BSC	**Design Calculation or Analysis Cover Sheet** *Complete only applicable items.*	1. QA: QA 2. Page 1

3. System Receipt Facility	4. Document Identifier 200-DBC-RF00-00200-000-00C
5. Title Receipt Facility Concrete Diaphragm Design	
6. Group Civil/Architectural/Structural (CSA)	
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8. Notes/Comments Table of Contents, Sections 1-4, & 7 – Checked by Surendra Goel Sections 5, 6, & Attachment A – Checked by Anant Varkekar	

Attachments	Total Number of Pages
Attachment A – RF Bldg. Roof and Floor Plans	6

RECORD OF REVISIONS

9. No.	10. Reason For Revision	11. Total # of Pgs.	12. Last Pg. #	13. Originator (Print/Sign/Date)	14. Checker (Print/Sign/Date)	15. EGS (Print/Sign/Date)	16. Approved/Accepted (Print/Sign/Date)
00A	Initial Issue	152	B6	R. Nelson	R. Baylosis	J. Bisset	R. Rajagopal
00B	Taking corrective action this calculation is revised to address CR 10960. The tables in Section 6.5 (pgs 121 and 122) are corrected to agree with the body of the calculation and the calculation of the diaphragm shear reinforcing is corrected. Incorrect original material is deleted. Minor editorial and typographical changes made. Corrected metal deck thickness. The required bending reinforcing is also recalculated to permit the accurate determination of the required diaphragm plus bending reinforcing. The procedures referenced on pgs. 4 and 5 are updated to the current revisions and the calculation is brought up to the current calculation requirements. Affects pages: 4, 5, 6, 8, 10, 11, 14, 16, 19, 20, 26, 28, 34, 40, 46, 52, 54 thru 59, 60, 63, 66, 68, 70, 72, 76, 78, 80, 84, 87, 89, 93, 95, 98, 99, 101, 106, 108, 109, 110, 112, 113, 115, 116, 118, 119, 121, 122, B1	152	B6	John Bisset	Dariusz Karpinski	Thomas Frankert	Raj Rajagopal
00C	Taking corrective action this calculation is revised to address CR 11530. The entire calculation is completely revised and reformatted. Attachments A & C from the previous calculation have been removed and Attachment B is renamed to Attachment A	77	A6	Tyson Day Tyson Day 1/29/08	Surendra Goel S C Goel 1/29/08 Anant Varkekar A Varkekar 1/29/08	Thomas Frankert Thomas Frankert 1/29/08	Raj Rajagopal R Rajagopal 1/29/08

DISCLAIMER

CONTENTS

1.0 PURPOSE

The purpose of this calculation is for the preliminary design of the concrete slabs and diaphragms for the Receipt Facility (RF). See the figures in Attachment A for plans and elevations of the RF structure. In this calculation a number of slabs representative of all the diaphragms will be designed. This number includes the following cases.

Case 1	Roof Diaphragm @ El. 100' : Col Lines 3-6/C-E (N/S & E/W dir):	18" slab
Case 2	Roof Diaphragm @ El. 72' : Col Lines 6-8/C-E (N/S & E/W dir):	18" slab
Case 3	Roof Diaphragm @ El. 64' : Col Lines 4-6/A-C (N/S dir) & 3-8/A-C (E/W dir): Also applies to Col Lines 4-6/E-F (N/S dir) & 3-8/E-F (E/W dir).	18" slab
Case 4	Roof Diaphragm @ El. 64' : Col Lines 2-3/C-E (N/S & E/W dir):	18" slab
Case 5	Roof Diaphragm @ El. 32' : Col Lines 8-9/C-E (N/S & E/W dir):	18" slab
Case 6	Floor Diaphragm @ El. 32' : Col Lines 4-6/A-C (N/S dir) & 3-8/A-C (E/W dir): Also applies to Col Lines 4-6/E-F (N/S dir) & 3-8/E-F (E/W dir).	18" slab
Case 7	Floor Diaphragm @ El. 32' : Col Lines 3-4/A-B (N/S & E/W dir):	18" slab
Case 8	Floor Diaphragm @ El. 32' : Col Lines 2-3/C-E (N/S & E/W dir):	18" slab
Case 9	Floor Diaphragm @ El. 32' : Col Lines 3-4/C-D (N/S & E/W dir): Also applies to Col Lines 3-6/C-E (N/S & E/W dir).	48" slab

2.0 REFERENCES

2.1 PROCEDURES/DIRECTIVES

2.1.1 BSC 2007. EG-PRO-3DP-G04B-00037, Rev. 10. *Calculations and Analyses*. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071018.0001.

2.1.2 BSC 2007. IT-PRO-0011, Rev. 7. Software Management. Las Vegas, Nevada: Bechtel SAIC Company. ACC: DOC.20070905.0007.

2.1.3 ORD (Office of Repository Development) 2007. Repository Project Management Automation Plan. 000-PLN-MGR0-00200-000, Rev. 00E. Las Vegas, Nevada: U.S. Department of Energy, Office of Repository Development. ACC: ENG.20070326.0019.

2.2 DESIGN INPUTS

2.2.1 ACI 349-01. 2001. *Code Requirements for Nuclear Safety Related Concrete Structures (ACI349-01) and Commentary (ACI 349R-01).* Farmington Hills, Michigan: American Concrete Institute. TIC: 252732. [ISBN 0-87031-041-0]

2.2.2 BSC (Bechtel SAIC Company) 2007. Project Design Criteria Document. 000-3DR-MGR0-00100-000-007. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071016.0005.

2.2.3 BSC (Bechtel SAIC Company) 2007. Basis of Design for the TAD Canister-Based Repository Design Concept. 000-3DR-MGR0-00300-000-001. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071002.0042.

2.2.4 MacGregor, J.G. 1997, Reinforced Concrete, Mechanics and Design. Prentice Hall International Series in Civil Engineering and Engineering mechanics. 3rd Edition. Upper Saddle River, New Jersey: Prentice Hall. TIC: 242587. [ISBN 0-13-233974-9]

2.2.5 Not Used

2.2.6 Not Used

2.2.7 BSC (Bechtel SAIC Company) 2004. *Analysis (SASSI) for Sample In-Structure Response Spectra for CHF with Two Closure Cells.* 190-SYC-SY00-01000-000-00A. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20041101.0001.

2.2.8 BSC (Bechtel SAIC Company) 2007. RF Seismic Analysis. 200-SYC-RF00-00400-000-00C. Las Vegas, Nevada: Bechtel SAIC Company. ACC ENG.20071228.0001.

2.2.9 BSC (Bechtel SAIC Company) 2007. *Seismic Analysis and Design Approach Document.* 000-30R-MGR0-02000-000-001. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071220.0029.

2.2.10 BSC (Bechtel SAIC Company) 2007. CRCF, IHF, RF, & WHF Port Slide Gate Mechanical Equipment Envelope. 000-MJ0-H000-00301-000 REV 00B. Las Vegas, Nevada: Bechtel SAIC Company. ACC: ENG.20071101.0015.

2.3 DESIGN CONSTRAINTS

None.

2.4 DESIGN OUTPUTS

The results of this calculation will be used for future structural drawings and calculations associated with the RF concrete slabs.

3.0 ASSUMPTIONS

3.1 ASSUMPTIONS REQUIRING VERIFICATION

3.1.1 Structural steel framing loads are assumed as follows.
Roofs at 32’, 64’, 72’ & 100’, Floors at 32’ (except bounded by column lines 3-6/C-E): 40 lbs/ft^2. Floor at 32’ bounded by column lines 3-6/C-E: 0 lbs/ft^2.

Rationale: Structural steel represents a small fraction of the total loads applied to the RF slabs. 40psf is a reasonable assumed value until the slab support steel has been designed. Actual steel weights will be used as the design matures in the detailed design phase of the project. Floor at 32’ bounded by column lines 3-6/C-E has no steel framing.

Where used: Assumption used in Section 6.2.2.

3.1.2 Equipment dead loads are assumed as 100 lbs/ft^2 and 10 lbs/ft^2 on the floor slabs and roof slabs, respectively. Equipment dead loads include HVAC equipment, Electrical equipment and so forth.

Rationale: The RF is not an equipment intensive structure with the major equipment for diaphragm design being the HVAC equipment. 100 lbs/ft^2 and 10 lbs/ft^2 are a reasonable assumption for this type of structure. Actual equipment weights will be used as the design matures in the detailed design phase of the project.

Where used: Assumption used in Section 6.2.3.

3.1.3 Roofing material dead load is assumed as 55 lbs/ft^2.

Rationale: This is a reasonable assumption that allows for lightweight concrete fill material to be applied over the concrete slab with an average thickness of 6 inches including a waterproof roofing membrane.

Where used: Assumption used in Section 6.2.4.

3.1.4 Live load is assumed as 100 lbs/ft^2 for floor live load and 40 lbs/ft^2 for roof live load.

Rationale: 100 lbs/ft^2 live load for floor and 40 lbs/ft^2 live load for roof are a reasonable assumption for a structure of this type. The primary source of live load is maintenance of HVAC and other equipment.

Where used: Assumption used in Section 6.2.5.

3.1.5 Floor slabs constructed on a 3" metal deck are assumed to have maximum span of 7'-0"

Rationale: It is a standard engineering practice to have spans of 6’ to 7’ for floor slabs constructed on a 3" metal deck. In the subsequent design the actual maximum slab span will be used for the diaphragm design.

Where used: Assumption used in Section 6.3.

3.1.6 The amplified slab acceleration for out-of-plane seismic loads is assumed as 2.0 times the slab acceleration obtained from the RF seismic analysis (Ref. 2.2.8).

Rationale: The tier-1 seismic analysis models did not include the effects of vertical floor flexibility, i.e. the floors were considered as rigid diaphragms. To obtain amplified vertical floor accelerations to be used in the design of floor slabs and supporting steel the following process was used.

A SASSI (System for Analysis of Soil-Structure Interaction) analysis was performed on the Canister Handling Facility (CHF) (Ref. 2.2.7) which developed in-structure response spectra at hard points on the walls. RF structure is similar to CHF structure. Using the 7% damped vertical response spectra given in fig. F-3 of Ref. 2.2.7, a response ratio between the wall ZPA (Zero Period Acceleration) and the in-structure response was computed at various frequencies. A plot was generated of response ratio versus frequency.

A study was performed for the CHF where floor frequencies were computed for various slab geometry’s (Ref. 2.2.7). Looking at the results of this study one can determine the fundamental vertical floor mode and obtain the frequency and mass participation for the various conditions studied. For an 18" floor with columns spaced at approx. 20' on centers the fundamental mode is approximately 25Hz with a mass participation of 50%. Thus 50% of the mass is responding at this frequency and 50% of the mass responds at the ZPA. The following equation may be written:

$$response = (.5 * mass * ZPA) + (.5 * Ratio * mass * ZPA)$$

Where *Ratio* = Acceleration @ 25 Hz / ZPA

Using the Response Ratio versus frequency plot described above the ratio for 25Hz was found to be 2.3. Using this value in the response equation above results in:

$$response = .5 * mass * ZPA + .5 * 2.3 * mass * ZPA$$

$$response = 1.65 * mass * ZPA$$

Where ZPA for the slab is the acceleration obtained from the RF seismic analysis (Ref. 2.2.8) at the floor level under consideration.

This procedure was done for various slabs and the results indicated that 2.0*ZPA is a reasonable approximation of the vertical floor amplification for this type of structural configuration.

Where used: Assumption used in Section 6.2.6

3.1.7 The plan, elevation and dimensions on the figures of Attachment A are used as the basis for the structural configuration of the RF building to be used in this calculation. These figures are based on Plant Design engineering sketches, 200-P0K-MGR0-10301-000-00B, 200-P0K-MGR0-10401-000-00B, 200-P0K-MGR0-10501-000-00B, 200-P0K-MGR0-10601-000-00B, 200-P0K-MGR0-10701-000-00B, Receipt Facility (RF) Preliminary Layout Ground Floor Plan, Second Floor Plan, Third Floor Plan, Section A, and Section B, respectively. The dimensions (including openings) are as shown in Attachment A.

Rationale–The rationale for this assumption is that further refinement of the general arrangements in the plant design engineering sketches (i.e., adding doors and corridors) will not greatly affect the dynamic response of the structure or the related design effort. The development of the general arrangements continues to be refined and the referenced sketches have been superseded; however the referenced sketches form the basis for the tier 1 analysis and related design work; the major rooms and wall locations and sizes are not expected to change. Properties computed in this calculation are adequate for the current initial design and analysis work. Before the tier 2 analysis and design effort begins the configurations will be reviewed against the latest committed or confirmed drawings and revised as required.

Where used: Assumption used in Section 6.4 & 6.5

3.2 ASSUMPTIONS NOT REQUIRING VERIFICATION

3.2.1 All slabs are assumed and designed as one-way slabs.

Rationale: Designing one-way slab instead of two-way slab is bounding. Reinforcing steel computed in the slab span direction will also be provided in the orthogonal direction.

Where used: Assumption used in Section 6.3.4.

3.2.2 Multiple span diaphragms when analyzing for in-plane loads are taken as simple span using the largest span.

Rationale: Taking simple span instead of multiple spans is conservative because moments computed as simple span envelopes the positive and negative moments computed as multiple spans.

Where used: Assumption used in Section 6.4.1 & 6.5.1.

3.2.3 **Wall Openings**

In the calculation (Section 6.2) for wall mass, it is assumed that the entire wall as solid, neglecting the small wall openings.

Rationale: This is a reasonable assumption considering that overall lengths of openings are small compared to the lengths of the walls and the impact on the accuracy of the analyses will be conservative.

4.0 METHODOLOGY

4.1 QUALITY ASSURANCE

This calculation was prepared in accordance with EG-PRO-3DP-G04B-00037, Calculations and Analyses (Ref.2.1.1). The RF has been classified as a structure that is important to safety (ITS), (Section 6.1.2) in the *Basis of Design for the TAD Canister-Based Repository Design Concept* (Ref. 2.2.3). Therefore, the approved version is designated as QA:QA.

4.2 USE OF SOFTWARE

Word 2003, which is part of the Microsoft Office 2003 suite of programs was used in this calculation. Microsoft Office 2003, as used in this calculation, is classified as Level 2 software usage as defined in IT-PRO-0011 (Ref. 2.1.2). Microsoft Office Professional and Microsoft Windows XP are listed on the *Repository Project Management Automation Plan* (Ref. 2.1.3).

Word 2003 was used in the text preparation of this document; no calculation functions contained in Word 2003 were used in this document.

MathCAD Version 13 was utilized to perform design calculations. MathCAD was operated on a PC system running the Windows XP operating system. MathCAD as used in this calculation is considered as level 2 software usage as defined in IT-PRO-0011 (Ref. 2.1.2). MathCAD Version 13 is listed in the *Repository Project Management Automation Plan* (Ref. 2.1.3).

All MathCAD input values and equations are stated in the calculation. Checking of the MathCAD calculations was done by using visual inspection and hand calculations to confirm the accuracy of the results.

4.3 DESIGN METHODOLOGY

Concrete slabs and diaphragms will be designed for the vertical floor loads (dead loads, live loads, equipment loads, steel decking loads, and roofing material loads) applied to the slab as well as the in-plane and out-of-plane loads imposed on the slabs under seismic loading conditions.

The RF structure is ITS (Section 4.1), therefore the design will be based on the requirements of ACI 349-01, Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-01), referred in this document as ACI 349 (Ref. 2.2.1).

Initially the reinforcing requirements for out-of-plane loads are calculated. Subsequently the reinforcing requirements for in-plane (diaphragm) loads are calculated. While calculating the in-plane (diaphragm) loads, any wall that is perpendicular to the direction of the in-plane excitation and its mass contributes an in-plane distributed load along its length to the diaphragm. A wall in the direction of the in-plane excitation acts to restrain the diaphragm and its mass does not contribute to in-plane loads. The results of these analyses are combined to determine the overall reinforcing requirements for the imposed loads. These reinforcing requirements are then compared to the ACI 349 (Ref. 2.2.1) minimum reinforcing requirements. The larger of the reinforcing required for the imposed load and minimum requirements will determine the final reinforcing requirements for the slab. It is recognized that the approach used to determine chord reinforcing steel is approximate, but conservative. From the Tier 2 finite element analysis, it is expected that a more economical distribution of reinforcing steel will be determined.

5.0 LIST OF ATTACHMENTS

	Number of Pages
Attachment A: RF Bldg. Roof and Floor Plans	6

6.0 BODY OF CALCULATION

6.1 UNITS, STRESSES AND VARIABLES

6.1.1 Units

psi : pounds per square inch	psf : pounds per square foot	pcf : pounds per cubic foot
kip : 1000 lbs	klf : kip per linear foot	

6.1.2 Stresses

$f_c := 5000 \cdot psi$	Compressive Strength of Concrete	(Ref. 2.2.2, Sect. 4.2.11.6.2)
$f_y := 60 \cdot ksi$	Yield Stress of Grade 60 Reinforcing Steel	(Ref. 2.2.2, Sect. 4.2.11.6.2)

6.1.3 Variables

As	Area of reinforcing steel (in^2)
b	Width of concrete section (inches)
clr	Clear cover over reinforcing bars (inches)
d	Distance from extreme compression fiber to centroid of reinforcement (inches)
d_{bar}	Diameter of reinforcing bar (inches)
h	Height of section or slab thickness (inches)
span	Span of slab (feet)
w	Uniform applied load on slab (lbs/ft)
w_{conc}	Unit weight of concrete
ρ	Reinforcing ratio = As/(b*d)
ρ_{req}	Computed required reinforcing ratio
ϕ_b	Strength reduction factor for bending
ϕ_{diaph}	Strength reduction factor for in-plane shear (diaphragm shear)
ϕ_s	Strength reduction factor for shear
ω	Reinforcing index = $\rho * (f_y / f_c)$

6.1.3 Variables (Continued)

A_{ch}	Required Chord Reinforcement Area (in^2)
A_{cv}	Shear Area (ft^2)
C	Constant = $\omega(1 - .59\omega)$
CF	Chord Force (kips)
DD	Depth of Diaphragm
E	Seismic Load
EDL	Equipment Dead Load
H	Height of Wall
L	Length of Wall
LL	Design Live Load
M	Moment
RMDL	Roofing Material Dead Load
RR	Reinforcement Ratio
S	Span
SACC	Seismic Acceleration
SDL	Slab Dead Load
SFDL	Steel Framing Dead Load
TDL	Total Dead Load
Tw	Thickness of Wall
U	Ultimate Load
V_c	Nominal Shear Strength Provided by Concrete
Vn	Nominal Shear Strength
Vs	Nominal Shear Strength Provided by Shear Reinforcement
W	Seismic Load per Foot
WW	Weight of Wall

6.2 DESIGN LOADS

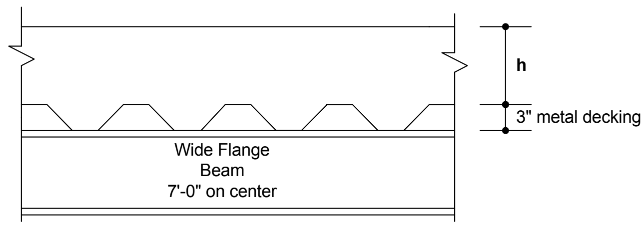
6.2.1 Dead Loads of Concrete Slabs

$w_{conc} := 150 \cdot pcf$ Unit weight of concrete (Reference 2.2.2, Section 4.2.11.6.6)

The roof and floor diaphragms of cases 1 through 8 are 18" thick concrete slabs are on a 3" metal deck as shown in the adjoining sketch:

$$SDL := \left[\left(\frac{h \cdot in}{12} + \frac{0.25 \cdot ft}{2}\right) \cdot w_{conc}\right]$$

where h is the slab thickness above the metal decking in inches



18" roof thick slab $SDL_{18r} := (1.5 \cdot ft + 0.125 \cdot ft) \cdot w_{conc}$ $SDL_{18r} = 244\,psf$ El 100', 72', & 64'

18" floor thick slab $SDL_{18f} := (1.5 \cdot ft + 0.125 \cdot ft) \cdot w_{conc}$ $SDL_{18f} = 244\,psf$ El 32'

Note : The 48" thick floor slab does not have metal deck or the wide flange support beams under the slab.

