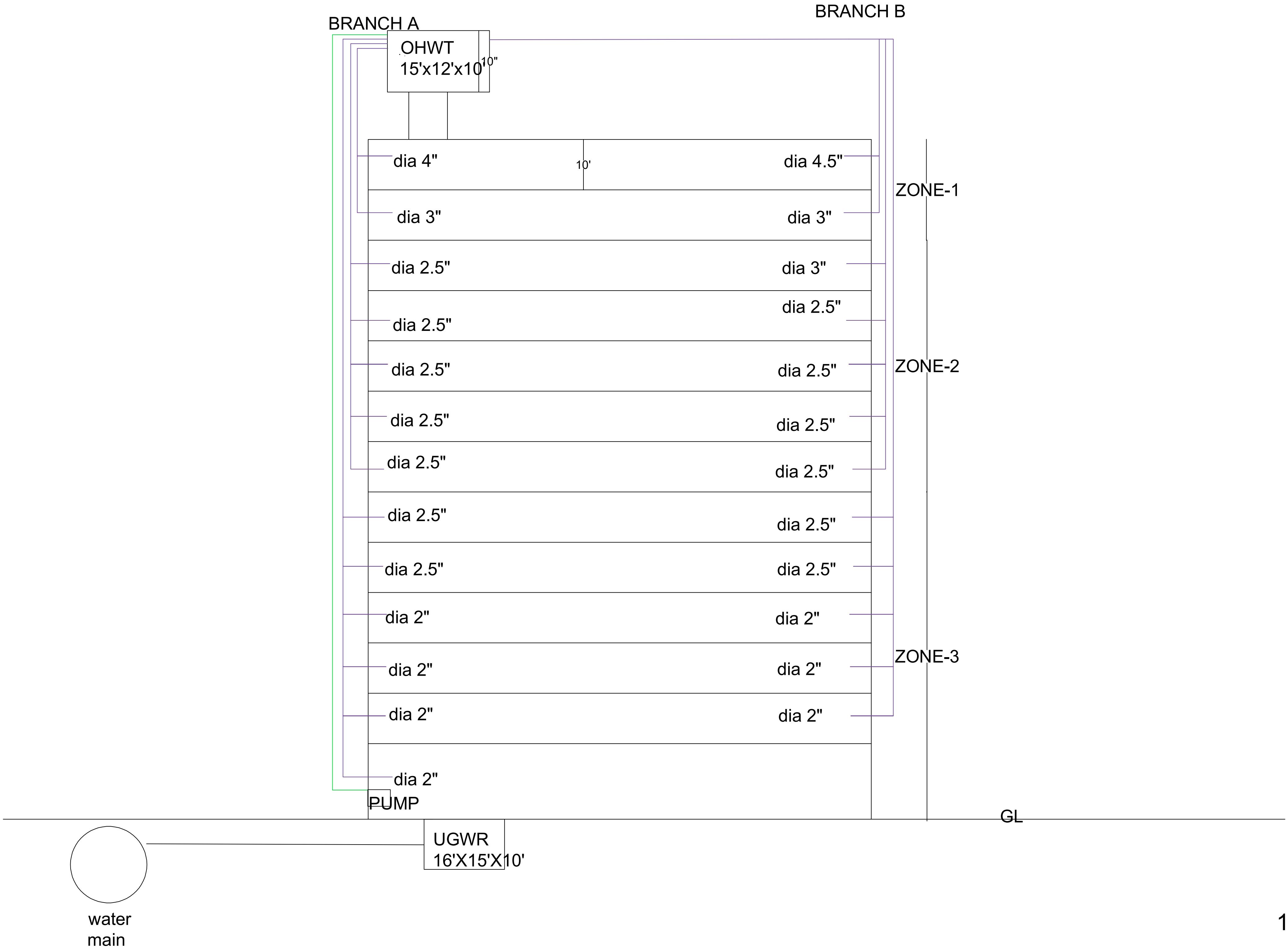
= 150+35.31+1 ft

= 186.31 ft

Let, pump efficiency, E=60%

Capacity of the pump= HQ/3960E=186.31*209.23/.6*3960=16.40 HP=17 HP

Pump capacity= 17 HP


BRANCH A
BRANCH B
OHWT
15'x12'x10'
10"
dia 4"
10'
dia 4.5"
ZONE-1
dia 3"
dia 3"
dia 2.5"
dia 3"
dia 2.5"
dia 2.5"
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ZONE-2
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ZONE-3
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PUMP
GL
UGWR
16'X15'X10'
water
main

