BSC

Design Calculation or Analysis Cover Sheet

Complete only applicable items.

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CRCF Diaphragm Design

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Attachments	Total Number of Pages
ATTACHMENT A : CRCF PLANS	3
ATTACHMENT B : PLAN – TYPICAL CHORD STEEL FOR DIAPHRAGM	1

RECORD OF REVISIONS

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DISCLAIMER

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ATTACHMENTS

1.0 PURPOSE

The purpose of this calculation is for the preliminary design of the concrete slabs/diaphragms for the Canister Receipt and Closure Facility (CRCF). See plant design drawings (references 2.2.7 to 2.2.11) for plans and sections of the CRCF structure.

In this calculation a representative sample of slabs will be designed. This sample includes the following cases. See attachment A.

Case 1: Roof Diaphragm @ El. 100': Col Lines 6-9/D-H (N/S dir) & 6-9/E-G (E/W dir): 18" slab

Case 2: Roof Diaphragm @ El. 72': Col Lines 9-12/E-G (N/S & E/W dir): 18" slab

Case 3: Roof Diaphragm @ El. 64': Col Lines 9-12/D-E (N/S & E/W dir): 18" slab
Also applies to Col Lines 9-12/G-H (N/S & E/W dir).

Case 4: Roof Diaphragm @ El. 64': Col Lines 2-3/D-E (N/S dir) & 2-6/D-E (E/W dir): 18" slab
Also applies to Col Lines 2-3/G-H (N/S dir) & 2-6/G-H (E/W dir).

Case 5: Roof Diaphragm @ El. 64': Col Lines 2-5/E-G (N/S & E/W dir): 33" slab

Case 6: Floor Diaphragm @ El. 32': Col Lines 6-9/E-G (N/S & E/W dir): 48" slab

Case 7: Roof Diaphragm @ El. 32': Col Lines 12-13/E-G (N/S & E/W dir): 18" slab
Also applies to Col Lines 1-2/E-G (N/S & E/W dir).

Case 8: Floor Diaphragm @ El. 32': Col Lines 2-3/D-E (N/S dir) & 2-12/D-E (E/W dir):18" slab
Also applies to Col Lines 2-3/G-H (N/S dir) & 2-12/G-H (E/W dir).

Results from these cases bound the diaphragm panels not covered in this calculation.

The results of this calculation yield preliminary design of all slabs/diaphragms for the CRCF. During the detailed design phase of the project a complete design of all slabs/diaphragms will be made using results from a detailed three dimensional finite element model.

2.0 REFERENCES

2.1 PROJECT PROCEDURES/DIRECTIVES

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2.2 DESIGN INPUTS

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2.2.7 BSC (Bechtel SAIC Company) 2007. *Canister Receipt and Closure Facility # 1 Preliminary Layout Ground Floor Plan.* 060-P0K-CR00-10101-000-00A.
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2.2.10 BSC (Bechtel SAIC Company) 2007. *Canister Receipt and Closure Facility # 1 Preliminary Layout Section A.* 060-P0K- CR00-10104-000-00A.
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2.2.14 BSC (Bechtel SAIC Company) 2004. *Analysis (SASSI) for Sample In-Structure Response Spectra for CHF with Two Closure Cells.*
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2.2 DESIGN CONSTRAINTS

None

2.3 DESIGN OUTPUTS

Results of this calculation will be used in the development of the CRCF concrete outline drawings. Drawing numbers have not yet been assigned to these drawings.

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 col lines 6-9/E-G): 40 lbs/ft^2
Floor at 32’ bounded by col lines 6-9/E-G: 0 lbs/ft^2

Rationale: Structural steel represents a small fraction of the total loads applied to the CRCF slabs. 40 lbs/ft^2 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 col lines 6-9/E-G has no steel framing.

3.1.2 Equipment dead loads are assumed as 100 lbs/ft^2 and 50 lbs/ft^2 on the floor slabs and the roof slabs, respectively. Equipment dead loads include HVAC equipment, Electrical equipment and so forth.

Rationale: The CRCF is not an equipment intensive structure with the major equipment for diaphragm design being the HVAC equipment. 100 lbs/ft^2 and 50 lbs/ft^2 is 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.

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

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

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 is a reasonable assumption for a structure of this type. The primary source of live load is maintenance of HVAC and other equipment.

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.

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 CRCF seismic analysis (Ref. 2.2.4).

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. 2.2.14) which developed in-structure response spectra at hard points on the walls. CRCF structure is similar to CHF structure. Using the 7% damped vertical response spectra given in fig. F-3 of Ref. 2.2.14, 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. 2.2.13). 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 CRCF seismic analysis (Ref. 2.2.4) 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.

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.

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.

4. METHODOLOGY

4.1 QUALITY ASSURANCE

This calculation was prepared in accordance with EG-PRO-3DP-G04B-00037, *Calculations and Analyses* (Ref. 2.1.1). Section 4.1.2 of the *Basis of Design for the TAD Canister–Based Repository Design Concept* (Ref. 2.2.2) classifies the CRCF structure as ITS. Therefore, the approved version of this calculation is designated as QA: QA.

4.2 USE OF SOFTWARE

Word 2000, which is part of the Microsoft Office 2000 suite of programs, was used in preparation of the calculation. Microsoft Office 2000 as used in the calculation is classified as Level 2 software usage as defined in IT-PRO-0011, *Software Management* (Ref. 2.1.2). Microsoft Office 2000 is also listed on the controlled Software Report (SW Tracking Number 610236-2000-00) as well as on 000-PLN-MGR0-00200-000, *Repository Project Management Automation Plan* (Ref. 2.1.3).

Mathcad 13 was utilized to perform the mathematical computations in this calculation. All Mathcad input values and equations are stated in the calculation. Checking of the Mathcad template was done by using a hand calculator to check the operations being performed by Mathcad.

Mathcad version 13 is listed on the controlled software report (SW Tracking Number 61116), as well as on 000-PLN-MGR0-00200-000, *Repository Project Management Automation Plan* (Ref. 2.1.3).

The software was executed on a PC system running Microsoft Windows 2000 operating system.

4.3 DESIGN APPROACH

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

The CRCF 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)*, hereinafter referred to as ACI 349 (Ref. 2.2.3).

Diaphragms transmit the horizontal seismic loads to the tributary shear walls. In-plane diaphragm loads are a result of horizontal seismic acceleration of the diaphragm itself and the walls tributary to the diaphragm perpendicular to the direction of the seismic acceleration. For example, under a North to South seismic acceleration a diaphragm must transfer horizontal seismic load equal to the mass of the diaphragm plus the mass of the East to West walls (tributary to the diaphragm) times the horizontal seismic acceleration to the North to South shear walls.

The diaphragm design is carried out in three steps.

- Reinforcing requirements for out-of-plane (bending) loads are calculated.
- Reinforcing requirements for in-plane (diaphragm) shear loads are calculated.
- Reinforcing requirements for in-plane (diaphragm) moments are calculated.

The results of the first and second steps are combined to determine the reinforcing requirements for the out-of-plane bending and in-plane shear loads. These reinforcing requirements are then compared to the ACI 349 (Ref. 2.2.3) minimum reinforcing requirements. The larger of the reinforcing required for the out-of-plane bending and in-plane shear loads and minimum requirements will determine the reinforcing requirements.

The results of the third step (in-plane moments) yield the chord steel required for the diaphragm.

5. LIST OF ATTACHMENTS

Note : Attachment A is based on plant design drawings (references 2.2.7 to 2.2.11).

6. 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. 2.2.1, Sect. 4.2.11.6.2)
$f_y := 60000 \cdot psi$	Yield Stress of Grade 60 Reinforcing Steel	(Ref. 2.2.1, 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

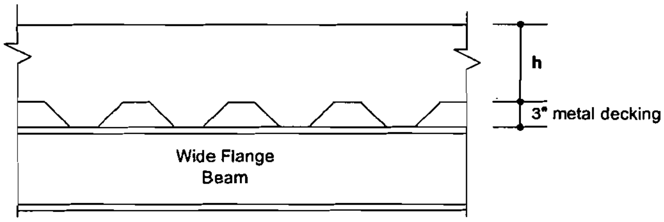
6.2.1 Dead Loads of Concrete Slabs

$w_{conc} := 150 \cdot pcf$ Unit weight of concrete (Reference 2.2.1, Section 4.2.11.6.6)

The slab dead load of concrete slab constructed on a 3" (0.25 ft) metal deck (18" & 33" thick slabs) :

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

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



18" thick roof slab	$SDL_{18r} := (1.5 \cdot ft + 0.125 \cdot ft) \cdot w_{conc}$	$SDL_{18r} = 244\,psf$	El 32', 64', 72' & 100'
18" thick floor slab	$SDL_{18f} := (1.5 \cdot ft + 0.125 \cdot ft) \cdot w_{conc}$	$SDL_{18f} = 244\,psf$	El 32'
33" thick roof slab	$SDL_{33r} := (2.75 \cdot ft + 0.125 \cdot ft) \cdot w_{conc}$	$SDL_{33r} = 431\,psf$	El 64'

Note : 48" thick floor slab does not have metal deck under the slab.

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

$$SDL := \begin{pmatrix} SDL_{18r} \\ SDL_{18f} \\ SDL_{33r} \\ SDL_{48f} \end{pmatrix} \quad \begin{array}{l} \text{Slab Dead Load : 18" roof slab} \\ \text{Slab Dead Load : 18" floor slab} \\ \text{Slab Dead Load : 33" roof slab} \\ \text{Slab Dead Load : 48" floor slab} \end{array} \qquad \boxed{SDL = \begin{pmatrix} 244 \\ 244 \\ 431 \\ 600 \end{pmatrix} psf}$$

6.2.2 Steel Framing Dead Load (Assumption 3.1.1)

$$SFDL := \begin{pmatrix} 40 \\ 40 \\ 40 \\ 0 \end{pmatrix} psf \quad \begin{array}{l} \text{Steel Framing Dead Load : 18" roof slab} \\ \text{Steel Framing Dead Load : 18" floor slab} \\ \text{Steel Framing Dead Load : 33" roof slab} \\ \text{Steel Framing Dead Load : 48" floor slab} \end{array} \qquad \boxed{SFDL = \begin{pmatrix} 40 \\ 40 \\ 40 \\ 0 \end{pmatrix} psf}$$

Note : 48" thick floor slab does not have steel framing under the slab.

6.2.3 Equipment Dead Load (Assumption 3.1.2)

$EDL_{fl} := 100psf$ Floor Equipment Dead Load $EDL_{rf} := 50psf$ Roof Equipment Dead Load

$$EDL := \begin{pmatrix} EDL_{rf} \\ EDL_{fl} \\ EDL_{rf} \\ EDL_{fl} \end{pmatrix} \quad \begin{array}{l} \text{Equipment Dead Load : 18" roof slab} \\ \text{Equipment Dead Load : 18" floor slab} \\ \text{Equipment Dead Load : 33" roof slab} \\ \text{Equipment Dead Load : 48" floor slab} \end{array} \qquad \boxed{EDL = \begin{pmatrix} 50 \\ 100 \\ 50 \\ 100 \end{pmatrix} psf}$$

6.2.4 Roofing Material Dead Load (Assumption 3.1.3)

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

Roofing Material Dead Load : 18" roof slab
Roofing Material Dead Load : 18" floor slab
Roofing Material Dead Load : 33" roof slab
Roofing Material Dead Load : 48" floor slab

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

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

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

Design Live Load : 18" roof slab
Design Live Load : 18" floor slab
Design Live Load : 33" roof slab
Design Live Load : 48" floor slab

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

6.2.6 Acceleration Factors for Seismic Loads (Ref. 2.2.4, table 13)

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'

$a100_x := 1.41$ (East/West)
$a100_y := 1.35$ (North/South)
$a100_z := 2 \cdot 0.82$ $a100_z = 1.64$ (Vertical)

Acceleration factors for elevation 72'

$a72_x := 0.96$ (East/West)
$a72_y := 1.00$ (North/South)
$a72_z := 2 \cdot 0.80$ $a72_z = 1.60$ (Vertical)

Acceleration factors for elevation 64'

$a64_x := 0.91$ (East/West)
$a64_y := 0.87$ (North/South)
$a64_z := 2 \cdot 0.76$ $a64_z = 1.52$ (Vertical)

Acceleration factors for elevation 32'

$a32_x := 0.67$ (East/West)
$a32_y := 0.70$ (North/South)
$a32_z := 2 \cdot 0.71$ $a32_z = 1.42$ (Vertical)

6.2.7 Loading Combinations (Ref. 2.2.1, sect. 4.2.11.4.5)

$U1 := 1.4D + 1.7 \cdot L$ (Ultimate Load for normal operating condition)

$U2 := D + Le + E$ (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. 2.2.6, sect. 8.3.1)

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

See plant design drawings (references 2.2.7 to 2.2.11) and attachment A for plans and sections of the CRCF structure.

Most of the 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" thk slabs (el 32',64',72' or 100') & 33" thk slab (el 64').

48" thick slab bounded by column lines 6, 9, E & G @ el. 32' (Ref. 2.2.8) does not have metal deck under the slab. It has more than two continuous spans with the maximum spans of 32' (between column lines E.3 & F and F & F.7).