48" thick floor slab $SDL_{48f} := (4 \cdot ft) \cdot w_{conc}$ $SDL_{48f} = 600\,psf$ El 32'

$$SDL := \begin{pmatrix} SDL_{18r} \\ SDL_{18f} \\ SDL_{48f} \end{pmatrix}$$

Slab Dead Load : 18" roof slab
Slab Dead Load : 18" floor slab
Slab Dead Load : 48" floor slab

$$\boxed{SDL = \begin{pmatrix} 244 \\ 244 \\ 600 \end{pmatrix} psf}$$

6.2.2 Steel Decking Dead Load (Assumption 3.1.1)

Note : There is no steel support decking for the 48" thick floor slab.

$$SFDL := \begin{pmatrix} 40 \\ 40 \\ 0 \end{pmatrix} psf$$

Steel Decking Dead Load : 18" roof slab
Steel Decking Dead Load : 18" floor slab
Steel Decking Dead Load : 48" floor slab

$$\boxed{SFDL = \begin{pmatrix} 40 \\ 40 \\ 0 \end{pmatrix} psf}$$

6.2.3 Equipment Dead Load (Assumption 3.1.2)

$EDL_{fl} := 100psf$ Floor Equipment Dead Load $EDL_{rf} := 10psf$ Roof Equipment Dead Load

$$EDL := \begin{pmatrix} EDL_{rf} \\ EDL_{fl} \\ EDL_{fl} \end{pmatrix}$$

Equipment Dead Load : 18" roof slab
Equipment Dead Load : 18" floor slab
Equipment Dead Load : 48" floor slab

$$\boxed{EDL = \begin{pmatrix} 10 \\ 100 \\ 100 \end{pmatrix} psf}$$

6.2.4 Roofing Material Dead Load (Assumption 3.1.3)

$$RMDL := \begin{pmatrix} 55 \\ 0 \\ 0 \end{pmatrix} psf$$

Roofing Material Dead Load : 18" roof slab
Roofing Material Dead Load : 18" floor slab
Roofing Material Dead Load : 48" floor slab

$$\boxed{RMDL = \begin{pmatrix} 55 \\ 0 \\ 0 \end{pmatrix} psf}$$

6.2.5 Design Live Load (Assumption 3.1.4)

$LL_{fl} := 100 \cdot psf$ Floor Design Live Load $LL_{rf} := 40 \cdot psf$ Roof Design Live Load

$$LL := \begin{pmatrix} LL_{rf} \\ LL_{fl} \\ LL_{fl} \end{pmatrix}$$

Design Live Load : 18" roof slab
Design Live Load : 18" floor slab
Design Live Load : 48" floor slab

$$\boxed{LL = \begin{pmatrix} 40 \\ 100 \\ 100 \end{pmatrix} psf}$$

6.2.6 Acceleration Factors for Seismic Loads (Ref. 2.2.8, Table 14)

Note : The amplified slab acceleration for out-of-plane seismic loads is 2xVertical acceleration at floor level. (Assumption 3.1.6)

Acceleration factors for Elevation 100'

$a100_x := 1.45$ (East/West)
$a100_y := 1.60$ (North/South)
$a100_z := 2 \cdot 0.79$ $a100_z = 1.58$ (Vertical)

Acceleration factors for Elevation 72'

$a72_x := 0.97$ (East/West)
$a72_y := 1.14$ (North/South)
$a72_z := 2 \cdot 0.72$ $a72_z = 1.44$ (Vertical)

Acceleration factors for Elevation 64'

$a64_x := 0.92$ (East/West)
$a64_y := 0.95$ (North/South)
$a64_z := 2 \cdot 0.72$ $a64_z = 1.44$ (Vertical)

Acceleration factors for Elevation 32'

$a32_x := 0.71$ (East/West)
$a32_y := 0.69$ (North/South)
$a32_z := 2 \cdot 0.67$ $a32_z = 1.34$ (Vertical)

6.2.7 Loading Combinations (Ref. 2.2.2, Sect. 4.2.11.4.5)

$U1 := 1.4TDL + 1.7LL$ (Ultimate Load for normal operating condition)

$U2 := TDL + Le + E$ (Ultimate Load for extreme abnormal condition)

Where $Le := 0.25 \cdot LL$ Live load present during an earthquake

Note : Use 25% of design live load (LL) in combination with seismic loads. (Ref. 2.2.9, Sect. 8.3.3)

6.3 SLAB DESIGN FOR OUT-OF-PLANE (VERTICAL) LOADS

See Attachment A for plans and sections of the RF structure.

All 18" slabs are constructed on a 3" metal deck with more than 2 continuous spans with a maximum span of 7'-0" (Assumption 3.1.5) for 18" thick slabs (El. 32', 64', 72' or 100').

48" thick slab bounded by column lines 3-6/C-E does not have a metal deck under the slab. It has more than two continuous spans with the maximum center to center span of 41' and clear spans of 37' (between column lines 3/C-D). The diaphragm and shear walls are monolithic concrete, clear span will be used as per Ref. 2.2.4, Figure 13-23.

6.3.1 Total Dead Loads (TDL)

Total dead loads (TDL) include slab dead load (SDL), steel framing dead load (SFDL), equipment dead load (EDL) and roofing material dead load (RMDL).

$TDL := SDL + SFDL + EDL + RMDL$

$$\boxed{TDL = \begin{pmatrix} 349 \\ 384 \\ 700 \end{pmatrix} psf}$$

Total Dead Load : 18" roof slab
Total Dead Load : 18" floor slab
Total Dead Load : 48" floor slab

6.3.2 Seismic Load (E)

Note : Use 25% of design live load (Le = 0.25LL, see Section 6.2.7) in combination with dead loads in calculating seismic loads. (Ref. 2.2.9, Sect. 8.3.3)

$$SACC_Z := \begin{pmatrix} a100_Z \\ a32_Z \\ a32_Z \end{pmatrix} \qquad \boxed{SACC_Z = \begin{pmatrix} 1.58 \\ 1.34 \\ 1.34 \end{pmatrix}}$$

Seismic Acceleration factor for 18" slab @ El 100', 72', & 64' in Z dir. Conservatively use the max value at El. 100'
Seismic Acceleration factor for 18" slab @ El 32' in Z dir
Seismic Acceleration factor for 48" floor slab @ El 32' in Z dir

Note: Conservatively consider the acceleration for 18" roof slabs for roof slabs at elevations 100', 72', & 64' to have the acceleration factors for seismic loading of the 100' elevation. The 18" roof and floor slabs located at elevation 32' has the acceleration factor for seismic loading of the 32' elevation

$E := \overrightarrow{\left[(TDL + Le) \cdot SACC_z\right]}$ $\quad E = \begin{pmatrix} 567 \\ 548 \\ 971 \end{pmatrix} psf$

Seismic Load for 18" slab @ El 100' , 72', & 64' in Z direction
Seismic Load for 18" slab @ El 32' in Z direction
Seismic Load for 48" floor slab @ El 32' in Z direction

6.3.3 Governing Vertical (Out-of-Plane) Ultimate Load Combination for Concrete Design (See Section 6.2.7.)

$U1 := 1.4 \cdot TDL + 1.7 \cdot LL$ $\quad U1 = \begin{pmatrix} 556 \\ 707 \\ 1150 \end{pmatrix} psf$

18" roof slab @ El. 100', 72', & 64'
18" slab @ El. 32'
48" floor slab @ El. 32'

$U2 := TDL + Le + E$ $\quad U2 = \begin{pmatrix} 926 \\ 956 \\ 1696 \end{pmatrix} psf$

18" roof slab @ El. 100', 72', & 64'
18" slab @ El. 32'
48" floor slab @ El. 32'

$ORIGIN \equiv 1$

Changing origin for matrices from 0,0 to 1,1

$U := \begin{pmatrix} \max(U1_1, U2_1) \\ \max(U1_2, U2_2) \\ \max(U1_3, U2_3) \end{pmatrix}$ $\quad U = \begin{pmatrix} 926 \\ 956 \\ 1696 \end{pmatrix} psf$

Gov Ult Load Comb. D+Le+E : 18" slab @ 100', 72', & 64'
Gov Ult Load Comb. D+Le+E : 18" slab @ El. 32'
Gov Ult Load Comb. D+Le+E : 48" floor slab @ El. 32'

6.3.4 Design of Slabs for Moments and Shears due to vertical loads

All slabs are designed as one-way slabs. (Assumption 3.2.1)

Using the equations for moments and shears from ACI 349 (Ref. 2.2.1 Sect. 8.3.3) :

Maximum positive moment = $wL^2/14$ (end span : discontinuous end integral with support governs)
Maximum negative moment at supports = $wL^2/10$ (more than two spans) **Governs**

Maximum shear force = 1.15wL/2 (end span governs)

$b := 1 \cdot ft$ Width for a 1' strip

$span_{18} := 7 \cdot ft$ Span for 18" slab. (Assumption 3.1.5)

$span_{48} := 37 \cdot ft$ Span for 48" floor slab, see Section 6.3.

$span := \begin{pmatrix} span_{18} \\ span_{18} \\ span_{48} \end{pmatrix}$ $\quad span = \begin{pmatrix} 7 \\ 7 \\ 37 \end{pmatrix} ft$

Span 18" roof slab
Span 18" floor slab
Span 48" floor slab

$$M_{max} := \overrightarrow{\frac{U \cdot b \cdot span^2}{10}} \qquad \boxed{M_{max} = \begin{pmatrix} 4535 \\ 4687 \\ 232251 \end{pmatrix} lbf \cdot ft} \text{ per ft}$$

Moment 18" slab @ El. 100', 72', & 64'
Moment 18" slab @ El. 32'
Moment 48" floor slab @ El. 32'

$$V_{max} := \overrightarrow{\frac{1.15 \cdot U \cdot b \cdot span}{2}} \qquad \boxed{V_{max} = \begin{pmatrix} 3725 \\ 3850 \\ 36093 \end{pmatrix} lbf} \text{ per ft}$$

Shear 18" slab @ El. 100', 72', & 64'
Shear 18" slab @ El. 32'
Shear 48" floor slab @ El. 32'

6.3.5 Check Shear Reinforcement Requirements

For 18" & 48" thick slabs the effective depth, d, can be calculated as:

d = h - 3/4" (clr) - 1.5xd_{bar} where h is the slab thickness above the metal deck
For a # 11 reinforcing bar d_{bar} = 1.41", this results in an effective depth, d, of 15.14" for 18" slabs.

The 48" thick slab has a depression of 18" in some areas (Ref. 2.2.10). For this condition the effective depth of the rebar, d = 48" - 18" - 0.75" - 1.5x1.41" = 27.14"

$d_{18r} := 15.14 \cdot in \quad d_{18f} := 15.14 \cdot in \quad d_{48f} := 27.14 \cdot in$

$$d := \begin{pmatrix} d_{18r} \\ d_{18f} \\ d_{48f} \end{pmatrix} \qquad \boxed{d = \begin{pmatrix} 15.14 \\ 15.14 \\ 27.14 \end{pmatrix} in}$$

Effective depth of rebar for 18" slab @ El. 100', 72', & 64'
Effective depth of rebar for 18" slab @ El. 32'
Effective depth of rebar for 48" floor slab @ El. 32'

<u>Shear strength of concrete,</u> $\phi_s V_c$:

$\phi_s := .85$ Strength reduction factor for transverse shear ACI 349 (Ref. 2.2.1, Sect. 9.3.2.3)

$b = 12\,in$ See Sect 6.3.4.

From ACI 349 Ref. 2.2.1, Sect. 11.3.1.1

$$\phi_{sV.c} := 2 \cdot \phi_s \cdot \sqrt{f_c \cdot psi} \cdot b \cdot d \qquad \boxed{\phi_{sV.c} = \begin{pmatrix} 21839 \\ 21839 \\ 39149 \end{pmatrix} lbf} \text{ per ft}$$

(Equation 11-3)

Shear Strength 18" slab @ El. 100', 72', & 64'
Shear Strength 18" slab @ El. 32'
Shear Strength 48" floor slab @ El. 32'

See Section 6.3.4 for V_{max}

$$V_{max} = \begin{pmatrix} 3725 \\ 3850 \\ 36093 \end{pmatrix} lbf \text{ per ft}$$

Shear 18" slab @ El. 100', 72', & 64'
Shear 18" slab @ El. 32'
Shear 48" floor slab @ El. 32'

$\phi_s \cdot V_c > V_{max}$, therefore no transverse shear reinforcement is required.

6.3.6 Compute Reinforcement Required for Bending

The moment capacity of a reinforced concrete slab is computed as : (Ref. 2.2.4, Equation 4-15)

$$Mu := \phi_b \cdot f_c \cdot b \cdot d^2 \cdot \omega \cdot (1 - .59 \cdot \omega)$$

Where $\omega := \dfrac{\rho \cdot f_y}{f_c}$ and $\rho := \dfrac{A_s}{b \cdot d}$

$\phi_b := .9$ Strength reduction factor for bending, ACI 349 (Ref. 2.2.1, Sect. 9.3.2.1)

Rearranging to solve for ω: $\omega(1 - .59\omega) := \dfrac{M_u}{\phi_b \cdot f_c \cdot b \cdot d^2}$

Letting the right hand side of the equation to be a constant

$$C1 := \overrightarrow{\frac{M_{max}}{\phi_b \cdot f_c \cdot b \cdot \vec{d}^2}} \qquad C1 = \begin{pmatrix} 0.00440 \\ 0.00454 \\ 0.07007 \end{pmatrix}$$

Constant for 18" roof slab
Constant for 18" floor slab
Constant for 48" floor slab

$C18r := C1_1$ $C18r = 0.00440$ Constant for 18" roof slab

$C18f := C1_2$ $C18f = 0.00454$ Constant for 18" floor slab

$C48f := C1_3$ $C48f = 0.07007$ Constant for 48" floor slab

Solving for ω:

To solve the polynomial in Mathcad first make a guess at the root, then use the root function in Mathcad to solve for the root based on the initial guess.

Solving for $\omega 18r$: 18" roof slab

Try $\omega 18r := .00300$ $f1(\omega 18r) := .59\cdot\omega 18r^2 - \omega 18r + C18r$

$root(f1(\omega 18r), \omega 18r) = 0.00441$ $\omega 18r\_act := root(f1(\omega 18r), \omega 18r)$

$\boxed{\omega 18r\_act = 0.00441}$ Actual Value of $\omega 18r$

Solving for $\omega 18f$: 18" floor slab

Try $\omega 18f := .00300$ $f2(\omega 18f) := .59\cdot\omega 18f^2 - \omega 18f + C18f$

$root(f2(\omega 18f), \omega 18f) = 0.00456$ thus $\omega 18f\_act := root(f2(\omega 18f), \omega 18f)$

$\boxed{\omega 18f\_act = 0.00456}$ Actual Value of $\omega 18f$

Solving for : $\omega 48f$ 48" floor slab

Try $\omega 48f := .00300$ $f3(\omega 48f) := .59\cdot\omega 48f^2 - \omega 48f + C48f$

$root(f3(\omega 48f), \omega 48f) = 0.07323$ thus $\omega 48f\_act := root(f3(\omega 48f), \omega 48f)$

$\boxed{\omega 48f\_act = 0.07323}$ Actual Value of $\omega 30f$

$$\omega := \begin{pmatrix} \omega 18r\_act \\ \omega 18f\_act \\ \omega 48f\_act \end{pmatrix} \qquad \boxed{\omega = \begin{pmatrix} 0.00441 \\ 0.00456 \\ 0.07323 \end{pmatrix}}$$

18" roof slab
18" floor slab
48" floor slab

<u>Required Reinforcing Index</u>

$$\rho_{req} := \frac{\omega\cdot f_c}{f_y} \qquad \boxed{\rho_{req} = \begin{pmatrix} 0.00037 \\ 0.00038 \\ 0.00610 \end{pmatrix}}$$

18" roof slab
18" floor slab
48" floor slab

Note : See Section 6.6.1.3 and 6.6.2.3 for combined reinforcement required for out of plane (vertical) and in-plane (horizontal) loads.

6.4 SLAB DIAPHRAGM DESIGN FOR IN-PLANE (HORIZONTAL) LOADS : ANALYSIS CASES 1 TO 4

6.4.1 Diaphragms for Analysis Cases 1 to 4

See Attachment A for plans of the RF showing the diaphragms.

Steel for diaphragms will be computed for following diaphragm panels in this section.

Case 1: Roof Diaphragm @ El.100':Col Lines 3-6/C-E (N/S & E/W dir): 18" slab
(See Attachment A, Sheet A6)

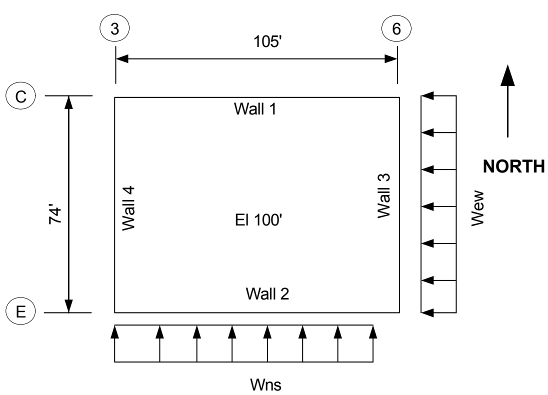
Case 2: Roof Diaphragm @ El. 72': Col Lines 6-8/C-E (N/S & E/W dir): 18" slab
(See Attachment A, Sheet A5)

Case 3: Roof Diaphragm @ El. 64': Col Lines 4-6/A-C (N/S dir) & 3-8/A-C (E/W dir): 18" slab
Also applies to Col Lines:

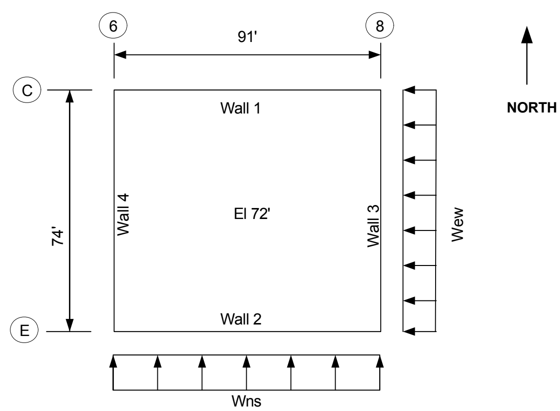
- 3-4/A-C, 6-7/A-C, 7-8/A-C, 3-4/E-F, 4-6/E-F, 6-7/E-F & 7-8/E-F (N/S dir)
- 3-8/E-F (E/W dir).

(See Attachment A, Sheet A4)

Case 4: Roof Diaphragm @ El. 64':Col Lines 2-3/C-E (N/S & E/W dir): 18" slab
(See Attachment A, Sheet A4)

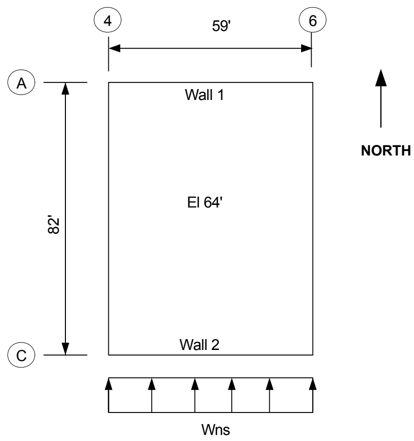


Case 1

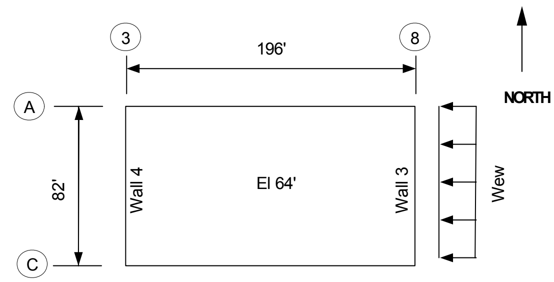


Case 2

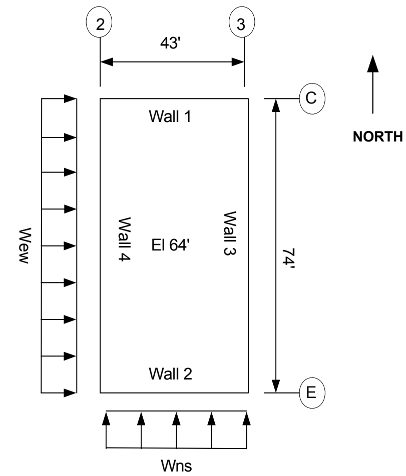
Note : Conservatively take diaphragm as simple span using the largest span. (Assumption 3.2.2)



Case 3 : N/S



Case 3 E/W dir



Case 4

Note : For case 3 in the E/W dir., conservatively take diaphragm as simple span using the largest span. (Assumption 3.2.2)

6.4.2 Governing Design Loads

Combine dead load and design live load for seismic load calculation.

$$U_3 := TDL + Le$$

$$\boxed{U_3 = \begin{pmatrix} 359 \\ 409 \\ 725 \end{pmatrix} psf}$$

18" roof slab
18" floor slab
48" floor slab

See Section 6.3.1 for TDL & Section 6.2.7 for Le.

$$U\_1to4 := \begin{pmatrix} U_{3_1} \\ U_{3_1} \\ U_{3_1} \\ U_{3_1} \end{pmatrix}$$

$$\boxed{U\_1to4 = \begin{pmatrix} 359 \\ 359 \\ 359 \\ 359 \end{pmatrix} psf}$$

Case 1
Case 2
Case 3
Case 4

$D + Le$ for 18" roof slabs

6.4.3 Weight of North and South Walls (per Foot) Tributary to Diaphragm : WW_{ns}

Walls perpendicular to the direction of the in-plane load will contribute distributed loads along the span of the diaphragm

6.4.3.1 Weight of Exterior Walls (per Foot) : WW_{ns_ext}

Tributary Height of Wall 1 (North wall) :

$$Hw1\_1 := \frac{100\cdot ft - 64\cdot ft}{2} \qquad \boxed{Hw1\_1 = 18\,ft} \quad \text{Case 1 : Col line C}$$

$$Hw1\_2 := \frac{72\cdot ft - 64ft}{2} \qquad \boxed{Hw1\_2 = 4\,ft} \quad \text{Case 2 : Col line C}$$

$$Hw1\_3 := \frac{64\cdot ft - 32ft}{2} \qquad \boxed{Hw1\_3 = 16\,ft} \quad \text{Case 3 : Col line A}$$

$$Hw1\_4 := \frac{64\cdot ft - 32\cdot ft}{2} \qquad \boxed{Hw1\_4 = 16\,ft} \quad \text{Case 4 : Col line C}$$

$$Hw1 := \begin{pmatrix} Hw1\_1 \\ Hw1\_2 \\ Hw1\_3 \\ Hw1\_4 \end{pmatrix} \qquad \boxed{Hw1 = \begin{pmatrix} 18 \\ 4 \\ 16 \\ 16 \end{pmatrix} ft} \quad \begin{matrix} \text{Case 1 : Col line C} \\ \text{Case 2 : Col line C} \\ \text{Case 3 : Col line A} \\ \text{Case 4 : Col line C} \end{matrix}$$

Thickness of Wall 1 (North wall) :

$$\boxed{Tw1 := \begin{pmatrix} 4 \\ 4 \\ 4 \\ 4 \end{pmatrix} \cdot ft} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Tributary Height of Wall 2 (South wall) :

$$Hw2\_1 := \frac{100\cdot ft - 64\cdot ft}{2} \qquad \boxed{Hw2\_1 = 18\,ft} \quad \text{Case 1 : Col line E}$$

$$Hw2\_2 := \frac{72\cdot ft - 64\cdot ft}{2} \qquad \boxed{Hw2\_2 = 4\,ft} \quad \text{Case 2 : Col line E}$$

$$Hw2\_3 := \frac{100\cdot ft - 64\cdot ft}{2} \qquad \boxed{Hw2\_3 = 18\,ft} \quad \text{Case 3 : Col line C}$$

$$Hw2\_4 := \frac{64\cdot ft - 32\cdot ft}{2} \qquad \boxed{Hw2\_4 = 16\,ft} \quad \text{Case 4 : Col line E}$$

$$Hw2 := \begin{pmatrix} Hw2\_1 \\ Hw2\_2 \\ Hw2\_3 \\ Hw2\_4 \end{pmatrix} \qquad \boxed{Hw2 = \begin{pmatrix} 18 \\ 4 \\ 18 \\ 16 \end{pmatrix} ft} \quad \begin{matrix} \text{Case 1 : Col line E} \\ \text{Case 2 : Col line E} \\ \text{Case 3 : Col line C} \\ \text{Case 4 : Col line E} \end{matrix}$$

Thickness of Wall 2 (South wall) :

$$\boxed{Tw2 := \begin{pmatrix} 4 \\ 4 \\ 4 \\ 4 \end{pmatrix} \cdot ft} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

$$WWns\_ext\_1to4 := \overrightarrow{\left[(Hw1 \cdot Tw1 + Hw2 \cdot Tw2) \cdot w_{conc}\right]} \qquad \boxed{WWns\_ext\_1to4 = \begin{pmatrix} 21.6 \\ 4.8 \\ 20.4 \\ 19.2 \end{pmatrix} klf} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Weight of North and South Walls (per foot)
Tributary to Diaphragm

6.4.3.2 Weight of Interior Walls (per Foot) : WW_{ns_int}

$$\boxed{S_{ns\_1to4} := \begin{pmatrix} 105 \\ 91 \\ 59 \\ 43 \end{pmatrix} \cdot ft} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Diaphragm Spans
North/South Seismic Acceleration
See Section 6.4.1.

