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LIST OF ACRONYMS

ACI	American Concrete Institute
A_{cv}	Area bounded by web thickness and length of section in the direction of the shear force considered.
ASCE	American Society of Civil Engineers
A_{sh}	Horizontal steel reinforcement area
A_{sv}	Vertical steel reinforcement area
BDBGM	Beyond Design Base Ground Motion (10,000 Year Return Period)
C/D	Capacity-to-Demand ratio
$C_{1\%}$	1% probability of failure capacity
$C_{50\%}$	Capacity at 50% probability of exceedance
$C_{98\%}$	Capacity at 98% probability of exceedance
C_{CDFM}	CDFM Capacity
CDFM	Conservative Deterministic Failure Margin
C_f	Frequency shift coefficient
C_{HCLPF}	HCLPF Capacity
CHF	Canister Handling Facility
Col.	Column
CRCF	Canister Receipt and Closure Facility
$C_{x\%}$	Capacity at x% probability of exceedance
D/C	Demand-to-Capacity ratio
D_{BDBGM}	Demand from BDBGM seismic load
DL	Dead Load
D_{NS}	Expected concurrent non-seismic demand
EL.	Elevation

E-W	East-West
f'_c	Concrete design strength at 28 days
f'_{cHCLPF}	Concrete compressive strength used in the HCLPF capacity calculations
F_s	Strength Margin Factor
f_y	Reinforcing steel yield strength = 60 ksi
$F\mu$	Energy Dissipation (Absorption) Factor
H	Height of wall between floor diaphragms
HCLPF	High-Confidence-of-Low-Probability-of-Failure
HVAC	Heating, Ventilation, and Air Conditioning
h_w	Height of wall or wall segment
Hz	Hertz
I-P	In-Plane
ITS	Important To Safety
LL	Live Load
l_w	Length of wall or wall segment
N	Effective number of strong nonlinear cycles
Na	Axial demand force
N-S	North-South
O-O-P	Out-of-Plane
PGA	Peak Ground Acceleration
PGA_h	Peak Horizontal Ground Acceleration
psi	Pounds per square inch
SADA	Seismic Analysis and Design Approach
SASSI	System for the Analysis of Soil-Structure Interaction
SPRA	Seismic Probabilistic Risk Assessment
SRSS	Square Root of Sum of Squares
SSI	Soil Structure Interaction

t_w	Thickness of wall segment
U1	E-W (X) direction displacement
U2	N-S (Y) direction displacement
V_c	Nominal shear strength provided by concrete
V_n	Nominal shear strength
V_s	Nominal shear strength provided by shear reinforcement
V_{uBDBGM}	In-plane shear demand force due to BDBGM loads
V_{uNS}	In-plane shear demand force due to non-seismic loads
YMP	Yucca Mountain Project
Z	Value of normal variant
ZPA	Zero Period Acceleration
β	Composite Variability
ϕV_n	In-plane shear code capacity
ρ_h	Horizontal reinforcing ratio
ρ_v	Vertical reinforcing ratio

1. PURPOSE

The purpose of this calculation is to perform a seismic fragility evaluation of the Canister Receipt and Closure Facility (CRCF) and to develop mean seismic fragility curves for the CRCF. The seismic fragility curve is developed using the 1% probability of unacceptable performance, $C_{1\%}$, approximated by the deterministically computed Conservative Deterministic Failure Margin (CDFM) methodology, and the composite logarithmic standard deviation, β (Ref. 2.2.4). The mean seismic fragility curve is defined in terms of the peak horizontal ground acceleration of the beyond design basis ground motion (BDBGM) given in Ref. 2.2.31.

The scope of this calculation includes the development of seismic fragility curves according to two sequences:

Building just short of collapse

The seismic fragility curve developed for this limit state should be used when the building collapse is the only event in the event sequence. For the purpose of this calculation, this building state is considered Limit State A (Table 1-1, Ref. 2.2.6).

Essentially elastic building behavior

The seismic fragility curve developed for this limit state should be used when essentially elastic behavior of the structure is required, such as ensuring building confinement integrity, together with other events that contribute to the critical event sequence. For the purpose of this calculation, this building state is considered Limit State D (Table 1-1, Ref. 2.2.6).

The seismic fragility curves developed in this calculation will be used in a limited probabilistic risk assessment of the CRCF.

The scope of this calculation does not include concrete and steel detailing.

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2.3 DESIGN CONSTRAINTS

None

2.4 DESIGN OUTPUTS

The seismic fragility curves developed in this calculation will be used in a limited seismic probabilistic risk assessment (SPRA) of the CRCF.

3. ASSUMPTIONS

3.1 ASSUMPTIONS REQUIRING VERIFICATION

3.1.1 The acceleration used for the out-of-plane seismic loads on walls is assumed as 1.5 times the maximum horizontal acceleration at the upper elevation of the wall.

Rationale: Based on preliminary CRCF Tier 2 SASSI analysis, a uniform acceleration equal to 1.5 times the maximum horizontal acceleration at the upper floor elevation for all other walls is a reasonable measure of the expected out-of-plane seismic accelerations for these walls.

3.1.2 NOT USED

3.1.3 NOT USED

3.1.4 Equipment dead loads are assumed as 50 psf on the floor and roof slabs. Equipment dead loads include HVAC equipment, electrical equipment, etc. Also, miscellaneous hanging equipment (cable trays, ductwork, etc.) is 10 psf.

Rationale: The CRCF is not an equipment intensive structure with the major equipment for diaphragm design being the HVAC equipment. 50 psf equipment load is a reasonable assumption for this type of structure.

3.1.5 Roofing material dead load is assumed as 15 psf.

Rationale: This is a reasonable assumption for built-up roofing material.

3.1.6 Live load is assumed as 100 psf for floor live load and 40 psf for roof live load.

Rationale: This is a reasonable assumption for this type of structure, as the primary source of live load is maintenance of HVAC and other equipment.

3.1.7 NOT USED

3.1.8 The amplified acceleration for out-of-plane seismic loads on a given slab is assumed as 2.0 times the vertical acceleration obtained from the CRCF seismic analysis (Ref. 2.2.5).

Rationale: The Tier-1 seismic analysis models did not include the effects of vertical floor flexibility on the seismic demands for the floor slabs and interior steel columns supporting the floor slabs. To obtain amplified vertical floor acceleration to be used in the design of floor slabs and supporting steel the following process was used.

A SASSI (System for the Analysis of Soil-Structure Interaction) analysis was performed on the Canister Handling Facility (CHF) (Ref. 2.2.34), a structure similar to the CRCF, which developed in-structure response spectra at hard points on the walls. Using the 7% damped vertical response spectra give in Fig. F-3 of Ref. 2.2.34, 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. 2.2.35). 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 approximately 20' on centers the fundamental mode is approximately 25 Hz with a mass participation of 50%. Thus, 50% of the mass is responding at this frequency and 50% of the mass responds at other frequencies. Assuming the remaining mass participates at the ZPA, the following equation can be written:

Response = (0.5*mass*ZPA) + (0.5*ratio*mass*ZPA)

Where ratio = (acceleration at 25 HZ) / ZPA

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

Response = 0.5*mass*ZPA + 0.5*2.3*mass*ZPA

Response = 1.65*mass*ZPA

Where the ZPA for the slab is the vertical acceleration from the CRCF seismic analysis (Ref. 2.2.5) at the floor level under consideration.

This procedure was carried out 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.

3.1.9 The final design of the CRCF is assumed to follow the ductile detailing requirements of ASCE 43-05 (Ref. 2.2.6).

Rationale: ASCE 43-05 (Ref. 2.2.6) is one of the design documents used for the CRCF structural analysis and design. Therefore, assuming that the detailing requirements of ASCE 43-05 will be followed is reasonable.

3.1.10 The forklift is assumed to not be included in the fragility event tree sequence. Therefore, the forklift weight is not included in the HCLPF capacity calculations.

Rationale: The operation of the forklift is a low frequency event. Therefore, neglecting the possibility of the forklift operation during the low frequency BDBGM seismic event is a reasonable assumption.

3.1.11 It is assumed that the concrete placed at different times will intentionally be roughened to a full amplitude of at least ¼ inches, as specified in Section 11.7.9 of Ref. 2.2.2.

Rationale: Typical construction practice on nuclear power plant jobs specifies that interfaces between concrete poured at different times be intentionally roughened. Therefore, assuming that this practice will be followed is reasonable. This requirement will be implemented in the detailed construction drawings and/or concrete specifications in the detailed design phase of the project.

3.2 ASSUMPTIONS NOT REQUIRING VERIFICATION

3.2.1 Unless otherwise noted, the out-of-plane analysis of all walls assumes simply supported vertical strips between diaphragms with uniform acceleration applied to the entire wall strip.

Rationale: Analyzing the walls as simply supported, one-way vertical beam strips with a uniform acceleration applied over the entire wall height, and neglecting two-way action of the wall panel, is bounding.

3.2.2 All slabs are assumed to be one-way slabs.

Rationale: Analyzing the floor slabs as one-way slabs instead of two-way slabs is bounding.

3.2.3 Multiple span diaphragms, when analyzing for in-plane forces, are taken as simply supported spans considering the largest span.

Rationale: Taking simple spans instead of multiple spans is bounding because the moments compute as simple spans envelopes the positive and negative moments compute as multiple spans.

3.2.4 The strong motion duration is assumed to be greater than 15 seconds. Therefore, per Table 4-2 of Ref. 2.2.42, the effective number of strong nonlinear cycles (N) is assumed as 4 and the frequency shift coefficient (C_f) is 2.7.

Rationale: This is a bounding assumption for the Fμ factor calculation for shear walls and the Fμ reduction calculation for ratcheting.

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. 2.2.3) classifies the CRCF structure as ITS. The approved record version of this document is designated QA:QA.

4.2 USE OF SOFTWARE

4.2.1 Word, Excel, and MathCAD

Word 2003 and Excel 2003, which are parts of the Microsoft Office Professional Edition 2003 suite of programs, were used in this calculation. Microsoft Office 2003 is classified as Level 2 software as defined in IT-PRO-0011, *Software Management*, (Ref. 2.1.2). Microsoft Office 2003 is listed on the current Level 2 Usage Controlled Software Report. Microsoft Office software with Software Track Number 610236-2003-00 is also listed in 000-PLN-MGR0-00200-000, *Repository Project Management Automation Plan*, (Ref. 2.1.3). Checking of the Excel computations in this calculation is performed using a hand calculator and/or by visual inspection.

MathCAD version 13 was utilized to perform mathematical computations in this calculation. MathCAD version 13 is classified as Level 2 software as defined in IT-PRO-0011, *Software Management*, (Ref. 2.1.2). All MathCAD input values and equations are stated in the calculations. Checking of the MathCAD results was done using a hand calculator, by comparison to known solutions, and/or by visual inspection.

MathCAD version 13 is listed on the Level 2 Usage Controlled Software Report (SW Tracking Number 61116-13-00), as well as in 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 2003 operating system.

4.2.2 SAP2000

SAP2000, Version 9, as used in this calculation, is classified as Level 1 software usage as defined in IT-PRO-0011 (Ref. 2.1.2). This software is a commercially available computer program qualified to perform static and dynamic analysis of structural systems. The software validation report is given in 11198-SVR-9.1.4-00-WIN2000 (Ref. 2.2.45). This software is listed in the Qualified and Controlled Software Report as qualified with

Software Tracking Number 11198-9.1.4-00 as well as the Repository Project Management Automation Plan (Ref. 2.1.3).

The software is operated on a PC system running the Windows 2000 operating system.

4.3 DESIGN APPROACH

4.3.1 Introduction

The seismic fragility curves for the CRCF are developed following the guidelines established in Appendix B, Section B3 of the Seismic Analysis and Design Approach Document (SADA) (Ref. 2.2.4). The sections of Appendix B applicable to this calculation are summarized below for traceability purposes.

A High-Confidence-Low-Probability-of-Failure capacity (C_{HCLPF}) is the ground motion level at which there is approximately 95% confidence of less than or about 5% probability of failure. Worded differently, the HCLPF capacity corresponds to approximately the 1% probability of failure point on the mean (composite) fragility curve ($C_{1\%}$). In this calculation, HCLPF capacities are calculated in terms of the peak horizontal ground acceleration (PGA_h) of the BDBGM seismic input motion defined in Ref. 2.2.31. Also, the HCLPF capacities are estimated using the Conservative Deterministic Failure Margin (CDFM) Method.

According to Ref. 2.2.28, the terms C_{HCLPF}, $C_{1\%}$, and C_{CDFM} are essentially interchangeable terms for the same Seismic Margin Capacity and are used as such in this calculation.

HCLPF calculations consider all failure modes in order to determine the "weakest link" of the structure. Based on experience with seismic probabilistic risk assessments of nuclear power plants, the HCLPF capacity is based on in-plane shear for shear walls and out-of-plane bending for slabs. In order to demonstrate the adequacy of the entire structure, additional evaluations must be carried out. The evaluations performed for the HCLPF capacity evaluation of the CRCF structure are as follows:

Primary HCLPF Capacity Calculations
- In-plane shear of shear walls
- Out-of-plane bending and out-of-plane shear of floor diaphragms

Additional HCLPF Capacity Evaluations
- Out-of-plane bending of shear walls
- In-plane bending and in-plane shear of floor diaphragms
- Axial force in combination with in-plane bending of walls

The HCLPF capacity for the entire CRCF structure will be governed by the lowest in-plane shear HCLPF capacity of the CRCF shear walls. Other failure mechanisms with HCLPF capacities lower than the governing shear wall will be modified by revising the reinforcing provided to increase the HCLPF capacity above the governing shear wall capacity.

The mean seismic fragility curves will then be controlled by the minimum HCLPF capacity of the CRCF shear walls.

4.3.2 Steps for Calculating Seismic Fragility Curves for the CRCF

The following steps describe the procedure used in this calculation to develop the seismic fragility curves for the CRCF.

Step 1: Determine HCLPF Capacity (C_{HCLPF})

The HCLPF capacity of any structural element is estimated from:

$$C_{HCLPF} = F_S * F_\mu * PGA_{BDBGM} \qquad \text{(Eq. 4-1)}$$

where

PGA_{BDBGM}: peak horizontal ground acceleration (g) of the Beyond Design Basis Ground Motion

F_S = computed strength margin factor

F_μ = inelastic energy dissipation factor

Step 1a: Strength Margin Factor (F_s)

The strength margin factor for an individual element is given by:

$$F_S = \frac{C_{98\%} - D_{NS}}{D_{BDBGM}} \qquad \text{(Eq. 4-2)}$$

where

$C_{98\%}$ = element capacity computed using code capacity acceptance criteria (including code specified strength reduction factors ϕ)

D_{NS} = expected concurrent non-seismic demand. For this calculation, the concurrent non-seismic demand is considered as the dead load plus 25% of the design live load.

D_{BDBGM} = seismic demand computed for the BDBGM input in accordance with the requirements of ASCE 4-98 (Ref. 2.2.32), Section 3.1.1.2. The seismic demand forces are retrieved from the seismic analysis results in Ref. 2.2.5. From Ref. 2.2.5, the 100-ft Alluvium Upper Bound soil condition case bounds all other soil case conditions. Therefore, all references to the seismic results developed in Ref. 2.2.5 refer to the results of the 100-ft Alluvium Upper Bound soil condition case.

If force redistribution to another element or a group of elements is possible, the strength margin factor for the group of elements is given by:

$$F_S = \frac{\sum_{i=1}^{n}(C_{98\%} - D_{NS})}{\sum_{i=1}^{n} D_{BDBGM}} \qquad \text{(Eq. 4-3)}$$

where

n = number of elements to which demands can be redistributed

Examples of when redistribution is possible are individual wall elements (piers) that comprise a wall with openings. Also, if the floor diaphragms can transmit the required demand, the strength margin factor for a series of adjacent shear walls can be computed using Equation 4-3.

Step 1b: Inelastic Energy Dissipation Factor (F_μ)

General estimates for the inelastic energy dissipation (absorption) factor for a range of structural elements and for a given Limit State are provided in ASCE/SEI 43-05, Tables 5-1 (Ref. 2.2.6). If applicable, the F_μ factors given in Table 5-1 of Ref. 2.2.6 are reduced to account for weak and/or soft story effects, as required by Section 5.1.2.1 of Ref. 2.2.6. Per Section 1.3 of Ref, 2.2.6, other methods for computing Fμ factors may be employed, such as the Effective Frequency Method discussed in Ref. 2.2.42 and Ref. 2.2.43.

For this calculation, Limit State A and Limit State D are considered. For Limit State D, the inelastic energy dissipation factor is 1.0 for all structural elements. Therefore, the Limit State D HCLPF capacities are obtained by dividing the Limit State A HCLPF capacity by the Fμ factor associated with the HCLPF capacity. See Section 6.8 for further discussion of Limit State D evaluations.

Step 2: Determine Minimum HCLPF Capacity

Determine the minimum HCLPF capacity as the minimum of the HCLPF capacities of the CRCF shear walls. Also, ensure that the HCLPF capacities for the other failure modes are greater than the minimum shear wall HCLPF capacity.

Step 3: Estimate the Fragility Logarithmic Standard Deviation (β)

The fragility logarithmic standard deviation (β) is estimated by judgment following the guidance in ASCE/SEI 43-05 (Ref. 2.2.6). For structures and major passive mechanical components mounted on the ground or at low elevations within structures, β typically ranges from 0.3 to 0.5.

Step 4: Develop Fragility Curves

The mean fragility curve is defined with a lognormal distribution with a $C_{1\%}$ capacity and a logarithmic standard deviation, β. Recognizing that $C_{1\%} \approx C_{HCLPF}$, the median capacity is given by:

$$C_{50\%} = C_{1\%}e^{2.326\beta} \qquad \text{(Eq. 4-4)}$$

where 2.326 is the number of standard normal variants that the 1% point lies below the 50% point (Ref. 2.2.28). For any other probability level x, the capacity is given by

$$C_{x\%} = C_{50\%}e^{Z\beta} \qquad \text{(Eq. 4-5)}$$

where Z is the number of standard normal variants from the mean to the x-level of performance. Equations 4-4 and 4-5 are used to develop the mean seismic fragility curves of the CRCF.

5 LIST OF ATTACHMENTS

Number of Pages

ATTACHMENT A: FLOOR PLAN AND WALL ELEVATIONS………………21

ATTACHMENT B: SHEAR WALL DESIGN SUMMARY TABLE.… ……………5

ATTACHMENT C: ………………………………………………………….CD
File: *BDBGM 100 UPPER BOUND RESULTS.xls* – Contains BDBGM seismic analysis results from Ref. 2.2.5 used in this calculation

ATTACHMENT D: ………………………………………………………….CD
Contains the SAP model and analysis results for the Beam Model developed in Section 6.3 for the HCLPF capacity evaluation of the slab at EL. 32'-0"

ATTACHMENT E: ………………………………………………………….CD
File: *CRCF – Fragility – In-Plane Shear Wall.xls* – Contains the results from the HCLPF capacity evaluation of the CRCF shear walls in Section 6.2
File: *CRCF – Shear Friction.xls* – Contains the results from the shear friction evaluation in Section 6.7

ATTACHMENT F: ………………………………………………………….CD
Contains the files used in the axial force in combination with in-plane bending of walls HCLPF evaluation of Section 6.5

ATTACHMENT G: Structural Steel Framing Schematics.… ……………………3

ATTACHMENT H: Establishment of hw for In-Plane Shear HCLPF Capacity Evaluations.… …………………………………………………1

6 BODY OF CALCULATION

6.1 SYMBOLS AND NOTATIONS

See LIST OF ACRONYMS section for the symbols and notations used in this calculation.

6.2 HCLPF CAPACITY CALCULATIONS FOR IN-PLANE SHEAR OF SHEAR WALLS

6.2.1 Seismic Analysis Results

The stick model and results of the CRCF seismic analysis (Ref. 2.2.5) are used in the in-plane shear HCLPF capacity calculation for the CRCF. The seismic analysis results for the 100-ft upper bound soil case from Attachment J of Ref. 2.2.5 are included in Attachment C. The seismic analysis performed a modal analysis using the SRSS method to combine the spatial components of the earthquake. The axial force and in-plane shear force for the seismic and non-seismic load cases are the analysis results required for the in-plane shear HCLPF capacity calculation. The SAP2000 results from Ref. 2.2.5 consider a negative axial force equal to a net compression. However, for the purposes of the in-plane shear HCLPF calculation, a negative (-) axial force indicates an element with net tension.

6.2.2 Element Properties

The element (stick) properties for each wall of the CRCF are determined from Attachment B of Ref. 2.2.5. The element properties retrieved from Ref. 2.2.5 are shown on the “Frag. Shear Calculation” sheet of the file “CRCF – Fragility – In-Plane Shear Wall.xls” included in Attachment E. The following table describes these element properties.

Table 6.2.1 Stick Element Properties

Excel Column*	Property Name	Description
B	Stick ID	Beam element name
C	Joint I	ID of the starting node of the beam element
D	Joint J	ID of the ending node of the beam element
E	Length	Length of the beam element (feet)
F, G, H	X, Y, Z Centroid	Global X,Y, and Z coordinate of the beam element centroid (feet)

* Source - sheet “Frag. Shear Calculation” in file “CRCF – Fragility - In-Plane Shear Wall.xls” in Attachment E

6.2.3 Wall Design Parameters

The CRCF Shear Wall Design calculation (Ref. 2.2.29) is used to retrieve the wall design parameters used in the shear wall design. The following table describes those parameters (Excel columns J-P) retrieved from Attachment B of Ref. 2.2.29. The horizontal and vertical steel areas for each stick element are shown in Attachment B. Also, the stick demand forces required for the in-plane shear HCLPF calculations (Excel columns Q, AD, AE, and AF) are discussed. These stick forces are retrieved from the results of the seismic analysis documented in Ref. 2.2.5 and are also given in Attachment C for usage in this calculation.

Table 6.2.2 Stick Element Wall Design Parameters and Stick Forces

Excel Column*	Parameter Name	Description
J	hw Revised?	(See Note 1)
K	h_w	height of wall segment (feet)
L	l_w	length of wall segment (feet)
M	H	distance between floor diaphragms (feet)
N	thick	wall thickness (feet)
O	A_{sv}	vertical wall reinforcement (in^2 / ft / face)
P	A_{sh}	horizontal wall reinforcement (in^2 / ft / face)
Q	Na	Maximum tension force on the stick element. (See Note 2)
AD	In-Plane Direction	NS = stick element is part of a N-S shear wall; EW = stick element is part of a E-W shear wall
AE	Vu_{NS}	In-plane shear due to the non-seismic load (DL+LL) (See Note 3)
AF	Vu_{BDBGM}	In-plane shear due to the BDBGM seismic load (BDBGM_SRSS) (See Note 3)

* Source - sheet "Frag. Shear Calculation" in file "CRCF – Fragility - In-Plane Shear Wall.xls" in Attachment E

Notes:

(1) ACI 349-01 (Ref. 2.2.2) equations were used to calculate the in-plane shear strengths for the CRCF walls in Ref. 2.2.29. These equations are based on hw/lw, but are not direct functions of hw/lw. However, the HCLPF capacities for in-plane shear are based on equation 4-3 given in ASCE/SEI 43-05 (Ref. 2.2.6) Section 4.2.3 (herein referred to as the Barda equation). The Barda equation is more sensitive to the value of hw/lw than the ACI 349-01 equations. Therefore, if required, the values of hw given in Ref. 2.2.29 are revised to produce a conservative HCLPF capacity. See Attachment H for further discussion of hw used in this evaluation.

(2) Calculated as the difference between the axial force due to the BDBGM_SRSS load (positive vertical direction case for maximum tension) and the axial force due to the DL+LL load from Ref. 2.2.5. See Sheet "Element Forces – Na Calc" in the Excel file "CRCF – Fragility - In-Plane Shear Wall.xls" included in Attachment E.

(3) See Sheet "Element Forces – Vn Calc" , columns Z and AA, in the Excel file "CRCF – Fragility - In-Plane Shear Wall.xls" included in Attachment E.

6.2.4 Strength Margin Factor (F_s) Calculation

The strength margin factor (F_s) for in-plane shear of the CRCF shear walls is based on three values: the in-plane shear demand from the concurrent non-seismic loads (Vu_{NS}), the in-plane shear demand from the BDBGM loads (Vu_{BDBGM}), and the in-plane shear capacity of the shear wall (ϕV_n). These values are determined for each stick element of the CRCF model as follows:

Vu_{NS} = in-plane shear force due to load case 'DL+LL' (from Ref. 2.2.5)
Vu_{BDBGM} = in-plane shear force due to the load case 'BDBGM_SRSS' (from Ref. 2.2.5)
ϕV_n = in-plane shear capacity (See the following discussion)

Per Section 4.2.3 of Ref. 2.2.6, for $h_w / l_w \leq 2.0$, ϕV_n is given as follows:

$$v_u = \phi\left[8.3\sqrt{f'_c} - 3.4\sqrt{f'_c}\,(\frac{h_w}{l_w} - 0.5) + \frac{N_a}{4l_w t_w} + \rho_{se} f_y\right] \qquad \text{(Eq. 6.2.1)}$$

where

ϕ = Capacity reduction factor (= 0.8)
v_u = Ultimate shear strength (psi)
f'_c = Concrete compressive strength (psi)
h_w = Wall height (in.)
l_w = Wall length (in.)
Na = Axial force (lb) (- = tension; + = compression)
t_w = Wall thickness (in.)
ρ_{se} = $A\rho_v + B\rho_h$ (shall not exceed 0.01)
f_y = Steel yield stress (psi)
ρ_v = Vertical steel reinforcement ratio
ρ_h = Horizontal steel reinforcement ratio
A,B = Constants given as follows:

$h_w/l_w \leq 0.5$ → A = 1.0, B = 0
$0.5 \leq h_w/l_w \leq 1.5$ → $A = -h_w/l_w + 1.5$, $B = h_w/l_w - 0.5$
$h_w/l_w \geq 1.5$ → A = 0, B = 1.0

Per ASCE 43-05 Section 4.2.3 (Ref. 2.2.6), the ultimate shear strength given in Eq. 6.2.1 shall not exceed $20\phi(f'_c)^{1/2}$.

The total shear capacity is

$$\phi V_n = v_u \cdot d \cdot t_w \qquad \text{(Eq. 6.2.2)}$$

where d is the distance from the extreme compression fiber to the center of force of all reinforcement in tension, and may be conservatively estimated from the following (Ref. 2.2.6):

$$d = 0.6 \cdot l_w \qquad \text{(Eq. 6.2.3)}$$

Per Section 4.2.3 of Ref. 2.2.6, for h_w / l_w > 2.0, ϕV_n is given as follows by ACI 349-01 (Ref. 2.2.2). Per section 11.3.2.3 of ACI 349-01, for members subject to significant axial tension, equation 11-8 shall be used and is given as follows:

$$v_u = \phi\left[2(1+\frac{N_a}{500A_g})\sqrt{f'_c} + \rho_h f_y\right] \qquad \text{(Eq. 6.2.4)}$$

where

ϕ = Capacity reduction factor (= 0.6)
v_u = Ultimate shear stress (psi)
f'_c = Concrete compressive strength (psi)
f_y = Steel yield stress (psi)
ρ_h = Horizontal steel reinforcement ratio
N_a = Axial force (lb) (- = tension)
A_g = gross area of wall = t_w x l_w

Per ACI 349-01 Section 21.6.5.6 (Ref. 2.2.2), the ultimate shear strength for wall piers sharing loading shall not exceed $8\phi(f'_c)^{1/2}$. Therefore, the ultimate shear strength for wall piers with hw/lw > 2.0 is limited to the smaller of Equation 6.2.4 and $8\phi(f'_c)^{1/2}$.

The total shear capacity is

$$\phi V_n = v_u \cdot d \cdot t_w \qquad \text{(Eq. 6.2.5)}$$

where d is the distance from the extreme compression fiber to the center of force of all reinforcements in tension, and may be estimated from the following (Ref. 2.2.2 Section 11.10.4):

$$d = 0.8 \cdot l_w \qquad \text{(Eq. 6.2.6)}$$

6.2.4.1 Concrete Compressive Strength (f'c)

Ref. 2.2.1 gives the 28-day concrete compressive strength as 5,000 psi for the design of ITS structures. For the HCLPF capacity evaluations the concrete compressive strength is specified as the code specified minimum strength or at the 95% exceedance of the actual strength if test data are available. Section 2, page 2-51 and 2-52 of Ref. 2.2.43 discusses typical concrete strength increase factors that meet the requirements for HCLPF capacity evaluations. On page 2-52 of Ref. 2.2.43, it is stated that "Concrete compressive strength increases are likely to range from 10% to 45% over minimum specified 28-day strengths…". Therefore, a concrete compressive strength increase factor of 1.10 is used in the HCLPF capacity evaluations in this calculation.

For concrete compressive strength, the 95% exceedance level is specified as follows -
28-day design strength: 5,000 psi
Increase factor: 1.10

f'_{cHCLPF} = 5,000psi x 1.10 = 5,500 psi ← **Concrete compressive strength used in the HCLPF capacity calculations**

6.2.5 Inelastic Energy Absorption (F_μ) Factor for Shear Walls

Table 5-1 of ASCE/SEI 43-05 (Ref. 2.2.6) gives the F_μ factor for shear controlled shear walls as 2.0 for Limit State A. Section 5.1.2.1 of Ref. 2.2.6 states that the F_μ factor for a structure with a weak or soft story must be reduced according to equation 5-2(a), where a weak story is one in which the story lateral strength (defined as the capacity (C) to demand (D) ratio (C/D) of the story) is less than 80% of the immediate story above.

The reduced F_μ is calculated according to equation 5-2(a) in Ref. 2.2.6, and is given as follows:

$$F_{\mu reduced} := 1 + 2\left(F_\mu - 1\right)\cdot\frac{n-k+1}{n\cdot(n+1)}$$

where

- F_μ = Inelastic energy absorption factor (= 2.0 for Limit State A per Table 5-1 in Ref. 2.2.6)
- n = Number of stories in the structure (= 3 for the CRCF)
- k = Story level of the highest weak story in the structure

Based on the C/D ratios given in Step 3 of the following calculation, the C/D of the first story is less than 80% of the second story (i.e. the 1st story is a weak story). Also, the C/D of the second story is less than 80% of the third story (i.e. the 2nd story is a weak story). The reduced F_μ is calculated with k = 1 and k = 2 to determine the effects of the weak story location on the reduced F_μ calculation.

$$F_\mu := 2.0$$

$$n := 3$$

$$F_\mu(k) := 1 + 2\cdot\left(F_\mu - 1\right)\cdot\frac{n-k+1}{n\cdot(n+1)}$$

1st story is the weak story $\quad F\mu_{1st} := F_\mu(1) \quad F\mu_{1st} = 1.50$

2nd story is the weak story $\quad F\mu_{2nd} := F_\mu(2) \quad F\mu_{2nd} = 1.33$

The reduced F_μ calculation shown above is quite conservative, given the conservative nature of the ASCE 43-05 F_μ factors.

In lieu of equation 5-2(a), Section 1.3 of Ref. 2.2.6 allows for the use of alternate methods in estimating the inelastic energy absorption capacity factors for shear walls structures. One such alternate method is the Effective Frequency Method discussed in Ref. 2.2.42 and Ref. 2.2.43.

The Effective Frequency Method is used to estimate the F_μ factor used in the shear wall HCLPF capacity calculations of the CRCF. F_μ factors are estimated for the N-S shear walls and the E-W shear walls. The following calculation follows the methodology discussed in Appendix M of Ref. 2.2.43

$\text{ORIGIN} := 1$ Set the array origin to 1

Step 1: Determine Displacements, Shear Demands, and Capacities for each story

Three stories are considered in this calculation:

Story 1: EL. 0' to EL. 32' (Stick Elements with starting nodes at EL. 0')
Story 2: EL. 32' to EL. 64' (Stick Elements with starting nodes at EL. 32')
Story 3: EL. 64' to EL. 100' (Stick Elements with starting nodes at EL. 64')

The stick elements from EL. 64' to EL. 72' feet exist only in a small portion of the CRCF. Therefore, these stick elements are not considered to constitute a separate story.

Story Drifts
The average story drifts at EL. 32', EL. 64' and EL. 100' due to the BDBGM load are used. The story drifts are defined as the difference between the average displacement at the respective elevation and the displacement at EL. 0'. The rigid body displacement of the soil-structure system does not contribute any displacement demand on the structure and thus is not included in the story drift calculation.

The U1 (E-W) average story drifts are used for the E-W shear wall F_μ calculation and the U2 (N-S) average story drifts are used for the N-S shear wall F_μ calculation.

$EL0_{dispNS} := 0.276in$ Cell "W7" in Sheet "Joint Displ - Fu Calc" included in Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E

$EL0_{dispEW} := 0.290in$ Cell "V7" in Sheet "Joint Displ - Fu Calc" included in Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E

$$story_{dispNS} := \begin{pmatrix} 0.370 \\ 0.425 \\ 0.478 \end{pmatrix} \cdot in$$

Average story displacements in the N-S direction. Calculated in Column W of Sheet "Joint Displ - Fu Calc" included in Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E

$$story_{dispEW} := \begin{pmatrix} 0.372 \\ 0.434 \\ 0.491 \end{pmatrix} \cdot in$$

Average story displacements in the E-W direction. Calculated in Column V of Sheet "Joint Displ - Fu Calc" included in Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E

$$story_{\Delta NS} := story_{dispNS} - EL0_{dispNS} \qquad story_{\Delta NS} = \begin{pmatrix} 0.094 \\ 0.149 \\ 0.202 \end{pmatrix} in$$

Relative story displacements in N-S direction

$$story_{\Delta EW} := story_{dispEW} - EL0_{dispEW} \qquad story_{\Delta EW} = \begin{pmatrix} 0.082 \\ 0.144 \\ 0.201 \end{pmatrix} in$$

Relative story displacements in E-W direction

Story Shear Demands
The summation of the in-plane shear demands at EL. 0' (Story 1), EL. 32' (Story 2), and EL. 64' (Story 3) due to the BDBGM load. The in-plane shear demands on the N-S stick elements are used for the F_μ calculation of the N-S shear walls and the in-plane shear demands on the E-W stick elements are used for the F_μ calculation of the E-W shear walls.

$$story_{VrNS} := \begin{pmatrix} 253200 \\ 129912 \\ 39418 \end{pmatrix} \cdot kip$$

Story demands on the N-S shear walls.
Calculated in Columns AK, AM, and AO of Sheet "Fu Calculation" included in Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E. The summation of all in-plane shear demands on the N-S sticks at EL. 0', EL. 32', and EL. 64' are given in Row 6 of the columns listed above.

$$story_{VrEW} := \begin{pmatrix} 250188 \\ 130249 \\ 39603 \end{pmatrix} \cdot kip$$

Story demands on the E-W shear walls.
Calculated in Columns AS, AU, and AW of Sheet "Fu Calculation" included in Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E. The summation of all in-plane shear demands on the E-W sticks at EL. 0', EL. 32', and EL. 64' are given in Row 6 of the columns listed above.

Story Capacities

The summation of the in-plane shear capacities for the stick elements at EL. 0' (Story 1), EL. 32' (Story 2), and EL. 64' (Story 3). The individual stick element capacities are calculated using the methodology discussed in Section 6.2.4.

$$story_{VcNS} := \begin{pmatrix} 348924 \\ 374909 \\ 158871 \end{pmatrix} \cdot kip$$

Story capacities of the N-S shear walls.
Calculated in Columns S, U, and W of Sheet "Fu Calculation" included in Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E. The summation of all capacities of the N-S sticks at EL. 0', EL. 32', and EL. 64' are given in Row 6 of the columns listed above.

$$story_{VcEW} := \begin{pmatrix} 410364 \\ 282460 \\ 124110 \end{pmatrix} \cdot kip$$

Story capacities of the E-W shear walls.
Calculated in Columns AA, AC, and AE of Sheet "Fu Calculation" included in Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E. The summation of all capacities of the E-W sticks at EL. 0', EL. 32', and EL. 64' are given in Row 6 of the columns listed above.

Step 2: Determine Structural Frequency and Structural Damping

Per the seismic modal analysis performed in Ref. 2.2.5, the structural frequencies of the CRCF are given as follows:

$Freq_{NS} := 5.84Hz$ Structural frequency in the N-S direction from Table 7 of Ref. 2.2.5 - BDBGM 100' Upper Bound soil spring case

$Freq_{EW} := 5.77Hz$ Structural frequency in the E-W direction from Table 7 of Ref. 2.2.5 - BDBGM 100' Upper Bound soil spring case

Also, 10% structural damping was considered in the BDBGM seismic analysis.

$\delta := 10\%$ Structural damping used in the BDBGM seismic analysis of Ref. 2.2.5

Step 3: Compute the elastic Capacity/Demand ratios for each story under the BDBGM loads

$$C/D_{NS} := \overrightarrow{\frac{story_{VcNS}}{story_{VrNS}}} \qquad C/D_{NS} = \begin{pmatrix} 1.38 \\ 2.89 \\ 4.03 \end{pmatrix}$$

C/D ratios for the N-S shear walls

$$C/D_{EW} := \overrightarrow{\frac{story_{VcEW}}{story_{VrEW}}} \qquad C/D_{EW} = \begin{pmatrix} 1.64 \\ 2.17 \\ 3.13 \end{pmatrix}$$

C/D ratios for the E-W shear walls

Step 4: Check for Weak Story

Per ASCE 43-05 Section 5.1.2.1 (Ref. 2.2.6), if the C/D ratio of a given story is less than 80% of the story above, then the structure has a weak story.

$$min_{1stNS} := 0.80 \cdot C/D_{NS_2} \qquad min_{1stNS} = 2.31$$

Minimum 1st story C/D for N-S walls

$$min_{1stEW} := 0.80 \cdot C/D_{EW_2} \qquad min_{1stEW} = 1.73$$

Minimum 1st story C/D for E-W walls

$$weak_{1stNS} := if\left(C/D_{NS_1} < min_{1stNS}, \text{"There is a Weak Story in N-S direction"}, \text{"No Weak Story"}\right)$$

$$weak_{1stEW} := if\left(C/D_{EW_1} < min_{1stEW}, \text{"There is a Weak Story in E-W direction"}, \text{"No Weak Story"}\right)$$

$$weak_{1stNS} = \text{"There is a Weak Story in N-S direction"}$$

$$weak_{1stEW} = \text{"There is a Weak Story in E-W direction"}$$

Therefore, the inelastic energy absorption factors must be reduced from those listed in Table 5-1 of Ref. 2.2.6. The Effective Frequency Method is used to estimate the inelastic energy absorption factors for the CRCF.

Step 5: Determine Elastic Displaced Shape at Onset of Yielding (δe)

According to the information in Step 3, the lowest C/D ratio are for the 1st story (1.38 for the N-S walls and 1.64 for the E-W walls). Therefore, yielding will initially occur in the 1st story walls and the elastic displaced shape at the onset of yielding is given by the following:

Use the 1st story C/D to calculate δe since this corresponds to the first element which reaches yield

$$\delta e_{NS} := \overrightarrow{\left(C/D_{NS_1} \cdot story_{\Delta NS}\right)} \qquad \delta e_{NS} = \begin{pmatrix} 0.130 \\ 0.205 \\ 0.278 \end{pmatrix} in$$

Elastic displaced shape in the N-S direction at the onset of yielding in the first story

$$\delta e_{EW} := \overrightarrow{\left(C/D_{EW_1} \cdot story_{\Delta EW}\right)} \qquad \delta e_{EW} = \begin{pmatrix} 0.134 \\ 0.236 \\ 0.330 \end{pmatrix} in$$

Elastic displaced shape in the E-W direction at the onset of yielding in the first story

Step 6: Estimate Inelastic Deformed Shape

According to Table 5-2 of Ref. 2.2.6, a permissible total story distortion for a shear controlled shear wall (hw/lw < 2.0) at Limit State A is 0.75%.

$$drift := 0.0075$$

Allowable drift at Limit State A

The inelastic displacement of the 1st story (H = 32 feet) is then given by:

$\delta_{T1} := \text{drift} \cdot 32\text{ft}$ $\quad \delta_{T1} = 2.880\,\text{in}$ Allowable inelastic displacement of the 1st story

A slightly conservative estimate of the inelastic deformed shape may be obtained by considering that all of the nonlinear drift occurs in the story with the lowest Capacity/Demand ratio (the 1st story) and the remaining stories maintain the same differential drifts determined in Step 5.

Differential drifts: $\Delta_{2NS} := \delta e_{NS_2} - \delta e_{NS_1}$ $\quad \Delta_{2NS} = 0.076\,\text{in}$ N-S differential drift of 2nd story

$\Delta_{3NS} := \delta e_{NS_3} - \delta e_{NS_2}$ $\quad \Delta_{3NS} = 0.073\,\text{in}$ N-S differential drift of 3rd story

$\Delta_{2EW} := \delta e_{EW_2} - \delta e_{EW_1}$ $\quad \Delta_{2EW} = 0.102\,\text{in}$ E-W differential drift of 2nd story

$\Delta_{3EW} := \delta e_{EW_3} - \delta e_{EW_2}$ $\quad \Delta_{3EW} = 0.093\,\text{in}$ E-W differential drift of 3rd story

Inelastic deformed shape:

$$\delta_{TNS} := \begin{pmatrix} \delta_{T1} \\ \delta_{T1} + \Delta_{2NS} \\ \delta_{T1} + \Delta_{2NS} + \Delta_{3NS} \end{pmatrix} \qquad \delta_{TNS} = \begin{pmatrix} 2.88 \\ 2.96 \\ 3.03 \end{pmatrix} \text{in}$$

Inelastic deformed shape in the N-S direction

$$\delta_{TEW} := \begin{pmatrix} \delta_{T1} \\ \delta_{T1} + \Delta_{2EW} \\ \delta_{T1} + \Delta_{2EW} + \Delta_{3EW} \end{pmatrix} \qquad \delta_{TEW} = \begin{pmatrix} 2.88 \\ 2.98 \\ 3.08 \end{pmatrix} \text{in}$$

Inelastic deformed shape in the E-W direction

Step 7: Estimate System Ductility
According to equation 6-1 in Ref. 2.2.43, the system ductility can be estimated using the following equation:

$$\mu := \frac{\sum_{i=1}^{n} \left(W_i \cdot \delta_{T.i}\right)}{\sum_{i=1}^{n} \left(W_i \cdot \delta_{e.i}\right)}$$

Where:
n = total number of stories
Wi = inertial weights applied at story i
δ_{Ti} = total drift at story i corresponding to the permissible total story distortion occurring in the critical story (1st story)
δ_{ei} = elastic drift at story i corresponding to an elastic Capacity/Demand ratio of unity for the critical story (1st story)
μ = total system ductility

Inertial Weights:

$W_{1st} := 96930\text{kip}$ Inertial weight for 1st story = Weight at EL. 32' from Ref. 2.2.5 Attachment J (Mass x 32.2 ft/sec^2)

$W_{2nd} := 60807 \cdot \text{kip}$ Inertial weight for 2nd story = Weight at EL. 64' from Ref. 2.2.5. Attachment J (Mass x 32.2 ft/sec^2)

$W_{3rd} := 22424\text{kip}$ Inertial weight for 3rd story = Weight at EL. 72' plus weight at EL. 100' from Ref. 2.2.5 Attachment J (Mass x 32.2 ft/sec^2). The stick elements from EL. 64' to EL.

72' feet exist only in a small portion of the CRCF. Therefore, these stick elements are not considered to constitute a separate story and the weight lumped at EL. 72' in the stick model (Ref. 2.2.5) is incorporated into the weight at EL. 100' for this evaluation.

$$\mu_{NS} := \frac{W_{1st} \cdot \delta_{TNS_1} + W_{2nd} \cdot \delta_{TNS_2} + W_{3rd} \cdot \delta_{TNS_3}}{W_{1st} \cdot \delta e_{NS_1} + W_{2nd} \cdot \delta e_{NS_2} + W_{3rd} \cdot \delta e_{NS_3}} \qquad \mu_{NS} = 16.84$$

N-S system ductility

$$\mu_{EW} := \frac{W_{1st} \cdot \delta_{TEW_1} + W_{2nd} \cdot \delta_{TEW_2} + W_{3rd} \cdot \delta_{TEW_3}}{W_{1st} \cdot \delta e_{EW_1} + W_{2nd} \cdot \delta e_{EW_2} + W_{3rd} \cdot \delta e_{EW_3}} \qquad \mu_{EW} = 15.22$$

E-W system ductility

Step 8: Estimate Fμ using the Effective Frequency Method

Considering that the force-deflection relationship on initial loading is elasto-perfectly plastic with an ultimate capacity equal to the story capacities defined in Step 1 then

The ratio of secant to elastic frequency is given by:

$$fs/f(\mu) := \sqrt{\frac{1}{\mu}}$$

(Equation M-6 in Ref. 2.2.43)

The ratio of effective frequency to elastic frequency is then given by:

$$fe/f(\mu, A) := (1 - A) + A \cdot fs/f(\mu)$$

(Equation M-7 in Ref. 2.2.43)

where:

$$A(C_F, \mu) := \text{if}\left[C_F \cdot (1 - fs/f(\mu)) \le 0.85, C_F \cdot (1 - fs/f(\mu)), 0.85\right]$$

(Equation M-7 in Ref. 2.2.43)

Per Assumption 3.2.4, the strong duration of the BDBDGM ground motion is assumed to be greater than 15 seconds. Therefore, per Table 4-2 of reference 2.2.42, the frequency shift coefficient, Cf, is 2.7.

$$C_F := 2.7$$

The effective damping may be estimated from:

$$\beta_e(\mu, \beta, \beta_H, A) := \left(\frac{fs/f(\mu)}{fe/f(\mu, A)}\right)^2 \cdot (\beta + \beta_H)$$

(Equation M-9 in Ref. 2.2.43)

where:

$\beta := \delta \qquad \beta = 0.10$ elastic damping = structural damping used in the BDBGM seismic analysis of Ref. 2.2.5

β_H is the pinched hysteric damping which, for shear wall structures, can be approximated by

$$\beta_H(\mu) := 11\% \cdot (1 - fs/f(\mu))$$

(Equation M-10 in Ref. 2.2.43)

The inelastic energy absorption factor is then given by:

$$F_{\mu} := \left(\frac{f_e}{f_s}\right)^2 \cdot \frac{S_A(f,\beta)}{S_A(f_e,\beta_e)}$$ (Equation M-13 in Ref. 2.2.43)

where:
f_e = effective frequency
f_s = secant frequency
$S_A(f,\beta)$ = spectral acceleration at the elastic structural frequency (f) and the elastic structural damping (β)
$S_A(f_e,\beta_e)$ = spectral acceleration at the effective frequency (f_e) and the effective structural damping (β_e)

Step 9: $F\mu$ Calculations for N-S Shear Walls

$f := Freq_{NS}$ $f = 5.84\,Hz$ Elastic Structural frequency in N-S direction

$\beta = 10.0\%$ Elastic Structural damping

$\mu_{NS} = 16.84$ System ductility in the N-S direction

Secant frequency:	$f_{sNS} := fs/f(\mu_{NS})\cdot f$	$f_{sNS} = 1.42\,Hz$
A:	$A_{NS} := A(C_F, \mu_{NS})$	$A_{NS} = 0.85$
Effective frequency:	$f_{eNS} := fe/f(\mu_{NS}, A_{NS})\cdot f$	$f_{eNS} = 2.09\,Hz$
Hysteric damping:	$\beta h_{NS} := \beta_H(\mu_{NS})$	$\beta h_{NS} = 8.32\%$
Effective damping:	$\beta e_{NS} := \beta_e(\mu_{NS}, \beta, \beta h_{NS}, A_{NS})$	$\beta e_{NS} = 8.53\%$

Spectral Acceleration at effective frequency and effective damping
The BDBGM (10,000 APE) damped design spectra in Ref. 2.2.31 are given for 0.5%, 1%, 2%, 3%, 5%, 7%, 10%, 15%, and 20% for a range of frequencies between 0.1 Hz and 100 Hz.

In order to obtain the spectral acceleration at the effective frequency (fe) and the effective damping (βe), the following procedure is used -

- Interpolate between the 2.009 Hz and 2.984 Hz spectral accelerations to obtain the spectral accelerations at 2.09 Hz (the effective frequency) for all damping values.
- Plot the spectral accelerations at 2.09 Hz (y-axis) versus damping (x-axis)
- Fit an equation to the plotted acceleration vs. damping values using Mathcad built-in equation-fitting functions.
- Determine the equation that best fits the spectral acceleration vs. damping data.
- Using the equation generated above, determine the spectral acceleration at the effective damping 8.53%.

$$\text{damping} := \begin{pmatrix} 0.5 \\ 1 \\ 2 \\ 3 \\ 5 \\ 7 \\ 10 \\ 15 \\ 20 \end{pmatrix} \cdot \%$$

Damping values at which the BDBGM horizontal spectra acceleration curves are provided in Ref. 2.2.31

$$Sa_{2.009Hz} := \begin{pmatrix} 3.3925 \\ 2.8626 \\ 2.3328 \\ 2.0229 \\ 1.6302 \\ 1.3908 \\ 1.1949 \\ 0.9722 \\ 0.8142 \end{pmatrix}$$

BDBGM horizontal spectral accelerations at 2.009 Hz for 0.5%, 1%, 2%, 3%, 5%, 7%, 10%, 15%, and 20% damping given in Ref. 2.2.31

$$Sa_{2.984Hz} := \begin{pmatrix} 4.2016 \\ 3.5044 \\ 2.8072 \\ 2.3994 \\ 1.9339 \\ 1.6479 \\ 1.4222 \\ 1.1657 \\ 0.9838 \end{pmatrix}$$

BDBGM horizontal spectral accelerations at 2.984 Hz for 0.5%, 1%, 2%, 3%, 5%, 7%, 10%, 15%, and 20% damping given in Ref. 2.2.31

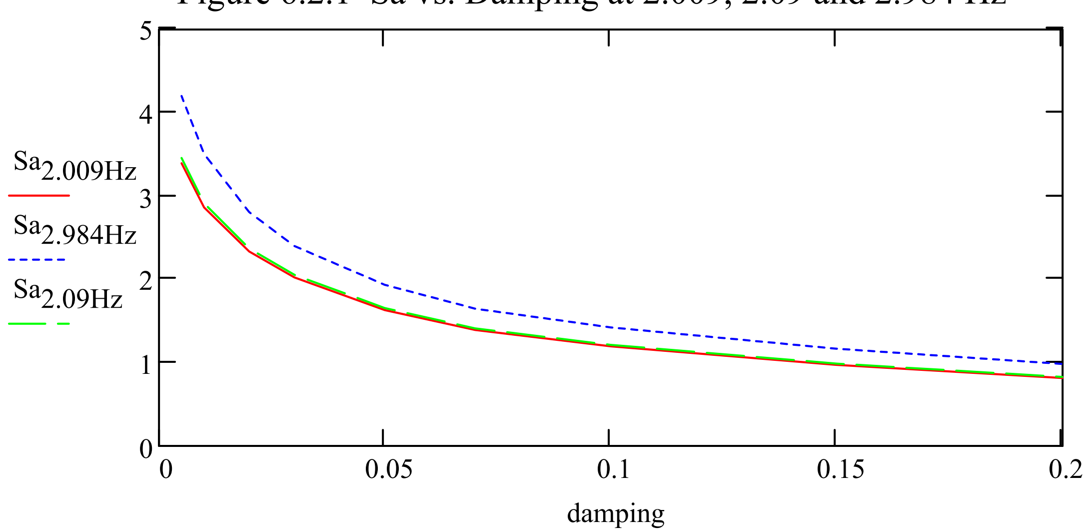
$$Sa_{2.09Hz} := \left| \begin{array}{l} x \leftarrow \begin{pmatrix} 2.009 \\ 2.984 \end{pmatrix} \\ z \leftarrow \dfrac{f_{eNS}}{Hz} \\ \text{for } i \in 1 .. \text{rows}(damping) \\ \quad \left| \begin{array}{l} y \leftarrow \begin{pmatrix} Sa_{2.009Hz_i} \\ Sa_{2.984Hz_i} \end{pmatrix} \\ result_i \leftarrow \text{linterp}(x, y, z) \end{array} \right. \\ result \end{array} \right.$$

Description:
This loop linearly interpolates between 2.009 Hz and 2.984 Hz to determine the Sa values at 2.09 Hz at all damping values.

$$Sa_{2.09Hz} = \begin{pmatrix} 3.4561 \\ 2.9131 \\ 2.3701 \\ 2.0525 \\ 1.6541 \\ 1.4110 \\ 1.2128 \\ 0.9874 \\ 0.8275 \end{pmatrix}$$

Spectral acceleration values at 2.09 Hz for the damping values

Figure 6.2.1 Sa vs. Damping at 2.009, 2.09 and 2.984 Hz



Fit a curve to the spectral acceleration vs. damping curve at 2.09Hz
Fit a curve using the linear, exponential, and power regression and then determine the equation that best fits the data.

Linear Regression: line function returns values in the form of a + bx

$$linear_{values} := line(damping, Sa_{2.09Hz}) \qquad linear_{values} = \begin{pmatrix} 2.6866 \\ -11.4873 \end{pmatrix}$$

$$linear_{Sa}(x) := linear_{values_1} + linear_{values_2} \cdot x$$

*Exponential Regression: expfit function returns values in the form of a*exp^{bx} + c*

$$exp_{values} := expfit(damping, Sa_{2.09Hz}) \qquad exp_{values} = \begin{pmatrix} 2.7273 \\ -28.7873 \\ 0.9540 \end{pmatrix}$$

$$exp_{Sa}(x) := exp_{values_1} \cdot e^{exp_{values_2} \cdot x} + exp_{values_3}$$

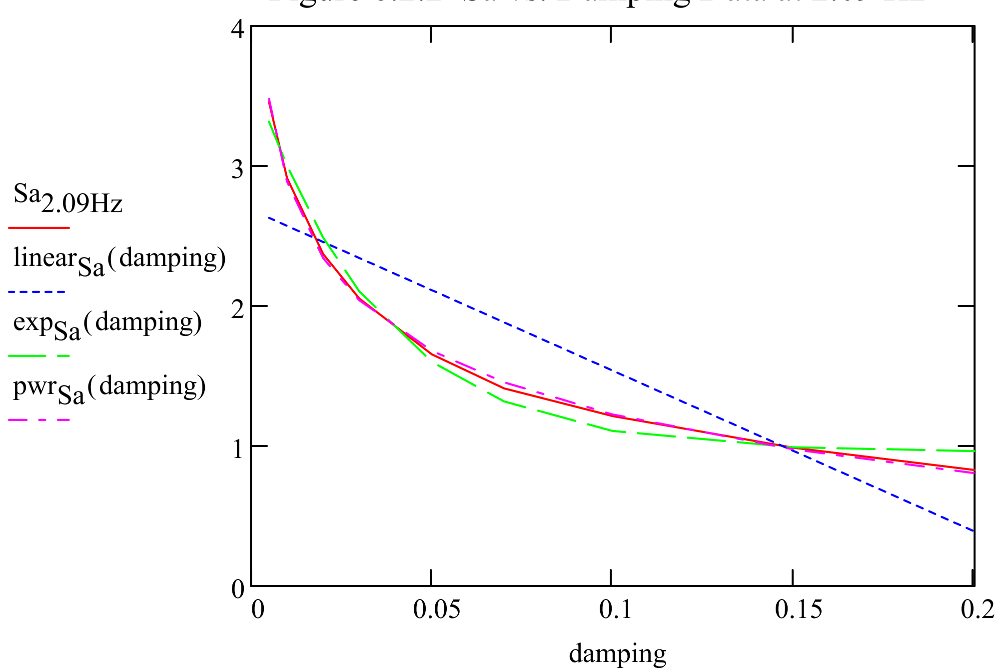
*Power Regression: pwrfit function returns values in the form of a*x^b + c*

$$guess := \begin{pmatrix} 0.2 \\ 2.0 \\ 0 \end{pmatrix}$$ initial guess values for a, b, and c for the power regression function

$$pwr_{values} := pwrfit(damping, Sa_{2.09Hz}, guess) \qquad pwr_{values} = \begin{pmatrix} 4.2164 \\ -0.1148 \\ -4.2676 \end{pmatrix}$$

$$pwr_{Sa}(x) := pwr_{values_1} \cdot x^{pwr_{values_2}} + pwr_{values_3}$$

Figure 6.2.2 Sa vs. Damping Data at 2.09 Hz



From Figure 6.2.2, the power regression equation best fits the spectral acceleration vs. damping data at 2.09 Hz. Therefore, this equation will be used to determine the spectral acceleration at 2.09 Hz and the effective damping (8.53 %).

$Sa(\text{damping}) := pwr_{Sa}(\text{damping})$ Spectral acceleration at 2.09 Hz as a function of damping

$\beta e_{NS} = 8.53\%$

$Sa_{effective} := Sa(\beta e_{NS})$ $\quad Sa_{effective} = 1.33$ Spectral acceleration at 2.09 Hz and the effective damping

$F\mu$ Calculation for N-S shear walls

$f = 5.84\,Hz$ Elastic frequency in the N-S direction

Determine BDBGM spectral acceleration at the elastic frequency of 5.84Hz and 10% damping by linearly interpolating between the 4.977 Hz and 5.995 Hz spectral values on the 10% damped curve given in Ref. 2.2.31.