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).

$$TDL := SDL + SFDL + EDL + RMDL \qquad \boxed{TDL = \begin{pmatrix} 389 \\ 384 \\ 576 \\ 700 \end{pmatrix} psf}$$

Total Dead Load : 18" roof slab
Total Dead Load : 18" floor slab
Total Dead Load : 33" roof 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. 2.2.6, sect. 8.3.1)

$$SACC_Z := \begin{pmatrix} a100_Z \\ a32_Z \\ a64_Z \\ a32_Z \end{pmatrix} \qquad \boxed{SACC_Z = \begin{pmatrix} 1.64 \\ 1.42 \\ 1.52 \\ 1.42 \end{pmatrix}}$$

Seismic Acceleration factor for 18" slab @ El 100' in Z dir
Seismic Acceleration factor for 18" slab @ El 32' in Z dir
Seismic Acceleration factor for 18" slab @ El 64' in Z dir
Seismic Acceleration factor for 48" slab @ El 32' in Z dir

$$E := \overrightarrow{\left[(TDL + Le)\cdot SACC_Z\right]} \qquad \boxed{E = \begin{pmatrix} 654 \\ 580 \\ 891 \\ 1029 \end{pmatrix} psf}$$

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

6.3.3 Governing Ultimate Load Combination for Concrete Design (See section 6.2.7.)

$$U1 := 1.4\cdot TDL + 1.7\cdot LL \qquad \boxed{U1 = \begin{pmatrix} 612 \\ 707 \\ 875 \\ 1150 \end{pmatrix} psf}$$

Ultimate Load Comb. 1.4D+1.7L : 18" roof slab
Ultimate Load Comb. 1.4D+1.7L : 18" floor slab
Ultimate Load Comb. 1.4D+1.7L : 33" roof slab
Ultimate Load Comb. 1.4D+1.7L : 48" floor slab

$$U2 := TDL + Le + E \qquad \boxed{U2 = \begin{pmatrix} 1053 \\ 989 \\ 1477 \\ 1755 \end{pmatrix} psf}$$

Ultimate Load Comb. D+Le+E : 18" roof slab
Ultimate Load Comb. D+Le+E : 18" floor slab
Ultimate Load Comb. D+Le+E : 33" roof slab
Ultimate Load Comb. D+Le+E : 48" floor slab

$ORIGIN \equiv 1$ Changing origin for matrices from 0,0 to 1,1

$$U := \begin{pmatrix} \max(U1_1, U2_1) \\ \max(U1_2, U2_2) \\ \max(U1_3, U2_3) \\ \max(U1_4, U2_4) \end{pmatrix}$$

$$\boxed{U = \begin{pmatrix} 1053 \\ 989 \\ 1477 \\ 1755 \end{pmatrix} psf}$$

Gov Ult Load Comb. D+Le+E : 18" roof slab
Gov Ult Load Comb. D+Le+E : 18" floor slab
Gov Ult Load Comb. D+Le+E : 33" roof slab
Gov Ult Load Comb. D+Le+E : 48" floor slab

6.3.4 Moments and Shears for Slabs

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

Using the equations for moments and shears from ACI 349 (Ref. 2.2.3 sect. 8.3) :

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

$span_{18or33} := 7 \cdot ft$ Span for 18" or 33" slabs. (Assumption 3.1.5)

$span_{48} := 32 \cdot ft$ $\boxed{span_{48} = 32\,ft}$ Span 48" floor slab, see section 6.3.

$$span := \begin{pmatrix} span_{18or33} \\ span_{18or33} \\ span_{18or33} \\ span_{48} \end{pmatrix}$$

$$\boxed{span = \begin{pmatrix} 7 \\ 7 \\ 7 \\ 32 \end{pmatrix} ft}$$

Span 18" roof slab
Span 18" floor slab
Span 33" roof slab
Span 48" floor slab

$$M_{max} := \overrightarrow{\frac{U \cdot b \cdot span^2}{10}}$$

$$\boxed{M_{max} = \begin{pmatrix} 5158 \\ 4847 \\ 7239 \\ 179661 \end{pmatrix} lbf \cdot ft} \text{ per ft}$$

Moment 18" roof slab
Moment 18" floor slab
Moment 33" roof slab
Moment 48" floor slab

$$V_{max} := \overrightarrow{\frac{1.15 \cdot U \cdot b \cdot span}{2}}$$

$$\boxed{V_{max} = \begin{pmatrix} 4237 \\ 3981 \\ 5946 \\ 32283 \end{pmatrix} lbf} \text{ per ft}$$

Shear 18" roof slab
Shear 18" floor slab
Shear 33" roof slab
Shear 48" floor slab

6.3.5 Check Shear Reinforcement Requirements

For 18" & 33" thick slabs the effective depth of the rebar, d, can be calculated as:

d = h - 3/4" (clr) - $1.5\text{x}\,d_{bar}$ where h is the slab thickness above the metal deck

For a # 11 reinforcing bar d_{bar} = 1.41", this results in an effective rebar depth, d, of 15.13" for 18" slab and 30.13" for 33" slab.

48" thick slab has a depression of 18" in some areas (Ref. 2.2.12). For this condition the effective depth of the rebar, d = 48" - 18" - 0.75" - 1.5x1.41" = 27.13"

$d_{18r} := 15.13\cdot in$ $\quad d_{18f} := 15.13\cdot in$ $\quad d_{33} := 30.13\cdot in$ $\quad d_{48} := 27.13\cdot in$

$$d := \begin{pmatrix} d_{18r} \\ d_{18f} \\ d_{33} \\ d_{48} \end{pmatrix} \qquad d = \begin{pmatrix} 15.13 \\ 15.13 \\ 30.13 \\ 27.13 \end{pmatrix} in$$

Effective depth of the rebar 18" roof slab
Effective depth of the rebar 18" floor slab
Effective depth of the rebar 33" roof slab
Effective depth of the rebar 48" floor slab

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

$\phi_s := .85$ Strength reduction factor for transverse shear ACI 349 (Ref. 2.2.3, sect. 9.3.2.3)

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

From ACI 349 Ref. 2.2.3, sect. 11.3.1.1

$$\phi_{sV.c} := 2\cdot\phi_s\cdot\sqrt{f_c\cdot psi}\cdot b\cdot d \qquad \phi_{sV.c} = \begin{pmatrix} 21825 \\ 21825 \\ 43462 \\ 39135 \end{pmatrix} lbf \text{ per ft}$$

Shear Strength 18" roof slab
Shear Strength 18" floor slab
Shear Strength 33" roof slab
Shear Strength 48" floor slab

See section 6.3.4 for V_{max}

$$V_{max} = \begin{pmatrix} 4237 \\ 3981 \\ 5946 \\ 32283 \end{pmatrix} lbf \text{ per ft}$$

Shear 18" roof slab
Shear 18" floor slab
Shear 33" roof slab
Shear 48" floor slab

$\phi_s\cdot V_c > V_{max}$, 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. 2.2.5, page 102)

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

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. 2.2.3, 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

$$C1 := \overrightarrow{\frac{M_{max}}{\phi_b \cdot f_c \cdot b \cdot d^2}} \qquad \boxed{C1 = \begin{pmatrix} 0.00501 \\ 0.00471 \\ 0.00177 \\ 0.05424 \end{pmatrix}}$$

Constant for 18" roof slab
Constant for 18" floor slab
Constant for 33" roof slab
Constant for 48" floor slab

$C18r := C1_1$	$C18r = 0.00501$	Constant for 18" roof slab
$C18f := C1_2$	$C18f = 0.00471$	Constant for 18" floor slab
$C33 := C1_3$	$C33 = 0.00177$	Constant for 33" roof slab
$C48 := C1_4$	$C48 = 0.05424$	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 $\omega 18r := .00300$ $f1(\omega 18r) := .59 \cdot \omega 18r^2 - \omega 18r + C18r$

$root(f1(\omega 18r), \omega 18r) = 0.00502$ thus $\omega 18r\_act := root(f1(\omega 18r), \omega 18r)$

$\boxed{\omega 18r\_act = 0.00502}$ Actual Value of $\omega 18r$

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

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

$\text{root}(f2(\omega 18f), \omega 18f) = 0.00472$ thus $\omega 18f\_act := \text{root}(f2(\omega 18f), \omega 18f)$

$\boxed{\omega 18f\_act = 0.00472}$ Actual Value of $\omega 18f$

Solving for $\omega 33$: 33" roof slab

Try $\omega 33 := .00300$ $f3(\omega 33) := .59 \cdot \omega 33^2 - \omega 33 + C33$

$\text{root}(f3(\omega 33), \omega 33) = 0.00177$ thus $\omega 33\_act := \text{root}(f3(\omega 33), \omega 33)$

$\boxed{\omega 33\_act = 0.00177}$ Actual Value of $\omega 33$

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

Try $\omega 48 := .00300$ $f4(\omega 48) := .59 \cdot \omega 48^2 - \omega 48 + C48$

$\text{root}(f4(\omega 48), \omega 48) = 0.0561$ thus $\omega 48\_act := \text{root}(f4(\omega 48), \omega 48)$

$\boxed{\omega 48\_act = 0.0561}$ Actual Value of $\omega 48$

$$\omega := \begin{pmatrix} \omega 18r\_act \\ \omega 18f\_act \\ \omega 33\_act \\ \omega 48\_act \end{pmatrix} \qquad \boxed{\omega = \begin{pmatrix} 0.00502 \\ 0.00472 \\ 0.00177 \\ 0.05610 \end{pmatrix}}$$

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

<u>Required Reinforcing Index</u>

$$\rho_{req} := \frac{\omega \cdot f_c}{f_y} \qquad \boxed{\rho_{req} = \begin{pmatrix} 0.00042 \\ 0.00039 \\ 0.00015 \\ 0.00467 \end{pmatrix}}$$

18" roof slab
18" floor slab
33" roof 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 CRCF showing the diaphragms.

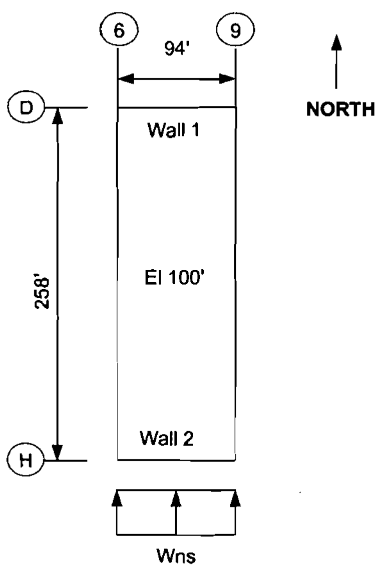
Steel for diaphragms will be computed for following diaphragm panels in this section.

Case 1:Roof Diaphragm @ El.100':Col Lines 6-9/D-H (N/S dir)&6-9/E-G (E/W dir):18" slab

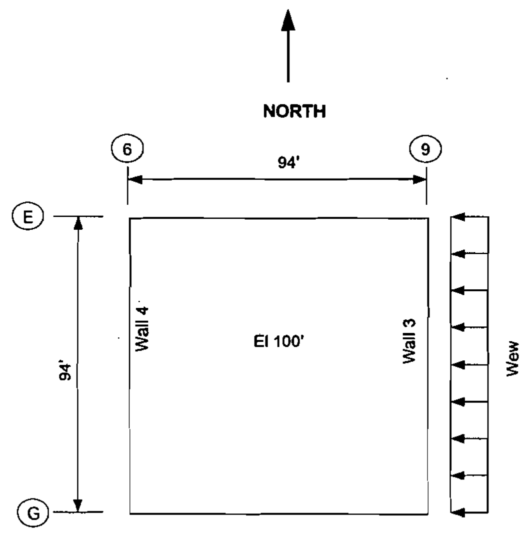
Case 2:Roof Diaphragm @ El. 72': Col Lines 9-12/E-G (N/S & E/W dir): 18" slab

Case 3:Roof Diaphragm @ El. 64': Col Lines 9-12/D-E (N/S & E/W dir): 18" slab
Also applies to Col Lines 9-12/G-H (N/S & E/W dir).

Case 4:Roof Diaphragm @ El. 64':Col Lines 2-3/D-E (N/S dir) & 2-6/D-E (E/W dir):18" slab
Also applies to Col Lines 2-3/G-H (N/S dir) & 2-6/G-H (E/W dir).

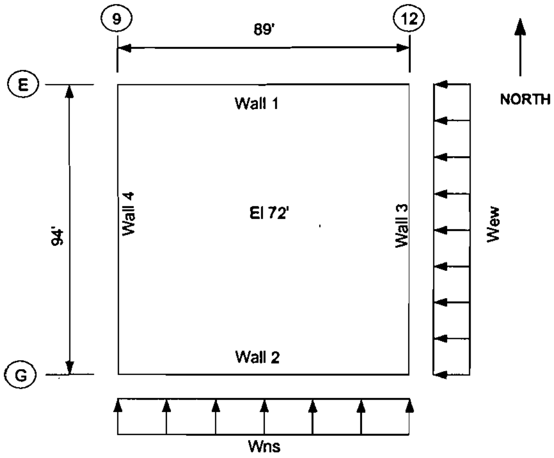


Case 1 : N/S

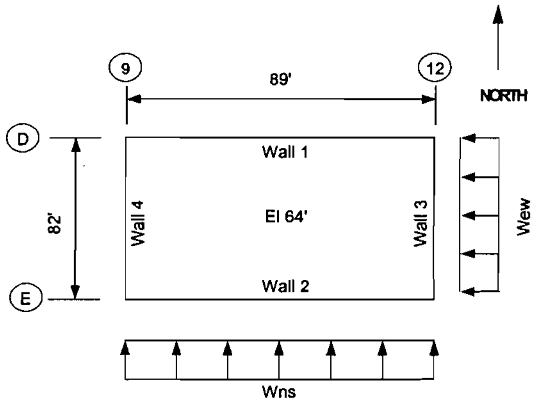


Case 1 : E/W

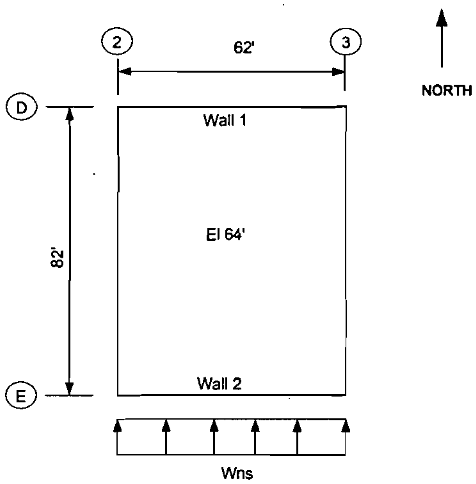
Note : For case 1 in the E/W direction the diaphragm is a 3 span system. (Ref. 2.2.9)
Conservatively take diaphragm as simple span using the largest span. (Assumption 3.2.2)



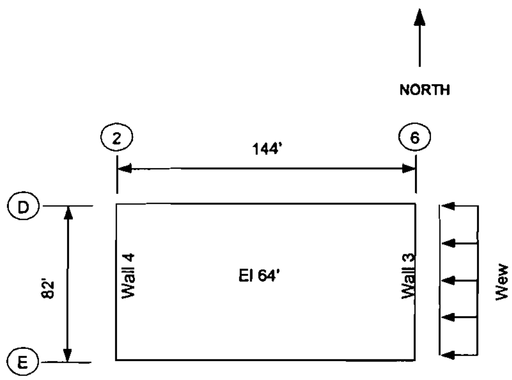
Case 2



Case 3



Case 4 : N/S



Case 4 : E/W

Note : For case 4 in the N/S direction the diaphragm is a 3 span system. (Ref. 2.2.9)
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.