Case 1 : No interior walls $\boxed{WWns\_int\_1 := 0.0 \cdot klf}$

Case 2 : No interior walls $\boxed{WWns\_int\_2 := 0.0 \cdot klf}$

Case 3 : Wall at column line B/3-4 & C/4-6:

Conservatively locate the wall B/3-4 to the evaluation for B/4-6 for additional weight on the diaphragm

$$H2ns_1 := \frac{64 \cdot ft - 32 \cdot ft}{2} \qquad H2ns_1 = 16\,ft \quad \text{Height}$$

$T2ns_1 := 4 \cdot ft$ Thickness

$L2ns_1 := 46 \cdot ft$ Length

$$H2ns_2 := \frac{64 \cdot ft - 32 \cdot ft}{2} \qquad H2ns_2 = 16\,ft \quad \text{Height}$$

$T2ns_2 := 4 \cdot ft$ Thickness

$L2ns_2 := 59 \cdot ft$ Length

$$WWns\_int\_3 := \left(\frac{H2ns_1 \cdot T2ns_1 \cdot L2ns_1 + H2ns_2 \cdot T2ns_2 \cdot L2ns_2}{S_{ns\_1to4_3}}\right) \cdot w_{conc}$$

$\boxed{WWns\_int\_3 = 17.1\,klf}$

Case 4 : No interior wall $\boxed{WWns\_int\_4 := 0.0 \cdot klf}$

$$WWns\_int\_1to4 := \begin{pmatrix} WWns\_int\_1 \\ WWns\_int\_2 \\ WWns\_int\_3 \\ WWns\_int\_4 \end{pmatrix} \qquad \boxed{WWns\_int\_1to4 = \begin{pmatrix} 0.0 \\ 0.0 \\ 17.1 \\ 0.0 \end{pmatrix} klf} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Weight of North and South Walls (per foot) Tributary to Diaphragm

6.4.3.3 Total Weight of North & South Walls Tributary to Diaphragm : WW_{ns}

$$WWns\_1to4 := WWns\_ext\_1to4 + WWns\_int\_1to4$$

Total weight of North & South Walls (per foot)
Tributary to Diaphragm

$$\boxed{WWns\_1to4 = \begin{pmatrix} 21.6 \\ 4.8 \\ 37.5 \\ 19.2 \end{pmatrix} klf}$$

Case 1
Case 2
Case 3
Case 4

6.4.4 Weight of East and West Walls (per Foot) Tributary to Diaphragm : WW_{ew}

6.4.4.1 Weight of Exterior Walls (per Foot) : WW_{ew_ext}

Tributary Height of Wall 3 (East wall) :

$$Hw3\_1 := \frac{100 \cdot ft - 72 \cdot ft}{2} \qquad \boxed{Hw3\_1 = 14\,ft}$$ Case 1 : Col line 6

$$Hw3\_2 := \frac{72 \cdot ft - 32 \cdot ft}{2} \qquad \boxed{Hw3\_2 = 20\,ft}$$ Case 2 : Col line 8

$$Hw3\_3 := \frac{64 \cdot ft - 32 \cdot ft}{2} \qquad \boxed{Hw3\_3 = 16\,ft}$$ Case 3 : Col line 8

$$Hw3\_4 := \frac{100 \cdot ft - 64 \cdot ft}{2} \qquad \boxed{Hw3\_4 = 18\,ft}$$ Case 4 : Col line 3

Thickness of Wall 3 (East wall) :

$$\boxed{Tw3 := \begin{pmatrix} 4 \\ 4 \\ 4 \\ 4 \end{pmatrix} \cdot ft}$$

Case 1
Case 2
Case 3
Case 4

$$Hw3 := \begin{pmatrix} Hw3\_1 \\ Hw3\_2 \\ Hw3\_3 \\ Hw3\_4 \end{pmatrix} \qquad \boxed{Hw3 = \begin{pmatrix} 14 \\ 20 \\ 16 \\ 18 \end{pmatrix} ft}$$

Case 1 : Col line 6
Case 2 : Col line 8
Case 3 : Col line 8
Case 4 : Col line 3

Tributary Height of Wall 4 (West wall) :

$$Hw4\_1 := \frac{100 \cdot ft - 64 \cdot ft}{2} \qquad \boxed{Hw4\_1 = 18\,ft}$$ Case 1 : Col line 3

$$Hw4\_2 := \frac{100 \cdot ft - 72ft}{2} \qquad \boxed{Hw4\_2 = 14\,ft}$$ Case 2 : Col line 6

$$Hw4\_3 := \frac{64 \cdot ft - 32ft}{2} \qquad \boxed{Hw4\_3 = 16\,ft}$$ Case 3 : Col line 3

$$Hw4\_4 := \frac{64 \cdot ft - 32 \cdot ft}{2} \qquad \boxed{Hw4\_4 = 16\,ft}$$ Case 4 : Col line 2

Thickness of Wall 4 (West wall) :

$$\boxed{Tw4 := \begin{pmatrix} 4 \\ 4 \\ 4 \\ 2 \end{pmatrix} \cdot ft}$$

Case 1
Case 2
Case 3
Case 4

$$Hw4 := \begin{pmatrix} Hw4\_1 \\ Hw4\_2 \\ Hw4\_3 \\ Hw4\_4 \end{pmatrix} \qquad \boxed{Hw4 = \begin{pmatrix} 18 \\ 14 \\ 16 \\ 16 \end{pmatrix} ft}$$

Case 1 : Col line 3
Case 2 : Col line 6
Case 3 : Col line 3
Case 4 : Col line 2

$$WWew\_ext\_1to4 := \overrightarrow{\left[(Hw3 \cdot Tw3 + Hw4 \cdot Tw4) \cdot w_{conc}\right]}$$

$$\boxed{WWew\_ext\_1to4 = \begin{pmatrix} 19.2 \\ 20.4 \\ 19.2 \\ 15.6 \end{pmatrix} klf}$$

Case 1
Case 2
Case 3
Case 4

Weight of East and West Walls (per foot)
Tributary to Diaphragm

6.4.4.2 Weight of Interior Walls (per Foot) : WW_{ew_int}

$$\boxed{S_{ew\_1to4} := \begin{pmatrix} 74 \\ 74 \\ 82 \\ 74 \end{pmatrix} \cdot ft}$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Spans
East/West Seismic Acceleration
See Section 6.4.1.

Case 1 : No interior walls

$$\boxed{WWew\_int\_1 := 0.0 \cdot klf}$$

Case 2 : Interior wall @ column line 6/C-E

$$H2ew := \frac{72 \cdot ft - 32 \cdot ft}{2} \qquad H2ew = 20\,ft \quad \text{Height}$$

$$T2ew := 4 \cdot ft \quad \text{Thickness}$$

$$L2ew := 74 \cdot ft \quad \text{Length}$$

$$WWew\_int\_2 := \frac{H2ew \cdot T2ew \cdot L2ew}{S_{ew\_1to4_2}} \cdot w_{conc}$$

$$\boxed{WWew\_int\_2 = 12.0\,klf}$$

Case 3 : Three (3) walls at column line 4/A-C, 4/E-F, 6/A-C, 6/E-F, 7/A-C, and 7/E-F

$$H3ew := \frac{64 \cdot ft - 32 \cdot ft}{2} \qquad H3ew = 16\,ft \quad \text{Height}$$

$$T3ew := 4 \cdot ft \quad \text{Thickness} \qquad L3ew := 82 \cdot ft \quad \text{Length}$$

$$WWew\_int\_3 := \left(\frac{H3ew \cdot T3ew \cdot L3ew \cdot w_{conc}}{S_{ew\_1to4_3}}\right) \cdot 3$$

$$\boxed{WWew\_int\_3 = 28.8\,klf}$$

Case 4 : Interior wall @ column line 3/C-E

$$H4ew := \frac{64 \cdot ft - 32 \cdot ft}{2} \qquad H4ew = 16\,ft \quad \text{Height}$$

$$T4ew := 4 \cdot ft \quad \text{Thickness} \qquad L4ew := 74 \cdot ft \quad \text{Length}$$

$$WWew\_int\_4 := \left(\frac{H4ew \cdot T4ew \cdot L4ew}{S_{ew\_1to4_4}}\right) \cdot w_{conc}$$

$$\boxed{WWew\_int\_4 = 9.6\,klf}$$

$$WWew\_int\_1to4 := \begin{pmatrix} WWew\_int\_1 \\ WWew\_int\_2 \\ WWew\_int\_3 \\ WWew\_int\_4 \end{pmatrix} \qquad \boxed{WWew\_int\_1to4 = \begin{pmatrix} 0.0 \\ 12.0 \\ 28.8 \\ 9.6 \end{pmatrix} klf} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Weight of East and West Walls (per foot) Tributary to Diaphragm

6.4.4.3 Total Weight of East & West Walls Tributary to Diaphragm : WW_{ew}

$$WWew\_1to4 := WWew\_ext\_1to4 + WWew\_int\_1to4 \qquad \boxed{WWew\_1to4 = \begin{pmatrix} 19.2 \\ 32.4 \\ 48.0 \\ 25.2 \end{pmatrix} klf} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Total weight of East & West Walls (per foot)
Tributary to Diaphragm

6.4.5 Design for North/South Seismic Acceleration

6.4.5.1 Moments and Shears

$$\boxed{S_{ns\_1to4} = \begin{pmatrix} 105 \\ 91 \\ 59 \\ 43 \end{pmatrix} ft} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Diaphragm Spans
North/South Seismic Acceleration
See Section 6.4.3.2.

$$\boxed{DD_{ns\_1to4} := \begin{pmatrix} 74 \\ 74 \\ 82 \\ 74 \end{pmatrix} \cdot ft} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Diaphragm Depths
North/South Seismic Acceleration
See Section 6.4.1.

$$SACC_{y\_1to4} := \begin{pmatrix} a100_y \\ a72_y \\ a64_y \\ a64_y \end{pmatrix} \begin{matrix} \text{El. 100'} \\ \text{El. 72'} \\ \text{El. 64'} \\ \text{El. 64'} \end{matrix} \qquad \boxed{SACC_{y\_1to4} = \begin{pmatrix} 1.60 \\ 1.14 \\ 0.95 \\ 0.95 \end{pmatrix}} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Seismic Acceleration Factors
in Y (North/South) Direction
See Section 6.2.6.

$$\boxed{U\_1to4 = \begin{pmatrix} 359 \\ 359 \\ 359 \\ 359 \end{pmatrix} psf} \quad \begin{matrix} \text{Case 1} & \text{18" roof slab} \\ \text{Case 2} & \text{18" roof slab} \\ \text{Case 3} & \text{18" roof slab} \\ \text{Case 4} & \text{18" roof slab} \end{matrix}$$

Vertical Load
Section 6.3.3

<u>Seismic Force in the North/South Direction</u>

$$W_{ns\_1to4} := \overrightarrow{\left[\left(\underbrace{U\_1to4}_{\text{Weight of Slab}} \cdot \underbrace{DD_{ns\_1to4}}_{\text{Diaphragm Depth}} + \underbrace{WWns\_1to4}_{\text{Weight of Walls}}\right) \cdot \underbrace{SACC_{y\_1to4}}_{\text{Seismic Acceleration}}\right]}$$

Horizontal Seismic Load for Diaphragm per Foot (N/S dir)

$$W_{ns\_1to4} = \begin{pmatrix} 77.0 \\ 35.7 \\ 63.6 \\ 43.5 \end{pmatrix} klf$$

Case 1
Case 2
Case 3
Case 4

Loads

For $U_{\_1to4}$, see Sec. 6.4.2
For DD_{ns_1to4}, see Sec. 6.4.1
For WW_{ns_1to4}, see Sec. 6.4.3.3
For $SACC_{y\_1to4}$, see Sec. 6.2.6

$$M_{ns\_1to4} := \overrightarrow{\frac{W_{ns\_1to4} \cdot S_{ns\_1to4}^{\;2}}{10}}$$

$$M_{ns\_1to4} = \begin{pmatrix} 84932 \\ 29593 \\ 22124 \\ 8036 \end{pmatrix} ft \cdot kip$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Moment

$$V_{ns\_1to4} := \overrightarrow{\frac{1.15 W_{ns\_1to4} \cdot S_{ns\_1to4}}{2}}$$

$$V_{ns\_1to4} = \begin{pmatrix} 4651 \\ 1870 \\ 2156 \\ 1075 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Shear

6.4.5.2 Check Shear Strength as per ACI 349 (Ref. 2.2.1, Sect. 21.6.5.6)

$$h_{1to4} := \begin{pmatrix} 18 \\ 18 \\ 18 \\ 18 \end{pmatrix} \cdot in$$

Case 1
Case 2
Case 3
Case 4

Slab Thickness
See Section 6.4.1.

$$A_{cv\_ns\_1to4} := \overrightarrow{\left(DD_{ns\_1to4} \cdot h_{1to4}\right)}$$

$$A_{cv\_ns\_1to4} = \begin{pmatrix} 15984 \\ 15984 \\ 17712 \\ 15984 \end{pmatrix} in^2$$

Case 1
Case 2
Case 3
Case 4

Shear Areas

$\phi_{diaph} := 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.1, Sect. 9.3.4)]

Check Limiting Diaphragm Shear Strength

$\phi V_{n\_max\_ns\_1to4} := \phi_{diaph} \cdot 8 \cdot A_{cv\_ns\_1to4} \cdot \sqrt{f_c \cdot psi}$ ACI 349 (Ref. 2.2.1, Sect. 21.6.5.6)

$\phi_{diaph} = 0.6$ ACI 349 (Ref 2.2.1, Sect 9.3.4)

$$\phi V_{n\_max\_ns\_1to4} = \begin{pmatrix} 5425 \\ 5425 \\ 6012 \\ 5425 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Shear Strength

$$V_{ns\_1to4} = \begin{pmatrix} 4651 \\ 1870 \\ 2156 \\ 1075 \end{pmatrix} kip$$

Shear See Section 6.4.5.1.

Compare limiting shear capacity per ACI 349 to shear requirements. If $\phi V_{n\_max\_ns\_1to4} > V_{ns\_1to4}$, no shear reinforcement is required.

$$\phi V_{n\_max\_ns_1} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ns\_1to4_1} > V_{ns\_1to4_1} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 1

$$\phi V_{n\_max\_ns_2} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ns\_1to4_2} > V_{ns\_1to4_2} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 2

$$\phi V_{n\_max\_ns_3} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ns\_1to4_3} > V_{ns\_1to4_3} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 3

$$\phi V_{n\_max\_ns_4} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ns\_1to4_4} > V_{ns\_1to4_4} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 4

$$\phi V_{n\_max\_ns} := \begin{pmatrix} \phi V_{n\_max\_ns_1} \\ \phi V_{n\_max\_ns_2} \\ \phi V_{n\_max\_ns_3} \\ \phi V_{n\_max\_ns_4} \end{pmatrix} \quad \phi V_{n\_max\_ns} = \begin{pmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{pmatrix}$$

Case 1
Case 2
Case 3
Case 4

$\phi V_{n\_max\_ns\_1to4} > V_{ns\_1to4}$ Limiting diaphragm shear strength satisfied.

6.4.5.3 Shear Reinforcement as per ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.1, Sect. 9.3.4)]

$$\frac{V_{ns\_1to4}}{\phi_{diaph}} := V_{c\_ns\_1to4} + V_{s\_ns\_1to4}$$

Total Shear Strength = Concrete Shear Strength + Reinforcing Steel Shear Strength

$$V_{c\_ns\_1to4} := 2 \cdot A_{cv\_ns\_1to4} \cdot \sqrt{f_c \cdot psi}$$

ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2, Equation 21-6)

$$\phi_{diaph} \cdot V_{c\_ns\_1to4} = \begin{pmatrix} 1356 \\ 1356 \\ 1503 \\ 1356 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Shear Strength

$$V_{ns\_1to4} = \begin{pmatrix} 4651 \\ 1870 \\ 2156 \\ 1075 \end{pmatrix} kip$$

Shear See Section 6.4.5.1.

Compare concrete shear capacity per ACI 349 to shear requirements. If $\phi_{diaph} \cdot V_{c\_ns\_1to4} > V_{ns\_1to4}$, no shear reinforcement is required.

$$\phi V_{c\_ns_1} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_1to4_1} > V_{ns\_1to4_1} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 1

$$\phi V_{c\_ns_2} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_1to4_2} > V_{ns\_1to4_2} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 2

$$\phi V_{c\_ns_3} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_1to4_3} > V_{ns\_1to4_3} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 3

$$\phi V_{c\_ns_4} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_1to4_4} > V_{ns\_1to4_4} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 4

$$\phi V_{c\_ns} := \begin{pmatrix} \phi V_{c\_ns_1} \\ \phi V_{c\_ns_2} \\ \phi V_{c\_ns_3} \\ \phi V_{c\_ns_4} \end{pmatrix} \qquad \phi V_{c\_ns} = \begin{pmatrix} \text{"NG"} \\ \text{"NG"} \\ \text{"NG"} \\ \text{"OK"} \end{pmatrix}$$

Case 1
Case 2
Case 3
Case 4

For cases 1 to 3 the concrete shear strength [$\phi_{diaph} \cdot V_{c\_ns\_1to4}$] is less than the shear demand [V_{ns_1to4}] therefore shear reinforcing is required. For case 4 the concrete shear strength is more than the shear demand, therefore shear reinforcing is not required.

$$V_{s\_ns\_1to4} := \frac{V_{ns\_1to4}}{\phi_{diaph}} - V_{c\_ns\_1to4}$$ MacGregor (Ref. 2.2.4, Page 207)

$$V_{s\_ns\_1to4} = \begin{pmatrix} 5491 \\ 856 \\ 1089 \\ -470 \end{pmatrix} kip \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

$$V_{s\_ns\_1to4rev_1} := \begin{array}{|l} V_{s\_ns\_1to4_1} \text{ if } V_{s\_ns\_1to4_1} \ge 0kip \\ 0.0 \cdot kip \text{ otherwise} \end{array}$$ Case 1

$$V_{s\_ns\_1to4rev_2} := \begin{array}{|l} V_{s\_ns\_1to4_2} \text{ if } V_{s\_ns\_1to4_2} \ge 0kip \\ 0.0 \cdot kip \text{ otherwise} \end{array}$$ Case 2

$$V_{s\_ns\_1to4rev_3} := \begin{array}{|l} V_{s\_ns\_1to4_3} \text{ if } V_{s\_ns\_1to4_3} \ge 0kip \\ 0.0 \cdot kip \text{ otherwise} \end{array}$$ Case 3

$$V_{s\_ns\_1to4rev_4} := \begin{array}{|l} V_{s\_ns\_1to4_4} \text{ if } V_{s\_ns\_1to4_4} \ge 0kip \\ 0.0 \cdot kip \text{ otherwise} \end{array}$$ Case 4

$$V_{s\_ns\_1to4rev} = \begin{pmatrix} 5491 \\ 856 \\ 1089 \\ 0 \end{pmatrix} kip \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Required Reinforcing Shear Strength
("0 kip" indicates that no shear reinforcing is required.)