$x := \begin{pmatrix} 4.977 \\ 5.995 \end{pmatrix}$ $\quad y := \begin{pmatrix} 1.7245 \\ 1.7519 \end{pmatrix}$ $\quad x_{loc} := \dfrac{f}{Hz}$ $\quad x_{loc} = 5.84$

$Sa_{elastic} := \text{linterp}(x, y, x_{loc})$ $\quad Sa_{elastic} = 1.75$

$Sa_{elastic} = 1.75$ BDBGM spectral acceleration at the elastic frequency of 5.84 Hz and 10% damping

$Sa_{effective} = 1.33$ BDBGM spectral acceleration at the effective frequency of 2.09 Hz and effective damping of 8.53%

$f_{eNS} = 2.09\,Hz$ Effective frequency in the N-S direction

$f_{sNS} = 1.42\,Hz$ Secant frequency in the N-S direction

$F_{\mu NS} := \left(\dfrac{f_{eNS}}{f_{sNS}}\right)^2 \cdot \dfrac{Sa_{elastic}}{Sa_{effective}}$ $\quad F_{\mu NS} = 2.83$ $F\mu$ factor for the N-S shear walls

Step 10: $F\mu$ Calculations for E-W Shear Walls

$f := Freq_{EW}$ $\quad f = 5.77\,Hz$ Elastic Structural frequency in E-W direction

$\beta = 10.0\%$ Elastic Structural damping

$\mu_{EW} = 15.22$ System ductility in the E-W direction

Secant frequency: $f_{sEW} := fs/f(\mu_{EW}) \cdot f$ $\quad f_{sEW} = 1.48\,Hz$

A: $A_{EW} := A(C_F, \mu_{EW})$ $\quad A_{EW} = 0.85$

Effective frequency: $f_{eEW} := fe/f\left(\mu_{EW}, A_{EW}\right)\cdot f$ $\quad f_{eEW} = 2.12\,\text{Hz}$

Hysteric damping: $\beta h_{EW} := \beta_H\left(\mu_{EW}\right)$ $\quad \beta h_{EW} = 8.18\,\%$

Effective damping: $\beta e_{EW} := \beta_e\left(\mu_{EW}, \beta, \beta h_{EW}, A_{EW}\right)$ $\quad \beta e_{EW} = 8.83\,\%$

Spectral Acceleration at effective frequency and effective damping
The BDBGM (10,000 APE) damped design spectra in Ref. 2.2.31 are given for 0.5%, 1%, 2%, 3%, 5%, 7%, 10%, 15%, and 20% for a range of frequencies between 0.1 Hz and 100 Hz.

In order to obtain the spectral acceleration at the effective frequency (fe) and the effective damping (βe), the following procedure is used -

- Interpolate between the 2.009 Hz and 2.984 Hz spectral accelerations to obtain the spectral accelerations at 2.12 Hz (the effective frequency) for all damping values.
- Plot the spectral accelerations at 2.12 Hz (y-axis) versus damping (x-axis)
- Fit an equation to the plotted acceleration vs. damping values using Mathcad built-in equation-fitting functions.
- Determine the equation that best fits the spectral acceleration vs. damping data.
- Using the equation generated above, determine the spectral acceleration at the effective damping 8.83%.

$$\text{damping} = \begin{pmatrix} 0.01 \\ 0.01 \\ 0.02 \\ 0.03 \\ 0.05 \\ 0.07 \\ 0.10 \\ 0.15 \\ 0.20 \end{pmatrix}$$

Damping values at which the BDBGM horizontal spectra acceleration curves are provided in Ref. 2.2.31

$$Sa_{2.009Hz} = \begin{pmatrix} 3.3925 \\ 2.8626 \\ 2.3328 \\ 2.0229 \\ 1.6302 \\ 1.3908 \\ 1.1949 \\ 0.9722 \\ 0.8142 \end{pmatrix}$$

BDBGM horizontal spectral accelerations at 2.009 Hz for 0.5%, 1%, 2%, 3%, 5%, 7%, 10%, 15%, and 20% damping given in Ref. 2.2.31

$$Sa_{2.984Hz} = \begin{pmatrix} 4.2016 \\ 3.5044 \\ 2.8072 \\ 2.3994 \\ 1.9339 \\ 1.6479 \\ 1.4222 \\ 1.1657 \\ 0.9838 \end{pmatrix}$$

BDBGM horizontal spectral accelerations at 2.984 Hz for 0.5%, 1%, 2%, 3%, 5%, 7%, 10%, 15%, and 20% damping given in Ref. 2.2.31

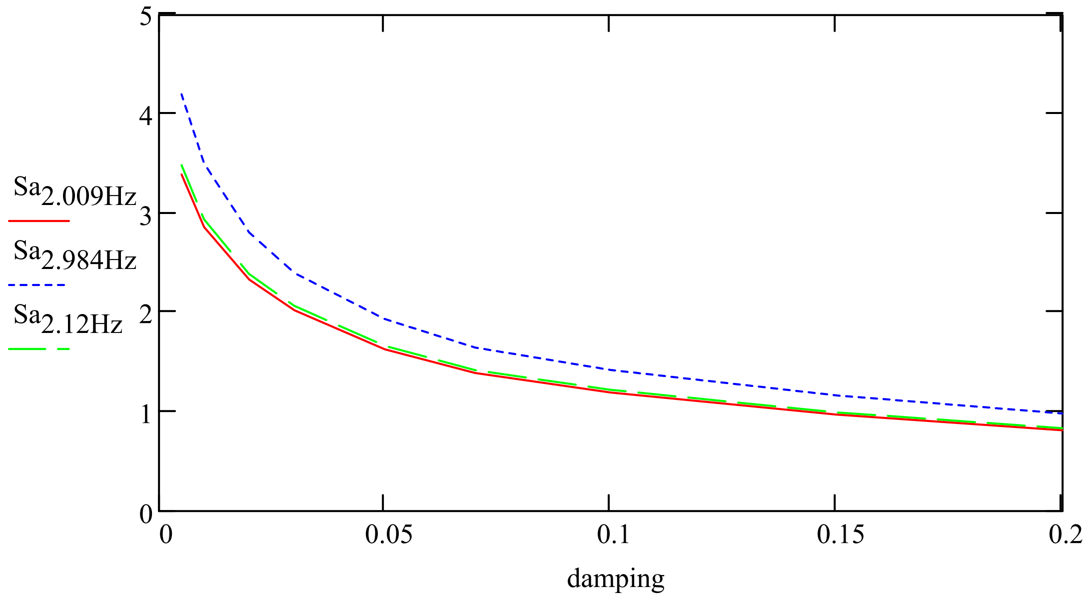
$$Sa_{2.12Hz} := \left| \begin{array}{l} x \leftarrow \begin{pmatrix} 2.009 \\ 2.984 \end{pmatrix} \\ z \leftarrow \dfrac{f_{eEW}}{Hz} \\ \text{for } i \in 1..\text{rows(damping)} \\ \quad \left| \begin{array}{l} y \leftarrow \begin{pmatrix} Sa_{2.009Hz_i} \\ Sa_{2.984Hz_i} \end{pmatrix} \\ \text{result}_i \leftarrow \text{linterp}(x, y, z) \end{array} \right. \\ \text{result} \end{array} \right.$$

Description:
This loop linearly interpolates between 2.009 Hz and 2.984 Hz to determine the Sa values at 2.12 Hz at all damping values.

$$Sa_{2.12Hz} = \begin{pmatrix} 3.4869 \\ 2.9375 \\ 2.3882 \\ 2.0668 \\ 1.6656 \\ 1.4208 \\ 1.2214 \\ 0.9948 \\ 0.8340 \end{pmatrix}$$

Spectral acceleration values at 2.12 Hz for the damping values

Figure 6.2.3 Sa vs. Damping at 2.009, 2.12 and 2.984 Hz



Fit a curve to the spectral acceleration vs. damping curve at 2.12Hz

Fit a curve using the linear, exponential, and power regression and then determine the equation that best fits the data.

Linear Regression: line function returns values in the form of a + bx

$$linear_{values} := line(damping, Sa_{2.12Hz}) \qquad linear_{values} = \begin{pmatrix} 2.7080 \\ -11.5848 \end{pmatrix}$$

$$linear_{Sa}(x) := linear_{values_1} + linear_{values_2} \cdot x$$

*Exponential Regression: expfit function returns values in the form of a*exp^{bx} + c*

$$exp_{values} := expfit(damping, Sa_{2.12Hz}) \qquad exp_{values} = \begin{pmatrix} 2.7538 \\ -28.9214 \\ 0.9623 \end{pmatrix}$$

$$exp_{Sa}(x) := exp_{values_1} \cdot e^{exp_{values_2} \cdot x} + exp_{values_3}$$

*Power Regression: pwrfit function returns values in the form of a*x^{b} + c*

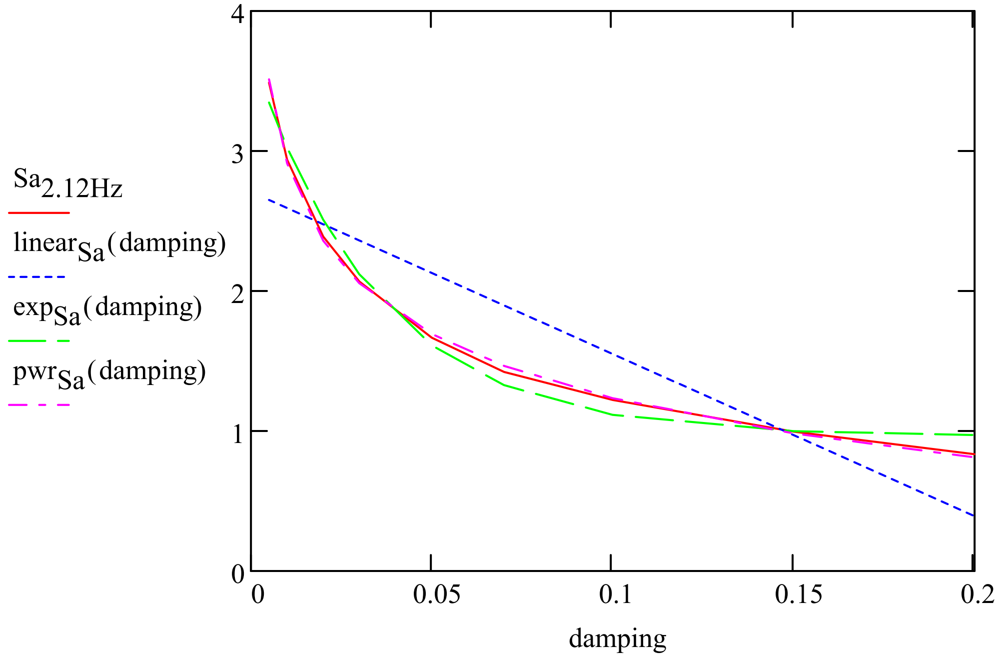
$$guess := \begin{pmatrix} 0.2 \\ 2.0 \\ 0 \end{pmatrix}$$

guess values for a, b, and c for the power regression function

$$pwr_{values} := pwrfit(damping, Sa_{2.12Hz}, guess \qquad pwr_{values} = \begin{pmatrix} 4.1343 \\ -0.1172 \\ -4.1816 \end{pmatrix}$$

$$pwr_{Sa}(x) := pwr_{values_1} \cdot x^{pwr_{values_2}} + pwr_{values_3}$$

Figure 6.2.4 Sa vs. Damping Data at 2.12 Hz



From the above plot, the power regression equation best fits the spectral acceleration vs. damping data at 2.12 Hz. Therefore, this equation will be used to determine the spectral acceleration at 2.12 Hz and the effective damping (8.83 %).

$Sa(damping) := pwr_{Sa}(damping)$ Spectral acceleration at 2.12 Hz as a function of damping

$\beta e_{EW} = 8.83\,\%$

$Sa_{effective} := Sa(\beta e_{EW})$ $Sa_{effective} = 1.31$ Spectral acceleration at 2.12 Hz and the effective damping

$F\mu$ Calculation for E-W shear walls

$f = 5.77\,Hz$ Elastic frequency in the E-W direction

Determine BDBGM spectral acceleration at the elastic frequency of 5.77Hz and 10% damping by linearly interpolating between the 4.977 Hz and 5.995 Hz spectral values on the 10% damped curve given in Ref. 2.2.31.

$x := \begin{pmatrix} 4.977 \\ 5.995 \end{pmatrix}$ $y := \begin{pmatrix} 1.7245 \\ 1.7519 \end{pmatrix}$ $x_{loc} := \dfrac{f}{Hz}$ $x_{loc} = 5.77$

$Sa_{elastic} := linterp(x, y, x_{loc})$ $Sa_{elastic} = 1.75$

$Sa_{elastic} = 1.75$ BDBGM spectral acceleration at the elastic frequency of 5.77 Hz and 10% damping

$Sa_{effective} = 1.31$ BDBGM spectral acceleration at the effective frequency of 2.12 Hz and effective damping of 8.83%

$f_{eEW} = 2.12\,Hz$ Effective frequency in the N-S direction

$f_{sEW} = 1.48\,Hz$ Secant frequency in the N-S direction

$$F_{\mu EW} := \left(\frac{f_{eEW}}{f_{sEW}}\right)^2 \cdot \frac{Sa_{elastic}}{Sa_{effective}}$$

$F_{\mu EW} = 2.74$ Fμ factor for the E-W shear walls

Step 11: Fu Calculation Summary

Limit State A Fμ per Equation 5-2(a) of Ref. 2.2.6 to account for weak story

$F\mu_{1st} = 1.50$ Fμ considering the 1st story is the highest weak story (k = 1)

$F\mu_{2nd} = 1.33$ Fμ considering the 2nd story is the highest weak story (k = 2)

Limit State A Fμ per Effective Frequency Method (accounts for weak story by isolating all inelastic deformation in 1st story)

N-S Shear Walls: $F_{\mu NS} = 2.83$

E-W Shear Walls: $F_{\mu EW} = 2.74$

Discussion
The above calculations give Fμ factors for Limit State A between 1.33 and 2.83. The following observations are made regarding the Fμ factors determined above.

1. Based on the direction of ASCE 43-05, both methods for calculating Fμ are acceptable.
2. Based on the discussion in C5.1.2.3 of Ref. 2.2.6 and Ref. 2.2.48, the Fμ factors given in ASCE 43-05 Table 5-1 are inherently conservative for all Limit States.
3. The Fμ factors tabulated in ASCE 43-05 Table 5-1 were derived from a series of literature reviews that aimed to develop force reduction factors (Fμ factors) (Ref. 2.2.48). One of the methods researched was the Effective Frequency Method.
4. The Effective Frequency Method accounts for weak story effects by concentrating all of the inelastic deformation in the story with the lowest C/D ratio.
5. Based on (2) and (3), using an Fμ = 2.0 for a shear wall at Limit State A is a conservative measure of the inelastic energy absorption that accounts for weak story effects.
6. Equation 5-2(a) of Ref. 2.2.6 conservatively assumes that all the inelastic displacement occurs in the highest weak story. This assumption does not account for the fact that the story with the lowest C/D ratio (typically the 1st story) will begin to yield before the highest weak story (2nd story in the CRCF). Therefore, an Fμ = 1.50 reasonably accounts for weak story effects per Equation 5-2(a) of Ref. 2.2.6.

An Fμ = 1.75 is used in the in-plane shear HCLPF evaluation of the CRCF at Limit State A. This Fμ value is a reasonable inelastic energy absorption factor for the CRCF shear walls that accounts for (1) the inherit conservatism of the Fμ factors in Table 5-1 of Ref. 2.2.6 and (2) the need to account for weak story effects in the CRCF.

6.2.6 In-Plane Shear HCLPF Calculations

The Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" included in Attachment E contains the in-plane shear HCLPF calculations for the CRCF. The majority of calculations are shown on sheet "Frag. Shear Calculation", while a summary of the HCLPF calculations at each floor story for each wall are calculated and shown on sheet "In-Plane Shear Frag Summary". The following sections verify the calculations on these Excel sheets through an independent calculation.

6.2.6.1 Independent HCLPF Capacity Calculation

To show the validity of the Excel calculations, an independent calculation of the HCLPF capacities for several stick elements is performed. Five stick elements are selected with different hw/lw values to ensure that all branches of the in-plane shear capacity calculation are covered. The stick elements selected for the sample calculation are as follows:

Table 6.2.3 Wall Design Parameters for Selected Stick Elements

Stick ID	h_w (ft)	l_w (ft)	h_w / l_w
2A.2	8.0	21.5	0.37
5A.1	32.0	48.0	0.67
G2.2	32.0	29.9	1.07
7A.2	32.0	19.0	1.68
4A.2	32.0	12.0	2.67

* Source - sheet "Frag. Shear Calculation" in file "CRCF-Fragility-In-Plane Shear Wall.xls" Attachment E

The Excel calculations are shown followed by the independent calculations for each stick element.

Table 6.2.4 Sample Output from Sheet "*Frag. Shear Calculation" of Excel file "CRCF – Fragility – In-Plane Shear Wall.xls"*

Stick Element Properties

Stick ID	Joint I	Joint J	Length	X Centroid	Y Centroid	Z Centroid
2A.2	33	115	8.00	49.00	90.75	4.00
5A.1	42	123	10.00	153.00	34.00	5.00
G2.2	205	306	12.00	180.13	82.00	38.00
7A.2	51	257	32.00	237.00	168.50	16.00
4A.2	40	242	32.00	137.00	129.00	16.00

Wall Design Parameters

							DL+ LL - BDBGM
Stick ID	hw (ft)	lw (ft)	H (ft)	thick (ft)	As_v (in^2/ft/face)	As_h (in^2/ft/face)	Na (kips)
2A.2	8.00	21.50	32.00	4.00	1.56	1.56	-1,148.38
5A.1	32.00	48.00	32.00	4.00	1.56	1.56	-2,466.79
G2.2	32.00	29.92	32.00	4.00	1.56	1.56	-498.62
7A.2	32.00	19.00	32.00	4.00	1.56	1.56	-28.76
4A.2	32.00	12.00	32.00	4.00	1.56	1.56	-26.76

f'c	5,500	psi
fy	60,000	psi
PGA	0.9138	g

Source - "*Frag. Shear Calculation" of Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" in Attachment E*

CRCF In-Plane Shear HCLPF Independent Calculations

$ORIGIN := 1$ Set the array origin to 1

$f_c := 5500psi$ Concrete compressive strength per section 6.2.4.1 of this calculation

$f_y := 60000psi$ Steel yield strength (Ref. 2.2.1, Section 4.2.11.6.2)

$PGA := 0.9138g$ BDBGM peak horizontal ground acceleration (Ref. 2.2.31)

Stick Properties

$$Stick\_ID := \begin{pmatrix} "2A.2" \\ "5A.1" \\ "G2.2" \\ "7A.2" \\ "4A.2" \end{pmatrix} \qquad Stick\_ID_1 = "2A.2"$$

$$h_w := \begin{pmatrix} 8.00 \\ 32.00 \\ 32.00 \\ 32.00 \\ 32.00 \end{pmatrix} \cdot ft \qquad l_w := \begin{pmatrix} 21.50 \\ 48.00 \\ 29.92 \\ 19.00 \\ 12.00 \end{pmatrix} \cdot ft \qquad \overrightarrow{\frac{h_w}{l_w}} = \begin{pmatrix} 0.37 \\ 0.67 \\ 1.07 \\ 1.68 \\ 2.67 \end{pmatrix}$$

$$t_w := \begin{pmatrix} 4 \\ 4 \\ 4 \\ 4 \\ 4 \end{pmatrix} \cdot ft \qquad A_{sv} := \begin{pmatrix} 1.56 \\ 1.56 \\ 1.56 \\ 1.56 \\ 1.56 \end{pmatrix} \cdot \frac{in^2}{ft} \qquad A_{sh} := \begin{pmatrix} 1.56 \\ 1.56 \\ 1.56 \\ 1.56 \\ 1.56 \end{pmatrix} \cdot \frac{in^2}{ft} \qquad N_a := \begin{pmatrix} -1148.38 \\ -2466.79 \\ -498.62 \\ -28.76 \\ -26.76 \end{pmatrix} \cdot kip$$

Table 6.2.5 Intermediate HCLPF Calculations for Selected Stick Elements

				$8\phi(f'c)^{0.5}$ (psi)	356			$20\phi(f'c)^{0.5}$ (psi)	1186.59			
				Concrete Capacity				Steel Capacity		Total Capacity		
				Vc (psi)				Vs (psi)		ϕVn = $C_{98\%}$ (kips)		
Stick ID	ρv	ρh	hw/lw	Barda - ASCE 43-05 Eq. 4-3	ACI 349-01 Eq. 11-8	A	B	Barda - ASCE 43-05 Eq. 4-3	ACI 349-01 Eq. 11-8	Barda - ASCE 43-05 Eq. 4-3	ACI 349-01 Eq. 11-8	ϕVn = $C_{98\%}$ (kips)
2A.2	0.0054	0.0054	0.37	624.61		1.00	0.00	325.00		5,645		5,645
5A.1	0.0054	0.0054	0.67	551.21		0.83	0.17	325.00		11,628		11,628
G2.2	0.0054	0.0054	1.07	464.71		0.43	0.57	325.00		6,533		6,533
7A.2	0.0054	0.0054	1.68	316.29		0.00	1.00	325.00		3,369		3,369
4A.2	0.0054	0.0054	2.67		147.18	0.00	1.00		325.00		1,567	1,567

These calculations are performed in sheet “Frag. Shear Calculation” of Excel file “CRCF – Fragility – In-Plane Shear Wall.xls” in Attachment E.

The following calculations verify the calculations given in Table 6.2.5.

HCLPF Calculations

Vertical Steel Reinforcement Ratio $\rho_v := \overrightarrow{\frac{2 \cdot A_{sv} \cdot 1ft}{t_w \cdot 1 \cdot ft}}$ $\rho_v = \begin{pmatrix} 0.0054 \\ 0.0054 \\ 0.0054 \\ 0.0054 \\ 0.0054 \end{pmatrix}$

Horizontal Steel Reinforcement Ratio $\rho_h := \overrightarrow{\frac{2 \cdot A_{sh} \cdot 1ft}{t_w \cdot 1 \cdot ft}}$ $\rho_h = \begin{pmatrix} 0.0054 \\ 0.0054 \\ 0.0054 \\ 0.0054 \\ 0.0054 \end{pmatrix}$

Concrete Shear Capacity

$$\nu_{cBarda}(i) := \left[8.3 \cdot \sqrt{f_c \cdot psi} - 3.4 \cdot \sqrt{f_c \cdot psi} \cdot \left(\frac{h_{w_i}}{l_{w_i}} - 0.5\right) + \frac{N_{a_i}}{4 \cdot l_{w_i} \cdot t_{w_i}}\right]$$

Equation 6.2.1 in Section 6.2.4 (applicable for h/l < 2)

Stick_ID$_3$ = "G2.2"

$\nu_{cBarda}(3) = 464.71$ psi **Identical to EXCEL Vc Barda capacity for Stick G2.2**

$$\nu_{c349}(i) := \left[2 \cdot \left(1 + \frac{N_{a_i}}{500 \cdot l_{w_i} \cdot t_{w_i}} \cdot \frac{1}{psi}\right) \cdot \sqrt{f_c \cdot psi}\right]$$

Equation 6.2.4 in Section 6.2.4 (applicable for h/l > 2)

$\nu_{c349}(5) = 147.18$ psi **Identical to EXCEL Vc ACI 349-01 capacity for Stick 4A.2**

Steel Shear Capacity

$$A(h,l) := \begin{cases} 1.0 & \text{if } \frac{h}{l} \le 0.5 \\ 0.0 & \text{if } \frac{h}{l} \ge 1.5 \\ \frac{h}{l}\cdot -1 + 1.5 & \text{otherwise} \end{cases}$$

$$B(h,l) := \begin{cases} 0.0 & \text{if } \frac{h}{l} \le 0.5 \\ 1.0 & \text{if } \frac{h}{l} \ge 1.5 \\ \frac{h}{l} - 0.5 & \text{otherwise} \end{cases}$$

In the Barda equation, the steel capacity is a function of the horizontal and vertical reinforcing ratios as functions of A and B. These equations are applicable for hw/lw < 2.0

$$A\left(h_{w_1}, l_{w_1}\right) = 1.00 \qquad B\left(h_{w_1}, l_{w_1}\right) = 0.00$$

$$A\left(h_{w_3}, l_{w_3}\right) = 0.43 \qquad B\left(h_{w_3}, l_{w_3}\right) = 0.57$$

$$\rho_{seBarda}(i) := \min\left(0.01, A\left(h_{w_i}, l_{w_i}\right)\cdot\rho_{v_i} + B\left(h_{w_i}, l_{w_i}\right)\cdot\rho_{h_i}\right)$$

$$\rho_{seBarda}(3) = 0.0054$$

Limit steel to 1% per ASCE 43-05 Section 4.2.3

$$\rho_{se349}(i) := \rho_{h_i}$$

$$\rho_{se349}(5) = 0.0054$$

$$V_{sBarda}(i) := \rho_{seBarda}(i)\cdot f_y$$

$$V_{s349}(i) := \rho_{se349}(i)\cdot f_y$$

In the ACI equations for steel capacdity the horizontal steel provides the capacity. It should be noted that ACI 349 requires the vertical steel to be greater than or equal to the horizontal steel. This equation is applicable for hw/lw > 2.0.

$$V_{sBarda}(3) = 325\ \text{psi}$$

Identical to EXCEL Vc Barda capacity for Stick G2.2

$$V_{s349}(5) = 325\ \text{psi}$$

Identical to EXCEL Vc ACI 349-01 capacity for Stick 4A.2

Total Shear Capacity

$$
\phi V_n := \left|
\begin{array}{l}
\text{for } i \in 1 .. \text{rows}(\text{Stick\_ID}) \\
\quad \left|
\begin{array}{l}
\text{if } \dfrac{h_{w_i}}{l_{w_i}} \le 2.0 \\
\quad \left|
\begin{array}{l}
a \leftarrow \nu_{cBarda}(i) \\
b \leftarrow V_{sBarda}(i) \\
v_{tot} \leftarrow \min\left(a + b, 20 \cdot \sqrt{f_c \cdot psi}\right) \\
Area \leftarrow 0.6 \cdot l_{w_i} \cdot t_{w_i} \\
\phi \leftarrow 0.80 \\
\phi V_{n_i} \leftarrow \phi \cdot v_{tot} \cdot Area
\end{array}
\right. \\
\text{otherwise} \\
\quad \left|
\begin{array}{l}
a \leftarrow \nu_{c349}(i) \\
b \leftarrow V_{s349}(i) \\
v_{tot} \leftarrow \min\left(a + b, 8 \cdot \sqrt{f_c \cdot psi}\right) \\
Area \leftarrow 0.8 \cdot l_{w_i} \cdot t_{w_i} \\
\phi \leftarrow 0.60 \\
\phi V_{n_i} \leftarrow \phi \cdot v_{tot} \cdot Area
\end{array}
\right.
\end{array}
\right. \\
\phi V_n
\end{array}
\right.
$$

$$\text{rows}(\text{Stick\_ID}) = 5$$

Description
This loop determines the shear capacity for each stick case. For hw/lw <= 2.0, the shear capacity is determined using the Barda equation. For hw/lw > 2.0, the shear capacity is determined using ACI 349-01 equations.

$$\phi V_n = \begin{pmatrix} 5645 \\ 11628 \\ 6533 \\ 3369 \\ 1567 \end{pmatrix} kip$$

All shear capacities are identical to those calculated in the EXCEL file

The following table is taken from calculations performed in Attachment E "CRCF – Fragility – In-Plane Shear Wall.xls" Sheet "Frag. Shear Calcualtion".

Table 6.2.6 HCLPF Calculations for Selected Stick Elements

		Vu_{BDBGM} > 0				
				Fμ (N-S)	1.75	
				Fμ (E-W)	1.75	
Stick ID	**In-Plane Direction**	**Vu_{NS}**	**Vu_{BDBGM}**	**Fs**	**Fμ**	**C_{HCLPF} (g)**
2A2	NS	-2.81	3691.80	1.53	1.75	2.44
5A1	NS	12.09	10629.48	1.09	1.75	1.75
G2.2	EW	-5.30	3850.64	1.70	1.75	2.71
7A2	NS	-0.18	1732.87	1.94	1.75	3.11
4A2	NS	-0.08	610.55	2.57	1.75	4.10

The following calculations verify the calculations given in Table 6.2.6.

Fs Calculation

Non-Seismic In-Plane Shear

$$V_{uNS} := \begin{pmatrix} -2.81 \\ 12.09 \\ -5.30 \\ -0.18 \\ -0.08 \end{pmatrix} \cdot kip$$

(Ref. 2.2.5 and Attachment E "CRCF - Fragility - In-Plane Shear Wall.xls" sheet ""Element Forces - Vn Calc.")

Seismic In-Plane Shear

$$V_{uBDBGM} := \begin{pmatrix} 3691.80 \\ 10629.48 \\ 3850.64 \\ 1732.87 \\ 610.55 \end{pmatrix} \cdot kip$$

(Ref. 2.2.5 and Attachment E "CRCF - Fragility - In-Plane Shear Wall.xls" sheet ""Element Forces - Vn Calc."

$$F_s := \overrightarrow{\frac{\phi V_n - \left|V_{uNS}\right|}{V_{uBDBGM}}} \qquad Stick\_ID = \begin{pmatrix} \text{"2A.2"} \\ \text{"5A.1"} \\ \text{"G2.2"} \\ \text{"7A.2"} \\ \text{"4A.2"} \end{pmatrix} \qquad F_s = \begin{pmatrix} 1.53 \\ 1.09 \\ 1.70 \\ 1.94 \\ 2.57 \end{pmatrix}$$

As calculated in Section 6.2.5, an F_{μ} = 1.75 is used for all stick elements.

$$F_{\mu} := \begin{pmatrix} 1.75 \\ 1.75 \\ 1.75 \\ 1.75 \\ 1.75 \end{pmatrix}$$

$$C_{HCLPF} := \overrightarrow{\left(F_s \cdot F_{\mu} \cdot PGA\right)}$$

$$Stick\_ID = \begin{pmatrix} \text{"2A.2"} \\ \text{"5A.1"} \\ \text{"G2.2"} \\ \text{"7A.2"} \\ \text{"4A.2"} \end{pmatrix} \qquad C_{HCLPF} = \begin{pmatrix} 2.44 \\ 1.75 \\ 2.71 \\ 3.11 \\ 4.10 \end{pmatrix} g$$

HCLPF capacities identical to those calculated in the Excel file

The preceeding calculations verify the formulations used in compute HCLPF capacities in Attachment E "CRCF-Fragility-In-Plane Shear Wall.xls" sheet "Frag. Shear Calculation".

6.2.6.2 Minimum HCLPF for Individual Stick Elements

Based on the results shown in Column AI on sheet "Frag. Shear Calculation" in file "CRCF – Fragility – In-Plane Shear Wall.xls" included in Attachment E, the minimum HCLPF for the CRCF shear walls is:

Minimum C_{HCLPF} = 1.75g (calculated for stick elements 5A.1 and 5A.4)

6.2.6.3 HCLPF Combinations for Different Levels

Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" included in Attachment E calculates the in-plane shear HCLPF for each stick element in the CRCF stick model. As stated in Step 1a of section 4.3.2, the Strength Margin Factor (F_S) may be re-calculated to reflect the redistribution of force between individual sticks comprising a given wall. This redistribution is permitted between individual wall piers because, as the rigid floor slab imposes a uniform displacement along the top of the entire wall, the individual piers will attract force based on their relative stiffness. As an individual wall pier begins to soften due to cracking, the force in that pier will redistribute to the adjacent piers in the wall.

Sheet "In-Plane Shear Frag Summary" in the Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" in Attachment E contains the calculations for F_s considering the redistribution of forces between each individual piers of each wall of the CRCF. These calculations are summarized below:

Level Definitions

Table 6.2.7 Level Definition Information

Excel Columns*	Value ID	Description
B	Level	This number represents the level number
C	Lower Node Bound	Lowest numerical value of the starting node for all stick elements at a given level in the CRCF stick model (Ref. 2.2.5)
D	Upper Node Bound	Highest numerical value of the starting node for all stick elements at a given level in the CRCF stick model (Ref. 2.2.5)
E	Z Bounds Lower (ft)	Lowest Z coordinate of a starting node point for all stick elements at a given level in the CRCF stick model (Ref. 2.2.5)
F	Z Bounds Upper (ft)	Highest Z coordinate of a starting node point for all stick elements at a given level in the CRCF stick model (Ref. 2.2.5)
G	Level	A repeat of the value contained in Column B used for the Excel VLOOKUP function contained in Column K

* Source - Sheet "In-Plane Shear Frag Summary" in Excel file "CRCF – Fragility – In-Plane Shear.Wall.xls" in Attachment E

For example, the following level definitions state that all elements with starting node numbers (Joint I) between 1 and 100 are located in Level 1 and that these nodes have Z coordinates = 0.0 feet. Similarly, all elements with starting node numbers (Joint I) between 101 and 199 are located in Level 2 and these nodes have Z coordinates between 0.001 and 31.999 feet. The stick elements in Levels 2 and 4 represent the piers from the floor slabs at EL. 0 and EL. 32 to the tops of the openings located within the walls.

Table 6.2.8 Level Definition Output

Level Definitions

Level	Node Bounds		Z Bounds (ft)		Level
	Lower	Upper	Lower	Upper	
1	1	100	0	0	1
2	101	199	0.001	31.999	2
3	200	299	32	32	3
4	300	399	32.001	63.999	4
5	400	499	64	64	5
6	500	599	72	72	6
7	600	699	100	100	7

* Source - Sheet "In-Plane Shear Frag Summary" in Excel file "CRCF – Fragility – In-Plane Shear.Wall.xls" in Attachment E

Note on Story Definitions:
The level definitions shown above indicate that there are 7 levels in the CRCF. However, there are three major stories in the CRCF: EL. 0'-0" to EL. 32'-0", EL. 32'-0" to EL. 64'-0", and EL. 64'-0" to EL. 100'-0". (The floor slab at EL. 72'-0" is not considered to constitute a separate Story because this slab is small in comparison to the floor slab at EL. 64'-0"). The CRCF Stories relate to the Levels defined above as follows:

Table 6.2.9 CRCF Story/Level Relationship

CRCF Story	CRCF Level
1	1
2	3
3	5

Column Line Definitions

Table 6.2.10 Column Line Definition Information

Excel Columns*	Value ID	Description
I	Stick ID	Name of the individual stick element
J	Joint I	ID of the starting node of the beam element
K	Level	Level that the Stick ID is located in. This value is determined based on the numerical value of Joint I and the Level Definitions
L	In-Plane Direction	In-Plane direction of the shear wall in which the Stick ID is located. This value is NS for walls running in the North-South direction and EW for wall running in the East-West direction
M	Column Line	Column line that defines the wall in which the individual stick element is located.

* Source - Sheet "In-Plane Shear Frag Summary" in Excel file "CRCF – Fragility – In-Plane Shear.Wall.xls" in Attachment E

For example, stick element '1A.1', with a starting node (Joint I) = 31, is located on the 1st level in the North-South (NS) direction and is located along column line 1. Also, stick element '2B.4', with a starting node = 320, is located on the 4th level in the North-South direction and is located along column line 2.

Table 6.2.11 Column Line Definition Sample
* Source – Sheet "In-Plane Frag. Summary" in Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" in Attachment E

Column Line Definitions

Stick ID	Joint I	Level	In-Plane Direction	Column Line
1A.1	31	1	NS	1
2B.4	320	4	NS	2

Individual HCLPF Data

Table 6.2.12 Individual HCLPF Data Definitions

Excel Columns*	Value ID	Description
N	$\phi V_n = C_{98\%}$	In-plane shear capacity; value taken from Column AC on Sheet "Frag. Shear Calculation"
O	V_{uNS}	In-plane shear due to non-seismic loads; value taken from Column AE on Sheet "Frag. Shear Calculation"
P	V_{uBDBGM}	In-plane shear due to BDBGM seismic load; value taken from Column AF on Sheet "Frag. Shear Calculation"
Q	F_s	Strength Margin Factor for the individual stick element; value taken from Column AG on Sheet "Frag. Shear Calculation"
R	F_μ	Inelastic Energy Absorption Factor; value taken from Column AH on Sheet "Frag. Shear Calculation"
S	C_{HCLPF} (g)	HCLPF capacity for individual stick element; value taken from Column AI on Sheet "Frag. Shear Calculation"

* Sheet "In-Plane Shear Frag Summary" in Excel file "CRCF – Fragility – In-Plane Shear.Wall.xls" in Attachment E

HCLPF Combination Calculations

Equation 4-3 given in Section 4.3.2 of this calculation is used to calculate strength margin factors considering redistribution of forces at each level of a given wall (column line). Table 6.2.13 below shows the Excel calculations performed for Column Line 5. To reproduce this calculation, a value of 5 is placed in cell "V7", a value of "NS" is placed in Cell "V5", and a value of 0.9138 is placed in cell "X3" in sheet "In-Plane Shear Frag. Summary" in the Excel file "CRCF – Fragility – In-Plane Shear Wall.xls". A sample hand calculation created using MathCAD is shown below to further validate the Excel results. References to columns are for sheet "In-Plane Shear Frag Summary" in Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" included in Attachment E.

Table 6.2.13 HCLPF Combination Sample Output

PGA_H	0.9138	g

In-Plane Direction	NS					
Column Line	5	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	$Fs_{Combined}$	$F\mu$	CHCLPF (g)
Level	1	35,485	31,188	1.14	1.75	1.82
	2	56,325	31,188	1.81	1.75	2.89
	3	59,820	20,244	2.95	1.75	4.73
	4	75,030	20,244	3.71	1.75	5.93
	5	0	0	NA	1.75	NA
	6	0	0	NA	1.75	NA
	7	0	0	NA	1.75	NA

* Source – Sheet "In-Plane Frag. Summary" in Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" in Attachment E

CRCF In-Plane Shear HCLPF Redistribution Calculations for Column Line 5

$ORIGIN := 1$ Set the array origin to 1

$f_c := 5500psi$ Concrete compressive strength

$f_y := 60000psi$ Steel yield strength

$PGA := 0.9138g$ BDBGM Peak ground acceleration

Properties for Column Line 5

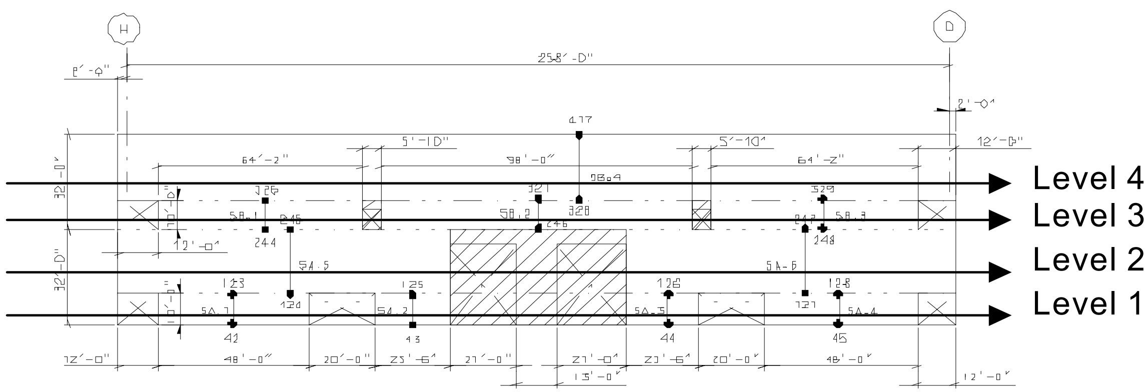
Information is retrieved from Column Line Definitions (Columns I to M)

$levels := 4$ Number of levels in column line 5

$$level\_num := \begin{pmatrix} 4 \\ 2 \\ 3 \\ 1 \end{pmatrix}$$ Number of individual sticks at each level

$$Stick\_ID := \begin{array}{c} \\ \\ \end{array}\begin{array}{cccc} \underline{\text{Level 1}} & \underline{\text{Level 2}} & \underline{\text{Level 3}} & \underline{\text{Level 4}} \end{array}$$

$$Stick\_ID := \begin{pmatrix} \text{"5A.1"} & \text{"5A.5"} & \text{"5B.1"} & \text{"5B,4"} \\ \text{"5A.2"} & \text{"5A.6"} & \text{"5B.2"} & \text{"NA"} \\ \text{"5A.3"} & \text{"NA"} & \text{"5B.3"} & \text{"NA"} \\ \text{"5A.4"} & \text{"NA"} & \text{"NA"} & \text{"NA"} \end{pmatrix}$$ Stick IDs at each level (each column represents a level)



Individual HCLPF Data for Column Line 5
Information is retrieved from Individual HCLPF Data (Columns N to S)

$$\phi V_n := \begin{pmatrix} 11628 & 28185 & 16573 & 75030 \\ 6136 & 28180 & 26668 & 0 \\ 6135 & 0 & 16579 & 0 \\ 11625 & 0 & 0 & 0 \end{pmatrix} \cdot kip$$

Shear Capacities for Column Line 5 Individual Stick Elements.
These values are from column AC on Sheet "Frag. Shear Calculation" in the Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E.

$$Vu_{NS} := \begin{pmatrix} 12 & 18 & 0 & 0 \\ 6 & -22 & 0 & 0 \\ -7 & 0 & 0 & 0 \\ -15 & 0 & 0 & 0 \end{pmatrix} \cdot kip$$

In-plane shear due to non-seismic loads for Column Line 5 Individual Stick Elements.
These values are from column AE on Sheet "Frag. Shear Calculation" in the Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E.

$$Vu_{BDBGM} := \begin{pmatrix} 10629 & 15594 & 5727 & 20244 \\ 4964 & 15594 & 8790 & 0 \\ 4964 & 0 & 5727 & 0 \\ 10629 & 0 & 0 & 0 \end{pmatrix} \cdot kip$$

In-plane shear due to BDBGM seismic loads for Column Line 5 Individual Stick Elements.
These values are from column AF on Sheet "Frag. Shear Calculation" in the Excel file "CRCF - Fragility - In-Plane Shear Wall.xls" included in Attachment E.

$F_{sRedistributed} :=$
```
for i ∈ 1 .. levels
    a ← 0.0
    b ← 0.0
    for j ∈ 1 .. level_num
        a ← a + (φVn[j,i] − |VuNS[j,i]|)
        b ← b + VuBDBGM[j,i]
    Fs[i] ← a / b
Fs
```

$$F_{sRedistributed} = \begin{pmatrix} 1.14 \\ 1.81 \\ 2.95 \\ 3.71 \end{pmatrix}$$

Strength Margin Factor identical to the Fs values shown in Table 6.2.18 for levels 1 to 4 for Column Line 5.

$F_\mu := 1.75$ Inelastic energy absorption used for the N-S shear walls stick elements

$$C_{HCLPFRedist} := F_{sRedistributed} \cdot F_\mu \cdot PGA$$

$$C_{HCLPFRedist} = \begin{pmatrix} 1.82 \\ 2.89 \\ 4.73 \\ 5.93 \end{pmatrix} g$$

Redistributed HCLPF capacities identical to those shown in Table 6.2.18 for levels 1 to 4 for Column Line 5.

Column Line HCLPF Summary

The following tables show the redistributed HCLPF calculations at each level for each column line. The N-S column lines and E-W column lines are shown separately.

Table 6.2.14 to Table 6.2.25 – Redistributed HCLPF Calculations for the North-South Column Lines
Table 6.2.26 to Table 6.2.32 – Redistributed HCLPF Calculations for the East-West Column Lines

Note: a "NA" value indicates that the column line does not contain individual stick elements at that level.

Table 6.2.14 Redistributed HCLPF Capacity Calculations – Column Line 1

Column Line	1	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	6,732	2,461	2.74	1.75	**4.37**
	2	0	0	NA	1.75	**NA**
	3	0	0	NA	1.75	**NA**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.15 Redistributed HCLPF Capacity Calculations – Column Line 2

Column Line	2	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	52,528	35,452	1.48	1.75	**2.37**
	2	55,913	34,725	1.61	1.75	**2.57**
	3	66,749	18,446	3.62	1.75	**5.79**
	4	74,881	18,446	4.06	1.75	**6.49**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.16 Redistributed HCLPF Capacity Calculations – Column Line 3

Column Line	3	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	38,794	27,556	1.41	1.75	**2.25**
	2	45,871	27,556	1.66	1.75	**2.66**
	3	39,050	11,980	3.26	1.75	**5.21**
	4	46,127	11,980	3.85	1.75	**6.16**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.17 Redistributed HCLPF Capacity Calculations – Column Line 4

Column Line	4	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	8,287	4,008	2.07	1.75	**3.31**
	2	0	0	NA	1.75	**NA**
	3	26,663	7,574	3.52	1.75	**5.63**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.18 Redistributed HCLPF Capacity Calculations – Column Line 5

Column Line	5	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	35,485	31,188	1.14	1.75	**1.82**
	2	56,325	31,188	1.81	1.75	**2.89**
	3	59,820	20,244	2.95	1.75	**4.73**
	4	75,030	20,244	3.71	1.75	**5.93**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.19 Redistributed HCLPF Capacity Calculations – Column Line 6

Column Line	6	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	38,818	30,790	1.26	1.75	**2.02**
	2	52,513	30,790	1.71	1.75	**2.73**
	3	50,923	21,035	2.42	1.75	**3.87**
	4	72,534	21,035	3.45	1.75	**5.51**
	5	66,614	16,972	3.93	1.75	**6.28**
	6	72,423	16,972	4.27	1.75	**6.82**
	7	0	0	NA	1.75	**NA**

Table 6.2.20 Redistributed HCLPF Capacity Calculations – Column Line 7

Column Line	7	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	6,736	3,466	1.94	1.75	**3.11**
	2	0	0	NA	1.75	**NA**
	3	0	0	NA	1.75	**NA**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.21 Redistributed HCLPF Capacity Calculations – Column Line 8

Column Line	8	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	Fμ	CHCLPF (g)
Level	1	41,096	32,779	1.25	1.75	**2.00**
	2	54,844	32,779	1.67	1.75	**2.68**
	3	0	0	NA	1.75	**NA**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.22 Redistributed HCLPF Capacity Calculations – Column Line 9

Column Line	9	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	Fμ	CHCLPF (g)
Level	1	39,200	32,614	1.20	1.75	**1.92**
	2	54,787	32,614	1.68	1.75	**2.69**
	3	65,418	23,950	2.73	1.75	**4.37**
	4	72,496	23,950	3.03	1.75	**4.84**
	5	66,687	18,853	3.54	1.75	**5.66**
	6	72,364	16,436	4.40	1.75	**7.04**
	7	0	0	NA	1.75	**NA**

Table 6.2.23 Redistributed HCLPF Capacity Calculations – Column Line 11

Column Line	11	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	Fμ	CHCLPF (g)
Level	1	25,978	14,665	1.77	1.75	**2.83**
	2	29,442	14,665	2.01	1.75	**3.21**
	3	0	0	NA	1.75	**NA**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.24 Redistributed HCLPF Capacity Calculations – Column Line 12

Column Line	12	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	Fμ	CHCLPF (g)
Level	1	49,804	36,307	1.37	1.75	**2.19**
	2	54,844	36,307	1.51	1.75	**2.42**
	3	66,217	26,683	2.48	1.75	**3.97**
	4	74,350	26,683	2.79	1.75	**4.46**
	5	25,569	3,593	7.12	1.75	**11.38**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.25 Redistributed HCLPF Capacity Calculations – Column Line 13

Column Line	13	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	Fμ	CHCLPF (g)
Level	**1**	4,987	1,915	2.60	1.75	**4.16**
	2	0	0	NA	1.75	**NA**
	3	0	0	NA	1.75	**NA**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.26 Redistributed HCLPF Capacity Calculations – Column Line D

Column Line	D	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	Fμ	CHCLPF (g)
Level	**1**	77,366	51,423	1.50	1.75	**2.41**
	2	104,402	51,423	2.03	1.75	**3.25**
	3	86,275	41,072	2.10	1.75	**3.36**
	4	102,522	41,072	2.50	1.75	**3.99**
	5	27,026	8,774	3.08	1.75	**4.93**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.27 Redistributed HCLPF Capacity Calculations – Column Line E

Column Line	E	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	Fμ	CHCLPF (g)
Level	**1**	86,859	49,066	1.77	1.75	**2.83**
	2	0	0	NA	1.75	**NA**
	3	53,151	24,825	2.14	1.75	**3.42**
	4	65,877	24,825	2.65	1.75	**4.24**
	5	34,953	11,071	3.16	1.75	**5.05**
	6	10,951	4,864	2.25	1.75	**3.60**
	7	0	0	NA	1.75	**NA**

Table 6.2.28 Redistributed HCLPF Capacity Calculations – Column Line E.3

Column Line	E.3	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	Fμ	CHCLPF (g)
Level	**1**	32,084	18,555	1.73	1.75	**2.77**
	2	0	0	NA	1.75	**NA**
	3	0	0	NA	1.75	**NA**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.29 Redistributed HCLPF Capacity Calculations – Column Line F

Column Line	F	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	32,109	18,593	1.73	1.75	**2.76**
	2	0	0	NA	1.75	**NA**
	3	0	0	NA	1.75	**NA**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.30 Redistributed HCLPF Capacity Calculations – Column Line F.7

Column Line	F.7	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	32,079	18,633	1.72	1.75	**2.75**
	2	0	0	NA	1.75	**NA**
	3	0	0	NA	1.75	**NA**
	4	0	0	NA	1.75	**NA**
	5	0	0	NA	1.75	**NA**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

Table 6.2.31 Redistributed HCLPF Capacity Calculations – Column Line G

Column Line	G	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	80,552	46,330	1.74	1.75	**2.78**
	2	18,872	8,886	2.12	1.75	**3.40**
	3	53,182	24,573	2.16	1.75	**3.46**
	4	65,879	24,573	2.68	1.75	**4.29**
	5	34,955	10,986	3.18	1.75	**5.09**
	6	10,951	4,864	2.25	1.75	**3.60**
	7	0	0	NA	1.75	**NA**

Table 6.2.32 Redistributed HCLPF Capacity Calculations – Column Line H

Column Line	H	$\Sigma(C_{98\%} - Vu_{NS})$	ΣVu_{BDBGM}	Fs	$F\mu$	CHCLPF (g)
Level	1	69,157	47,589	1.45	1.75	**2.32**
	2	95,688	47,589	2.01	1.75	**3.22**
	3	89,702	39,779	2.26	1.75	**3.61**
	4	102,511	39,779	2.58	1.75	**4.12**
	5	27,026	8,773	3.08	1.75	**4.93**
	6	0	0	NA	1.75	**NA**
	7	0	0	NA	1.75	**NA**

6.2.7 In-Plane Shear HCLPF Capacity Summary

Section 6.2.6.2 indicates that the minimum in-plane shear HCLPF capacity for an individual stick element is **1.75g** and was calculated for stick elements 5A.1 and 5A.4 located in the shear wall along column line 5.

Table 6.2.14 to Table 6.2.32 in Section 6.2.6.3 indicates that the minimum in-plane shear HCLPF capacity for an entire wall is **1.82g** and was calculated for the 1st story of the shear wall along column line 5.

6.3 HCLPF CAPACITY EVALUATIONS FOR DIAPHRAGMS

General Information and Data

$ORIGIN := 1$ Set the array origin to 1

$f_c := 5500psi$ Concrete compressive strength per section 6.2.4.1 of this calculation

$f_y := 60000psi$ Steel yield strength (Ref. 2.2.1, Section 4.2.11.6.2)

$PGA_h := 0.9138g$ BDBGM peak horizontal ground acceleration (Ref. 2.2.31)

$cover := 0.75in$ Concrete clear cover for bottom bars of slabs per Section 7.7.1 of Ref. 2.2.2

$$d_b(num) := \left| \begin{array}{l} 0.375in \text{ if } num = 3 \\ 0.500in \text{ if } num = 4 \\ 0.625in \text{ if } num = 5 \\ 0.750in \text{ if } num = 6 \\ 0.875in \text{ if } num = 7 \\ 1.00in \text{ if } num = 8 \\ 1.128in \text{ if } num = 9 \\ 1.270in \text{ if } num = 10 \\ 1.410in \text{ if } num = 11 \\ 0.0in \text{ otherwise} \end{array} \right.$$

$$A_{sbar}(num) := \left| \begin{array}{l} 0.11in^2 \text{ if } num = 3 \\ 0.20in^2 \text{ if } num = 4 \\ 0.31in^2 \text{ if } num = 5 \\ 0.44in^2 \text{ if } num = 6 \\ 0.60in^2 \text{ if } num = 7 \\ 0.79in^2 \text{ if } num = 8 \\ 1.00in^2 \text{ if } num = 9 \\ 1.27in^2 \text{ if } num = 10 \\ 1.56in^2 \text{ if } num = 11 \\ 0.0in^2 \text{ otherwise} \end{array} \right.$$

Table A-1 of Ref. 2.2.36

6.3.1 HCLPF Capacity for Diaphragms - Out-of-Plane Forces

Similar to Ref. 2.2.30, five slab cases are considered in the out-of-plane diaphragm fragility calculation. These cases bound all diaphragms for out-of-plane considerations in the CRCF. Also, the out-of-plane HCLPF capacity evaluation does not include the HCLPF capacity calculation of the steel decking. The purpose of the steel decking is to support the weight of the wet concrete during placement and is not considered in the slab HCLPF capacity evaluations.

Note: Case 4 and Case 5 are the slab at EL. 32' bounded by grid lines 6-9 and E-G. This slab is 48" in some areas, but in other areas there is a 18" depression due to the presence of the slide gates (Ref. 2.2.37). The uniform loadings are the same for Cases 4 and 5. However, the demands and capacities for Case 4 are those associated with the full section (48") and Case 5 considers the demands and capacities associated with the reduced section (30").

The diaphragm cases are determined from Ref. 2.2.30 and from drawings in Ref. 2.2.7 thru Ref. 2.2.26.

$case\_num := 5$

$$\text{Cases} := \begin{pmatrix} \text{"18" roof slab at EL. 100'"} \\ \text{"18" floor slab at EL. 32'"} \\ \text{"33" roof slab at EL. 64'"} \\ \text{"48" floor slab at EL. 32"} \\ \text{"30" floor slab at EL. 32"} \end{pmatrix}$$

(Note: 18" roof slab at EL. 100 bounds 18" roof slab at EL. 64")

6.3.1.1 Loads

Concrete Dead Loads

$w_{conc} := 150\text{pcf}$ Unit weight of concrete Ref. 2.2.1, Section 4.2.11.6.6

All slabs, except for the 4-ft thick slab at EL. 32', are constructed on a 3" (0.25 ft) metal deck. Consider 1/2 the depth of the deck in the slab dead load calculation.

For the Case 4 and Case 5 slab dead load, consider 48" and 30" thickness, respectively.

$$\text{SDL} := \begin{pmatrix} 18\text{in} + 1.5\cdot\text{in} \\ 18\cdot\text{in} + 1.5\cdot\text{in} \\ 33\cdot\text{in} + 1.5\cdot\text{in} \\ 48\cdot\text{in} \\ 30\text{in} \end{pmatrix}\cdot w_{conc} \qquad \text{SDL} = \begin{pmatrix} 244 \\ 244 \\ 431 \\ 600 \\ 375 \end{pmatrix}\text{psf}$$

Steel Framing Dead Loads

$$\text{SFDL} := \begin{pmatrix} 50 \\ 50 \\ 80 \\ 0 \\ 0 \end{pmatrix}\cdot\text{psf} \qquad \text{SFDL} = \begin{pmatrix} 50.0 \\ 50.0 \\ 80.0 \\ 0.0 \\ 0.0 \end{pmatrix}\text{psf}$$

See Section 6.4.1 of this calculation for structural steel weight (Conservatively consider steel framing loads in the out-of-plane diaphragm evaluation). Case 4 and 5 are not supported by structural steel.

Equipment Dead Loads + Miscellaneous Hanging Equipment Load

$$\text{EDL} := \begin{pmatrix} 50 + 10 \\ 50 + 10 \\ 50 + 10 \\ 50 + 10 \\ 50 + 10 \end{pmatrix}\cdot\text{psf} \qquad \text{EDL} = \begin{pmatrix} 60.0 \\ 60.0 \\ 60.0 \\ 60.0 \\ 60.0 \end{pmatrix}\text{psf}$$

(Assumption 3.1.4)

Roof Material Dead Loads

$$RMDL := \begin{pmatrix} 15 \\ 0 \\ 15 \\ 0 \\ 0 \end{pmatrix} \cdot psf$$ (Assumption 3.1.5) $$RMDL = \begin{pmatrix} 15.0 \\ 0.0 \\ 15.0 \\ 0.0 \\ 0.0 \end{pmatrix} psf$$

Design Live Loads

$$LL := \begin{pmatrix} 40 \\ 100 \\ 40 \\ 100 \\ 100 \end{pmatrix} \cdot psf$$ (Assumption 3.1.6) $$LL = \begin{pmatrix} 40.0 \\ 100.0 \\ 40.0 \\ 100.0 \\ 100.0 \end{pmatrix} psf$$

- 40 psf live load for roof slabs
- 100 psf live load for floor slabs

Acceleration Factors for Seismic Loads

The following accelerations are the maximum horizontal (envelope of X and Y directions) and vertical accelerations at each elevation due to the BDBGM_SRSS seismic load case (Ref. 2.2.5). See Attachment C for the seismic analysis results of Ref. 2.2.5. Note: Table 15 of Ref. 2.2.5 gives the X and Y maximum accelerations at the center of gravity of each elevation. The following values are the maximum accelerations for all nodes at each elevation.

$$Acc_h := \begin{pmatrix} 1.31 \\ 1.56 \\ 1.60 \\ 1.83 \end{pmatrix} \cdot g$$ EL. 32', EL. 64', EL. 72', EL. 100' — Max. Horizontal accelerations due to BDBGM_SRSS load in Ref. 2.2.5

$$Acc_v := \begin{pmatrix} 1.06 \\ 1.07 \\ 1.05 \\ 1.10 \end{pmatrix} \cdot g$$ EL. 32', EL. 64', EL. 72', EL. 100' — Max. Vertical accelerations due to BDBGM_SRSS load in Ref. 2.2.5

$amplify := 2$ Vertical amplification factor (Assumption 3.1.8)

Seismic Load Combination

$$EQ_{LC} := DL + 0.25LL$$ Per Ref. 2.2.4, 25% of the design live load is considered to act concurrently with the seismic load.