$U_3 := TDL + Le$

$$\boxed{U_3 = \begin{pmatrix} 399 \\ 409 \\ 586 \\ 725 \end{pmatrix} psf}$$

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

See section 6.3.1 for TDL & section 6.2.7 for Le.

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

$$\boxed{U\_1to4 = \begin{pmatrix} 399 \\ 399 \\ 399 \\ 399 \end{pmatrix} psf}$$

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}

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

Tributary Height of Wall 1 (North wall) :

$$Hw1\_1 := \frac{100\cdot ft - 64\cdot ft}{2} + \frac{64\cdot ft - 32ft}{2} \qquad \boxed{Hw1\_1 = 34\,ft}$$ Case 1 : Col line D

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

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

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

Thickness of Wall 1 (North wall) :

$$\boxed{Tw1 := \begin{pmatrix} 4 \\ 4 \\ 4 \\ 4 \end{pmatrix}\cdot ft}$$

Case 1
Case 2
Case 3
Case 4

$$Hw1 := \begin{pmatrix} Hw1\_1 \\ Hw1\_2 \\ Hw1\_3 \\ Hw1\_4 \end{pmatrix} \qquad \boxed{Hw1 = \begin{pmatrix} 34 \\ 4 \\ 16 \\ 16 \end{pmatrix} ft}$$

Case 1 : Col line D
Case 2 : Col line E
Case 3 : Col line D
Case 4 : Col line D

Tributary Height of Wall 2 (South wall) :

$$Hw2\_1 := \frac{100\cdot ft - 64\cdot ft}{2} + \frac{64\cdot ft - 32ft}{2} \qquad \boxed{Hw2\_1 = 34\,ft}$$ Case 1 : Col line H

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

$$Hw2\_3 := \frac{72\cdot ft - 64\cdot ft}{2} + \frac{64\cdot ft - 32ft}{2} \qquad \boxed{Hw2\_3 = 20\,ft}$$ Case 3 : Col line E

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

Thickness of Wall 2 (South wall) :

$$\boxed{Tw2 := \begin{pmatrix} 4 \\ 4 \\ 4 \\ 4 \end{pmatrix}\cdot ft}$$

Case 1
Case 2
Case 3
Case 4

$$Hw2 := \begin{pmatrix} Hw2\_1 \\ Hw2\_2 \\ Hw2\_3 \\ Hw2\_4 \end{pmatrix} \qquad \boxed{Hw2 = \begin{pmatrix} 34 \\ 4 \\ 20 \\ 16 \end{pmatrix} ft}$$

Case 1 : Col line H
Case 2 : Col line G
Case 3 : Col line E
Case 4 : Col line E

$$WWns\_ext\_1to4 := \overrightarrow{\left[(Hw1\cdot Tw1 + Hw2\cdot Tw2)\cdot w_{conc}\right]} \qquad \boxed{WWns\_ext\_1to4 = \begin{pmatrix} 40.8 \\ 4.8 \\ 21.6 \\ 19.2 \end{pmatrix} klf}$$

Case 1
Case 2
Case 3
Case 4

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

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

$$S_{ns\_1to4} := \begin{pmatrix} 94 \\ 89 \\ 89 \\ 62 \end{pmatrix} \cdot ft$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Spans
North/South Seismic Acceleration
See section 6.4.1.

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

Case 2 : Interior wall @ col line 10 & wing walls @ col lines E and G (Ref. 2.2.8 & 2.2.9)

Interior wall @ col line 10

$H2nsa := \frac{72 \cdot ft - 32 \cdot ft}{2}$ $H2nsa = 20\ ft$ Height

$T2nsa := 2 \cdot ft$ Thickness

$L2nsa := 94 \cdot ft$ Length

Wing walls @ col lines E and G

$H2nsb := 100 \cdot ft - 72 \cdot ft$ $H2nsb = 28\ ft$ Height

$T2nsb := 8 \cdot ft$ Combined Thickness of wing walls

$L2nsb := 40 \cdot ft$ Length

$$WWns\_int\_2 := \frac{H2nsa \cdot T2nsa \cdot L2nsa + H2nsb \cdot T2nsb \cdot L2nsb}{S_{ns\_1to4_2}} \cdot w_{conc}$$ $\boxed{WWns\_int\_2 = 21.4\ klf}$

Case 3 : No interior wall $\boxed{WWns\_int\_3 := 0.0 \cdot klf}$

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

$$WWns\_int\_1to4 := \begin{pmatrix} WWns\_int\_1 \\ WWns\_int\_2 \\ WWns\_int\_3 \\ WWns\_int\_4 \end{pmatrix}$$

$$WWns\_int\_1to4 = \begin{pmatrix} 0.0 \\ 21.4 \\ 0.0 \\ 0.0 \end{pmatrix} klf$$

Case 1
Case 2
Case 3
Case 4

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

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

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

$$WWns\_1to4 = \begin{pmatrix} 40.8 \\ 26.2 \\ 21.6 \\ 19.2 \end{pmatrix} klf$$

Case 1
Case 2
Case 3
Case 4

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

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 9

$$Hw3\_2 := \frac{72\cdot ft - 32\cdot ft}{2} \qquad \boxed{Hw3\_2 = 20\,ft}$$ Case 2 : Col line 12

$$Hw3\_3 := \frac{64\cdot ft - 32\cdot ft}{2} \qquad \boxed{Hw3\_3 = 16\,ft}$$ Case 3 : Col line 12

$$Hw3\_4 := \frac{100\cdot ft - 64\cdot ft}{2} + \frac{64\cdot ft - 32\cdot ft}{2} \qquad \boxed{Hw3\_4 = 34\,ft}$$ Case 4 : Col line 6

$$Hw3 := \begin{pmatrix} Hw3\_1 \\ Hw3\_2 \\ Hw3\_3 \\ Hw3\_4 \end{pmatrix} \qquad \boxed{Hw3 = \begin{pmatrix} 14 \\ 20 \\ 16 \\ 34 \end{pmatrix} ft}$$

Case 1 : Col line 9
Case 2 : Col line 12
Case 3 : Col line 12
Case 4 : Col line 6

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

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 6

$$Hw4\_2 := \frac{100\cdot ft - 72ft}{2} + \frac{72\cdot ft - 32ft}{2} \qquad \boxed{Hw4\_2 = 34\,ft}$$ Case 2 : Col line 9

$$Hw4\_3 := \frac{100\cdot ft - 68ft}{2} + \frac{68\cdot ft - 32ft}{2} \qquad \boxed{Hw4\_3 = 34\,ft}$$ Case 3 : Col line 9

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

$$Hw4 := \begin{pmatrix} Hw4\_1 \\ Hw4\_2 \\ Hw4\_3 \\ Hw4\_4 \end{pmatrix} \qquad \boxed{Hw4 = \begin{pmatrix} 18 \\ 34 \\ 34 \\ 16 \end{pmatrix} ft}$$

Case 1 : Col line 6
Case 2 : Col line 9
Case 3 : Col line 9
Case 4 : Col line 2

Thickness of Wall 4 (West wall) :

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

Case 1
Case 2
Case 3
Case 4

$$WWew\_ext\_1to4 := \overrightarrow{\left[(Hw3\cdot Tw3 + Hw4\cdot Tw4)\cdot w_{conc}\right]} \qquad \boxed{WWew\_ext\_1to4 = \begin{pmatrix} 19.2 \\ 32.4 \\ 30.0 \\ 30.0 \end{pmatrix} klf}$$

Case 1
Case 2
Case 3
Case 4

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

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

$$\boxed{S_{ew\_1to4} := \begin{pmatrix} 94 \\ 94 \\ 82 \\ 82 \end{pmatrix} \cdot ft}$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Spans
East/West Seismic Acceleration
See section 6.4.1.

Case 1 : No interior wall

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

Case 2 : Interior wall @ col line 10

(Ref. 2.2.8 & 2.2.9)

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

Height

$$T2ew := 2 \cdot ft$$

Thickness

$$L2ew := 94 \cdot ft$$

Length

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

$$\boxed{WWew\_int\_2 = 6.0\,klf}$$

Case 3 : No interior wall

$$\boxed{WWew\_int\_3 := 0.0 \cdot klf}$$

Case 4 : Interior walls @ col lines 3 & 5

(Ref. 2.2.7 & 2.2.8)

$$H4 := \frac{64 \cdot ft - 32 \cdot ft}{2} \qquad H4 = 16\ ft$$

Height

$$T4 := 8 \cdot ft$$

Combined Thickness
of Walls Col 3 and 5

$$L4 := 82 \cdot ft$$

Length

$$WWew\_int\_4 := \frac{H4 \cdot T4 \cdot L4 \cdot w_{conc}}{S_{ew\_1to4_4}}$$

$$\boxed{WWew\_int\_4 = 19.2\,klf}$$

$$WWew\_int\_1to4 := \begin{pmatrix} WWew\_int\_1 \\ WWew\_int\_2 \\ WWew\_int\_3 \\ WWew\_int\_4 \end{pmatrix}$$

$$\boxed{WWew\_int\_1to4 = \begin{pmatrix} 0.0 \\ 6.0 \\ 0.0 \\ 19.2 \end{pmatrix} klf}$$

Case 1
Case 2
Case 3
Case 4

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

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

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

$$\boxed{WWew\_1to4 = \begin{pmatrix} 19.2 \\ 38.4 \\ 30.0 \\ 49.2 \end{pmatrix} klf}$$

Case 1
Case 2
Case 3
Case 4

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

6.4.5 Design for North/South Seismic Acceleration

6.4.5.1 Moments and Shears

$$\boxed{S_{ns\_1to4} = \begin{pmatrix} 94 \\ 89 \\ 89 \\ 62 \end{pmatrix} ft}$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Spans
North/South Seismic Acceleration
See section 6.4.3.2.

$$\boxed{DD_{ns\_1to4} := \begin{pmatrix} 258 \\ 94 \\ 82 \\ 82 \end{pmatrix} \cdot ft}$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Depths
North/South Seismic Acceleration
See section 6.4.1.

$$SACC_{y\_1to4} := \begin{pmatrix} a100_y \\ a72_y \\ a64_y \\ a64_y \end{pmatrix} \begin{matrix} \text{El. 100'} \\ \text{El. 72'} \\ \text{El. 64'} \\ \text{El. 64'} \end{matrix} \qquad \boxed{SACC_{y\_1to4} = \begin{pmatrix} 1.35 \\ 1.00 \\ 0.87 \\ 0.87 \end{pmatrix}}$$

Case 1
Case 2
Case 3
Case 4

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

$$W_{ns\_1to4} := \overrightarrow{\left[\left(\underbrace{U_{\_1to4} \cdot DD_{ns\_1to4}}_{\text{Wt of slab}} + \underbrace{WWns\_1to4}_{\text{Wt of walls}}\right) \cdot SACC_{y\_1to4}\right]}$$

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

$$\boxed{W_{ns\_1to4} = \begin{pmatrix} 194.0 \\ 63.7 \\ 47.2 \\ 45.2 \end{pmatrix} klf}$$

Case 1
Case 2
Case 3
Case 4

Loads

$$M_{ns\_1to4} := \overrightarrow{\frac{W_{ns\_1to4} \cdot S_{ns\_1to4}^{\,2}}{8}} \qquad \boxed{M_{ns\_1to4} = \begin{pmatrix} 214234 \\ 63091 \\ 46772 \\ 21695 \end{pmatrix} ft \cdot kip}$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Moment

$$V_{ns\_1to4} := \overrightarrow{\frac{W_{ns\_1to4} \cdot S_{ns\_1to4}}{2}} \qquad \boxed{V_{ns\_1to4} = \begin{pmatrix} 9116 \\ 2836 \\ 2102 \\ 1400 \end{pmatrix} kip}$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Shear

6.4.5.2 Check Shear Strength as per ACI 349 (Ref. 2.2.3, 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.

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

$$A_{cv\_ns\_1to4} = \begin{pmatrix} 387 \\ 141 \\ 123 \\ 123 \end{pmatrix} ft^2$$

Case 1
Case 2
Case 3
Case 4

Shear
Areas

$\phi_{diaph} := 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.3, sect. 9.3.4)]

<u>Check Limiting Diaphragm Shear Strength</u>

$\phi V_{n\_max\_ns\_1to4} := \phi_{diaph} \cdot 8 \cdot A_{cv\_ns\_1to4} \cdot \sqrt{f_c \cdot psi}$ ACI 349 (Ref. 2.2.3, sect. 21.6.5.6)

$$\phi V_{n\_max\_ns\_1to4} = \begin{pmatrix} 18915 \\ 6891 \\ 6012 \\ 6012 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Shear
Strength

$$V_{ns\_1to4} = \begin{pmatrix} 9116 \\ 2836 \\ 2102 \\ 1400 \end{pmatrix} kip$$

Shear
See section 6.4.5.1.

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

6.4.5.3 Shear Reinforcement as per ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$\dfrac{V_{ns\_1to4}}{\phi_{diaph}} := V_{c\_ns\_1to4} + V_{s\_ns\_1to4}$ 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. 2.2.3, sect. 21.6.5.2)

$$\phi_{diaph} \cdot V_{c\_ns\_1to4} = \begin{pmatrix} 4729 \\ 1723 \\ 1503 \\ 1503 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Shear
Strength

$$V_{ns\_1to4} = \begin{pmatrix} 9116 \\ 2836 \\ 2102 \\ 1400 \end{pmatrix} kip$$

Shear
See section 6.4.5.1.

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.