6.4.5.4 Required Reinforcement

$$A_{cv\_ns\_1to4} = \begin{pmatrix} 15984 \\ 15984 \\ 17712 \\ 15984 \end{pmatrix} in^2$$

Case 1	Shear Areas
Case 2	Sections 6.4.5.2
Case 3	
Case 4	

$f_y = 60\ ksi$

$$\rho_{req\_ns\_1to4} := \frac{V_{s\_ns\_1to4rev}}{A_{cv\_ns\_1to4} \cdot f_y}$$ ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$$\rho_{req\_ns\_1to4} = \begin{pmatrix} 0.00573 \\ 0.00089 \\ 0.00102 \\ 0.00000 \end{pmatrix}$$

Case 1
Case 2
Case 3
Case 4

Required Reinforcement
(total required on 2 faces)

Note : See section 6.6.1.3 for total steel required for out of plane (vertical) and in-plane (horizontal) loads.

6.4.6 Design for East/West Seismic Acceleration

6.4.6.1 Moments and Shears

$$S_{ew\_1to4} = \begin{pmatrix} 74 \\ 74 \\ 82 \\ 74 \end{pmatrix} ft$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Spans
East/West Seismic Acceleration
See Section 6.4.4.2.

$$DD_{ew\_1to4} := \begin{pmatrix} 105 \\ 91 \\ 196 \\ 43 \end{pmatrix} \cdot ft$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Depths
East/West Seismic Acceleration
See Section 6.4.1.

$$SACC_{x\_1to4} := \begin{pmatrix} a100_x \\ a72_x \\ a64_x \\ a64_x \end{pmatrix} \begin{matrix} \text{El. 100'} \\ \text{El. 72'} \\ \text{El. 64'} \\ \text{El. 64'} \end{matrix} \qquad SACC_{x\_1to4} = \begin{pmatrix} 1.45 \\ 0.97 \\ 0.92 \\ 0.92 \end{pmatrix}$$

Case 1
Case 2
Case 3
Case 4

Seismic Acceleration Factors
in X (East/West) Direction
See Section 6.2.6.

$$U_{\_1to4} = \begin{pmatrix} 359 \\ 359 \\ 359 \\ 359 \end{pmatrix} psf$$

Case 1	18" roof slab
Case 2	18" roof slab
Case 3	18" roof slab
Case 4	18" roof slab

Vertical Load
Section 6.3.3

Seismic Force in the East/West Direction

$$W_{ew\_1to4} := \overrightarrow{\left[\left(U_{\_1to4} \cdot DD_{ew\_1to4} + WWew\_1to4\right) \cdot SACC_{x\_1to4}\right]}$$

Weight of Slab | Diaphragm Depth | Weight of Walls | Seismic Acceleration

Horizontal Seismic Load for Diaphragm per Foot (E/W dir)

$$W_{ew\_1to4} = \begin{pmatrix} 82.5 \\ 63.1 \\ 108.8 \\ 37.4 \end{pmatrix} klf$$

Case 1
Case 2
Case 3
Case 4

Loads

For $U_{\_1to4}$, see Sec. 6.4.2
For DD_{ew_1to4}, see Sec. 6.4.1
For WW_{ew_1to4}, see Sec. 6.4.4.3
For $SACC_{x\_1to4}$, see Sec. 6.2.6

$$M_{ew\_1to4} := \overrightarrow{\frac{W_{ew\_1to4} \cdot S_{ew\_1to4}^{\;2}}{10}}$$

$$M_{ew\_1to4} = \begin{pmatrix} 45155 \\ 34551 \\ 73191 \\ 20467 \end{pmatrix} ft \cdot kip$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Moment

$$V_{ew\_1to4} := \overrightarrow{\frac{1.15 \cdot W_{ew\_1to4} \cdot S_{ew\_1to4}}{2}}$$

$$V_{ew\_1to4} = \begin{pmatrix} 3509 \\ 2685 \\ 5132 \\ 1590 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Shear

6.4.6.2 Check Shear Strength as per ACI 349 (Ref. 2.2.1, Sect. 21.6.5.6)

$$h_{1to4} = \begin{pmatrix} 18 \\ 18 \\ 18 \\ 18 \end{pmatrix} in$$

Case 1
Case 2
Case 3
Case 4

Slab Thickness
See Section 6.4.5.2.

$$A_{cv\_ew\_1to4} := \overrightarrow{\left(DD_{ew\_1to4} \cdot h_{1to4}\right)}$$

$$A_{cv\_ew\_1to4} = \begin{pmatrix} 22680 \\ 19656 \\ 42336 \\ 9288 \end{pmatrix} in^2$$

Case 1
Case 2
Case 3
Case 4

Shear Areas

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.1, Sect. 9.3.4)]

Check Limiting Diaphragm Shear Strength

$\phi V_{n\_max\_ew\_1to4} := \phi_{diaph} \cdot 8 \cdot A_{cv\_ew\_1to4} \cdot \sqrt{f_c \cdot psi}$ ACI 349 (Ref. 2.2.1, Sect. 21.6.5.6)

$\phi_{diaph} = 0.6$ ACI 349 (Ref 2.2.1, Sect 9.3.4)

$\phi V_{n\_max\_ew\_1to4} = \begin{pmatrix} 7698 \\ 6671 \\ 14369 \\ 3152 \end{pmatrix} kip$ Case 1, Case 2, Case 3, Case 4 — Shear Strength

$V_{ew\_1to4} = \begin{pmatrix} 3509 \\ 2685 \\ 5132 \\ 1590 \end{pmatrix} kip$ Shear See Section 6.4.6.1.

Compare limiting shear capacity per ACI 349 to shear requirements. If $\phi V_{n\_max\_ew\_1to4} > V_{ew\_1to4}$, no shear reinforcement is required.

$\phi V_{n\_max\_ew_1} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_1to4_1} > V_{ew\_1to4_1} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 1

$\phi V_{n\_max\_ew_2} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_1to4_2} > V_{ew\_1to4_2} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 2

$\phi V_{n\_max\_ew_3} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_1to4_3} > V_{ew\_1to4_3} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 3

$\phi V_{n\_max\_ew_4} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_1to4_4} > V_{ew\_1to4_4} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 4

$\phi V_{n\_max\_ew} := \begin{pmatrix} \phi V_{n\_max\_ew_1} \\ \phi V_{n\_max\_ew_2} \\ \phi V_{n\_max\_ew_3} \\ \phi V_{n\_max\_ew_4} \end{pmatrix}$ $\phi V_{n\_max\_ew} = \begin{pmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{pmatrix}$ Case 1, Case 2, Case 3, Case 4

$\phi V_{n\_max\_ew\_1to4} > V_{ew\_1to4}$ Limiting diaphragm shear strength satisfied.

6.4.6.3 Shear Reinforcement as per ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$$\frac{V_{ew\_1to4}}{\phi_{diaph}} := V_{c\_ew\_1to4} + V_{s\_ew\_1to4}$$

Total Shear Strength = Concrete Shear Strength + Reinforcing Steel Shear Strength

$$V_{c\_ew\_1to4} := 2 \cdot A_{cv\_ew\_1to4} \cdot \sqrt{f_c \cdot psi}$$

ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$$\phi_{diaph} \cdot V_{c\_ew\_1to4} = \begin{pmatrix} 1924 \\ 1668 \\ 3592 \\ 788 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Shear Strength

$$V_{ew\_1to4} = \begin{pmatrix} 3509 \\ 2685 \\ 5132 \\ 1590 \end{pmatrix} kip$$

Shears See Section 6.4.6.1.

Compare concrete shear capacity per ACI 349 to shear requirements. If $\phi_{diaph} \cdot V_{c\_ew\_1to4} > V_{ew\_1to4}$, no shear reinforcement is required.

$$\phi V_{c\_ew_1} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_1to4_1} > V_{ew\_1to4_1} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 1

$$\phi V_{c\_ew_2} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_1to4_2} > V_{ew\_1to4_2} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 2

$$\phi V_{c\_ew_3} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_1to4_3} > V_{ew\_1to4_3} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 3

$$\phi V_{c\_ew_4} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_1to4_4} > V_{ew\_1to4_4} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 4

$$\phi V_{c\_ew} := \begin{pmatrix} \phi V_{c\_ew_1} \\ \phi V_{c\_ew_2} \\ \phi V_{c\_ew_3} \\ \phi V_{c\_ew_4} \end{pmatrix}$$

$$\phi V_{c\_ew} = \begin{pmatrix} \text{"NG"} \\ \text{"NG"} \\ \text{"NG"} \\ \text{"NG"} \end{pmatrix}$$

Case 1
Case 2
Case 3
Case 4

For cases 1 to 4 the concrete shear strength [$\phi_{diaph} \cdot V_{c\_ew\_1to4}$] is less than the shear demand [V_{ew_1to4}] therefore shear reinforcing is required.

$$V_{s\_ew\_1to4} := \frac{V_{ew\_1to4}}{\phi_{diaph}} - V_{c\_ew\_1to4}$$ MacGregor (Ref. 2.2.4, Page 207)

$$\boxed{V_{s\_ew\_1to4} = \begin{pmatrix} 2640 \\ 1695 \\ 2567 \\ 1337 \end{pmatrix} kip}$$

Case 1
Case 2
Case 3
Case 4

Required Reinforcing Shear Strength

6.4.6.4 Required Reinforcement

$$\boxed{A_{cv\_ew\_1to4} = \begin{pmatrix} 22680 \\ 19656 \\ 42336 \\ 9288 \end{pmatrix} in^2}$$

Case 1
Case 2
Case 3
Case 4

Shear Areas

$f_y = 60\,ksi$

$$\rho_{req\_ew\_1to4} := \frac{V_{s\_ew\_1to4}}{A_{cv\_ew\_1to4} \cdot f_y}$$ ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$$\boxed{\rho_{req\_ew\_1to4} = \begin{pmatrix} 0.00194 \\ 0.00144 \\ 0.00101 \\ 0.00240 \end{pmatrix}}$$

Case 1
Case 2
Case 3
Case 4

Required Reinforcement
(total required on 2 faces)

Note : See Section 6.6.1.3 for combined reinforcement required
for out of plane (vertical) and in-plane (horizontal) loads.

6.4.7 Design Chord Steel

6.4.7.1 North/South Seismic Acceleration

$$\boxed{M_{ns\_1to4} = \begin{pmatrix} 84932 \\ 29593 \\ 22124 \\ 8036 \end{pmatrix} ft \cdot kip}$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Moment
See Section 6.4.5.1.

$$\boxed{DD_{ns\_1to4} = \begin{pmatrix} 74 \\ 74 \\ 82 \\ 74 \end{pmatrix} ft}$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Depths
See Section 6.4.5.1.

Chord Force (CF):

$$CF_{ns\_1to4} := \frac{M_{ns\_1to4}}{.9 \cdot DD_{ns\_1to4}}$$

Note : 0.9 x Depth is taken as lever arm from centroid of compression stress block to centroid of chord reinforcing steel, thus chord steel will be provided over a width equal to 10% of the diaphragm depth.

$$\boxed{CF_{ns\_1to4} = \begin{pmatrix} 1275 \\ 444 \\ 300 \\ 121 \end{pmatrix} kip}$$

Case 1
Case 2
Case 3
Case 4

Chord Forces

Required Chord Steel (A_{ch}):

$\phi_b = 0.9$

Strength reduction factor for bending
ACI 349 (Ref. 2.2.1, Sect. 9.3.2.1)

$f_y = 60\ ksi$

$$A_{ch\_ns\_1to4} := \frac{CF_{ns\_1to4}}{\phi_b \cdot f_y}$$

$$\boxed{A_{ch\_ns\_1to4} = \begin{pmatrix} 23.6 \\ 8.2 \\ 5.6 \\ 2.2 \end{pmatrix} in^2}$$

Case 1
Case 2
Case 3
Case 4

Required Chord Steel

Provided Chord Steel (A_{ch}):

Use 16 & 6 # 11 bars for cases 1 & 2 respectively and use 6 & 4 # 9 bars for cases 3 & 4 (N/S Seismic Acceleration).

Chord steel provided = 25.0, 9.4, 6.0, & 4.0 in^2 for cases 1, 2, 3 & 4 respectively (N/S Seismic Acceleration).

See Section 6.6.1.7 for chord steel provided.

6.4.7.2 East/West Seismic Acceleration

$$M_{ew\_1to4} = \begin{pmatrix} 45155 \\ 34551 \\ 73191 \\ 20467 \end{pmatrix} ft \cdot kip$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Moment
See Section 6.4.6.1.

$$DD_{ew\_1to4} = \begin{pmatrix} 105 \\ 91 \\ 196 \\ 43 \end{pmatrix} ft$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Depths
See Section 6.4.6.1.

Chord Force (CF):

$$CF_{ew\_1to4} := \frac{M_{ew\_1to4}}{.9 \cdot DD_{ew\_1to4}}$$

Note : 0.9 x Depth is taken as lever arm from centroid of compression stress block to centroid of chord reinforcing steel, thus chord steel will be provided over a width equal to 10% of the diaphragm depth.

$$CF_{ew\_1to4} = \begin{pmatrix} 478 \\ 422 \\ 415 \\ 529 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Chord Forces

Required Chord Steel (A_{ch}):

$\phi_b = 0.9$

Strength reduction factor for bending
ACI 349 (Ref. 2.2.1, Sect. 9.3.2.1)

$f_y = 60\,ksi$

$$A_{ch\_ew\_1to4} := \frac{CF_{ew\_1to4}}{\phi_b \cdot f_y}$$

$$A_{ch\_ew\_1to4} = \begin{pmatrix} 8.8 \\ 7.8 \\ 7.7 \\ 9.8 \end{pmatrix} in^2$$

Case 1
Case 2
Case 3
Case 4

Required Chord Steel

Provided Chord Steel (A_{ch}):

Use 6 # 11 bars for case 1 and 8, 8, & 10 # 9 bars for cases 2, 3 & 4 respectively (E/W Seismic Acceleration).

Chord steel provided= 9.4, 8.0, 8.0 & 10.0 in^2 for cases 1, 2, 3 & 4 respectively (E/W Seismic Acceleration).

See Section 6.6.1.7 for chord steel provided.

6.5 SLAB DIAPHRAGM DESIGN FOR IN-PLANE (HORIZONTAL) LOADS : ANALYSIS CASES 5 TO 8

6.5.1 Diaphragms for Analysis Cases 5 to 8

See Attachment A for plans of the RF showing the diaphragms.

Steel for diaphragms will be computed for following diaphragm panels in this section.

Case 5: Roof Diaphragm @ El. 32': Col Lines 8-9/C-E (N/S & E/W dir): 18" slab
(See Attachment A, Sheet A3)

Case 6: Floor Diaphragm @ El. 32': Col Lines 4-6/E-F (N/S dir) & 3-8/E-F (E/W dir): 18" slab
Also applies to Col Lines:

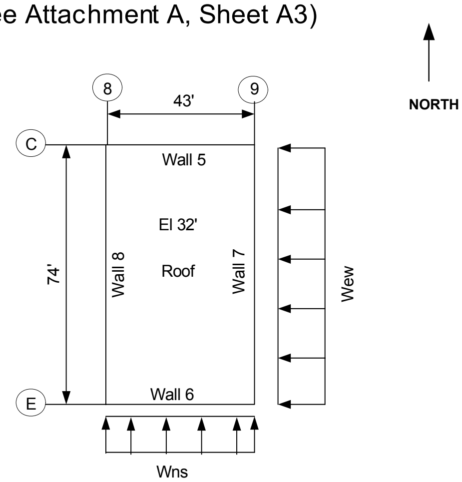
- 3-4/E-F, 6-7/E-F, 7-8/E-F, 4-6/A-C, 6-7/A-C, & 7-8/A-C(N/S dir)
- 4-8/A-C (E/W dir).

See Attachment A, Sheet A3)

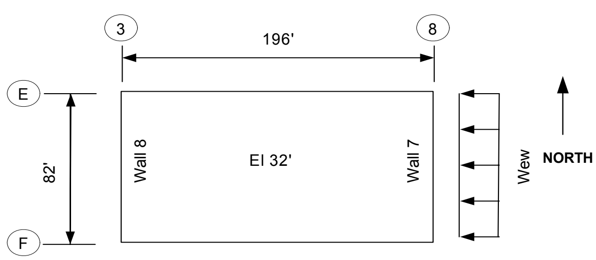
Case 7: Floor Diaphragm @ El. 32’: Col Lines 3-4/A-B (N/S & E/W dir): 18” slab
See Attachment A, Sheet A3)

Case 8: Floor Diaphragm @ El. 32': Col Lines 2-3/C-E (N/S & E/W dir): 18" slab
See Attachment A, Sheet A3)

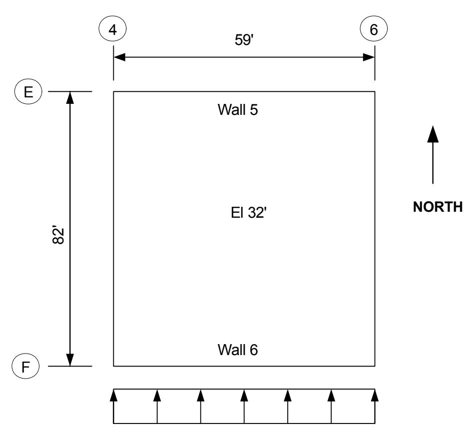
Case 9: Floor Diaphragm @ El.32': Col Lines 3-6/C-E (N/S & E/W dir): 48" slab
See Attachment A, Sheet A3)



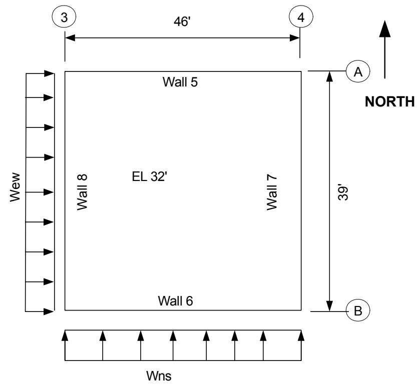
Case 5



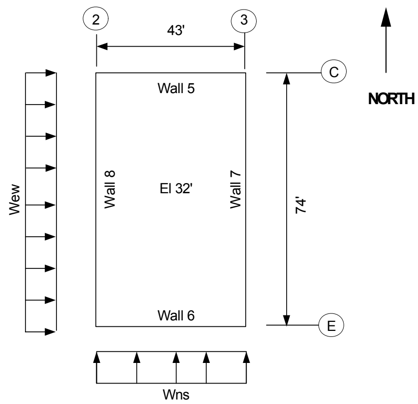
Case 6 E/W



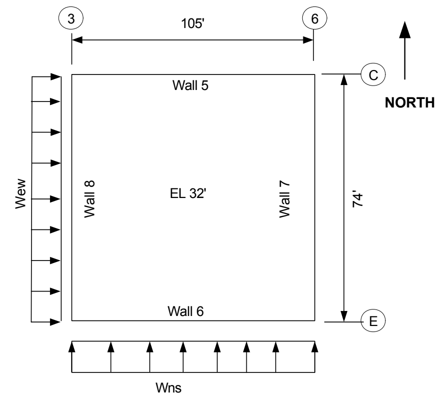
Case 6 N/S



Case 7:



Case 8:



Case 9:

Note : For case 9 in the N/S direction the diaphragm is a 3 span system.
Conservatively take diaphragm as simple span using the largest span. (Assumption 3.2.2)

6.5.2 Governing Design Loads

Combine dead load and design live load for seismic load combination. See Section 6.4.2.

$$\boxed{U_3 = \begin{pmatrix} 359 \\ 409 \\ 725 \end{pmatrix} \text{psf}} \qquad \begin{matrix} \text{18" roof slab} \\ \text{18" floor slab} \\ \text{48" floor slab} \end{matrix}$$

$$U\_5to9 := \begin{pmatrix} {U_3}_1 \\ {U_3}_2 \\ {U_3}_2 \\ {U_3}_2 \\ {U_3}_3 \end{pmatrix} \qquad \boxed{U\_5to9 = \begin{pmatrix} 359 \\ 409 \\ 409 \\ 409 \\ 725 \end{pmatrix} \text{psf}} \qquad \begin{matrix} \text{Case 5} & \text{18" roof slab} \\ \text{Case 6} & \text{18" floor slab} \\ \text{Case 7} & \text{18" floor slab} \\ \text{Case 8} & \text{18" floor slab} \\ \text{Case 9} & \text{48" floor slab} \end{matrix}$$

6.5.3 Weight of North and South Walls (per Foot) Tributary to Diaphragm : WW_{ns}

6.5.3.1 Weight of Exterior Walls (per Foot) : WW_{ns_ext}

Tributary Height of Wall 5 (North wall) :

$$Hw5\_5 := \frac{32\cdot ft - 0\cdot ft}{2} \qquad \boxed{Hw5\_5 = 16\,ft}$$ Case 5 : Col line C

Case 6: No Exterior Walls: $\boxed{Hw5\_6 := 0\cdot ft}$ Case 6 : Col line E

$$Hw5\_7 := \frac{64\cdot ft - 32\cdot ft}{2} + \frac{32ft - 0ft}{2} \qquad \boxed{Hw5\_7 = 32\,ft}$$ Case 7 : Col line A

$$Hw5\_8 := \frac{64\cdot ft - 32\cdot ft}{2} + \frac{32ft - 0ft}{2} \qquad \boxed{Hw5\_8 = 32\,ft}$$ Case 8 : Col line C

Case 9: No Exterior Walls $\boxed{Hw5\_9 := 0\cdot ft}$ Case 9 : Col line C

$$Hw5 := \begin{pmatrix} Hw5\_5 \\ Hw5\_6 \\ Hw5\_7 \\ Hw5\_8 \\ Hw5\_9 \end{pmatrix} \qquad \boxed{Hw5 = \begin{pmatrix} 16 \\ 0 \\ 32 \\ 32 \\ 0 \end{pmatrix} ft}$$

Case 5 : Col line C
Case 6 : Col line E
Case 7 : Col line A
Case 8 : Col line C
Case 9 : Col line C

Thickness of Wall 5 (North wall) :

$$\boxed{Tw5 := \begin{pmatrix} 4 \\ 0 \\ 4 \\ 4 \\ 0 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Tributary Height of Wall 6 (South wall) :

$$Hw6\_5 := \frac{32\cdot ft - 0\cdot ft}{2} \qquad \boxed{Hw6\_5 = 16\,ft}$$ Case 5 : Col line E

$$Hw6\_6 := \frac{64\cdot ft - 32\cdot ft}{2} + \frac{32\cdot ft - 0ft}{2} \qquad \boxed{Hw6\_6 = 32\,ft}$$ Case 6 : Col line F

Case 7: No exterior walls: $\boxed{Hw6\_7 := 0\cdot ft}$ Case 7 : Col line B

$$Hw6\_8 := \frac{64\cdot ft - 32\cdot ft}{2} + \frac{32\cdot ft - 0ft}{2} \qquad \boxed{Hw6\_8 = 32\,ft}$$ Case 8 : Col line E

Case 9: No exterior Walls $\boxed{Hw6\_9 := 0\cdot ft}$ Case 9 : Col line E

$$Hw6 := \begin{pmatrix} Hw6\_5 \\ Hw6\_6 \\ Hw6\_7 \\ Hw6\_8 \\ Hw6\_9 \end{pmatrix} \qquad \boxed{Hw6 = \begin{pmatrix} 16 \\ 32 \\ 0 \\ 32 \\ 0 \end{pmatrix} ft}$$

Case 5 : Col line E
Case 6 : Col line F
Case 7 : Col line B
Case 8 : Col line E
Case 9 : Col line E

Thickness of Wall 6 (South wall) :

$$\boxed{Tw6 := \begin{pmatrix} 4 \\ 4 \\ 0 \\ 4 \\ 0 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8
Case 9

$$WWns\_ext\_5to9 := \overrightarrow{\left[(Hw5 \cdot Tw5 + Hw6 \cdot Tw6) \cdot w_{conc}\right]}$$

$$WWns\_ext\_5to9 = \begin{pmatrix} 19.2 \\ 19.2 \\ 19.2 \\ 38.4 \\ 0.0 \end{pmatrix} klf$$

Case 5
Case 6
Case 7
Case 8
Case 9

Weight of North and South Walls (per foot) Tributary to Diaphragm

6.5.3.2 Weight of Interior Walls (per Foot) : WW_{ns_int}

$$S_{ns\_5to9} := \begin{pmatrix} 43 \\ 59 \\ 46 \\ 43 \\ 105 \end{pmatrix} \cdot ft$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Spans
North/South Seismic Acceleration
See Section 6.5.1.