Total Dead Loads

The total dead load (DL) includes the slab dead load, steel framing dead load, equipment dead load, and roofing material dead load.

$$DL := SDL + SFDL + EDL + RMDL \qquad DL^T = (369 \quad 354 \quad 586 \quad 660 \quad 435)\,psf$$

Seismic Loads

$$SACC_z := amplify \cdot \begin{pmatrix} Acc_{v_4} \\ Acc_{v_1} \\ Acc_{v_2} \\ Acc_{v_1} \\ Acc_{v_1} \end{pmatrix} \quad Cases = \begin{pmatrix} \text{"18" roof slab at EL. 100'"} \\ \text{"18" floor slab at EL. 32'"} \\ \text{"33" roof slab at EL. 64'"} \\ \text{"48" floor slab at EL. 32"} \\ \text{"30" floor slab at EL. 32"} \end{pmatrix} \quad SACC_z = \begin{pmatrix} 2.20 \\ 2.12 \\ 2.14 \\ 2.12 \\ 2.12 \end{pmatrix} g$$

Amplified vertical seismic accelerations for Cases 1 to 5

$$E := \overrightarrow{\left[(DL + 0.25 \cdot LL) \cdot \frac{SACC_z}{g}\right]} \qquad E^T = (833 \quad 803 \quad 1276 \quad 1452 \quad 975)\,psf$$

Non-Seismic Loads

$$w_{uNS} := DL + 0.25LL \qquad {w_{uNS}}^T = (379 \quad 379 \quad 596 \quad 685 \quad 460)\,psf$$

BDBGM Seismic Loads

$$w_{uBDBGM} := E \qquad {w_{uBDBGM}}^T = (833 \quad 803 \quad 1276 \quad 1452 \quad 975)\,psf$$

6.3.1.2 Moment and Shear Demands for Slabs

See structural design drawings (Ref. 2.2.7 to Ref. 2.2.26) and Attachment A for plan and wall elevations of the CRCF structure.

Slab Cases 1 to 3

Ref. 2.2.38 contains the calculations for the structural steel framing system. The beam spacing in Ref. 2.2.38 ranges from 5.3 feet to 6.67 feet. Therefore, the maximum spans considered for the slabs constructed on the 3" metal deck with more than 2 continuous spans is 7'-0". The 18" slabs at EL. 32, 64, 72, and 100 and the 33" slab at EL. 64 are such slabs.

All slabs are treated as one-way slabs (Assumption 3.2.2).

Using the equations for moments and shear from ACI 349-01 (Ref. 2.2.2 Section 8.3),

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

Max. shear force = 1.15wL/2 (shear in end members at face of interior support)

Slab Case 4 and 5

The 48" thick slab bounded by col. lines 6-9/E-G at EL. 32 is not supported by structural steel framing or a metal deck. It has more than two continuous spans, assuming one-way action in the N-S direction (Assumption 3.2.), with spans of 15', 32', 32', and 15' between column lines E & G.

A SAP2000 model (Ref. 2.2.44) is developed in Attachment D to determine the maximum moments and shears for Slab Case 4 and Case 5. In this model, the beam stiffness variation along the length of beam due to the 18" depression for the slide gates (see Ref. 2.2.9) is considered. Also, the uniform loading on the beam strip in this model is arbitraily set equal to 1 kip/ft. The moments and shears for the seismic and non-seismic loading can be determined using results from this analysis by simple ratio.

The maximum moments and shears at the reduced 30" section (in the area of the 18" depression) and the full section (48" slab) are determined and checked against the respective capacities at these locations in the HCLPF evaluation.

$b := 1\text{ft}$ Perform all calculations for 1-ft wide strip

$$\text{Cases} = \begin{pmatrix} \text{"18" roof slab at EL. 100'"} \\ \text{"18" floor slab at EL. 32'"} \\ \text{"33" roof slab at EL. 64'"} \\ \text{"48" floor slab at EL. 32"} \\ \text{"30" floor slab at EL. 32"} \end{pmatrix} \qquad \text{span} := 7\text{ft}$$

Note:
Moments and shear for Case 4 and 5 are determined from the SAP2000 analysis in Attachment D. Therefore, the span value is used for Cases 1 to 3, which are 7-ft spans.

Maximum moment and shear for Cases 1 to 3

$$Mu_{max}(w, L) := \frac{w \cdot L^2}{10} \cdot b$$

$$Vu_{max}(w, L) := \frac{1.15 w \cdot L}{2} \cdot b$$

$$w_{uNS} \cdot b = \begin{pmatrix} 0.38 \\ 0.38 \\ 0.60 \\ 0.69 \\ 0.46 \end{pmatrix} \frac{kip}{ft} \qquad w_{uBDBGM} \cdot b = \begin{pmatrix} 0.83 \\ 0.80 \\ 1.28 \\ 1.45 \\ 0.98 \end{pmatrix} \frac{kip}{ft}$$

Case 1 Design Moments and Shears:

$$M_{uNS_1} := \overrightarrow{Mu_{max}\left(w_{uNS_1}, \text{span}\right)} \qquad M_{uNS_1} = 1.86\, kip \cdot ft$$

$$M_{uBDBGM_1} := \overrightarrow{Mu_{max}\left(w_{uBDBGM_1}, \text{span}\right)} \qquad M_{uBDBGM_1} = 4.08\, kip \cdot ft$$

$$V_{uNS_1} := \overrightarrow{Vu_{max}\left(w_{uNS_1}, \text{span}\right)} \qquad V_{uNS_1} = 1.52\, kip$$

$$V_{uBDBGM_1} := \overrightarrow{Vu_{max}\left(w_{uBDBGM_1}, \text{span}\right)} \qquad V_{uBDBGM_1} = 3.35\, kip$$

Case 2 Design Moments and Shears:

$$M_{uNS_2} := \overrightarrow{Mu_{max}(w_{uNS_2}, span)} \qquad M_{uNS_2} = 1.86\,kip\cdot ft$$

$$M_{uBDBGM_2} := \overrightarrow{Mu_{max}(w_{uBDBGM_2}, span)} \qquad M_{uBDBGM_2} = 3.93\,kip\cdot ft$$

$$V_{uNS_2} := \overrightarrow{Vu_{max}(w_{uNS_2}, span)} \qquad V_{uNS_2} = 1.52\,kip$$

$$V_{uBDBGM_2} := \overrightarrow{Vu_{max}(w_{uBDBGM_2}, span)} \qquad V_{uBDBGM_2} = 3.23\,kip$$

Case 3 Design Moments and Shears:

$$M_{uNS_3} := \overrightarrow{Mu_{max}(w_{uNS_3}, span)} \qquad M_{uNS_3} = 2.92\,kip\cdot ft$$

$$M_{uBDBGM_3} := \overrightarrow{Mu_{max}(w_{uBDBGM_3}, span)} \qquad M_{uBDBGM_3} = 6.25\,kip\cdot ft$$

$$V_{uNS_3} := \overrightarrow{Vu_{max}(w_{uNS_3}, span)} \qquad V_{uNS_3} = 2.40\,kip$$

$$V_{uBDBGM_3} := \overrightarrow{Vu_{max}(w_{uBDBGM_3}, span)} \qquad V_{uBDBGM_3} = 5.14\,kip$$

Case 4 and Case 5:
A SAP2000 model is created to determine the moments and shear for the slab for Case 4 and Case 5. (See Attachment D for the SAP2000 model and results). The beam model represents a 1-ft wide strip of the slab at EL. 32'-0" between grid lines 6 and 6.8 and running from grid line E to grid line G. This strip bounds the response of the slab in this region.

Figure 6.3.1 shows a plan view of the beam strip location at EL. 32'-0".

Figure 6.3.2 shows the joint IDs and the restraint locations of the model. The joint IDs correspond to the following locations that can be identified on the plan view shown in Figure 6.3.1:
Joint 1 - Wall E
Joint 2 - Wall E.3
Joint 3 - Location where slab between Wall E.3 and Wall F goes from 48" to 30"
Joint 4 - Location where slab between Wall E.3 and Wall F goes from 30" to 48"
Joint 5 - Wall F
Joint 6 - Location where slab between Wall F and Wall F.7 goes from 48" to 30"
Joint 7 - Location where slab between Wall F and Wall F.7 goes from 30" to 48"
Joint 8 - Wall F.7
Joint 9 - Wall G
The joint restraints are pinned connections.

Figure 6.3.1 Plan View Showing Location of Case 4/Case 5 Beam Strip at EL. 32'-0" (Ref. 2.2.9)

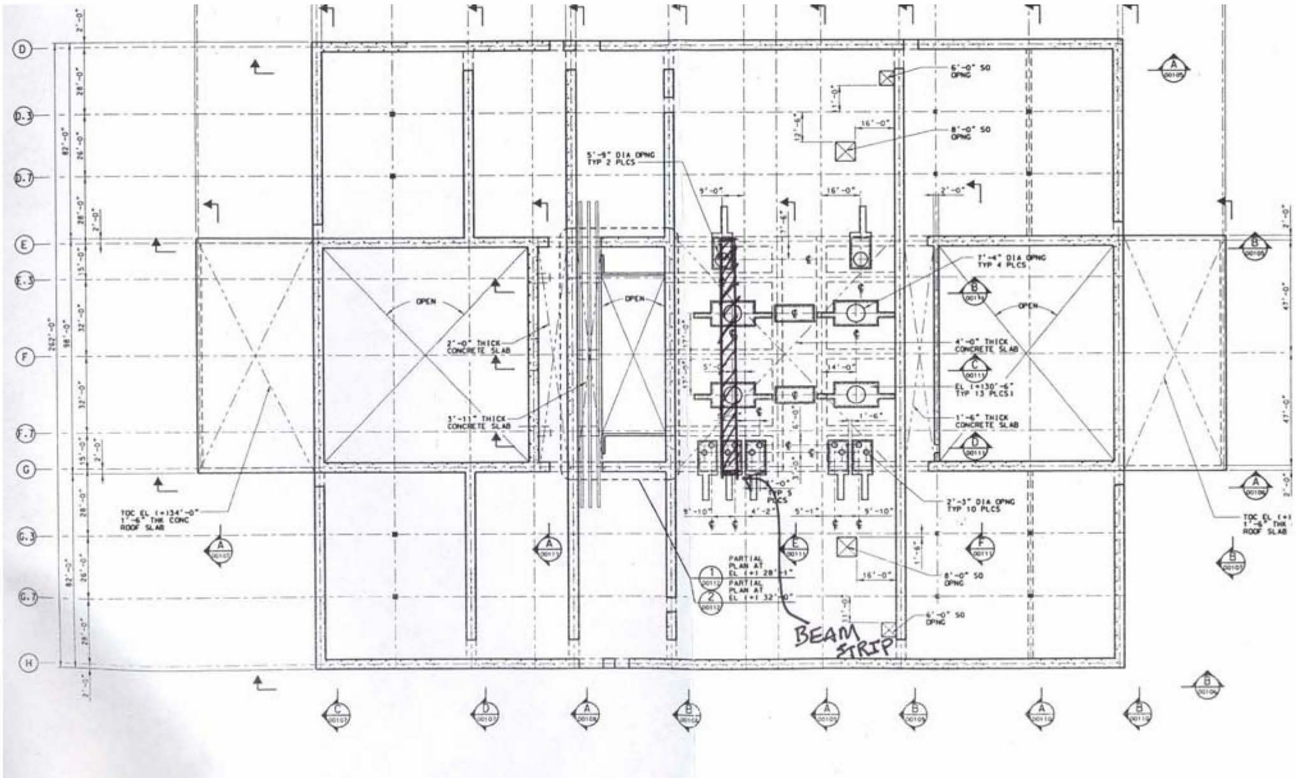


Figure 6.3.2 Joint IDs and Restraints of SAP2000 Beam Model

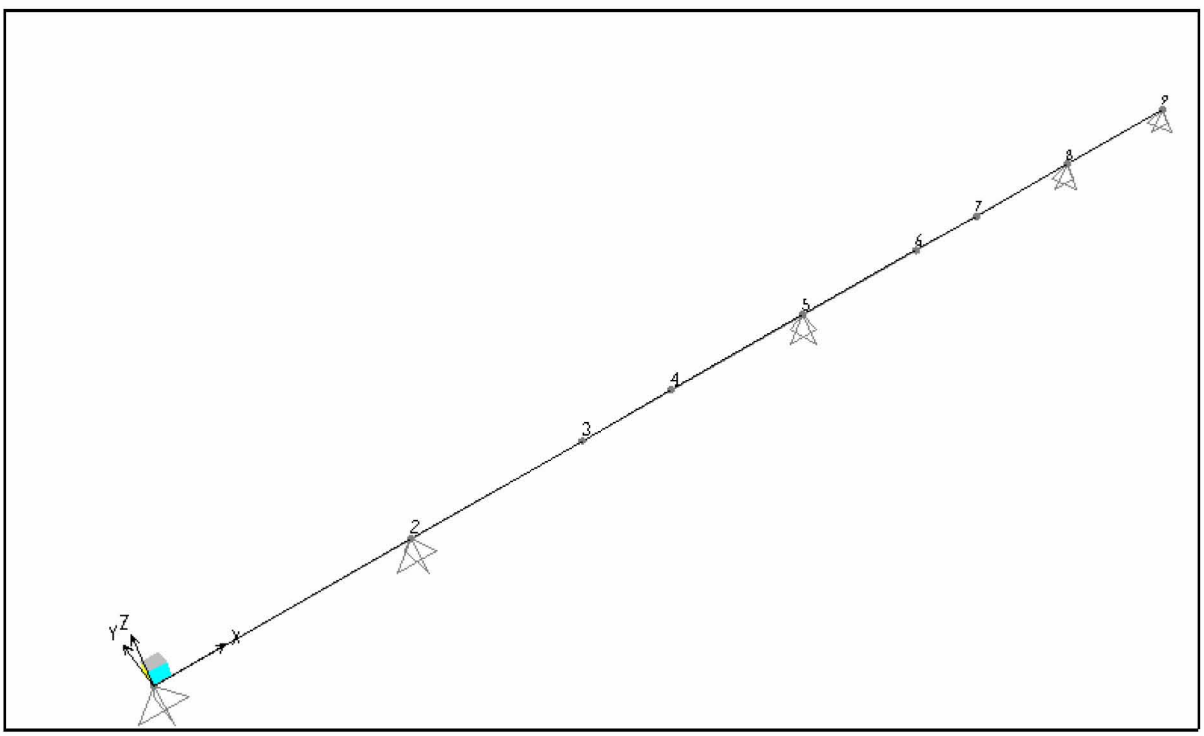
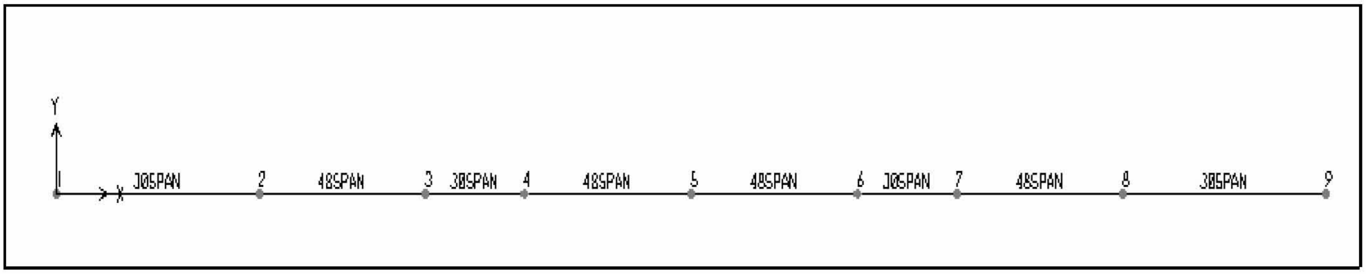


Figure 6.3.3 shows the section properties IDs used in the beam model. The 30SPAN corresponds to the section with a 30" depth and the 48SPAN corresponds to the section with a 48" depth.

Figure 6.3.3 Joint IDs and Frame Sections of SAP2000 Beam Model



For a uniform loading of 1 kip-ft, the moment and shear diagrams for the SAP2000 beam model are shown in Figure 6.3.4 and Figure 6.3.5, respectively.

Figure 6.3.4 Moment Diagram due to 1 kip-ft Loading

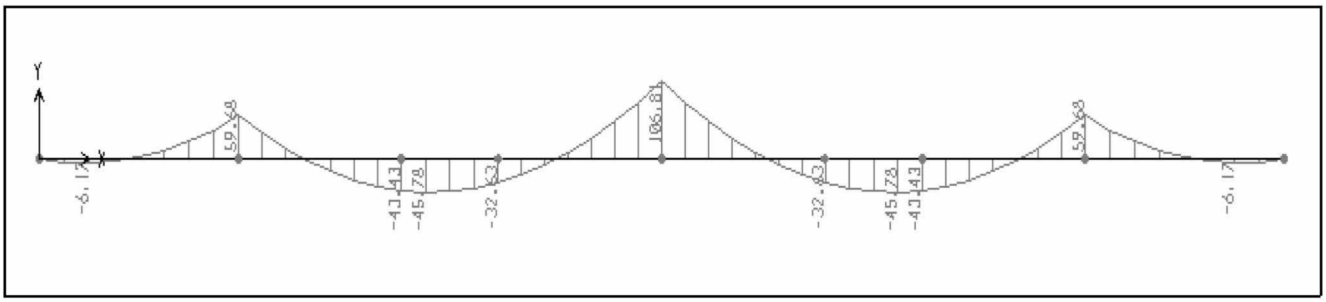
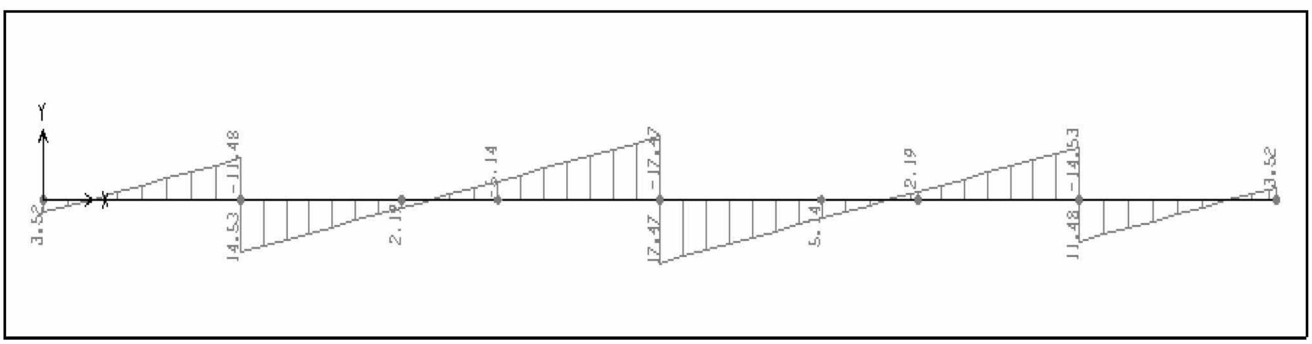


Figure 6.3.5 Shear Diagram due to 1 kip-ft Loading



Based on the moment and shear diagram shown above, the maximum moments and shears for a 1-kip uniform load placed over each span are as follows:

Case 4: Max. moment on 48" thick section: moment at joint 5 = 106 kip-ft
Max. shear on 48" thick section: shear at joint 5 = 17.5 kips
Case 5: Max. moment at 30" thick section: moment at joint 2 = 59 kip-ft
Max. shear on 30" thick section: shear at the end of the member from joint 1 to joint 2 = 11.5 kip

The maximum moments and shear occuring at the location of a wall/slab interface are conservatively taken at the centerline of the supporting wall and not a the face of the wall.

Case 4 and Case 5: Uniform load on beam strip for Non-Seismic Loads per ft width:

$$w_{uNS_4} \cdot b = 0.69 \frac{kip}{ft}$$

$$w_{uNS_5} \cdot b = 0.46 \frac{kip}{ft}$$

Case 4 and Case 5: Uniform load on beam strip for BDBGM Loads per ft width:

$$w_{uBDBGM_4} \cdot b = 1.45 \frac{kip}{ft}$$

$$w_{uBDBGM_5} \cdot b = 0.98 \frac{kip}{ft}$$

Maximum moment ratios:
The maximum moments computed from the SAP2000 beam model (Attachment C) are multiplied by the following ratios to determine the maximum moments used in the HCLPF calculations.

$$ratio_{NS4} := \frac{w_{uNS_4} \cdot b}{1 \cdot \frac{kip}{ft}} \qquad ratio_{NS4} = 0.69$$

$$ratio_{BDBGM4} := \frac{w_{uBDBGM_4} \cdot b}{1 \cdot \frac{kip}{ft}} \qquad ratio_{BDBGM4} = 1.45$$

$$ratio_{NS5} := \frac{w_{uNS_5} \cdot b}{1 \cdot \frac{kip}{ft}} \qquad ratio_{NS5} = 0.46$$

$$ratio_{BDBGM5} := \frac{w_{uBDBGM_5} \cdot b}{1 \cdot \frac{kip}{ft}} \qquad ratio_{BDBGM5} = 0.98$$

Case 4 Design Moments and Shears:

48" section -

Non-seismic loads $M_{uNS_4} := 106 kip \cdot ft \cdot ratio_{NS4}$ $\qquad M_{uNS_4} = 72.6\, kip \cdot ft$

$V_{uNS_4} := 17.5 kip \cdot ratio_{NS4}$ $\qquad V_{uNS_4} = 12.0\, kip$

BDBGM loads $M_{uBDBGM_4} := 106 kip \cdot ft \cdot ratio_{BDBGM4}$ $\qquad M_{uBDBGM_4} = 153.9\, kip \cdot ft$

$V_{uBDBGM_4} := 17.5 kip \cdot ratio_{BDBGM4}$ $\qquad V_{uBDBGM_4} = 25.4\, kip$

Case 5 Design Moments and Shears:

30" section -

Non-seismic loads

$M_{uNS_5} := 59\text{kip}\cdot\text{ft}\cdot\text{ratio}_{NS5}$ $\qquad M_{uNS_5} = 27.1\,\text{kip}\cdot\text{ft}$

$V_{uNS_5} := 11.5\text{kip}\cdot\text{ratio}_{NS5}$ $\qquad V_{uNS_5} = 5.3\,\text{kip}$

BDBGM loads

$M_{uBDBGM_5} := 59\text{kip}\cdot\text{ft}\cdot\text{ratio}_{BDBGM5}$ $\qquad M_{uBDBGM_5} = 57.5\,\text{kip}\cdot\text{ft}$

$V_{uBDBGM_5} := 11.5\text{kip}\cdot\text{ratio}_{BDBGM5}$ $\qquad V_{uBDBGM_5} = 11.2\,\text{kip}$

Non-Seismic Demands for Case 1 to Case 5

$$M_{uNS} = \begin{pmatrix} 1.9 \\ 1.9 \\ 2.9 \\ 72.6 \\ 27.1 \end{pmatrix} \text{kip}\cdot\text{ft}$$

$$V_{uNS} = \begin{pmatrix} 1.5 \\ 1.5 \\ 2.4 \\ 12.0 \\ 5.3 \end{pmatrix} \text{kip}$$

Seismic Demands for Case 1 to Case 5

$$M_{uBDBGM} = \begin{pmatrix} 4.1 \\ 3.9 \\ 6.3 \\ 153.9 \\ 57.5 \end{pmatrix} \text{kip}\cdot\text{ft}$$

$$V_{uBDBGM} = \begin{pmatrix} 3.4 \\ 3.2 \\ 5.1 \\ 25.4 \\ 11.2 \end{pmatrix} \text{kip}$$

6.3.1.3 Slab Design Capacities ($C_{98\%}$)

For the moment and shear capacity calculation for Case 5, the effective depth (d) is calculated considering the 18" depression for the slide gates (48" - 18" = 30").

$$\text{slab}_{bar} := \begin{pmatrix} 7 \\ 7 \\ 9 \\ 10 \\ 10 \end{pmatrix} \qquad \text{slab}_{space} := \begin{pmatrix} 12 \\ 12 \\ 12 \\ 6 \\ 6 \end{pmatrix} \cdot \text{in}$$

Slab reinforcing bar size and spacing from Section 7.1 of Ref. 2.2.30

$$\text{Cases} = \begin{pmatrix} \text{"18" roof slab at EL. 100'"} \\ \text{"18" floor slab at EL. 32'"} \\ \text{"33" roof slab at EL. 64'"} \\ \text{"48" floor slab at EL. 32"} \\ \text{"30" floor slab at EL. 32"} \end{pmatrix}$$

Effective depth:

$$d := \begin{pmatrix} 18in \\ 18in \\ 33in \\ 48in \\ 30\cdot in \end{pmatrix} - \text{cover} - 1.5\cdot\overrightarrow{d_b\left(slab_{bar}\right)} \qquad d = \begin{pmatrix} 15.94 \\ 15.94 \\ 30.56 \\ 45.34 \\ 27.35 \end{pmatrix} in$$

Slab reinforcement per width (b)

$$A_s := \overrightarrow{\left(A_{sbar}\left(slab_{bar}\right)\cdot\frac{b}{slab_{space}}\right)} \qquad A_s^T = (0.60\ \ 0.60\ \ 1.00\ \ 2.54\ \ 2.54)\,in^2$$

Compression block depth

$$a := \overrightarrow{\frac{A_s\cdot f_y}{0.85\cdot f_c\cdot b}} \qquad a^T = (0.64\ \ 0.64\ \ 1.07\ \ 2.72\ \ 2.72)\,in$$

Moment Capacity = $\phi Mn = \phi C_{98\%}$

$\phi_b := 0.9$ Strength reduction factor for transverse bending per Ref. 2.2.2

$$\phi M_n := \overrightarrow{\left[\phi_b \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right]} \qquad \phi M_n = \begin{pmatrix} 42.2 \\ 42.2 \\ 135.1 \\ 502.8 \\ 297.0 \end{pmatrix} kip \cdot ft$$

Transverse Shear Capacity = $\phi Vn = \phi C_{98\%}$

$\phi_s := 0.85$ Strength reduction factor for transverse shear per Ref. 2.2.2

$$\phi V_n := \phi_s \cdot 2 \cdot \sqrt{f_c \cdot psi} \cdot b \cdot d \qquad \phi V_n^T = (24.1 \quad 24.1 \quad 46.2 \quad 68.6 \quad 41.4)\,kip$$

6.3.1.4 Strength Margin Factor - Out-of-Plane Bending (Fs_{Mom}) and Out-of-Plane Shear (Fs_{Shear})

Per Equation 4-2 of Section 4.3.2 of this calculation -

$$F_{sMom} := \overrightarrow{\frac{\phi M_n - M_{uNS}}{M_{uBDBGM}}} \qquad F_{sMom} = \begin{pmatrix} 9.9 \\ 10.2 \\ 21.1 \\ 2.8 \\ 4.7 \end{pmatrix}$$

$$F_{sShear} := \overrightarrow{\frac{\phi V_n - V_{uNS}}{V_{uBDBGM}}} \qquad F_{sShear} = \begin{pmatrix} 6.7 \\ 7.0 \\ 8.5 \\ 2.2 \\ 3.2 \end{pmatrix}$$

6.3.1.5 Inelastic Energy Absorption Factor - F_μ)

$$\frac{span}{d} = \begin{pmatrix} 5.3 \\ 5.3 \\ 2.7 \\ 1.9 \\ 3.1 \end{pmatrix}$$

All span-to-depth ratios are less than 10. Therefore, use $F\mu$ = 2.25 for limit state A. (Slab/wall moment frames with span/depth ratios less than 10) per Table 5-1 of Ref. 2.2.6.

$F_\mu := 2.25$ $F\mu$ for Limit State A per Table 5-1 of Ref. 2.2.6

However, per ASCE 43-05 Section C5.1.2.3, the $F\mu$ factor for slabs with significant gravity loads subject to vertical seismic motion must be reduced to account for ratcheting effects. The following is a derivation of an effective $F\mu$ for floor slabs that accounts for potential ratcheting effects.

When the static loading is sufficiently large that oscillatory dynamic loads result in nonlinear response in one direction only (i.e., no nonlinear response reversals), then with multiple nonlinear cycles, the nonlinear response ratchets in that one direction. On the first nonlinear cycle, the total displacement d_{t1} is:

$$\delta_{t1} := \delta_e + \delta_{p1} \qquad (6.3.1)$$

where δ_e is the elastic displacement and δ_{p1} is the plastic displacement for this first nonlinear cycle. The elastic displacement δ_e is recovered on dynamic response reversal. However, if the static load is sufficiently large that there is no reversal of nonlinear response, then the plastic displacement δ_{p1} is not recovered. Defining the first cycle ductility factor μ_{c1} by:

$$\mu_{c1} := \left(\frac{\delta_{t1}}{\delta_e}\right) \quad \text{and} \quad \mu_{c1} := 1 + \frac{\delta_{p1}}{\delta_e} \qquad (6.3.2)$$

the factor f_1 of non-recovered first cycle response is given by:

$$f_1 := \left(\frac{\delta_{p1}}{\delta_e}\right) \quad \text{and} \quad f_1 := \left(\mu_{c1} - 1\right) \qquad (6.3.3)$$

For N cycles, the total ductility factor μ is:

$$\mu := \left(\frac{\delta_t}{\delta_e}\right) \quad \text{and} \quad \mu := 1 + \sum_{i=1}^{N} \left(f_i\right) \qquad (6.3.4)$$

Assuming the equivalent of N number of equal nonlinear response cycles each with cyclic ductility μ_c, the total ratcheted ductility factor μ is:

$$\mu := 1 + N \cdot \left(\mu_c - 1\right) \qquad (6.3.5)$$

For a total ductility μ, the permissible cyclic ductility μ_c per cycle is:

$$\mu_c := 1 + \frac{\mu - 1}{N} \qquad (6.3.6)$$

Assuming $F\mu$ is roughly proportional to μ, the permissible effective $F_{\mu e}$ for situations of one-way ratcheting is:

$$F_{\mu e} := 1 + \frac{F_\mu - 1}{N} \qquad (6.3.7)$$

where $F\mu$ is the permissible $F\mu$ for situations with complete reversal of nonlinear response.

For actual situations, Equation (6.3.7) is conservatively biased for three reasons. First, unless the static load is a large fraction of the capacity, a portion of the plastic deformation is recoverable on dynamic response reversal. Secondly, $F\mu$ is closer to being proportional to μ^{β} where β is less than or equal to one. Third, each actual nonlinear response cycle does not go to the full cyclic ductility μ_c.

Table 4-2 of NUREG/CR-3805 (Ref. 2.2.42) provides guidance on the number N of equivalent full nonlinear response cycles as a function of strong motion duration. Per Assumption 3.2.4, the strong duration of the BDBDGM ground motion is assumed to be greater than 15 seconds. Therefore, per Table 4-2 of reference 2.2.42, a value of N = 4 is used.

$N := 4$ (Assumption 3.2.4)

$$F_{\mu e} := 1 + \frac{F_{\mu} - 1}{N} \qquad F_{\mu e} = 1.31$$

$F_{\mu oopM} := F_{\mu e}$ $F\mu$ factor for out-of-plane bending of slabs

$F_{\mu oopS} := 1.0$ Transverse shear is a brittle failure mechanism and thus no inelastic energy absoprtion is considered.

6.3.1.6 HCLPF for Out-of-Plane Bending of Diaphragms

$$HCLPF_M := F_{sMom} \cdot F_{\mu e} \cdot PGA_h \qquad HCLPF_M^{\,T} = (11.84 \quad 12.29 \quad 25.36 \quad 3.35 \quad 5.63)\,g$$

$$HCLPF_V := F_{sShear} \cdot F_{\mu oopS} \cdot PGA_h \qquad HCLPF_V^{\,T} = (6.15 \quad 6.39 \quad 7.80 \quad 2.04 \quad 2.94)\,g$$

$$HCLPF_{oopM} := \min\left(HCLPF_M\right)$$

$$HCLPF_{oopV} := \min\left(HCLPF_V\right)$$

$HCLPF_{oopM} = 3.35\,g$ with an $F\mu$ $F_{\mu oopM} = 1.31$ ***Minimum HCLPF for out-of-plane bending of the CRCF diaphragms***

$HCLPF_{oopV} = 2.04\,g$ with an $F\mu$ $F_{\mu oopS} = 1.00$ ***Minimum HCLPF for out-of-plane shear of the CRCF diaphragms***

6.3.2 HCLPF Capacity for Diaphragms - In-Plane Forces

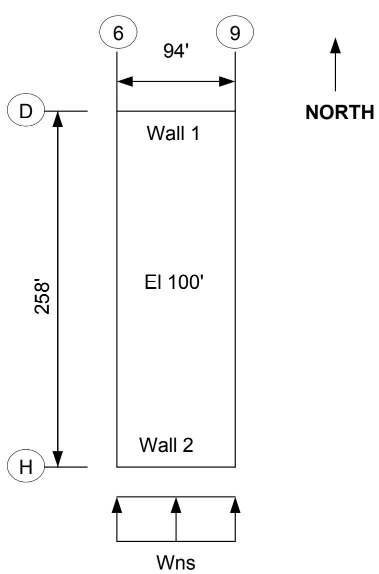
Similar to Ref. 2.2.30, eight cases are considered in the in-plane diaphragm fragility calculation. Analysis cases 1 - 4 consider the 18" roof diaphragms at EL. 64', 72' and 100' and analysis cases 5 - 8 consider the 33" slab at EL. 64', the 48" slab at EL. 32', and the 18" slabs at EL. 32. For each analysis case, the diaphragm is evaluated with the BDBGM acting in the north-south (N-S) direction and the east-west (E-W) direction. Therefore, the horizontal spans and depths considered for the in-plane evaluation of the diaphragm are different, depending on the direction of seismic load considered. For graphical representation of the cases considered see Attachment A of Ref. 2.2.30.

6.3.2.1 Diaphragms Properties and Loads for Analysis Cases 1 to 4

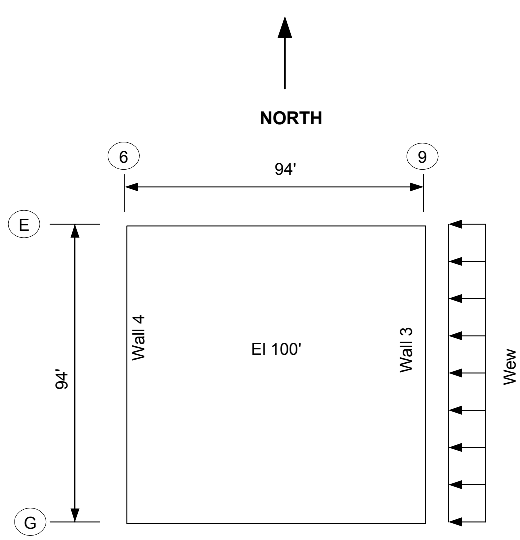
$thick := 18in$ All slabs considered in analysis cases 1 to 4 are 18" thick.

$case\_num := 8$

$$Cases_{1to4} := \begin{pmatrix} \text{"Case 1 (N-S): 18" roof @ EL. 100': Col. Line 6-9/D-H"} \\ \text{"Case 1 (E-W): 18" roof @ EL. 100': Col. Line 6-9/E-G"} \\ \text{"Case 2 (N-S): 18" roof @ EL. 72': Col. Line 9-12/E-G"} \\ \text{"Case 2 (E-W): 18" roof @ EL. 72': Col. Line 9-12/E-G"} \\ \text{"Case 3 (N-S): 18" roof slab @ EL. 64': Col. Line 9-12/D-E(\& G-H)"} \\ \text{"Case 3 (E-W): 18" roof slab @ EL. 64': Col. Line 9-12/D-E(\& G-H)"} \\ \text{"Case 4 (N-S): 18" roof slab at EL. 64': Col. Line 2-3/D-E(\& G-H)"} \\ \text{"Case 4 (E-W): 18" roof slab at EL. 64': Col. Line 2-6/D-E(\& G-H)"} \end{pmatrix}$$

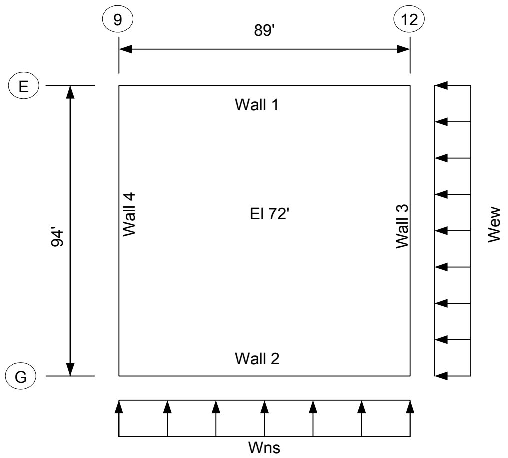


Case 1 : N/S

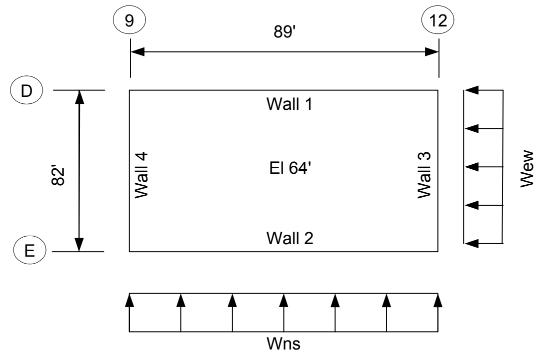


Case 1 : E/W

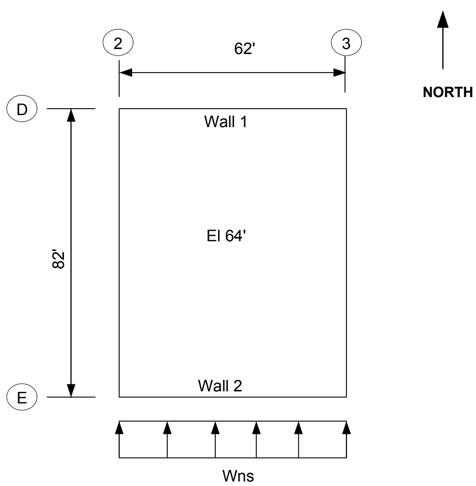
Note : For case 1 in the E/W direction the diaphragm is a 3 span system. (Ref. 2.2.11)
Conservatively take diaphragm as simple span using the largest span. (Assumption 3.2.3)



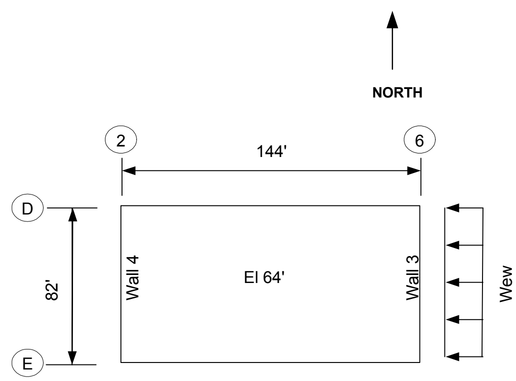
Case 2



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. 2.2.10)
Conservatively take diaphragm as simple span using the largest span. (Assumption 3.2.3)

Diaphragm horizontal span

Diaphragm depth

Chord Steel (Total # of bars)
- All chord steel for Cases 1 to 4 is # 9 bars per Ref. 2.2.30, Section 6.6.1.7

$$span_{hor} := \begin{pmatrix} 94ft \\ 94ft \\ 89ft \\ 94ft \\ 89ft \\ 82ft \\ 62ft \\ 82ft \end{pmatrix} \qquad depth := \begin{pmatrix} 258ft \\ 94ft \\ 94ft \\ 89ft \\ 82ft \\ 89ft \\ 82ft \\ 144ft \end{pmatrix} \qquad As_{chord} := \begin{pmatrix} 18 \\ 20 \\ 14 \\ 19 \\ 12 \\ 12 \\ 6 \\ 12 \end{pmatrix} \cdot 1.00in^2$$

Slab Steel

All slab steel for 18" slabs is # 7 bars @ 12" o.c., both ways, top & bottom (Ref. 2.2.30, Section 7.1.1)

$$As_{top} := \frac{A_{sbar}(7)}{ft} \qquad As_{top} = 0.60\ \frac{1}{ft}\ in^2$$

$$As_{bot} := \frac{A_{sbar}(7)}{ft} \qquad As_{bot} = 0.60\ \frac{1}{ft}\ in^2$$

Maximum Horizontal Acceleration at diaphragm elevation

Per. Section 6.3.1.1, the maximum horizontal accelerations are EL. 32', 64', 72', and 100' are as follows:

$$Acc_h = \begin{pmatrix} 1.31 \\ 1.56 \\ 1.60 \\ 1.83 \end{pmatrix} g \qquad \begin{array}{l} \text{EL. 32'} \\ \text{EL. 64'} \\ \text{EL. 72'} \\ \text{EL. 100'} \end{array}$$

$$ah_{1to4} := \begin{pmatrix} Acc_{h_4} \\ Acc_{h_4} \\ Acc_{h_3} \\ Acc_{h_3} \\ Acc_{h_2} \\ Acc_{h_2} \\ Acc_{h_2} \\ Acc_{h_2} \end{pmatrix} \qquad ah_{1to4} = \begin{pmatrix} 1.83 \\ 1.83 \\ 1.60 \\ 1.60 \\ 1.56 \\ 1.56 \\ 1.56 \\ 1.56 \end{pmatrix} g$$

Max. horizontal BDBGM accelerations at the respective elevations for analysis cases 1 to 4

Governing Design Loads

Combine dead load and 25% of design live load for seismic load combination

$$w_{uNS} = \begin{pmatrix} 378.8 \\ 378.8 \\ 596.3 \\ 685.0 \\ 460.0 \end{pmatrix} psf \qquad Cases = \begin{pmatrix} \text{"18" roof slab at EL. 100'"} \\ \text{"18" floor slab at EL. 32'"} \\ \text{"33" roof slab at EL. 64'"} \\ \text{"48" floor slab at EL. 32"} \\ \text{"30" floor slab at EL. 32"} \end{pmatrix}$$

See Section 6.3.1.1 for calculation of w_{uNS}

$$w_{u1to4} := w_{uNS_1} \qquad w_{u1to4} = 378.8\,psf$$

Non-Seismic load for 18" roof slab applies for analysis cases 1 to 4

Wall Weight Tributary to Diaphragm

- For seismic excitation in N-S dir., Section 6.4.3.3 of Ref. 2.2.30 provides the wall weight trib. to the diaphragms for analysis cases 1 to 4

- For seismic excitation in E-W dir., Section 6.4.4.3 of Ref. 2.2.30 provides the wall weight trib. to the diaphragms for analysis cases 1 to 4

$$weight_{trib1to4} := \begin{pmatrix} 40.8 \\ 19.2 \\ 26.2 \\ 38.4 \\ 21.6 \\ 30.0 \\ 19.2 \\ 49.2 \end{pmatrix} \cdot \frac{kip}{ft}$$

6.3.2.2 In-Plane Moment and Shear Demands for Analysis Cases 1 to 4

- Treat the diaphragm as a deep beam with length = $span_{hor}$.
- Uniform Seismic Load on beam w = ((DL + 0.25LL)*depth + $weight_{trib}$) * horizontal acceleration/g

$$w_{diaphragm1to4} := \overrightarrow{\left[\left(w_{u1to4} \cdot depth + weight_{trib1to4}\right) \cdot \frac{ah_{1to4}}{g}\right]}$$

$$w_{diaphragm1to4}^{T} = (253.5 \quad 100.3 \quad 98.9 \quad 115.4 \quad 82.1 \quad 99.4 \quad 78.4 \quad 161.8)\,klf$$

Diaphragm Moment

$$Mom_{E1to4} := \overrightarrow{\frac{w_{diaphragm1to4} \cdot span_{hor}^{2}}{8}}$$

$$Mom_{E1to4}^{T} = (279976 \quad 110769 \quad 97908 \quad 127431 \quad 81335 \quad 83534 \quad 37672 \quad 136022)\,kip \cdot ft$$

Chord Force - determined by dividing the diaphragm moment by a lever arm equal to 90% of the diaphragm depth (i.e. the chord steel is provided over a width equal to 10% of the diaphragm depth per Ref. 2.2.30 and the distance between the center of reinforcing and the center of the compression block is approximated as 0.9*d)

$$Chord_{E} := \frac{\overrightarrow{Mom_{E1to4}}}{0.9 \cdot depth}$$

$$Chord_{E}^{T} = (1205.8 \quad 1309.3 \quad 1157.3 \quad 1590.9 \quad 1102.1 \quad 1042.9 \quad 510.5 \quad 1049.6)\,kip$$

Diaphragm Shear

$$Shear_{E} := \overrightarrow{\frac{w_{diaphragm1to4} \cdot span_{hor}}{2}}$$

$$Shear_{E}^{T} = (11914 \quad 4714 \quad 4400 \quad 5423 \quad 3655 \quad 4075 \quad 2430 \quad 6635)\,kip$$

6.3.2.3 Design Capacities ($C_{98\%}$)

- The in-plane moment demand on the diaphragm (translated to a chord force) is carried by the cord steel and slab steel within an area equal to 10% of the diaphragm depth.
- The in-plane shear demand on the diaphragm is carried by the concrete and the slab steel along the depth of the diaphragm.
- A check is made to show that the in-plane shear capacity of the concrete alone can carry 40% of the seismic demand. This check will ensure that the slab steel considered in both the in-plane moment and in-plane shear capacity is not considered twice.

Chord Capacity

The moment demand is translated to a chord force carried by the chord steel and the slab steel located within 10% of the diaphragm depth. The capacity of this steel is used in the fragility evaluation for in-plane moment on the diaphragms.

$\phi := 0.9$ Strength reduction factor for bending per ACI 349-01 (Ref. 2.2.2 Section 9.3.2.1)

$$As_{chord}^{T} = (18.0\ \ 20.0\ \ 14.0\ \ 19.0\ \ 12.0\ \ 12.0\ \ 6.0\ \ 12.0)\,in^2$$

$$\phi Tn_{chord} := \overrightarrow{\left(\phi \cdot As_{chord} \cdot f_y\right)} \qquad \phi Tn_{slab} := \left[\phi \cdot \left(As_{top} + As_{bot}\right) \cdot f_y\right] \cdot 0.10 \cdot depth$$

Slab steel capacity within 10% of the diaphragm depth

$$\phi Tn_{chord} = \begin{pmatrix} 972 \\ 1080 \\ 756 \\ 1026 \\ 648 \\ 648 \\ 324 \\ 648 \end{pmatrix} kip$$

Chord steel capacity

$$\phi Tn_{slab} = \begin{pmatrix} 1672 \\ 609 \\ 609 \\ 577 \\ 531 \\ 577 \\ 531 \\ 933 \end{pmatrix} kip$$

Slab steel capacity within 10% of depth

In-Plane Shear Capacity of Diaphragms

$h_w := span_{hor} \qquad l_w := depth$ height and length of diaphragms

Steel Reinforcement Ratio: slab steel is the same in both directions

$$\rho_v := \overrightarrow{\frac{As_{top} + As_{bot}}{thick}} \qquad \rho_v = 0.0056$$

$$\rho_h := \rho_v \qquad \rho_h = 0.0056$$

Concrete Shear Capacity

$N_a := 0kip$ Neglect in-plane compression/tension forces in diaphragms

$$\nu_{cBarda}(i) := \left[8.3\cdot\sqrt{f_c\cdot psi} - 3.4\cdot\sqrt{f_c\cdot psi}\cdot\left(\frac{h_{w_i}}{l_{w_i}} - 0.5\right) + \frac{N_a}{4\cdot l_{w_i}\cdot thick}\right]$$

For hw/lw < 2.0; use Barda (Eq. 6.2.1 in Section 6.2.4)

$$\nu_{c349}(i) := \left(2\cdot\sqrt{f_c\cdot psi}\right)$$

For hw/lw > 2.0; use ACI 349-01 capacity (Eq. 21-6 in Section 21.6.5.2 of Ref. 2.2.2)

Steel Shear Capacity

$$A(h,l) := \begin{cases} 1.0 & \text{if } \frac{h}{l} \le 0.5 \\ 0.0 & \text{if } \frac{h}{l} \ge 1.5 \\ \frac{h}{l}\cdot -1 + 1.5 & \text{otherwise} \end{cases}$$

$$B(h,l) := \begin{cases} 0.0 & \text{if } \frac{h}{l} \le 0.5 \\ 1.0 & \text{if } \frac{h}{l} \ge 1.5 \\ \frac{h}{l} - 0.5 & \text{otherwise} \end{cases}$$

$$A\left(h_{w_1}, l_{w_1}\right) = 1.00 \qquad B\left(h_{w_1}, l_{w_1}\right) = 0.00$$

$$A\left(h_{w_3}, l_{w_3}\right) = 0.55 \qquad B\left(h_{w_3}, l_{w_3}\right) = 0.45$$

$$\rho_{seBarda}(i) := \min\left(0.01, A\left(h_{w_i}, l_{w_i}\right)\cdot\rho_v + B\left(h_{w_i}, l_{w_i}\right)\cdot\rho_h\right)$$

Limit steel ratio 1%

$$\rho_{se349}(i) := \rho_h$$

$$V_{sBarda}(i) := \rho_{seBarda}(i)\cdot f_y \qquad \nu_{cBarda}(1) = 649.8\ psi$$

$$V_{s349}(i) := \rho_{se349}(i)\cdot f_y \qquad \nu_{c349}(1) = 148.3\ psi$$

Total Shear Capacity

$$\phi V_n := \left| \begin{array}{l} \text{for } i \in 1 .. \text{case\_num} \\ \quad \left| \begin{array}{l} \text{if } \dfrac{h_{w_i}}{l_{w_i}} \le 2.0 \\ \quad \left| \begin{array}{l} a \leftarrow \nu_{cBarda}(i) \\ b \leftarrow V_{sBarda}(i) \\ v_{tot} \leftarrow \min\left(a + b, 20 \cdot \sqrt{f_c \cdot psi}\right) \\ Area \leftarrow 0.6 \cdot l_{w_i} \cdot thick \\ \phi \leftarrow 0.80 \\ \phi V_{n_i} \leftarrow \phi \cdot v_{tot} \cdot Area \end{array} \right. \\ \text{otherwise} \\ \quad \left| \begin{array}{l} a \leftarrow \nu_{c349}(i) \\ b \leftarrow V_{s349}(i) \\ v_{tot} \leftarrow \min\left(a + b, 8 \cdot \sqrt{f_c \cdot psi}\right) \\ Area \leftarrow l_{w_i} \cdot thick \\ \phi \leftarrow 0.60 \\ \phi V_{n_i} \leftarrow \phi \cdot v_{tot} \cdot Area \end{array} \right. \end{array} \right. \\ \phi V_n \end{array} \right.$$

$$\text{case\_num} = 8.0$$

Description
This loop determines the shear capacity for each diaphragm case. For hw/lw <= 2.0, the shear capacity is determined using the Barda equation. For hw/lw > 2.0, the shear capacity is determined using ACI 349-01 equations.

$$\phi V_n = \begin{pmatrix} 26297 \\ 8019 \\ 8150 \\ 7462 \\ 6812 \\ 7775 \\ 7518 \\ 13905 \end{pmatrix} \text{kip}$$

Total in-plane shear capacity of the diaphragms

Concrete Shear Capacity and 40% Seismic Demand

Determine if concrete capacity is enough to carry 40% of the in-plane diaphragm shear.

$$
\begin{aligned}
\phi V_{conc} :=\ & \text{for } i \in 1 .. \text{case\_num} \\
& \quad \text{if } \frac{h_{w_i}}{l_{w_i}} \le 2.0 \\
& \quad\quad a \leftarrow v_{cBarda}(i) \\
& \quad\quad b \leftarrow 0.0\text{psi} \\
& \quad\quad v_{tot} \leftarrow \min\left(a + b, 20 \cdot \sqrt{f_c \cdot \text{psi}}\right) \\
& \quad\quad \text{Area} \leftarrow 0.6 \cdot l_{w_i} \cdot \text{thick} \\
& \quad\quad \phi \leftarrow 0.80 \\
& \quad\quad \phi V_{n_i} \leftarrow \phi \cdot v_{tot} \cdot \text{Area} \\
& \quad \text{otherwise} \\
& \quad\quad a \leftarrow v_{c349}(i) \\
& \quad\quad b \leftarrow 0.0\text{psi} \\
& \quad\quad v_{tot} \leftarrow \min\left(a + b, 8 \cdot \sqrt{f_c \cdot \text{psi}}\right) \\
& \quad\quad \text{Area} \leftarrow l_{w_i} \cdot \text{thick} \\
& \quad\quad \phi \leftarrow 0.60 \\
& \quad\quad \phi V_{n_i} \leftarrow \phi \cdot v_{tot} \cdot \text{Area} \\
& \phi V_n
\end{aligned}
$$

Description

This loop determines the shear capacity for each diaphragm case considering only the contribution of the concrete. For hw/lw <= 2.0, the shear capacity is determined using the Barda equation. For hw/lw > 2.0, the shear capacity is determined using ACI 349-01 equations.

$$
\phi V_{conc} = \begin{pmatrix} 17380 \\ 4770 \\ 4901 \\ 4386 \\ 3978 \\ 4700 \\ 4684 \\ 8929 \end{pmatrix} \text{kip}
$$

In-plane shear capacity of the diaphragm concrete

$$\frac{0.4 \cdot Shear_E}{\phi V_{conc}} = \begin{pmatrix} 0.27 \\ 0.40 \\ 0.36 \\ 0.49 \\ 0.37 \\ 0.35 \\ 0.21 \\ 0.30 \end{pmatrix}$$

All D/C ratios are less than 1.0. Therefore, the distributed slab steel is not required to carry the shear due to the 40% seismic load and the full distributed slab steel can be used to carry the chord force from the 100% seismic load.