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

$$V_{s\_ns\_1to4} = \begin{pmatrix} 7313 \\ 1855 \\ 999 \\ -172 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

$$V_{s\_ns\_1to4rev} := \left| \begin{array}{l} V_{s\_ns\_1to4} \ \text{if } V_{s\_ns\_1to4_4} \geq 0kip \\ \begin{pmatrix} V_{s\_ns\_1to4_1} \\ V_{s\_ns\_1to4_2} \\ V_{s\_ns\_1to4_3} \\ 0.0 \cdot kip \end{pmatrix} \end{array} \right.$$

$$V_{s\_ns\_1to4rev} = \begin{pmatrix} 7313 \\ 1855 \\ 999 \\ 0 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

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

6.4.5.4 Required Reinforcement

$$\rho_{req\_ns\_1to4} := \frac{V_{s\_ns\_1to4rev}}{A_{cv\_ns\_1to4} \cdot f_y}$$

ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$$\rho_{req\_ns\_1to4} = \begin{pmatrix} 0.00219 \\ 0.00152 \\ 0.00094 \\ 0.00000 \end{pmatrix}$$

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

$$S_{ew\_1to4} = \begin{pmatrix} 94 \\ 94 \\ 82 \\ 82 \end{pmatrix} ft \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Diaphragm Spans
East/West Seismic Acceleration
See section 6.4.4.2.

$$DD_{ew\_1to4} := \begin{pmatrix} 94 \\ 89 \\ 89 \\ 144 \end{pmatrix} \cdot ft \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Diaphragm Depths
East/West Seismic Acceleration
See section 6.4.1.

$$SACC_{x\_1to4} := \begin{pmatrix} a100_x \\ a72_x \\ a64_x \\ a64_x \end{pmatrix} \begin{matrix} \text{El. 100'} \\ \text{El. 72'} \\ \text{El. 64'} \\ \text{El. 64'} \end{matrix} \qquad SACC_{x\_1to4} = \begin{pmatrix} 1.41 \\ 0.96 \\ 0.91 \\ 0.91 \end{pmatrix} \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

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

$$W_{ew\_1to4} := \overrightarrow{\left[\left(\underbrace{U_{\_1to4} \cdot DD_{ew\_1to4}}_{\text{Wt of slab}} + \underbrace{WWew\_1to4}_{\text{Wt of walls}}\right) \cdot SACC_{x\_1to4}\right]}$$

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

$$W_{ew\_1to4} = \begin{pmatrix} 79.9 \\ 70.9 \\ 59.6 \\ 97.0 \end{pmatrix} klf \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Loads

$$M_{ew\_1to4} := \overrightarrow{\frac{W_{ew\_1to4} \cdot S_{ew\_1to4}^{\ 2}}{8}} \qquad M_{ew\_1to4} = \begin{pmatrix} 88274 \\ 78346 \\ 50089 \\ 81549 \end{pmatrix} ft \cdot kip \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Diaphragm Moment

$$V_{ew\_1to4} := \overrightarrow{\frac{W_{ew\_1to4} \cdot S_{ew\_1to4}}{2}} \qquad V_{ew\_1to4} = \begin{pmatrix} 3756 \\ 3334 \\ 2443 \\ 3978 \end{pmatrix} kip \qquad \begin{matrix} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{matrix}$$

Diaphragm Shear

6.4.6.2 Check Shear Strength as per ACI 349 (Ref. 2.2.3, 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.

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

$$A_{cv\_ew\_1to4} = \begin{pmatrix} 141 \\ 134 \\ 134 \\ 216 \end{pmatrix} ft^2$$

Case 1
Case 2
Case 3
Case 4

Shear
Areas

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.3, sect. 9.3.4)]

<u>Check Limiting Diaphragm Shear Strength</u>

$$\phi V_{n\_max\_ew\_1to4} := \phi_{diaph} \cdot 8 \cdot A_{cv\_ew\_1to4} \cdot \sqrt{f_c \cdot psi}$$ ACI 349 (Ref. 2.2.3, sect. 21.6.5.6)

$$\phi V_{n\_max\_ew\_1to4} = \begin{pmatrix} 6891 \\ 6525 \\ 6525 \\ 10557 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Shear
Strength

$$V_{ew\_1to4} = \begin{pmatrix} 3756 \\ 3334 \\ 2443 \\ 3978 \end{pmatrix} kip$$

Shear
See section 6.4.6.1.

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

6.4.6.3 Shear Reinforcement as per ACI 349 (Ref. 2.2.3, 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. 2.2.3, sect. 21.6.5.2)

$$\phi_{diaph} \cdot V_{c\_ew\_1to4} = \begin{pmatrix} 1723 \\ 1631 \\ 1631 \\ 2639 \end{pmatrix} kip$$

Case 1
Case 2
Case 3
Case 4

Shear
Strength

$$V_{ew\_1to4} = \begin{pmatrix} 3756 \\ 3334 \\ 2443 \\ 3978 \end{pmatrix} kip$$

Shears
See section 6.4.6.1.

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.

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

$$\boxed{V_{s\_ew\_1to4} = \begin{pmatrix} 3389 \\ 2838 \\ 1354 \\ 2231 \end{pmatrix} \text{kip}}$$

Case 1
Case 2
Case 3
Case 4

Required Reinforcing Shear Strength

6.4.6.4 Required Reinforcement

$$\rho_{req\_ew\_1to4} := \frac{V_{s\_ew\_1to4}}{A_{cv\_ew\_1to4} \cdot f_y}$$

ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$$\boxed{\rho_{req\_ew\_1to4} = \begin{pmatrix} 0.00278 \\ 0.00246 \\ 0.00117 \\ 0.00120 \end{pmatrix}}$$

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

See attachment B for typical chord steel for diaphragm.

6.4.7.1 North/South Seismic Acceleration

$$M_{ns\_1to4} = \begin{pmatrix} 214234 \\ 63091 \\ 46772 \\ 21695 \end{pmatrix} ft \cdot kip$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Moment
See section 6.4.5.1.

$$DD_{ns\_1to4} = \begin{pmatrix} 258 \\ 94 \\ 82 \\ 82 \end{pmatrix} ft$$

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 witdh equal to 10% of the diaphragm depth. See attachment B.

$$CF_{ns\_1to4} = \begin{pmatrix} 923 \\ 746 \\ 634 \\ 294 \end{pmatrix} kip$$

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. 2.2.3, sect. 9.3.2.1)

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

$$A_{ch\_ns\_1to4} = \begin{pmatrix} 17.1 \\ 13.8 \\ 11.7 \\ 5.4 \end{pmatrix} in^2$$

Case 1
Case 2
Case 3
Case 4

Required Chord Steel

Provided Chord Steel (A_{ch}):

Use 18, 14,12 & 6 - # 9 bars for cases 1, 2, 3 & 4 respectively (N/S Seismic Acceleration).

Chord steel provided = 18, 14, 12 & 6 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

$$M_{ew\_1to4} = \begin{pmatrix} 88274 \\ 78346 \\ 50089 \\ 81549 \end{pmatrix} ft \cdot kip$$

Case 1
Case 2
Case 3
Case 4

Diaphragm Moment
See section 6.4.6.1.

$$DD_{ew\_1to4} = \begin{pmatrix} 94 \\ 89 \\ 89 \\ 144 \end{pmatrix} ft$$

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 witdh equal to 10% of the diaphragm depth. See attachment B.

$$CF_{ew\_1to4} = \begin{pmatrix} 1043 \\ 978 \\ 625 \\ 629 \end{pmatrix} kip$$

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. 2.2.3, sect. 9.3.2.1)

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

$$A_{ch\_ew\_1to4} = \begin{pmatrix} 19.3 \\ 18.1 \\ 11.6 \\ 11.7 \end{pmatrix} in^2$$

Case 1
Case 2
Case 3
Case 4

Required Chord Steel

Provided Chord Steel (A_{ch}):

Use 20, 19, 12 & 12 - # 9 bars for cases 1, 2, 3 & 4 respectively (E/W Seismic Acceleration).

Chord steel provided = 20,19,12 & 12 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 CRCF showing the diaphragms.

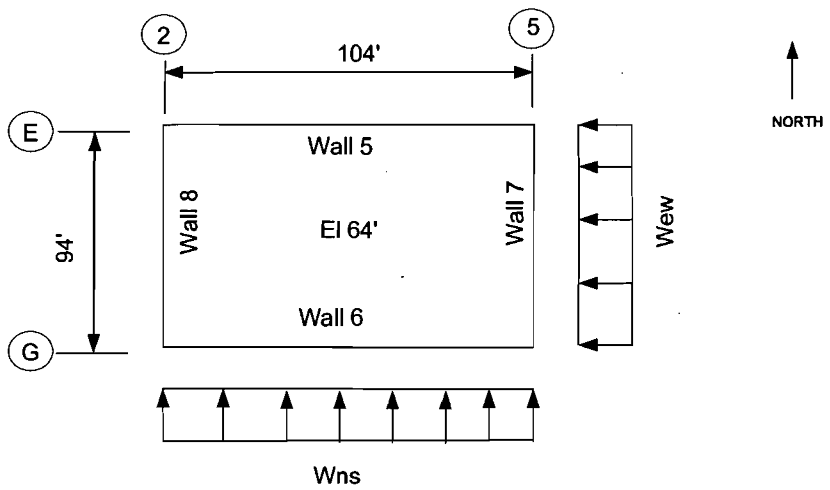
Steel for diaphragms will be computed for following diaphragm panels in this section.

Case 5: Roof Diaphragm @ El. 64':Col Lines 2-5/E-G (N/S & E/W dir): 33" slab

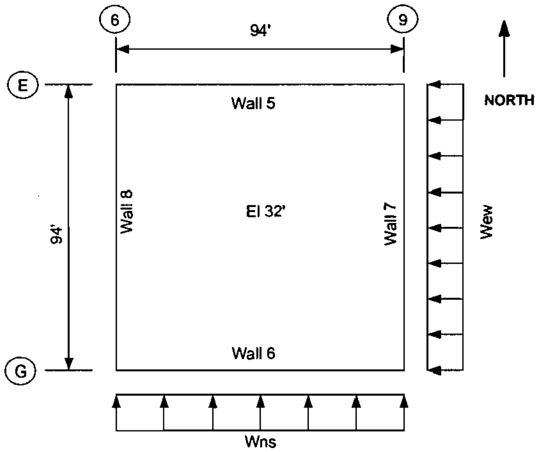
Case 6: Floor Diaphragm @ El. 32':Col Lines 6-9/E-G (N/S & E/W dir): 48" slab

Case 7: Roof Diaphragm @ El. 32': Col Lines 12-13/E-G (N/S & E/W dir): 18" slab
Also applies to Col Lines 1-2/E-G (N/S & E/W dir).

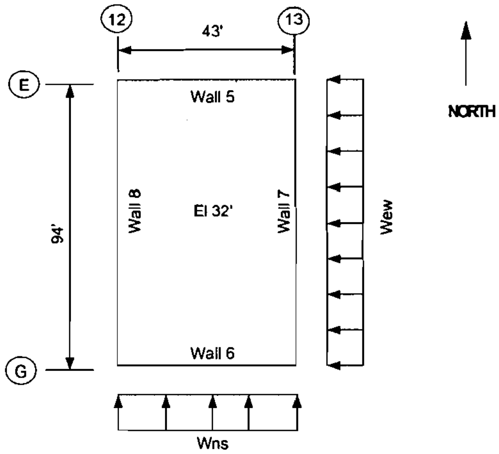
Case 8: Floor Diaphragm @ El.32':Col Lines 2-3/D-E (N/S dir)&2-12/D-E(E/W dir):18" slab
Also applies to Col Lines 2-3/G-H (N/S dir) & 2-12/G-H (E/W dir).



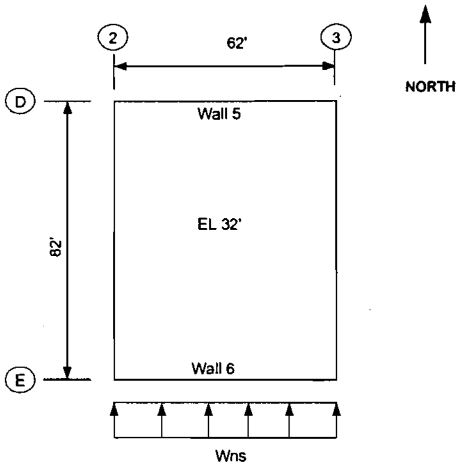
Case 5



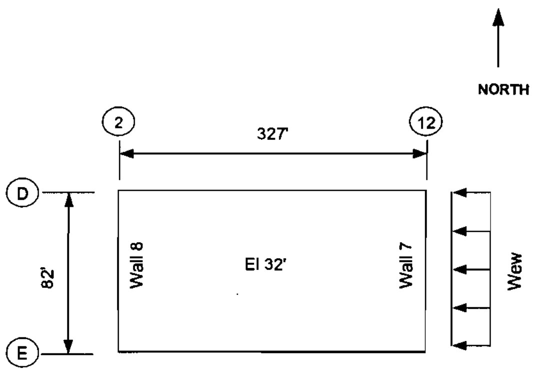
Case 6



Case 7



Case 8 : N/S



Case 8 : E/W

Note : For case 8 in the N/S direction the diaphragm is a 7 span system. (Ref. 2.2.7)
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.

$$\boxed{U_3 = \begin{pmatrix} 399 \\ 409 \\ 586 \\ 725 \end{pmatrix} psf}$$

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

$$U\_5to8 := \begin{pmatrix} U_{3_3} \\ U_{3_4} \\ U_{3_1} \\ U_{3_2} \end{pmatrix} \qquad \boxed{U\_5to8 = \begin{pmatrix} 586 \\ 725 \\ 399 \\ 409 \end{pmatrix} psf}$$

Case 5	33" roof slab
Case 6	48" floor slab
Case 7	18" roof slab
Case 8	18" floor slab

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) :

$$Hw5\_5 := \frac{64\cdot ft - 32ft}{2} \qquad \boxed{Hw5\_5 = 16\,ft}$$ Case 5 : Col line E

$$Hw5\_6 := \frac{32\cdot ft - 0ft}{2} \qquad \boxed{Hw5\_6 = 16\,ft}$$ Case 6 : Col line E

$$Hw5\_7 := \frac{32\cdot ft - 0ft}{2} \qquad \boxed{Hw5\_7 = 16\,ft}$$ Case 7 : Col line E

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

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

Case 5 : Col line E
Case 6 : Col line E
Case 7 : Col line E
Case 8 : Col line D

Thickness of Wall 5 (North wall) :

$$\boxed{Tw5 := \begin{pmatrix} 4 \\ 4 \\ 2 \\ 4 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8

Tributary Height of Wall 6 (South wall) :

$$Hw6\_5 := \frac{64\cdot ft - 32\cdot ft}{2} \qquad \boxed{Hw6\_5 = 16\,ft}$$ Case 5 : Col line G

$$Hw6\_6 := \frac{32\cdot ft - 0ft}{2} \qquad \boxed{Hw6\_6 = 16\,ft}$$ Case 6 : Col line G

$$Hw6\_7 := \frac{32\cdot ft - 0ft}{2} \qquad \boxed{Hw6\_7 = 16\,ft}$$ Case 7 : Col line G

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

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

Case 5 : Col line G
Case 6 : Col line G
Case 7 : Col line G
Case 8 : Col line E

Thickness of Wall 6 (South wall) :

$$\boxed{Tw6 := \begin{pmatrix} 4 \\ 4 \\ 2 \\ 4 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8

$$WWns\_ext\_5to8 := \overrightarrow{\left[(Hw5\cdot Tw5 + Hw6\cdot Tw6)\cdot w_{conc}\right]} \qquad \boxed{WWns\_ext\_5to8 = \begin{pmatrix} 19.2 \\ 19.2 \\ 9.6 \\ 38.4 \end{pmatrix} klf}$$

Case 5
Case 6
Case 7
Case 8

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

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

$$S_{ns\_5to8} := \begin{pmatrix} 104 \\ 94 \\ 43 \\ 62 \end{pmatrix} \cdot ft$$ Case 5, Case 6, Case 7, Case 8

Diaphragm Spans
North/South Seismic Acceleration
See section 6.5.1.