Case 5 : No walls

$$WWns\_int\_5 := 0.0 \cdot klf$$

Case 6 : Wall at column line E/4-6:

$$H6ns := \frac{64 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0 \cdot ft}{2} \qquad H6ns = 32\,ft \qquad \text{Height}$$

$$T6ns := 4 \cdot ft \quad \text{Thickness} \qquad L6ns := 59 \cdot ft \quad \text{Length}$$

$$WWns\_int\_6 := \left(\frac{H6ns \cdot T6ns \cdot L6ns}{S_{ns\_5to9_2}}\right) \cdot w_{conc}$$

$$WWns\_int\_6 = 19.2\,klf$$

Case 7: Wall at column line B/3-4:

$$H7ns := \frac{64 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0 \cdot ft}{2} \qquad H7ns = 32\,ft \qquad \text{Height}$$

$$T7ns := 4 \cdot ft \quad \text{Thickness} \qquad L7ns := 46 \cdot ft \quad \text{Length}$$

$$WWns\_int\_7 := \left(\frac{H7ns \cdot T7ns \cdot L7ns}{S_{ns\_5to9_3}}\right) \cdot w_{conc}$$

$$WWns\_int\_7 = 19.2\,klf$$

Case 8 : No walls

$$WWns\_int\_8 := 0.0 \cdot klf$$

Case 9: Walls at column line C/3-6, E/3-6, D/3-4 & D/5-6:

$H9ns_1 := \frac{64\cdot ft - 32\cdot ft}{2} + \frac{32\cdot ft - 0\cdot ft}{2}$ $H9ns_1 = 32\,ft$ Height Walls along col line C & E

$T9ns_1 := 4\cdot ft$ Thickness $L9ns_1 := 105\cdot ft$ Length

$H9ns_2 := \frac{32\cdot ft - 0\cdot ft}{2}$ $H9ns_2 = 16\,ft$ Height

$T9ns_2 := 4\cdot ft$ Thickness $L9ns_2 := 87\cdot ft$ Total Length of col line D

Consider the weight of wall on col line D spread over the entire span.

$$WWns\_int\_9 := \frac{\left(2\cdot H9ns_1\cdot T9ns_1\cdot L9ns_1\right) + \left(H9ns_2\cdot T9ns_2\cdot L9ns_2\right)}{S_{ns\_5to9_5}}\cdot w_{conc}$$

$\boxed{WWns\_int\_9 = 46.4\,klf}$

$$WWns\_int\_5to9 := \begin{pmatrix} WWns\_int\_5 \\ WWns\_int\_6 \\ WWns\_int\_7 \\ WWns\_int\_8 \\ WWns\_int\_9 \end{pmatrix} \qquad \boxed{WWns\_int\_5to9 = \begin{pmatrix} 0.0 \\ 19.2 \\ 19.2 \\ 0.0 \\ 46.4 \end{pmatrix} klf} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Weight of North and South Walls (per foot)
Tributary to Diaphragm

6.5.3.3 Total Weight of North & South Walls Tributary to Diaphragm : WW_{ns}

$$WWns\_5to9 := WWns\_ext\_5to9 + WWns\_int\_5to9 \qquad \boxed{WWns\_5to9 = \begin{pmatrix} 19.2 \\ 38.4 \\ 38.4 \\ 38.4 \\ 46.4 \end{pmatrix} klf} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Total weight of North & South Walls (per foot)
Tributary to Diaphragm

6.5.4 Weight of East and West Walls (per Foot) Tributary to Diaphragm : WW_{ew}

6.5.4.1 Weight of Walls (per Foot) : WW_{ew_ext}

Tributary Height of Wall 7 (East wall) :

$$Hw7\_5 := \frac{32 \cdot ft - 0 \cdot ft}{2}$$ $\boxed{Hw7\_5 = 16\,ft}$ Case 5 : Col line 9

$$Hw7\_6 := \frac{64 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0 \cdot ft}{2}$$ $\boxed{Hw7\_6 = 32\,ft}$ Case 6 : Col line 8

Case 7: No exterior wall: $\boxed{Hw7\_7 := 0 \cdot ft}$ Case 7 : Col line 4

Case 8: No exterior wall: $\boxed{Hw7\_8 := 0 \cdot ft}$ Case 8 : Col line 3

Case 9: No exterior wall: $\boxed{Hw7\_9 := 0 \cdot ft}$ Case 9 : Col line 6

$$Hw7 := \begin{pmatrix} Hw7\_5 \\ Hw7\_6 \\ Hw7\_7 \\ Hw7\_8 \\ Hw7\_9 \end{pmatrix} \qquad \boxed{Hw7 = \begin{pmatrix} 16 \\ 32 \\ 0 \\ 0 \\ 0 \end{pmatrix} ft}$$

Case 5 : Col line 9
Case 6 : Col line 8
Case 7 : Col line 4
Case 8 : Col line 3
Case 9: Col line 6

Thickness of Wall 7 (East wall) :

$$\boxed{Tw7 := \begin{pmatrix} 2 \\ 4 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Tributary Height of Wall 8 (West wall) :

Case 5: No exterior wall: $\boxed{Hw8\_5 := 0 \cdot ft}$ Case 5 : Col line 8

$$Hw8\_6 := \frac{64 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0ft}{2}$$ $\boxed{Hw8\_6 = 32\,ft}$ Case 6 : Col line 3

$$Hw8\_7 := \frac{64 \cdot ft - 32ft}{2} + \frac{32 \cdot ft - 0ft}{2}$$ $\boxed{Hw8\_7 = 32\,ft}$ Case 7 : Col line 3

$$Hw8\_8 := \frac{64 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0ft}{2}$$ $\boxed{Hw8\_8 = 32\,ft}$ Case 8 : Col line 2

Case 9: No exterior wall: $\boxed{Hw8\_9 := 0 \cdot ft}$ Case 9 : Col line 3

$$Hw8 := \begin{pmatrix} Hw8\_5 \\ Hw8\_6 \\ Hw8\_7 \\ Hw8\_8 \\ Hw8\_9 \end{pmatrix} \qquad \boxed{Hw8 = \begin{pmatrix} 0 \\ 32 \\ 32 \\ 32 \\ 0 \end{pmatrix} ft}$$

Case 5 : Col line 8
Case 6 : Col line 3
Case 7 : Col line 3
Case 8 : Col line 2
Case 9 : Col line 3

Thickness of Wall 8 (West wall) :

$$\boxed{Tw8 := \begin{pmatrix} 0 \\ 4 \\ 4 \\ 2 \\ 0 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8
Case 9

$WWew\_ext\_5to9 := \overrightarrow{\left[(Hw7 \cdot Tw7 + Hw8 \cdot Tw8) \cdot w_{conc}\right]}$

Weight of East and West Walls (per foot)
Tributary to Diaphragm

$$\boxed{WWew\_ext\_5to9 = \begin{pmatrix} 4.8 \\ 38.4 \\ 19.2 \\ 9.6 \\ 0.0 \end{pmatrix} klf}$$

Case 5
Case 6
Case 7
Case 8
Case 9

6.5.4.2 Weight of Interior Walls (per Foot) : WW_{ew_int}

$$\boxed{S_{ew\_5to9} := \begin{pmatrix} 74 \\ 82 \\ 39 \\ 74 \\ 74 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Spans
East/West Seismic Acceleration
See Section 6.5.1.

<u>Case 5 : Wall at column line 8/C-E</u>

$H5ew := \dfrac{72 \cdot ft - 32 \cdot ft}{2} + \dfrac{32 \cdot ft - 0ft}{2}$ $H5ew = 36\,ft$ Height

$L5ew := 74 \cdot ft$ Length $T5ew := 4 \cdot ft$ Thickness

$WWew\_int\_5 := \dfrac{H5ew \cdot T5ew \cdot L5ew}{S_{ew\_5to9_1}} \cdot w_{conc}$

$\boxed{WWew\_int\_5 = 21.6\,klf}$

<u>Case 6 : Wall (3) at col lines 4, 6, & 7</u>

$H6ew := \dfrac{64 \cdot ft - 32 \cdot ft}{2} + \dfrac{32 \cdot ft - 0ft}{2}$ $H6ew = 32\,ft$ Height $T6ew := 4 \cdot ft$ Thickness

$L_{ew} := 82 \cdot ft$ 3 Walls @ column lines 4, 6, & 7

$L6ew := 3 \cdot L_{ew}$ $L6ew = 246\,ft$ Total length

$WWew\_int\_6 := \dfrac{H6ew \cdot T6ew \cdot L6ew}{S_{ew\_5to9_2}} \cdot w_{conc}$

$\boxed{WWew\_int\_6 = 57.6\,klf}$

<u>Case 7 : Wall at column line 4/A-B</u>

$H7ew := \dfrac{64 \cdot ft - 32 \cdot ft}{2} + \dfrac{32 \cdot ft - 0ft}{2}$ $H7ew = 32\,ft$ Height

$L7ew := 39 \cdot ft$ Length $T7ew := 4 \cdot ft$ Thickness

$WWew\_int\_7 := \dfrac{H7ew \cdot T7ew \cdot L7ew}{S_{ew\_5to9_3}} \cdot w_{conc}$

$\boxed{WWew\_int\_7 = 19.2\,klf}$

Case 8 : Wall at column line 3/C-E

$$H8ew := \frac{64 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0ft}{2} \qquad H8ew = 32\,ft \qquad \text{Height}$$

$L8ew := 74 \cdot ft$ Length $T8ew := 4 \cdot ft$ Thickness

$$WWew\_int\_8 := \frac{H8ew \cdot T8ew \cdot L8ew}{S_{ew\_5to9_4}} \cdot w_{conc}$$

$$\boxed{WWew\_int\_8 = 19.2\,klf}$$

Case 9 : Walls at column lines 3/C-E, 4/C-E, 5/C-E & 6/C-E

$$H9ew_1 := \frac{64 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0ft}{2} \qquad H9ew_1 = 32\,ft \qquad \text{Height}$$

$L9ew_1 := 74 \cdot ft$ Length $T9ew_1 := 4 \cdot ft$ Thickness

$$H9ew_2 := \frac{32 \cdot ft - 0ft}{2} \qquad H9ew_2 = 16\,ft \qquad \text{Height}$$

$L9ew_2 := 74 \cdot ft$ Length $T9ew_2 := 4 \cdot ft$ Thickness

$$WWew\_int\_9 := \frac{\left(2 \cdot H9ew_1 \cdot T9ew_1 \cdot L9ew_1\right) + \left(2\,H9ew_2 \cdot T9ew_2 \cdot L9ew_2\right)}{S_{ew\_5to9_5}} \cdot w_{conc}$$

$$\boxed{WWew\_int\_9 = 57.6\,klf}$$

$$WWew\_int\_5to9 := \begin{pmatrix} WWew\_int\_5 \\ WWew\_int\_6 \\ WWew\_int\_7 \\ WWew\_int\_8 \\ WWew\_int\_9 \end{pmatrix} \qquad \boxed{WWew\_int\_5to9 = \begin{pmatrix} 21.6 \\ 57.6 \\ 19.2 \\ 19.2 \\ 57.6 \end{pmatrix} klf} \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Weight of East and West Walls (per foot)
Tributary to Diaphragm

6.5.4.3 Total Weight of East & West Walls Tributary to Diaphragm : WW_{ew}

$$WWew\_5to9 := WWew\_ext\_5to9 + WWew\_int\_5to9 \qquad \boxed{WWew\_5to9 = \begin{pmatrix} 26.4 \\ 96.0 \\ 38.4 \\ 28.8 \\ 57.6 \end{pmatrix} klf} \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Total weight of East & West Walls(per foot)
Tributary to Diaphragm

6.5.5 Design for North/South Seismic Acceleration

6.5.5.1 Moments and Shears

$$S_{ns\_5to9} = \begin{pmatrix} 43 \\ 59 \\ 46 \\ 43 \\ 105 \end{pmatrix} ft$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Spans
North/South Seismic Acceleration
See Section 6.5.3.2.

$$DD_{ns\_5to9} := \begin{pmatrix} 74 \\ 82 \\ 39 \\ 74 \\ 74 \end{pmatrix} \cdot ft$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Depths
North/South Seismic Acceleration
See Section 6.5.1.

$$SACC_{y\_5to9} := \begin{pmatrix} a32_y \\ a32_y \\ a32_y \\ a32_y \\ a32_y \end{pmatrix}$$

El. 32'
El. 32'
El. 32'
El. 32'
El. 32'

$$SACC_{y\_5to9} = \begin{pmatrix} 0.69 \\ 0.69 \\ 0.69 \\ 0.69 \\ 0.69 \end{pmatrix}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Seismic Acceleration
Factors in
Y (North/South) Direction
See Section 6.2.6.

$$U_{\_5to9} = \begin{pmatrix} 359 \\ 409 \\ 409 \\ 409 \\ 725 \end{pmatrix} psf$$

Case 5 18" roof slab
Case 6 18" floor slab
Case 7 18" floor slab
Case 8 18" floor slab
Case 9 48" floor slab

Vertical Loads
Section 6.4.2

<u>Seismic Force in the North/South Direction</u>

$$W_{ns\_5to9} := \overrightarrow{\left[\left(\underbrace{U_{\_5to9}}_{\text{Weight of Slab}} \cdot \underbrace{DD_{ns\_5to9}}_{\text{Diaphragm Depth}} + \underbrace{WWns\_5to9}_{\text{Weight of Walls}}\right) \cdot \underbrace{SACC_{y\_5to9}}_{\text{Seismic Acceleration}}\right]}$$

Horizontal Seismic Load for diaphragm per Foot (N/S dir)

$$W_{ns\_5to9} = \begin{pmatrix} 31.6 \\ 49.6 \\ 37.5 \\ 47.4 \\ 69.0 \end{pmatrix} klf$$

Case 5
Case 6
Case 7
Case 8
Case 9

Loads

For $U_{\_5to9}$, see Sec. 6.5.2
For DD_{ns_5to9}, see Sec. 6.5.1
For WWns_5to9, see Sec. 6.5.3.3
For $SACC_{y\_5to9}$, see Sec. 6.2.6

$$M_{ns\_5to9} := \overrightarrow{\frac{W_{ns\_5to9} \cdot S_{ns\_5to9}^{2}}{10}} \qquad \boxed{M_{ns\_5to9} = \begin{pmatrix} 5837 \\ 17274 \\ 7934 \\ 8758 \\ 76076 \end{pmatrix} ft \cdot kip}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Moment

$$V_{ns\_5to9} := \overrightarrow{\frac{1.15W_{ns\_5to9} \cdot S_{ns\_5to}}{2}} \qquad \boxed{}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Shear

6.5.5.2 Check Shear Strength as per ACI 349 (Ref. 2.2.1, Sect. 21.6.5.6)

$$\boxed{h_{5to9} := \begin{pmatrix} 18 \\ 18 \\ 18 \\ 18 \\ 48 \end{pmatrix} \cdot in}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Slab Thickness, See Section 6.5.1.

Note: Shear strength for case 9 is to be considered as the full depth of 48" as the maximum shear force is outside the 18" depression in the slab.

$$A_{cv\_ns\_5to9} := \overrightarrow{\left(DD_{ns\_5to9} \cdot h_{5to9}\right)}$$

$$\boxed{A_{cv\_ns\_5to9} = \begin{pmatrix} 15984 \\ 17712 \\ 8424 \\ 15984 \\ 42624 \end{pmatrix} in^{2}}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Shear Areas

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.1, Sect. 9.3.4)]

Check Limiting Diaphragm Shear Strength

$\phi V_{n\_max\_ns\_5to9} := \phi_{diaph} \cdot 8 \cdot A_{cv\_ns\_5to9} \cdot \sqrt{f_c \cdot psi}$ ACI 349 (Ref. 2.2.1, Sect. 21.6.5.6)

$\phi_{diaph} = 0.6$ ACI 349 (Ref 2.2.1, Sect 9.3.4)

$$\phi V_{n\_max\_ns\_5to9} = \begin{pmatrix} 5425 \\ 6012 \\ 2859 \\ 5425 \\ 14467 \end{pmatrix} kip \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Shear Strength

$$V_{ns\_5to9} = \begin{pmatrix} 780 \\ 1683 \\ 992 \\ 1171 \\ 4166 \end{pmatrix} kip$$

Shear See Section 6.5.5.1.

Compare limiting shear capacity per ACI 349 to shear requirements. If $\phi V_{n\_max\_ns\_5to9} > V_{ns\_5to9}$, no shear reinforcement is required.

$\phi V_{n\_max\_ns_1} :=$ "OK" if $\phi V_{n\_max\_ns\_5to9_1} > V_{ns\_5to9_1}$; "NG" otherwise — Case 5

$\phi V_{n\_max\_ns_2} :=$ "OK" if $\phi V_{n\_max\_ns\_5to9_2} > V_{ns\_5to9_2}$; "NG" otherwise — Case 6

$\phi V_{n\_max\_ns_3} :=$ "OK" if $\phi V_{n\_max\_ns\_5to9_3} > V_{ns\_5to9_3}$; "NG" otherwise — Case 7

$\phi V_{n\_max\_ns_4} :=$ "OK" if $\phi V_{n\_max\_ns\_5to9_4} > V_{ns\_5to9_4}$; "NG" otherwise — Case 8

$\phi V_{n\_max\_ns_5} :=$ "OK" if $\phi V_{n\_max\_ns\_5to9_5} > V_{ns\_5to9_5}$; "NG" otherwise — Case 9

$$\phi V_{n\_max\_ns} := \begin{pmatrix} \phi V_{n\_max\_ns_1} \\ \phi V_{n\_max\_ns_2} \\ \phi V_{n\_max\_ns_3} \\ \phi V_{n\_max\_ns_4} \\ \phi V_{n\_max\_ns_5} \end{pmatrix} \qquad \phi V_{n\_max\_ns} = \begin{pmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{pmatrix} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

$\phi V_{n\_max\_ns\_5to9} > V_{ns\_5to9}$ Limiting diaphragm shear strength satisfied.

6.5.5.3 Shear Reinforcement as per ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.1, Sect. 9.3.4)]

$$\frac{V_{ns\_5to9}}{\phi_{diaph}} := V_{c\_ns\_5to9} + V_{s\_ns\_5to9}$$

Total Shear Strength = Concrete Shear Strength + Reinforcing Steel Shear Strength

$$V_{c\_ns\_5to9} := 2 \cdot A_{cv\_ns\_5to9} \cdot \sqrt{f_c \cdot psi}$$

ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$$\phi_{diaph} \cdot V_{c\_ns\_5to9} = \begin{pmatrix} 1356 \\ 1503 \\ 715 \\ 1356 \\ 3617 \end{pmatrix} kip \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Shear Strength

$$V_{ns\_5to9} = \begin{pmatrix} 780 \\ 1683 \\ 992 \\ 1171 \\ 4166 \end{pmatrix} kip$$

Shear See Section 6.5.5.1.