6.3.2.4 Strength Margin Factor

Per Equation 4-2 of Section 4.3.2 of this calculation -

$$Fs_{chord} := \overrightarrow{\frac{\phi Tn_{chord} + \phi Tn_{slab} - 0kip}{Chord_E}}$$

Chord force due to non-seismic demand is negligible and is set equal to 0 kips and all slab steel within 10% of the diaphragm depth is considered.

$$Fs_{chord}^{T} = (2.19 \quad 1.29 \quad 1.18 \quad 1.01 \quad 1.07 \quad 1.17 \quad 1.68 \quad 1.51)$$

$$Fs_{shear} := \overrightarrow{\frac{\phi V_n - 0kip}{Shear_E}}$$

In-plane shear force due to non-seismic demand is negligible and is set equal to 0 kips.

$$Fs_{shear}^{T} = (2.21 \quad 1.70 \quad 1.85 \quad 1.38 \quad 1.86 \quad 1.91 \quad 3.09 \quad 2.10)$$

6.3.2.5 Inelastic Energy Absorption Factor - F_μ)

- $F\mu$ factors for in-plane shear of diaphragms is determined from Table 5-1 of Ref. 2.2.6 for Limit State A for reinforced concrete shear walls.
- $F\mu$ factors for in-plane bending of diaphragms is determined from Table 5-1 of Ref. 2.2.6 for Limit State A for slab/wall moment frames, beams and walls of reinforced concrete.

$$\frac{span_{hor}}{depth}^{T} = (0.4 \quad 1.0 \quad 0.9 \quad 1.1 \quad 1.1 \quad 0.9 \quad 0.8 \quad 0.6)$$

All span-to-depth ratios (identical to hw/lw terminology for walls) are less than 2.0. Therefore, use $F\mu$ = 2.0 for in-plane shear and $F\mu$ = 2.25 for in-plane bending (Cord Force)

$F_{\mu shear} := 2.0 \qquad F_{\mu chord} := 2.25$ Table 5-1 of Ref. 2.2.6 for Limit State A

6.3.2.6 HCLPF for In-Plane Bending and Shear of Diaphragms

$$HCLPF_{chord} := Fs_{chord} \cdot F_{\mu chord} \cdot PGA_h$$

$$HCLPF_{chord}{}^{T} = (4.51 \;\; 2.65 \;\; 2.43 \;\; 2.07 \;\; 2.20 \;\; 2.41 \;\; 3.45 \;\; 3.10)\,g$$

$$HCLPF_{shear} := Fs_{shear} \cdot F_{\mu shear} \cdot PGA_{h}$$

$$HCLPF_{shear}{}^{T} = (4.03 \;\; 3.11 \;\; 3.38 \;\; 2.51 \;\; 3.41 \;\; 3.49 \;\; 5.65 \;\; 3.83)\,g$$

$$HCLPF_{Case1to4Bending} := \min\left(HCLPF_{chord}\right)$$

$$HCLPF_{Case1to4Shear} := \min\left(HCLPF_{shear}\right)$$

$$HCLPF_{Case1to4Bending} = 2.07\,g$$

$$HCLPF_{Case1to4Shear} = 2.51\,g$$

Minimum HCLPF for in-plane bending and in-plane shear of the diaphragms for Analysis Cases 1 to 4

$Cases_{1to4_{4}}$ = "Case 2 (E-W): 18" roof @ EL. 72': Col. Line 9-12/E-G"

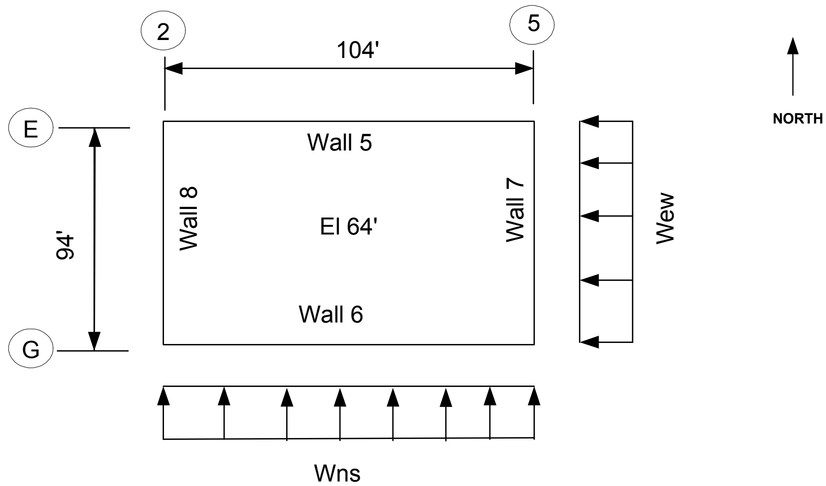
$$HCLPF_{chord_{4}} = 2.07\,g$$

6.3.2.7 Diaphragms Properties and Loads for Analysis Cases 5 to 8

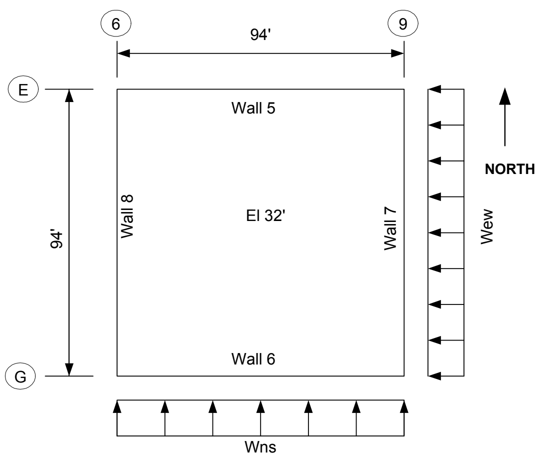
case_num := 8

$Cases_{5to8}$:=

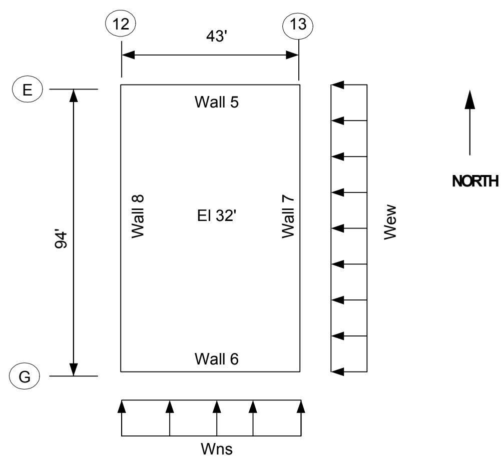
"Case 5 (N-S): 33" roof @ EL. 64': Col. Line 2-5/E-G"
"Case 5 (E-W): 33" roof @ EL. 64': Col. Line 2-5/E-G"
"Case 6 (N-S): 48" floor @ EL. 32': Col. Line 6-9/E-G"
"Case 6 (E-W): 48" floor @ EL. 32': Col. Line 6-9/E-G"
"Case 7 (N-S): 18" roof @ EL. 32': Col. Line 12-13(& 1-2)/E-G"
"Case 7 (E-W): 18" roof @ EL. 32': Col. Line 12-13(& 1-2)/E-G"
"Case 8 (N-S): 18" floor at EL. 32': Col. Line 2-3/D-E(& G-H)"
"Case 8 (E-W): 18" floor at EL. 32': Col. Line 2-12/D-E(& G-H)"



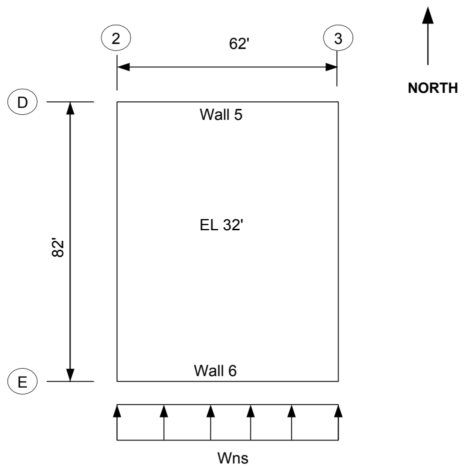
Case 5

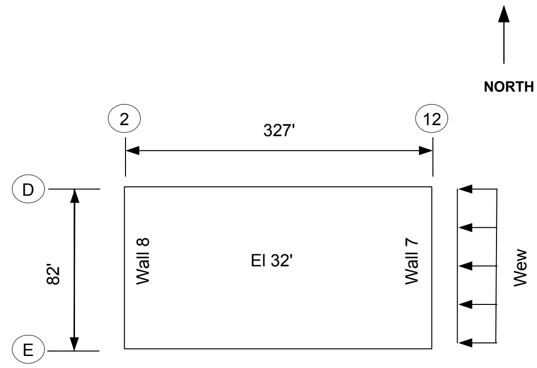


Case 6



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. 2.2.9)
Conservatively take diaphragm as simple span using the largest span. (Assumption 3.2.3)

$$\text{thick} := \begin{pmatrix} 2.75 \\ 2.75 \\ 4 \\ 4 \\ 1.5 \\ 1.5 \\ 1.5 \\ 1.5 \end{pmatrix} \cdot \text{ft}$$

Slab thickness for analysis cases 5 to 8

Diaphragm horizontal span

Diaphragm depth

Chord Steel (Total # of bars)
- All chord steel for Cases 5 to 8 is # 9 bars per Ref. 2.2.30, Section 6.6.2.7

$$\text{span}_{\text{hor}} := \begin{pmatrix} 104 \\ 94 \\ 94 \\ 94 \\ 43 \\ 94 \\ 62 \\ 82 \end{pmatrix} \cdot \text{ft} \qquad \text{depth} := \begin{pmatrix} 94 \\ 104 \\ 94 \\ 94 \\ 94 \\ 43 \\ 82 \\ 327 \end{pmatrix} \cdot \text{ft} \qquad \text{As}_{\text{chord}} := \begin{pmatrix} 22 \\ 18 \\ 22 \\ 25 \\ 2 \\ 16 \\ 7 \\ 10 \end{pmatrix} \cdot 1.00\text{in}^2$$

Slab Steel
- Slab steel for Analysis Cases 5 to 8 from Ref. 2.2.30, Section 7.1.1.

- Top and bottom steel is the same per Ref. 2.2.30, Section 7.1.1

$$As_{top} := \begin{pmatrix} A_{sbar}(9) \\ A_{sbar}(9) \\ A_{sbar}(10)\cdot 2 \\ A_{sbar}(10)\cdot 2 \\ A_{sbar}(7) \\ A_{sbar}(7) \\ A_{sbar}(7) \\ A_{sbar}(7) \end{pmatrix} \cdot \frac{1}{ft}$$

Case 5: #9 @ 12" top and bottom, each way

Case 6: #10 @ 6" top and bottom, each way

Case 7 & 8: #7 @ 12" top and bottom, each way

$$As_{top} = \begin{pmatrix} 1.00 \\ 1.00 \\ 2.54 \\ 2.54 \\ 0.60 \\ 0.60 \\ 0.60 \\ 0.60 \end{pmatrix} \frac{1}{ft} in^2$$

$$As_{bot} := As_{top} \qquad As_{bot}^{T} = (1.00 \;\; 1.00 \;\; 2.54 \;\; 2.54 \;\; 0.60 \;\; 0.60 \;\; 0.60 \;\; 0.60)\frac{in^2}{ft}$$

<u>Maximum Horizontal Acceleration at diaphragm elevation</u>

Per. Section 6.3.1.1, the maximum horizontal accelerations are EL. 32', 64', 72', and 100' are as follows:

$$Acc_h = \begin{pmatrix} 1.31 \\ 1.56 \\ 1.60 \\ 1.83 \end{pmatrix} g \qquad \begin{matrix} \text{EL. 32'} \\ \text{EL. 64'} \\ \text{EL. 72'} \\ \text{EL. 100'} \end{matrix}$$

$$ah_{5to8} := \begin{pmatrix} Acc_{h_2} \\ Acc_{h_2} \\ Acc_{h_1} \\ Acc_{h_1} \\ Acc_{h_1} \\ Acc_{h_1} \\ Acc_{h_1} \\ Acc_{h_1} \end{pmatrix} \qquad ah_{5to8} = \begin{pmatrix} 1.56 \\ 1.56 \\ 1.31 \\ 1.31 \\ 1.31 \\ 1.31 \\ 1.31 \\ 1.31 \end{pmatrix} g$$

Max. horizontal BDBGM accelerations at the respective elevations for analysis cases 5 to 8

<u>Governing Design Loads</u>

Combine dead load and 25% of design live load for seismic load combination

$$w_{uNS} = \begin{pmatrix} 378.8 \\ 378.8 \\ 596.3 \\ 685.0 \\ 460.0 \end{pmatrix} psf \qquad Cases = \begin{pmatrix} \text{"18" roof slab at EL. 100'"} \\ \text{"18" floor slab at EL. 32'"} \\ \text{"33" roof slab at EL. 64'"} \\ \text{"48" floor slab at EL. 32"} \\ \text{"30" floor slab at EL. 32"} \end{pmatrix}$$

See Section 6.3.1.1 for definition of w_{uNS}

$$w_{u5to8} := \begin{pmatrix} w_{uNS_3} \\ w_{uNS_3} \\ w_{uNS_4} \\ w_{uNS_4} \\ w_{uNS_1} \\ w_{uNS_1} \\ w_{uNS_2} \\ w_{uNS_2} \end{pmatrix}$$

Case 5: EL. 64' roof slab

Case 6: EL. 32' 48" slab

Case 7: EL. 100' roof slab

Case 7: EL. 32' 18" roof slab

$$w_{u5to8} = \begin{pmatrix} 596.3 \\ 596.3 \\ 685.0 \\ 685.0 \\ 378.8 \\ 378.8 \\ 378.8 \\ 378.8 \end{pmatrix} psf$$

Non-Seismic load for analysis cases 5 to 8

Wall Weight Tributary to Diaphragm

- For seismic excitation in N-S dir., Section 6.5.3.3 of Ref. 2.2.30 provides the wall weight trib. to the diaphragms for analysis cases 5 to 8
- For seismic excitation in E-W dir., Section 6.5.4.3 of Ref. 2.2.30 provides the wall weight trib. to the diaphragms for analysis cases 5 to 8

$$weight_{trib5to8} := \begin{pmatrix} 27.9 \\ 28.8 \\ 61.7 \\ 83.3 \\ 9.60 \\ 26.4 \\ 38.4 \\ 129.6 \end{pmatrix} \cdot \frac{kip}{ft}$$

6.3.2.8 In-Plane Moment and Shear Demands for Analysis Cases 5 to 8

- Treat the diaphragm as a deep beam with length = $span_{hor}$.
- Uniform Seismic Load on beam w = ((DL + 0.25LL)*depth + $weight_{trib}$) * horizontal acceleration/g

$$w_{diaphragm5to8} := \overrightarrow{\left[\left(w_{u5to8} \cdot depth + weight_{trib5to8}\right) \cdot \frac{ah_{5to8}}{g}\right]}$$

$$w_{diaphragm5to8}^{T} = (131.0 \quad 141.7 \quad 165.2 \quad 193.5 \quad 59.2 \quad 55.9 \quad 91.0 \quad 332.0)\,klf$$

Diaphragm Moment $$Mom_E := \overrightarrow{\frac{w_{diaphragm5to8} \cdot span_{hor}^{2}}{8}}$$

$$Mom_E^{T} = (177055 \quad 156467 \quad 182439 \quad 213692 \quad 13686 \quad 61763 \quad 43720 \quad 279064)\,kip \cdot ft$$

Chord Force - determined by dividing the diaphragm moment by a lever arm equal to 90% of the diaphragm depth (i.e. the chord steel is provided over a width equal to 10% of the diaphragm depth per Ref. 2.2.30)

$$Chord_E := \overrightarrow{\frac{Mom_E}{0.9 \cdot depth}}$$

$$Chord_E^{T} = (2092.9 \quad 1671.7 \quad 2156.5 \quad 2525.9 \quad 161.8 \quad 1595.9 \quad 592.4 \quad 948.2)\,kip$$

Diaphragm Shear $$Shear_E := \overrightarrow{\frac{w_{diaphragm5to8} \cdot span_{hor}}{2}}$$

$$Shear_E^{T} = (6810 \quad 6658 \quad 7763 \quad 9093 \quad 1273 \quad 2628 \quad 2821 \quad 13613)\,kip$$

6.3.2.9 Design Capacities ($C_{98\%}$)

- The in-plane moment demand on the diaphragm (translated to a chord force) is carried by the cord steel and slab steel within an area equal to 10% of the diaphragm depth.
- The in-plane shear demand on the diaphragm is carried by the concrete and the slab steel along the depth of the diaphragm.
- A check is made to show that the in-plane shear capacity of the concrete alone can carry 40% of the seismic demand. This check will ensure that the slab steel considered in both the in-plane moment and in-plane shear capacity is not considered twice.

Chord Capacity
The moment demand is translated to a chord force carried by the chord steel and the slab steel located within 10% of the diaphragm depth. The capacity of this steel is used in the fragility evaluation for in-plane moment on the diaphragms.

$\phi := 0.9$ Strength reduction factor for bending per ACI 349-01 (Ref. 2.2.2 Section 9.3.2.1)

$$As_{chord}^{T} = (22.0 \quad 18.0 \quad 22.0 \quad 25.0 \quad 2.0 \quad 16.0 \quad 7.0 \quad 10.0)\,in^2$$

$$\phi Tn_{chord} := \overrightarrow{\left(\phi \cdot As_{chord} \cdot f_y\right)}$$

$$\phi Tn_{slab} := \overrightarrow{\left[\left[\phi\cdot\left(As_{top} + As_{bot}\right)\cdot f_y\right]\cdot 0.10\cdot depth\right]}$$

Slab steel capacity within 10% of the diaphragm depth

$$\phi Tn_{chord} = \begin{pmatrix} 1188 \\ 972 \\ 1188 \\ 1350 \\ 108 \\ 864 \\ 378 \\ 540 \end{pmatrix} kip$$

Chord steel capacity

$$\phi Tn_{slab} = \begin{pmatrix} 1015 \\ 1123 \\ 2579 \\ 2579 \\ 609 \\ 279 \\ 531 \\ 2119 \end{pmatrix} kip$$

Slab steel capacity within 10% of depth

In-Plane Shear Capacity

$h_w := span_{hor}$ $\quad l_w := depth$ height and length of diaphragms

Steel Reinforcement Ratio: slab steel is the same in both directions

$$\rho_v := \overrightarrow{\frac{As_{top} + As_{bot}}{thick}}$$

$$\rho_h := \rho_v$$

$$\rho_v = \begin{pmatrix} 0.0051 \\ 0.0051 \\ 0.0088 \\ 0.0088 \\ 0.0056 \\ 0.0056 \\ 0.0056 \\ 0.0056 \end{pmatrix} \qquad \rho_h = \begin{pmatrix} 0.0051 \\ 0.0051 \\ 0.0088 \\ 0.0088 \\ 0.0056 \\ 0.0056 \\ 0.0056 \\ 0.0056 \end{pmatrix}$$

Concrete Shear Capacity

$N_a := 0kip$ Neglect in-plane compression/tension forces in diaphragms

$$\nu_{cBarda}(i) := \left[8.3\cdot\sqrt{f_c\cdot psi} - 3.4\cdot\sqrt{f_c\cdot psi}\cdot\left(\frac{h_{w_i}}{l_{w_i}} - 0.5\right) + \frac{N_a}{4\cdot l_{w_i}\cdot thick_i}\right]$$

For hw/lw < 2.0; use Barda (Eq. 6.2.1 in Section 6.2.4)

$$\nu_{c349}(i) := \left(2\cdot\sqrt{f_c\cdot psi}\right)$$

For hw/lw > 2.0; use ACI 349 capacity (Eq. 21-6 in Section 21.6.5.2 of Ref. 2.2.2)

Steel Shear Capacity

$$A(h,l) := \begin{cases} 1.0 & \text{if } \frac{h}{l} \le 0.5 \\ 0.0 & \text{if } \frac{h}{l} \ge 1.5 \\ \frac{h}{l} \cdot -1 + 1.5 & \text{otherwise} \end{cases} \qquad B(h,l) := \begin{cases} 0.0 & \text{if } \frac{h}{l} \le 0.5 \\ 1.0 & \text{if } \frac{h}{l} \ge 1.5 \\ \frac{h}{l} - 0.5 & \text{otherwise} \end{cases}$$

$$A\left(h_{w_1}, l_{w_1}\right) = 0.39 \qquad B\left(h_{w_1}, l_{w_1}\right) = 0.61$$

$$A\left(h_{w_3}, l_{w_3}\right) = 0.50 \qquad B\left(h_{w_3}, l_{w_3}\right) = 0.50$$

$$\rho_{seBarda}(i) := \min\left(0.01, A\left(h_{w_i}, l_{w_i}\right)\cdot \rho_{v_i} + B\left(h_{w_i}, l_{w_i}\right)\cdot \rho_{h_i}\right)$$ Limit steel ratio 1%

$$\rho_{se349}(i) := \rho_{h_i}$$

$$V_{sBarda}(i) := \rho_{seBarda}(i)\cdot f_y$$

$$V_{s349}(i) := \rho_{se349}(i)\cdot f_y$$

Total Shear Capacity

$$
\begin{aligned}
\phi V_n := \; & \text{for } i \in 1 .. \text{case\_num} \\
& \quad \text{if } \frac{h_{w_i}}{l_{w_i}} \le 2.0 \\
& \quad\quad a \leftarrow v_{cBarda}(i) \\
& \quad\quad b \leftarrow V_{sBarda}(i) \\
& \quad\quad v_{tot} \leftarrow \min\left(a + b, 20 \cdot \sqrt{f_c \cdot psi}\right) \\
& \quad\quad Area \leftarrow 0.6 \cdot l_{w_i} \cdot thick_i \\
& \quad\quad \phi \leftarrow 0.80 \\
& \quad\quad \phi V_{n_i} \leftarrow \phi \cdot v_{tot} \cdot Area \\
& \quad \text{otherwise} \\
& \quad\quad a \leftarrow v_{c349}(i) \\
& \quad\quad b \leftarrow V_{s349}(i) \\
& \quad\quad v_{tot} \leftarrow \min\left(a + b, 8 \cdot \sqrt{f_c \cdot psi}\right) \\
& \quad\quad Area \leftarrow l_{w_i} \cdot thick_i \\
& \quad\quad \phi \leftarrow 0.60 \\
& \quad\quad \phi V_{n_i} \leftarrow \phi \cdot v_{tot} \cdot Area \\
& \phi V_n
\end{aligned}
$$

$$\text{case\_num} = 8.0$$

Description
This loop determines the shear capacity for each diaphragm case. For hw/lw <= 2.0, the shear capacity is determined using the Barda equation. For hw/lw > 2.0, the shear capacity is determined using ACI 349-01 equations.

$$\phi V_n = \begin{pmatrix} 13681 \\ 16146 \\ 26473 \\ 26473 \\ 9352 \\ 2684 \\ 7518 \\ 34301 \end{pmatrix} \text{kip}$$

Total in-plane shear capacity of the diaphragms

Concrete Shear Capacity and 40% Seismic Demand

Determine if concrete capacity is enough to carry 40% of the in-plane diaphragm shear.

$$\phi V_{conc} := \left| \begin{array}{l} \text{for } i \in 1..\text{case\_num} \\ \quad \left| \begin{array}{l} \text{if } \dfrac{h_{w_i}}{l_{w_i}} \le 2.0 \\ \quad \left| \begin{array}{l} a \leftarrow \nu_{cBarda}(i) \\ b \leftarrow 0.0\text{psi} \\ v_{tot} \leftarrow \min\left(a+b, 20\cdot\sqrt{f_c\cdot \text{psi}}\right) \\ \text{Area} \leftarrow 0.6\cdot l_{w_i}\cdot \text{thick}_i \\ \phi \leftarrow 0.80 \\ \phi V_{n_i} \leftarrow \phi\cdot v_{tot}\cdot \text{Area} \end{array} \right. \\ \text{otherwise} \\ \quad \left| \begin{array}{l} a \leftarrow \nu_{c349}(i) \\ b \leftarrow 0.0\text{psi} \\ v_{tot} \leftarrow \min\left(a+b, 8\cdot\sqrt{f_c\cdot \text{psi}}\right) \\ \text{Area} \leftarrow l_{w_i}\cdot \text{thick}_i \\ \phi \leftarrow 0.60 \\ \phi V_{n_i} \leftarrow \phi\cdot v_{tot}\cdot \text{Area} \end{array} \right. \end{array} \right. \\ \phi V_n \end{array} \right.$$

Description

This loop determines the shear capacity for each diaphragm case considering only the contribution of the concrete. For hw/lw <= 2.0, the shear capacity is determined using the Barda equation. For hw/lw > 2.0, the shear capacity is determined using ACI 349-01 equations.

$$\phi V_{conc} = \begin{pmatrix} 8266 \\ 10155 \\ 12721 \\ 12721 \\ 6104 \\ 827 \\ 4684 \\ 23000 \end{pmatrix} \text{kip}$$

In-plane shear capacity of the diaphragm concrete

$$\frac{0.4 \cdot Shear_E}{\phi V_{conc}} = \begin{pmatrix} 0.33 \\ 0.26 \\ 0.24 \\ 0.29 \\ 0.08 \\ 1.27 \\ 0.24 \\ 0.24 \end{pmatrix}$$

$Cases5to8_6$ = "Case 7 (E-W): 18" roof @ EL. 32': Col. Line 12-13(& 1-2)/E-G"

All D/C ratios are less than 1.0 except for Case 7 in the E-W direction. That is, the diaphragm concrete in the E-W direction can not carry 40% of the BDBGM seismic force in that direction. Therefore, to prevent double-counting of the Case 7 slab steel capacity, the following steel reinforcement changes are made to these slabs -

- Increase the chord steel for N-S seismic excitation from 2 - #9 bars to 6 - #9 bars
- Increase the chord steel for E-W seismic excitation from 16 - #9 bars to 22 - #9 bars
- Decrease the steel spacing from #7 @ 12" top and bottom, each way to #7 @ 9" top and bottom, each way

For the fragility evaluation of Analysis Case 7 in the N-S direction, the slab steel is not considered to carry the chord force caused by the N-S seismic excitation. This will allow for the entire slab steel to be included in the in-plane shear capacity needed to transmit the E-W seismic force in the diaphragm.

For all other cases, the slab steel is not required to carry the shear due to the 40% seismic load and the full slab steel capacity can be used to carry the chord force from the 100% seismic load.

$As_{chord_5} := 6 \cdot 1.00in^2$ Increase N-S seismic acceleration chord steel to 6 - #9 bars

$As_{chord_6} := 22 \cdot 1.00in^2$ Increase E-W seismic acceleration chord steel to 22 - #9 bars

$$As_{top_5} := A_{sbar}(7) \cdot \frac{12}{9} \cdot \frac{1}{ft} \qquad As_{bot_5} := A_{sbar}(7) \cdot \frac{12}{9} \cdot \frac{1}{ft}$$

$$As_{top_6} := A_{sbar}(7) \cdot \frac{12}{9} \cdot \frac{1}{ft} \qquad As_{bot_6} := A_{sbar}(7) \cdot \frac{12}{9} \cdot \frac{1}{ft}$$

Increase the slab steel for Case 7 to #7 bars @ 9"

Redefine chord steel and slab steel axial capacity with new chord and slab steel for Case 7

$$\phi Tn_{chord} := \overrightarrow{\left(\phi \cdot As_{chord} \cdot f_y\right)}$$

$$\phi Tn_{slab} := \overrightarrow{\left[\left[\phi\cdot\left(As_{top} + As_{bot}\right)\cdot f_y\right]\cdot 0.10\cdot depth\right]}$$

Slab steel capacity within 10% of the diaphragm depth

Define total steel capacity used to carry the chord force

$$\phi Tn_{total} := \phi Tn_{chord} + \phi Tn_{slab}$$

$$\phi Tn_{total_5} := \phi Tn_{chord_5}$$

Do not use the slab steel to carry the N-S seismic acceleration chord force because this steel is required to carry the 40% seismic acceleration in the E-W direction.

$$\phi Tn_{total}{}^{T} = (2203.2 \quad 2095.2 \quad 3766.6 \quad 3928.6 \quad 324.0 \quad 1559.5 \quad 909.4 \quad 2659.0)\, kip$$

Redefine slab shear capacity with new slab steel for Case 7

$$\rho_v := \overrightarrow{\frac{As_{top} + As_{bot}}{thick}}$$

$$\rho_h := \rho_v$$

$$\rho_v = \begin{pmatrix} 0.0051 \\ 0.0051 \\ 0.0088 \\ 0.0088 \\ 0.0074 \\ 0.0074 \\ 0.0056 \\ 0.0056 \end{pmatrix} \qquad \rho_h = \begin{pmatrix} 0.0051 \\ 0.0051 \\ 0.0088 \\ 0.0088 \\ 0.0074 \\ 0.0074 \\ 0.0056 \\ 0.0056 \end{pmatrix}$$

$$\rho_{seBarda}(i) := \min\left(0.01, A\left(h_{w_i}, l_{w_i}\right)\cdot\rho_{v_i} + B\left(h_{w_i}, l_{w_i}\right)\cdot\rho_{h_i}\right)$$

Limit steel ratio 1%

$$\rho_{se349}(i) := \rho_{h_i}$$

$$V_{sBarda}(i) := \rho_{seBarda}(i)\cdot f_y$$

$$V_{s349}(i) := \rho_{se349}(i)\cdot f_y$$

Total Shear Capacity

$$\phi V_n := \begin{array}{|l} \text{for } i \in 1..\text{case\_num} \\ \quad \begin{array}{|l} \text{if } \dfrac{h_{w_i}}{l_{w_i}} \le 2.0 \\ \quad \begin{array}{|l} a \leftarrow v_{cBarda}(i) \\ b \leftarrow V_{sBarda}(i) \\ v_{tot} \leftarrow \min\left(a + b, 20\cdot\sqrt{f_c\cdot psi}\right) \\ Area \leftarrow 0.6\cdot l_{w_i}\cdot thick_i \\ \phi \leftarrow 0.80 \\ \phi V_{n_i} \leftarrow \phi\cdot v_{tot}\cdot Area \end{array} \\ \text{otherwise} \\ \quad \begin{array}{|l} a \leftarrow v_{c349}(i) \\ b \leftarrow V_{s349}(i) \\ v_{tot} \leftarrow \min\left(a + b, 8\cdot\sqrt{f_c\cdot psi}\right) \\ Area \leftarrow l_{w_i}\cdot thick_i \\ \phi \leftarrow 0.60 \\ \phi V_{n_i} \leftarrow \phi\cdot v_{tot}\cdot Area \end{array} \end{array} \\ \phi V_n \end{array}$$

$case\_num = 8.0$

Description
This loop determines the shear capacity for each diaphragm case. For hw/lw <= 2.0, the shear capacity is determined using the Barda equation. For hw/lw > 2.0, the shear capacity is determined using ACI 349-01 equations.

$$\phi V_n = \begin{pmatrix} 13681 \\ 16146 \\ 26473 \\ 26473 \\ 10435 \\ 3303 \\ 7518 \\ 34301 \end{pmatrix} kip$$

Total in-plane shear capacity of the diaphragms

6.3.2.10 Strength Margin Factor

Per Equation 4-2 of Section 4.3.2 of this calculation -

$$Fs_{chord} := \overrightarrow{\frac{\phi Tn_{total} - 0kip}{Chord_E}}$$

Chord force due to non-seismic demand is negligible and is set equal to 0 kips and all slab steel within 10% of the diaphragm depth is considered.

$$Fs_{chord}^{\ T} = \begin{pmatrix} 1.05 & 1.25 & 1.75 & 1.56 & 2.00 & 0.98 & 1.53 & 2.80 \end{pmatrix}$$

$$Fs_{shear} := \overrightarrow{\frac{\phi V_n - 0kip}{Shear_E}}$$

In-plane shear force due to non-seismic demand is negligible and is set equal to 0 kips.

$$Fs_{shear}^{T} = (2.01 \quad 2.42 \quad 3.41 \quad 2.91 \quad 8.20 \quad 1.26 \quad 2.67 \quad 2.52)$$

6.3.2.11 Inelastic Energy Absorption Factor - F_μ)

- $F\mu$ factors for in-plane shear of diaphragms is determined from Table 5-1 of Ref. 2.2.6 for Limit State A for reinforced concrete shear walls.
- $F\mu$ factors for in-plane bending of diaphragms is determined from Table 5-1 of Ref. 2.2.6 for Limit State A for slab/wall moment frames, beams and walls of reinforced concrete.

$$\frac{span_{hor}}{depth}^{T} = (1.1 \quad 0.9 \quad 1.0 \quad 1.0 \quad 0.5 \quad 2.2 \quad 0.8 \quad 0.3)$$

Except for Case 7 in the E-W direction, all span-to-depth ratios (identical to hw/lw terminology for walls) are less than 2.0. Use $F\mu$ = 2.0 for in-plane shear and $F\mu$ = 2.25 for in-plane bending (Cord Force) for all cases. Using a $F\mu$ = 2.0 for the shear of Case 7 in the E-W direction is conservative because Table 5-1 of Ref. 2.2.6 permits an $F\mu$ for bending controlled walls(slab) (i.e. hw/lw > 2) for Limit State A between 2.25 and 2.50.

$$F_{\mu shear} := 2.0 \qquad F_{\mu chord} := 2.25$$

Table 5-1 of Ref. 2.2.6 for Limit State A

6.3.2.12 HCLPF for In-Plane Bending and Shear of Diaphragms

Use the horizontal PGA for the in-plane diaphragm HCLPF calculations

$$HCLPF_{chord} := Fs_{chord} \cdot F_{\mu chord} \cdot PGA_h$$

$$HCLPF_{chord}^{T} = (2.16 \quad 2.58 \quad 3.59 \quad 3.20 \quad 4.12 \quad 2.01 \quad 3.16 \quad 5.77)\,g$$

$$HCLPF_{shear} := Fs_{shear} \cdot F_{\mu shear} \cdot PGA_h$$

$$HCLPF_{shear}^{T} = (3.67 \quad 4.43 \quad 6.23 \quad 5.32 \quad 14.98 \quad 2.30 \quad 4.87 \quad 4.61)\,g$$

$$HCLPF_{Case5to8Bending} := \min(HCLPF_{chord})$$

$$HCLPF_{Case5to8Shear} := \min(HCLPF_{shear})$$

$$HCLPF_{Case5to8Bending} = 2.01\,g$$

$$HCLPF_{Case5to8Shear} = 2.30\,g$$

Minimum HCLPF for in-plane bending and in-plane shear of the diaphragms for Analysis Cases 5 to 8

$Cases_{5to8_6}$ = "Case 7 (E-W): 18" roof @ EL. 32': Col. Line 12-13(& 1-2)/E-G"

6.3.3 HCLPF Capacity Evaluations for Diaphragms - Summary

The results from the HCLPF capacity evaluations for the CRCF diaphragms are as follows -

- The minimum HCLPF capacity for the out-of-plane bending failure mechanism of the CRCF diaphragms is

$HCLPF_{oopM} = 3.35\,g$ with an Fμ $F_{\mu oopM} = 1.31$

- The minimum HCLPF capacity for the out-of-plane shear failure mechanism of the CRCF diaphragms is

$HCLPF_{oopV} = 2.04\,g$ with an Fμ $F_{\mu oopS} = 1.00$

- The minimum HCLPF capacity for the in-plane failure mechanisms (in-plane bending and in-plane shear) of the CRCF diaphragms is

$$HCLPF_{ipb} := \min\left(HCLPF_{Case1to4Bending}, HCLPF_{Case5to8Bending}\right)$$

$HCLPF_{ipb} = 2.01\,g$ Minimum HCLPF capacity for in-plane bending of the CRCF diaphragms

$$F_{\mu chord} = 2.25$$

$$HCLPF_{ips} := \min\left(HCLPF_{Case1to4Shear}, HCLPF_{Case5to8Shear}\right)$$

$HCLPF_{ips} = 2.30\,g$ Minimum HCLPF capacity for in-plane shear of the CRCF diaphragms

$$F_{\mu shear} = 2.00$$

$$HCLPF_{ip} := \min\left(HCLPF_{ipb}, HCLPF_{ips}\right)$$

$HCLPF_{ip} = 2.01\,g$ Minimum HCLPF capacity for in-plane bending and in-plane shear of the CRCF diaphragms

- In order to achieve the above shown HCLPF capacities, the slab reinforcement and chord steel reinforcement of the CRCF must be changed from that established in Ref. 2.2.30. These changes are as follows -

  <u>18" roof slabs at EL. 32' between col. line 12-13/E-G and between col. line 1-2/E-G</u>

  <u>Chord Reinforcement</u>
  Provide 6 - #9 bars for N-S seismic excitation chord reinforcement
  Provide 22 - #9 bars for E-W seismic excitation chord reinforcement

  <u>Slab Reinforcement</u>
  Provide #7 @ 9" on centers, both ways, top and bottom

- The above listed HCLPF capacities are larger than the HCLPF capacity for the CRCF shear walls determined in Section 6.2.7. Therefore, the HCLPF capacity of the diaphragms is not the controlling HCLPF capacity of the CRCF.

6.4 HCLPF CAPACITY EVALUATIONS FOR STRUCTURAL STEEL

General Information and Data

$\mathrm{ORIGIN} := 1$ Set the array origin to 1

$PGA_h := 0.9138g$ BDBGM peak horizontal ground acceleration (Ref. 2.2.31)

Structural Steel Material Properties

$F_y := 50\mathrm{ksi}$ Structural steel yield strength for ASTM A992 (Ref. 2.2.1)

$F_u := 65\mathrm{ksi}$ Structural steel ultimate tensile strength for ASTM A992 (Ref. 2.2.1)

$F_{y36} := 36\mathrm{ksi}$ Structural plate yield strength, ASTM A36 for plate girders & stiffeners (Ref. 2.2.1)

$F_{u36} := 58\mathrm{ksi}$ Structural steel ultimate strength for ASTM A36 (Ref. 2.2.1)

$E_s := 29000\mathrm{ksi}$ Steel modulus of elasticity (Ref. 2.2.1)

$\rho_s := 490\mathrm{pcf}$ Unit weight of structural steel (Ref. 2.2.1)

Concrete Material Properties

$\rho_c := 150\mathrm{pcf}$ Unit weight of concrete (Ref. 2.2.1)

Design Loads

Dead Loads

$$DL_{slab} := \begin{pmatrix} 18\mathrm{in} + \dfrac{0.25\mathrm{ft}}{2} \\ 2.75\mathrm{ft} + \dfrac{0.25\mathrm{ft}}{2} \end{pmatrix} \cdot \rho_c$$

Concrete weight of 18" slab with 3" metal decking

Concrete weight of 33" slab with 3" metal decking

$$DL_{slab} = \begin{pmatrix} 243.75 \\ 431.25 \end{pmatrix} \mathrm{psf}$$

Miscellaneous Dead Loads

$DL_{equip} := 50\mathrm{psf}$ Equipment load (Assumption 3.1.4)

$DL_{hang} := 10\mathrm{psf}$ Miscellaneous hanging equipment load (Assumption 3.1.4)

$DL_{deck} := 3.58\mathrm{psf}$ 3" steel decking weight per Ref. 2.2.39.

$DL_{roof} := 15psf$ Roofing material load (Assumption 3.1.5)

Live Loads

$LL := 100psf$ Floor live load (Assumption 3.1.6)

$LL_r := 40psf$ Roof live load (Assumption 3.1.6)

Forklift Load

Per Assumption 3.1.10, the operation of the forklift is not included in the failure event sequence. Therefore, the forklift weight is not included for the beam calculations.

Seismic Loads

The following accelerations are the maximum vertical accelerations at each elevation due to the BDBGM_SRSS seismic load (Ref. 2.2.5). See Section 6.3.1 of this calculation for further discussion.

$$Acc_v := \begin{pmatrix} 1.06 \\ 1.07 \\ 1.05 \\ 1.10 \end{pmatrix} \cdot g \qquad \begin{matrix} \text{EL. 32'} \\ \text{EL. 64'} \\ \text{EL. 72'} \\ \text{EL. 100'} \end{matrix}$$

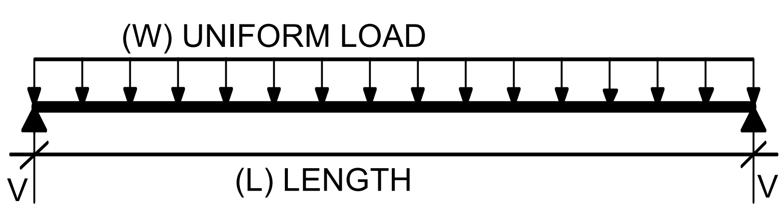
$amplify := 2.0$ Vertical amplification factor (Assumption 3.1.8)

Seismic Load Combination

$EQ_{LC} := DL + 0.25LL$ Per Ref. 2.2.4, 25% of the design live load is considered to act concurrently with the seismic load.

6.4.1 HCLPF Capacity Evaluations for Structural Beams

The beam cases for the HCLPF capacity evaluations are identical to the beam cases considered in the structural steel design calculation (Ref. 2.2.38). Cases 1 to 7 are roof beams while cases 8 to 10 are floor beams. Beam Case 11 (W18x35), consisting of the floor beams around the openings at EL. 32', is bounded by the other W18x35 at EL. 32' (Beam Case 8). Therefore, Beam Case 11 from Ref. 2.2.38 is not considered in the HCLPF capacity evaluations for the CRCF structural beams.



Typical Beam loading diagram for simply supported beam (Ref. 2.2.40)

$$Beam_{Cases} := \begin{pmatrix} \text{"B1 - EL. 64' - Gridline 2 to 5 \& 9 to 12 from D to E \& G to H"} \\ \text{"B2 - EL. 64' - Gridline 5 to 6 from D to E \& G to H"} \\ \text{"B3 - EL. 64' - Gridline 4 to 6 from E to G"} \\ \text{"B4 - EL. 72' - Gridline 9 to 12 from E to G"} \\ \text{"B5 - EL. 64' - Gridline 2 to 4 from E to G"} \\ \text{"B6 - EL. 100' - Gridline 6 to 9 from D to H"} \\ \text{"B7 - EL. 36' - Gridline 12 to 13 from E to G at Vestibule"} \\ \text{"B8 - EL. 32' - Gridline 2-5 \& 6-12 from D-E \& G-H, 9-10 from E-G"} \\ \text{"B9 - EL. 32' - Gridline 5-6 from D-E \& G-H "} \\ \text{"B10 - EL. 32' - Gridline 9-10 from E-G"} \end{pmatrix}$$

Moment and shear capacity are per Ref. 2.2.46 multiplied by the appropriate stress increase factor given in Table Q1.5.7.1 of Ref. 2.2.46. For bending in beams, the stress increase factor used in this evaluation is 1.5. For beam shear, the stress increase factor is 1.4.

The steel decking with the concrete slab provides continuous support against lateral movement of the compression flange along the entire beam length. Therefore, the beam strength is controlled by the yielding of the member and all other failure mechanisms do not control.

$Mpx = 1.5*Fa_b * Sx$
where: $Fa_b = 0.66 \times Fy$
Sx = Section modulus per Table 1-1 of Ref. 2.2.40

$Vnx = 1.4*Fa_s * Aw$
where: $Fa_s = 0.4 \times Fy$
Aw = area of web = d x tw

$Fa_b := 0.66 \cdot F_y \qquad Fa_b = 33.00\,ksi$ Section Q1.5 Ref. 2.2.46

$Fa_s := 0.40 \cdot F_y \qquad Fa_s = 20.00\,ksi$

$$Beam_{Size} := \begin{pmatrix} \text{"W16x31"} \\ \text{"W24x68"} \\ \text{"W24x55"} \\ \text{"W18x40"} \\ \text{"W12x30"} \\ \text{"W18x55"} \\ \text{"W21x44"} \\ \text{"W18x35"} \\ \text{"W18x40"} \\ \text{"W10x39"} \end{pmatrix} \qquad M_{px} := 1.5 \cdot Fa_b \cdot \begin{pmatrix} 47.2 \\ 154 \\ 114 \\ 68.4 \\ 38.6 \\ 98.3 \\ 81.6 \\ 57.6 \\ 68.4 \\ 42.1 \end{pmatrix} \cdot in^3 \qquad V_{nx} := 1.4 \cdot Fa_s \cdot \begin{pmatrix} 15.9 \cdot 0.275 \\ 23.7 \cdot 0.415 \\ 23.6 \cdot 0.395 \\ 17.9 \cdot 0.315 \\ 12.3 \cdot 0.260 \\ 18.1 \cdot 0.390 \\ 20.7 \cdot 0.350 \\ 17.7 \cdot 0.30 \\ 17.9 \cdot 0.315 \\ 9.92 \cdot 0.315 \end{pmatrix} \cdot in^2$$

$$M_{px}^{T} = \begin{pmatrix} 194.70 & 635.25 & 470.25 & 282.15 & 159.22 & 405.49 & 336.60 & 237.60 & 282.15 & 173.66 \end{pmatrix} kip \cdot ft$$

$$V_{nx}^{T} = \begin{pmatrix} 122.43 & 275.39 & 261.02 & 157.88 & 89.54 & 197.65 & 202.86 & 148.68 & 157.88 & 87.49 \end{pmatrix} kip$$

$$trib_{Beam} := \begin{pmatrix} 6.5 \\ 6.0 \\ 4.5 \\ 6.5 \\ 5.29 \\ 6.43 \\ 6.67 \\ 6.5 \\ 6.5 \\ 6.4 \end{pmatrix} \cdot ft \qquad L_{Beam} := \begin{pmatrix} 14.5 \\ 26 \\ 22.5 \\ 17.75 \\ 12.0 \\ 21.17 \\ 18.8 \\ 14.5 \\ 12.0 \\ 13.0 \end{pmatrix} \cdot ft$$

Tributary width and beam lengths per Figure G.1 to G.3 in Attachment G and Ref. 2.2.38.

Structural Steel Framing Loadings

$$\text{Area} := \begin{pmatrix} 96\text{ft}\cdot 78\text{ft} \\ 36\cdot\text{ft}\cdot 78\cdot\text{ft} \\ 48\cdot\text{ft}\cdot 90\cdot\text{ft} \\ 85\cdot\text{ft}\cdot 90\cdot\text{ft} \\ 84\cdot\text{ft}\cdot 90\cdot\text{ft} \\ 90\cdot\text{ft}\cdot 254\cdot\text{ft} \\ 40\cdot\text{ft}\cdot 90\text{ft} \\ 96\cdot\text{ft}\cdot 78\cdot\text{ft} \\ 36\cdot\text{ft}\cdot 78\cdot\text{ft} \\ 13\cdot\text{ft}\cdot 90\cdot\text{ft} \end{pmatrix} \qquad \text{Area} = \begin{pmatrix} 7488 \\ 2808 \\ 4320 \\ 7650 \\ 7560 \\ 22860 \\ 3600 \\ 7488 \\ 2808 \\ 1170 \end{pmatrix} \text{ft}^2$$

Areas per Figure G.1 to G.3 in Attachment G

The truss weight is not supported by the beams. Therefore, do not include the truss weight when calculating the steel dead load for each beam case. However, the girder weight is included.

$$\text{Steel}_{DL} := \begin{bmatrix} 31\text{plf}\cdot 14.5\text{ft}\cdot(63) + 116\cdot\text{plf}\cdot 26\cdot\text{ft}\cdot(15) + 210\text{plf}\cdot 58\text{ft}\cdot(2) + 359\text{plf}\cdot 38\text{ft}\cdot(2) \\ 300\text{plf}\cdot 36\text{ft}\cdot(2) + 68\text{plf}\cdot 26\text{ft}\cdot(15) \\ 393\text{plf}\cdot 36\text{ft}\cdot 3 + 55\text{plf}\cdot 90\text{ft}\cdot(7) \\ 40\text{plf}\cdot(12 + 70)\cdot\text{ft}\cdot 14 \\ 30\text{plf}\cdot 84\cdot\text{ft}\cdot 16 \\ 55\text{plf}\cdot 21\text{ft}\cdot(12\cdot 13) \\ 359\text{plf}\cdot 40\text{ft}\cdot(4) + 44\text{plf}\cdot 94\text{ft}\cdot(5) \\ 35\text{plf}\cdot 96\text{ft}\cdot(9) + 118\text{plf}\cdot 78\cdot\text{ft}\cdot(5) + 232\text{plf}\cdot 58\text{ft}\cdot(2) + 393\text{plf}\cdot 38\text{ft}\cdot(2) \\ 40\text{plf}\cdot 36\text{ft}\cdot(9) + 130\text{plf}\cdot 78\text{ft}\cdot(2) + 359\text{plf}\cdot 36\text{ft}\cdot 2 \\ 39\text{plf}\cdot 13\text{ft}\cdot(13) + 235\text{plf}\cdot 94\text{ft}\cdot(1)\cdot 0 \end{bmatrix}$$

$$\text{Steel}_{DL}{}^{T} = \begin{pmatrix} 125.20 & 48.12 & 77.09 & 45.92 & 40.32 & 180.18 & 78.12 & 133.04 & 59.09 & 6.59 \end{pmatrix} \text{kip}$$

$$\text{DL}_{steel} := \overrightarrow{\frac{\text{Steel}_{DL}}{\text{Area}}}$$

$$\text{DL}_{steel}{}^{T} = \begin{pmatrix} 16.72 & 17.14 & 17.85 & 6.00 & 5.33 & 7.88 & 21.70 & 17.77 & 21.04 & 5.63 \end{pmatrix} \text{psf}$$

A uniform loading of 25 psf can be used for the structural beam weight. Conservatively considering 50 psf for the structural steel weight will be bounding for the diaphragm calculations in Section 6.3. Also, using 80 psf for the steel weight in areas with steel trusses and beams is bounding.

Non-Seismic Loads

Dead load of concrete slab for Beam Cases 1 to 10

Beam Case 3 and 5 support 3-ft slabs, all other cases support 1.5-ft slabs

$$DL_s := \begin{pmatrix} DL_{slab_1} \\ DL_{slab_1} \\ DL_{slab_2} \\ DL_{slab_1} \\ DL_{slab_2} \\ DL_{slab_1} \\ DL_{slab_1} \\ DL_{slab_1} \\ DL_{slab_1} \\ DL_{slab_1} \end{pmatrix} \qquad DL_s = \begin{pmatrix} 243.75 \\ 243.75 \\ 431.25 \\ 243.75 \\ 431.25 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \end{pmatrix} psf$$

Live load for Beam Cases 1 to 10

$$LL_{Beam} := \begin{pmatrix} LL_r \\ LL_r \\ LL_r \\ LL_r \\ LL_r \\ LL_r \\ LL_r \\ LL \\ LL \\ LL \end{pmatrix} \qquad LL_{Beam} = \begin{pmatrix} 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 100.00 \\ 100.00 \\ 100.00 \end{pmatrix} psf$$

Total Dead Load for Beam Cases 1 to 10 including roofing dead load for all cases

$$DL_{Beam} := DL_s + DL_{equip} + DL_{hang} + DL_{deck} + DL_{steel} + DL_{roof}$$

$$DL_{Beam}^{\ T} = \begin{pmatrix} 339.05 & 339.47 & 527.68 & 328.33 & 515.16 & 330.21 & 344.03 & 340.10 & 343.37 & 327.96 \end{pmatrix} psf$$

Subtract the roofing material dead load from Cases 8 to 10 (the floor beam cases)

$$DL_{Beam_8} := DL_{Beam_8} - DL_{roof} \qquad DL_{Beam_8} = 325.10\,psf$$

$$DL_{Beam_9} := DL_{Beam_9} - DL_{roof} \qquad DL_{Beam_9} = 328.37\,psf$$

$$DL_{Beam_{10}} := DL_{Beam_{10}} - DL_{roof} \qquad DL_{Beam_{10}} = 312.96\,psf$$

Non-Seismic Loads for Beam Cases 1 to 10

$$DL_{Beam}^{\ T} = \begin{pmatrix} 339.05 & 339.47 & 527.68 & 328.33 & 515.16 & 330.21 & 344.03 & 325.10 & 328.37 & 312.96 \end{pmatrix} psf$$

$$LL_{Beam}^{\ T} = \begin{pmatrix} 40.00 & 40.00 & 40.00 & 40.00 & 40.00 & 40.00 & 40.00 & 100.00 & 100.00 & 100.00 \end{pmatrix} psf$$

Seismic Loads

Use the enveloped vertical acceleration at EL. 64' for the vertical acceleration at EL. 72 case b/c the EL. 64' value is higher.

$$Acc_{Beam} := amplify \cdot \begin{pmatrix} Acc_{v_2} \\ Acc_{v_2} \\ Acc_{v_2} \\ Acc_{v_2} \\ Acc_{v_2} \\ Acc_{v_4} \\ Acc_{v_1} \\ Acc_{v_1} \\ Acc_{v_1} \\ Acc_{v_1} \end{pmatrix} \qquad Acc_{Beam} = \begin{pmatrix} 2.14 \\ 2.14 \\ 2.14 \\ 2.14 \\ 2.14 \\ 2.20 \\ 2.12 \\ 2.12 \\ 2.12 \\ 2.12 \end{pmatrix} g$$

Amplified vertical acceleration for each beam case

Seismic Load = (DL + 25% LL) * Acc per Ref. 2.2.4

$$E_{Beam} := \overrightarrow{\left[\left(DL_{Beam} + 0.25 \cdot LL_{Beam}\right)\frac{Acc_{Beam}}{g}\right]}$$

Seismic demand tributary to each beam case

$$E_{Beam}^{\ T} = \begin{pmatrix} 746.97 & 747.86 & 1150.63 & 724.03 & 1123.85 & 748.47 & 750.54 & 742.21 & 749.15 & 716.48 \end{pmatrix} psf$$

Maximum uniform load on beams

$$wu_{max}(w, tributary) := \overrightarrow{(w \cdot tributary)}$$

$$wu_{NS} := wu_{max}\left(DL_{Beam} + 0.25LL_{Beam}, trib_{Beam}\right)$$

Uniform beam loading for non-seismic loads

$$wu_{NS}^{\ T} = \begin{pmatrix} 2.27 & 2.10 & 2.42 & 2.20 & 2.78 & 2.19 & 2.36 & 2.28 & 2.30 & 2.16 \end{pmatrix} \frac{kip}{ft}$$

$$wu_{BDBGM} := wu_{max}\left(E_{Beam}, trib_{Beam}\right)$$

Uniform beam loading for BDBGM loads

$$wu_{BDBGM}^{\ T} = \begin{pmatrix} 4.86 & 4.49 & 5.18 & 4.71 & 5.95 & 4.81 & 5.01 & 4.82 & 4.87 & 4.59 \end{pmatrix} \frac{kip}{ft}$$

Maximum moment and shear

$$M_u(w, L) := \overrightarrow{\frac{w \cdot L^2}{8}}$$

Max. moment on simply-supported, uniformly loaded beam

$$V_u(w, L) := \overrightarrow{\frac{w \cdot L}{2}}$$

Max. shear on simply-supported, uniformly loaded beam

$M_{uNS} := M_u(wu_{NS}, L_{Beam})$ Max. moment due to non-seismic loads

$M_{uNS}^T = (59.63\ \ 177.18\ \ 153.11\ \ 86.61\ \ 50.01\ \ 122.55\ \ 104.33\ \ 59.81\ \ 41.34\ \ 45.69)\ \text{kip}\cdot\text{ft}$

$V_{uNS} := V_u(wu_{NS}, L_{Beam})$ Max. shear due to non-seismic loads

$V_{uNS}^T = (16.45\ \ 27.26\ \ 27.22\ \ 19.52\ \ 16.67\ \ 23.16\ \ 22.20\ \ 16.50\ \ 13.78\ \ 14.06)\ \text{kip}$

$M_{uBDBGM} := M_u(wu_{BDBGM}, L_{Beam})$ Max. moment due to seismic loads

$M_{uBDBGM}^T = (127.60\ \ 379.16\ \ 327.66\ \ 185.34\ \ 107.01\ \ 269.61\ \ 221.17\ \ 126.79\ \ 87.65\ \ 96.87)\ \text{kip}\cdot\text{ft}$

$V_{uBDBGM} := V_u(wu_{BDBGM}, L_{Beam})$ Max. shear due to seismic loads

$V_{uBDBGM}^T = (35.20\ \ 58.33\ \ 58.25\ \ 41.77\ \ 35.67\ \ 50.94\ \ 47.06\ \ 34.98\ \ 29.22\ \ 29.81)\ \text{kip}$

Strength Margin Factor for Beams (Fs_{Beam})

$Fs_{MomBeam} := \dfrac{M_{px} - M_{uNS}}{M_{uBDBGM}}$ Strength margin factor for moment

$Fs_{MomBeam}^T = (1.06\ \ 1.21\ \ 0.97\ \ 1.06\ \ 1.02\ \ 1.05\ \ 1.05\ \ 1.40\ \ 2.75\ \ 1.32)$

$Fs_{ShearBeam} := \dfrac{V_{nx} - V_{uNS}}{V_{uBDBGM}}$ Strength margin factor for shear

$Fs_{ShearBeam}^T = (3.01\ \ 4.25\ \ 4.01\ \ 3.31\ \ 2.04\ \ 3.43\ \ 3.84\ \ 3.78\ \ 4.93\ \ 2.46)$

Inelastic Energy Absorption Factor for Beams ($F\mu$)

Per Table 5-1 of Ref. 2.2.6

$F_{\mu Mom} := 5.25$ $F\mu$ for beams of SMRF steel moment frames - Limit State A

Based on Section 6.3 of this calculation, the $F\mu$ for structural beams and girders must be reduced to account for ratcheting effects. Using equation 6.3.7 in Section 6.3, the revised $F\mu$ is given by:

$N := 4$ Number of equal nonlinear response cycles (Assumption 3.1.11)

$F_{\mu e} := 1 + \dfrac{F_{\mu Mom} - 1}{N}$ $F_{\mu e} = 2.06$

$F_{\mu MomBeam} := F_{\mu e}$ $F\mu$ for bending in steel beams and girders

$F_{\mu Shear} := 1.0$ Shear failure is a brittle failure thus no inelastic energy absorption is considered.

HCLPF Capacity for Beams ($HCLPF_{Beams}$)

$HCLPF_{MomBeams} := Fs_{MomBeam} \cdot F_{\mu e} \cdot PGA_h$ HCLPF for bending moment of beams

$HCLPF_{ShearBeams} := Fs_{ShearBeam} \cdot F_{\mu Shear} \cdot PGA_h$ HCLPF for shear of beams

$$HCLPF_{MomBeams}^{T} = \begin{pmatrix} 2.00 & 2.28 & 1.82 & 1.99 & 1.92 & 1.98 & 1.98 & 2.64 & 5.18 & 2.49 \end{pmatrix} g$$

$$HCLPF_{ShearBeams}^{T} = \begin{pmatrix} 2.75 & 3.89 & 3.67 & 3.03 & 1.87 & 3.13 & 3.51 & 3.45 & 4.51 & 2.25 \end{pmatrix} g$$

Minimum HCLPF Capacity for Beams - Moment and Shear

$HCLPF_{BeamsM} := \min(HCLPF_{MomBeams})$ $HCLPF_{BeamsM} = 1.82\,g$

$HCLPF_{BeamsS} := \min(HCLPF_{ShearBeams})$ $HCLPF_{BeamsS} = 1.87\,g$

6.4.2 HCLPF Capacity Evaluations for Structural Girders

The girder cases for the HCLPF capacity evaluations are identical to the girder cases considered in the structural steel design calculation (Ref. 2.2.38). Cases 1 to 6 are roof girders while cases 7 to 12 are floor girders.

Girder Case 11 consists of the floor girder along column line 10 that is used to support the construction loads from the wall from EL. 32' to EL. 72'. After the concrete wall along column line 10 has been set, the wall acts as a deep beam spanning from column line E to G. Therefore, Girder Case 11 is not considered in the HCLPF capacity evaluations.

Girder Case 13 and 14 (W18x35, W21x44), consisting of the floor girders around the openings at EL. 32', are bounded by Beam Case 8 (W18x35 at EL. 32'). Also, Girder Case 15 (W33x118), consisting of floor girders around the openings at EL. 32', are bounded by Girder Case 7 (W33x118 at EL. 32'). Therefore, Girder Cases 13, 14, and 15 from Ref. 2.2.38 are not considered in the HCLPF capacity evaluations for the CRCF structural girders.

See Figure G.1 to G.3 in Attachment G for the locations of the steel girder cases.

$$Girder_{Cases} := \begin{pmatrix} \text{"G1 - EL. 64' - Gridline 2 to 5 \& 9 to 12 from D to E \& G to H"} \\ \text{"G2 - EL. 64' - Gridline 2 to 5 from D to E \& G to H"} \\ \text{"G3 - EL. 64' - Gridline 3 to 5 \& 10 to 12 from D to E \& G to H"} \\ \text{"G4 - EL. 64' - Gridline 5 to 6 from D to H"} \\ \text{"G5 - EL. 64' - Gridline 4 to 5 from E to G"} \\ \text{"G6 - EL. 32' - Gridline 1 to 2 from E to G at Vestibule"} \\ \text{"G7 - EL. 32' - Gridline 2 to 5 \& 6 to 12 from D to E \& G to H"} \\ \text{"G8 - EL. 32' - Gridline 3 to 5 \& 10 to 12 from D to E \& G to H"} \\ \text{"G9 - EL. 32' - Gridline 5 to 6 from D to E \& G to H "} \\ \text{"G10 - EL. 32' - Gridline 5 to 6 from D to E \& G to H"} \\ \text{"G12 - EL. 32' - Gridline 2 to 4 \& 6 to 9 from D to E \& G to H"} \end{pmatrix}$$

Allowable moment and shear capacity are per Ref. 2.2.46 multiplied by the appropriate stress increase factor given in Table Q1.5.7.1 of Ref. 2.2.46. For bending in beams, the stress increase factor used in this evaluation is 1.5. For beam shear, the stress increase factor is 1.4.

The steel decking provides continuous support against lateral movement of the compression flange along the entire beam length. Therefore, the beam strength is controlled by the yielding of the member and all other failure mechanisms do not control.