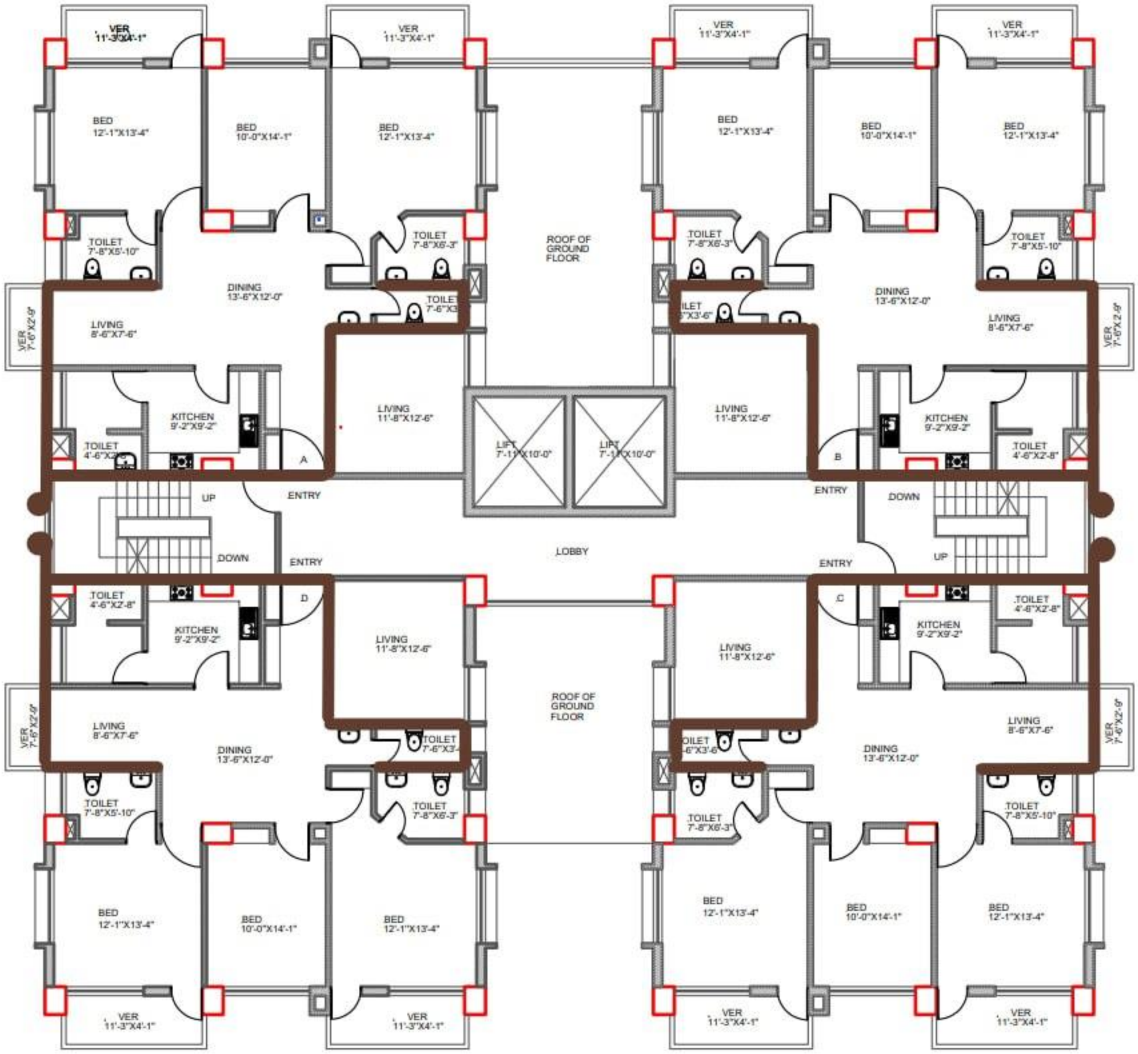


Figure: Soil Branch

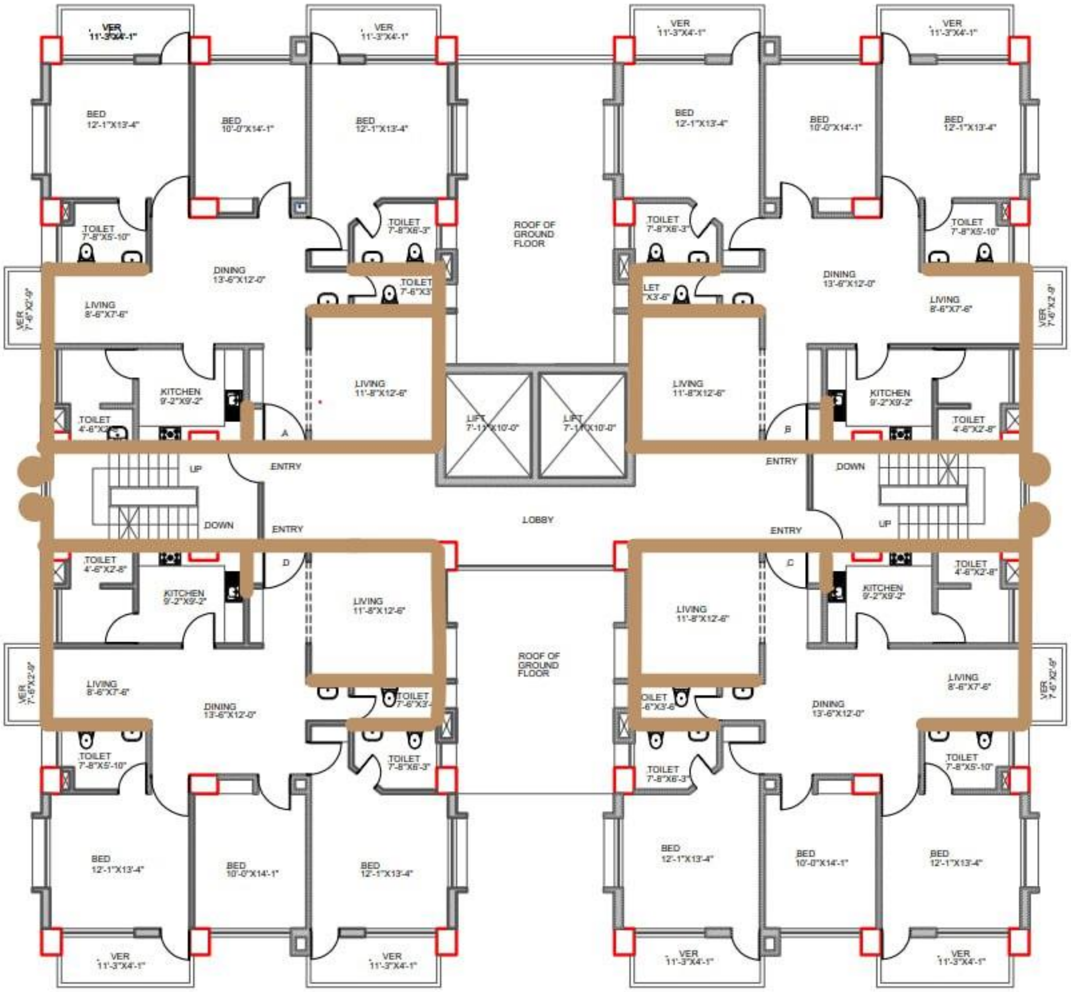


Figure: Waste Branch

Total DFU in different rooms:

Room	Fixture	Drainage Fixture Unit
Toilet	Waste stack fixture: • Wash basin • Shower head • Floor Traps Soil stack fixtures: • Flush tank • Bidet	Waste stack DFU=3+1+1=5 Soil stack DFU=3+2=5
kitchen	• Kitchen sink	2 (waste stack)
Dining room	• Wash basin	1 (waste stack)
Ground floor	Waste stack fixture: • Wash basin • Floor Traps Soil stack fixtures: • Flush tank • Bidet	Waste stack DFU=1+1=2 Soil stack DFU=3+2=5

Total waste stack DFU per apartment= 5x4+2+1=23

Hence per floor waste stack DFU= 23 x 2= 46

Total soil stack DFU per apartment= 5x4=20

Hence per floor soil stack DFU = 20x2 = 40

Diameter and slope of horizontal pipes in each apartment:

For Soil stack: Using single branch:

Pipe Number	Cumulative DFU	Size (mm)	Slope (mm/m)
1-5 (toilet 1)	5	50	20
2-3 (toilet 2)	5	50	20
3-4 (toilet 2+3)	10	65	20
4-5 (toilet 2+3+4)	15	75	10
5-0 (toilet 1+2+3+4)	20	75	10

But designing same branch for different diameters is not economical. Instead, we will use two separate brunch pipes. Toilet 1 and 2 will be connected with vent stack by a 50 mm branch (slope 20 mm/m), while toilet 3, 4 and 5 will connect with a 75 mm pipe (branch ii, slope 20 mm/m).

For Waste stack:

Pipe Number	Cumulative DFU	Size(mm)	Slope(mm/m)
1-2 (toilet 2)	5	50	20
2-4 (toilet 2+3+ dining)	11	65	20
3-4 (kitchen)	2	50	20
4-6 (toilet 2+3+ dining+kitchen)	13	75	10
6-7 (toilet 2+3+4+ dining+kitchen)	18	75	10
5-7 (toilet 1)	5	50	20
7-0 (toilet 1+2+3+4+ dining+kitchen)	23	100	10

But designing same branch for different diameters is not economical. Instead, we will use two separate brunch pipes. Toilet 2,3, dining and kitchen will be connected with vent stack by a 65 mm branch (slope 20 mm/m), while toilet 1 will connect with a 50 mm pipe (branch ii, slope 20 mm/m).

Table: Diameter of Vertical Pipes						
Floor	**Soil Stack (minimum 100mm diameter)**			**Waste Stack (minimum 75 mm diameter)**		
	Total Fixture Unit	**Fixture Connected to that Story**	**Diameter, mm**	**Total fixture unit**	**Fixture Connected to that Story**	**Diameter, mm**
12	20	20	100	46	46	125
11	40	20		92	46	
10	60	20		138	46	
9	80	20		184	46	
8	100	20		230	46	
7	120	20		276	46	
6	140	20		322	46	
5	160	20		368	46	
4	180	20		414	46	
3	200	20		460	46	
2	220	20		506	46	
1	240	20		552	46	
GF	250	10		556	4	

Sample Calculation:

For soil stack, development length at 12th floor

= 63.5’ (furthest point from soil stack) + 10’ (Floor height) +6.5’ (above roof soil stack height)

= 80’ = 24.38 meter

For waste stack, development length at 12th floor

= 63.5’ (furthest point from waste stack) + 10’ (Floor height) +6.5’ (above roof waste stack height)

= 80’ = 24.38 meter

Minimum diameter of vent stack for waste pipe

= max of (25mm, 2/3 x branch waste pipe dia)

= max of (25, 2/3 *100)

= 66.67 mm = 75 mm

Table: Diameter of Vent Stacks										
Floor	Vent Stack for Soil Stack					Vent Stack for Waste Stack				
	Total Fixture Unit	Fixture Connected to that Story	Diameter, mm	Development Length, m	Diameter, mm(Minimum 75 mm)	Total fixture unit	Fixture Connected to that Story	Diameter, mm	Development Length, m	Diameter, mm(Minimum 75 mm)
12	250	20	100	24.384	75	556	46	125	24.384	100
11	230	20		27.432	75	510	46		27.432	100
10	210	20		30.48	75	464	46		30.48	100
9	190	20		33.528	75	418	46		33.528	100
8	170	20		36.576	75	372	46		36.576	100
7	150	20		39.624	75	326	46		39.624	100
6	130	20		42.672	75	280	46		42.672	100
5	110	20		45.72	75	234	46		45.72	100
4	90	20		48.768	75	188	46		48.768	100
3	70	20		51.816	75	142	46		51.816	100
2	50	20		54.864	75	96	46		54.864	100
1	30	20		57.912	75	50	46		57.912	100
GF	10	10		42.97	75	4	4		42.97	100

Sample Calculation:

For soil stack, development length at 12th floor

= 63.5’ (furthest point from soil stack) + 10’ (Floor height) +6.5’ (above roof soil stack height)