Case 5 : Interior wall @ col line 4 (Ref. 2.2.7 & 2.2.8)

$H5ns := \frac{64 \cdot ft - 32 \cdot ft}{2}$ $H5ns = 16\ ft$ Height

$T5ns := 4 \cdot ft$ Thickness $L5ns := 94 \cdot ft$ Length

$WWns\_int\_5 := \frac{H5ns \cdot T5ns \cdot L5ns}{S_{ns\_5to8_1}} \cdot w_{conc}$ $\boxed{WWns\_int\_5 = 8.7\,klf}$

Case 6 : Interior walls @ col lines 7, 8, E.3, F and F.7 (Ref. 2.2.7 & 2.2.8)

$H6ns := \frac{32 \cdot ft - 0 \cdot ft}{2}$ $H6ns = 16\ ft$ Height $T6ns := 4 \cdot ft$ Thickness

$L7or8 := 94 \cdot ft$ 2 Walls @ 7 or 8 $Lgth := (44 + 32)ft$ 3 Walls @ E.3, F or F.7 between col lines 6 & 7 and 8 & 9

$L6ns := 2 \cdot L7or8 + 3 \cdot Lgth$ $L6ns = 416\ ft$ Total length

$WWns\_int\_6 := \frac{H6ns \cdot T6ns \cdot L6ns}{S_{ns\_5to8_2}} \cdot w_{conc}$ $\boxed{WWns\_int\_6 = 42.5\,klf}$

Case 7 : No interior wall $\boxed{WWns\_int\_7 := 0.0 \cdot klf}$

Case 8 : No interior wall $\boxed{WWns\_int\_8 := 0.0 \cdot klf}$

$$WWns\_int\_5to8 := \begin{pmatrix} WWns\_int\_5 \\ WWns\_int\_6 \\ WWns\_int\_7 \\ WWns\_int\_8 \end{pmatrix} \qquad \boxed{WWns\_int\_5to8 = \begin{pmatrix} 8.7 \\ 42.5 \\ 0.0 \\ 0.0 \end{pmatrix} klf}$$ Case 5, Case 6, Case 7, Case 8

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

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

$$WWns\_5to8 := WWns\_ext\_5to8 + WWns\_int\_5to8 \qquad \boxed{WWns\_5to8 = \begin{pmatrix} 27.9 \\ 61.7 \\ 9.6 \\ 38.4 \end{pmatrix} klf}$$ Case 5, Case 6, Case 7, Case 8

Total weight of North & South Walls (exterior & interior) (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 Exterior Walls (per Foot) : WW_{ew_ext}

Tributary Height of Wall 7 (East wall) :

$Hw7\_5 := \frac{64 \cdot ft - 32 \cdot ft}{2}$ $\quad Hw7\_5 = 16\,ft$ Case 5 : Col line 5

$Hw7\_6 := \frac{72 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0 \cdot ft}{2}$ $\quad Hw7\_6 = 36\,ft$ Case 6 : Col line 9

$Hw7\_7 := \frac{32 \cdot ft - 0 \cdot ft}{2}$ $\quad Hw7\_7 = 16\,ft$ Case 7 : Col line 13

$Hw7\_8 := \frac{64 \cdot ft - 32 \cdot ft}{2} + \frac{32 \cdot ft - 0 \cdot ft}{2}$ $\quad Hw7\_8 = 32\,ft$ Case 8 : Col line 12

$$Hw7 := \begin{pmatrix} Hw7\_5 \\ Hw7\_6 \\ Hw7\_7 \\ Hw7\_8 \end{pmatrix} \qquad Hw7 = \begin{pmatrix} 16 \\ 36 \\ 16 \\ 32 \end{pmatrix} ft$$

Case 5 : Col line 5
Case 6 : Col line 9
Case 7 : Col line 13
Case 8 : Col line 12

Thickness of Wall 7 (East wall) :

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

Case 5
Case 6
Case 7
Case 8

Tributary Height of Wall 8 (West wall) :

$Hw8\_5 := \frac{64 \cdot ft - 32 \cdot ft}{2}$ $\quad Hw8\_5 = 16\,ft$ Case 5 : Col line 2

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

$Hw8\_7 := \frac{72 \cdot ft - 32ft}{2} + \frac{32 \cdot ft - 0ft}{2}$ $\quad Hw8\_7 = 36\,ft$ Case 7 : Col line 12

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

$$Hw8 := \begin{pmatrix} Hw8\_5 \\ Hw8\_6 \\ Hw8\_7 \\ Hw8\_8 \end{pmatrix} \qquad Hw8 = \begin{pmatrix} 16 \\ 32 \\ 36 \\ 32 \end{pmatrix} ft$$

Case 5 : Col line 2
Case 6 : Col line 6
Case 7 : Col line 12
Case 8 : Col line 2

Thickness of Wall 8 (West wall) :

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

Case 5
Case 6
Case 7
Case 8

$$WWew\_ext\_5to8 := \overrightarrow{\left[(Hw7 \cdot Tw7 + Hw8 \cdot Tw8) \cdot w_{conc}\right]} \qquad WWew\_ext\_5to8 = \begin{pmatrix} 19.2 \\ 40.8 \\ 26.4 \\ 38.4 \end{pmatrix} klf$$

Case 5
Case 6
Case 7
Case 8

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

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

$$\boxed{S_{ew\_5to8} := \begin{pmatrix} 94 \\ 94 \\ 94 \\ 82 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Spans
East/West Seismic Acceleration
See section 6.5.1.

Case 5 : Interior wall @ col line 4 (Ref. 2.2.7 & 2.2.8)

$H5ew := \dfrac{64 \cdot ft - 32 \cdot ft}{2}$ $\quad H5ew = 16\ ft$ Height

$T5ew := 4 \cdot ft$ Thickness $\quad L5ew := 94 \cdot ft$ Length

$WWew\_int\_5 := \dfrac{H5ew \cdot T5ew \cdot L5ew}{S_{ew\_5to8_1}} \cdot w_{conc}$ $\quad \boxed{WWew\_int\_5 = 9.6\,klf}$

Case 6 : Interior walls @ col lines 7, 8, E.3, F and F.7 (Ref. 2.2.7 & 2.2.8)

Note : Spans for North and South walls (94') is same as spans for East and West walls (94').

$WWew\_int\_6 := WWns\_int\_6$ See section 6.5.3.2. $\boxed{WWew\_int\_6 = 42.5\,klf}$

Case 7 : No interior wall $\boxed{WWew\_int\_7 := 0.0 \cdot klf}$

Case 8 : Interior walls @ col lines 3, 5, 6, 8, 9 (4' thk walls) & 11 (2' thk wall) (Ref. 2.2.7 & 2.2.8)

$H8a := \dfrac{64 \cdot ft - 32 \cdot ft}{2} + \dfrac{32 \cdot ft - 0 \cdot ft}{2}$ $\quad H8a = 32\ ft$ Height of walls @ col 3, 5, 6 & 9

$H8b := \dfrac{32 \cdot ft - 0 \cdot ft}{2}$ $\quad H8b = 16\ ft$ Height of walls @ col 8 & 11

$T8a := 4 \cdot 4 \cdot ft$ $\quad T8a = 16\ ft$ Combined thickness of walls @ col 3, 5, 6 & 9

$T8b := 4 \cdot ft + 2 \cdot ft$ $\quad T8b = 6\ ft$ Combined thickness of walls @ col 8 & 11 $\quad L8 := 82 \cdot ft$ Length

$WWew\_int\_8 := \dfrac{(H8a \cdot T8a + H8b \cdot T8b) \cdot L8 \cdot w_{conc}}{S_{ew\_5to8_4}}$ $\quad \boxed{WWew\_int\_8 = 91.2\,klf}$

$$WWew\_int\_5to8 := \begin{pmatrix} WWew\_int\_5 \\ WWew\_int\_6 \\ WWew\_int\_7 \\ WWew\_int\_8 \end{pmatrix} \qquad \boxed{WWew\_int\_5to8 = \begin{pmatrix} 9.6 \\ 42.5 \\ 0.0 \\ 91.2 \end{pmatrix} klf}$$

Case 5
Case 6
Case 7
Case 8

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

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

$$WWew\_5to8 := WWew\_ext\_5to8 + WWew\_int\_5to8 \qquad \boxed{WWew\_5to8 = \begin{pmatrix} 28.8 \\ 83.3 \\ 26.4 \\ 129.6 \end{pmatrix} klf}$$

Case 5
Case 6
Case 7
Case 8

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

6.5.5 Design for North/South Seismic Acceleration

6.5.5.1 Moments and Shears

$$\boxed{S_{ns\_5to8} = \begin{pmatrix} 104 \\ 94 \\ 43 \\ 62 \end{pmatrix} ft}$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Spans
North/South Seismic Acceleration
See section 6.5.3.2.

$$\boxed{DD_{ns\_5to8} := \begin{pmatrix} 94 \\ 94 \\ 94 \\ 82 \end{pmatrix} \cdot ft}$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Depths
North/South Seismic Acceleration
See section 6.5.1.

$$SACC_{y\_5to8} := \begin{pmatrix} a64_y \\ a32_y \\ a32_y \\ a32_y \end{pmatrix} \begin{matrix} \text{El. 64'} \\ \text{El. 32'} \\ \text{El. 32'} \\ \text{El. 32'} \end{matrix} \qquad \boxed{SACC_{y\_5to8} = \begin{pmatrix} 0.87 \\ 0.70 \\ 0.70 \\ 0.70 \end{pmatrix}}$$

Case 5
Case 6
Case 7
Case 8

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

$$W_{ns\_5to8} := \overrightarrow{\left[\left(\underbrace{U_{\_5to8} \cdot DD_{ns\_5to8}}_{\text{Wt of slab}} + \underbrace{WWns\_5to8}_{\text{Wt of walls}}\right) \cdot SACC_{y\_5to8}\right]}$$

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

$$\boxed{W_{ns\_5to8} = \begin{pmatrix} 72.2 \\ 90.9 \\ 33.0 \\ 50.3 \end{pmatrix} klf}$$

Case 5
Case 6
Case 7
Case 8

Loads

$$M_{ns\_5to8} := \overrightarrow{\frac{W_{ns\_5to8} \cdot S_{ns\_5to8}^{\,2}}{8}} \qquad \boxed{M_{ns\_5to8} = \begin{pmatrix} 97610 \\ 100382 \\ 7617 \\ 24189 \end{pmatrix} ft \cdot kip}$$

Case 5
Case 6
Case 7
Case 8

Diaphragm
Moment

$$V_{ns\_5to8} := \overrightarrow{\frac{W_{ns\_5to8} \cdot S_{ns\_5to8}}{2}} \qquad \boxed{V_{ns\_5to8} = \begin{pmatrix} 3754 \\ 4272 \\ 709 \\ 1561 \end{pmatrix} kip}$$

Case 5
Case 6
Case 7
Case 8

Diaphragm
Shear

6.5.5.2 Check Shear Strength as per ACI 349 (Ref. 2.2.3, sect. 21.6.5.6)

$$\boxed{h_{5to8} := \begin{pmatrix} 33 \\ 30 \\ 18 \\ 18 \end{pmatrix} \cdot in}$$

Case 5
Case 6 Slab Thickness, See section 6.5.1.
Note : Used 30" (minimum thickness) for 48" slab (case 6).
Case 7
Case 8

$$A_{cv\_ns\_5to8} := \overrightarrow{\left(DD_{ns\_5to8} \cdot h_{5to8}\right)}$$

$$\boxed{A_{cv\_ns\_5to8} = \begin{pmatrix} 259 \\ 235 \\ 141 \\ 123 \end{pmatrix} ft^2}$$

Case 5
Case 6 Shear Areas
Case 7
Case 8

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.3, sect. 9.3.4)]

Check Limiting Diaphragm Shear Strength

$$\phi V_{n\_max\_ns\_5to8} := \phi_{diaph} \cdot 8 \cdot A_{cv\_ns\_5to8} \cdot \sqrt{f_c \cdot psi}$$ ACI 349 (Ref. 2.2.3, sect. 21.6.5.6)

$$\boxed{\phi V_{n\_max\_ns\_5to8} = \begin{pmatrix} 12634 \\ 11486 \\ 6891 \\ 6012 \end{pmatrix} kip}$$

Case 5
Case 6 Shear Strength
Case 7
Case 8

$$\boxed{V_{ns\_5to8} = \begin{pmatrix} 3754 \\ 4272 \\ 709 \\ 1561 \end{pmatrix} kip}$$

Shear
See section 6.5.5.1.

$\phi V_{n\_max\_ns\_5to8} > V_{ns\_5to8}$ Limiting diaphragm shear strength satisfied.