Mpx = 1.5*Fa_b * Sx
where: Fa_b = 0.66 x Fy
Sx = Section modulus per Table 1-1 of Ref. 2.2.40

Vnx = 1.4*Fa_s * Aw
where: Fa_s = 0.4 x Fy
Aw = area of web = d x tw

$Fa_b = 33.00\,\mathrm{ksi}$

$Fa_s = 20.00\,\mathrm{ksi}$

Note: Table 1-1 of Ref. 2.2.40 does not include the shape for Cases 3, 4, 5, 6, 8, & 10 (W36x359, W36x300, W36x393). The allowable bending capacities of these sections are determined by obtaining the Sx factor in the allowable stress design selection table in Part 2 of Ref. 2.2.41. The allowable shear capacities of these sections is determined using the above equation for shear.

$$Girder_{Size} := \begin{pmatrix} \text{"W30x116"} \\ \text{"W36x210"} \\ \text{"W36x359"} \\ \text{"W36x300"} \\ \text{"W36x393"} \\ \text{"W36x359"} \\ \text{"W33x118"} \\ \text{"W36x393"} \\ \text{"W33x130"} \\ \text{"W36x359"} \\ \text{"W36x232"} \end{pmatrix} \quad M_{pxGirder} := 1.5\cdot Fa_b \cdot \begin{pmatrix} 329 \\ 719 \\ 1320 \\ 1110 \\ 1450 \\ 1320 \\ 359 \\ 1450 \\ 406 \\ 1320 \\ 809 \end{pmatrix} \cdot in^3 \quad V_{nxGirder} := 1.4\cdot Fa_s \cdot \begin{pmatrix} 30\cdot 0.565 \\ 36.7\cdot .830 \\ 37.4\cdot 1.12 \\ 36.74\cdot 0.945 \\ 37.8\cdot 1.22 \\ 37.4\cdot 1.12 \\ 32.9\cdot 0.55 \\ 37.8\cdot 1.22 \\ 33.1\cdot .580 \\ 37.4\cdot 1.12 \\ 37.1\cdot 0.870 \end{pmatrix} \cdot in^2$$

$$M_{pxGirder}{}^{T} = \begin{pmatrix} 1357 & 2966 & 5445 & 4579 & 5981 & 5445 & 1481 & 5981 & 1675 & 5445 & 3337 \end{pmatrix} kip\cdot ft$$

$$V_{nxGirder}{}^{T} = \begin{pmatrix} 475 & 853 & 1173 & 972 & 1291 & 1173 & 507 & 1291 & 538 & 1173 & 904 \end{pmatrix} kip$$

<u>Girder Loading Configuration</u>
The girder loading configuration used in the HCLPF capacity evaluation is identical to the loading configuration used in the girder design of Ref. 2.2.38.

The major loadings on the girders come from the shear reaction of the beams framing into the girder. These shear reactions, calculated in Section 6.4.1, include the effects of dead load, live load, and seismic load. In the case of girders that support a tributary width of the slab (Girder Cases 2, 3, 8, 10, and 12 which are girder parallel to beams) the self-weight of the girder is the only steel framing load included in the dead load because the other structural steel weight is already included in the shear reactions from the beams.

For the girder cases that do not support a tributary width of the floor or roof slab, the $trib_{Girder}$ length is set equal to 0.0 feet.

$$trib_{Girder} := \begin{pmatrix} 0 \\ 6.5 \\ 6.5 \\ 0 \\ 0 \\ 0 \\ 0 \\ 6.5 \\ 0 \\ 6.5 \\ 6.5 \end{pmatrix} \cdot ft \qquad L_{Girder} := \begin{pmatrix} 26 \\ 29 \\ 38 \\ 36 \\ 36 \\ 46 \\ 26 \\ 38 \\ 26 \\ 36 \\ 29 \end{pmatrix} \cdot ft$$

Tributary width and beam lengths per Figure G.1 to G.3 in Attachment G and Ref. 2.2.38.

Non-Seismic Loads

Dead load of concrete slab for Girder Cases 1 to 12
All girders support 18" slabs

Live Load for Girder Cases 1 to 12
Girder Cases 1 to 6 support roof slabs while Case 7 to 12 support floor slabs

$$DL_{sGirder} := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix} \cdot DL_{slab_1} \qquad DL_{sGirder} = \begin{pmatrix} 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \\ 243.75 \end{pmatrix} psf$$

$$LL_{Girder} := \begin{pmatrix} LL_r \\ LL_r \\ LL_r \\ LL_r \\ LL_r \\ LL_r \\ LL \\ LL \\ LL \\ LL \\ LL \end{pmatrix} \qquad LL_{Girder} = \begin{pmatrix} 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 100.00 \\ 100.00 \\ 100.00 \\ 100.00 \\ 100.00 \end{pmatrix} psf$$

Tributary Girder Self-weight

Girder self-weight is determined as a psf acting over the tributary area of the girder. When the girder self-weight is multiplied by the tributary area, the resulting uniform loading on the girder will be the girder self-weight

$$trib_{Girder} = \begin{pmatrix} 0.00 \\ 6.50 \\ 6.50 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 6.50 \\ 0.00 \\ 6.50 \\ 6.50 \end{pmatrix} ft \quad DL_{GirderSteel} := \begin{pmatrix} 0 \cdot psf \\ \overrightarrow{\dfrac{210plf}{trib_{Girder_2}}} \\ \overrightarrow{\dfrac{359plf}{trib_{Girder_3}}} \\ 0psf \\ 0psf \\ 0psf \\ 0psf \\ \overrightarrow{\dfrac{393plf}{trib_{Girder_8}}} \\ 0psf \\ \overrightarrow{\dfrac{359plf}{trib_{Girder_{10}}}} \\ \overrightarrow{\dfrac{232plf}{trib_{Girder_{11}}}} \end{pmatrix} \quad DL_{GirderSteel} = \begin{pmatrix} 0.00 \\ 32.31 \\ 55.23 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 60.46 \\ 0.00 \\ 55.23 \\ 35.69 \end{pmatrix} psf$$

Note: Array location 11 is associated with Girder Case 12 (i.e. Girder Case 11 is not considered in the HCLPF capacity evaluation).

Total Dead Load for Girder Cases 1 to 12 including roofing dead load for all cases (As noted above, the only structural steel weight included is the girder self-weight.

$$DL_{Girder} := DL_{sGirder} + DL_{equip} + DL_{hang} + DL_{deck} + DL_{roof} + DL_{GirderSteel}$$

$$DL_{Girder}^{T} = \begin{pmatrix} 322.33 & 354.64 & 377.56 & 322.33 & 322.33 & 322.33 & 322.33 & 382.79 & 322.33 & 377.56 & 358.02 \end{pmatrix} psf$$

Subtract the roofing material dead load from Cases 7 to 12 (the floor beam cases)

$DL_{Girder_7} := DL_{Girder_7} - DL_{roof}$ $\qquad DL_{Girder_7} = 307.33\,psf$

$DL_{Girder_8} := DL_{Girder_8} - DL_{roof}$ $\qquad DL_{Girder_8} = 367.79\,psf$

$DL_{Girder_9} := DL_{Girder_9} - DL_{roof}$ $\qquad DL_{Girder_9} = 307.33\,psf$

$DL_{Girder_{10}} := DL_{Girder_{10}} - DL_{roof}$ $\qquad DL_{Girder_{10}} = 362.56\,psf$

$DL_{Girder_{11}} := DL_{Girder_{11}} - DL_{roof}$ $\qquad DL_{Girder_{11}} = 343.02\,psf$

Non-Seismic Loads Tributary to Girder Cases 1 to 12

$$DL_{Girder} = \begin{pmatrix} 322.33 \\ 354.64 \\ 377.56 \\ 322.33 \\ 322.33 \\ 322.33 \\ 307.33 \\ 367.79 \\ 307.33 \\ 362.56 \\ 343.02 \end{pmatrix} psf \qquad LL_{Girder} = \begin{pmatrix} 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 40.00 \\ 100.00 \\ 100.00 \\ 100.00 \\ 100.00 \\ 100.00 \end{pmatrix} psf$$

Dead and Live Load acting tributary to the girders

Seismic Loads
Girder Cases 1 to 5 are located at EL. 64', while Cases 6 to 12 are located at EL. 32'

$$Acc_{Girder} := amplify \cdot \begin{pmatrix} Acc_{v_2} \\ Acc_{v_2} \\ Acc_{v_2} \\ Acc_{v_2} \\ Acc_{v_2} \\ Acc_{v_1} \\ Acc_{v_1} \\ Acc_{v_1} \\ Acc_{v_1} \\ Acc_{v_1} \\ Acc_{v_1} \end{pmatrix} \qquad Acc_{Girder} = \begin{pmatrix} 2.14 \\ 2.14 \\ 2.14 \\ 2.14 \\ 2.14 \\ 2.12 \\ 2.12 \\ 2.12 \\ 2.12 \\ 2.12 \\ 2.12 \end{pmatrix} g$$

Amplified vertical seismic acceleration for each girder case

Seismic Load = (DL + 25% LL) * Acc per Ref. 2.2.4

$$E_{Girder} := \overrightarrow{\left[\left(DL_{Girder} + 0.25 \cdot LL_{Girder}\right)\frac{Acc_{Girder}}{g}\right]}$$

Seismic demand tributary to each girder case

$$E_{Girder}^{T} = \begin{pmatrix} 711.2 & 780.3 & 829.4 & 711.2 & 711.2 & 704.5 & 704.5 & 832.7 & 704.5 & 821.6 & 780.2 \end{pmatrix} psf$$

Maximum Moment and Shears on Girders

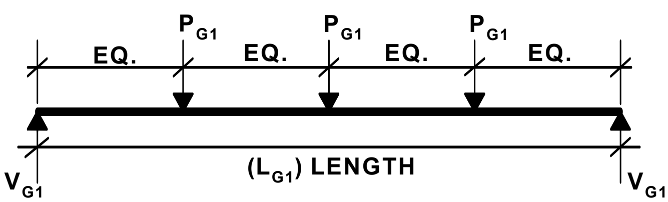
The maximum moment and shears on the girders due to the concentrated loading from the beams (or girders) framing into the girder are determined using the Table 3-22a - Concentrated Load Equivalents (Ref. 2.2.40) for simple beams. For each girder case, the a and c coefficients are tabulated depending on the concentrated loading configuration on the girder.

The maximum moment and shears due to the uniform loading on the girder case are calculated using Mmax = w*L^2 / 8 (a = 0.125) and Vmax = w*L/2 (c = 0.50)

The maximum moment and shear on the girder are calculated by superimposing the maximum moments and shears from the concentrated loadings and from the uniform loading.

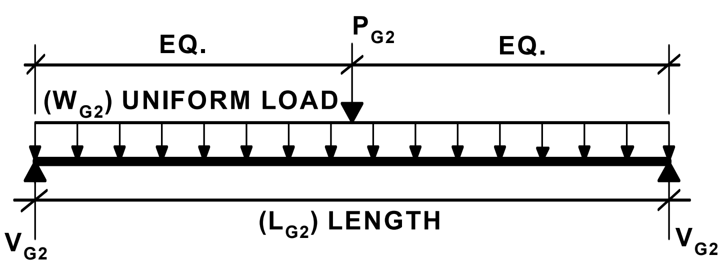
Concentrated Loading Configuration
The concentrated loading configuration on each girder are determined as follows:

Case 1: 3 equally spaced concentrated loadings from Beam Case 1
PG1 = 2 x shear reaction of Beam Case 1 (one reaction on each side of girder)



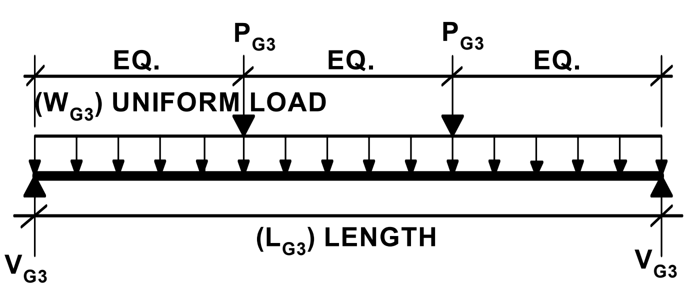
Girder Case 1 Sketch

Case 2: Single concentrated loadings from Girder Case 1
PG2 = 2 x Shear reaction of Girder Case 1 (one reaction on each side of girder)



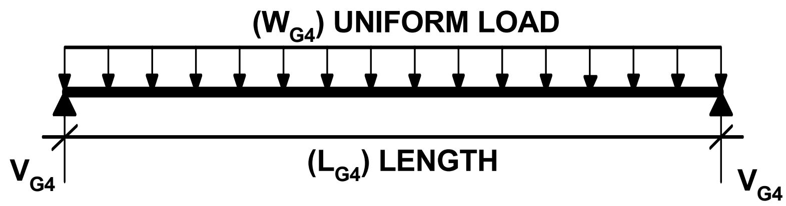
Girder Case 2 Sketch

Case 3: 2 equally spaced concentrated loadings from Girder Case 1
PG3 = 2 x Shear reaction of Girder Case 1 (one reaction on each side of girder)



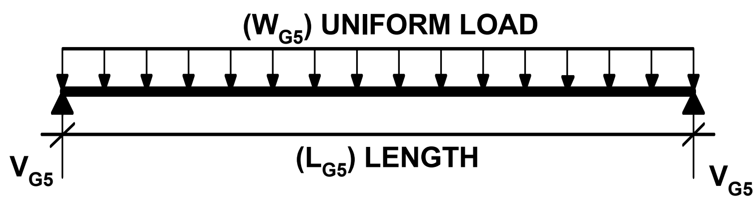
Girder Case 3 Sketch

Case 4: 5 equally spaced concentrated loadings from Beam Case 2 treated as a uniform loading
wG4 = 5 x Shear reaction of Beam Case 2 (use uniformly loaded beam coefficients of
a = 0.125, c = 0.5)



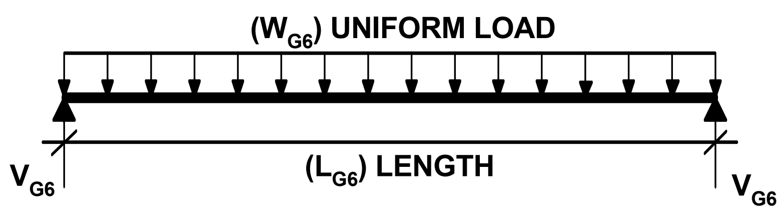
Girder Case 4 Sketch

Case 5: 7 equally spaced concentrated loadings from Beam Case 3 treated as a uniform loading
wG5 = 7 x Shear reaction of Beam Case 3 (use uniformly loaded beam coefficients of
a = 0.125, c = 0.5)



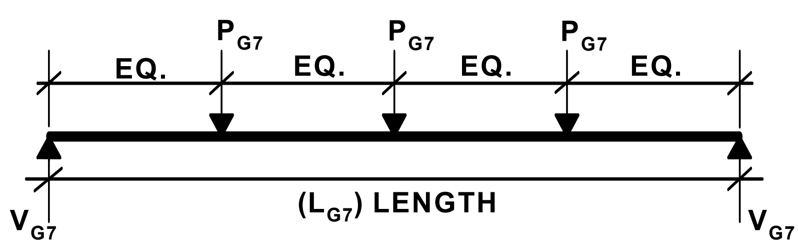
Girder Case 5 Sketch

Case 6: 6 equally spaced concentrated loadings from Beam Case 7 treated as a uniform loading
wG6 = 6 x Shear reaction of Beam Case 7 (use uniformly loaded beam coefficients of
a = 0.125, c = 0.5)



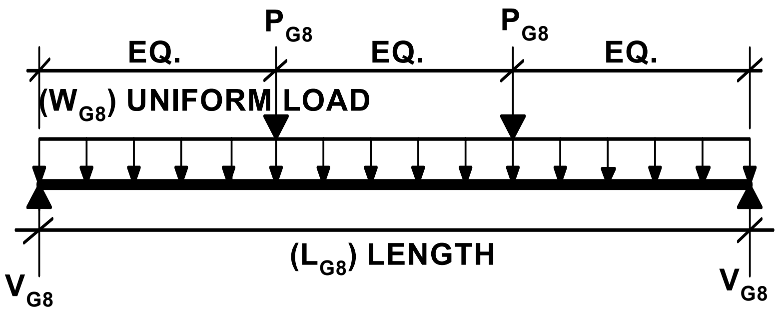
Girder Case 6 Sketch

Case 7: 3 equally spaced concentrated loadings from Beam Case 8
PG7 = 2 x shear reaction of Beam Case 8 (one reaction on each side of girder)



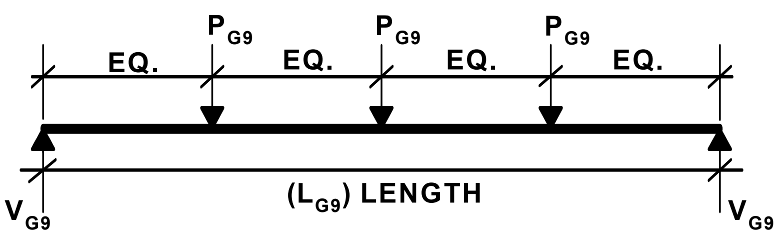
Girder Case 7 Sketch

Case 8: 2 equally spaced concentrated loadings from Girder Case 7
PG8 = 2 x shear reaction of Girder Case 7 (one reaction on each side of girder)



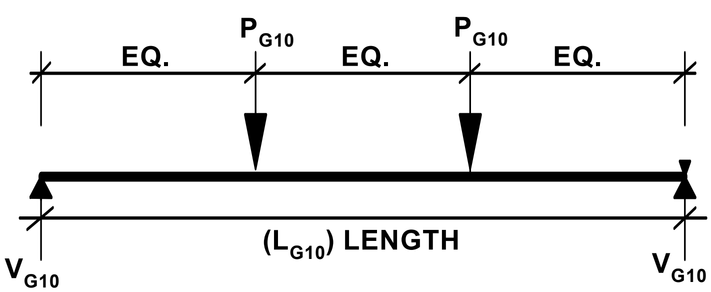
Girder Case 8 Sketch

Case 9: 3 equally spaced concentrated loadings from Beam Case 9
PG9 = 2 x shear reaction of Beam Case 9 (one reaction on each side of girder)



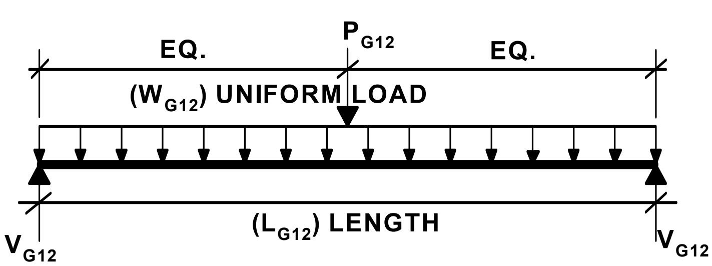
Girder Case 9 Sketch

Case 10: 2 equally spaced concentrated loadings from Girder Case 9
PG10 = 2 x shear reaction of Girder Case 9 (one reaction on each side of girder)



Girder Case 10 Sketch

Case 12: Single concentrated loadings from Girder Case 7
PG12 = 2 x Shear reaction of Girder Case 7 (one reaction on each side of girder)



Girder Case 11 Sketch

Coefficients for Concentrated Loadings for Simple Beams (Girder Cases 1 to 12)

$$a_{Concentrated} := \begin{pmatrix} 0.5 \\ 0.25 \\ 0.333 \\ 0.125 \\ 0.125 \\ 0.125 \\ 0.5 \\ 0.333 \\ 0.5 \\ 0.333 \\ 0.25 \end{pmatrix} \qquad c_{Concentrated} := \begin{pmatrix} 1.5 \\ 0.5 \\ 1.0 \\ 0.5 \\ 0.5 \\ 0.5 \\ 1.5 \\ 1.0 \\ 1.5 \\ 1.0 \\ 0.5 \end{pmatrix}$$

a and c values used in Table 3-22a - Concentrated Load Equivalents (Ref. 2.2.40) for simple beams to determine maximum moment and shear for each girder case

Concentrated Loadings

Non-Seismic Loadings

Girder Case 1	$PG1_{NS} := 2 \cdot V_{uNS_1}$	$PG1_{NS} = 32.90\,kip$
Girder Case 2	$PG2_{NS} := 2 \cdot PG1_{NS}$	$PG2_{NS} = 65.80\,kip$
Girder Case 3	$PG3_{NS} := 2 \cdot PG1_{NS}$	$PG3_{NS} = 65.80\,kip$
Girder Case 4	$PG4_{NS} := 2 \cdot V_{uNS_2} \cdot 5$	$PG4_{NS} = 272.58\,kip$
Girder Case 5	$PG5_{NS} := 2 \cdot V_{uNS_3} \cdot 7$	$PG5_{NS} = 381.08\,kip$
Girder Case 6	$PG6_{NS} := 2 \cdot V_{uNS_7} \cdot 6$	$PG6_{NS} = 266.36\,kip$
Girder Case 7	$PG7_{NS} := 2 \cdot V_{uNS_8}$	$PG7_{NS} = 33.00\,kip$
Girder Case 8	$PG8_{NS} := 2 \cdot PG7_{NS}$	$PG8_{NS} = 65.99\,kip$
Girder Case 9	$PG9_{NS} := 2 \cdot V_{uNS_9}$	$PG9_{NS} = 27.56\,kip$
Girder Case 10	$PG10_{NS} := 2 \cdot PG9_{NS}$	$PG10_{NS} = 55.13\,kip$
Girder Case 12	$PG12_{NS} := 2 \cdot PG7_{NS}$	$PG12_{NS} = 65.99\,kip$

Total load from beams is considered as uniform on Girder Case 4, 5, and 6

$$P_{GirderNS} := \begin{pmatrix} PG1_{NS} \\ PG2_{NS} \\ PG3_{NS} \\ PG4_{NS} \\ PG5_{NS} \\ PG6_{NS} \\ PG7_{NS} \\ PG8_{NS} \\ PG9_{NS} \\ PG10_{NS} \\ PG12_{NS} \end{pmatrix} \qquad P_{GirderNS} = \begin{pmatrix} 32.90 \\ 65.80 \\ 65.80 \\ 272.58 \\ 381.08 \\ 266.36 \\ 33.00 \\ 65.99 \\ 27.56 \\ 55.13 \\ 65.99 \end{pmatrix} \text{kip}$$

Seismic Loadings

Girder Case 1	$PG1_{BDBGM} := 2 \cdot V_{uBDBGM_1}$	$PG1_{BDBGM} = 70.40\ \text{kip}$
Girder Case 2	$PG2_{BDBGM} := 2 \cdot PG1_{BDBGM}$	$PG2_{BDBGM} = 140.80\ \text{kip}$
Girder Case 3	$PG3_{BDBGM} := 2 \cdot PG1_{BDBGM}$	$PG3_{BDBGM} = 140.80\ \text{kip}$
Girder Case 4	$PG4_{BDBGM} := 2 \cdot V_{uBDBGM_2} \cdot 5$	$PG4_{BDBGM} = 583.33\ \text{kip}$
Girder Case 5	$PG5_{BDBGM} := 2 \cdot V_{uBDBGM_3} \cdot 7$	$PG5_{BDBGM} = 815.51\ \text{kip}$
Girder Case 6	$PG6_{BDBGM} := 2 \cdot V_{uBDBGM_7} \cdot 6$	$PG6_{BDBGM} = 564.69\ \text{kip}$
Girder Case 7	$PG7_{BDBGM} := 2 \cdot V_{uBDBGM_8}$	$PG7_{BDBGM} = 69.95\ \text{kip}$
Girder Case 8	$PG8_{BDBGM} := 2 \cdot PG7_{BDBGM}$	$PG8_{BDBGM} = 139.91\ \text{kip}$
Girder Case 9	$PG9_{BDBGM} := 2 \cdot V_{uBDBGM_9}$	$PG9_{BDBGM} = 58.43\ \text{kip}$
Girder Case 10	$PG10_{BDBGM} := 2 \cdot PG9_{BDBGM}$	$PG10_{BDBGM} = 116.87\ \text{kip}$
Girder Case 12	$PG12_{BDBGM} := 2 \cdot PG7_{BDBGM}$	$PG12_{BDBGM} = 139.91\ \text{kip}$

Total load from beams is considered as uniform on Girder Case 4, 5, and 6

$$P_{GirderBDBGM} := \begin{pmatrix} PG1_{BDBGM} \\ PG2_{BDBGM} \\ PG3_{BDBGM} \\ PG4_{BDBGM} \\ PG5_{BDBGM} \\ PG6_{BDBGM} \\ PG7_{BDBGM} \\ PG8_{BDBGM} \\ PG9_{BDBGM} \\ PG10_{BDBGM} \\ PG12_{BDBGM} \end{pmatrix} \qquad P_{GirderBDBGM} = \begin{pmatrix} 70.40 \\ 140.80 \\ 140.80 \\ 583.33 \\ 815.51 \\ 564.69 \\ 69.95 \\ 139.91 \\ 58.43 \\ 116.87 \\ 139.91 \end{pmatrix} \text{kip}$$

Uniform Loadings

$wu_{GirderNS} := wu_{max}\left(DL_{Girder} + 0.25LL_{Girder}, trib_{Girder}\right)$ Uniform girder loading for non-seismic loads

$$wu_{GirderNS}^{T} = \begin{pmatrix} 0.00 & 2.37 & 2.52 & 0.00 & 0.00 & 0.00 & 0.00 & 2.55 & 0.00 & 2.52 & 2.39 \end{pmatrix} \frac{kip}{ft}$$

$wu_{GirderBDBGM} := wu_{max}\left(E_{Girder}, trib_{Girder}\right)$ Uniform girder loading for BDBGM loads

$$wu_{GirderBDBGM}^{T} = \begin{pmatrix} 0.00 & 5.07 & 5.39 & 0.00 & 0.00 & 0.00 & 0.00 & 5.41 & 0.00 & 5.34 & 5.07 \end{pmatrix} \frac{kip}{ft}$$

Maximum Moment and Shear on Girder

Non-Seismic Loadings

$$Mu_{GirderNS} := \overrightarrow{\frac{wu_{GirderNS} \cdot L_{Girder}^{2}}{8}} + \overrightarrow{\left(a_{Concentrated} \cdot P_{GirderNS} \cdot L_{Girder}\right)}$$

$$Vu_{GirderNS} := \overrightarrow{\frac{wu_{GirderNS} \cdot L_{Girder}}{2}} + \overrightarrow{\left(c_{Concentrated} \cdot P_{GirderNS}\right)}$$

$$Mu_{GirderNS} = \begin{pmatrix} 427.67 \\ 726.18 \\ 1287.29 \\ 1226.63 \\ 1714.85 \\ 1531.59 \\ 428.96 \\ 1295.92 \\ 358.32 \\ 1068.95 \\ 729.93 \end{pmatrix} kip \cdot ft \qquad Vu_{GirderNS} = \begin{pmatrix} 49.35 \\ 67.27 \\ 113.66 \\ 136.29 \\ 190.54 \\ 133.18 \\ 49.49 \\ 114.50 \\ 41.34 \\ 100.47 \\ 67.68 \end{pmatrix} kip$$

Moment and shear due to non-seismic uniform loading and concentrated loading

Seismic Loadings

$$Mu_{GirderBDBGM} := \overrightarrow{\frac{wu_{GirderBDBGM} \cdot L_{Girder}^{2}}{8}} + \overrightarrow{\left(a_{Concentrated} \cdot P_{GirderBDBGM} \cdot L_{Girder}\right)}$$

$$Vu_{GirderBDBGM} := \overrightarrow{\frac{wu_{GirderBDBGM} \cdot L_{Girder}}{2}} + \overrightarrow{\left(c_{Concentrated} \cdot P_{GirderBDBGM}\right)}$$

$$Mu_{GirderBDBGM} = \begin{pmatrix} 915.22 \\ 1554.03 \\ 2754.80 \\ 2624.98 \\ 3669.78 \\ 3246.97 \\ 909.39 \\ 2747.35 \\ 759.64 \\ 2266.18 \\ 1547.44 \end{pmatrix} kip \cdot ft \qquad Vu_{GirderBDBGM} = \begin{pmatrix} 105.60 \\ 143.95 \\ 243.23 \\ 291.66 \\ 407.75 \\ 282.35 \\ 104.93 \\ 242.75 \\ 87.65 \\ 213.00 \\ 143.49 \end{pmatrix} kip$$

Moment and shear due to seismic uniform loading and concentrated loading

Strength Margin Factor for Girders (Fs_{Girder})

$$Fs_{MomGirder} := \frac{M_{pxGirder} - Mu_{GirderNS}}{Mu_{GirderBDBGM}}$$

$$Fs_{MomGirder}{}^{T} = (1.02\ \ 1.44\ \ 1.51\ \ 1.28\ \ 1.16\ \ 1.21\ \ 1.16\ \ 1.71\ \ 1.73\ \ 1.93\ \ 1.68)$$

$$Fs_{ShearGirder} := \frac{V_{nxGirder} - Vu_{GirderNS}}{Vu_{GirderBDBGM}}$$

$$Fs_{ShearBeam}{}^{T} = (3.01\ \ 4.25\ \ 4.01\ \ 3.31\ \ 2.04\ \ 3.43\ \ 3.84\ \ 3.78\ \ 4.93\ \ 2.46)$$

Inelastic Energy Absorption Factor for Beams(Girders) (F_μ)

Per Table 5-1 of Ref. 2.2.6

$F_{\mu Mom} := 5.25$ F_μ for beams (girders) of SMRF steel moment frames - Limit State A

Based on Section 6.3 of this calculation, the F_μ for structural beams and girders must be reduced to account for ratcheting effects. Using equation 6.3.7 in Section 6.3, the revised F_μ is given by:

$N := 4$ Number of equal nonlinear response cycles (Assumption 3.2.4)

$$F_{\mu e} := 1 + \frac{F_{\mu Mom} - 1}{N} \qquad F_{\mu e} = 2.06$$

$F_{\mu MomGirder} := F_{\mu e}$ F_μ for bending in steel beams and girders

$F_{\mu Shear} := 1.0$ Shear failure is a brittle failure thus no inelastic energy absorption is considered.

HCLPF Capacity for Girders ($HCLPF_{Girders}$)

$$HCLPF_{MomGirder} := Fs_{MomGirder} \cdot F_{\mu e} \cdot PGA_h$$

$$HCLPF_{ShearGirder} := Fs_{ShearGirder} \cdot F_{\mu Shear} \cdot PGA_h$$

$$HCLPF_{MomGirder}{}^{T} = (1.91\ \ 2.72\ \ 2.84\ \ 2.41\ \ 2.19\ \ 2.27\ \ 2.18\ \ 3.21\ \ 3.27\ \ 3.64\ \ 3.18)\,g$$

$$HCLPF_{ShearGirder}{}^{T} = (3.68\ \ 4.99\ \ 3.98\ \ 2.62\ \ 2.47\ \ 3.36\ \ 3.98\ \ 4.43\ \ 5.17\ \ 4.60\ \ 5.32)\,g$$

Minimum HCLPF Capacity for Girders - Moment and Shear

$$HCLPF_{GirderM} := \min(HCLPF_{MomGirder}) \qquad HCLPF_{GirderM} = 1.91\,g$$

$$HCLPF_{GirderS} := \min(HCLPF_{ShearGirder}) \qquad HCLPF_{GirderS} = 2.47\,g$$

HCLPF Capacity for Girders - Summary

$HCLPF_{GirderM} = 1.91\,g$ (Bending)

$HCLPF_{GirderS} = 2.47\,g$ (Shear)

6.4.3 HCLPF Capacity Calculations for Interior Steel Columns

Case 1 - Columns at gridlines D.3-2.5, D.7-2.5, G.3-2.5, and G.7-2.5, elevation from 0'-0" to 64'-0". Supports second floor and roof framing members.

Case 2 - Columns at gridlines D.3-6.8, D.7-6.8, G.3-6.8, and G.7-6.8, elevation from 0'-0" to 32'-0". Supports floor framing members.

Case 3 - Columns at gridlines D.3-10, D.7-10, G.3-10, and G.7-10, elevation from 0'-0" to 64'-0". Supports second floor and roof framing members.

Case 4 - Columns at gridlines E.3-10, F-10, and F.7-10, elevation from 0'-0" to 32'-0". Supports second floor and roof framing members.

Case 5 - Columns at gridlines D.3-11, D.7-11, G.3-11, and G.7-11, elevation from 32'-0" to 64'-0". Supports roof framing members.

Column Size	Section Area	Radius of Gyration about weak axis	Stability Factor

$$\text{Column}_{\text{Size1}} := \begin{pmatrix} \text{"W14x257"} \\ \text{"W14x145"} \\ \text{"W14x233"} \\ \text{"W14x311"} \\ \text{"W14x176"} \end{pmatrix} \quad a := \begin{pmatrix} 75.6 \\ 42.7 \\ 68.5 \\ 91.4 \\ 51.8 \end{pmatrix} \cdot in^2 \quad r := \begin{pmatrix} 4.13 \\ 3.98 \\ 4.10 \\ 4.20 \\ 4.02 \end{pmatrix} \cdot in \quad K := \begin{pmatrix} 1.0 \\ 1.0 \\ 1.0 \\ 1.0 \\ 1.0 \end{pmatrix}$$

Height of Column (Unbraced Column Length between Floors)

$$L := \begin{pmatrix} 29 \\ 29 \\ 29 \\ 29 \\ 29 \end{pmatrix} \cdot ft$$

$$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E_s}{F_y}} \qquad C_c = 107$$

Equation Q1.5-1 Ref. 2.2.46

Allowable stress for compression members
Equations Q1.5-1 of Ref. 2.2.46

$$\overrightarrow{\frac{K \cdot L}{r}} = \begin{pmatrix} 84.26 \\ 87.44 \\ 84.88 \\ 82.86 \\ 86.57 \end{pmatrix}$$

$$F_{cr}(K, L, r) := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows}(K) \\ \quad \left| \begin{array}{l} \text{result}_i \leftarrow \dfrac{\left[1 - \dfrac{\left(\dfrac{K_i \cdot L_i}{r_i}\right)^2}{2 \cdot C_c^{\,2}}\right] \cdot F_y}{\dfrac{5}{3} + \dfrac{3 \cdot \dfrac{K_i \cdot L_i}{r_i}}{8 \cdot C_c} - \dfrac{\left(\dfrac{K_i \cdot L_i}{r_i}\right)^3}{8 \cdot C_c^{\,3}}} \quad \text{if } C_c \geq \dfrac{K_i \cdot L_i}{r_i} \\ \text{result}_i \leftarrow \dfrac{12 \cdot \pi^2 \cdot E_s}{23 \cdot \left(\dfrac{K_i \cdot L_i}{r_i}\right)^2} \quad \text{if } C_c < \dfrac{K_i \cdot L_i}{r_i} \end{array} \right. \\ \text{result} \end{array} \right.$$

$$F_{crit} := F_{cr}(K, L, r) \qquad F_{crit} = \begin{pmatrix} 18.15 \\ 17.48 \\ 18.02 \\ 18.44 \\ 17.67 \end{pmatrix} \text{ksi}$$

Allowable compressive force

$$P_n := \overrightarrow{\left(F_{crit} \cdot a\right)} \qquad P_n = \begin{pmatrix} 1371.91 \\ 746.57 \\ 1234.33 \\ 1684.97 \\ 915.16 \end{pmatrix} \text{kip}$$

Roof Tributary Area

$$A_{rf} := \begin{pmatrix} 26 \cdot 29 \\ 0 \\ 26 \cdot 25 \\ 32 \cdot 8.875 \\ 26 \cdot 36 \end{pmatrix} \cdot \text{ft}^2$$

Floor Tributary Area

$$A_{flr} := \begin{pmatrix} 26 \cdot 29 \\ 26 \cdot 29 \\ 26 \cdot 25 \\ 6.5 \cdot 32 \\ 0 \end{pmatrix} \cdot \text{ft}^2$$

Wall Load

$$\text{wall} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 384 \\ 0 \end{pmatrix} \cdot \text{kip}$$

Case 4 column supports the wall from EL. 32' to EL. 72' along column line 10. (32' ft trib length x 2 ft thick x 40 ft height x 0.150 kcf = 384 kips)

Non-Seismic Demand

The dead and live loads tributary to the columns are taken from the dead and live loads calculated for the beams (DL_{Beam}). The dead loads include the slab weight, equipment weight, hanging equipment weight, the beam and girder weight, the steel decking and roofing material (if the column supports roof).

$$DL_{Beam}^{T} = \begin{pmatrix} 339.1 & 339.5 & 527.7 & 328.3 & 515.2 & 330.2 & 344.0 & 325.1 & 328.4 & 313.0 \end{pmatrix} psf$$

$$Beam_{Cases} = \begin{pmatrix} \text{"B1 - EL. 64' - Gridline 2 to 5 \& 9 to 12 from D to E \& G to H"} \\ \text{"B2 - EL. 64' - Gridline 5 to 6 from D to E \& G to H"} \\ \text{"B3 - EL. 64' - Gridline 4 to 6 from E to G"} \\ \text{"B4 - EL. 72' - Gridline 9 to 12 from E to G"} \\ \text{"B5 - EL. 64' - Gridline 2 to 4 from E to G"} \\ \text{"B6 - EL. 100' - Gridline 6 to 9 from D to H"} \\ \text{"B7 - EL. 36' - Gridline 12 to 13 from E to G at Vestibule"} \\ \text{"B8 - EL. 32' - Gridline 2-5 \& 6-12 from D-E \& G-H, 9-10 from E-G"} \\ \text{"B9 - EL. 32' - Gridline 5-6 from D-E \& G-H "} \\ \text{"B10 - EL. 32' - Gridline 9-10 from E-G"} \end{pmatrix}$$

Roof Dead Loads

$$ColumnDL_{rf} := \begin{pmatrix} DL_{Beam_1} \\ 0psf \\ DL_{Beam_1} \\ DL_{Beam_4} \\ DL_{Beam_1} \end{pmatrix} \qquad ColumnDL_{rf} = \begin{pmatrix} 339.05 \\ 0.00 \\ 339.05 \\ 328.33 \\ 339.05 \end{pmatrix} psf$$

Floor Dead Loads

$$ColumnDL_{flr} := \begin{pmatrix} DL_{Beam_8} \\ DL_{Beam_8} \\ DL_{Beam_8} \\ DL_{Beam_{10}} \\ 0psf \end{pmatrix} \qquad ColumnDL_{flr} = \begin{pmatrix} 325.10 \\ 325.10 \\ 325.10 \\ 312.96 \\ 0.00 \end{pmatrix} psf$$

Roof Live Loads

$$\mathrm{ColumnLL}_{rf} := \begin{pmatrix} LL_{Beam_1} \\ 0\,psf \\ LL_{Beam_1} \\ LL_{Beam_4} \\ LL_{Beam_1} \end{pmatrix} \qquad \mathrm{ColumnLL}_{rf} = \begin{pmatrix} 40.00 \\ 0.00 \\ 40.00 \\ 40.00 \\ 40.00 \end{pmatrix} psf$$

Floor Live Loads

$$\mathrm{ColumnLL}_{flr} := \begin{pmatrix} LL_{Beam_8} \\ LL_{Beam_8} \\ LL_{Beam_8} \\ LL_{Beam_{10}} \\ 0\,psf \end{pmatrix} \qquad \mathrm{ColumnLL}_{flr} = \begin{pmatrix} 100.00 \\ 100.00 \\ 100.00 \\ 100.00 \\ 0.00 \end{pmatrix} psf$$

The vertical seismic accelerations for the columns are taken as the maximum vertical acceleration from the BDBGM_SRSS load combination (Ref. 2.2.5) at the upper elevation of the column.

$$Acc_v = \begin{pmatrix} 1.06 \\ 1.07 \\ 1.05 \\ 1.10 \end{pmatrix} g \qquad \begin{matrix} \text{EL. 32'} \\ \text{EL. 64'} \\ \text{EL. 72'} \\ \text{EL. 100'} \end{matrix}$$

Roof Vertical Seismic Accelerations

$$Acc_{rf} := amplify \cdot \begin{pmatrix} Acc_{v_2} \\ 0g \\ Acc_{v_2} \\ Acc_{v_3} \\ Acc_{v_2} \end{pmatrix} \qquad Acc_{rf} = \begin{pmatrix} 2.14 \\ 0.00 \\ 2.14 \\ 2.10 \\ 2.14 \end{pmatrix} g$$

- Case 2 column does not support a roof slab
- Amplify the vertical seismic load on the column per Assumption 3.1.8

Floor Vertical Seismic Accelerations

$$Acc_{flr} := amplify \cdot \begin{pmatrix} Acc_{v_1} \\ Acc_{v_1} \\ Acc_{v_1} \\ Acc_{v_1} \\ 0g \end{pmatrix} \qquad Acc_{flr} = \begin{pmatrix} 2.12 \\ 2.12 \\ 2.12 \\ 2.12 \\ 0.00 \end{pmatrix} g$$

- Amplify the vertical seismic load on the column per Assumption 3.1.8
- Case 5 column does not support a floor slab at EL. 32'-0".

Column Non-Seismic Axial Load

Non-seismic roof and floor loads

$$D_{NSrf} := \overrightarrow{\left[\left(ColumnDL_{rf} + 0.25 \cdot ColumnLL_{rf}\right) \cdot A_{rf}\right]}$$

$$D_{NSflr} := \overrightarrow{\left[\left(ColumnDL_{flr} + 0.25 \cdot ColumnLL_{flr}\right) \cdot A_{flr}\right]}$$

$$D_{NSrf} = \begin{pmatrix} 263.18 \\ 0.00 \\ 226.88 \\ 96.09 \\ 326.71 \end{pmatrix} kip$$

Non-seismic load acting the column supporting a roof

$$D_{NSflr} = \begin{pmatrix} 263.97 \\ 263.97 \\ 227.56 \\ 70.30 \\ 0.00 \end{pmatrix} kip$$

Non-seismic load acting the column supporting a floor

$$P_{NS} := \left(D_{NSrf} + D_{NSflr} + wall\right)$$

$$P_{NS} = \begin{pmatrix} 527.16 \\ 263.97 \\ 454.45 \\ 550.38 \\ 326.71 \end{pmatrix} kip$$

Total non-seismic load acting on the columns

Column Seismic Axial Load

$$P_{BDBGM} := \overrightarrow{\left(D_{NSrf} \cdot \frac{Acc_{rf}}{g}\right)} + \overrightarrow{\left(D_{NSflr} \cdot \frac{Acc_{flr}}{g}\right)} + \overrightarrow{\left(wall \cdot \frac{Acc_{rf}}{g}\right)}$$

The dead load from Wall 10 is amplified by the acceleration at the EL. 64' roof.

$$P_{BDBGM} = \begin{pmatrix} 1122.84 \\ 559.62 \\ 967.96 \\ 1157.21 \\ 699.16 \end{pmatrix} \text{kip}$$

Total seismic load acting on the columns

Strength Margin Factor (F_s)

$$C_{98\%} := 1.5 \cdot P_n$$

1.5 = Capacity increase factor for axial compression of columns per Table Q1.5.7.1 of Ref. 2.2.49.

$$C_{98\%} = \begin{pmatrix} 2057.86 \\ 1119.86 \\ 1851.50 \\ 2527.46 \\ 1372.75 \end{pmatrix} \text{kip}$$

$$Fs_{Column} := \frac{C_{98\%} - P_{NS}}{P_{BDBGM}} \qquad Fs_{Column} = \begin{pmatrix} 1.36 \\ 1.53 \\ 1.44 \\ 1.71 \\ 1.50 \end{pmatrix}$$

Inelastic Energy Absorption Factor ($F\mu$)

$$F\mu_{Column} := 1.0$$

Per Table 5-1 of Ref. 2.2.5, the inelastic energy absorption factor for compression

Column HCLPF Capacity

$$HCLPF_{Column} := \overrightarrow{\left(Fs_{Column} \cdot F\mu_{Column} \cdot PGA_h\right)} \qquad HCLPF_{Column} = \begin{pmatrix} 1.25 \\ 1.40 \\ 1.32 \\ 1.56 \\ 1.37 \end{pmatrix} \text{g}$$

Conclusion: The minimum HCLPF capacity for the CRCF columns is less than the HCLPF capacity of the in-plane shear walls (HCLPF = 1.82g). Therefore, increase the column sizes to obtain a HCLPF capacity of, at the least, 1.82g.

$$\text{Column}_{\text{Size}} := \begin{pmatrix} \text{"W14x398"} \\ \text{"W14x233"} \\ \text{"W14x370"} \\ \text{"W14x426"} \\ \text{"W14x257"} \end{pmatrix} \qquad a := \begin{pmatrix} 117 \\ 68.5 \\ 109 \\ 125 \\ 75.6 \end{pmatrix} \cdot \text{in}^2 \qquad r := \begin{pmatrix} 4.31 \\ 4.10 \\ 4.27 \\ 4.34 \\ 4.13 \end{pmatrix} \cdot \text{in} \qquad K := \begin{pmatrix} 1.0 \\ 1.0 \\ 1.0 \\ 1.0 \\ 1.0 \end{pmatrix}$$

Revised columns sizes and properties

$$F_{crit} := F_{cr}(K, L, r) \qquad F_{crit} = \begin{pmatrix} 18.86 \\ 18.02 \\ 18.71 \\ 18.98 \\ 18.15 \end{pmatrix} \text{ksi}$$

Allowable compressive force

$$P_n := \overrightarrow{\left(F_{crit} \cdot a\right)} \qquad P_n = \begin{pmatrix} 2207.04 \\ 1234.33 \\ 2039.51 \\ 2371.94 \\ 1371.91 \end{pmatrix} \text{kip}$$

Strength Margin Factor (F_s)

$$C_{98\%} := 1.5 \cdot P_n$$

1.5 = Capacity increase factor for axial compression of columns per Table Q1.5.7.1 of Ref. 2.2.49.

$$C_{98\%} = \begin{pmatrix} 3310.56 \\ 1851.50 \\ 3059.27 \\ 3557.91 \\ 2057.86 \end{pmatrix} \text{kip}$$

$$Fs_{Column} := \frac{C_{98\%} - P_{NS}}{P_{BDBGM}} \qquad Fs_{Column} = \begin{pmatrix} 2.48 \\ 2.84 \\ 2.69 \\ 2.60 \\ 2.48 \end{pmatrix}$$

Inelastic Energy Absorption Factor ($F\mu$)

$$F\mu_{Column} := 1.0$$

Per Table 5-1 of Ref. 2.2.5, the inelastic energy absorption factor for compression

Column HCLPF Capacity

$$HCLPF_{Column} := \overrightarrow{\left(Fs_{Column} \cdot F\mu_{Column} \cdot PGA_h\right)} \qquad HCLPF_{Column} = \begin{pmatrix} 2.27 \\ 2.59 \\ 2.46 \\ 2.37 \\ 2.26 \end{pmatrix} g$$

HCLPF Capacity for Columns - Summary

$$HCLPF_{Columns} := \min\left(HCLPF_{Column}\right)$$

$$HCLPF_{Columns} = 2.26\, g$$

Conclusion: All column HCLPF capacities are greater than the minimum HCLPF for the in-plane shear walls of the CRCF (1.82g). Therefore, the minimum HCLPF of the CRCF is controlled by the in-plane shear walls. The CRCF column sizes are given as follows:

$$Column_{Size} = \begin{pmatrix} \text{"W14x398"} \\ \text{"W14x233"} \\ \text{"W14x370"} \\ \text{"W14x426"} \\ \text{"W14x257"} \end{pmatrix}$$

Case 1 - Columns at gridlines D.3-2.5, D.7-2.5, G.3-2.5, and G.7-2.5, elevation from 0'-0" to 64'-0". Supports second floor and roof framing members.

Case 2 - Columns at gridlines D.3-6.8, D.7-6.8, G.3-6.8, and G.7-6.8, elevation from 0'-0" to 32'-0". Supports floor framing members.

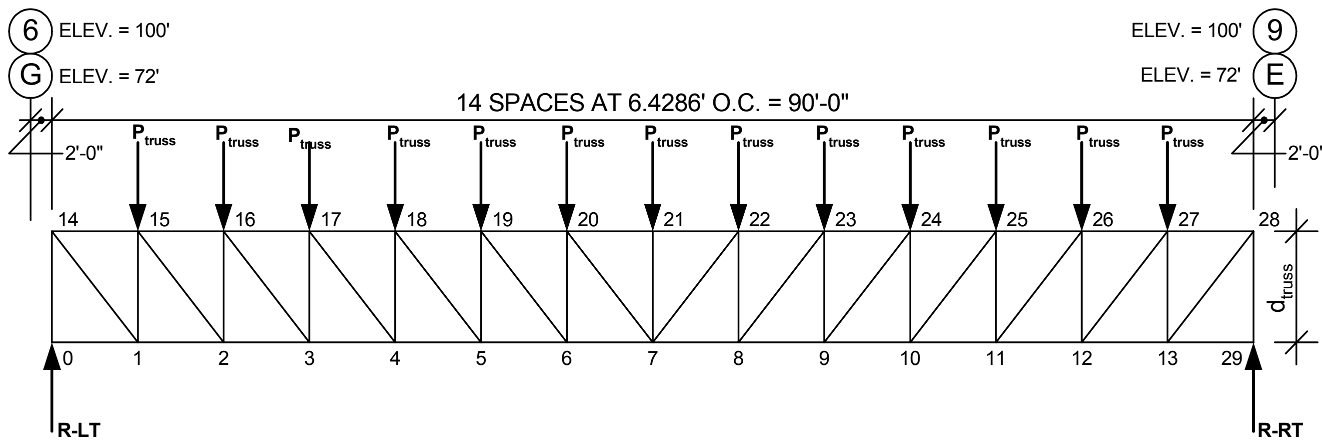
Case 3 - Columns at gridlines D.3-10, D.7-10, G.3-10, and G.7-10, elevation from 0'-0" to 64'-0". Supports second floor and roof framing members.

Case 4 - Columns at gridlines E.3-10, F-10, and F.7-10, elevation from 0'-0" to 32'-0". Supports second floor and roof framing members.

Case 5 - Columns at gridlines D.3-11, D.7-11, G.3-11, and G.7-11, elevation from 32'-0" to 64'-0". Supports roof framing members.

6.4.4 HCLPF Capacity Calculations for Truss T1

ROOF ELEV. 72'-0" - Truss Design between Gridline G to E, from 10 to 12, and
ROOF ELEV. 100'-0" - Truss Design between Gridline 6 to 9, from D to H.
Nominal concrete slab thickness = 1'-6"



TRUSS T1

$L_{truss} := 90ft$ — truss length

$sp := \frac{L_{truss}}{14}$ $\quad sp = 6.43\ ft$ — panel point spacing

$trib_{t1} := 21.17ft$ — tributary loading (T1 Grids 6-9/D-H)

$d_{truss} := 78.46in$ — truss depth center of top chord to center of bottom chord (overall depth D = 8'-0")

$h := \sqrt{sp^2 + d_{truss}^2}$ $\quad h = 9.17\ ft$ — diagonal web length

$D := d_{truss} + 0.5\cdot(17.54in + 17.54in)$ $\quad D = 8.00\ ft$ — maximum outer dimensions of truss

$cos\theta := \frac{d_{truss}}{h}$ $\quad cos\theta = 0.71$

Truss Self-Weight (Per Ref. 2.2.38)

$T1_{weight} := 2\cdot 342plf\cdot 89ft + 15\cdot 170plf\cdot d_{truss} + 14\cdot 170plf\cdot h$ $\quad T1_{weight} = 99.37\ kip$

$DL_{Truss1} := \frac{T1_{weight}}{L_{truss}}$ $\quad DL_{Truss1} = 1104.13\ plf$

Non-seismic load on truss T1 is the dead and live loads from Beam Case 6

$DL_{T1} := DL_{Beam_6}$ $\quad DL_{T1} = 330.21\ psf$

$LL_{T1} := LL_{Beam_6}$ $\quad LL_{T1} = 40.00\ psf$

$NS_{T1} := DL_{T1} + 0.25\cdot LL_{T1}$ $\quad NS_{T1} = 340.21\ psf$ — Non-seismic load acting tributary to Truss T1

Seismic load on truss T1

$Acc_{T1} := amplify\cdot Acc_{v_4}$ $\quad Acc_{T1} = 2.20\ g$ — Amplified vertical seismic acceleration at EL. 100'-0"

$Seismic_{T1} := NS_{T1} \cdot \frac{Acc_{T1}}{g}$ $\quad Seismic_{T1} = 748.47\,psf$ Seismic load acting tributary to Truss T1

$P_{ns\_truss} := \left(NS_{T1} \cdot trib_{t1} + DL_{Truss1}\right) \cdot sp$ $\quad P_{ns\_truss} = 53.40\,kip$ truss non-seismic panel point load

$P_{s\_truss} := \left(Seismic_{T1} \cdot trib_{t1}\right) \cdot sp$ $\quad P_{s\_truss} = 101.86\,kip$ truss seismic panel point load

$R_{ns} := P_{ns\_truss} \cdot 6.5$ $\quad R_{ns} = 347.09\,kip$ truss non-seismic reaction

$R_s := P_{s\_truss} \cdot 6.5$ $\quad R_s = 662.10\,kip$ truss seismic reaction

$$m20\_21_{ns} := \frac{R_{ns} \cdot 7(sp) - \left[\sum_{n=1}^{6} \left(n \cdot sp \cdot P_{ns\_truss}\right)\right]}{d_{truss}} \qquad m20\_21_{ns} = 1286.3\,kip$$

maximum non-seismic top-chord compression demand

$$m20\_21_{s} := \frac{R_{s} \cdot 7(sp) - \left[\sum_{n=1}^{6} \left(n \cdot sp \cdot P_{s\_truss}\right)\right]}{d_{truss}} \qquad m20\_21_{s} = 2453.7\,kip$$

maximum seismic top-chord compression demand

$$m6\_7_{ns} := \frac{R_{ns} \cdot 7(sp) - \left[\sum_{n=1}^{6} \left(n \cdot sp \cdot P_{ns\_truss}\right)\right]}{d_{truss}} \qquad m6\_7_{ns} = 1286.3\,kip$$

maximum non-seismic bottom-chord tension demand

$$m6\_7_{s} := \frac{R_{s} \cdot 7(sp) - \left[\sum_{n=1}^{6} \left(n \cdot sp \cdot P_{s\_truss}\right)\right]}{d_{truss}} \qquad m6\_7_{s} = 2453.7\,kip$$

maximum seismic bottom-chord tension demand

$m0\_14_{ns} := R_{ns}$ $\quad m0\_14_{ns} = 347.09\,kip$ maximum non-seismic vertical web compression demand

$m0\_14_{s} := R_{s}$ $\quad m0\_14_{s} = 662.10\,kip$ maximum seismic vertical web compression demand

$m1\_14_{ns} := R_{ns} \cdot \cos\theta^{-1}$ $\quad m1\_14_{ns} = 486.76\,kip$ maximum non-seismic diagonal web compression demand

$m1\_14_{s} := R_{s} \cdot \cos\theta^{-1}$ $\quad m1\_14_{s} = 928.52\,kip$ maximum seismic diagonal web compression demand

TRUSS COMPONENTS

TRUSS T1	member size	section area	radius of gyration	stability factor	member length
Top Chord Diag. Web Vert. Web Bott. Chord	$T1 := \begin{pmatrix} \text{"W14x342"} \\ \text{"W12x170"} \\ \text{"W12x170"} \\ \text{"W14x342"} \end{pmatrix}$	$a := \begin{pmatrix} 101.0 \\ 50.0 \\ 50.0 \\ 101.0 \end{pmatrix} in^2$	$r := \begin{pmatrix} 4.24 \\ 3.22 \\ 3.22 \\ 4.24 \end{pmatrix} in$	$K := \begin{pmatrix} 0.65 \\ 1.0 \\ 1.0 \\ 0.65 \end{pmatrix}$	$L := \begin{pmatrix} sp \\ h \\ d_{truss} \\ sp \end{pmatrix}$

Note: The top and bottom chord are considered as continuously supported axial members. Therefore, per Table C-C2.2 of Ref. 2.2.40, the K factor is 0.65.

Flexural buckling allowable stress for compression members

$$F_{crT1} := F_{cr}(K, L, r)$$

Allowable compressive force

$$P_n := \overrightarrow{(F_{crT1} \cdot a)} \qquad P_n = \begin{pmatrix} 2938.71 \\ 1331.11 \\ 1391.06 \\ 2938.71 \end{pmatrix} kip$$

Allowable Tensile Load

$$P_{tensile} := 0.60 \cdot F_y \cdot a \qquad P_{tensile} = \begin{pmatrix} 3030.00 \\ 1500.00 \\ 1500.00 \\ 3030.00 \end{pmatrix} kip$$

Q1.5.1.1 of Ref. 2.2.46

Member Capacities

$$P_{nT1} := \begin{pmatrix} P_{n_1} \\ P_{n_2} \\ P_{n_3} \\ P_{tensile_4} \end{pmatrix} \begin{matrix} \text{Compression} \\ \text{Compression} \\ \text{Compression} \\ \text{Tension} \end{matrix} \qquad P_{nT1} = \begin{pmatrix} 2939 \\ 1331 \\ 1391 \\ 3030 \end{pmatrix} kip$$

Non-Seismic Demand

$$P_{ns} := \begin{pmatrix} m20\_21_{ns} \\ m0\_14_{ns} \\ m1\_14_{ns} \\ m6\_7_{ns} \end{pmatrix} \qquad P_{ns} = \begin{pmatrix} 1286.30 \\ 347.09 \\ 486.76 \\ 1286.30 \end{pmatrix} kip$$

Seismic Demand

$$P_s := \begin{pmatrix} m20\_21_s \\ m0\_14_s \\ m1\_14_s \\ m6\_7_s \end{pmatrix} \qquad P_s = \begin{pmatrix} 2453.70 \\ 662.10 \\ 928.52 \\ 2453.70 \end{pmatrix} kip$$

$C_{98\%T1} := 1.5 \cdot P_{nT1}$ 1.5 = Capacity increase factor for axial compression of columns per Table Q1.5.7.1 of Ref. 2.2.49. Conservatively use 1.5 increase factor for the bottom chord (tension member)

$$C_{98\%T1} = \begin{pmatrix} 4408.06 \\ 1996.66 \\ 2086.59 \\ 4545.00 \end{pmatrix} kip$$

Strength Margin Factor (F_s)

$$F_{sT1} := \frac{C_{98\%T1} - P_{ns}}{P_s} \qquad F_{sT1} = \begin{pmatrix} 1.27 \\ 2.49 \\ 1.72 \\ 1.33 \end{pmatrix}$$

Inelastic Energy Absorption Factor ($F\mu$)

$F\mu := 5.25$ The truss is treated as a deep flexural member. Therefore, per Table 5-1 of Ref. 2.2.5, the inelastic energy absorption factor for beams and columns of SMRF steel moment frames at Limit State A is used.