Compare concrete shear capacity per ACI 349 to shear requirements. If $\phi_{diaph} \cdot V_{c\_ns\_5to9} > V_{ns\_5to9}$, no shear reinforcement is required.

$$\phi V_{c\_ns_1} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_5to9_1} > V_{ns\_5to9_1} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 5

$$\phi V_{c\_ns_2} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_5to9_2} > V_{ns\_5to9_2} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 6

$$\phi V_{c\_ns_3} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_5to9_3} > V_{ns\_5to9_3} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 7

$$\phi V_{c\_ns_4} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_5to9_4} > V_{ns\_5to9_4} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 8

$$\phi V_{c\_ns_5} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ns\_5to9_5} > V_{ns\_5to9_5} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Case 9

$$\phi V_{n\_max\_ns} := \begin{pmatrix} \phi V_{c\_ns_1} \\ \phi V_{c\_ns_2} \\ \phi V_{c\_ns_3} \\ \phi V_{c\_ns_4} \\ \phi V_{c\_ns_5} \end{pmatrix} \qquad \boxed{\phi V_{n\_max\_ns} = \begin{pmatrix} \text{"OK"} \\ \text{"NG"} \\ \text{"NG"} \\ \text{"OK"} \\ \text{"NG"} \end{pmatrix}} \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

For cases 5 & 8 the concrete shear strength [$\phi_{diaph} \cdot V_{c\_ns\_5to9}$] is more than the shear demand [V_{ns_5to9}] therefore shear reinforcing is not required. For cases 6, 7 & 9 the concrete shear strength is less than the shear demand, therefore shear reinforcing is required.

$$V_{s\_ns\_5to9} := \frac{V_{ns\_5to9}}{\phi_{diaph}} - V_{c\_ns\_5to9}$$ MacGregor (Ref 2.2.4, Page 207)

$$\boxed{V_{s\_ns\_5to9} = \begin{pmatrix} -960 \\ 301 \\ 462 \\ -309 \\ 915 \end{pmatrix} \text{kip}} \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

$$\boxed{V_{s\_ns\_5to9rev_1} := \begin{cases} V_{s\_ns\_5to9_1} & \text{if } V_{s\_ns\_5to9_1} \ge 0\text{kip} \\ 0.0 \cdot \text{kip} & \text{otherwise} \end{cases}} \qquad \text{Case 5}$$

$$\boxed{V_{s\_ns\_5to9rev_2} := \begin{cases} V_{s\_ns\_5to9_2} & \text{if } V_{s\_ns\_5to9_2} \ge 0\text{kip} \\ 0.0 \cdot \text{kip} & \text{otherwise} \end{cases}} \qquad \text{Case 6}$$

$$\boxed{V_{s\_ns\_5to9rev_3} := \begin{cases} V_{s\_ns\_5to9_3} & \text{if } V_{s\_ns\_5to9_3} \ge 0\text{kip} \\ 0.0 \cdot \text{kip} & \text{otherwise} \end{cases}} \qquad \text{Case 7}$$

$$\boxed{V_{s\_ns\_5to9rev_4} := \begin{cases} V_{s\_ns\_5to9_4} & \text{if } V_{s\_ns\_5to9_4} \ge 0\text{kip} \\ 0.0 \cdot \text{kip} & \text{otherwise} \end{cases}} \qquad \text{Case 8}$$

$$\boxed{V_{s\_ns\_5to9rev_5} := \begin{cases} V_{s\_ns\_5to9_5} & \text{if } V_{s\_ns\_5to9_5} \ge 0\text{kip} \\ 0.0 \cdot \text{kip} & \text{otherwise} \end{cases}} \qquad \text{Case 9}$$

$$V_{s\_ns\_5to9rev} = \begin{pmatrix} 0 \\ 301 \\ 462 \\ 0 \\ 915 \end{pmatrix} kip \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Required Reinforcing Shear Strength
("0 kip" indicates that no shear reinforcing is required.)

6.5.5.4 Required Reinforcement

$$A_{cv\_ns\_5to9} = \begin{pmatrix} 15984 \\ 17712 \\ 8424 \\ 15984 \\ 42624 \end{pmatrix} in^2 \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Shear
Areas

$f_y = 60\ ksi$

$$\rho_{req\_ns\_5to9} := \frac{V_{s\_ns\_5to9rev}}{A_{cv\_ns\_5to9} \cdot f_y}$$

ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$$\rho_{req\_ns\_5to9} = \begin{pmatrix} 0.00000 \\ 0.00028 \\ 0.00091 \\ 0.00000 \\ 0.00036 \end{pmatrix} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Required Reinforcement
(total required on 2 faces)

Note : See Section 6.6.2.3 for total steel required for out of plane
(vertical) and in-plane (horizontal) loads.

6.5.6 Design for East/West Seismic Acceleration

6.5.6.1 Moments and Shears

$$S_{ew\_5to9} = \begin{pmatrix} 74 \\ 82 \\ 39 \\ 74 \\ 74 \end{pmatrix} ft \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Diaphragm Spans
East/West Seismic Acceleration
See Section 6.5.4.2.

$$DD_{ew\_5to9} := \begin{pmatrix} 43 \\ 196 \\ 46 \\ 43 \\ 105 \end{pmatrix} \cdot ft \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Diaphragm Depths
East/West Seismic Acceleration
See Section 6.5.1.

$$SACC_{x\_5to9} := \begin{pmatrix} a32_x \\ a32_x \\ a32_x \\ a32_x \\ a32_x \end{pmatrix} \quad \begin{matrix} \text{El. 32'} \\ \text{El. 32'} \\ \text{El. 32'} \\ \text{El. 32'} \\ \text{El. 32'} \end{matrix}$$

$$SACC_{x\_5to9} = \begin{pmatrix} 0.71 \\ 0.71 \\ 0.71 \\ 0.71 \\ 0.71 \end{pmatrix} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Seismic Acceleration
Factors in
X (East/West) Direction
See Section 6.2.6.

$$U_{\_5to9} = \begin{pmatrix} 359 \\ 409 \\ 409 \\ 409 \\ 725 \end{pmatrix} psf \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix} \quad \begin{matrix} \text{18" roof slab} \\ \text{18" floor slab} \\ \text{18" floor slab} \\ \text{18" floor slab} \\ \text{48" floor slab} \end{matrix}$$

Vertical Loads
Section 6.4.2

Seismic Force in the East/West Direction

$$W_{ew\_5to9} := \overrightarrow{\left[\left(\underset{\text{Weight of Slab}}{U_{\_5to9}} \cdot \underset{\text{Diaphragm Depth}}{DD_{ew\_5to9}} + \underset{\text{Weight of Walls}}{WWew\_5to9}\right) \cdot \underset{\text{Seismic Acceleration}}{SACC_{x\_5to9}}\right]}$$

Horizontal Seismic Load for diaphragm per Foot (E/W dir)

$$W_{ew\_5to9} = \begin{pmatrix} 29.7 \\ 125.0 \\ 40.6 \\ 32.9 \\ 94.9 \end{pmatrix} \text{klf} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Loads

For $U_{\_5to9}$, see Sec. 6.4.2
For DD_{ew_5to9}, see Sec. 6.5.1
For WWew_5to9, see Sec. 6.5.4.3
For $SACC_{x\_5to9}$, see Sec. 6.2.6

$$M_{ew\_5to9} := \overrightarrow{\frac{W_{ew\_5to9} \cdot S_{ew\_5to9}^{\;2}}{10}} \qquad M_{ew\_5to9} = \begin{pmatrix} 16262 \\ 84078 \\ 6177 \\ 18031 \\ 51992 \end{pmatrix} \text{ft} \cdot \text{kip} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Diaphragm Moment

$$V_{ew\_5to9} := \overrightarrow{\frac{1.15 W_{ew\_5to9} \cdot S_{ew\_5to9}}{2}} \qquad V_{ew\_5to9} = \begin{pmatrix} 1264 \\ 5896 \\ 911 \\ 1401 \\ 4040 \end{pmatrix} \text{kip} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Diaphragm Shear

6.5.6.2 Check Shear Strength as per ACI 349 (Ref. 2.2.1, Sect. 21.6.5.6)

$$h_{5to9} = \begin{pmatrix} 18 \\ 18 \\ 18 \\ 18 \\ 48 \end{pmatrix} \text{in} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Slab Thickness, See Section 6.5.5.2

Note: Shear strength for case 9 is to be considered as the full depth of 48" as the maximum shear force is outside the 18" depression in the slab.

$$A_{cv\_ew\_5to9} := \overrightarrow{\left(DD_{ew\_5to9} \cdot h_{5to9}\right)} \qquad A_{cv\_ew\_5to9} = \begin{pmatrix} 9288 \\ 42336 \\ 9936 \\ 9288 \\ 60480 \end{pmatrix} \text{in}^2 \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Shear Areas

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.1, Sect. 9.3.4)]

Check Limiting Diaphragm Shear Strength

$\phi V_{n\_max\_ew\_5to9} := \phi_{diaph} \cdot 8 \cdot A_{cv\_ew\_5to9} \cdot \sqrt{f_c \cdot psi}$ ACI 349 (Ref. 2.2.1, Sect. 21.6.5.6)

$$\phi V_{n\_max\_ew\_5to9} = \begin{pmatrix} 3152 \\ 14369 \\ 3372 \\ 3152 \\ 20528 \end{pmatrix} kip \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Shear Strength

$$V_{ew\_5to9} = \begin{pmatrix} 1264 \\ 5896 \\ 911 \\ 1401 \\ 4040 \end{pmatrix} kip$$

Shear See Section 6.5.5.1.

Compare limiting shear capacity per ACI 349 to shear requirements. If $\phi V_{n\_max\_ew\_5to9} > V_{ew\_5to9}$, no shear reinforcement is required.

$$\phi V_{n\_max\_ew_1} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_5to9_1} > V_{ew\_5to9_1} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 5

$$\phi V_{n\_max\_ew_2} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_5to9_2} > V_{ew\_5to9_2} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 6

$$\phi V_{n\_max\_ew_3} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_5to9_3} > V_{ew\_5to9_3} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 7

$$\phi V_{n\_max\_ew_4} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_5to9_4} > V_{ew\_5to9_4} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 8

$$\phi V_{n\_max\_ew_5} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n\_max\_ew\_5to9_5} > V_{ew\_5to9_5} \\ \text{"NG"} & \text{otherwise} \end{cases}$$ Case 9

$$\phi V_{n\_max\_ew} := \begin{pmatrix} \phi V_{n\_max\_ew_1} \\ \phi V_{n\_max\_ew_2} \\ \phi V_{n\_max\_ew_3} \\ \phi V_{n\_max\_ew_4} \\ \phi V_{n\_max\_ew_5} \end{pmatrix} \quad \phi V_{n\_max\_ew} = \begin{pmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{pmatrix} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

$\phi V_{n\_max\_ew\_5to9} > V_{ew\_5to9}$ Limiting diaphragm shear strength satisfied.

6.5.6.3 Shear Reinforcement as per ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.1, Sect. 9.3.4)]

$\frac{V_{ew\_5to9}}{\phi_{diaph}} := V_{c\_ew\_5to9} + V_{s\_ew\_5to9}$ Total Shear Strength = Concrete Shear Strength + Reinforcing Steel Shear Strength

$V_{c\_ew\_5to9} := 2 \cdot A_{cv\_ew\_5to9} \cdot \sqrt{f_c \cdot psi}$ ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$\phi_{diaph} \cdot V_{c\_ew\_5to9} = \begin{pmatrix} 788 \\ 3592 \\ 843 \\ 788 \\ 5132 \end{pmatrix} kip$ Case 5, Case 6, Case 7, Case 8, Case 9 — Shear Strength

$V_{ew\_5to9} = \begin{pmatrix} 1264 \\ 5896 \\ 911 \\ 1401 \\ 4040 \end{pmatrix} kip$ Shear See Section 6.5.5.1.

Compare concrete shear capacity per ACI 349 to shear requirements. If $\phi_{diaph} \cdot V_{c\_ew\_5to9} > V_{ew\_5to9}$, no shear reinforcement is required.

$\phi V_{c\_ew_1} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_5to9_1} > V_{ew\_5to9_1} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 5

$\phi V_{c\_ew_2} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_5to9_2} > V_{ew\_5to9_2} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 6

$\phi V_{c\_ew_3} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_5to9_3} > V_{ew\_5to9_3} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 7

$\phi V_{c\_ew_4} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_5to9_4} > V_{ew\_5to9_4} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 8

$\phi V_{c\_ew_5} := \begin{cases} \text{"OK"} & \text{if } \phi_{diaph} \cdot V_{c\_ew\_5to9_5} > V_{ew\_5to9_5} \\ \text{"NG"} & \text{otherwise} \end{cases}$ Case 9

$$\phi V_{n\_max\_ew} := \begin{pmatrix} \phi V_{c\_ew_1} \\ \phi V_{c\_ew_2} \\ \phi V_{c\_ew_3} \\ \phi V_{c\_ew_4} \\ \phi V_{c\_ew_5} \end{pmatrix} \qquad \boxed{\phi V_{n\_max\_ew} = \begin{pmatrix} \text{"NG"} \\ \text{"NG"} \\ \text{"NG"} \\ \text{"NG"} \\ \text{"OK"} \end{pmatrix}} \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

For case 9 the concrete shear strength [$\phi_{diaph} \cdot V_{c\_ns\_5to9}$] is more than the shear demand [V_{ns_5to9}] therefore shear reinforcing is not required. For cases 5, 6, 7 & 8 the concrete shear strength is less than the shear demand, therefore shear reinforcing is required.

$$V_{s\_ew\_5to9} := \frac{V_{ew\_5to9}}{\phi_{diaph}} - V_{c\_ew\_5to9}$$ MacGregor (Ref 2.2.4, Page 207)

$$\boxed{V_{s\_ew\_5to9} = \begin{pmatrix} 792 \\ 3839 \\ 113 \\ 1022 \\ -1820 \end{pmatrix} kip} \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

$$V_{s\_ew\_5to9rev_1} := \begin{cases} V_{s\_ew\_5to9_1} & \text{if } V_{s\_ew\_5to9_1} \geq 0kip \\ 0.0 \cdot kip & \text{otherwise} \end{cases}$$ Case 5

$$V_{s\_ew\_5to9rev_2} := \begin{cases} V_{s\_ew\_5to9_2} & \text{if } V_{s\_ew\_5to9_2} \geq 0kip \\ 0.0 \cdot kip & \text{otherwise} \end{cases}$$ Case 6

$$V_{s\_ew\_5to9rev_3} := \begin{cases} V_{s\_ew\_5to9_3} & \text{if } V_{s\_ew\_5to9_3} \geq 0kip \\ 0.0 \cdot kip & \text{otherwise} \end{cases}$$ Case 7

$$V_{s\_ew\_5to9rev_4} := \begin{cases} V_{s\_ew\_5to9_4} & \text{if } V_{s\_ew\_5to9_4} \geq 0kip \\ 0.0 \cdot kip & \text{otherwise} \end{cases}$$ Case 8

$$V_{s\_ew\_5to9rev_5} := \begin{cases} V_{s\_ew\_5to9_5} & \text{if } V_{s\_ew\_5to9_5} \geq 0kip \\ 0.0 \cdot kip & \text{otherwise} \end{cases}$$ Case 9

$$V_{s\_ew\_5to9rev} = \begin{pmatrix} 792 \\ 3839 \\ 113 \\ 1022 \\ 0 \end{pmatrix} kip \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Required Reinforcing Shear Strength
("0 kip" indicates that no shear reinforcing is required.)

6.5.6.4 Required Reinforcement

$$A_{cv\_ew\_5to9} = \begin{pmatrix} 9288 \\ 42336 \\ 9936 \\ 9288 \\ 60480 \end{pmatrix} in^2 \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Shear
Areas

$f_y = 60\ ksi$

$$\rho_{req\_ew\_5to9} := \frac{V_{s\_ew\_5to9rev}}{A_{cv\_ew\_5to9} \cdot f_y}$$

ACI 349 (Ref. 2.2.1, Sect. 21.6.5.2)

$$\rho_{req\_ew\_5to9} = \begin{pmatrix} 0.00142 \\ 0.00151 \\ 0.00019 \\ 0.00183 \\ 0.00000 \end{pmatrix} \quad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Required Reinforcement
(total required on 2 faces)

Note : See Section 6.6.2.3 for total steel required for out of plane
(vertical) and in-plane (horizontal) loads.

6.5.7 Design Chord Steel

6.5.7.1 North/South Seismic Acceleration

$$M_{ns\_5to9} = \begin{pmatrix} 5837 \\ 17274 \\ 7934 \\ 8758 \\ 76076 \end{pmatrix} ft \cdot kip$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Moment
See Section 6.5.5.1.

$$DD_{ns\_5to9} = \begin{pmatrix} 74 \\ 82 \\ 39 \\ 74 \\ 74 \end{pmatrix} ft$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Depths
See Section 6.5.5.1.

Chord Force (CF):

$$CF_{ns\_5to9} := \frac{M_{ns\_5to9}}{.9 \cdot DD_{ns\_5to9}}$$

Note : 0.9 x Depth is taken as lever arm from centroid of compression stress block to centroid of chord reinforcing steel, thus chord steel will be provided over a width equal to 10% of the diaphragm depth.

$$CF_{ns\_5to9} = \begin{pmatrix} 88 \\ 234 \\ 226 \\ 132 \\ 1142 \end{pmatrix} kip$$

Case 5
Case 6
Case 7
Case 8
Case 9

Chord Forces

Required Chord Steel (A_{ch}):

$\phi_b = 0.9$

Strength reduction factor for bending
ACI 349 (Ref. 2.2.1, Sect. 9.3.2.1)

$f_y = 60\ ksi$

$$A_{ch\_ns\_5to9} := \frac{CF_{ns\_5to9}}{\phi_b \cdot f_y}$$

$$A_{ch\_ns\_5to9} = \begin{pmatrix} 1.6 \\ 4.3 \\ 4.2 \\ 2.4 \\ 21.2 \end{pmatrix} in^2$$

Case 5
Case 6
Case 7
Case 8
Case 9

Required Chord Steel

Provided Chord Steel (A_{ch}):

Use 2, 6, 6 & 4 # 9 bars for cases 5, 6, 7 & 8 and use 14 # 11 bars for case 9 respectively (N/S Seismic Acceleration).

Chord steel provided = 2.0, 6.0, 6.0, 4.0 & 21.8 in^2 for cases 5, 6, 7, 8 & 9 respectively (N/S Seismic Acceleration).

See Section 6.6.2.7 for chord steel provided.

6.5.7.2 East/West Seismic Acceleration

$$\boxed{M_{ew\_5to9} = \begin{pmatrix} 16262 \\ 84078 \\ 6177 \\ 18031 \\ 51992 \end{pmatrix} ft \cdot kip}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Moment
See Section 6.5.6.1.

$$\boxed{DD_{ew\_5to9} = \begin{pmatrix} 43 \\ 196 \\ 46 \\ 43 \\ 105 \end{pmatrix} ft}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Diaphragm Depths
See Section 6.5.6.1.

Chord Force (CF):

$$CF_{ew\_5to9} := \frac{M_{ew\_5to9}}{.9 \cdot DD_{ew\_5to9}}$$

Note : 0.9 x Depth is taken as lever arm from centroid of compression stress block to centroid of chord reinforcing steel, thus chord steel will be provided over a width equal to 10% of the diaphragm depth.

$$\boxed{CF_{ew\_5to9} = \begin{pmatrix} 420 \\ 477 \\ 149 \\ 466 \\ 550 \end{pmatrix} kip}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Chord Forces

Required Chord Steel (A_{ch}):

$\phi_b = 0.9$ Strength reduction factor for bending ACI 349 (Ref. 2.2.1, Sect. 9.3.2.1)

$f_y = 60\ ksi$

$$A_{ch\_ew\_5to9} := \frac{CF_{ew\_5to9}}{\phi_b \cdot f_y} \qquad \boxed{A_{ch\_ew\_5to9} = \begin{pmatrix} 7.8 \\ 8.8 \\ 2.8 \\ 8.6 \\ 10.2 \end{pmatrix} in^2} \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

Required Chord Steel

Provided Chord Steel (A_{ch}):

Use 8 & 4 # 9 bars for cases 5 & 7 and use 6, 6 & 8 # 11 bars for cases 6, 8 & 9 respectively (E/W Seismic Acceleration).

Chord steel provided = 8.0, 9.4, 4.0, 9.4 & 12.5 in^2 for cases 5, 6, 7, 8 & 9 respectively (E/W Seismic Acceleration).

See Section 6.6.2.7 for chord steel provided.

6.6 REINFORCEMENT FOR SLAB AND CHORD

6.6.1 Analysis Cases 1 to 4

See Attachment A for plans of the RF showing the diaphragms.

Case 1: Roof Diaphragm @ El.100':Col Lines 3-6/C-E (N/S & E/W dir): 18" slab
(See Attachment A, Sheet A6)

Case 2: Roof Diaphragm @ El. 72': Col Lines 6-8/C-E (N/S & E/W dir): 18" slab
(See Attachment A, Sheet A5)

Case 3: Roof Diaphragm @ El. 64': Col Lines 4-6/A-C (N/S dir) & 3-8/A-C (E/W dir): 18" slab
Also applies to Col Lines:

- 2-3/A-C, 6-7/A-C, 7-8/A-C, 2-3/E-F, 4-6/E-F, 6-7/E-F & 7-8/E-F (N/S dir)
- 3-8/E-F (E/W dir).