6.5.5.3 Shear Reinforcement as per ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$$\frac{V_{ns\_5to8}}{\phi_{diaph}} := V_{c\_ns\_5to8} + V_{s\_ns\_5to8}$$

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

$$V_{c\_ns\_5to8} := 2 \cdot A_{cv\_ns\_5to8} \cdot \sqrt{f_c \cdot psi}$$ ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$$\boxed{\phi_{diaph} \cdot V_{c\_ns\_5to8} = \begin{pmatrix} 3159 \\ 2871 \\ 1723 \\ 1503 \end{pmatrix} kip}$$

Case 5
Case 6 Shear Strength
Case 7
Case 8

$$\boxed{V_{ns\_5to8} = \begin{pmatrix} 3754 \\ 4272 \\ 709 \\ 1561 \end{pmatrix} kip}$$

Shear
See section 6.5.5.1.

For cases 5, 6 and 8 the concrete shear strength [$\phi_{diaph} \cdot V_{c\_ns\_5to8}$] is less than the shear demand [V_{ns_5to8}] therefore shear reinforcing is required. For case 7 the concrete shear strength is more than the shear demand, therefore shear reinforcing is not required.

$$V_{s\_ns\_5to8} := \frac{V_{ns\_5to8}}{\phi_{diaph}} - V_{c\_ns\_5to8}$$

$$V_{s\_ns\_5to8} = \begin{pmatrix} 993 \\ 2334 \\ -1690 \\ 96 \end{pmatrix} kip \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{matrix}$$

$$V_{s\_ns\_5to8rev} := \begin{array}{|l} V_{s\_ns\_5to8} \text{ if } V_{s\_ns\_5to8_3} \geq 0kip \\ \begin{pmatrix} V_{s\_ns\_5to8_1} \\ V_{s\_ns\_5to8_2} \\ 0.0 \cdot kip \\ V_{s\_ns\_5to8_4} \end{pmatrix} \end{array}$$

$$V_{s\_ns\_5to8rev} = \begin{pmatrix} 993 \\ 2334 \\ 0 \\ 96 \end{pmatrix} kip \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{matrix}$$

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

6.5.5.4 Required Reinforcement

$$\rho_{req\_ns\_5to8} := \frac{V_{s\_ns\_5to8rev}}{A_{cv\_ns\_5to8} \cdot f_y}$$

ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$$\rho_{req\_ns\_5to8} = \begin{pmatrix} 0.00044 \\ 0.00115 \\ 0 \\ 0.00009 \end{pmatrix} \qquad \begin{matrix} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{matrix}$$

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

$$S_{ew\_5to8} = \begin{pmatrix} 94 \\ 94 \\ 94 \\ 82 \end{pmatrix} ft$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Spans
East/West Seismic Acceleration
See section 6.5.4.2.

$$DD_{ew\_5to8} := \begin{pmatrix} 104 \\ 94 \\ 43 \\ 327 \end{pmatrix} \cdot ft$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Depths
East/West Seismic Acceleration
See section 6.5.1.

$$SACC_{x\_5to8} := \begin{pmatrix} a64_x \\ a32_x \\ a32_x \\ a32_x \end{pmatrix} \begin{matrix} \text{El. 64'} \\ \text{El. 32'} \\ \text{El. 32'} \\ \text{El. 32'} \end{matrix} \qquad SACC_{x\_5to8} = \begin{pmatrix} 0.91 \\ 0.67 \\ 0.67 \\ 0.67 \end{pmatrix}$$

Case 5
Case 6
Case 7
Case 8

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

$$W_{ew\_5to8} := \overrightarrow{\left[\left(\underbrace{U_{\_5to8} \cdot DD_{ew\_5to8}}_{\text{Wt of slab}} + \underbrace{WWew\_5to8}_{\text{Wt of walls}}\right) \cdot SACC_{x\_5to8}\right]}$$

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

$$W_{ew\_5to8} = \begin{pmatrix} 81.7 \\ 101.5 \\ 29.2 \\ 176.4 \end{pmatrix} klf$$

Case 5
Case 6
Case 7
Case 8

Loads

$$M_{ew\_5to8} := \overrightarrow{\frac{W_{ew\_5to8} \cdot S_{ew\_5to8}^{\,2}}{8}} \qquad M_{ew\_5to8} = \begin{pmatrix} 90227 \\ 112064 \\ 32225 \\ 148252 \end{pmatrix} ft \cdot kip$$

Case 5
Case 6
Case 7
Case 8

Diaphragm
Moment

$$V_{ew\_5to8} := \overrightarrow{\frac{W_{ew\_5to8} \cdot S_{ew\_5to8}}{2}} \qquad V_{ew\_5to8} = \begin{pmatrix} 3839 \\ 4769 \\ 1371 \\ 7232 \end{pmatrix} kip$$

Case 5
Case 6
Case 7
Case 8

Diaphragm
Shear

6.5.6.2 Check Shear Strength as per ACI 349 (Ref. 2.2.3, sect. 21.6.5.6)

$$h_{5to8} = \begin{pmatrix} 33 \\ 30 \\ 18 \\ 18 \end{pmatrix} in$$

Case 5
Case 6
Case 7
Case 8

Slab Thickness
See section 6.5.5.2.

$$A_{cv\_ew\_5to8} := \overrightarrow{\left(DD_{ew\_5to8} \cdot h_{5to8}\right)}$$

$$A_{cv\_ew\_5to8} = \begin{pmatrix} 286 \\ 235 \\ 65 \\ 490 \end{pmatrix} ft^2$$

Case 5
Case 6
Case 7
Case 8

Shear
Areas

$\phi_{diaph} = 0.6$ [Strength reduction factor for in-plane shear ACI 349 (Ref. 2.2.3, sect. 9.3.4)]

<u>Check Limiting Diaphragm Shear Strength</u>

$$\phi V_{n\_max\_ew\_5to8} := \phi_{diaph} \cdot 8 \cdot A_{cv\_ew\_5to8} \cdot \sqrt{f_c \cdot psi}$$

ACI 349 (Ref. 2.2.3, sect. 21.6.5.6)

$$\phi V_{n\_max\_ew\_5to8} = \begin{pmatrix} 13978 \\ 11486 \\ 3152 \\ 23973 \end{pmatrix} kip$$

Case 5
Case 6
Case 7
Case 8

Shear
Strength

$$V_{ew\_5to8} = \begin{pmatrix} 3839 \\ 4769 \\ 1371 \\ 7232 \end{pmatrix} kip$$

Shear
See section 6.5.6.1.

$\phi V_{n\_max\_ew\_5to8} > V_{ew\_5to8}$ Limiting diaphragm shear strength satisfied.

6.5.6.3 Shear Reinforcement as per ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$$\frac{V_{ew\_5to8}}{\phi_{diaph}} := V_{c\_ew\_5to8} + V_{s\_ew\_5to8}$$

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

$$V_{c\_ew\_5to8} := 2 \cdot A_{cv\_ew\_5to8} \cdot \sqrt{f_c \cdot psi}$$

ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$$\phi_{diaph} \cdot V_{c\_ew\_5to8} = \begin{pmatrix} 3495 \\ 2871 \\ 788 \\ 5993 \end{pmatrix} kip$$

Case 5
Case 6
Case 7
Case 8

Shear
Strength

$$V_{ew\_5to8} = \begin{pmatrix} 3839 \\ 4769 \\ 1371 \\ 7232 \end{pmatrix} kip$$

Shear
See section 6.5.6.1.

For cases 5 to 8 the concrete shear strength [$\phi_{diaph} \cdot V_{c\_ew\_5to8}$] is less than the shear demand [V_{ew_5to8}] therefore shear reinforcing is required.

$$V_{s\_ew\_5to8} := \frac{V_{ew\_5to8}}{\phi_{diaph}} - V_{c\_ew\_5to8}$$

$$V_{s\_ew\_5to8} = \begin{pmatrix} 575 \\ 3162 \\ 972 \\ 2064 \end{pmatrix} kip$$

Case 5
Case 6
Case 7
Case 8

Required Reinforcing Shear Strength

6.5.6.4 Required Reinforcement

$$\rho_{req\_ew\_5to8} := \frac{V_{s\_ew\_5to8}}{A_{cv\_ew\_5to8} \cdot f_y}$$

ACI 349 (Ref. 2.2.3, sect. 21.6.5.2)

$$\rho_{req\_ew\_5to8} = \begin{pmatrix} 0.00023 \\ 0.00156 \\ 0.00174 \\ 0.00049 \end{pmatrix}$$

Case 5
Case 6
Case 7
Case 8

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

See attachment B for typical chord steel for diaphragm.

6.5.7.1 North/South Seismic Acceleration

$$M_{ns\_5to8} = \begin{pmatrix} 97610 \\ 100382 \\ 7617 \\ 24189 \end{pmatrix} ft \cdot kip$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Moment
See section 6.5.5.1.

$$DD_{ns\_5to8} = \begin{pmatrix} 94 \\ 94 \\ 94 \\ 82 \end{pmatrix} ft$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Depths
See section 6.5.5.1.

Chord Force (CF):

$$CF_{ns\_5to8} := \frac{M_{ns\_5to8}}{.9 \cdot DD_{ns\_5to8}}$$

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 witdh equal to 10% of the diaphragm depth. See attachment B.

$$CF_{ns\_5to8} = \begin{pmatrix} 1154 \\ 1187 \\ 90 \\ 328 \end{pmatrix} kip$$

Case 5
Case 6
Case 7
Case 8

Chord Forces

Required Chord Steel (A_{ch}):

$\phi_b = 0.9$

Strength reduction factor for bending
ACI 349 (Ref. 2.2.3, sect. 9.3.2.1)

$$A_{ch\_ns\_5to8} := \frac{CF_{ns\_5to8}}{\phi_b \cdot f_y}$$

$$A_{ch\_ns\_5to8} = \begin{pmatrix} 21.4 \\ 22.0 \\ 1.7 \\ 6.1 \end{pmatrix} in^2$$

Case 5
Case 6
Case 7
Case 8

Required Chord Steel

Provided Chord Steel (A_{ch}):

Use 22, 22, 2 & 7 - # 9 bars for cases 5, 6, 7 & 8 respectively (N/S Seismic Acceleration).

Chord steel provided = 22, 22, 2 & 7 in^2 for cases 5, 6, 7 & 8 respectively (N/S Seismic Acceleration).

See section 6.6.2.7 for chord steel provided.

6.5.7.2 East/West Seismic Acceleration

$$M_{ew\_5to8} = \begin{pmatrix} 90227 \\ 112064 \\ 32225 \\ 148252 \end{pmatrix} ft \cdot kip$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Moment
See section 6.5.6.1.

$$DD_{ew\_5to8} = \begin{pmatrix} 104 \\ 94 \\ 43 \\ 327 \end{pmatrix} ft$$

Case 5
Case 6
Case 7
Case 8

Diaphragm Depths
See section 6.5.6.1.

<u>Chord Force (CF):</u>

$$CF_{ew\_5to8} := \frac{M_{ew\_5to8}}{.9 \cdot DD_{ew\_5to8}}$$

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 witdh equal to 10% of the diaphragm depth. See attachment B.

$$CF_{ew\_5to8} = \begin{pmatrix} 964 \\ 1325 \\ 833 \\ 504 \end{pmatrix} kip$$

Case 5
Case 6
Case 7
Case 8

Chord Forces

<u>Required Chord Steel</u> (A_{ch}):

$\phi_b = 0.9$

Strength reduction factor for bending
ACI 349 (Ref. 2.2.3, sect. 9.3.2.1)

$$A_{ch\_ew\_5to8} := \frac{CF_{ew\_5to8}}{\phi_b \cdot f_y}$$

$$A_{ch\_ew\_5to8} = \begin{pmatrix} 17.9 \\ 24.5 \\ 15.4 \\ 9.3 \end{pmatrix} in^2$$

Case 5
Case 6
Case 7
Case 8

Required Chord Steel

<u>Provided Chord Steel</u> (A_{ch}):

Use 18, 25, 16 & 10 - # 9 bars for cases 5, 6, 7 & 8 respectively (E/W Seismic Acceleration).

Chord steel provided = 18,25,16 & 10 in^2 for cases 5, 6, 7 & 8 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 CRCF showing the diaphragms.

Case 1:Roof Diaphragm @ El.100':Col Lines 6-9/D-H (N/S dir)&6-9/E-G (E/W dir):18" slab

Case 2:Roof Diaphragm @ El. 72': Col Lines 9-12/E-G (N/S & E/W dir): 18" slab

Case 3:Roof Diaphragm @ El. 64': Col Lines 9-12/D-E (N/S & E/W dir): 18" slab
Also applies to Col Lines 9-12/G-H (N/S & E/W dir).

Case 4:Roof Diaphragm @ El. 64':Col Lines 2-3/D-E (N/S dir) & 2-6/D-E (E/W dir):18" slab
Also applies to Col Lines 2-3/G-H (N/S dir) & 2-6/G-H (E/W dir).

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.

$$\rho_{req\_1to4} := \begin{pmatrix} \rho_{req_1} \\ \rho_{req_1} \\ \rho_{req_1} \\ \rho_{req_1} \end{pmatrix} \begin{array}{l} \text{18" roof slab} \\ \text{18" roof slab} \\ \text{18" roof slab} \\ \text{18" roof slab} \end{array} \qquad \boxed{\rho_{req\_1to4} = \begin{pmatrix} 0.00042 \\ 0.00042 \\ 0.00042 \\ 0.00042 \end{pmatrix}} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

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>

$$\boxed{\rho_{req\_ns\_1to4} = \begin{pmatrix} 0.00219 \\ 0.00152 \\ 0.00094 \\ 0.00000 \end{pmatrix}} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

<u>E/W Seismic</u>

$$\boxed{\rho_{req\_ew\_1to4} = \begin{pmatrix} 0.00278 \\ 0.00246 \\ 0.00117 \\ 0.00120 \end{pmatrix}} \begin{array}{l} \text{Case 1} \\ \text{Case 2} \\ \text{Case 3} \\ \text{Case 4} \end{array}$$

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

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

$$\boxed{\rho_{req\_comb\_1to4} = \begin{pmatrix} 0.00181 \\ 0.00165 \\ 0.00101 \\ 0.00102 \end{pmatrix}}$$ Case 1, Case 2, Case 3, Case 4

$$d_{1to4} := \begin{pmatrix} d_{18r} \\ d_{18r} \\ d_{18r} \\ d_{18r} \end{pmatrix}$$ See section 6.3.5. Case 1, Case 2, Case 3, Case 4

$$\boxed{d_{1to4} = \begin{pmatrix} 15.13 \\ 15.13 \\ 15.13 \\ 15.13 \end{pmatrix} in}$$

$b = 1\ ft$ See section 6.3.4.

$$As_{req\_1to4} := \overrightarrow{\left[\left(\rho_{req\_comb\_1to4}\right)\cdot b\cdot d_{1to4}\right]}$$

$$\boxed{As_{req\_1to4} = \begin{pmatrix} 0.33 \\ 0.30 \\ 0.18 \\ 0.18 \end{pmatrix} in^2}$$ per foot Case 1, Case 2, Case 3, Case 4

6.6.1.4 Minimum Reinforcement Req'd

ACI 349 (Ref. 2.2.3, sect. 21.6.2.1) specifies that the minimum reinforcement for diaphragms shall be in conformance with ACI 349 (Ref. 2.2.3, sect. 7.12).