Per Section 6.3.1.5 and Equation 6.3.7 of this calculation, the $F\mu$ must be reduced to account for ratcheting effects.

$N := 4$ (Assumption 3.2.4)

$$F_{\mu e} := 1 + \frac{F\mu - 1}{N} \qquad F_{\mu e} = 2.06$$

$F_{\mu Truss} := F_{\mu e}$ $F_{\mu Truss} = 2.06$ $F\mu$ factor used for truss

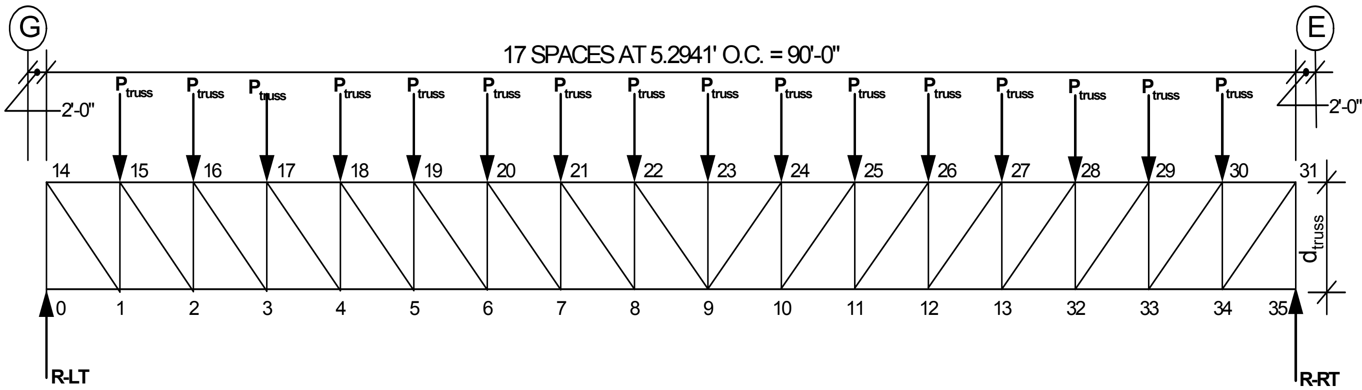
HCLPF Capacity

$$HCLPF_{T1} := \overrightarrow{\left(F_{sT1} \cdot F_{\mu Truss} \cdot PGA_h\right)} \qquad HCLPF_{T1} = \begin{pmatrix} 2.40 \\ 4.70 \\ 3.25 \\ 2.50 \end{pmatrix} g$$

6.4.5 HCLPF Capacity Calculations for Truss T2

TRUSS T2 CALCULATION (Total of 6 T2 trusses required)

ROOF ELEV. 64'-0" - Truss Design between Gridline G to E, from 2 to 4.
Nominal concrete slab thickness = 2'-9"



TRUSS T2

$L_{truss2} := 90ft$ — truss length

$sp2 := \dfrac{L_{truss2}}{17}$ $\quad sp2 = 5.29\,ft$ — panel point spacing

$trib_{t2} := 12ft$ — tributary loading (T2 Grids 2-4/E-G)

$d_{truss2} := 78.46in$ — truss depth

$h2 := \sqrt{sp2^2 + d_{truss2}^2}$ $\quad h = 9.17\,ft$ — diagonal web length

$D2 := d_{truss} + 0.5\cdot(17.54in + 17.54in)$ $\quad D = 8.00\,ft$ — maximum outer dimensions of truss

$\cos\theta := \dfrac{d_{truss2}}{h2}$ $\quad \cos\theta = 0.78$

Truss Self-Weight (Per Ref. 2.2.38)

$T2_{weight} := 2\cdot 342plf\cdot 89ft + 15\cdot 170plf\cdot d_{truss} + 14\cdot 170plf\cdot h$ $\quad T2_{weight} = 99.37\,kip$

$DL_{Truss2} := \dfrac{T2_{weight}}{L_{truss2}}$ $\quad DL_{Truss2} = 1104.13\,plf$

Non-seismic load on truss T2 is the dead and live loads from Beam Case 5

$DL_{T2} := DL_{Beam_5}$ $\quad DL_{T2} = 515.16\,psf$

$LL_{T2} := LL_{Beam_5}$ $\quad LL_{T2} = 40.00\,psf$

$NS_{T2} := DL_{T2} + 0.25\cdot LL_{T2}$ $\quad NS_{T2} = 525.16\,psf$ — Non-seismic load acting tributary to Truss T2

Seismic load on truss T2

$Acc_{T2} := amplify\cdot Acc_{v_2}$ $\quad Acc_{T2} = 2.14\,g$ — Amplified vertical seismic acceleration at EL. 64'-0"

$Seismic_{T2} := NS_{T2}\cdot\dfrac{Acc_{T2}}{g}$ $\quad Seismic_{T2} = 1123.85\,psf$ — Seismic load acting tributary to Truss T2

$P_{ns\_truss2} := \left(NS_{T2} \cdot trib_{t2} + DL_{Truss2}\right) \cdot sp2$ $P_{ns\_truss2} = 39.21\,kip$ truss non-seismic panel point load

$P_{s\_truss2} := \left(Seismic_{T2} \cdot trib_{t2}\right) \cdot sp2$ $P_{s\_truss2} = 71.40\,kip$ truss seismic panel point load

$R_{ns2} := P_{ns\_truss2} \cdot 8$ $R_{ns2} = 313.67\,kip$ truss non-seismic reaction

$R_{s2} := P_{s\_truss2} \cdot 8$ $R_{s2} = 571.18\,kip$ truss seismic reaction

$$m22\_23_{ns2} := \frac{R_{ns2} \cdot 9(sp2) - \left[\sum_{n=1}^{8} \left(n \cdot sp2 \cdot P_{ns\_truss2}\right)\right]}{d_{truss2}}$$

$m22\_23_{ns2} = 1142.9\,kip$ maximum non-seismic top-chord compression demand (Moment about point 9)

$$m22\_23_{s2} := \frac{R_{s2} \cdot 9(sp2) - \left[\sum_{n=1}^{8} \left(n \cdot sp2 \cdot P_{s\_truss2}\right)\right]}{d_{truss2}}$$

$m22\_23_{s2} = 2081.2\,kip$ maximum seismic top-chord compression demand (Moment about point 9)

$$m8\_9_{ns2} := \frac{R_{ns2} \cdot 8(sp2) - \left[\sum_{n=1}^{7} \left(n \cdot sp2 \cdot P_{ns\_truss2}\right)\right]}{d_{truss2}}$$

$m8\_9_{ns2} = 1142.9\,kip$ maximum non-seismic bottom-chord tension demand (Moment about point 22)

$$m8\_9_{s2} := \frac{R_{s2} \cdot 8(sp2) - \left[\sum_{n=1}^{7} \left(n \cdot sp2 \cdot P_{s\_truss2}\right)\right]}{d_{truss2}}$$

$m8\_9_{s2} = 2081.2\,kip$ maximum seismic bottom-chord tension demand (Moment about point 22)

$m0\_14_{ns2} := R_{ns2}$ $m0\_14_{ns2} = 313.67\,kip$ maximum non-seismic vertical web compression demand

$m0\_14_{s2} := R_{s2}$ $m0\_14_{s2} = 571.18\,kip$ maximum seismic vertical web compression demand

$m1\_14_{ns2} := R_{ns2} \cdot \cos\theta^{-1}$ $m1\_14_{ns2} = 403.60\,kip$ maximum non-seismic diagonal web compression demand

$m1\_14_{s2} := R_{s2} \cdot \cos\theta^{-1}$ $m1\_14_{s2} = 734.94\,kip$ maximum seismic diagonal web compression demand

TRUSS COMPONENTS

member	section	radius of	stability	member

TRUSS T2	size	area	gyration	factor	length
Top Chord	"W14x342"	101.0	4.24	0.65	sp2
Diag. Web	"W12x170"	50.0	3.22	1.0	h2
Vert. Web	"W12x170"	50.0	3.22	1.0	d_{truss2}
Bott. Chord	"W14x342"	101.0	4.24	0.65	sp2

$$T2 := \begin{pmatrix} \text{"W14x342"} \\ \text{"W12x170"} \\ \text{"W12x170"} \\ \text{"W14x342"} \end{pmatrix} \quad a := \begin{pmatrix} 101.0 \\ 50.0 \\ 50.0 \\ 101.0 \end{pmatrix} in^2 \quad r := \begin{pmatrix} 4.24 \\ 3.22 \\ 3.22 \\ 4.24 \end{pmatrix} in \quad K := \begin{pmatrix} 0.65 \\ 1.0 \\ 1.0 \\ 0.65 \end{pmatrix} \quad L := \begin{pmatrix} sp2 \\ h2 \\ d_{truss2} \\ sp2 \end{pmatrix}$$

Note: The top and bottom chord are considered as continuously supported axial members. Therefore, per Table C-C2.2 of Ref. 2.2.40, the K factor is 0.65.

Flexural buckling allowable stress for compression members

$$F_{crT2} := F_{cr}(K, L, r)$$

Allowable compressive load

$$P_n := \overrightarrow{(F_{crT2} \cdot a)} \qquad P_n = \begin{pmatrix} 2957.06 \\ 1349.20 \\ 1391.06 \\ 2957.06 \end{pmatrix} kip$$

Allowable tensile load

$$P_{tensile} := 0.60 F_y \cdot a$$ Q1.5.1.1 of Ref. 2.2.46

Member Capacities

$$P_{nT2} := \begin{pmatrix} P_{n_1} \\ P_{n_2} \\ P_{n_3} \\ P_{tensile_4} \end{pmatrix} \qquad P_{nT2} = \begin{pmatrix} 2957 \\ 1349 \\ 1391 \\ 3030 \end{pmatrix} kip$$

Non-Seismic Demand

$$P_{ns} := \begin{pmatrix} m22\_23_{ns2} \\ m0\_14_{ns2} \\ m1\_14_{ns2} \\ m8\_9_{ns2} \end{pmatrix} \qquad P_{ns} = \begin{pmatrix} 1142.91 \\ 313.67 \\ 403.60 \\ 1142.91 \end{pmatrix} kip$$

Seismic Demand

$$P_s := \begin{pmatrix} m22\_23_{s2} \\ m0\_14_{s2} \\ m1\_14_{s2} \\ m8\_9_{s2} \end{pmatrix} \qquad P_s = \begin{pmatrix} 2081.19 \\ 571.18 \\ 734.94 \\ 2081.19 \end{pmatrix} kip$$

$C_{98\%T2} := 1.5 \cdot P_{nT2}$ 1.5 = Capacity increase factor for axial compression of columns per Table Q1.5.7.1 of Ref. 2.2.49. Conservatively use 1.5 increase factor for the bottom chord (tension member)

$$C_{98\%T2} = \begin{pmatrix} 4435.58 \\ 2023.80 \\ 2086.59 \\ 4545.00 \end{pmatrix} kip$$

Strength Margin Factor (F_s)

$$F_{sT2} := \frac{C_{98\%T2} - P_{ns}}{P_s} \qquad F_{sT2} = \begin{pmatrix} 1.58 \\ 2.99 \\ 2.29 \\ 1.63 \end{pmatrix}$$

Inelastic Energy Absorption Factor ($F\mu$)

$F\mu := 5.25$ The truss is treated as a deep flexural member. Therefore, per Table 5-1 of Ref. 2.2.5, the inelastic energy absorption factor for beams and columns of SMRF steel moment frames at Limit State A is used.

Per Section 6.3.1.5 and Equation 6.3.7 of this calculation, the $F\mu$ must be reduced to account for ratcheting effects.

$N := 4$ (Assumption 3.2.4)

$$F_{\mu e} := 1 + \frac{F\mu - 1}{N} \qquad F_{\mu e} = 2.06$$

$F_{\mu Truss} := F_{\mu e}$ $F\mu$ factor used for truss

HCLPF Capacity

$$HCLPF_{T2} := \overrightarrow{\left(F_{sT2} \cdot F_{\mu Truss} \cdot PGA_h\right)} \qquad HCLPF_{T2} = \begin{pmatrix} 2.98 \\ 5.64 \\ 4.32 \\ 3.08 \end{pmatrix} g$$

HCLPF Capacity for Trusses - Summary

$$HCLPF_{Trusses} := \min\left(HCLPF_{T1}, HCLPF_{T2}\right)$$

$HCLPF_{Trusses} = 2.40\,g$

BEAMS **GIRDERS** **COLUMNS**

$$Beam_{Size} = \begin{pmatrix} "W16x31" \\ "W24x68" \\ "W24x55" \\ "W18x40" \\ "W12x30" \\ "W18x55" \\ "W21x44" \\ "W18x35" \\ "W18x40" \\ "W10x39" \end{pmatrix} \quad Girder_{Size} = \begin{pmatrix} "W30x116" \\ "W36x210" \\ "W36x359" \\ "W36x300" \\ "W36x393" \\ "W36x359" \\ "W33x118" \\ "W36x393" \\ "W33x130" \\ "W36x359" \\ "W36x232" \end{pmatrix} \quad Column_{Size} = \begin{pmatrix} "W14x398" \\ "W14x233" \\ "W14x370" \\ "W14x426" \\ "W14x257" \end{pmatrix}$$

TRUSS T1 **TRUSS T2**

$$T1 = \begin{pmatrix} "W14x342" \\ "W12x170" \\ "W12x170" \\ "W14x342" \end{pmatrix} \quad T2 = \begin{pmatrix} "W14x342" \\ "W12x170" \\ "W12x170" \\ "W14x342" \end{pmatrix}$$

6.4.6: Structural Steel HCLPF Capacity Evaluation Summary

The minimum HCLPF capacities for the structural steel beams, girders, columns, and truss members are as follows -

$HCLPF_{BeamsM} = 1.82\,g$ HCLPF capacity for beam bending

$F_{\mu MomBeam} = 2.06$ Fμ for bending moment in beams

$HCLPF_{BeamsS} = 1.87\,g$ HCLPF capacity for beam shear

$F_{\mu Shear} = 1.00$ Fμ for shear in beams

$HCLPF_{GirderM} = 1.91\,g$ HCLPF capacity for girder bending

$F_{\mu MomGirder} = 2.06$ Fμ for bending moment in girders

$HCLPF_{GirderS} = 2.47\,g$ HCLPF capacity for girder shear

$F_{\mu Shear} = 1.00$ Fμ for shear in girders

$HCLPF_{Columns} = 2.26\,g$ HCLPF capacity for columns

$F\mu_{Column} = 1.00$ Fμ for columns

$HCLPF_{Trusses} = 2.40\,g$ HCLPF capacity for truss members

$F_{\mu Truss} = 2.06$ Fμ for trusses

Increasing the columns sizes is required to ensure that the HCLPF capacity of the shear walls is the controlling HCLPF capacity for the CRCF.

Columns C1, C2, C3, C4 and C5 were increased in size as follows
Column C1 = W14x257 to W14x398
Column C2 = W14x145 to W14x233
Column C3 = W14x233 to W14x370
Column C4 = W14x311 to W14x426
Column C5 = W14x176 to W14x257

All other member sizes are the same.

After changing the columns sizes as indicated above, all HCLPF capacities for the interior structural steel of the CRCF are greater than or equal to the minimum HLCPF capacity of the CRCF shear walls (1.82g). Therefore, the minimum HCLPF capacity of the CRCF is not controlled by the structural steel.

6.5 HCLPF CAPACITY EVALUATIONS FOR AXIAL FORCE IN COMBINATION WITH IN-PLANE BENDING OF WALLS

For the HCLPF evaluation for the individual stick element under combined effects of in-plane bending and axial force, a strain-compatible analysis is performed to obtain an interaction diagram. The strain and stress diagrams considered in this analysis are shown in Figure 6.5.1. A point on this diagram represents the axial load capacity, Pc, and the corresponding moment capacity, Mc, for a given eccentricity The next step is to calculate the in-plane bending moment and the axial force corresponding to the minimum in-plane shear HCLPF value for the entire structure (HCLPF = 1.82g for Wall 5 at EL. 0'-0"). That is to say the axial load, P, and bending moment, M, in the wall when the Wall 5 reaches its HCLPF capacity. These moments and axial forces are given as:

$$P_T := P_{NS} + \frac{P_{HCLPF}}{F\mu} \qquad \text{(Eq 6.5.1)}$$

$$M_T := M_{NS} + \frac{M_{HCLPF}}{F\mu} \qquad \text{(Eq 6.5.2)}$$

where:
P_T = total axial demand
P_{NS} = non-seismic axial force
P_{HCLPF} = seismic axial force once the minimum HCLPF level in the shear walls has been reached
M_T = total in-plane bending moment demand
M_{NS} = non-seismic in-plane bending moment
M_{HCLPF} = seismic in-plane bending moment once the minimum HCLPF level in the shear walls has been reached
$F\mu$ = energy dissipation factor considered in the determination of the minimum HCLPF of the shear walls (1.75)

P_{HCLPF} and M_{HCLPF} are given as follows:

$$P_{HCLPF} := P_{BDBGM} \cdot \frac{HCLPF}{PGA_h}$$

$$M_{HCLPF} := M_{BDBGM} \cdot \frac{HCLPF}{PGA_h}$$

where:
P_{BDBGM} = axial force due to BDBGM seismic loading
M_{BDBGM} = in-plane bending moment due to BDBGM seismic loading
HCLPF = minimum HCLPF capacity of the CRCF shear walls (= 1.82g according to Section 6.2.7 of this calculation)
PGA_h = horizontal peak ground acceleration of BDBGM seismic ground motion. (= 0.9138g according to Ref. 2.2.31)

The P_{HCLPF} and M_{HCLPF} formulas above normalize the BDBGM in-plane bending moment (M_{BDBGM}) and axial force (P_{BDBGM}) to the seismic level at which the in-plane shear walls reaches its minimum HCLPF capacity.

The total axial force, P_T, and in-plane bending moment, M_T, calculated above are then plotted on the interaction diagram. If the point falls within the diagram, the section is adequate. If not, the interaction diagram is altered, either through increasing the vertical reinforcement of the wall or by considering any vertical steel lumped at the wall ends due to openings or flange effects of perpendicular walls, and the evaluation is repeated until all points lie within the interaction diagram.

The following steps describe the procedure for generating the interaction diagram and plotting P_T and M_T points in order to determine if the wall is adequate. All references to Excel sheets and Excel columns contained within these sheets is for the Excel file "CRCF - Fragility - In-Plane Bending and Axial Force.xls" included in Attachment F.

Step 1: Input the required wall design parameters (sheet "Interaction Diagram")
The input are the cells highlighted in light turquoise (bluish color). All information contained in cells highlighted in yellow and tan are determined by built-in Excel formulas (such as vlookup and hlookup).

Cell "C7" and "C8" contain the concrete compressive strength and steel yield strength used in the evaluation.

Column "B" - number of the stick given in column C.
Column "C" - stick element ID
Column "D" - wall length
Column "E" - wall thickness
Column "F" - vertical reinforcement of wall

Cell "J2" - minimum HCLPF for the CRCF shear walls given in Section 6.2.7.
Cell "J3" - horizontal PGA
Cell "J4" - F_μ factor used in the determination of the HCLPF value given in Section 6.2.7

Cell "J6" - number of the stick for which the interaction diagram is created

Cell "K8" - number of vertical reinforcing bars lumped at each end of the wall that are to be included in the creation of the interaction diagram. If no additional vertical bars are considered, this value = 0. Note: if considering additional lumped vertical reinforcement, this additional reinforcement must be placed at each end of the wall.
Cell "K10" - bar number of the lumped vertical reinforcement
Cell "K11" - percentage of the wall length where the additional reinforcement is placed.

The following shows a snapshot of the information contained in the file "CRCF - Fragility - In-Plane Bending and Axial Force.xls" included in Attachment F. This snapshot shows that the interaction diagram for stick element "5A.2" is generated with 4-#11 bars added at each end of the wall placed over a length equal to 5% of the wall.

	B	C	D	E	F	G	H	I	J	K
2								HCLPF	1.82	g
3								PGA	0.9138	g
4								F□	1.75	
5										
6							Requested Stick No.		28	5A.2
7	f'c	5,500	psi	See section 6.2.4.1 of this calculation						
8	fy	60,000	psi	Steel yield strength per Ref. 2.2.1			No. extra Vert. Bars on each end of			4
9							wall (= 0 if no extra bars)			
10		Data from figures in Attachment A			Data from Attachment B		Extra Bar No.			11
11							Extra Bar Location (% lw)			5.0%
12		Wall Design Parameters								

Step 2: Determine In-Plane Bending and Axial Force on the wall from BDBGM seismic and non-seismic loads
The controlling load combinations for this evaluation are:

Non-Seismic ($D_{Nonseismic}$) = Load case DL+LL in seismic analysis results from Ref. 2.2.5

Seismic (D_{BDBGM}) = Seismic load from seismic analysis results from Ref. 2.2.5. (Load cases "BDBGM 20-10HX", "BDBGM-20-10HY" and "BDBGM-VERT" are the X (E-W), Y (N-S), and vertical direction seismic load respectively.)

For this evaluation, the seismic loads are combined using the 100% - 40% - 40% rule. The following are the seismic load combinations, where EQX = "BDBGM 20-10HX", EQY = "BDBGM-20-10HY", and EQZ = "BDBGM-VERT".

The following are the seismic load combinations used in the In-Plane Bending + Axial HCLPF evaluation.

Load Case #
1 = 1.0EQX + 0.4EQY + 0.4EQZ
2 = 1.0EQX + 0.4EQY - 0.4EQZ
3 = 1.0EQX - 0.4EQY - 0.4EQZ
4 = 1.0EQX - 0.4EQY + 0.4EQZ
5 = -1.0EQX + 0.4EQY + 0.4EQZ
6 = -1.0EQX + 0.4EQY - 0.4EQZ
7 = -1.0EQX - 0.4EQY - 0.4EQZ
8 = -1.0EQX - 0.4EQY + 0.4EQZ

9 = 0.4EQX + 1.0EQY + 0.4EQZ
10 = 0.4EQX + 1.0EQY - 0.4EQZ
11 = 0.4EQX - 1.0EQY - 0.4EQZ
12 = 0.4EQX - 1.0EQY + 0.4EQZ
13 = -0.4EQX + 1.0EQY + 0.4EQZ
14 = -0.4EQX + 1.0EQY - 0.4EQZ
15 = -0.4EQX - 1.0EQY - 0.4EQZ
16 = -0.4EQX - 1.0EQY + 0.4EQZ

17 = 0.4EQX + 0.4EQY + 1.0EQZ
18 = 0.4EQX + 0.4EQY - 1.0EQZ
19 = 0.4EQX - 0.4EQY - 1.0EQZ
20 = 0.4EQX - 0.4EQY + 1.0EQZ
21 = -0.4EQX + 0.4EQY + 1.0EQZ
22 = -0.4EQX + 0.4EQY - 1.0EQZ
23 = -0.4EQX - 0.4EQY - 1.0EQZ
24 = -0.4EQX - 0.4EQY + 1.0EQZ

The BDBGM seismic results for load cases "DL+LL", "BDBGM 20-10HX", "BDBGM-20-10HY" and "BDBGM-VERT" for the individual stick elements are given on the sheet "Element Forces - Frames SAP2000". The axial force (P), in-plane shear (V2), and in-plane bending moment (M3) for each stick element for the "DL+LL" and each of the 24 seismic load combinations are given on sheet "Stick Forces 100+40+40".

On Sheet "Interaction Diagram", columns I-K from rows 15 to 38 contain the P, V2, and M3 results for the 24 seismic load cases associated with the stick element identified in cell "K6". These values are retrieved from the results contained on the sheet "Stick Forces 100+40+40".

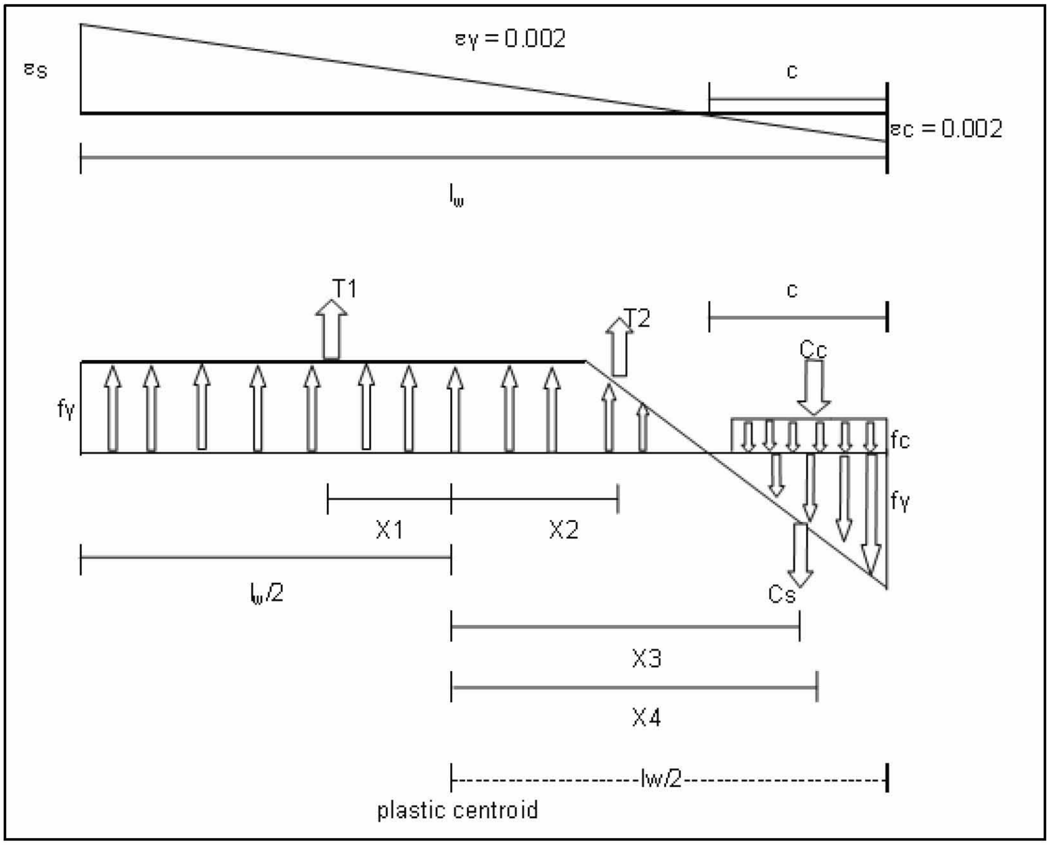
On Sheet "Interaction Diagram", columns N-P from rows 15 to 38 contain the P, V2, and M3 results for the "DL+LL" load case associated with the stick element identified in cell "K6". Notice the P, V2, and M3 values are the same for each seismic load case. (i.e. the "DL+LL" is the same for all seismic load combinations").

On Sheet "Interaction Diagram", column S rows 15 to 38 contain the P_T (labeled P_{total}) results calculated using equation 6.5.1 Column T rows 15 to 38 contain the M_T (labeled M_{total}) results calculated using equation 6.5.2.

These results are in units of $10e^3$ kips and $10e^3$ kip-ft, respectively, and are the design load cases plotted on the interaction diagram developed in Step 3.

Step 3: Develop Interaction Diagram
Figure 6.5.1 shows the strain and stress diagrams used to develop the interaction diagram

Figure 6.5.1 Stress-Strain Diagram used in Interaction Diagram Development



- The concrete strain is limited to 0.002 per SADA (Ref. 2.2.4).
- For a given wall length (lw), wall thickness (tw), and vertical reinforcement uniformly spaced along the length of wall, the distance from the compression face to the neutral axis (c) is arbitrarily selected.
- Next, using similar triangles, the following values are determined

es_max = maximum strain in steel
X1 = distance from plastic centroid to center of tension steel that has yielded
X2 = distance from plastic centroid to center of tension steel that has not yielded
X3 = distance from plastic centroid to center of compression steel
X4 = distance from plastic centroid to center of compression block
X1extra = distance from plastic centroid to center of tension steel lumped at the wall end that has yielded
X2extra = distance from plastic centroid to center of tension steel lumped at the wall end that has not yielded
X3extra = distance from plastic centroid to center of compression steel lumped at the wall end

- Next, the following forces acting on the wall are determined

T1 = force in tension steel that has yielded
T2 = force in tension steel that has not yielded
Cs = force in compression steel
Cc = force in concrete compression block
T1extra = force in tension steel lumped at the wall end that has yielded
T2extra = force in tension steel lumped at the wall end that has not yielded
Csextra = force in compression steel lumped at the wall end

The vertical steel lumped at the wall ends is considered to be placed within a certain length of the wall defined as a percentage of the total wall length. Also, this evaluation considers that equal amounts of steel are lumped at each end of the wall.

The total axial capacity of the wall is then given by

P = T1 + T2 + Cs + Cc + T1extra + T2extra + Csextra

and the total moment capacity of the wall is given by

M = (T1 x X1) + (T2 x X2) + (Cs x X3) + (Cc x X4) + (T1extra x X1extra) + (T2extra x X2extra) + (Csextra x X3extra)

The appropriate ϕ factors are then applied to P and M to determine the design capacities for the assumed c value. Another c value is assumed and the proceeding steps are repeated until the entire interaction diagram is developed.

The following are the derivations for X1, X2, X3, X4, T1, T2, Cs, Cc, P, and M. These are the equations contained in the appropriate columns in sheet "Shear Wall Design Template" in the file "CRCF - Fragility - In-Plane Bending and Axial Force.xls" included in Attachment F.

Wall Design Parameters

$\varepsilon_c := 0.002$ Limit concrete compressive strength to 0.002 per SADA (Ref. 2.2.4)

$f_c := 5500\text{psi}$ Concrete compressive strength per section 6.2.4.1 of this calculation

$E_s := 29000\text{ksi}$ Steel elastic modulus (Ref. 2.2.1)

$f_y := 60\text{ksi}$ Steel yield stress (Ref. 2.2.1)

$$\beta_1 := \left| \begin{array}{l} 0.85 \quad \text{if } f_c \le 4000\text{psi} \\ 1.05 - 0.05\cdot\dfrac{f_c}{1000}\cdot\dfrac{1}{\text{psi}} \quad \text{if } 4000\text{psi} < f_c < 8000\text{psi} \\ 0.65 \quad \text{otherwise} \end{array} \right.$$

(Section 4-2 in Ref. 2.2.36)

$\beta_1 = 0.78$

X1 = distance from plastic centroid to center of tension steel that has yielded

$$X1(c, l_w) := \left| \begin{array}{l} -c \quad \text{if } c \le \dfrac{l_w}{2} \\ 0\text{ft} \quad \text{otherwise} \end{array} \right.$$

(Column J)

X2 = distance from plastic centroid to center of tension steel that has not yielded

$$X2(c, l_w) := \left| \begin{array}{l} \text{result} \leftarrow \dfrac{l_w}{2} - \dfrac{5}{3}\cdot c \quad \text{if } c \le \dfrac{l_w}{2} \\ \text{result} \leftarrow -\left(\dfrac{l_w}{6} + \dfrac{1}{3}c\right) \quad \text{if } \dfrac{l_w}{2} < c \le l_w \\ \text{result} \leftarrow 0\text{ft} \quad \text{if } c > l_w \\ \text{result} \end{array} \right.$$

(Column K)

X3 = distance from plastic centroid to center of compression steel

$$X3(c, l_w) := \begin{cases} \dfrac{l_w}{2} - \dfrac{c}{3} & \text{if } c \le l_w \\ \text{otherwise} & \\ \quad \begin{array}{l} y_c \leftarrow \dfrac{c - l_w}{c} \cdot \varepsilon_c \\ C_T \leftarrow y_c \cdot l_w + \dfrac{1}{2} \cdot l_w \cdot (\varepsilon_c - y_c) \\ M1 \leftarrow (l_w \cdot y_c) \cdot \dfrac{l_w}{2} \\ M2 \leftarrow \dfrac{1}{2} \cdot l_w \cdot (\varepsilon_c - y_c) \cdot \dfrac{1}{3} \cdot l_w \\ R \leftarrow \dfrac{M1 + M2}{C_T} \\ \dfrac{l_w}{2} - R \end{array} & \end{cases}$$

(Column L)

X4 = distance from plastic centroid to center of compression block

$$X4(c, l_w) := \frac{l_w}{2} - \frac{\beta_1 \cdot c}{2}$$

(Column M)

X1extra = distance from plastic centroid to center of tension steel lumped at the wall end that has yielded

$$X1extra(c, l_w, \%) := \begin{cases} -\dfrac{l_w}{2} \cdot (1 - \%) & \text{if } c < \dfrac{l_w}{2} \\ 0ft & \text{otherwise} \end{cases}$$

(Column U)

X2extra = distance from plastic centroid to center of tension steel lumped at the wall end that has not yielded

$$X2extra(c, l_w, \%) := \begin{cases} \dfrac{-l_w}{2} \cdot (1 - \%) & \text{if } c \ge \dfrac{l_w}{2} \\ 0ft & \text{otherwise} \end{cases}$$

(Column V)

X3extra = distance from plastic centroid to center of compression steel lumped at the wall end

$$X3extra(c, l_w, \%) := \frac{l_w}{2} \cdot (1 - \%)$$

(Column W)

T1 = force due to tension steel that has yielded

$$T1(c, l_w, t_w, \rho_v) := \begin{cases} -\left[(E_s \cdot \varepsilon_c) \cdot \rho_v \cdot t_w \cdot (l_w - 2c)\right] & \text{if } c < \dfrac{l_w}{2} \\ 0kip & \text{otherwise} \end{cases}$$

(Column C)

T2 = force due to tension steel that has not yielded

$$T2(c, l_w, t_w, \rho_v) := \left| \begin{array}{l} \text{result} \leftarrow -\frac{1}{2}(E_s \cdot \varepsilon_c) \cdot \rho_v \cdot t_w \cdot c \quad \text{if } c \le \frac{l_w}{2} \\ \text{if } \frac{l_w}{2} < c < l_w \\ \quad \left| \begin{array}{l} \varepsilon_{steel} \leftarrow \frac{l_w - c}{c} \cdot \varepsilon_c \\ \text{result} \leftarrow -\frac{1}{2} \cdot (E_s \cdot \varepsilon_{steel}) \cdot \rho_v \cdot t_w \cdot (l_w - c) \end{array} \right. \\ \text{result} \leftarrow 0\text{kip} \quad \text{if } c \ge l_w \\ \text{result} \end{array} \right.$$

(Column D)

Cs = force due to compression steel

$$C_s(c, l_w, t_w, \rho_v) := \left| \begin{array}{l} \text{result} \leftarrow \frac{1}{2}(E_s \cdot \varepsilon_c) \cdot \rho_v \cdot t_w \cdot c \quad \text{if } c \le l_w \\ \text{otherwise} \\ \quad \left| \begin{array}{l} \varepsilon_{c1} \leftarrow \frac{c - l_w}{c} \cdot \varepsilon_c \\ C_{s1} \leftarrow E_s \cdot \varepsilon_{c1} \cdot \rho_v \cdot t_w \cdot l_w \\ C_{s2} \leftarrow \frac{1}{2} \cdot E_s \cdot (\varepsilon_c - \varepsilon_{c1}) \cdot \rho_v \cdot t_w \cdot l_w \\ \text{result} \leftarrow C_{s1} + C_{s2} \end{array} \right. \\ \text{result} \end{array} \right.$$

(Column E)

Cc = force due to concrete compression block

$$C_c(c, t_w) := 0.85 \cdot f_c \cdot \beta_1 \cdot c \cdot t_w$$

(Column F)

T1extra = force in tension steel lumped at the wall end that has yielded

num = number of vertical bars lumped at each end of the wall
As = area of bar lumped at the end of each wall

$$T1extra(c, l_w, num, A_s, \%) := \left| \begin{array}{l} -num \cdot A_s \cdot (\varepsilon_c \cdot E_s) \quad \text{if } c < \frac{l_w}{2} \cdot \left(1 - \frac{\%}{2}\right) \\ 0\text{kip} \quad \text{otherwise} \end{array} \right.$$

(Column R)

T2extra = force in tension steel lumped at the wall end that has not yielded

$$T2extra(c, l_w, num, A_s, \%) := \left| \begin{array}{l} result \leftarrow 0kip \quad \text{if } c \le \frac{l_w}{2}\left(1 - \frac{\%}{2}\right) \\ \text{if } \frac{l_w}{2}\cdot\left(1 - \frac{\%}{2}\right) < c \le l_w\cdot\left(1 - \frac{\%}{2}\right) \\ \quad \left| \begin{array}{l} \varepsilon_{T2} \leftarrow \left[\left(1 - \frac{\%}{2}\right)\cdot l_w - c\right]\cdot\frac{\varepsilon_c}{c} \\ result \leftarrow -num\cdot A_s\cdot\varepsilon_{T2}\cdot E_s \end{array} \right. \\ \text{if } c > l_w\cdot\left(1 - \frac{\%}{2}\right) \\ \quad \left| \begin{array}{l} \varepsilon_{T2} \leftarrow \frac{c - l_w}{c}\cdot\varepsilon_c \\ result \leftarrow num\cdot A_s\cdot\left(\varepsilon_{T2}\cdot E_s\right) \end{array} \right. \\ result \leftarrow 0kip \quad \text{otherwise} \\ result \end{array} \right.$$

(Column S)

Csextra = force in compression steel lumped at the wall end

$$Csextra(c, l_w, num, A_s, \%) := \left| \begin{array}{l} result \leftarrow -num\cdot A_s\cdot\left(\varepsilon_c\cdot E_s\right) \quad \text{if } c \le \frac{\%\cdot l_w}{2} \\ result \leftarrow num\cdot A_s\cdot\left[\frac{\varepsilon_c}{c}\cdot\left(c - \frac{\%\cdot l_w}{2}\right)\cdot E_s\right] \quad \text{otherwise} \\ result \end{array} \right.$$

(Column T)

Total Nominal Axial Capacity considering only the uniform vertical wall reinforcement

$$P_u(c, l_w, t_w, \rho_v) := T1(c, l_w, t_w, \rho_v) + T2(c, l_w, t_w, \rho_v) + C_s(c, l_w, t_w, \rho_v) + C_c(c, t_w)$$

(Column H)

Total Nominal In-Plane Bending Moment Capacity considering only the uniform vertical wall reinforcement

$$M_u(c, l_w, t_w, \rho_v) := T1(c, l_w, t_w, \rho_v)\cdot X1(c, l_w) + T2(c, l_w, t_w, \rho_v)\cdot X2(c, l_w) \ldots \\ + C_s(c, l_w, t_w, \rho_v)\cdot X3(c, l_w) + C_c(c, t_w)\cdot X4(c, l_w)$$

(Column P)

Nominal Axial Capacity of the reinforcement lumped at the wall ends

(Column X)

$$P_{extra}(c, l_w, t_w, num, A_s, \%) := T1extra(c, l_w, num, A_s, \%) + T2extra(c, l_w, num, A_s, \%) + Csextra(c, l_w, num, A_s, \%)$$

Nominal In-Plane Bending Capacity of the reinforcement lumped at the wall ends

(Column Y)

$$M_{extra}(c, l_w, t_w, num, A_s, \%) := T1extra(c, l_w, num, A_s, \%)\cdot X1extra(c, l_w, \%) + T2extra(c, l_w, num, A_s, \%)\cdot X2extra(c, l_w, \%) \ldots \\ + Csextra(c, l_w, num, A_s, \%)\cdot X3extra(c, l_w, \%)$$

Total Nominal Axial Capacity

$$P_T(c, l_w, t_w, \rho_v, num, A_s, \%) := P_u(c, l_w, t_w, \rho_v) + P_{extra}(c, l_w, t_w, num, A_s, \%)$$

Total Nominal In-Plane Bending Moment Capacity

$$M_T(c, l_w, t_w, \rho_v, num, A_s, \%) := M_u(c, l_w, t_w, \rho_v) + M_{extra}(c, l_w, t_w, num, A_s, \%)$$

Design Capacities

ACI 349-01 Section 9.3.2.2(b) (Ref. 2.2.2) states that the ϕ shall be 0.7 for axial compression. ϕ shall be permitted to be increased linearly to 0.90 as ϕPn decreases from the lesser of 0.10f'cAg or ϕPb to zero, where ϕPb is the axial capacity at the balanced load condition (c = lw/2 for the walls with uniform vertical reinforcement and ec = ey).

$$\phi(c, l_w, t_w, \rho_v, num, A_s, \%) := \left| \begin{array}{l} \phi_t \leftarrow 0.90 \\ \phi_c \leftarrow 0.70 \\ c_{balance} \leftarrow \dfrac{l_w}{2} \\ \phi P_b \leftarrow \phi_c \cdot P_T(c_{balance}, l_w, t_w, \rho_v, num, A_s, \%) \\ A_g \leftarrow t_w \cdot l_w \\ \phi P_{min} \leftarrow \min(\phi P_b, 0.10 \cdot A_g \cdot f_c) \\ \phi P_n \leftarrow \phi_c \cdot P_T(c, l_w, t_w, \rho_v, num, A_s, \%) \\ \phi \leftarrow \phi_c \ \text{ if } \ \phi P_n \ge \phi P_{min} \\ \phi \leftarrow \phi_t \ \text{ if } \ \phi P_n \le 0kip \\ \text{if } \ 0kip < \phi P_n < \phi P_{min} \\ \quad \left| \begin{array}{l} x \leftarrow \begin{pmatrix} 0kip \\ \phi P_{min} \end{pmatrix} \\ y \leftarrow \begin{pmatrix} \phi_t \\ \phi_c \end{pmatrix} \\ \phi \leftarrow \text{linterp}(x, y, \phi P_n) \end{array} \right. \\ \phi \end{array} \right.$$

(Column AG)

Axial Design Capacity (Column AD)

$$\phi P_T(c, l_w, t_w, \rho_v, num, A_s, \%) := \left| \begin{array}{l} A_g \leftarrow t_w \cdot l_w \\ \phi P_{max} \leftarrow 0.70 \cdot \left[0.85 \cdot f_c \cdot (A_g - \rho_v \cdot A_g - 2 \cdot num \cdot A_s) + E_s \cdot \varepsilon_c \cdot (\rho_v \cdot A_g + 2 \cdot num \cdot A_s)\right] \\ \phi P_n \leftarrow \phi(c, l_w, t_w, \rho_v, num, A_s, \%) \cdot P_T(c, l_w, t_w, \rho_v, num, A_s, \%) \\ result \leftarrow 0.80 \cdot \phi P_{max} \ \text{ if } \ \phi P_n \ge 0.80 \cdot \phi P_{max} \\ result \leftarrow \phi P_n \ \text{ if } \ \phi P_n < 0.80 \cdot \phi P_{max} \\ result \end{array} \right.$$

In-Plane Bending Design Capacity

$$\phi M_T(c, l_w, t_w, \rho_v, num, A_s, \%) := \phi(c, l_w, t_w, \rho_v, num, A_s, \%) \cdot M_T(c, l_w, t_w, \rho_v, num, A_s, \%)$$ (Column AE)

Test of Interaction Diagram Equations - Generate Interaction Diagram for Wall 5A.2

Case 1 - Only vertical wall reinforcement
For Case 1, the Excel file "Sample - Wall 5A.2.xls" in Attachment F is used.

Wall 5A.2 design parameters $num := 0$ $A_s := 0in^2$ No vertical steel lumped at each end of the wall for Case 1

$t_w := 4ft$ $\rho_v := \frac{2 \cdot 1.56 \cdot in^2}{t_w \cdot 1 \cdot ft}$ $\rho_v = 0.005417$ Stick 5A.2 contains #11 @ 12" c.c EF vertically

$l_w := 23.5ft$

$$c/l_w := \begin{pmatrix} 0.001 \\ 0.005 \\ 0.01 \\ 0.025 \\ 0.03 \\ 0.035 \\ 0.04 \\ 0.045 \\ 0.05 \\ 0.055 \\ 0.06 \\ 0.065 \\ 0.07 \\ 0.075 \\ 0.10 \\ 0.15 \\ 0.20 \\ 0.25 \\ 0.30 \\ 0.35 \\ 0.40 \\ 0.45 \\ 0.50 \\ 0.52 \\ 0.55 \\ 0.60 \\ 0.65 \\ 0.70 \\ 0.75 \\ 0.80 \\ 0.85 \\ 0.90 \\ 0.95 \\ 1.0 \\ 1.02 \\ 1.05 \\ 1.1 \\ 1.15 \\ 1.2 \\ 1.25 \end{pmatrix}$$

c/lw values considered
in the interaction generation.

These values are given in column A
of sheet "Shear Wall Design Template" in the
excel files contained in Attachment F.

For the verification of the Excel formulas, all
references to Excel columns are to the file
"Sample - Wall 5A.2.xls" in Attachment F.

$$c := c/l_w \cdot l_w \qquad c = \begin{pmatrix} 0.024 \\ 0.118 \\ 0.235 \\ 0.588 \\ 0.705 \\ 0.822 \\ 0.940 \\ 1.057 \\ 1.175 \\ 1.292 \\ 1.410 \\ 1.528 \\ 1.645 \\ 1.763 \\ 2.350 \\ 3.525 \\ 4.700 \\ 5.875 \\ 7.050 \\ 8.225 \\ 9.400 \\ 10.575 \\ 11.750 \\ 12.220 \\ 12.925 \\ 14.100 \\ 15.275 \\ 16.450 \\ 17.625 \\ 18.800 \\ 19.975 \\ 21.150 \\ 22.325 \\ 23.500 \\ 23.970 \\ 24.675 \\ 25.850 \\ 27.025 \\ 28.200 \\ 29.375 \end{pmatrix} \text{ft}$$

Identical
values
given in
Column B

$$\overrightarrow{T1(c, l_w, t_w, \rho_v)} = \begin{pmatrix} -4244 \\ -4210 \\ -4168 \\ -4040 \\ -3997 \\ -3955 \\ -3912 \\ -3870 \\ -3827 \\ -3785 \\ -3742 \\ -3700 \\ -3657 \\ -3615 \\ -3402 \\ -2977 \\ -2552 \\ -2126 \\ -1701 \\ -1276 \\ -851 \\ -425 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \text{kip}$$

Identical values given in Column C

$$\overrightarrow{T2(c, l_w, t_w, \rho_v)} = \begin{pmatrix} -2 \\ -11 \\ -21 \\ -53 \\ -64 \\ -74 \\ -85 \\ -96 \\ -106 \\ -117 \\ -128 \\ -138 \\ -149 \\ -159 \\ -213 \\ -319 \\ -425 \\ -532 \\ -638 \\ -744 \\ -851 \\ -957 \\ -1063 \\ -942 \\ -783 \\ -567 \\ -401 \\ -273 \\ -177 \\ -106 \\ -56 \\ -24 \\ -6 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \text{kip}$$

Identical values given in Column D

$$\overrightarrow{C_s(c,l_w,t_w,\rho_v)} = \begin{pmatrix} 2 \\ 11 \\ 21 \\ 53 \\ 64 \\ 74 \\ 85 \\ 96 \\ 106 \\ 117 \\ 128 \\ 138 \\ 149 \\ 159 \\ 213 \\ 319 \\ 425 \\ 532 \\ 638 \\ 744 \\ 851 \\ 957 \\ 1063 \\ 1106 \\ 1169 \\ 1276 \\ 1382 \\ 1488 \\ 1595 \\ 1701 \\ 1807 \\ 1914 \\ 2020 \\ 2126 \\ 2168 \\ 2228 \\ 2320 \\ 2404 \\ 2481 \\ 2552 \end{pmatrix} \text{kip}$$

Identical values give in Column E

$$\overrightarrow{C_c(c,t_w)} = \begin{pmatrix} 49 \\ 245 \\ 490 \\ 1226 \\ 1471 \\ 1716 \\ 1962 \\ 2207 \\ 2452 \\ 2697 \\ 2943 \\ 3188 \\ 3433 \\ 3678 \\ 4904 \\ 7356 \\ 9809 \\ 12261 \\ 14713 \\ 17165 \\ 19617 \\ 22069 \\ 24521 \\ 25502 \\ 26973 \\ 29426 \\ 31878 \\ 34330 \\ 36782 \\ 39234 \\ 41686 \\ 44138 \\ 46590 \\ 49043 \\ 50023 \\ 51495 \\ 53947 \\ 56399 \\ 58851 \\ 61303 \end{pmatrix} \text{kip}$$

Identical values give in Column F

$$\overrightarrow{P_u(c, l_w, t_w, \rho_v)} = \begin{pmatrix} -4.20 \\ -3.96 \\ -3.68 \\ -2.81 \\ -2.53 \\ -2.24 \\ -1.95 \\ -1.66 \\ -1.38 \\ -1.09 \\ -0.80 \\ -0.51 \\ -0.22 \\ 0.06 \\ 1.50 \\ 4.38 \\ 7.26 \\ 10.13 \\ 13.01 \\ 15.89 \\ 18.77 \\ 21.64 \\ 24.52 \\ 25.67 \\ 27.36 \\ 30.13 \\ 32.86 \\ 35.54 \\ 38.20 \\ 40.83 \\ 43.44 \\ 46.03 \\ 48.60 \\ 51.17 \\ 52.19 \\ 53.72 \\ 56.27 \\ 58.80 \\ 61.33 \\ 63.85 \end{pmatrix} 10^3 \cdot \text{kip}$$

Identical values give in Column H

$$\overrightarrow{X1(c, l_w)} = \begin{pmatrix} -0.02 \\ -0.12 \\ -0.24 \\ -0.59 \\ -0.71 \\ -0.82 \\ -0.94 \\ -1.06 \\ -1.18 \\ -1.29 \\ -1.41 \\ -1.53 \\ -1.65 \\ -1.76 \\ -2.35 \\ -3.53 \\ -4.70 \\ -5.88 \\ -7.05 \\ -8.22 \\ -9.40 \\ -10.58 \\ -11.75 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \end{pmatrix} \text{ft}$$

Identical values give in Column J

$$\overrightarrow{X2(c, l_w)} = \begin{pmatrix} 11.71 \\ 11.55 \\ 11.36 \\ 10.77 \\ 10.58 \\ 10.38 \\ 10.18 \\ 9.99 \\ 9.79 \\ 9.60 \\ 9.40 \\ 9.20 \\ 9.01 \\ 8.81 \\ 7.83 \\ 5.88 \\ 3.92 \\ 1.96 \\ -0.00 \\ -1.96 \\ -3.92 \\ -5.88 \\ -7.83 \\ -7.99 \\ -8.23 \\ -8.62 \\ -9.01 \\ -9.40 \\ -9.79 \\ -10.18 \\ -10.58 \\ -10.97 \\ -11.36 \\ -11.75 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \end{pmatrix} \text{ft}$$

Identical values given in Column K

$$\overrightarrow{X3(c, l_w)} = \begin{pmatrix} 11.74 \\ 11.71 \\ 11.67 \\ 11.55 \\ 11.52 \\ 11.48 \\ 11.44 \\ 11.40 \\ 11.36 \\ 11.32 \\ 11.28 \\ 11.24 \\ 11.20 \\ 11.16 \\ 10.97 \\ 10.58 \\ 10.18 \\ 9.79 \\ 9.40 \\ 9.01 \\ 8.62 \\ 8.23 \\ 7.83 \\ 7.68 \\ 7.44 \\ 7.05 \\ 6.66 \\ 6.27 \\ 5.88 \\ 5.48 \\ 5.09 \\ 4.70 \\ 4.31 \\ 3.92 \\ 3.77 \\ 3.56 \\ 3.26 \\ 3.01 \\ 2.80 \\ 2.61 \end{pmatrix} \text{ft}$$

Nearly identical values given in Column L.

For the values where c > lw, the values are not exactly identical because a value of 0.3333 is used in the Excel equation given in Column 0 while a value of 1/3 is used in the X3 definition in this Mathcad sheet.

$$\overrightarrow{X4(c, l_w)} = \begin{pmatrix} 11.74 \\ 11.70 \\ 11.66 \\ 11.52 \\ 11.48 \\ 11.43 \\ 11.39 \\ 11.34 \\ 11.29 \\ 11.25 \\ 11.20 \\ 11.16 \\ 11.11 \\ 11.07 \\ 10.84 \\ 10.38 \\ 9.93 \\ 9.47 \\ 9.02 \\ 8.56 \\ 8.11 \\ 7.65 \\ 7.20 \\ 7.01 \\ 6.74 \\ 6.29 \\ 5.83 \\ 5.38 \\ 4.92 \\ 4.46 \\ 4.01 \\ 3.55 \\ 3.10 \\ 2.64 \\ 2.46 \\ 2.19 \\ 1.73 \\ 1.28 \\ 0.82 \\ 0.37 \end{pmatrix} \mathrm{ft}$$

Identical values given in Column M

$$\overrightarrow{M_u(c, l_w, t_w, \rho_v)} = \begin{pmatrix} 0.68 \\ 3.37 \\ 6.70 \\ 16.54 \\ 19.76 \\ 22.96 \\ 26.12 \\ 29.25 \\ 32.36 \\ 35.44 \\ 38.48 \\ 41.50 \\ 44.49 \\ 47.45 \\ 61.82 \\ 88.38 \\ 112.04 \\ 132.81 \\ 150.67 \\ 165.63 \\ 177.70 \\ 186.87 \\ 193.13 \\ 194.91 \\ 196.98 \\ 198.86 \\ 198.69 \\ 196.44 \\ 192.08 \\ 185.59 \\ 176.95 \\ 166.14 \\ 153.15 \\ 137.98 \\ 131.30 \\ 120.62 \\ 101.07 \\ 79.31 \\ 55.35 \\ 29.17 \end{pmatrix} 10^3 \cdot \mathrm{kip} \cdot \mathrm{ft}$$

Identical values given in Column P

$$\overrightarrow{\phi\left(c, l_w, t_w, \rho_v, 0, 0in^2, 0.05\right)} = \begin{pmatrix} 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.87 \\ 0.82 \\ 0.76 \\ 0.71 \\ 0.70 \end{pmatrix}$$

Identical values given in Column AG

$$\overrightarrow{\phi P_T\left(c, l_w, t_w, \rho_v, 0, 0in^2, 0.05\right)} = \begin{pmatrix} -3.78 \\ -3.57 \\ -3.31 \\ -2.53 \\ -2.27 \\ -2.01 \\ -1.76 \\ -1.50 \\ -1.24 \\ -0.98 \\ -0.72 \\ -0.46 \\ -0.20 \\ 0.06 \\ 1.31 \\ 3.58 \\ 5.54 \\ 7.19 \\ 9.11 \\ 11.12 \\ 13.14 \\ 15.15 \\ 17.16 \\ 17.97 \\ 19.15 \\ 21.09 \\ 23.00 \\ 24.88 \\ 26.74 \\ 28.58 \\ 30.41 \\ 32.22 \\ 34.02 \\ 35.82 \\ 36.53 \\ 37.61 \\ 37.63 \\ 37.63 \\ 37.63 \\ 37.63 \end{pmatrix} 1000 \cdot kip$$

Identical values given in Column AD

$$\overrightarrow{\phi M_T\left(c, l_W, t_W, \rho_V, 0, 0in^2, 0.05\right)} = \begin{pmatrix} 0.61 \\ 3.03 \\ 6.03 \\ 14.89 \\ 17.79 \\ 20.66 \\ 23.51 \\ 26.33 \\ 29.12 \\ 31.89 \\ 34.64 \\ 37.35 \\ 40.04 \\ 42.65 \\ 53.89 \\ 72.26 \\ 85.55 \\ 94.22 \\ 105.47 \\ 115.94 \\ 124.39 \\ 130.81 \\ 135.19 \\ 136.43 \\ 137.89 \\ 139.20 \\ 139.08 \\ 137.51 \\ 134.46 \\ 129.91 \\ 123.86 \\ 116.30 \\ 107.21 \\ 96.59 \\ 91.91 \\ 84.44 \\ 70.75 \\ 55.52 \\ 38.74 \\ 20.42 \end{pmatrix} 1000 \cdot kip \cdot ft$$

Identical values
given in Column AE

$$P_{Case1} := \overrightarrow{\phi P_T(c, l_w, t_w, \rho_v, num, A_s, 0.05)}$$

$$M_{Case1} := \overrightarrow{\phi M_T(c, l_w, t_w, \rho_v, num, A_s, 0.05)}$$

Figure 6.5.2 - Interaction Diagram for Stick 5A.2 for Case 1

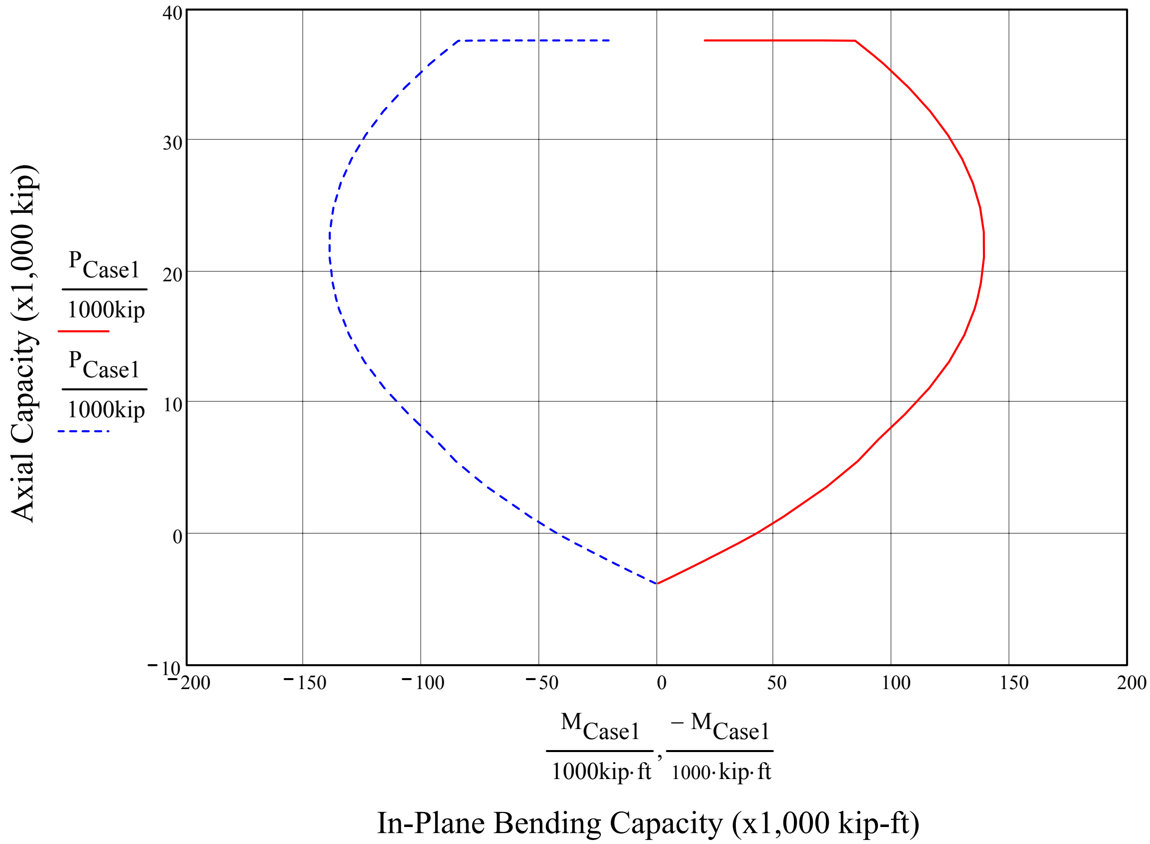
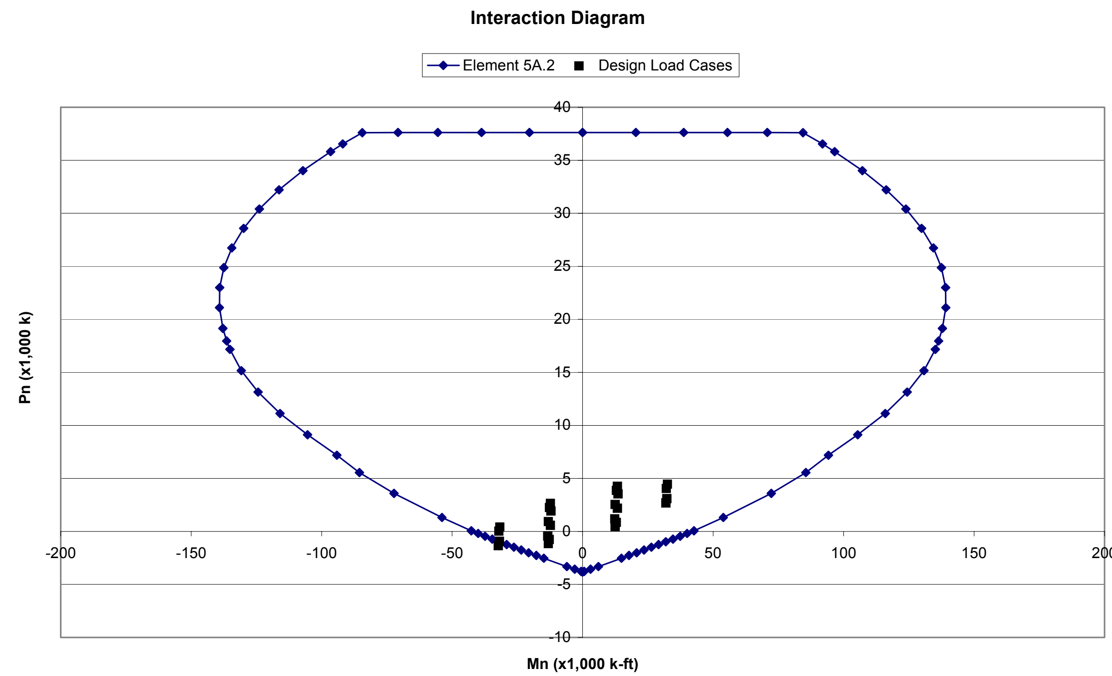


Figure 6.5.3 shows the interaction diagram for stick element 5A.2 calculated using the equations given on sheet "Shear Wall Design Template" in the file "Sample - Wall 5A.2.xls" included in Attachment F. A comparison of Figure 6.5.2 and Figure 6.5.3 shows that the formulas used in the Excel sheet "Shear Wall Design Template" included in Attachment F are correct.