(See Attachment A, Sheet A4)

Case 4: Roof Diaphragm @ El. 64':Col Lines 2-3/C-E (N/S & E/W dir): 18" slab
(See Attachment A, Sheet A4)

6.6.1.1 Slab Reinforcement for Out-of-Plane (Vertical) Loads : See Section 6.3.6.

Note : Reinforcement is required for one face of diaphragm.

$$\rho_{req\_1to4} := \begin{pmatrix} \rho_{req_1} \\ \rho_{req_1} \\ \rho_{req_1} \\ \rho_{req_1} \end{pmatrix} \begin{array}{l} \text{18" roof slab} \\ \text{18" roof slab} \\ \text{18" roof slab} \\ \text{18" roof slab} \end{array}$$

$$\boxed{\rho_{req\_1to4} = \begin{pmatrix} 0.00037 \\ 0.00037 \\ 0.00037 \\ 0.00037 \end{pmatrix}} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

6.6.1.2 Slab Reinforcement for In-Plane (Horizontal) Loads : See Sections 6.4.5.4 and 6.4.6.4.

Note : Reinforcement is required for total two faces of diaphragm.

<u>N/S Seismic</u>

$$\boxed{\rho_{req\_ns\_1to4} = \begin{pmatrix} 0.00573 \\ 0.00089 \\ 0.00102 \\ 0.00000 \end{pmatrix}} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

<u>E/W Seismic</u>

$$\boxed{\rho_{req\_ew\_1to4} = \begin{pmatrix} 0.00194 \\ 0.00144 \\ 0.00101 \\ 0.00240 \end{pmatrix}} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

6.6.1.3 Combined Reinforcement Req'd (Reinforcement required per face of diaphragm)

Distribute the reinforcement required for in-plane shear equally in each face.

$$\rho_{req\_comb\_1to4} := \begin{pmatrix} \rho_{req\_1to4_1} + \dfrac{\max\left(\rho_{req\_ns\_1to4_1}, \rho_{req\_ew\_1to4_1}\right)}{2} \\ \rho_{req\_1to4_2} + \dfrac{\max\left(\rho_{req\_ns\_1to4_2}, \rho_{req\_ew\_1to4_2}\right)}{2} \\ \rho_{req\_1to4_3} + \dfrac{\max\left(\rho_{req\_ns\_1to4_3}, \rho_{req\_ew\_1to4_3}\right)}{2} \\ \rho_{req\_1to4_4} + \dfrac{\max\left(\rho_{req\_ns\_1to4_4}, \rho_{req\_ew\_1to4_4}\right)}{2} \end{pmatrix} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

$$\boxed{\rho_{req\_comb\_1to4} = \begin{pmatrix} 0.00323 \\ 0.00109 \\ 0.00088 \\ 0.00157 \end{pmatrix}} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

$$d_{1to4} := \begin{pmatrix} d_{18r} \\ d_{18r} \\ d_{18r} \\ d_{18r} \end{pmatrix} \quad \text{See Section 6.3.5.} \quad \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array} \qquad \boxed{d_{1to4} = \begin{pmatrix} 15.14 \\ 15.14 \\ 15.14 \\ 15.14 \end{pmatrix} in}$$

$b = 12\ in$

See Section 6.3.4.

$$As_{req\_1to4} := \overrightarrow{\left[\left(\rho_{req\_comb\_1to4}\right) \cdot b \cdot d_{1to4}\right]} \qquad \boxed{As_{req\_1to4} = \begin{pmatrix} 0.59 \\ 0.20 \\ 0.16 \\ 0.28 \end{pmatrix} in^2} \text{ per foot} \quad \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

6.6.1.4 Minimum Reinforcement Req'd

ACI 349 (Ref. 2.2.1, Sect. 10.5.1) specifies that at every section of a flexural member where tensile reinforcement is required by analysis, the area A_s provided shall not be less than equation 10-3.

ACI 349 (Ref 2.2.1, Sect. 10.5.3) specifies that the minimum tensile reinforcement is satisfied by increasing the reinforcement requirement by 1/3

$$\boxed{h_{1to4} = \begin{pmatrix} 18 \\ 18 \\ 18 \\ 18 \end{pmatrix} in}$$

Case 1
Case 2
Case 3
Case 4

slab thickness, see Section 6.4.5.2.

$b = 12\,in$	See Section 6.3.4.	
$f_c = 5000\,psi$	Compressive Strength of Concrete	(Ref. 2.2.2, Sect. 4.2.11.6.2)
$f_y = 60000\,psi$	Yield Stress of Grade 60 Reinforcing Steel	(Ref. 2.2.2, Sect. 4.2.11.6.2)

$$As_{req\_1to4\_ACI} := \begin{pmatrix} \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{1to4_1}}{f_y}, \dfrac{4\cdot As_{req\_1to4_1}}{3}\right) \\ \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{1to4_2}}{f_y}, \dfrac{4\cdot As_{req\_1to4_2}}{3}\right) \\ \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{1to4_3}}{f_y}, \dfrac{4\cdot As_{req\_1to4_3}}{3}\right) \\ \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{1to4_4}}{f_y}, \dfrac{4\cdot As_{req\_1to4_4}}{3}\right) \end{pmatrix}$$

Case 1
Case 2
Case 3
Case 4

$$\boxed{As_{req\_1to4\_ACI} = \begin{pmatrix} 0.64 \\ 0.26 \\ 0.21 \\ 0.38 \end{pmatrix} in^2}$$

per foot

Case 1
Case 2
Case 3
Case 4

ACI 349 (Ref. 2.2.1, Sect. 7.12.5) specifies that where reinforcement is required the ratio of reinforcement provided on the tension face shall not be less than .0018 times the gross concrete area.

$$As_{req\_1to4\_min} := \begin{pmatrix} \max\left(0.0018 \cdot b \cdot h_{1to4_1}, As_{req\_1to4\_ACI_1}\right) \\ \max\left(0.0018 \cdot b \cdot h_{1to4_2}, As_{req\_1to4\_ACI_2}\right) \\ \max\left(0.0018 \cdot b \cdot h_{1to4_3}, As_{req\_1to4\_ACI_3}\right) \\ \max\left(0.0018 \cdot b \cdot h_{1to4_4}, As_{req\_1to4\_ACI_4}\right) \end{pmatrix} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Ref 2.2.1 (Sect 7.12.5 & 10.5.1, Eq. 10-3

$$\boxed{As_{req\_1to4\_min} = \begin{pmatrix} 0.64 \\ 0.39 \\ 0.39 \\ 0.39 \end{pmatrix} in^2} \text{ per foot} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Minimum Reinforcement Req'd

6.6.1.5 Slab Reinforcement Req'd

$$As_{reqd\_1to4} := \begin{pmatrix} \max\left(As_{req\_1to4_1}, As_{req\_1to4\_min_1}\right) \\ \max\left(As_{req\_1to4_2}, As_{req\_1to4\_min_2}\right) \\ \max\left(As_{req\_1to4_3}, As_{req\_1to4\_min_3}\right) \\ \max\left(As_{req\_1to4_4}, As_{req\_1to4\_min_4}\right) \end{pmatrix}$$

$$\boxed{As_{reqd\_1to4} = \begin{pmatrix} 0.64 \\ 0.39 \\ 0.39 \\ 0.39 \end{pmatrix} in^2} \text{ per foot}$$

6.6.1.6 Slab Reinforcement Provided

Provide # 9 bars @ 12" on center, both ways, top & bottom for case 1 and # 7 bars @ 12" on center, both ways, top & bottom for cases 2 to 4.

$$\boxed{As_{prov\_1to4} := \begin{pmatrix} 1.00 \\ 0.60 \\ 0.60 \\ 0.60 \end{pmatrix} \cdot in^2} \text{per foot}$$

Case 1 : 18" thick roof slab @ El. 100'

Case 2 : 18" thick roof slab @ El. 72'

Case 3 : 18" thick roof slab @ El. 64'

Case 4 : 18" thick roof slab @ El. 64'

Reinforcement provided for cases 1 to 4 is more than required.

$$RR_{1to4} := \overrightarrow{\left(\frac{As_{reqd\_1to4}}{As_{prov\_1to4}}\right)}$$

$$\boxed{RR_{1to4} = \begin{pmatrix} 0.64 \\ 0.65 \\ 0.65 \\ 0.65 \end{pmatrix}} \text{ Reinforcement Ratio} \quad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

6.6.1.7 Chord Steel Provided : See Sections 6.4.7.1 & 6.4.7.2.

Chord Steel Case 1 (El. 100')

16 - # 11 bars along column lines C/E (between column lines 3 & 6)
6 - # 11 bars along column lines 3/6 (between column lines C & E)

Chord Steel Case 2 (El. 72')

6 - # 11 bars along column lines C/E (between column lines 6 & 8)
8 - # 9 bars along column lines 6/8 (between column lines C & E)

Chord Steel Case 3 (El. 64')

6 - # 9 bars along column lines A/C & E/F (between column lines 3 & 8)
8 - # 9 bars along column lines 3/8 (between column lines A/C & E/F)

Chord Steel Case 4 (El. 64')

4- # 9 bars along column lines C/E (between column lines 2 & 3)
10 - # 9 bars along column lines 2/3 (between column lines C & E)

6.6.2 Analysis Cases 5 to 9

See Attachment A for plans of the RF showing the diaphragms.

Case 5: Roof Diaphragm @ El. 32': Col Lines 8-9/C-E (N/S & E/W dir): 18" slab
(See Attachment A, Sheet A3)

Case 6: Floor Diaphragm @ El. 32': Col Lines 4-6/E-F (N/S dir) & 3-8/E-F (E/W dir): 18" slab
Also applies to Col Lines:

- 2-3/E/F, 6-7/E-F, 7-8/E-F, 4-6/A-C, 6-7/E-F, & 7-8/E-F(N/S dir)
- 3-8/A-C (E/W dir).

See Attachment A, Sheet A3)

Case 7: Floor Diaphragm @ El. 32’: Col Lines 3-4/A-B (N/S & E/W dir): 18” slab
See Attachment A, Sheet A3)

Case 8: Floor Diaphragm @ El. 32': Col Lines 2-3/C-E (N/S & E/W dir): 18" slab
See Attachment A, Sheet A3)

Case 9: Floor Diaphragm @ El.32': Col Lines 3-4/C-D (N/S & E/W dir): 48" slab
Also applies to Col Lines 3-6/C-E
See Attachment A, Sheet A3)

6.6.2.1 Slab Reinforcement for Out-of-Plane (Vertical) Loads : See Section 6.3.6.

Note : Reinforcement is required for one face of diaphragm.

$$\rho_{req\_5to9} := \begin{pmatrix} \rho_{req_1} \\ \rho_{req_2} \\ \rho_{req_2} \\ \rho_{req_2} \\ \rho_{req_3} \end{pmatrix} \begin{matrix} \text{18" roof slab} \\ \text{18" floor slab} \\ \text{18" floor slab} \\ \text{18" floor slab} \\ \text{48" floor slab} \end{matrix}$$

$$\boxed{\rho_{req\_5to9} = \begin{pmatrix} 0.00037 \\ 0.00038 \\ 0.00038 \\ 0.00038 \\ 0.0061 \end{pmatrix}} \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

6.6.2.2 Slab Reinforcement for In-Plane (Horizontal) Loads : See Sections 6.5.5.4 and 6.5.6.4.

Note : Reinforcement is required for total two faces of diaphragm.

<u>N/S Seismic</u>

$$\boxed{\rho_{req\_ns\_5to9} = \begin{pmatrix} 0.00000 \\ 0.00028 \\ 0.00091 \\ 0.00000 \\ 0.00036 \end{pmatrix}} \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

<u>E/W Seismic</u>

$$\boxed{\rho_{req\_ew\_5to9} = \begin{pmatrix} 0.00142 \\ 0.00151 \\ 0.00019 \\ 0.00183 \\ 0.00000 \end{pmatrix}} \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

6.6.2.3 Combined Reinforcement Req'd

(Reinforcement required per face of diaphragm)

Distribute the reinforcement required for in-plane shear equally in each face.

$$\rho_{req\_comb\_5to9} := \begin{pmatrix} \rho_{req\_5to9_1} + \dfrac{\max\left(\rho_{req\_ns\_5to9_1}, \rho_{req\_ew\_5to9_1}\right)}{2} \\ \rho_{req\_5to9_2} + \dfrac{\max\left(\rho_{req\_ns\_5to9_2}, \rho_{req\_ew\_5to9_2}\right)}{2} \\ \rho_{req\_5to9_3} + \dfrac{\max\left(\rho_{req\_ns\_5to9_3}, \rho_{req\_ew\_5to9_3}\right)}{2} \\ \rho_{req\_5to9_4} + \dfrac{\max\left(\rho_{req\_ns\_5to9_4}, \rho_{req\_ew\_5to9_4}\right)}{2} \\ \rho_{req\_5to9_5} + \dfrac{\max\left(\rho_{req\_ns\_5to9_5}, \rho_{req\_ew\_5to9_5}\right)}{2} \end{pmatrix} \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

$$\boxed{\rho_{req\_comb\_5to9} = \begin{pmatrix} 0.00108 \\ 0.00114 \\ 0.00084 \\ 0.00130 \\ 0.00628 \end{pmatrix}} \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

$$d_{5to9} := \begin{pmatrix} d_{18r} \\ d_{18f} \\ d_{18f} \\ d_{18f} \\ d_{48f} \end{pmatrix}$$ See Section 6.3.5.

$$\boxed{d_{5to9} = \begin{pmatrix} 15.14 \\ 15.14 \\ 15.14 \\ 15.14 \\ 27.14 \end{pmatrix} in}$$

Case 5
Case 6
Case 7
Case 8
Case 9

$b = 12\,in$ See Section 6.3.4.

$$As_{req\_5to9} := \overrightarrow{\left[\left(\rho_{req\_comb\_5to9}\right) \cdot b \cdot d_{5to9}\right]}$$

$$\boxed{As_{req\_5to9} = \begin{pmatrix} 0.20 \\ 0.21 \\ 0.15 \\ 0.24 \\ 2.05 \end{pmatrix} in^2}$$ per foot

Case 5
Case 6
Case 7
Case 8
Case 9

6.6.2.4 Minimum Reinforcement Req'd

ACI 349 (Ref. 2.2.1, Sect. 10.5.1) specifies that at every section of a flexural member where tensile reinforcement is required by analysis, the area A_s provided shall not be less than equation 10-3.

ACI 349 (Ref 2.2.1, Sect. 10.5.3) specifies that the minimum tensile reinforcement is satisfied by increasing the reinforcement requirement by 1/3

$$\boxed{h_{5to9} = \begin{pmatrix} 18 \\ 18 \\ 18 \\ 18 \\ 48 \end{pmatrix} in}$$

Case 5
Case 6
Case 7
Case 8
Case 9

Slab Thickness, See Section 6.5.1.

$b = 12\,in$ See Section 6.3.4.

$f_c = 5000\,psi$ Compressive Strength of Concrete (Ref. 2.2.2, Sect. 4.2.11.6.2)

$f_y = 60000\,psi$ Yield Stress of Grade 60 Reinforcing Steel (Ref. 2.2.2, Sect. 4.2.11.6.2)

$$As_{req\_5to9\_ACI} := \begin{pmatrix} \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{5to9_1}}{f_y}, \dfrac{4\cdot As_{req\_5to9_1}}{3}\right) \\ \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{5to9_2}}{f_y}, \dfrac{4\cdot As_{req\_5to9_2}}{3}\right) \\ \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{5to9_3}}{f_y}, \dfrac{4\cdot As_{req\_5to9_3}}{3}\right) \\ \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{5to9_4}}{f_y}, \dfrac{4\cdot As_{req\_5to9_4}}{3}\right) \\ \min\left(\dfrac{3\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d_{5to9_5}}{f_y}, \dfrac{4\cdot As_{req\_5to9_5}}{3}\right) \end{pmatrix} \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

$$\boxed{As_{req\_5to9\_ACI} = \begin{pmatrix} 0.26 \\ 0.28 \\ 0.2 \\ 0.31 \\ 1.15 \end{pmatrix} in^2} \text{ per foot } \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

ACI 349 (Ref. 2.2.1, Sect. 7.12.5) specifies that where reinforcement is required the ratio of reinforcement provided on the tension face shall not be less than .0018 times the gross concrete area.

$$As_{req\_5to9\_min} := \begin{pmatrix} \max\left(0.0018\cdot b\cdot h_{5to9_1}, As_{req\_5to9\_ACI_1}\right) \\ \max\left(0.0018\cdot b\cdot h_{5to9_2}, As_{req\_5to9\_ACI_2}\right) \\ \max\left(0.0018\cdot b\cdot h_{5to9_3}, As_{req\_5to9\_ACI_3}\right) \\ \max\left(0.0018\cdot b\cdot h_{5to9_4}, As_{req\_5to9\_ACI_4}\right) \\ \max\left(0.0018\cdot b\cdot h_{5to9_5}, As_{req\_5to9\_ACI_5}\right) \end{pmatrix} \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix}$$

$$\boxed{As_{req\_5to9\_min} = \begin{pmatrix} 0.39 \\ 0.39 \\ 0.39 \\ 0.39 \\ 1.15 \end{pmatrix} in^2} \text{ per foot } \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{matrix} \quad \text{Minimum Reinforcement Req'd}$$

6.6.2.5 Slab Reinforcement Req'd

$$As_{reqd\_5to9} := \begin{pmatrix} \max\left(As_{req\_5to9_1}, As_{req\_5to9\_min_1}\right) \\ \max\left(As_{req\_5to9_2}, As_{req\_5to9\_min_2}\right) \\ \max\left(As_{req\_5to9_3}, As_{req\_5to9\_min_3}\right) \\ \max\left(As_{req\_5to9_4}, As_{req\_5to9\_min_4}\right) \\ \max\left(As_{req\_5to9_5}, As_{req\_5to9\_min_5}\right) \end{pmatrix}$$

$$\boxed{As_{reqd\_5to9} = \begin{pmatrix} 0.39 \\ 0.39 \\ 0.39 \\ 0.39 \\ 2.05 \end{pmatrix} in^2} \text{ per foot} \quad \begin{array}{l} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{array}$$

6.6.2.6 Slab Reinforcement Provided

Provide # 7 bars @ 12" on center, both ways, top & bottom for cases 5, 6, 7 & 8.
Provide # 11 bars @ 6" on center, both ways, top & bottom for case 9.

$$\boxed{As_{prov\_5to9} := \begin{pmatrix} 0.60 \\ 0.60 \\ 0.60 \\ 0.60 \\ 3.12 \end{pmatrix} \cdot in^2} \text{ per foot}$$

Case 5 : 18" thick roof slab @ El. 32'
Case 6 : 18" thick floor slab @ El. 32'
Case 7 : 18" thick floor slab @ El. 32'
Case 8 : 18" thick floor slab @ El. 32'
Case 9 : 48" thick floor slab @ El. 32'

Reinforcement provided for cases 5 to 9 is more than required.

$$RR_{5to9} := \overrightarrow{\left(\frac{As_{reqd\_5to9}}{As_{prov\_5to9}}\right)}$$

$$\boxed{RR_{5to9} = \begin{pmatrix} 0.65 \\ 0.65 \\ 0.65 \\ 0.65 \\ 0.66 \end{pmatrix}} \quad \begin{array}{l} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \\ \text{Case 9} \end{array}$$

Reinforcement Ratio

6.6.2.7 Chord Steel Provided : See Sections 6.5.7.1 & 6.5.7.2.

See Attachment A for typical chord steel for diaphragm.

Chord Steel Case 5 (El. 32')

2- # 9 bars along column lines C/E (between column lines 8 & 9)
8- # 9 bars along column lines 8/9 (between column lines C & E)

Chord Steel Case 6 (El. 32')

6- # 9 bars along column lines A/C & E/F (between column lines 3 & 8)
6- # 11 bars along column lines 3/8 (between column lines A/C & E/F)

Chord Steel Case 7 (El. 32')

6- # 9 bars along column lines A/B (between column lines 3 & 4)
4- # 9 bars along column lines 3/4 (between column lines A & B)

Chord Steel Case 8 (El. 32')

4- # 9 bars along column lines C/E (between column lines 2 & 3)
6- # 11 bars along column lines 2/3 (between column lines C & E)

Chord Steel Case 9 (El. 32')

14- # 11 bars along column lines C/E (between column lines 3 & 6)
8- # 11 bars along column lines 3/6 (between column lines C & E)

6.7 Reinforcement Summary

Case No.	Diaphragm (a) N/S direction	Diaphragm (a) E/W direction	Slab Reinforcement (b)(c)	Chord Reinforcement N/S Direction (d)(e)	Chord Reinforcement E/W Direction (d)(e)
Case 1	3-6/C-E	3-6/C-E	# 9 @ 12" on center, top and bottom, both directions	16 - #11	6 - #11
Case 2	6-8/C-E	6-8/C-E	# 7 at 12" on center, top and bottom, both directions	6 - #11	8 - #9
Case 3	3-4/A-C & 3-4/E-F 4-6/A-C & 4-6/E-F 6-7/A-C & 6-7/E-F 7-8/A-C & 7-8/E-F	3-8/A-C & 3-8/E-F	#7at 12" on center, top and bottom, both directions	6 - #9	8 - #9
Case 4	2-3/C-E	2-3/C-E	# 7 at 12" on center, top and bottom, both directions	4 - #9	10 - #9
Case 5	8-9/C-E	8-9/C-E	# 7 at 12" on center, top and bottom, both directions	2 - #9	8 - #9
Case 6	3-4/E-F 4-6/A-C & 4-6/E-F 6-7/A-C & 6-7/E-F 7-8/A-C & 7-8/E-F	4-8/A-C & 3-8/E-F	#7at 12" on center, top and bottom, both directions	6 - #9	6 - #11
Case 7	3-4/A-B	3-4/A-B	# 7 at 12" on center, top and bottom, both directions	6 - #9	4 - #9
Case 8	2-3/C-E	2-3/C-E	# 7 at 12" on center, top and bottom, both directions	4 - #9	6 - #11
Case 9	3-6/C-E	3-6/C-E	# 11 at 6" on center, top and bottom, both directions	14 - #11	8 - #11

Notes:
a. Attachment A
b. Slab Reinforcement Cases 1-4, Section 6.6.1.6
c. Slab Reinforcement Cases 5-9, Section 6.6.2.6
d. Slab Reinforcement Cases 1-4, Section 6.6.1.7
e. Slab Reinforcement Cases 5-9, Section 6.6.2.7

7.0 RESULTS AND CONCLUSIONS

7.1 RESULTS

The results from this calculation are as follows.