= 80’ = 24.38 meter

For waste stack, development length at 12th floor

= 63.5’ (furthest point from waste stack) + 10’ (Floor height) +6.5’ (above roof waste stack height)

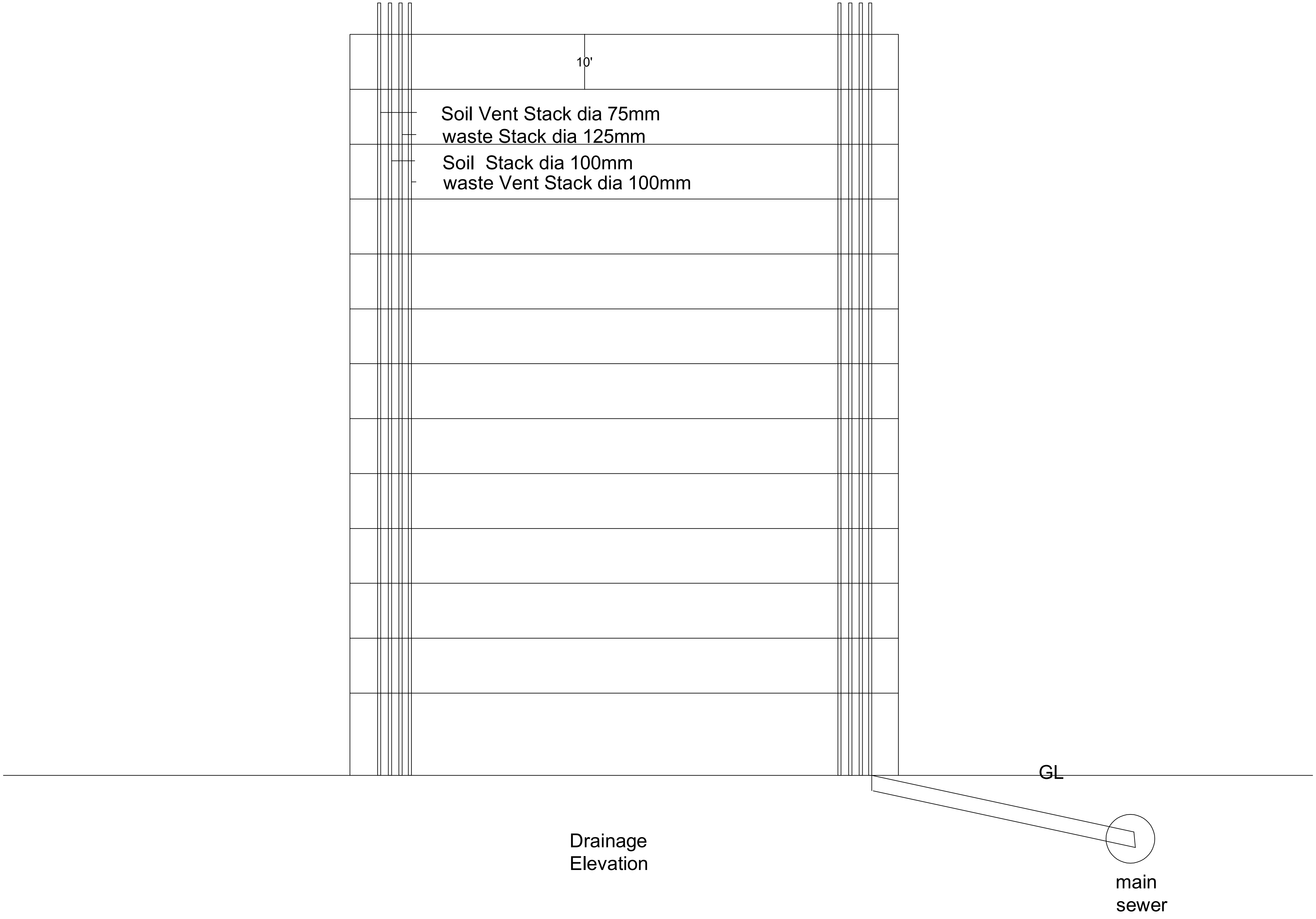
= 80’ = 24.38 meter

Minimum diameter of vent stack for waste pipe

= max of (25mm, 2/3 x branch waste pipe dia)

= max of (25, 2/3 *100)

= 66.67 mm = 75 mm



Drainage
Elevation