Figure 6.5.3 - Interaction Diagram for Stick 5A.2 for Case 1 in file "Sample - Wall 5A.2.xls"



Case 2 - Vertical wall reinforcement and lumped vertical steel at each end of the wall
For Case 2, the Excel file "Sample - Wall 5A.2 - Add Bars.xls" in Attachment F is used.

Wall 5A.2 design parameters

$t_w := 4ft$

$l_w := 23.5ft$

$\rho_v := \dfrac{2 \cdot 1.56 \cdot in^2}{t_w \cdot 1 \cdot ft}$ $\qquad \rho_v = 0.005417$ $\qquad$ Stick 5A.2 contains #11 @ 12" c.c EF vertically

$num := 4 \quad A_s := 1.56in^2$ $\qquad$ 4 - #11 bars lumped at each end of the wall for Case 2

$\% := 0.05$ $\qquad$ Consider the lumped steel at each end of the wall is contained within a distance equal to 5% of the wall length

The values for X1, X2, X3, X4, T1, T2, Cs, Cc are not a function of the vertical steel lumped at each end of the wall. Therefore, these values are the same as those shown in Case 1. For Case 2, a value of 4 is input into Cell "K8", a value of 11 (#11 bars are lumped at each end of the wall) is input into Cell "K10", and a value of 5% is input into Cell "K11" (all cell references are to those cells in sheet "Interaction Diagram").

$$\overrightarrow{T1extra\left(c, l_w, num, A_s, \%\right)} = \begin{pmatrix} -361.92 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \end{pmatrix} \text{kip}$$

Identical values give in column R of sheet "Shear Wall Design Template".

$$\overrightarrow{T2extra\left(c, l_w, num, A_s, \%\right)} = \begin{pmatrix} 0.00 \\ -343.82 \\ -316.68 \\ -279.67 \\ -226.20 \\ -180.96 \\ -142.18 \\ -108.58 \\ -79.17 \\ -53.22 \\ -30.16 \\ -9.52 \\ 0.00 \\ 7.10 \\ 17.23 \\ 32.90 \\ 47.21 \\ 60.32 \\ 72.38 \end{pmatrix} \text{kip}$$

Identical values give in column S of sheet "Shear Wall Design Template"

$$\overrightarrow{\mathrm{Csextra}\left(c, l_w, num, A_s, \%\right)} = \begin{pmatrix} -361.92 \\ -361.92 \\ -361.92 \\ -361.92 \\ 60.32 \\ 103.41 \\ 135.72 \\ 160.85 \\ 180.96 \\ 197.41 \\ 211.12 \\ 222.72 \\ 232.66 \\ 241.28 \\ 271.44 \\ 301.60 \\ 316.68 \\ 325.73 \\ 331.76 \\ 336.07 \\ 339.30 \\ 341.81 \\ 343.82 \\ 344.52 \\ 345.47 \\ 346.84 \\ 348.00 \\ 348.99 \\ 349.86 \\ 350.61 \\ 351.28 \\ 351.87 \\ 352.40 \\ 352.87 \\ 353.05 \\ 353.30 \\ 353.69 \\ 354.05 \\ 354.38 \\ 354.68 \end{pmatrix} \mathrm{kip}$$

Identical values give in column T of sheet "Shear Wall Design Template"

$$\overrightarrow{X1extra(c, l_w, \%)} = \begin{pmatrix} -11.16 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \\ 0.00 \end{pmatrix} \text{ft}$$

Identical values give in column U of sheet "Shear Wall Design Template"

$$\overrightarrow{X2extra(c, l_w, \%)} = \begin{pmatrix} 0.00 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \\ -11.16 \end{pmatrix} \text{ft}$$

Identical values give in column V of sheet "Shear Wall Design Template"

$$\overrightarrow{X3extra\left(c,l_w,\%\right)} = \begin{pmatrix} 11.16 \\ 11.16 \end{pmatrix} \text{ft}$$

Identical values give in column W of sheet "Shear Wall Design Template"

$$\overrightarrow{\phi\left(c,l_w,t_w,\rho_v,num,A_s,\%\right)} = \begin{pmatrix} 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.90 \\ 0.87 \\ 0.82 \\ 0.76 \\ 0.71 \\ 0.70 \end{pmatrix}$$

Identical values give in Column AG

$$\overrightarrow{\phi P_T(c, l_w, t_w, \rho_v, \mathrm{num}, A_s, \%)} = \begin{pmatrix} -4.43 \\ -4.22 \\ -3.96 \\ -3.18 \\ -2.54 \\ -2.25 \\ -1.96 \\ -1.68 \\ -1.40 \\ -1.13 \\ -0.86 \\ -0.59 \\ -0.32 \\ -0.05 \\ 1.23 \\ 3.54 \\ 5.51 \\ 7.17 \\ 9.09 \\ 11.10 \\ 13.12 \\ 15.14 \\ 17.16 \\ 17.99 \\ 19.20 \\ 21.18 \\ 23.12 \\ 25.03 \\ 26.91 \\ 28.77 \\ 30.61 \\ 32.45 \\ 34.26 \\ 36.07 \\ 36.79 \\ 37.86 \\ 38.00 \\ 38.00 \\ 38.00 \\ 38.00 \end{pmatrix} 1000 \cdot \mathrm{kip}$$

Identical values give in Column AD

$$\overrightarrow{\phi M_T(c, l_w, t_w, \rho_v, \mathrm{num}, A_s, \%)} = \begin{pmatrix} 0.61 \\ 3.03 \\ 6.03 \\ 14.89 \\ 22.03 \\ 25.34 \\ 28.51 \\ 31.58 \\ 34.58 \\ 37.51 \\ 40.39 \\ 43.23 \\ 46.02 \\ 48.77 \\ 60.17 \\ 78.43 \\ 91.43 \\ 99.76 \\ 110.89 \\ 121.40 \\ 129.87 \\ 136.30 \\ 140.57 \\ 141.60 \\ 142.77 \\ 143.68 \\ 143.22 \\ 141.35 \\ 138.04 \\ 133.27 \\ 127.02 \\ 119.28 \\ 110.04 \\ 99.35 \\ 94.62 \\ 87.06 \\ 73.25 \\ 57.91 \\ 41.04 \\ 22.63 \end{pmatrix} 1000 \cdot \mathrm{kip} \cdot \mathrm{ft}$$

Identical values give in Column AE

$$P_{Case2} := \overrightarrow{\phi P_T(c, l_w, t_w, \rho_v, num, A_s, \%)}$$

$$M_{Case2} := \overrightarrow{\phi M_T(c, l_w, t_w, \rho_v, num, A_s, \%)}$$

Figure 6.5.4 - Interaction Diagram for Stick 5A.2 for Case 1 and Case 2

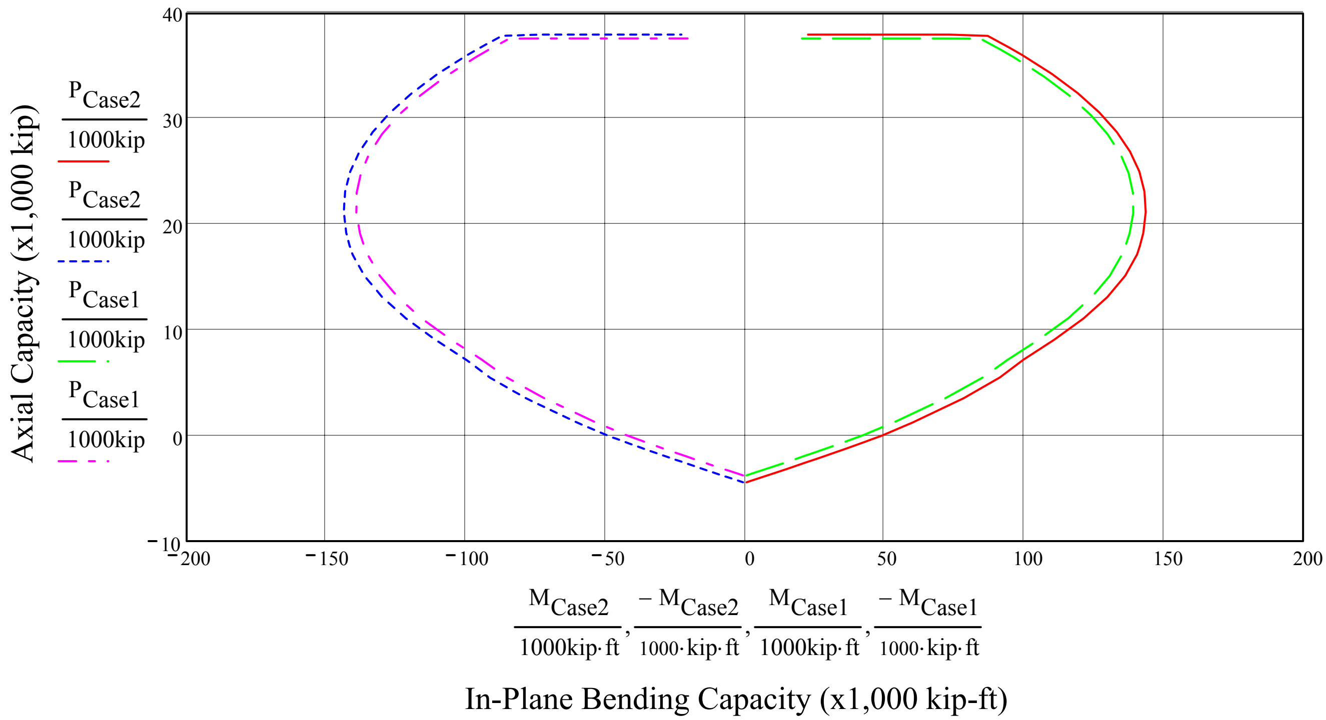
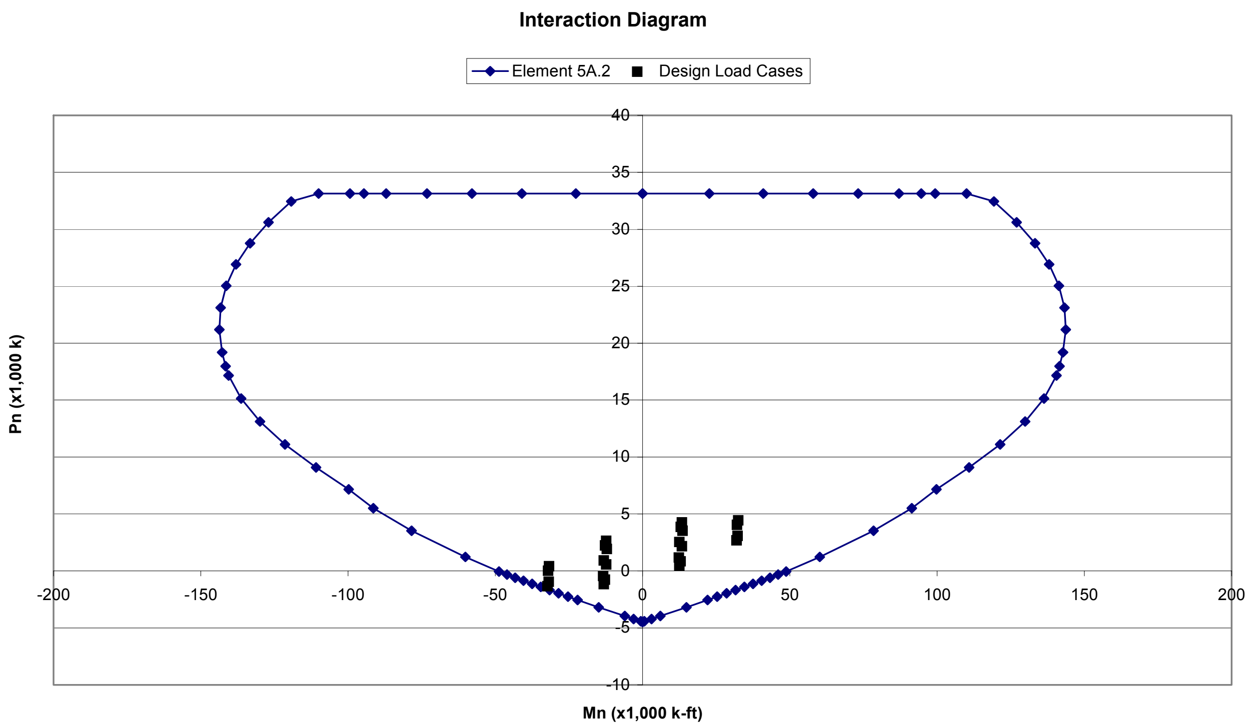


Figure 6.5.4 shows the interaction diagram for stick element 5A.2 calculated using the equations given on sheet "Shear Wall Design Template" in the file "Sample - Wall 5A.2 - Add Bars.xls" included in Attachment F with the addition of 4-#11 bars lumped within 5% of the wall length. A comparison of Figure 6.5.4 and Figure 6.5.5 shows that the formulas used in the Excel sheets included in Attachment F are correct.

Figure 6.5.5 - Interaction Diagram for Stick 5A.2 Case 2 in file "Sample - Wall 5A.2 - Add Bars.xls



Step 4 - Plot the PT and MT values on the Interaction Diagram for individual stick elements

Subfolders included in the Attachment F show the the P_T and M_T points plotted on the Interaction Diagram for each stick element. Each wall of the CRCF has a subfloder in Attachment F that shows the interaction diagram for each of the individual stick element comprising the wall. A review of these plots shows that 20 walls had points that laid on or outside of the interaction diagram. These walls are listed below:

1) Wall 2A.2
2) Wall 2A.4
3) Wall 4A.1
4) Wall 4A.3
5) Wall 5A.1
6) Wall 5A.2
7) Wall 5A.3
8) Wall 5A.4
9) Wall 6A.2
10) Wall 6A.3
11) Wall 7A.1
12) Wall 7A.2
13) Wall 8A.2
14) Wall 8A.3
15) Wall 9A.1
16) Wall 9A.4
17) Wall D1.2
18) Wall H1.4
19) Wall 12A.2
20) Wall 12A.3

As expected, the above walls are all located at EL. 0'-0" near openings.

For the above listed walls, the capacity of additional vertical reinforcing bars are added to the interaction diagram until all points lie within the interaction diagram. For this evaluation, a value of 5% of the total wall length is considered as the length of wall where the additional lumped steel is added. Cell "K8" on sheet "Interaction Diagram" located in the applicable Excel file contained in Attachment F contains the number of additional bars required at each end of the wall segment in order for all points to lie inside the interaction diagram. The files that contain the "modified" interaction diagram are titled "Additional - Wall XXX - CRCF - Fragility - In-Plane Bending and Axial Force.xls", where XXX is the wall ID.

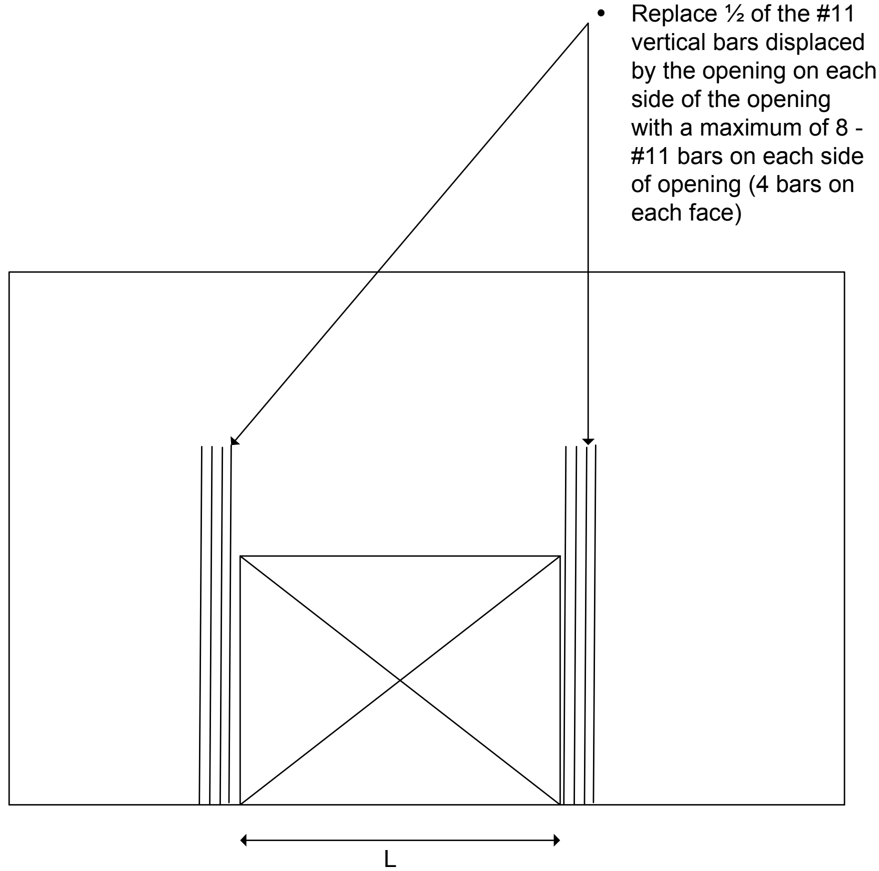
A review of these "modified" interaction diagram plots included in Attachment F shows between 2 and 8 additional vertical reinforcing bars are required for the walls to have all P_T and M_T points inside the interaction diagram. The list below shows the number of #11 bars required for each wall segment in order for the P_T and M_T points to lie within the interaction diagram.

Wall ID	Additional #11 Vertical Reinforcing Bars
1) Wall 2A.2	2
2) Wall 2A.4	2
3) Wall 4A.1	4
4) Wall 4A.3	4
5) Wall 5A.1	4
6) Wall 5A.2	4
7) Wall 5A.3	4
8) Wall 5A.4	4
9) Wall 6A.2	8
10) Wall 6A.3	8
11) Wall 7A.1	2
12) Wall 7A.2	2
13) Wall 8A.2	6
14) Wall 8A.3	6
15) Wall 9A.1	4
16) Wall 9A.4	4
17) Wall D1.2	3
18) Wall H1.4	6
19) Wall 12A.2	4
20) Wall 12A.3	4

The largest number of #11 vertical bars required (8) is for walls "6A.2" and "6A.3". As shown on the drawings in Attachment A, these wall segments are piers located between two large openings. A review of the other walls listed above shows that they are also located near openings. Typical reinforcing detailing practice around openings is to replace 1/2 of the vertical bars that were displaced by the opening at the end of the wall on both sides of the opening. Following this practice for the 20'-0" openings on the south end of wall "6A.2" and on the north end of wall "6A.3" would lead to 20 additional vertical reinforcing bars lumped at the wall ends (2-#11 bars / foot x (1/2 x 20 feet) = 20 - #11 bars), which is more than enough to satisfy the vertical reinforcing bar requirements shown above. The additional vertical reinforcing requirements for the other walls listed above would also be satisfied if this opening detailing practice is followed.

Therefore, for the openings at EL. 0'-0", Figure 6.5.6 shows a detail required to meet the HCLPF capacity requirements for in-plane bending and axial force on the walls of the CRCF.

Figure 6.5.6 - Typical Reinforcing Detail Around Wall Openings at EL. 0-0" Required for HCLPF Evaluation



6.5.1 HCLPF Capacity Evaluations for Axial Force in Combination with In-Plane Bending of Walls - Summary

The results from the HCLPF capacity evaluations for axial force in combination with in-plane bending of the CRCF shear wall are as follows -

- All CRCF shear walls above elevation 0'-0" can carry the in-plane bending moment plus in-plane axial force corresponding to the HCLPF capacity of the CRCF shear walls without additional vertical wall reinforcing.
- Figure 6.5.6 shows the reinforcing detailing requirements around the openings at EL. 0'-0" in order for the shear walls at this elevation to carry the in-plane bending moment plus in-plane axial force corresponding to the HCLPF capacity of the CRCF shear walls.

6.6 HCLPF CAPACITY EVALUATIONS FOR OUT-OF-PLANE FAILURE MECHANISMS OF WALLS

General Information and Data

$\text{ORIGIN} := 1$	Set the array origin to 1
$f_c := 5500\text{psi}$	Concrete compressive strength per section 6.2.4.1 of this calculation
$f_y := 60000\text{psi}$	Steel yield strength (Ref. 2.2.1, Section 4.2.11.6.2)
$PGA_h := 0.9138g$	BDBGM peak horizontal ground acceleration (Ref. 2.2.31)
$\text{cover} := 2.0\text{in}$	Minimum clear cover for concrete walls exposed to earth with #6 to #11 bars per Section 7.7.1 of Ref. 2.2.2
$\rho := 150\text{pcf}$	Concrete density (Ref. 2.2.1, Section 4.2.11.6.6)

Table A-1 of Ref. 2.2.36

$$d_b(\text{num}) := \begin{array}{|l} 0.375\text{in} \;\text{ if }\; \text{num} = 3 \\ 0.500\text{in} \;\text{ if }\; \text{num} = 4 \\ 0.625\text{in} \;\text{ if }\; \text{num} = 5 \\ 0.750\text{in} \;\text{ if }\; \text{num} = 6 \\ 0.875\text{in} \;\text{ if }\; \text{num} = 7 \\ 1.00\text{in} \;\text{ if }\; \text{num} = 8 \\ 1.128\text{in} \;\text{ if }\; \text{num} = 9 \\ 1.270\text{in} \;\text{ if }\; \text{num} = 10 \\ 1.410\text{in} \;\text{ if }\; \text{num} = 11 \\ 0.0\text{in} \;\text{ otherwise} \end{array} \qquad A_{sbar}(\text{num}) := \begin{array}{|l} 0.11\text{in}^2 \;\text{ if }\; \text{num} = 3 \\ 0.20\text{in}^2 \;\text{ if }\; \text{num} = 4 \\ 0.31\text{in}^2 \;\text{ if }\; \text{num} = 5 \\ 0.44\text{in}^2 \;\text{ if }\; \text{num} = 6 \\ 0.60\text{in}^2 \;\text{ if }\; \text{num} = 7 \\ 0.79\text{in}^2 \;\text{ if }\; \text{num} = 8 \\ 1.00\text{in}^2 \;\text{ if }\; \text{num} = 9 \\ 1.27\text{in}^2 \;\text{ if }\; \text{num} = 10 \\ 1.56\text{in}^2 \;\text{ if }\; \text{num} = 11 \\ 0.0\text{in}^2 \;\text{ otherwise} \end{array}$$

<u>Out-of-Plane Wall HCLPF Calculation Steps</u>
The HCLPF capacity evaluation for the out-of-plane (o-o-p) failure mechanism for the CRCF walls is performed as follows:

- Define vertical wall strips at each elevation that are representative of all walls (thickness, reinforcement, etc.)
- Considering a 1-ft wall strip width, determine the o-o-p moment and shear on the wall using the wall weight and a uniform lateral acceleration equal to the maximum lateral BDBGM acceleration at the floor elevation at the top of the wall strip multiplied by the appropriate amplification factor (see Assumption 3.1.1).
- Calculate the moment and shear capacity of the wall strip.
- Calculate the shear corresponding to the development of the plastic moment capacity of the wall strip and show that the wall can transmit this shear.
- Calculate the Fs, F_μ, and HCLPF for each wall strip. The minimum is the HCLPF capacity for out-of-plane bending of the CRCF walls.

This evaluation is conservative for the following reasons:
1. A uniform acceleration is applied to the entire wall strip height.

2. Only one face of vertical steel reinforcement is considered in the capacity.
3. Two-way action of the wall panel is ignored.
4. The entire wall panel is not excited at the amplified acceleration.

Note: This analysis does not consider any additional o-o-p demand on the wall strips due to the out-of-plane response of wall sections above adjacent wall openings. The wall strips adjacent to the wall openings will be reinforced with the vertical reinforcement that is removed by the opening. This reinforcement will provide enough capacity to prevent these wall strips from controlling the HCLPF capacity for o-o-p bending failure of the walls. Therefore, HCLPF capacities for the wall strips adjacent to wall openings are not explicitly calculated.

6.6.1 Wall Strip Cases

Note: reinforcement listed in the Case descriptions are the vertical wall reinforcement
All wall design parameters (thickness, reinforcement, etc.) is per the CRCF Shear Wall Design (Ref. 2.2.29). Also, all calculations are based on a 1-ft wide wall strip

$$\text{Cases} := \begin{pmatrix} \text{"Case 1: 2 ft wall - EL. 0' to EL. 32' - \#11 @ 12" o.c."} \\ \text{"Case 2: 4 ft wall - EL. 0' to EL. 32' - \#11 @ 12" o.c."} \\ \text{"Case 3: 4 ft wall - EL. 32' to EL. 64' - \#11 @ 12" o.c."} \\ \text{"Case 4: 4 ft wall - EL. 32' to EL. 72' - \#11 @ 12" o.c."} \\ \text{"Case 5: 4 ft wall - EL. 64' to EL. 100' - \#11 @ 12" o.c."} \\ \text{"Case 6: 4 ft wall - EL. 32' to EL. 100' - \#11 @ 9" o.c."} \end{pmatrix}$$

These cases cover all combinations of wall spans and reinforcements found in the CRCF.

$$\text{amplify}_{oop} := \begin{pmatrix} 1.5 \\ 1.5 \\ 1.5 \\ 1.5 \\ 1.5 \\ 1.5 \end{pmatrix}$$

Out-of-plane seismic amplification factors (Assumption 3.1.1)

$$\text{wall}_b := 11$$

All wall reinforcing steel is #11 bars

$$\text{thick} := \begin{pmatrix} 2 \\ 4 \\ 4 \\ 4 \\ 4 \\ 4 \end{pmatrix} \cdot \text{ft} \qquad \text{height} := \begin{pmatrix} 32 \\ 32 \\ 32 \\ 40 \\ 36 \\ 68 \end{pmatrix} \cdot \text{ft} \qquad As_v := \begin{pmatrix} A_{sbar}(\text{wall}_b) \\ A_{sbar}(\text{wall}_b) \\ A_{sbar}(\text{wall}_b) \\ A_{sbar}(\text{wall}_b) \\ A_{sbar}(\text{wall}_b) \\ A_{sbar}(\text{wall}_b) \cdot \dfrac{12}{9} \end{pmatrix} \qquad As_v = \begin{pmatrix} 1.56 \\ 1.56 \\ 1.56 \\ 1.56 \\ 1.56 \\ 2.08 \end{pmatrix} \text{in}^2$$

Seismic Loads
The uniform o-o-p seismic load equals the wall thickness x 1-ft x density x seismic acceleration x amplification factor

Acceleration Factors for Seismic Loads
See Section 6.3.1.1 for the calculation of the maximum horizontal seismic accelerations given below.

$$Acc_h := \begin{pmatrix} 1.31 \\ 1.56 \\ 1.60 \\ 1.83 \end{pmatrix} \cdot g$$

EL. 32'
EL. 64'
EL. 72'
EL. 100'

$$SACC_h := \overrightarrow{\left[amplify_{oop} \cdot \begin{pmatrix} Acc_{h_1} \\ Acc_{h_1} \\ Acc_{h_2} \\ Acc_{h_3} \\ Acc_{h_4} \\ Acc_{h_4} \end{pmatrix} \right]} \qquad SACC_h = \begin{pmatrix} 1.97 \\ 1.97 \\ 2.34 \\ 2.40 \\ 2.75 \\ 2.75 \end{pmatrix} g$$

Amplified horizontal acceleration for each wall strip case.

$$w_{uBDBGM} := \overrightarrow{\left[(thick \cdot 1 \cdot ft \cdot \rho) \cdot \frac{SACC_h}{g} \right]} \qquad w_{uBDBGM} = \begin{pmatrix} 0.59 \\ 1.18 \\ 1.40 \\ 1.44 \\ 1.65 \\ 1.65 \end{pmatrix} \frac{kip}{ft}$$

Uniform amplified BDBGM loading applied to the wall strip.

6.6.2 Moment and Shear Demands for Slabs

Max. moment = $wL^2/8$

Max. shear force
For the out-of-plane shear evaluation, the maximum shear demand is computed as the shear corresponding to the plastic moment capacity of the beam. The shear is evaluated at d away from the face, where d is the effective wall depth.

$$M_{uBDBGM} := \overrightarrow{\frac{w_{uBDBGM} \cdot height^2}{8}} \qquad M_{uBDBGM} = \begin{pmatrix} 75.5 \\ 150.9 \\ 179.7 \\ 288.0 \\ 266.8 \\ 952.0 \end{pmatrix} kip \cdot ft$$

Max. moment due to amplified BDBGM uniform loading on wall strip

6.6.3 Design Capacities ($C_{98\%}$)

Effective depth: $d := thick - cover - 1.5 \cdot d_b(wall_b)$

Conservatively consider that the vertical bar is the inside bar (i.e. subtract 1.5 bar diameters when determining the effective depth.)

$$d^T = (19.89 \;\; 43.89 \;\; 43.89 \;\; 43.89 \;\; 43.89 \;\; 43.89)\, in$$

Compression block depth (per ft) $a := \dfrac{As_v \cdot f_y}{0.85 \cdot f_c \cdot 12in}$ $\quad a^T = (1.67 \;\; 1.67 \;\; 1.67 \;\; 1.67 \;\; 1.67 \;\; 2.22)\, in$

Moment Capacity = $\phi Mn = \phi C_{98\%}$

$\phi_b := 0.9$ Strength reduction factor for transverse bending per Ref. 2.2.2

$$\phi M_n := \overrightarrow{\left[\phi_b \cdot As_v \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right]} \qquad \phi M_n^T = (133.7 \;\; 302.2 \;\; 302.2 \;\; 302.2 \;\; 302.2 \;\; 400.4)\, kip \cdot ft$$

Shear Capacity = $\phi Vn = \phi C_{98\%}$

$\phi_s := 0.85$ Strength reduction factor for transverse shear per Ref. 2.2.2

$$\phi V_n := \phi_s \cdot 2 \cdot \sqrt{f_c \cdot psi} \cdot 12in \cdot d \qquad \phi V_n^T = (30.1 \;\; 66.4 \;\; 66.4 \;\; 66.4 \;\; 66.4 \;\; 66.4)\, kip$$

Plastic Moment Capacity

The plastic moment capacity is set equal to the nominal moment capacity (no strength reduction factor).

$$M_p := \frac{\phi M_n}{\phi_b} \qquad M_p^T = (148.6 \;\; 335.8 \;\; 335.8 \;\; 335.8 \;\; 335.8 \;\; 444.8)\, kip \cdot ft$$

Shear Corresponding to the Plastic Moment Capacity

- Calculate the uniform load required to cause the beam to reach its plastic moment capacity (wplastic).
- Then calculate the shear demand at d away from the face of the support due to wplastic per Ref. 2.2.2 Section 11.1.3

$$w_{plastic} := \frac{M_p \cdot 8}{height^2} \qquad w_{plastic}^T = (1.16 \;\; 2.62 \;\; 2.62 \;\; 1.68 \;\; 2.07 \;\; 0.77)\, \frac{kip}{ft}$$

$$V_{plastic} := \overrightarrow{\left[w_{plastic} \cdot \left(\frac{height}{2} - d\right)\right]}$$

Shear at "d" away from the face of the support when the beam strip reaches its plastic moment capacity.

$$V_{plastic}^{T} = \begin{pmatrix} 16.7 & 32.4 & 32.4 & 27.4 & 29.7 & 23.4 \end{pmatrix} kip$$

Check shear D/C corresponding to the plastic moment capacity

$$\frac{V_{plastic}}{\phi V_n} = \begin{pmatrix} 0.55 \\ 0.49 \\ 0.49 \\ 0.41 \\ 0.45 \\ 0.35 \end{pmatrix}$$

All shear D/C are less than 1.0. Therefore, the wall strip can carry the shear that occurs when the plastic moment capacity of the wall is reached.

6.6.4 Strength Margin Factor - Out-of-Plane Bending (Fs_{Mom})

Per Equation 4-2 of Section 4.3.2 of this calculation and neglecting o-o-p bending on the wall due to non-seismic loading, the strength margin factor for o-o-p bending of the wall strips is -

$$F_s := \overrightarrow{\frac{\phi M_n - 0kip}{M_{uBDBGM}}}$$

$$F_s^{T} = \begin{pmatrix} 1.77 & 2.00 & 1.68 & 1.05 & 1.13 & 0.42 \end{pmatrix}$$

Note: Non-seismic demand for out-of-plane bending is taken as 0.0 kip-ft

6.6.5 Inelastic Energy Absorption Factor - F_μ)

$$\frac{height}{thick} = \begin{pmatrix} 16.0 \\ 8.0 \\ 8.0 \\ 10.0 \\ 9.0 \\ 17.0 \end{pmatrix}$$

Per Ref. 2.2.6, Section 5.1.2.3, the $F\mu$ factors for out-of-plane behavior of walls shall be taken from the values for concrete moment frames. For Limit State A, for span to depth ratios greater than 15.0, $F\mu$ = 2.5 and for span to depth ratios less than 10, $F\mu$ = 2.25. For span to depth ratios between 10 and 15, interpolate between 2.25 and 2.5. Note: the $F\mu$ values are for slab/wall moment frames and not SMRF reinforce concrete moment frames.

Linear interpolation ranges for $F\mu$ determination when height-to-thickness is between 10 and 15

$$\text{yrange} := \begin{pmatrix} 2.25 \\ 2.5 \end{pmatrix} \qquad \text{xrange} := \begin{pmatrix} 10.0 \\ 15.0 \end{pmatrix} \qquad \text{linterp}(\text{xrange}, \text{yrange}, 12.5) = 2.375$$

For example, for a span-to-depth ratio of 12.5, Fμ = 2.375

$$F_{\mu} := \left| \begin{array}{l} \text{for } i \in 1 .. \text{rows(Cases)} \\ \quad \left| \begin{array}{l} \text{value}_i \leftarrow 2.25 \text{ if } \dfrac{\text{height}_i}{\text{thick}_i} \le 10.0 \\ \text{value}_i \leftarrow 2.5 \text{ if } \dfrac{\text{height}_i}{\text{thick}_i} \ge 15.0 \\ \text{value}_i \leftarrow \text{linterp}\left(\text{xrange}, \text{yrange}, \dfrac{\text{height}_i}{\text{thick}_i}\right) \text{ otherwise} \end{array} \right. \\ \text{value} \end{array} \right.$$

$$F_{\mu} = \begin{pmatrix} 2.50 \\ 2.25 \\ 2.25 \\ 2.25 \\ 2.25 \\ 2.50 \end{pmatrix}$$

Fμ for Limit State A per Table 5-1 of Ref. 2.2.6

6.6.6 HCLPF Capacity for Out-of-Plane Bending of Walls

Use the horizontal PGA for the out-of-plane bending HCLPF calculation

$$\text{HCLPF}_{\text{Mom}} := \overrightarrow{\left(F_s \cdot F_{\mu} \cdot \text{PGA}_h\right)} \qquad \text{HCLPF}_{\text{Mom}}^{\ T} = (4.05 \ \ 4.12 \ \ 3.46 \ \ 2.16 \ \ 2.33 \ \ 0.96)\, g$$

$$\text{HCLPF}_{\text{oop}} := \min\left(\text{HCLPF}_{\text{Mom}}\right)$$

$$\text{HCLPF}_{\text{oop}} = 0.96\, g$$

Minimum HCLPF for out-of-plane bending of the CRCF walls

Cases_6 = "Case 6:   4 ft wall - EL. 32' to EL. 100' - #11 @ 9" o.c."

The HCLPF capacity for Case 6 was calculated as 1.32 g, which is less than the HCLPF capacity of the CRCF shear walls. Case 6 is associated with the wall between EL. 32' and EL. 100' along column lines D and H between column lines 6 and 9. This wall panel is actually a two way slab. Since the HCLPF capacity using the conservative one way beam strip is less than the in-plane shear HCLPF capacity a more refined analysis of the wall panel considered for Case 6 is required.

6.6.7 HCLPF Capacity for Out-of-plane Bending of Wall Panel between EL. 32' and EL. 100'

ACI 349-01 Section 13.5.1 states that a slab system shall be designed by any procedure satisfying conditions of equilibrium and geometric compatibility. The commentary of Section 13.5.1 states that yield line analysis is an acceptable design method. Therefore, the HCLPF capacity of the entire panel between column lines 6 and 9 between EL. 32' and EL. 100' (~ 94 feet x 68 feet) will be evaluated considering the ultimate uniform loading of the panel as the $C_{98\%}$ capacity.

The following steps for the yield line evaluation are similar to those discussed in Section 15-3 of Ref. 2.2.36.

Steps involved in the yield line evaluation

1. Determine negative moment reinforcement (top bars) per foot along the wall panel boundaries
2. Determine positive moment reinforcement (bottom bars) per foot
3. Develop Yield Line Equations as followings

- Select a trial yield line pattern
- Give the panel a virtual displacement
- Compute the external work done by the uniform load (w) moving along the displaced shape defined in Step 4 ($W_{external} = w_r \times \delta_r$) δ_r = displacement of resultant load w_r
- Compute the internal work done by the yield lines rotating through the displaced shaped defined in Step 4 ($W_{internal} = m \times \theta \times L$) where L = length of yield line, θ = angle change of yield line

4. Equate the external and internal work, solving for the uniform load (w).
5. Repeat Steps 3 to 7 until the lowest uniform load is found.

Panel Dimensions

$L := 94\text{ft}$

$b := 68\text{ft}$

Step 1: Determine negative moment reinforcement (top bars) along the wall panel boundaries

The wall panel is bounded by the EL. 32' floor slab and the EL. 100' roof slab at the top and bottom (edges C-D and A-B in Figure 6.6.1 and Figure 6.6.2). For the yield line evaluation, the negative moment reinforcement along line A-B (edge 4) is controlled by the moment capacities of the 18" roof slab at EL. 100' (#7 @ 12") because the roof slab will develop its moment capacity before the 4-ft wall panel will develop its negative moment capacity along line A-B. The negative moment capacity along the other wall panel boundaries (lines A-C, C-D, and B-D) are controlled by the wall panel reinforcement because the strengths of the perpendicular walls along col. lines 6 and 9 and the slab at EL. 32' are comparable to that of the wall panel along these edges.

Wall Panel Parameter
t = 4 feet
Vertical Steel = # 11 @ 9"
Horizontal Steel = #11 @ 12"

EL. 100' Roof Slab Steel
t = 1.5 feet
#7 @12"

$t_{panel} := 4\text{ft}$

$\text{cover} = 2.0\,\text{in}$

$d_{panel} := t_{panel} - \text{cover} - 1.5\cdot d_b(11)$ $\quad d_{panel} = 43.9\,\text{in}$

$$a_{panel} := \frac{A_{sbar}(11)\cdot f_y}{0.85\cdot f_c\cdot 12\text{in}} \qquad a_{panel} = 1.7\,\text{in}$$

Edge 1 (A-C) Negative Moment Capacity

$$M_{AC} := 0.9 \cdot \left(A_{sbar}(11) \cdot f_y\right) \cdot \left(d_{panel} - \frac{a_{panel}}{2}\right) \qquad M_{AC} = 302.2\,\text{kip} \cdot \text{ft}$$

Edge 2 (B-D) Negative Moment Capacity (same reinforcement as Edge 1 (A-C))

$$M_{BD} := M_{AC} \qquad M_{BD} = 302.2\,\text{kip} \cdot \text{ft}$$

Edge 3 (C-D) Negative Moment Capacity

$$a_{CD} := a_{panel} \cdot \frac{12}{9} \qquad a_{CD} = 2.2\,\text{in}$$

a for #11 @ 9" = a for #11 @12" x 12/9

$$M_{CD} := 0.9 \cdot \left(\frac{12}{9} \cdot A_{sbar}(11) \cdot f_y\right) \cdot \left(d_{panel} - \frac{a_{CD}}{2}\right) \qquad M_{CD} = 400.4\,\text{kip} \cdot \text{ft}$$

Edge 4 (A-B) Negative Moment Capacity (negative moment capacity of roof slab @ EL. 100')

$$t_{slab} := 1.5\text{ft}$$

$$\text{cover} = 2.0\,\text{in}$$

$$d_{slab} := t_{slab} - \text{cover} - 1.5 \cdot d_b(7) \qquad d_{slab} = 14.7\,\text{in}$$

$$a_{slab} := \frac{A_{sbar}(7) \cdot f_y}{0.85 \cdot f_c \cdot 12\text{in}} \qquad a_{slab} = 0.6\,\text{in}$$

$$M_{AB} := 0.9 \cdot \left(A_{sbar}(7) \cdot f_y\right) \cdot \left(d_{slab} - \frac{a_{slab}}{2}\right) \qquad M_{AB} = 38.8\,\text{kip} \cdot \text{ft}$$

Step 2: Determine positive moment reinforcement (bottom bars)
The positive moment capacity at the center of the panel is provided by #11 @ 9" vertically and #11 @ 12" horizontally

Vertical Reinforcement #11 @ 9" $\qquad M_y := M_{CD} \qquad M_y = 400.4\,\text{kip} \cdot \text{ft}$

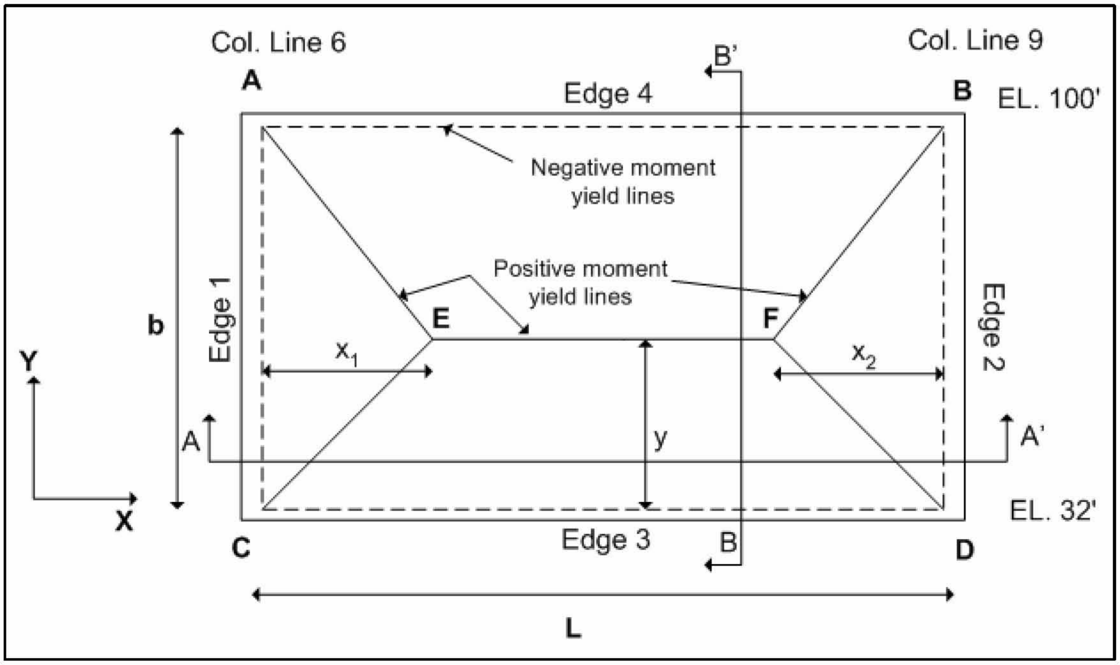
Horizontal Reinforcement #11 @ 12" $\qquad M_x := M_{AC} \qquad M_x = 302.2\,\text{kip} \cdot \text{ft}$

Step 3: Develop Yield Line Equations

Select a trial yield line pattern (Fig. 6.6.1)
Depending on the vertical and horizontal reinforcement, as well as the panel dimensions, either the yield line pattern in Figure 6.6.1 or Figure 6.6.4 will determine the ultimate uniform load capacity of the panel. First, the yield line pattern in Figure 6.6.1 will be considered and the uniform load capacity will be determined for a set of x1, x2, and y values.

Next, the yield line pattern in Figure 6.6.4 will be considered and the uniform load capacity will be determined for a set of y1, y2, and x values. The minimum uniform load capacity of the panel for both sets of yield line patterns will determine the ultimate uniform load capacity of the panel.

Figure 6.6.1 - Yield Line 1 Schematic



Give the panel a virtual displacement (Fig. 6.6.1)
Line E-F in Figure 6.6.1 is given a virtual displacement of δ

Figure 6.6.2 and Figure 6.6.3 show sections through Figure 6.6.1 to illustrate the displacement shape of the panel under the virtual displacement.

$\delta_p := 1 \cdot ft$

Figure 6.6.2 - Section A-A' of Yield Line 1

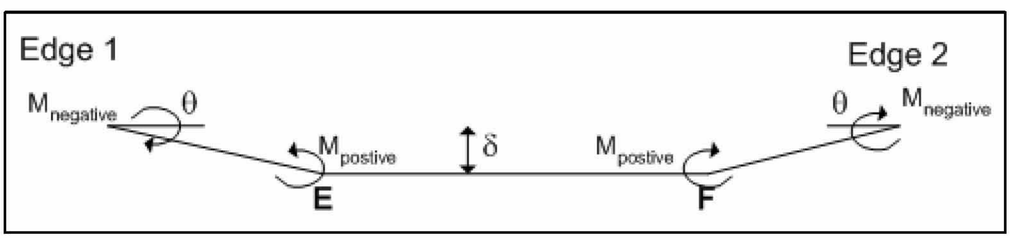
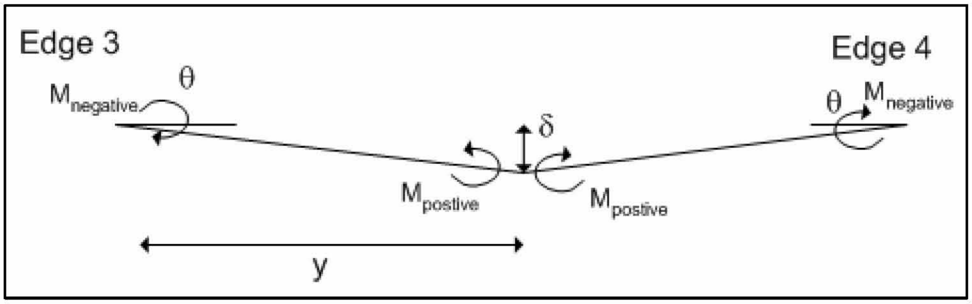


Figure 6.6.3 - Section B-B' of Yield Line 1



Compute the external work (Fig. 6.6.1)

Segment A-C-E (Edge 1)

Load on segment A-C-E: $W_{ACE}(w, x_1, b) := w \cdot \left(\frac{1}{2} \cdot x_1 \cdot b\right)$

Deflection of centroid of segment A-C-E: $\Delta_{ACE} := \frac{\delta_p}{3}$ Triangular segment

External work done on segment A-C-E: $WE_{ACE}(w, x_1, b) := W_{ACE}(w, x_1, b) \, \Delta_{ACE}$

Segment B-F-D (Edge 2)

Load on segment B-F-D: $W_{BFD}(w, x_2, b) := w \cdot \left(\frac{1}{2} \cdot x_2 \cdot b\right)$

Deflection of centroid of segment B-F-D: $\Delta_{BFD} := \frac{\delta_p}{3}$ Triangular segment

External work done on segment B-F-D: $WE_{BFD}(w, x_2, b) := W_{BFD}(w, x_2, b) \, \Delta_{BFD}$

Segment C-D-F-E (Edge 3)

Load on segment C-D-F-E: $W_{CDFE1}(w, x_1, y) := w \cdot \left(\frac{1}{2} \cdot x_1 \cdot y\right)$ ... in left triangular area

$W_{CDFE2}(w, x_2, y) := w \cdot \left(\frac{1}{2} \cdot x_2 \cdot y\right)$ ... in right triangular area

$W_{CDFE3}(w, x_1, x_2, y, L) := w \cdot \left[\left(L - x_1 - x_2\right) \cdot y\right]$ ... in central rectangular area

Deflection of centroid of segment C-D-F-E:

$$\Delta_1 := \frac{\delta_p}{3}$$ Triangular segment

$$\Delta_2 := \frac{\delta_p}{3}$$ Triangular segment

$$\Delta_3 := \frac{\delta_p}{2}$$ Rectangular segment

External work done on segment C-D-F-E:

$$WE_{CDFE12}(w, x_1, x_2, y, L) := W_{CDFE1}(w, x_1, y)\,\Delta_1 + W_{CDFE2}(w, x_2, y)\,\Delta_2$$ ... work done by triangular segments

$$WE_{CDFE}(w, x_1, x_2, y, L) := WE_{CDFE12}(w, x_1, x_2, y, L) + W_{CDFE3}(w, x_1, x_2, y, L)\,\Delta_3$$ ... Total work done by panel CDFE

Segment A-B-F-E (Edge 4)

Load on segment A-B-F-E:

$$W_{ABFE1}(w, x_1, y, b) := w\cdot\left[\frac{1}{2}\cdot x_1\cdot(b-y)\right]$$

$$W_{ABFE2}(w, x_2, y, b) := w\cdot\left[\frac{1}{2}\cdot x_2\cdot(b-y)\right]$$

$$W_{ABFE3}(w, x_1, x_2, y, L, b) := w\cdot\left[\left(L - x_1 - x_2\right)\cdot(b-y)\right]$$

Deflection of centroid of segment A-B-F-E:

$$\Delta_1 := \frac{\delta_p}{3}$$ Triangular segment

$$\Delta_2 := \frac{\delta_p}{3}$$ Triangular segment

$$\Delta_3 := \frac{\delta_p}{2}$$ Rectangular segment

External work done on segment A-B-F-E:

$$WE_{ABFE12}(w, x_1, x_2, y, L, b) := W_{ABFE1}(w, x_1, y, b)\,\Delta_1 + W_{ABFE2}(w, x_2, y, b)\,\Delta_2$$ ... work done by triangular segments

$$WE_{ABFE}(w, x_1, x_2, y, L, b) := WE_{ABFE12}(w, x_1, x_2, y, L, b) + W_{ABFE3}(w, x_1, x_2, y, L, b)\,\Delta_3$$ ... Total work done by panel ABFE

Total External Work

Segments A-C-E and B-F-D

$$WE_X(w, x_1, x_2, b) := WE_{ACE}(w, x_1, b) + WE_{BFD}(w, x_2, b)$$

Segments C-D-F-E and A-B-F-E

$$WE_Y(w, x_1, x_2, y, L, b) := WE_{CDFE}(w, x_1, x_2, y, L) + WE_{ABFE}(w, x_1, x_2, y, L, b)$$

Total External Work

$$WE_{YL1}(w, x_1, x_2, y, b, L) := WE_X(w, x_1, x_2, b) + WE_Y(w, x_1, x_2, y, L, b)$$

Compute the internal work (Fig. 6.6.1)

Segment A-C-E (Edge 1 Reinforcement)

Negative moment capacity $M_{AC} = 302.2\,\text{kip}\cdot\text{ft}$

Positive moment capacity $M_x = 302.2\,\text{kip}\cdot\text{ft}$

Rotation of yield line:
For small displacements, the rotation of the negative moment yield line is given by:
$\theta = \delta / x_1$ (rad/ft)
Where: δ = virtual displacement
x1 = distance from edge to yield line to virtual displacement location

$$WI_{ACE}(M_n, M_p, x_1, b) := \left(M_n \cdot \frac{\delta_p}{x_1} \cdot \frac{1}{ft}\right) \cdot b + \left(M_p \cdot \frac{\delta_p}{x_1} \cdot \frac{1}{ft}\right) \cdot b$$

Segment B-F-D (Edge 2 Reinforcement)

Negative moment capacity $M_{BD} = 302.2\,\text{kip}\cdot\text{ft}$

Positive moment capacity $M_x = \blacksquare\,\text{kip}\cdot\text{ft}$

$$WI_{BFD}(M_n, M_p, x_2, b) := \left(M_n \cdot \frac{\delta_p}{x_2} \cdot \frac{1}{ft}\right) \cdot b + \left(M_p \cdot \frac{\delta_p}{x_2} \cdot \frac{1}{ft}\right) \cdot b$$

Segment C-D-F-E (Edge 3 Reinforcement)

Negative moment capacity $M_{CD} = 400.4\,\text{kip}\cdot\text{ft}$

Positive moment capacity $M_y = 400.4\,\text{kip}\cdot\text{ft}$

$$WI_{CDFE}(M_n, M_p, y, L) := \left(M_n \cdot \frac{\delta_p}{y} \cdot \frac{1}{ft}\right) \cdot L + \left(M_p \cdot \frac{\delta_p}{y} \cdot \frac{1}{ft}\right) \cdot L$$

Segment A-B-F-E (Edge 4 Reinforcement)

Negative moment capacity $M_{AB} = 38.8\,\text{kip}\cdot\text{ft}$

Positive moment capacity $M_y = 400.4\,\text{kip}\cdot\text{ft}$

$$WI_{ABFE}(M_n, M_p, y, b, L) := \left(M_n \cdot \frac{\delta_p}{b - y} \cdot \frac{1}{ft}\right) \cdot L + \left(M_p \cdot \frac{\delta_p}{b - y} \cdot \frac{1}{ft}\right) \cdot L$$

Total Internal Work

Segments A-C-E (Edge 1 Reinforcement) and B-F-D (Edge 2 Reinforcement)

$$WI_{12}(M_{n1}, M_{n2}, M_{p1}, M_{p2}, x_1, x_2, b) := WI_{ACE}(M_{n1}, M_{p1}, x_1, b) + WI_{BFD}(M_{n2}, M_{p2}, x_2, b)$$

Segments C-D-F-E (Edge 3 Reinforcement) and A-B-F-E (Edge 4 Reinforcement)

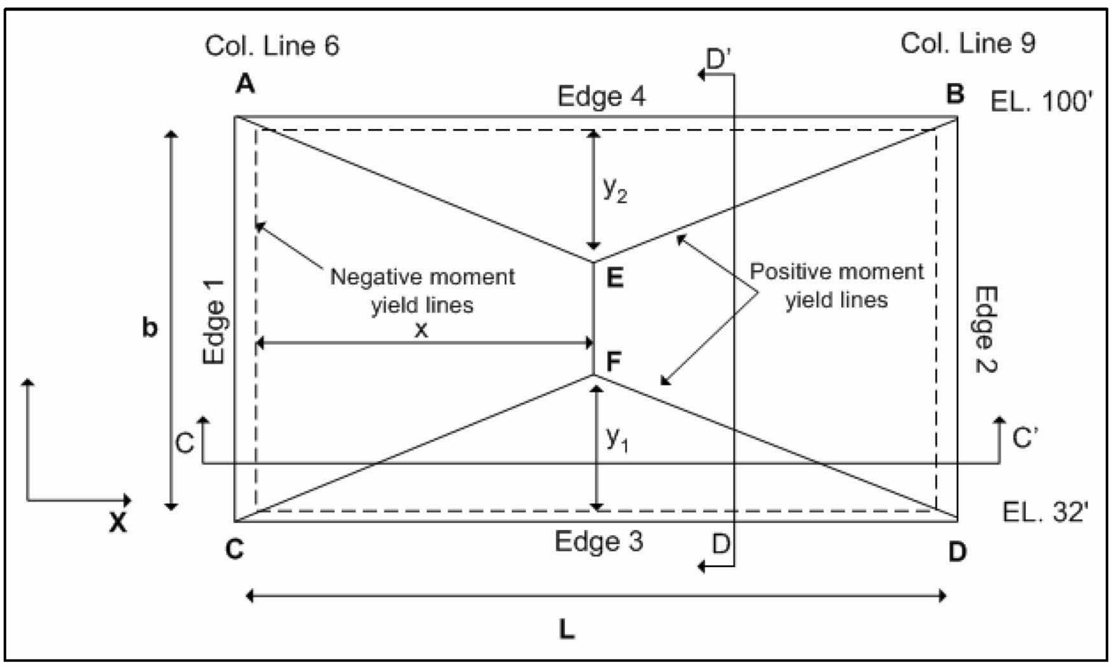
$$WI_{34}(M_{n3}, M_{n4}, M_{p3}, M_{p4}, y, b, L) := WI_{CDFE}(M_{n3}, M_{p3}, y, L) + WI_{ABFE}(M_{n4}, M_{p4}, y, b, L)$$

Total Internal Work

$$WI_{YL1}(M_{n1}, M_{n2}, M_{n3}, M_{n4}, M_{p1}, M_{p2}, M_{p3}, M_{p4}, x_1, x_2, y, b, L) := \begin{pmatrix} WI_{12}(M_{n1}, M_{n2}, M_{p1}, M_{p2}, x_1, x_2, b) \ldots \\ + WI_{34}(M_{n3}, M_{n4}, M_{p3}, M_{p4}, y, b, L) \end{pmatrix}$$

<u>Select a trial yield line pattern (Figure 6.6.4)</u>
Next, the yield line pattern in Figure 6.6.4 will be considered and the uniform load capacity will be determined. The minimum uniform load capacity of the panel for both sets of yield line patterns will determine the ultimate uniform load capacity of the panel.