ACI 349 (Ref. 2.2.3, 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.

$$\boxed{h_{1to4} = \begin{pmatrix} 18 \\ 18 \\ 18 \\ 18 \end{pmatrix} in}$$ Case 1, Case 2, Case 3, Case 4 slab thickness, see section 6.4.5.2.

$b = 1\ ft$ See section 6.3.4.

$$As_{req\_1to4\_min} := 0.0018\cdot b\cdot h_{1to4}$$

$$\boxed{As_{req\_1to4\_min} = \begin{pmatrix} 0.39 \\ 0.39 \\ 0.39 \\ 0.39 \end{pmatrix} in^2}$$ per foot Case 1, Case 2, Case 3, Case 4 Minimum Reinforcement Req'd

6.6.1.5 Slab Reinforcement Req'd

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

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

Case 1
Case 2
Case 3
Case 4

6.6.1.6 Slab Reinforcement Provided

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

$$\boxed{As_{prov\_1to4} := \begin{pmatrix} 0.60 \\ 0.60 \\ 0.60 \\ 0.60 \end{pmatrix} \cdot in^2}$$ 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.

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

$$\boxed{RR_{1to4} = \begin{pmatrix} 0.65 \\ 0.65 \\ 0.65 \\ 0.65 \end{pmatrix}}$$ Reinforcement Ratio

Case 1
Case 2
Case 3
Case 4

6.6.1.7 Chord Steel Provided : See sections 6.4.7.1 & 6.4.7.2.

See attachment B for typical chord steel for diaphragm.

<u>Chord Steel Case 1 (El. 100')</u>

18 - # 9 bars along column lines D/H (between col lines 6 & 9)
20 - # 9 bars along column lines 6/9 (between col lines D & H)

<u>Chord Steel Case 2 (El. 72')</u>

14 - # 9 bars along column lines E/G (between col lines 9 & 12)
19 - # 9 bars along column lines 9/12 (between col lines E & G)

<u>Chord Steel Case 3 (El. 64')</u>

12 - # 9 bars along column lines D/E & G/H (between col lines 9 & 12)
12 - # 9 bars along column lines 9/12 (between col lines D/E & G/H)

<u>Chord Steel Case 4 (El. 64')</u>

6 - # 9 bars along column lines D/E & G/H (between col lines 2 & 6)
12 - # 9 bars along column lines 2/6 (between col lines D/E & G/H)

6.6.2 Analysis Cases 5 to 8

See attachment A for plans of the CRCF showing the diaphragms.

Case 5: Roof Diaphragm @ El. 64':Col Lines 2-5/E-G (N/S & E/W dir): 33" slab

Case 6: Floor Diaphragm @ El. 32':Col Lines 6-9/E-G (N/S & E/W dir): 48" slab

Case 7: Roof Diaphragm @ El. 32': Col Lines 12-13/E-G (N/S & E/W dir): 18" slab
Also applies to Col Lines 1-2/E-G (N/S & E/W dir).

Case 8: Floor Diaphragm @ El.32':Col Lines 2-3/D-E (N/S dir)&2-12/D-E(E/W dir):18" slab
Also applies to Col Lines 2-3/G-H (N/S dir) & 2-12/G-H (E/W dir).

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.

$$\rho_{req\_5to8} := \begin{pmatrix} \rho_{req_3} \\ \rho_{req_4} \\ \rho_{req_1} \\ \rho_{req_2} \end{pmatrix} \begin{array}{l} \text{33" roof slab} \\ \text{48" floor slab} \\ \text{18" roof slab} \\ \text{18" floor slab} \end{array}$$

$$\boxed{\rho_{req\_5to8} = \begin{pmatrix} 0.00015 \\ 0.00467 \\ 0.00042 \\ 0.00039 \end{pmatrix}} \begin{array}{l} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{array}$$

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.

N/S Seismic

$$\boxed{\rho_{req\_ns\_5to8} = \begin{pmatrix} 0.00044 \\ 0.00115 \\ 0 \\ 0.00009 \end{pmatrix}} \begin{array}{l} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{array}$$

E/W Seismic

$$\boxed{\rho_{req\_ew\_5to8} = \begin{pmatrix} 0.00023 \\ 0.00156 \\ 0.00174 \\ 0.00049 \end{pmatrix}} \begin{array}{l} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{array}$$

6.6.2.3 Combined Reinforcement Req'd

(Reinforcement required per face of diaphragm)

$$\rho_{req\_comb\_5to8} := \begin{pmatrix} \rho_{req\_5to8_1} + \dfrac{\max\left(\rho_{req\_ns\_5to8_1}, \rho_{req\_ew\_5to8_1}\right)}{2} \\ \rho_{req\_5to8_2} + \dfrac{\max\left(\rho_{req\_ns\_5to8_2}, \rho_{req\_ew\_5to8_2}\right)}{2} \\ \rho_{req\_5to8_3} + \dfrac{\max\left(\rho_{req\_ns\_5to8_3}, \rho_{req\_ew\_5to8_3}\right)}{2} \\ \rho_{req\_5to8_4} + \dfrac{\max\left(\rho_{req\_ns\_5to8_4}, \rho_{req\_ew\_5to8_4}\right)}{2} \end{pmatrix} \begin{array}{l} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{array}$$

$$\rho_{req\_comb\_5to8} = \begin{pmatrix} 0.00037 \\ 0.00545 \\ 0.00129 \\ 0.00064 \end{pmatrix}$$ Case 5, Case 6, Case 7, Case 8

$$d_{5to8} := \begin{pmatrix} d_{33} \\ d_{48} \\ d_{18r} \\ d_{18f} \end{pmatrix}$$ See section 6.3.5.

$$d_{5to8} = \begin{pmatrix} 30.13 \\ 27.13 \\ 15.13 \\ 15.13 \end{pmatrix} in$$ Case 5, Case 6, Case 7, Case 8

$b = 1\ ft$ See section 6.3.4.

$$As_{req\_5to8} := \overrightarrow{\left[\left(\rho_{req\_comb\_5to8}\right) \cdot b \cdot d_{5to8}\right]}$$

$$As_{req\_5to8} = \begin{pmatrix} 0.13 \\ 1.78 \\ 0.23 \\ 0.12 \end{pmatrix} in^2$$ per foot Case 5, Case 6, Case 7, Case 8

6.6.2.4 Minimum Reinforcement Req'd

ACI 349 (Ref. 2.2.3, sect. 21.6.2.1) specifies that the minimum reinforcement for diaphragms shall be in conformance with ACI 349 (Ref. 2.2.3, sect. 7.12).

ACI 349 (Ref. 2.2.3, 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.

$$h_{5to8max} := \begin{pmatrix} 33 \\ 48 \\ 18 \\ 18 \end{pmatrix} \cdot in$$ Case 5, Case 6, Case 7, Case 8

Slab Thickness, See section 6.5.1.
Note : Used 48" (maximum thickness) for 48" slab.

$b = 1\ ft$ See section 6.3.4.

$$As_{req\_5to8\_min} := 0.0018 \cdot b \cdot h_{5to8max}$$

$$As_{req\_5to8\_min} = \begin{pmatrix} 0.71 \\ 1.04 \\ 0.39 \\ 0.39 \end{pmatrix} in^2$$ per foot Case 5, Case 6, Case 7, Case 8

Minimum Reinforcement Req'd

6.6.2.5 Slab Reinforcement Req'd

$$As_{reqd\_5to8} := \begin{pmatrix} \max\left(As_{req\_5to8_1}, As_{req\_5to8\_min_1}\right) \\ \max\left(As_{req\_5to8_2}, As_{req\_5to8\_min_2}\right) \\ \max\left(As_{req\_5to8_3}, As_{req\_5to8\_min_3}\right) \\ \max\left(As_{req\_5to8_4}, As_{req\_5to8\_min_4}\right) \end{pmatrix}$$

$$\boxed{As_{reqd\_5to8} = \begin{pmatrix} 0.71 \\ 1.78 \\ 0.39 \\ 0.39 \end{pmatrix} in^2} \quad \text{per foot} \quad \begin{array}{l} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{array}$$

6.6.2.6 Slab Reinforcement Provided

Provide # 9 bars @ 12" on center, both ways, top & bottom for case 5.
Provide # 10 bars @ 6" on center, both ways, top & bottom for case 6.
Provide # 7 bars @ 12" on center, both ways, top & bottom for cases 7 & 8.

$$\boxed{As_{prov\_5to8} := \begin{pmatrix} 1.00 \\ 2.54 \\ 0.60 \\ 0.60 \end{pmatrix} \cdot in^2} \quad \text{per foot}$$

Case 5 : 33" thick roof slab @ el. 64'
Case 6 : 48" thick floor slab @ el. 32'
Case 7 : 18" thick roof slab @ el. 32'
Case 8 : 18" thick floor slab @ el. 32'

Reinforcement provided for cases 5 to 8 is more than required.

$$RR_{5to8} := \overrightarrow{\left(\frac{As_{reqd\_5to8}}{As_{prov\_5to8}}\right)}$$

$$\boxed{RR_{5to8} = \begin{pmatrix} 0.71 \\ 0.70 \\ 0.65 \\ 0.65 \end{pmatrix}} \quad \text{Reinforcement Ratio} \quad \begin{array}{l} \text{Case 5} \\ \text{Case 6} \\ \text{Case 7} \\ \text{Case 8} \end{array}$$

6.6.2.7 Chord Steel Provided : See sections 6.5.7.1 & 6.5.7.2.

See attachment B for typical chord steel for diaphragm.

Chord Steel Case 5 (El. 64')

22 - # 9 bars along column lines E/G (between col lines 2 & 6)
18 - # 9 bars along column lines 2/5 (between col lines E & G)

Chord Steel Case 6 (El. 32')

22 - # 9 bars along column lines E/G (between col lines 6 & 9)
25 - # 9 bars along column lines 6/9 (between col lines E & G)

Chord Steel Case 7 (El. 32')

2 - # 9 bars along column lines E/G (between col lines 1 & 2 and 12 & 13)
16 - # 9 bars along column lines 1/2 & 12/13 (between col lines E & G)

Chord Steel Case 8 (El. 32')

7 - # 9 bars along column lines D/E & G/H (between col lines 2 & 12)
10 - # 9 bars along column lines 2/12 (between col lines D/E & G/H)

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', 72', 64' & 32':

  # 7 reinforcing bars @ 12" on centers, both ways, top & bottom

- 33" thick slab @ el. 64':

  # 9 reinforcing bars @ 12" on centers, both ways, top & bottom

- 48" thick slabs @ el. 32':

  # 10 reinforcing bars @ 6" on centers, both ways, top & bottom

- Reinforcement Ratio = Reinforcement required / Reinforcement provided

  Cases 1, 2, 3, 4, 7 & 8 : 0.65 Case 5 : 0.71 Case 6 : 0.70

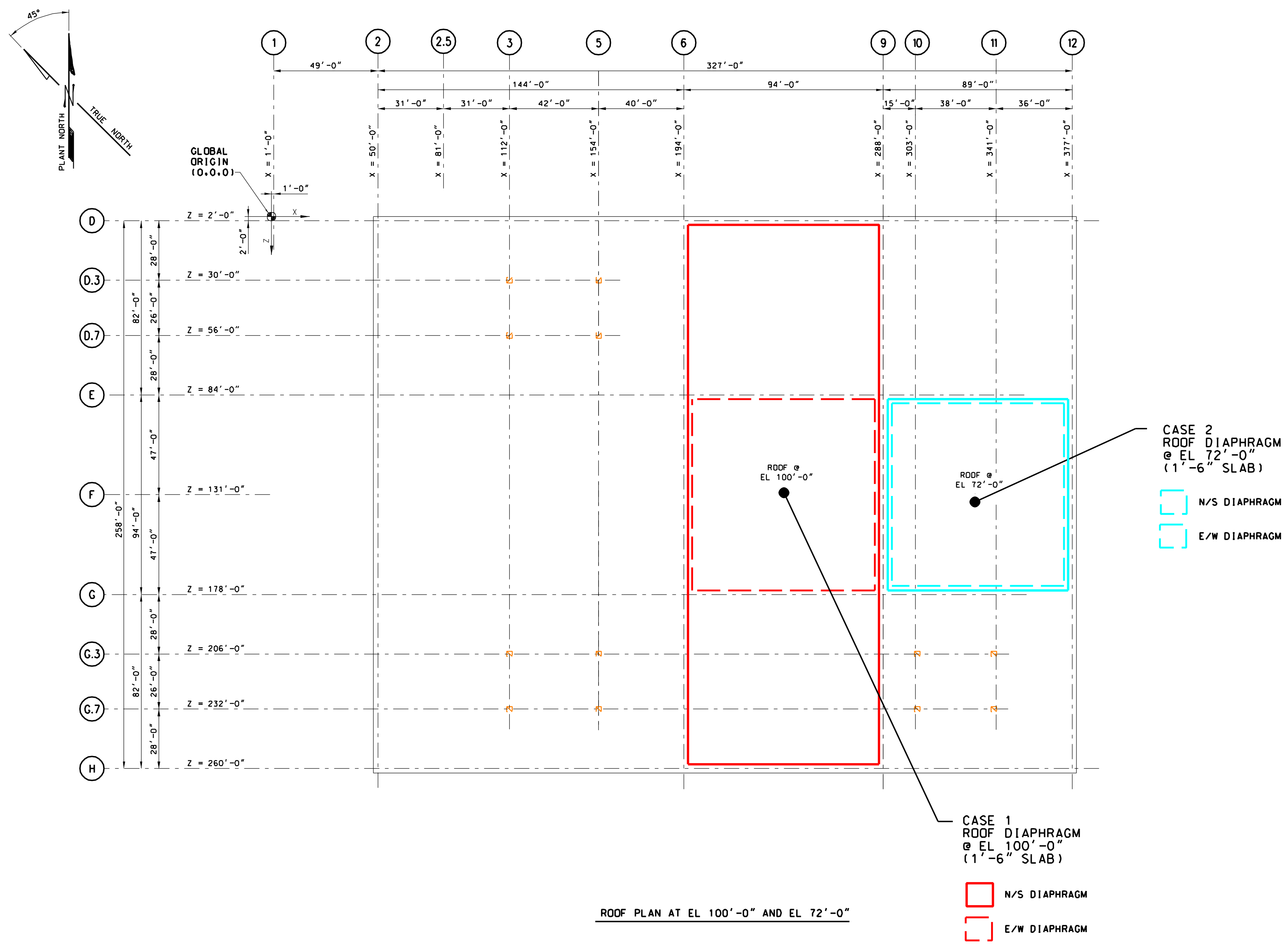
7.1.2 Chord reinforcement provided :

- See sections 6.6.1.7 and 6.6.2.7.