7.1.1 Slab reinforcement provided : See sections 6.6.1.6 and 6.6.2.6.

- 18” thick slabs @ El. 100’:

  # 9 reinforcing bars @ 12” on center, both ways, top & bottom

- 18” thick slabs @ El. 72’, 64’ & 32’:

  # 7 reinforcing bars @ 12” on center, both ways, top & bottom

- 48” thick slab @ El. 32’:

  # 11 reinforcing bars @ 6” on center, both ways, top & bottom

- Reinforcing Ratio = Reinforcement required / Reinforcment provided

  Cases 2, 3, 4, 5, 6, 7 & 8: 0.65 Case 1: 0.64 Case 9: 0.66

7.1.2 Chord reinforcement provided:

- See sections 6.6.1.7 and 6.6.2.7.

7.2 CONCLUSIONS

Results from this calculation demonstrate that for the slabs investigated a reasonable slab design is achieved for the imposed design loads. The slab reinforcement provided in section 7.1.1 of this calculation is reasonable for the type of structure under consideration and the types of loads applied to this structure. The Reinforcement Ratio (see section 7.1.1) shows that there is adequate margin for use in consideration of larger seismic events in the probabilistic risk assessment.

Chord reinforcement provided in section 7.1.2 is based on conservatively placing the reinforcement so that the lever arm from centroid of compression stress block to centroid of chord reinforcement is 0.9 x Depth of diaphragm. During the detailed design phase of the project three dimensional finite element analysis could yield reduction in the chord reinforcment required.

ATTACHMENT A
RF BLDG. ROOF AND FLOOR PLANS

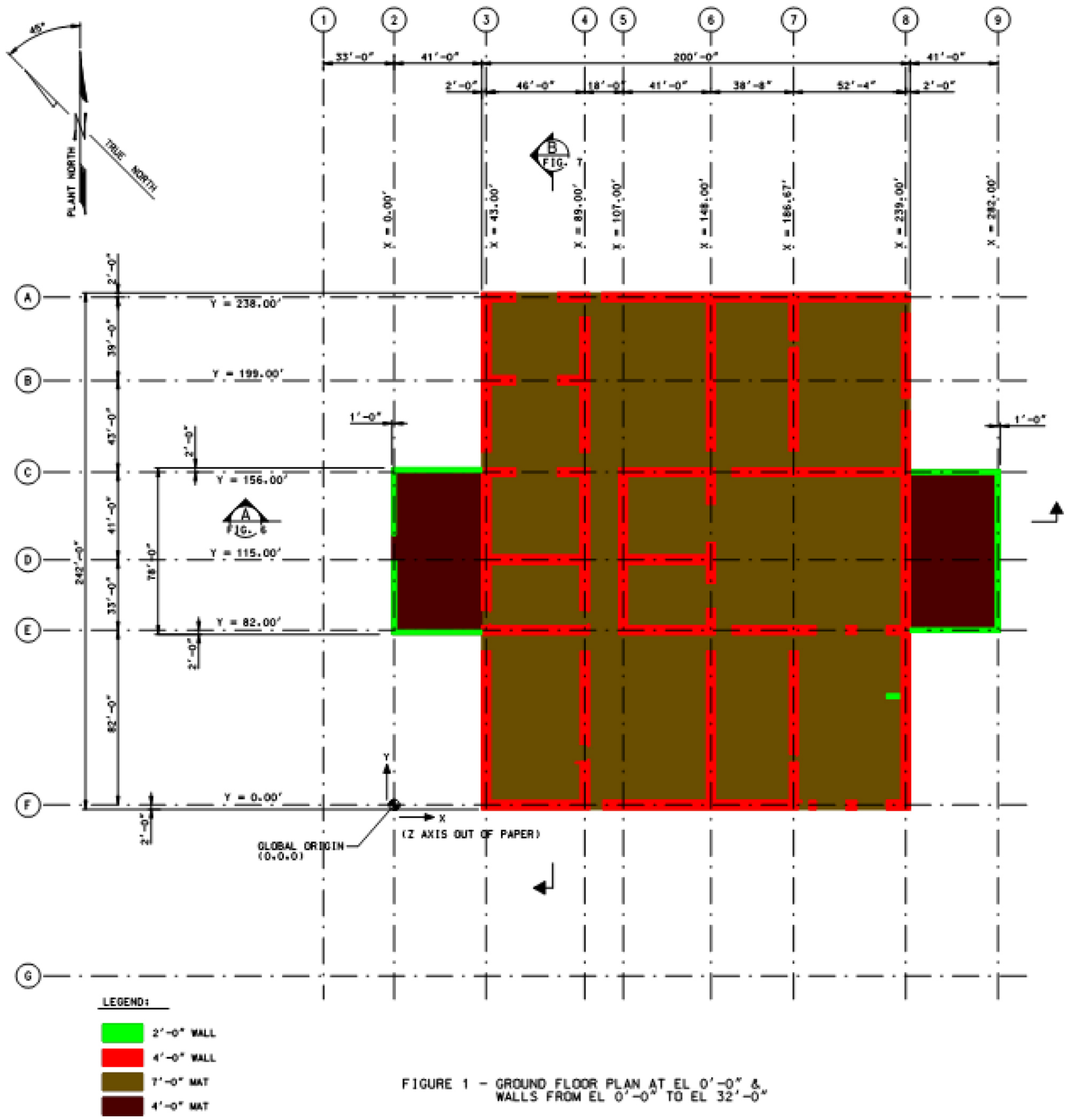


FIGURE 1 – GROUND FLOOR PLAN AT EL 0'-0" & WALLS FROM EL 0'-0" TO EL 32'-0"

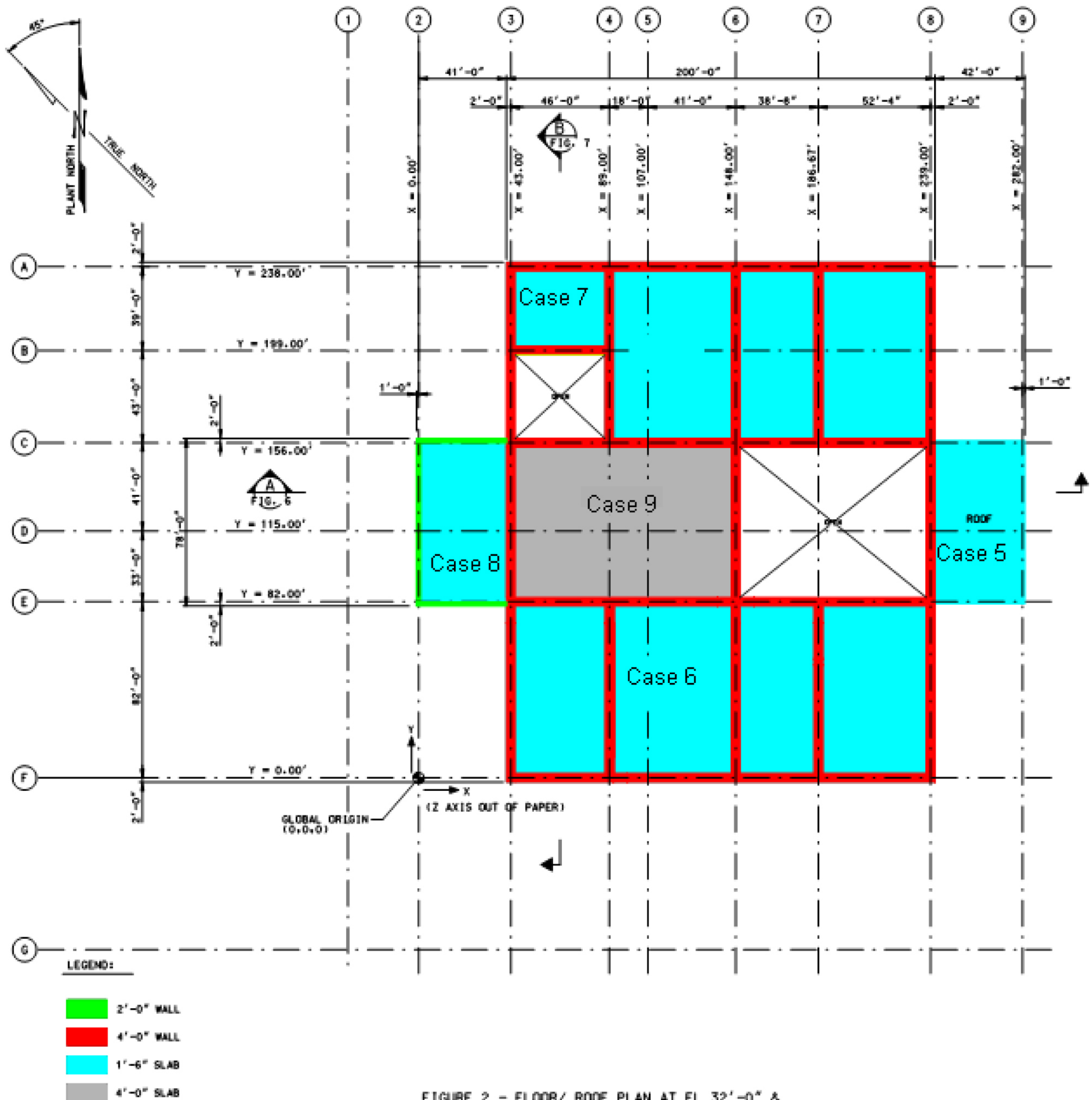


FIGURE 2 - FLOOR/ ROOF PLAN AT EL 32'-0" & WALLS FROM EL 32'-0" TO EL 64'-0"

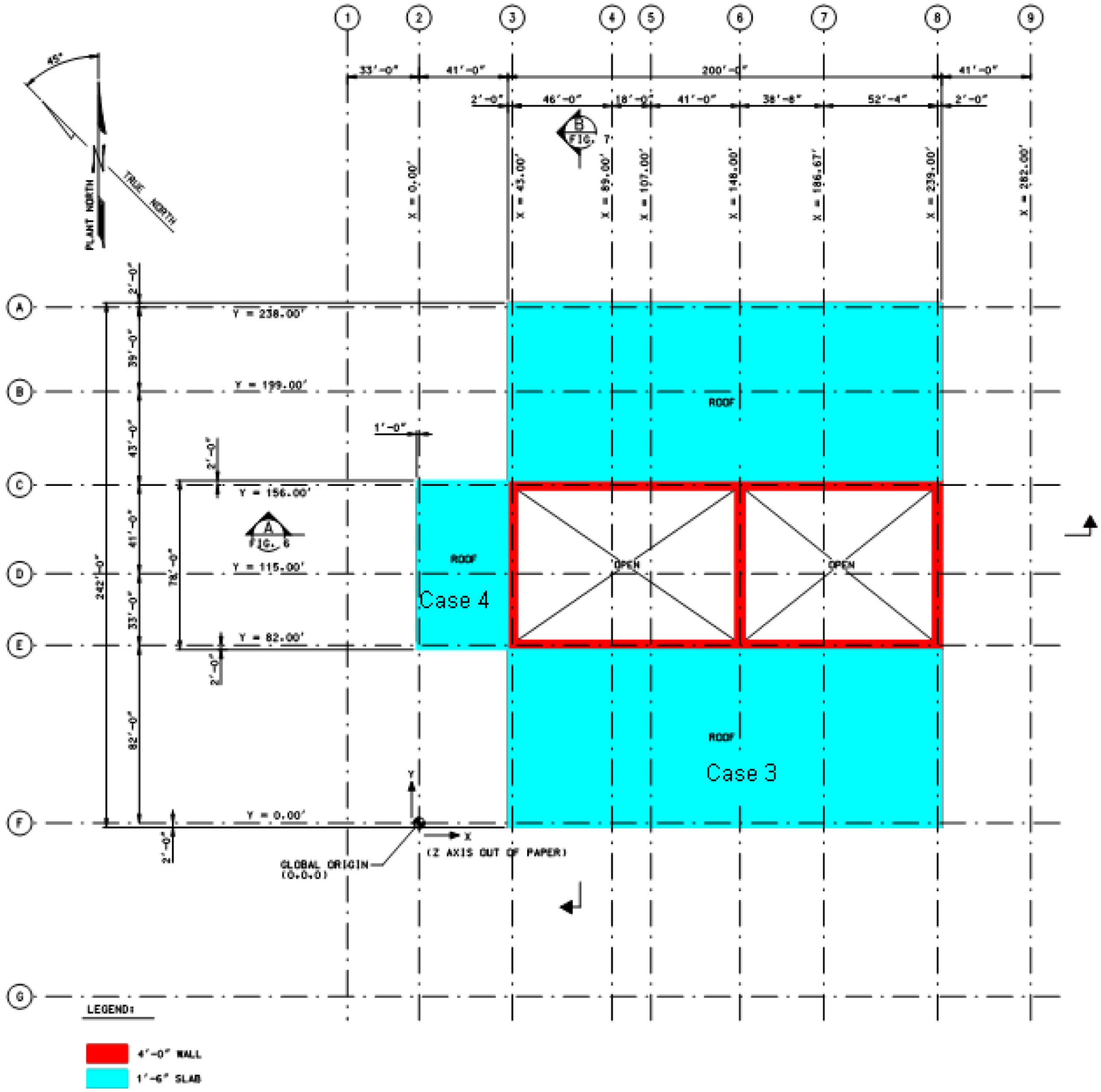


FIGURE 3 - ROOF PLAN AT EL 64'-0" &
WALLS FROM EL 64-0" TO EL 72'-0" & 100'-0"

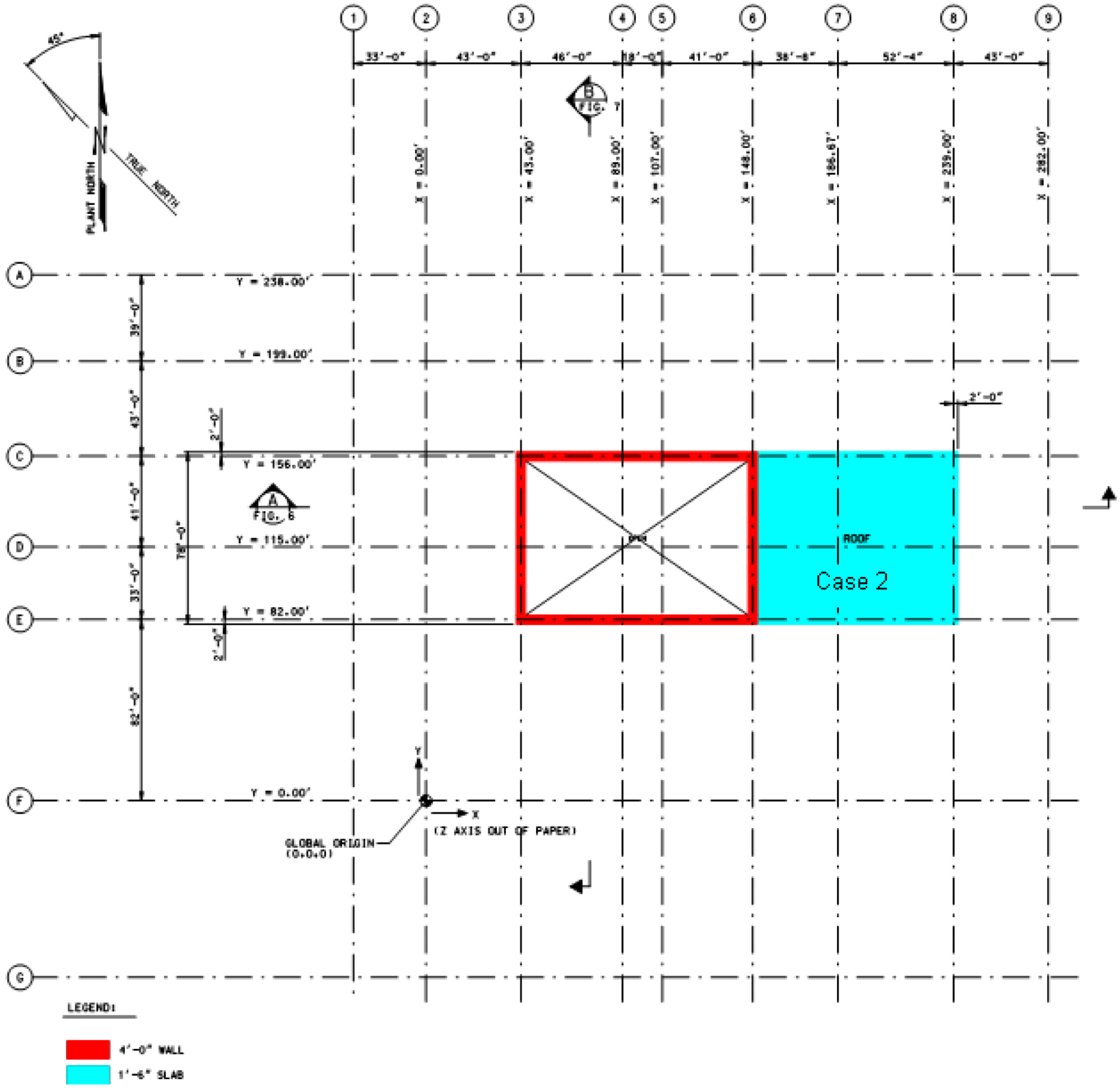


FIGURE 4 - ROOF PLAN AT EL 72'-0" & WALLS FROM EL 72-0" TO EL 100'-0"

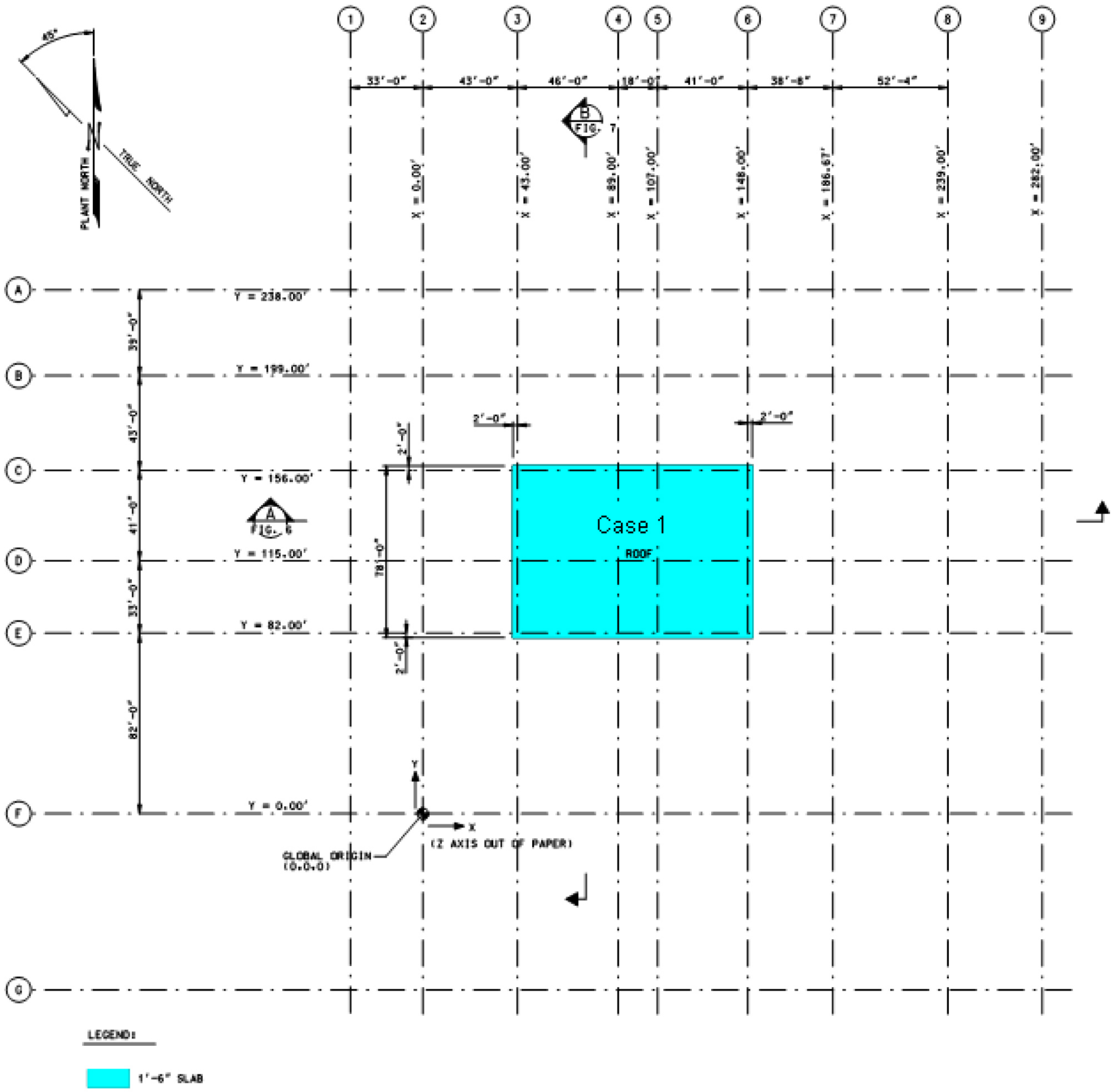


FIGURE 5 – ROOF PLAN AT EL 100'-0"