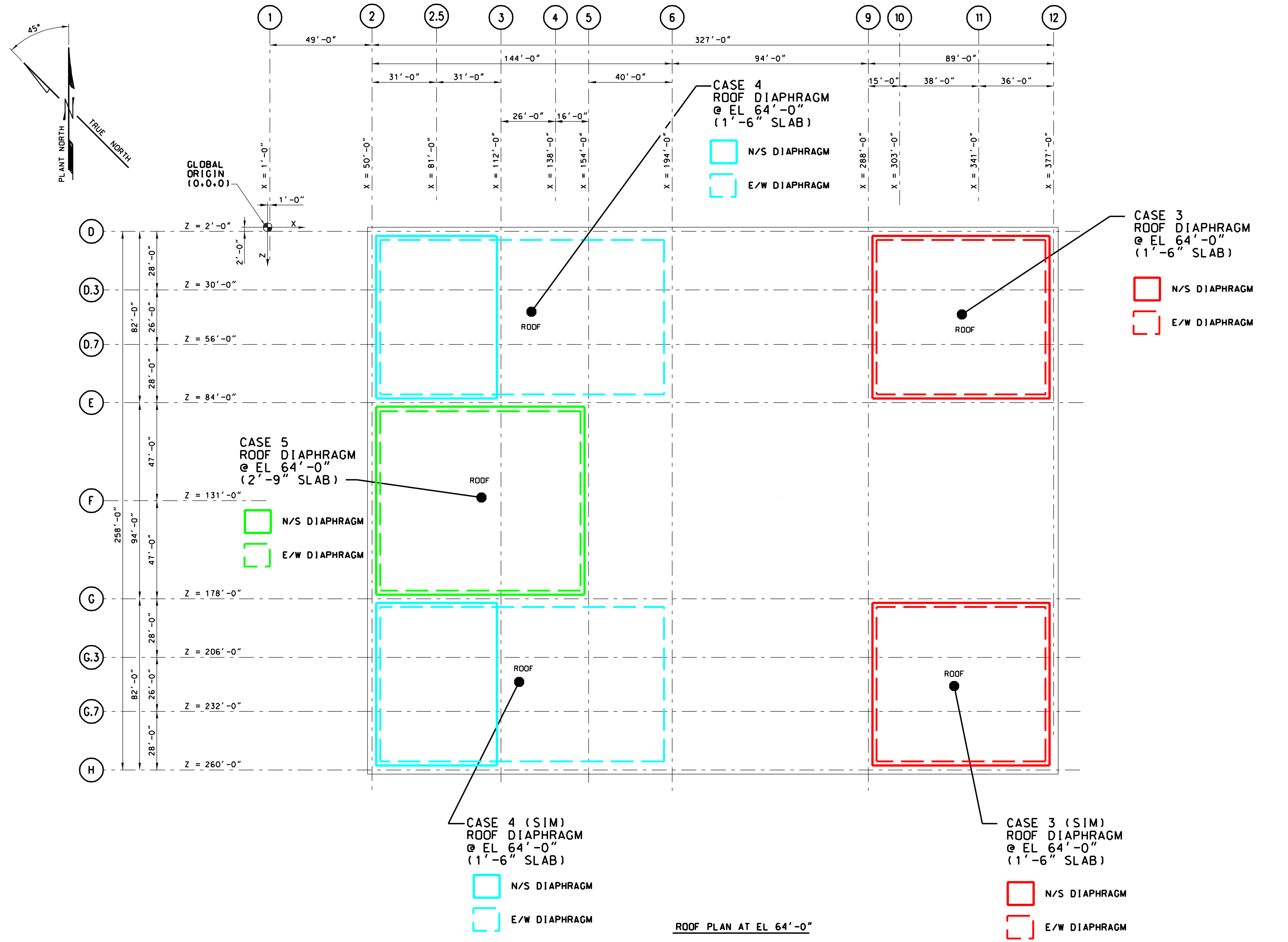
7.2 CONCLUSIONS

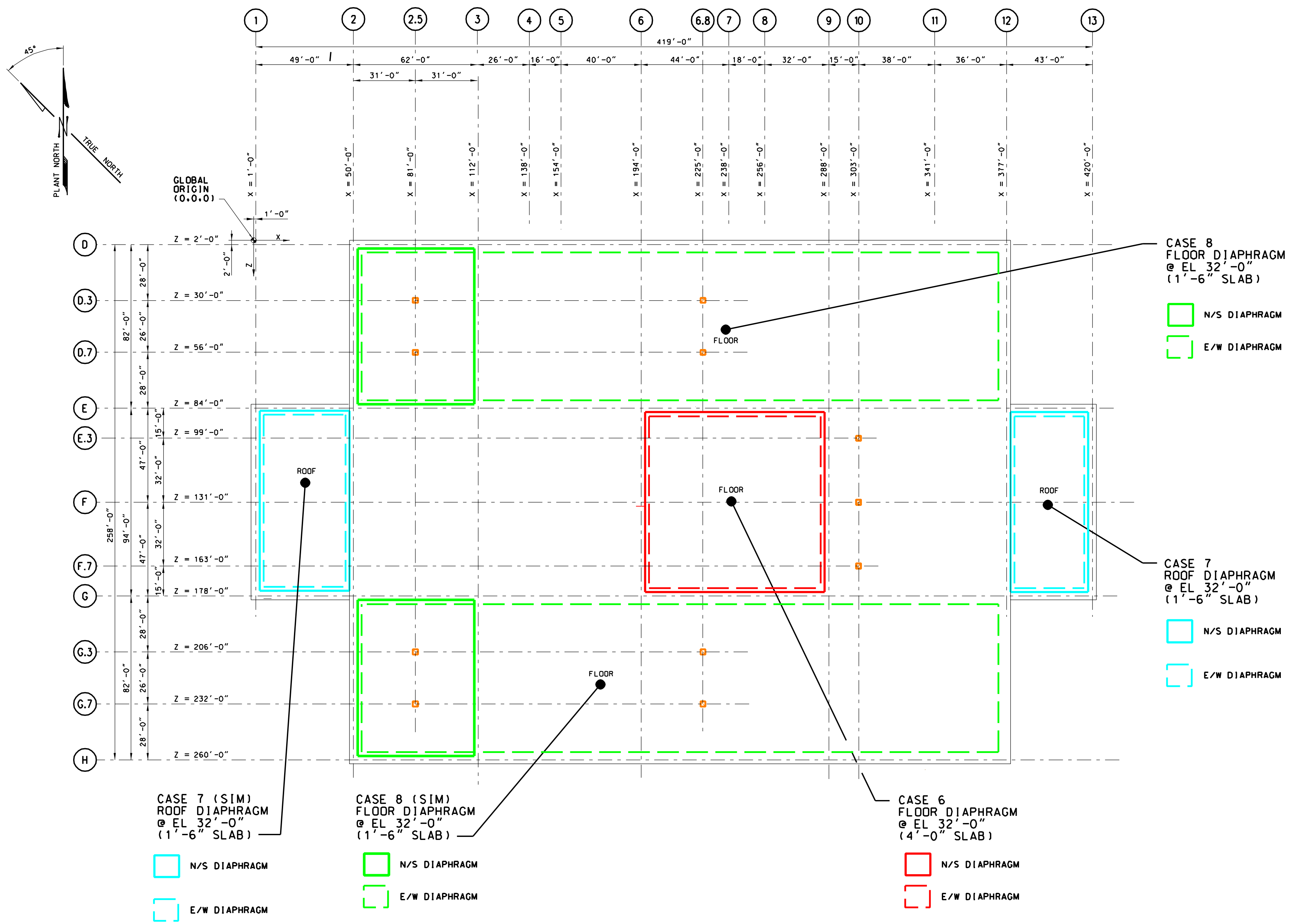
Results from this calculation demonstrate that for the slabs investigated a reasonable slab design is achieved for the imposed design loads. The slab reinforcement provided in section 7.1.1 of the calculation is reasonable for the type of structure under consideration and the types of loads applied to 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 putting the reinforcement as shown in attachment B 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 model will yield reduction in the chord reinforcement required.

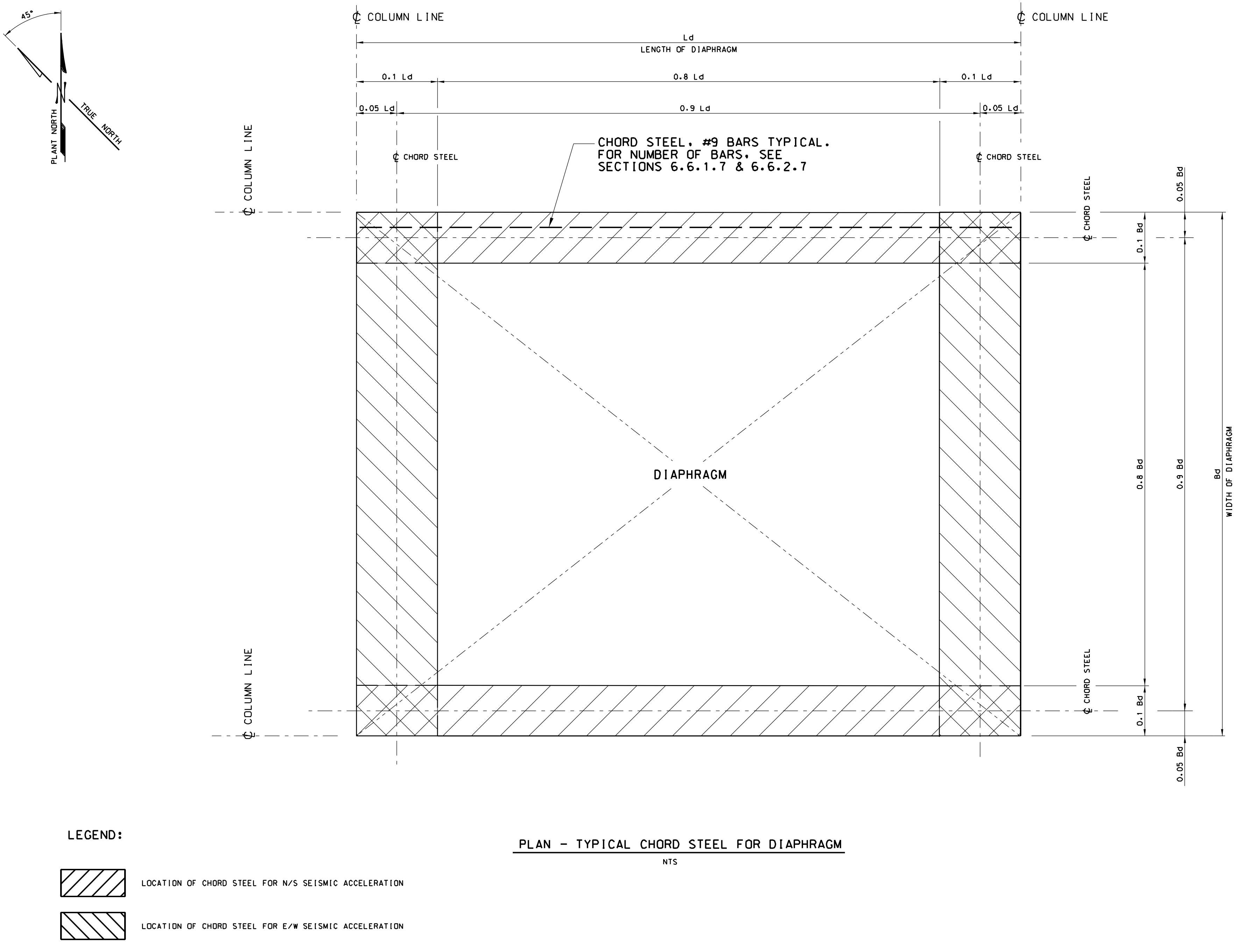


ROOF PLAN AT EL 100'-0" AND EL 72'-0"


CASE 4
ROOF DIAPHRAGM
@ EL 64'-0"
(1'-6" SLAB)
N/S DIAPHRAGM
E/W DIAPHRAGM
CASE 3
ROOF DIAPHRAGM
@ EL 64'-0"
(1'-6" SLAB)
CASE 5
ROOF DIAPHRAGM
@ EL 64'-0"
(2'-9" SLAB)
CASE 4 (SIM)
CASE 3 (SIM)
GLOBAL ORIGIN (0,0,0)
PLANT NORTH
TRUE NORTH
45°
ROOF
ROOF PLAN AT EL 64'-0"



FLOOR AND ROOF PLANS AT EL 32'-0" UNO



PLAN - TYPICAL CHORD STEEL FOR DIAPHRAGM

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