Figure 6.6.4 - Yield Line 2 Schematic



<u>Give the panel a virtual displacement (Fig. 6.6.4)</u>
Line E-F in Figure 6.6.4 is given a virtual displacement of δ

Figure 6.6.5 and Figure 6.6.6 show sections through Figure 6.6.4 to illustrate the displacement shape of the panel under the virtual displacement.

$\delta_p := 1 \cdot ft$

Figure 6.6.5 - Section C-C' of Yield Line 2

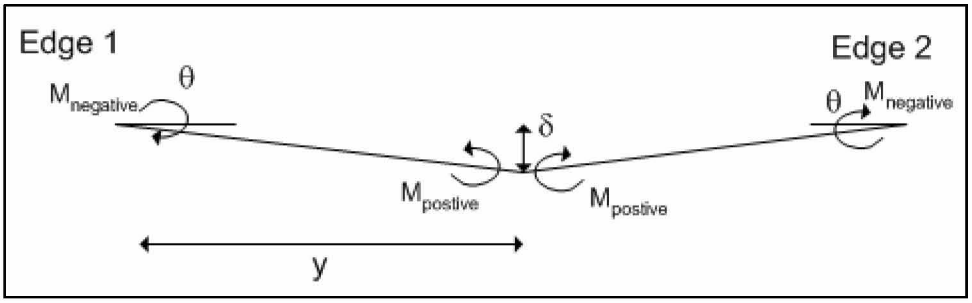
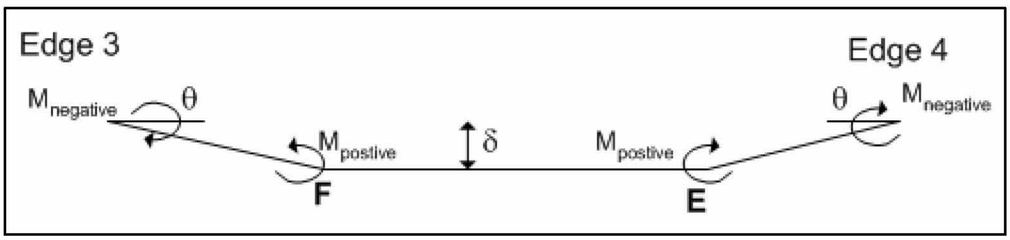


Figure 6.6.6- Section D-D' of Yield Line 2



Compute the external work (Fig. 6.6.4)

Segment C-D-F (Edge 3)

Load on segment C-D-F: $W_{CDF}(w, y_1, L) := w \cdot \left(\frac{1}{2} \cdot y_1 \cdot L\right)$

Deflection of centroid of segment C-D-F: $\Delta_{CDF} := \frac{\delta_p}{3}$ Triangular segment

External work done on segment C-D-F: $WE_{CDF}(w, y_1, L) := W_{CDF}(w, y_1, L)\, \Delta_{CDF}$

Segment A-B-E (Edge 4)

Load on segment A-B-E: $W_{ABE}(w, y_2, L) := w \cdot \left(\frac{1}{2} \cdot y_2 \cdot L\right)$

Deflection of centroid of segment A-B-E: $\Delta_{ABE} := \frac{\delta_p}{3}$ Triangular segment

External work done on segment A-B-E: $WE_{ABE}(w, y_2, L) := W_{ABE}(w, y_2, L)\, \Delta_{ABE}$

Segment A-C-F-E (Edge 1)

Load on segment A-C-F-E:

$$W_{ACFE1}(w, y_1, x) := w \cdot \left(\frac{1}{2} \cdot y_1 \cdot x\right)$$ ... lower triangular segment

$$W_{ACFE2}(w, y_2, x) := w \cdot \left(\frac{1}{2} \cdot y_2 \cdot x\right)$$ ... upper triangular segment

$$W_{ACFE3}(w, y_1, y_2, x, b) := w \cdot \left[\left(b - y_1 - y_2\right) \cdot x\right]$$ ... rectangular segment

Deflection of centroid of segment A-C-F-E:

$$\Delta_1 := \frac{\delta_p}{3}$$ Triangular segment

$$\Delta_2 := \frac{\delta_p}{3}$$ Triangular segment

$$\Delta_3 := \frac{\delta_p}{2}$$ Rectangular segment

External work done on segment A-C-F-E:

$$WE_{ACFE12}(w, y_1, y_2, x, b) := W_{ACFE1}(w, y_1, x)\,\Delta_1 + W_{ACFE2}(w, y_2, x)\,\Delta_2$$ ... triangular segments

$$WE_{ACFE}(w, y_1, y_2, x, b) := WE_{ACFE12}(w, y_1, y_2, x, b) + W_{ACFE3}(w, y_1, y_2, x, b)\,\Delta_3$$ ... Total work done by panel ACFE

Segment B-D-F-E (Edge 2)

Load on segment B-D-F-E:

$$W_{BDFE1}(w, y_1, x, L) := w \cdot \left[\frac{1}{2} \cdot y_1 \cdot (L - x)\right]$$ ... lower triangular segment

$$W_{BDFE2}(w, y_2, x, L) := w \cdot \left[\frac{1}{2} \cdot y_2 \cdot (L - x)\right]$$ ... upper triangular segment

$$W_{BDFE3}(w, y_1, y_2, x, b, L) := w \cdot \left[\left(b - y_1 - y_2\right) \cdot (L - x)\right]$$ ... rectangular segment

Deflection of centroid of segment B-D-F-E:

$$\Delta_1 := \frac{\delta_p}{3}$$ Triangular segment

$$\Delta_2 := \frac{\delta_p}{3}$$ Triangular segment

$$\Delta_3 := \frac{\delta_p}{2}$$ Rectangular segment

External work done on segment B-D-F-E:

$$WE_{BDFE12}(w,y_1,y_2,x,b,L) := W_{BDFE1}(w,y_1,x,L)\,\Delta_1 + W_{BDFE2}(w,y_2,x,L)\,\Delta_2$$ ... work done by triangular segments

$$WE_{BDFE}(w,y_1,y_2,x,b,L) := WE_{BDFE12}(w,y_1,y_2,x,b,L) + W_{BDFE3}(w,y_1,y_2,x,b,L)\,\Delta_3$$ ... Total work done by panel BDFE

Total External Work

Segments C-D-F and A-B-E

$$WE_X(w,y_1,y_2,L) := WE_{CDF}(w,y_1,L) + WE_{ABE}(w,y_2,L)$$

Segments A-C-F-E and B-D-F-E

$$WE_Y(w,y_1,y_2,x,b,L) := WE_{ACFE}(w,y_1,y_2,x,b) + WE_{BDFE}(w,y_1,y_2,x,b,L)$$

Total External Work

$$WE_{YL2}(w,y_1,y_2,x,L,b) := WE_X(w,y_1,y_2,L) + WE_Y(w,y_1,y_2,x,b,L)$$

Compute the internal work (Fig. 6.6.4)

Segment C-D-F (Edge 3 Reinforcement)

Negative moment capacity $M_{CD} = 400.4\,\text{kip}\cdot\text{ft}$

Positive moment capacity $M_y = 400.4\,\text{kip}\cdot\text{ft}$

Rotation of yield line:
For small displacements, the rotation of the negative moment yield line is given by:
$\theta = \delta / x_1$ (rad/ft)
Where: δ = virtual displacement
x1 = distance from edge to yield line to virtual displacement location

$$WI_{CDF}(M_n,M_p,y_1,L) := \left(M_n\cdot\frac{\delta_p}{y_1}\cdot\frac{1}{ft}\right)\cdot L + \left(M_p\cdot\frac{\delta_p}{y_1}\cdot\frac{1}{ft}\right)\cdot L$$

Segment A-B-E (Edge 4 Reinforcement)

Negative moment capacity $M_{AB} = 38.8\,\text{kip}\cdot\text{ft}$

Positive moment capacity $M_y = 400.4\,\text{kip}\cdot\text{ft}$

$$WI_{ABE}(M_n,M_p,y_2,L) := \left(M_n\cdot\frac{\delta_p}{y_2}\cdot\frac{1}{ft}\right)\cdot L + \left(M_p\cdot\frac{\delta_p}{y_2}\cdot\frac{1}{ft}\right)\cdot L$$

Segment A-C-F-E (Edge 1 Reinforcement)

Negative moment capacity $M_{AC} = 302.2\,\text{kip}\cdot\text{ft}$

Positive moment capacity $M_x = 302.2\,\text{kip}\cdot\text{ft}$

$$WI_{ACFE}(M_n, M_p, x, b) := \left(M_n \cdot \frac{\delta_p}{x} \cdot \frac{1}{ft}\right) \cdot b + \left(M_p \cdot \frac{\delta_p}{x} \cdot \frac{1}{ft}\right) \cdot b$$

Segment B-D-F-E (Edge 2 Reinforcement)

Negative moment capacity $M_{BD} = 302.2\,kip \cdot ft$

Positive moment capacity $M_x = 302.2\,kip \cdot ft$

$$WI_{BDFE}(M_n, M_p, x, L, b) := \left(M_n \cdot \frac{\delta_p}{L - x} \cdot \frac{1}{ft}\right) \cdot b + \left(M_p \cdot \frac{\delta_p}{L - x} \cdot \frac{1}{ft}\right) \cdot b$$

Total Internal Work

Segments C-D-F (Edge 3 Reinforcement) and A-B-E (Edge 4 Reinforcement)

$$WI_{34}(M_{n3}, M_{n4}, M_{p3}, M_{p4}, y_1, y_2, L) := WI_{CDF}(M_{n3}, M_{p3}, y_1, L) + WI_{ABE}(M_{n4}, M_{p4}, y_2, L)$$

Segments A-C-F-E (Edge 1 Reinforcement) and B-D-F-E (Edge 2 Reinforcement)

$$WI_{12}(M_{n1}, M_{n2}, M_{p1}, M_{p2}, x, L, b) := WI_{ACFE}(M_{n1}, M_{p1}, x, b) + WI_{BDFE}(M_{n2}, M_{p2}, x, L, b)$$

Total Internal Work

$$WI_{YL2}(M_{n1}, M_{n2}, M_{n3}, M_{n4}, M_{p1}, M_{p2}, M_{p3}, M_{p4}, y_1, y_2, x, L, b) := \begin{pmatrix} WI_{34}(M_{n3}, M_{n4}, M_{p3}, M_{p4}, y_1, y_2, L) \ldots \\ + WI_{12}(M_{n1}, M_{n2}, M_{p1}, M_{p2}, x, L, b) \end{pmatrix}$$

Test External and Internal work equations

The following calculations test the yield line equations previously developed using problems with know solutions.

Test 1 - Example 15-3 on page 687 of Ref. 2.2.36

This test will use the external and internal work equations for yield line 1 (Figure 6.6.1) and yield line 2 (Figure 6.6.4)

Yield Line 1 (Figure 6.6.1)

- Positive reinforcement only (pinned edges, all Mn values = 0 kip-ft) and is equal for all panel segments
- m = Mp1 = Mp2 = Mp3 = Mp4
- b = L (square panel)
- x1 = L/2
- x2 = L/2
- y = L/2

$$External(w, Length) := WE_{YL1}\left(w, \frac{Length}{2}, \frac{Length}{2}, \frac{Length}{2}, Length, Length\right)$$

$$External(w, Length) \rightarrow \frac{1}{3} \cdot w \cdot Length^2 \cdot ft$$

Equal to $w*L^2*\delta / 3$ (ft-kips) given in Step 3 of Example 15-3 on pg. 687 of Ref. 2.2.36 (δ = 1 ft)

$M_n := 0\,kip \cdot ft$

$$\text{Internal}(M_n, m, \text{Length}) := WI_{YL1}\left(M_n, M_n, M_n, M_n, m, m, m, m, \frac{\text{Length}}{2}, \frac{\text{Length}}{2}, \frac{\text{Length}}{2}, \text{Length}, \text{Length}\right)$$

$$\text{Internal}(M_n, m, \text{Length}) \rightarrow 8 \cdot m$$

Equal to 4*(2*m*δ) (ft-kips) given in Step 4 of Example 15-3 on pg. 688 of Ref. 2.2.36 (δ = 1 ft)

Equate external and internal work and solve for the moment capacity required to carry the uniform loading (w) over the square panel with length (L)

Given

$$\text{External}(w, \text{Length}) - \text{Internal}(M_n, m, \text{Length}) = 0$$

$$M(w, \text{Length}, m) := \text{Find}(m)$$

Find the moment capacity (m) in terms of the uniform load (w) and the length (L)

$$M(w, \text{Length}, m) \rightarrow \frac{1}{24} \cdot w \cdot \text{Length}^2 \cdot \text{ft}$$

Thus, the reinforcement in both directions of this panel should be designed for w*L^2 / 24.
This value exactly matches the result in Step 5 of Example 15-3 on pg. 688 of Ref. 2.2.36

Yield Line 2 (Figure 6.6.4)

- Positive reinforcement only (all Mn values = 0 kip-ft) and is equal for all panel segments
- m = Mp1 = Mp2 = Mp3 = Mp4
- b = L (square panel)
- y1 = L/2
- y2 = L/2
- x = L/2

$$\text{External}(w, \text{Length}) := WE_{YL2}\left(w, \frac{\text{Length}}{2}, \frac{\text{Length}}{2}, \frac{\text{Length}}{2}, \text{Length}, \text{Length}\right)$$

$$\text{External}(w, \text{Length}) \rightarrow \frac{1}{3} \cdot w \cdot \text{Length}^2 \cdot \text{ft}$$

Equal to w*L^2*δ / 3 (ft-kips) given in Step 3 of Example 15-3 on pg. 687 of Ref. 2.2.36 (δ = 1 ft)

$$M_n := 0 \text{kip} \cdot \text{ft}$$

$$\text{Internal}(M_n, m, \text{Length}) := WI_{YL2}\left(M_n, M_n, M_n, M_n, m, m, m, m, \frac{\text{Length}}{2}, \frac{\text{Length}}{2}, \frac{\text{Length}}{2}, \text{Length}, \text{Length}\right)$$

$$\text{Internal}(M_n, m, \text{Length}) \rightarrow 8 \cdot m$$

Equal to 4*(2*m*δ) (ft-kips) given in Step 4 of Example 15-3 on pg. 688 of Ref. 2.2.36 (δ = 1 ft)

Equate external and internal work and solve for the moment capacity required to carry the uniform loading (w) over the square panel with length (L)

Given

$\text{External}(w, \text{Length}) - \text{Internal}\left(M_n, m, \text{Length}\right) = 0$

$M(w, \text{Length}, m) := \text{Find}(m)$

Find the moment capacity (m) in terms of the uniform load (w) and the length (L)

$M(w, \text{Length}, m) \rightarrow \frac{1}{24} \cdot w \cdot \text{Length}^2 \cdot ft$

Thus, the reinforcement in both directions of this panel should be designed for w*L^2 / 24.
This value exactly matches the result in Step 5 of Example 15-3 on pg. 688 of Ref. 2.2.36

Test 2 - Case 1 of Example 18.9.1 on page 747 of Ref. 2.2.27
This test will use the external and internal work equations for yield line 1 (Figure 6.6.1) and yield line 2 (Figure 6.6.4)

- L = 25 feet
- b = 20 feet

Yield Line 1 (Figure 6.6.1)

- Mn1 = 3.125 kip-ft (Mn1 + Mn2 = 6.25 kip-ft)
- Mn2 = 3.125 kip-ft
- Mn3 = 2.0 kip-ft (Mn3 + Mn4 = 4 kip-ft)
- Mn4 = 2.0 kip-ft
- Mp1 = 3.125 kip-ft (Mp1 + Mp2 = 6.25 kip-ft)
- Mp2 = 3.125 kip-ft
- Mp3 = 2.0 kip-ft (Mp3 + Mp4 = 4 kip-ft)
- Mp4 = 2.0 kip-ft

$\text{Case1}_{EYL1}\left(w, x_1, x_2, y\right) := WE_{YL1}\left(w, x_1, x_2, y, 20ft, 25ft\right)$

$\text{Case1}_{EYL2}\left(w, y_1, y_2, x\right) := WE_{YL2}\left(w, y_1, y_2, x, 25ft, 20ft\right)$

$M_{n1} := 3.125 kip \cdot ft \quad M_{p1} := 3.125 kip \cdot ft$

$M_{n2} := 3.125 kip \cdot ft \quad M_{p2} := 3.125 kip \cdot ft$

$M_{n3} := 2.0 kip \cdot ft \quad M_{p3} := 2.0 kip \cdot ft$

$M_{n4} := 2.0 kip \cdot ft \quad M_{p4} := 2.0 kip \cdot ft$

$\text{Case1}_{IYL1}\left(x_1, x_2, y\right) := WI_{YL1}\left(M_{n1}, M_{n2}, M_{n3}, M_{n4}, M_{p1}, M_{p2}, M_{p3}, M_{p4}, x_1, x_2, y, 20ft, 25ft\right)$

$\text{Case1}_{IYL2}\left(y_1, y_2, x\right) := WI_{YL2}\left(M_{n1}, M_{n2}, M_{n3}, M_{n4}, M_{p1}, M_{p2}, M_{p3}, M_{p4}, y_1, y_2, x, 25ft, 20ft\right)$

$w := 0.25 ksf$

To solve for w, a searching technique is used to determine the minimum uniform load capacity of the yield line pattern. A initial value of 0.25 ksf is set for w. This initial value does not affect the searching technique.

Setting external work equal to internal work for yield line pattern 1 yields:

Given

$\text{Case1}_{EYL1}(w, x_1, x_2, y) - \text{Case1}_{IYL1}(x_1, x_2, y) = 0$

$\text{Case1}_{YL1uniform}(w, x_1, x_2, y) := \text{Find}(w)$

$$
\begin{aligned}
\text{result}(\text{func}, L, b, \text{step}) := \;& \text{result}_1 \leftarrow 1000000000 \\
& x1_{start} \leftarrow \frac{L}{ft} \cdot \frac{1}{step} \\
& x1_{end} \leftarrow \frac{L}{ft}\left(1 - \frac{1}{step}\right) \\
& x2_{start} \leftarrow \frac{L}{ft} \cdot \frac{1}{step} \\
& y_{start} \leftarrow \frac{b}{ft} \cdot \frac{1}{step} \\
& y_{end} \leftarrow \frac{b}{ft} \cdot \left(1 - \frac{1}{step}\right) \\
& \text{for } i \in x1_{start} .. x1_{end} \\
& \quad x_1 \leftarrow i \\
& \quad \text{for } j \in x2_{start} .. \left(\frac{L}{ft} - x_1\right) \\
& \quad\quad x_2 \leftarrow j \\
& \quad\quad \text{for } k \in y_{start} .. y_{end} \\
& \quad\quad\quad y \leftarrow k \\
& \quad\quad\quad \text{temp} \leftarrow \text{func}(w, x_1 \cdot ft, x_2 \cdot ft, y \cdot ft) \\
& \quad\quad\quad \text{if } \text{temp} \cdot \frac{1}{ksf} \le \text{result}_1 \\
& \quad\quad\quad\quad \text{result}_1 \leftarrow \text{temp} \cdot \frac{1}{ksf} \\
& \quad\quad\quad\quad \text{result}_2 \leftarrow x_1 \\
& \quad\quad\quad\quad \text{result}_3 \leftarrow x_2 \\
& \quad\quad\quad\quad \text{result}_4 \leftarrow y \\
& \text{result}
\end{aligned}
$$

Description
This loop determines the minimum uniform load capacity and corresponding yield line pattern of the wall panel.

The yield line pattern is defined by the input variable "func" and variables, x1, x2, and y.

The initial yield line pattern is determined by setting the x1 value equal to L/step, the x2 value equal to L/step, and the y value equal to b/step. The x1,x2, and y values are then incremented by 1 foot to determine the next yield line pattern.

For each set of x1, x2, y values, the uniform load capacity (w) of the yield line pattern is returned from "func". If the w value is smaller than the previously stored uniform load capacity (stored in the first row of the array "result"), then this value is stored as the new minimum uniform load capacity of the panel.

result_1 = minimum uniform load capacity

result_2 = x1 value of the yield line pattern causing the minimum uniform load capacity (or y1 value if Yield Line Pattern 2 is defined in the input variable func)

result_3 = x2 value of the yield line pattern causing the minimum uniform load capacity (or y2 value if Yield Line Pattern 2 is defined in the input variable func)

result_4 = y value of the yield line pattern causing the minimum uniform load capacity (or x value if Yield Line Pattern 2 is defined in the input variable func)

$L := 25ft$

$b := 20ft$

$step := 10$

The initial yield line pattern is:
x1 = L/step = 25ft/10 = 2.5 feet
x2 = L/step = 25ft/10 = 2.5 feet
y = b/step = 20ft/10 = 2.0 feet

$$\mathrm{result}\left(\mathrm{Case1}_{\mathrm{YL1uniform}}, L, b, \mathrm{step}\right) = \begin{pmatrix} 0.240 \\ 12.500 \\ 12.500 \\ 10.000 \end{pmatrix}$$

Now solve Case 1 using the Yield Line 2 (Figure 6.6.4) internal and external work equations

Given

$$\mathrm{Case1}_{\mathrm{EYL2}}\left(w, y_1, y_2, x\right) - \mathrm{Case1}_{\mathrm{IYL2}}\left(y_1, y_2, x\right) = 0$$

$$\mathrm{Case1}_{\mathrm{YL2uniform}}\left(w, y_1, y_2, x\right) := \mathrm{Find}(w)$$

$$\mathrm{result}\left(\mathrm{Case1}_{\mathrm{YL2uniform}}, b, L, \mathrm{step}\right) = \begin{pmatrix} 0.240 \\ 10.000 \\ 10.000 \\ 12.500 \end{pmatrix}$$

Test 2 Summary of Results:

Using the yield line equations for yield line 1 (Figure 6.6.1), the ultimate uniform load capacity of the wall panel is 0.240 ksf and the yield line dimensions are x1 = 12.5 ft, x2 = 12.5 ft, and y = 10.00 ft.

Using the yield line equations for yield line 2 (Figure 6.6.4), the ultimate uniform load capacity of the wall panel is 0.241 ksf and the yield line dimensions are y1 = 10.00 ft, y2 = 10.00 ft, and x = 12.50 ft.

The solution for Case 1 of Example 18.9.1 on page 748 of Ref. 2.2.27 calculates the ultimate uniform load capacity of the wall panel as 0.240 ksf and the yield line dimensions are x1 = 12.5 ft, x2 = 12.5 ft, and y = 10.0 ft (based on Figure 6.6.1 terminology) and y1 = 10 ft, y2 = 10 ft, and x = 12.5 ft (based on Figure 6.6.4 terminology).

The ultimate uniform load capacity is a good match with Case 1 of Example 18.9.1 of Ref. 2.2.27. Subsequent test value of the "step" variable show that the ultimate uniform load capacity is relatively insensitive to the "exact" yield line pattern. This point is expanded on further by using a set of known x1(y1), x2(y), and y(x) values and solving the $\mathrm{Case1}_{\mathrm{uniform}}$ function for w.

Yield Line 1 - Figure 6.6.1

Trial yield line 1
Trial yield line 2
Trial yield line 3
Trial yield line 4
Trial yield line 5

$$x_1 := \begin{pmatrix} 12.10 \\ 12.25 \\ 12.5 \\ 12.6 \\ 12.8 \end{pmatrix} \cdot \mathrm{ft} \qquad x_2 := \begin{pmatrix} 12.10 \\ 12.25 \\ 12.5 \\ 12.6 \\ 12.8 \end{pmatrix} \cdot \mathrm{ft} \qquad y := \begin{pmatrix} 10.08 \\ 10.01 \\ 10.0 \\ 9.8 \\ 9.7 \end{pmatrix} \cdot \mathrm{ft}$$

$$\mathrm{Case1}_{\mathrm{YL1uniform}}\left(w, {x_1}_1, {x_2}_1, y_1\right) = 0.24013\,\mathrm{ksf}$$

$$\mathrm{Case1}_{\mathrm{YL1uniform}}\left(w, {x_1}_2, {x_2}_2, y_2\right) = 0.24005\,\mathrm{ksf}$$

$Case1_{YL1uniform}(w, x1_3, x2_3, y_3) = 0.24000\,ksf$

$Case1_{YL1uniform}(w, x1_4, x2_4, y_4) = 0.24006\,ksf$

$Case1_{YL1uniform}(w, x1_5, x2_5, y_5) = 0.24018\,ksf$

The ultimate uniform load capacity for the trial yield lines calculated using the yield line 1 equations (Figure 6.6.1) are all reasonably close to that calculated from the closed-form solution in Case 1 of Example 18.9.1 on page 748 of Ref. 2.2.27 (Example 18.9.1 Case 1 yield line dimensions are those for Trial yield line 3).

Yield Line 2 - Figure 6.6.4

Trial yield line 1
Trial yield line 2
Trial yield line 3
Trial yield line 4
Trial yield line 5

$$y_1 := \begin{pmatrix} 10.08 \\ 10.01 \\ 10.0 \\ 9.8 \\ 9.7 \end{pmatrix} \cdot ft \qquad y_2 := \begin{pmatrix} 9.08 \\ 10.01 \\ 10.0 \\ 9.8 \\ 9.7 \end{pmatrix} \cdot ft \qquad x := \begin{pmatrix} 12.10 \\ 12.25 \\ 12.5 \\ 12.6 \\ 12.8 \end{pmatrix} \cdot ft$$

$Case1_{YL2uniform}(w, y1_1, y2_1, x_1) = 0.24067\,ksf$

$Case1_{YL2uniform}(w, y1_2, y2_2, x_2) = 0.24005\,ksf$

$Case1_{YL2uniform}(w, y1_3, y2_3, x_3) = 0.24000\,ksf$

$Case1_{YL2uniform}(w, y1_4, y2_4, x_4) = 0.24006\,ksf$

$Case1_{YL2uniform}(w, y1_5, y2_5, x_5) = 0.24018\,ksf$

The ultimate uniform load capacity for the trial yield lines calculated using the yield line 2 equations (Figure 6.6.4) are all reasonably close to that calculated from the closed-form solution in Case 1 of Example 18.9.1 on page 748 of Ref. 2.2.27 (Example 18.9.1 Case 1 yield line dimensions are those for Trial yield line 3).

Test 3 - Case 2 of Example 18.9.1 on page 747 of Ref. 2.2.27
Use Yield Line 1 equations (Figure 6.6.1)

- L = 25 feet
- b = 20 feet
- Mn1 = 1.0 kip-ft (Mn1 + Mn2 = 2 kip-ft)
- Mn2 = 1.0 kip-ft
- Mn3 = 2.0 kip-ft (Mn3 + Mn4 = 4 kip-ft)
- Mn4 = 2.0 kip-ft
- Mp1 = 1.0 kip-ft (Mp1 + Mp2 = 2 kip-ft)
- Mp2 = 1.0 kip-ft
- Mp3 = 2.0 kip-ft (Mp3 + Mp4 = 4 kip-ft)
- Mp4 = 2.0 kip-ft

$Case2_E(w, x_1, x_2, y) := WE_{YL1}(w, x_1, x_2, y, 20ft, 25ft)$

$M_{n1} := 1.0kip \cdot ft \qquad M_{p1} := 1.0kip \cdot ft$

$M_{n2} := 1.0kip \cdot ft \qquad M_{p2} := 1.0kip \cdot ft$

$M_{n3} := 2.0 \cdot kip \cdot ft \qquad M_{p3} := 2.0kip \cdot ft$

$M_{n4} := 2.0\,\text{kip}\cdot\text{ft} \qquad M_{p4} := 2.0\,\text{kip}\cdot\text{ft}$

$\text{Case2}_I(x_1, x_2, y) := \text{WI}_{YL1}(M_{n1}, M_{n2}, M_{n3}, M_{n4}, M_{p1}, M_{p2}, M_{p3}, M_{p4}, x_1, x_2, y, 20\text{ft}, 25\text{ft})$

$w := 1\,\text{ksf}$

Given

$\text{Case2}_E(w, x_1, x_2, y) - \text{Case2}_I(x_1, x_2, y) = 0$

$\text{Case2}_{uniform}(w, x_1, x_2, y) := \text{Find}(w)$

$L := 25\text{ft}$

$b := 20\text{ft}$

$step := 10$

The initial yield line pattern is:
x1 = L/step = 25ft/10 = 2.5 feet
x2 = L/step = 25ft/10 = 2.5 feet
y = b/step = 20ft/10 = 2.0 feet

$$\text{result}(\text{Case2}_{uniform}, L, b, step) = \begin{pmatrix} 0.152 \\ 8.500 \\ 8.500 \\ 10.000 \end{pmatrix}$$

Test 3 Summary of Results: The ultimate uniform load capacity of the wall panel is 0.152 ksf and the yield line dimensions (See Figure 6.6.1) x1 = 8.5 ft, x2 = 8.5 ft, and y = 10.00 ft.

The solution for Case 2 of Example 18.9.1 on page 748-749 of Ref. 2.2.27 calculates the ultimate uniform load capacity of the wall panel as 0.152 ksf and the yield line dimensions are x1 = 8.884 ft, x2 = 8.884 ft, and y = 10.0 ft.

The ultimate uniform load capacity matches exactly with the Case 2 of Example 18.9.1 of Ref. 2.2.27. The difference in the yield line dimensions is attributed to the search algorithm used above and also to the fact that the the ultimate uniform load capacity is relatively insensitive to the "exact" yield line pattern. This last point is expanded on further by using a set of known x1, x2, and y values and solving the $\text{Case2}_{uniform}$ function for w.

Trial yield line 1
Trial yield line 2
Trial yield line 3
Trial yield line 4
Trial yield line 5

$$x_1 := \begin{pmatrix} 9.10 \\ 9.0 \\ 8.884 \\ 8.7 \\ 8.5 \end{pmatrix}\cdot\text{ft} \qquad x_2 := \begin{pmatrix} 9.10 \\ 9.0 \\ 8.884 \\ 8.7 \\ 8.5 \end{pmatrix}\cdot\text{ft} \qquad y := \begin{pmatrix} 10.08 \\ 10.01 \\ 10.0 \\ 9.8 \\ 10 \end{pmatrix}\cdot\text{ft}$$

$\text{Case2}_{uniform}(w, x_{1_1}, x_{2_1}, y_1) = 0.15207\,\text{ksf}$

The ultimate uniform load capacity for the trial yield

$\text{Case2}_{\text{uniform}}\left(w, x1_2, x2_2, y_2\right) = 0.15205\,\text{ksf}$

$\text{Case2}_{\text{uniform}}\left(w, x1_3, x2_3, y_3\right) = 0.15204\,\text{ksf}$

$\text{Case2}_{\text{uniform}}\left(w, x1_4, x2_4, y_4\right) = 0.15210\,\text{ksf}$

$\text{Case2}_{\text{uniform}}\left(w, x1_5, x2_5, y_5\right) = 0.15213\,\text{ksf}$

lines are all reasonably close to that calculated from the closed-form solution in Case 2 of Example 18.9.1 on page 749 of Ref. 2.2.27 (Example 18.9.1 Case 2 yield line dimensions are those for Trial yield line 3).

Test 4 - Case 3 of Example 18.9.1 on page 747 of Ref. 2.2.27

Use Yield Line 2 equations (Figure 6.6.4)

- L = 25 feet
- b = 20 feet
- Mn1 = 4.0 kip-ft (Mn1 + Mn2 = 8 kip-ft)
- Mn2 = 4.0 kip-ft
- Mn3 = 2.0 kip-ft (Mn3 + Mn4 = 4 kip-ft)
- Mn4 = 2.0 kip-ft
- Mp1 = 4.0 kip-ft (Mp1 + Mp2 = 8 kip-ft)
- Mp2 = 4.0 kip-ft
- Mp3 = 2.0 kip-ft (Mp3 + Mp4 = 4 kip-ft)
- Mp4 = 2.0 kip-ft

$\text{Case3}_E\left(w, y_1, y_2, x\right) := \text{WE}_{\text{YL2}}\left(w, y_1, y_2, x, 25\text{ft}, 20\text{ft}\right)$

$M_{n1} := 4.0\text{kip}\cdot\text{ft} \qquad M_{p1} := 4.0\text{kip}\cdot\text{ft}$

$M_{n2} := 4.0\text{kip}\cdot\text{ft} \qquad M_{p2} := 4.0\text{kip}\cdot\text{ft}$

$M_{n3} := 2.0\cdot\text{kip}\cdot\text{ft} \qquad M_{p3} := 2.0\text{kip}\cdot\text{ft}$

$M_{n4} := 2.0\text{kip}\cdot\text{ft} \qquad M_{p4} := 2.0\text{kip}\cdot\text{ft}$

$\text{Case3}_I\left(y_1, y_2, x\right) := \text{WI}_{\text{YL2}}\left(M_{n1}, M_{n2}, M_{n3}, M_{n4}, M_{p1}, M_{p2}, M_{p3}, M_{p4}, y_1, y_2, x, 25\text{ft}, 20\text{ft}\right)$

$w := 1\text{ksf}$

Given

$\text{Case3}_E\left(w, y_1, y_2, x\right) - \text{Case3}_I\left(y_1, y_2, x\right) = 0$

$\text{Case3}_{\text{uniform}}\left(w, y_1, y_2, x\right) := \text{Find}(w)$

$L := 25\text{ft}$

$b := 20\text{ft}$

$\text{step} := 10$

The initial yield line pattern is:
y1 = b/step = 20ft/10 = 2.0 feet
y2 = b/step = 20ft/10 = 2.0 feet
y = L/step = 25ft/10 = 2.5 feet

$$\mathrm{result}\left(\mathrm{Case3}_{\mathrm{uniform}}, b, L, \mathrm{step}\right) = \begin{pmatrix} 0.273 \\ 9.000 \\ 9.000 \\ 12.500 \end{pmatrix}$$

Test 4 Summary of Results: The ultimate uniform load capacity of the wall panel is 0.273 ksf and the yield line dimensions (See Figure 6.6.4) y1 = 9.0 ft, y2 = 9.0 ft, and x = 12.5 ft.

The solution for Case 3 of Example 18.9.1 on page 748-749 of Ref. 2.2.27 calculates the ultimate uniform load capacity of the wall panel as 0.273 ksf and the yield line dimensions are y1 = 9.375 ft, y2 = 9.375 ft, and x = 12.5 ft.

The ultimate uniform load capacity matches exactly with the Case 3 of Example 18.9.1 of Ref. 2.2.27. The difference in the yield line dimensions is attributed to the search algorithm used above and also to the fact that the the ultimate uniform load capacity is relatively insensitive to the "exact" yield line pattern. This last point is expanded on further by using a set of known y1, y2, and x values and solving the $\mathrm{Case3}_{\mathrm{uniform}}$ function for w.

Trial yield line 1
Trial yield line 2
Trial yield line 3
Trial yield line 4
Trial yield line 5

$$y_1 := \begin{pmatrix} 9.0 \\ 9.125 \\ 9.375 \\ 8.95 \\ 8.75 \end{pmatrix} \cdot \mathrm{ft} \qquad y_2 := \begin{pmatrix} 9.0 \\ 9.125 \\ 9.375 \\ 8.95 \\ 8.75 \end{pmatrix} \cdot \mathrm{ft} \qquad x := \begin{pmatrix} 12.5 \\ 12.4 \\ 12.5 \\ 12.0 \\ 11.8 \end{pmatrix} \cdot \mathrm{ft}$$

$$\mathrm{Case3}_{\mathrm{uniform}}\left(w, y_{1_1}, y_{2_1}, x_1\right) = 0.27327\,\mathrm{ksf}$$

$$\mathrm{Case3}_{\mathrm{uniform}}\left(w, y_{1_2}, y_{2_2}, x_2\right) = 0.27317\,\mathrm{ksf}$$

$$\mathrm{Case3}_{\mathrm{uniform}}\left(w, y_{1_3}, y_{2_3}, x_3\right) = 0.27307\,\mathrm{ksf}$$

$$\mathrm{Case3}_{\mathrm{uniform}}\left(w, y_{1_4}, y_{2_4}, x_4\right) = 0.27356\,\mathrm{ksf}$$

$$\mathrm{Case3}_{\mathrm{uniform}}\left(w, y_{1_5}, y_{2_5}, x_5\right) = 0.27410\,\mathrm{ksf}$$

The ultimate uniform load capacity for the trial yield lines are all reasonably close to that calculated from the closed-form solution in Case 2 of Example 18.9.1 on page 749 of Ref. 2.2.27 (Example 18.9.1 Case 2 yield line dimensions are those for Trial yield line 3).

Having confirmed the yield line equations developed against known solutions, apply the equations to the actual Case 6 wall panel

<u>Step 4: Equate the external and internal work assuming Yield Line Pattern 1</u>

The properties of the wall panel are given below -

- b = 68 feet
- L = 94 feet
- Mn1 = M_{AC} (#11 @ 12" horizontally)
- Mn2 = M_{BD} (#11 @ 12" horizontally)
- Mn3 = M_{CD} (#11 @ 9" vertically)
- Mn4 = M_{AB} (#7 @ 12") [Negative moment along edge 4 (EL. 100') is controlled by the roof slab capacity]
- Mp1 = M_x (#11 @ 12" horizontally)

- Mp2 = M_x (#11 @ 12" horizontally)
- Mp3 = M_y (#11 @ 9" horizontally)
- Mp4 = M_y (#11 @ 9" horizontally)

$L := 94\text{ft}$

$b := 68\text{ft}$

$M_{n1} := M_{AC}$ $\quad M_{n1} = 302.2\,\text{kip}\cdot\text{ft}$

$M_{n2} := M_{BD}$ $\quad M_{n2} = 302.2\,\text{kip}\cdot\text{ft}$

$M_{n3} := M_{CD}$ $\quad M_{n3} = 400.4\,\text{kip}\cdot\text{ft}$

$M_{n4} := M_{AB}$ $\quad M_{n4} = 38.8\,\text{kip}\cdot\text{ft}$

$M_{p1} := M_x$ $\quad M_{p1} = 302.2\,\text{kip}\cdot\text{ft}$

$M_{p2} := M_x$ $\quad M_{p2} = 302.2\,\text{kip}\cdot\text{ft}$

$M_{p3} := M_y$ $\quad M_{p3} = 400.4\,\text{kip}\cdot\text{ft}$

$M_{p4} := M_y$ $\quad M_{p4} = 400.4\,\text{kip}\cdot\text{ft}$

Yield Line 1 (Figure 6.6.1) - determine the ultimate uniform load capacity of the wall panel using the equations derived for yield line 1 (Figure 6.6.1)

$$\text{WallPanel}_{\text{External}}(w, x_1, x_2, y) := \text{WE}_{\text{YL1}}(w, x_1, x_2, y, b, L)$$

$$\text{WallPanel}_{\text{Internal}}(x_1, x_2, y) := \text{WI}_{\text{YL1}}(M_{n1}, M_{n2}, M_{n3}, M_{n4}, M_{p1}, M_{p2}, M_{p3}, M_{p4}, x_1, x_2, y, b, L)$$

Given

$$\text{WallPanel}_{\text{External}}(w, x_1, x_2, y) - \text{WallPanel}_{\text{Internal}}(x_1, x_2, y) = 0$$

$$\text{WallPanel}_{\text{Ultimate}}(w, x_1, x_2, y) := \text{Find}(w)$$

$L = 94.0\text{ ft}$

$b = 68.0\text{ ft}$

$\text{step} := 10$

The initial yield line pattern is:
x1 = L/step = 94ft/10 = 9.4 feet
x2 = L/step = 94ft/10 = 9.4 feet
y = b/step = 68ft/10 = 6.8 feet

Step 5: Solve for the uniform load (w) assuming Yield Line Pattern 1

$$\text{Results}_{\text{YieldLine1}} := \text{result}(\text{WallPanel}_{\text{Ultimate}}, L, b, \text{step})$$

$$\text{Results}_{\text{YieldLine1}} = \begin{pmatrix} 2.362 \\ 39.400 \\ 39.400 \\ 38.800 \end{pmatrix}$$

... Yield Line Pattern 1 results

Step 4a: Equate the external and internal work assuming Yield Line Pattern 2

$WallPanel_{External}(w, y_1, y_2, x) := WE_{YL2}(w, y_1, y_2, x, L, b)$

$WallPanel_{Internal}(y_1, y_2, x) := WI_{YL2}(M_{n1}, M_{n2}, M_{n3}, M_{n4}, M_{p1}, M_{p2}, M_{p3}, M_{p4}, y_1, y_2, x, L, b)$

Given

$WallPanel_{External}(w, y_1, y_2, x) - WallPanel_{Internal}(y_1, y_2, x) = 0$

$WallPanel_{Ultimate}(w, y_1, y_2, x) := Find(w)$

$L = 94.0\ ft$

$b = 68.0\ ft$

$step := 10$

The initial yield line pattern is:
y1 = b/step = 68ft/10 = 6.8 feet
y2 = b/step = 68ft/10 = 6.8 feet
x = L/step = 94ft/10 = 9.4 feet

Step 5a: Solve for the uniform load (w) assuming Yield Line Pattern 2

$Results_{YieldLine2} := result(WallPanel_{Ultimate}, b, L, step)$

$$Results_{YieldLine2} = \begin{pmatrix} 2.397 \\ 38.800 \\ 28.800 \\ 47.400 \end{pmatrix}$$

... Yield Line Pattern 2 results

The ultimate uniform load capacity of the wall panel is the minimum of the ultimate uniform load capacities calculated from the yield line 1 and yield line 2 patterns.

$w_{ultimate} := 2.362ksf$

Capacity of the wall panel $C_{98\%} := w_{ultimate}$

Total demand on the panel due to BDBGM acceleration applied uniformly to the entire panel

$$D_{BDBGM} := \frac{SACC_{h_6}}{g} \cdot thick_6 \cdot \rho \qquad D_{BDBGM} = 1.6\,ksf$$

Strength Margin Factor for Case 6 Wall Panel

$F_{sCase6} := \dfrac{C_{98\%}}{D_{BDBGM}}$ $\qquad F_{sCase6} = 1.43$

Inelastic Energy Absorption Factor for the Case 6 Wall Panel
The inelastic energy absorption factor for the wall panel is conservatively taken as that for a column of a SMRF reinforced concrete moment frame at Limit State A (F_μ = 2.0 from Table 5-1 of Ref. 2.2.6). This is a conservative value because the axial load in the wall panel is not expected to be as large as would typically be seen in a SMRF moment frame column.

$F_{\mu Case6} := 2.0$

HCLPF Capacity for Case 6 Wall Panel
Use the horizontal PGA for the Wall Panel

$HCLPF_{Case6} := F_{sCase6} \cdot F_{\mu Case6} \cdot PGA_h$ $\qquad HCLPF_{Case6} = 2.62\,g$

Replace the HCLPF capacity calculated for Case 6 as a wall strip with that calculated considering the yield line capacity of the wall.

$HCLPF_{Mom_6} := HCLPF_{Case6}$ $\qquad HCLPF_{Mom}^{T} = \begin{pmatrix} 4.05 & 4.12 & 3.46 & 2.16 & 2.33 & 2.62 \end{pmatrix} g$

Out-of-Plane Shear HCLPF Capacity of the Wall Panel

Determine the HCLPF capacity using the the punching shear capacity of the entire panel

$A_{perimeter} := (b - d_{panel}) \cdot 2d_{panel} + (L - d_{panel}) \cdot 2 \cdot d_{panel}$ $\qquad A_{perimeter} = 1131.4\,ft^2$ Punching shear perimeter of the wall panel at d/2 away from the face

$\phi V_{np} := \phi_s \cdot 4 \cdot \sqrt{f_c \cdot psi} \cdot A_{perimeter}$ $\qquad \phi V_{np} = 41080.7\,kip$

Total shear acting on the panel caused by the BDBGM seismic load

$V_{BDBGM} := D_{BDBGM} \cdot b \cdot L$ $\qquad V_{BDBGM} = 10527.6\,kip$

Out-of-Plane Shear Strength Margin Factor

$F_{sShear} := \dfrac{\phi V_{np}}{V_{BDBGM}}$ $\qquad F_{sShear} = 3.90$

Inelastic Energy Absorption Factor for Out-of-Plane Shear
Out-of-plane shear is a brittle failure mechanism thus no inelastic energy absorption factor is considered.

$F_{\mu Shear} := 1.0$

HCLPF Capacity for Case 6 Wall Panel
Use the horizontal PGA for the Wall Panel

$HCLPF_{Shear} := F_{sShear} \cdot F_{\mu Shear} \cdot PGA_h$ $\qquad HCLPF_{Shear} = 3.57\,g$ > HLCPF for out-of-plane bending of the wall panel

6.6.8 HCLPF Capacity Calculations for Out-of-Plane Failure Mechanisms of Walls - Summary

$HCLPF_{oop} := \min(HCLPF_{Mom})$

$HCLPF_{Mom}{}^{T} = (4.05 \;\; 4.12 \;\; 3.46 \;\; 2.16 \;\; 2.33 \;\; 2.62)\, g$

$HCLPF_{oop} = 2.16\, g$ HCLPF corresponding to Case 4 and an $F\mu$ = $F_{\mu_4} = 2.25$

- The minimum HCLPF capacity for out-of-plane failure mechanisms of the CRCF walls is 2.16g.
- The above listed HCLPF capacities are larger than the HCLPF capacity for the CRCF shear walls (1.82g). Therefore, the in-plane shear HCLPF capacity of the CRCF controls over the out-of-plane bending HCLPF capacity of the CRCF walls.

6.7 SHEAR FRICTION EVALUATION

For the HCLPF capacity calculations for the CRCF shear walls, the in-plane shear capacity of walls is determined by equations 6.2.1 to 6.2.6. However, shear friction is to be evaluated "where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times" (ACI 349-01, Section 11.7.1 [Ref. 2.2.2]).

The last example in the above quotation, "interface between two concrete cast at different times" is interpreted to apply to construction joints of shear wall structures. For the purpose of the HCLPF capacity evaluations, the wall-basemat junction is evaluated for shear friction based on the recommendations of EPRI 6041, Appendix L (Ref. 2.2.43). The shear friction HCLPF capacity evaluation of only the wall-basemat junction at EL. 0'-0" is bounding because the other elevations have the same wall thickness and same vertical reinforcement as EL. 0'-0" and the in-plane shear demand at EL. 0'-0" is higher.

EPRI 6041, Appendix L (Ref. 2.2.43) states, in summary, that:

1. Sliding shear is not an issue if the diagonal shear capacity exceeds $7\sqrt{f'c}$.
2. Sliding shear may occur if the applied shear is between $3\sqrt{f'c}$ and $7\sqrt{f'c}$.

The Excel file “CRCF – Shear Friction.xls” included in Attachment E contains the evaluations for shear friction of the stick elements terminating at EL. 0’-0”. Two checks are made:

1. The diagonal shear capacity of the segment is determined. If the diagonal shear capacity is greater than $7\sqrt{f'c}$, then sliding shear will not occur.

2. If the diagonal shear capacity is less than $7\sqrt{f'c}$, then the shear friction capacity of the wall is determined per ACI 349-01 Section 11.7.4 (Ref. 2.2.2) and checked against the actual in-plane shear demand on the wall. If the vertical reinforcement can transmit the in-plane shear demand through shear friction, then the wall is adequate.

Column J in the Excel file “CRCF – Shear Friction.xls” included in Attachment E shows that only 2 wall segments at EL. 0-0” (“2A.3” and “4A.2”) have diagonal shear capacities less than $7\sqrt{f'c}$. Column O shows that the shear friction demand-to-capacity ratios for these walls are less than 1.0. Therefore, sliding shear will not occur in the CRCF under the BDBGM seismic loads.

The data and calculations contained within the Excel file “CRCF – Shear Friction.xls” in Attachment E are shown in Table 6.7.1.

Table 6.7.1 Shear Friction Evaluation Information

Excel Column*	Parameter Name	Description
B	Stick ID	Stick elements located at EL. 0'-0"
C	Joint I	Starting joint of the element located at EL. 0'-0"
D	A_{sv}	vertical wall reinforcement (in^2 / ft / face)
E	l_w	length of wall segment (feet)
F	thick	wall thickness (feet)
G	hw/lw	Height-to-length ratio for stick element determined in column T in sheet "Frag. Shear Calculation" in Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" in Attachment E.
H	Vc	Nominal shear strength determined as the sum of column U and Y for hw/lw < 2.0 or column V and Z if hw/lw > 2.0 in sheet "Frag. Shear Calculation" in Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" in Attachment E.
J	Greater than > $7\sqrt{f'c}$	If Vc is greater than $7\sqrt{f'c}$ a value of 0 is entered. If Vc is less than $7\sqrt{f'c}$, a value of 1 is entered.
L	Vu_{BDBGM}	In-plane shear due to the BDBGM seismic load retrieved from column AF in sheet "Frag. Shear Calculation" in Excel file "CRCF – Fragility – In-Plane Shear Wall.xls" in Attachment E.
M	VnSF	Nominal shear friction capacity determined as- Min($2A_{sv}f_y\mu$, 0.2f'cAc, 800Ac(lbs)) (Eq. 11-25 & Section 11.7.5 in Ref. 2.2.2) where μ = 1.0
O	Vu_{BDBGM}/VnSF	If Vc < $7\sqrt{f'c}$, then Column L divided by column M. If Vc > $7\sqrt{f'c}$, then no value is entered in this column

* Source - Sheet "Shear Friction" in file "CRCF – Shear Friction.xls" included in Attachment E

6.8 LIMIT STATE D HCLPF CAPACITY

Table 6.8.1 contains the minimum HCLPF capacities calculated for each failure mode of the CRCF and the Limit State A Fμ factor associated with the HCLPF capacity. The Limit State D HCLPF capacity for each failure mechanism is determined by dividing the HCLPF capacity by the Limit State A Fμ factor (i.e. the Limit State D Fμ factor = 1.0).

It is recognized that the BDBGM seismic analysis in Ref. 2.2.5 used 10% structural damping (Response Level 3 damping for reinforced concrete structures per Table 3-2 of ASCE 43-05 (Ref. 2.2.6)). Table 3-4 of Ref. 2.2.6 gives the maximum response level for damping for Limit State D as Response Level 2, which, according to Table 3-2 of Ref. 2.2.6, would equate to 7% structural damping for the CRCF BDBGM structural analysis. Section 7.1.5 of Ref. 2.2.47 states that the soil damping for soil structure interaction mode shapes is 20%. Using the BDBGM seismic analysis results of Ref. 2.2.5 with 10% structural damping for the Limit State D evaluation, rather that 7%, will result in an insignificant underestimation of the structural response under the BDBGM seismic loads because the energy dissipation of the soil-structure system in Ref. 2.2.5 is dominated by the soil damping, rather than the structural damping.

Based on the results give in Table 6.8.1, the minimum HCLPF capacity for Limit State D is 0.88g.

Table 6.8.1 Minimum HCLPF Capacities – Limit State A & D

<table>
<tr><th colspan="2" rowspan="2">Failure Mechanism</th><th rowspan="2">Calculation Section</th><th colspan="2">Limit State A</th><th rowspan="2">Limit State D HCLPF</th></tr>
<tr><th>HCLPF</th><th>Fμ</th></tr>
<tr><td colspan="2">In-Plane Shear of Shear Walls*</td><td>6.2.6.2</td><td>1.75g</td><td>1.75</td><td>1.00g</td></tr>
<tr><td rowspan="4">Diaphragms</td><td>O-O-P Bending</td><td rowspan="4">6.3.3</td><td>3.35g</td><td>1.31</td><td>2.56g</td></tr>
<tr><td>O-O-P Shear</td><td>2.04g</td><td>1.00</td><td>2.04g</td></tr>
<tr><td>I-P Bending</td><td>2.01g</td><td>2.25</td><td>0.89g</td></tr>
<tr><td>I-P Shear</td><td>2.30g</td><td>2.00</td><td>1.15g</td></tr>
<tr><td rowspan="4">Structural Steel</td><td>Bending (Beam & Girders)</td><td rowspan="4">6.4.6</td><td>1.82g</td><td>2.06</td><td>0.88g</td></tr>
<tr><td>Shear (Beam & Girders)</td><td>1.87g</td><td>1.00</td><td>1.87</td></tr>
<tr><td>Columns</td><td>2.08g</td><td>1.00</td><td>2.08g</td></tr>
<tr><td>Trusses</td><td>2.40g</td><td>2.06</td><td>1.17g</td></tr>
<tr><td colspan="2">Axial Force in Combination with In-Plane Bending of Walls</td><td>6.5.1</td><td colspan="3">See Section 6.5.1 for Limit State D Results**</td></tr>
<tr><td colspan="2">O-O-P Bending of Walls</td><td>6.6.8</td><td>2.16g</td><td>2.25</td><td>0.96g</td></tr>
</table>

* The Limit State D HCLPF for in-plane shear of the shear walls is based on HCLPF capacity of the individual wall elements and not the HCLPF capacity considering redistribution within a given wall.

** The HCLPF Capacity Evaluation for Axial Force in Combination with In-Plane Bending of Walls was performed by factoring the BDBGM forces by a ratio of the minimum HCLPF capacity to the Limit State A Fμ factor for the shear walls. This ratio is the same if the Limit State D HCLPF capacity and the Limit State D Fμ factor were used to factor the BDBGM forces. Therefore, the results for this evaluation are the same for Limit State A and Limit State D.

6.9 CRCF MEAN FRAGILITY CURVES

The HCLPF capacity of the CRCF is 1.82g for Limit State A and 0.88g for Limit State D. Based on equation 4-4 and equation 4-5 in Section 4.3 of this calculation and considering a composite logarithmic standard deviation, β, of 0.3, the mean fragility curve data for the Limit State A and Limit State D are given in Table 6.9.1 and Table 6.9.2, respectively. Figure 6.9.1 and Figure 6.9.2 show the Limit State A and Limit State D mean fragility curves, respectively. The mean fragility curves are in terms of the peak BDBGM horizontal ground acceleration.

The mean fragility curve given in Table 6.9.1 and Figure 6.9.1 should be used when the CRCF structure is just short of collapse (i.e. excessive cracking in the shear walls is permitted). The mean fragility curve given in Table 6.9.2 and Figure 6.9.2 should be used when essentially elastic building response is required as discussed in Section 1 of this calculation.

Figure 6.9.1 Mean Fragility Curve for CRCF at Limit State A
HCLPF = 1.82g, β = 0.3

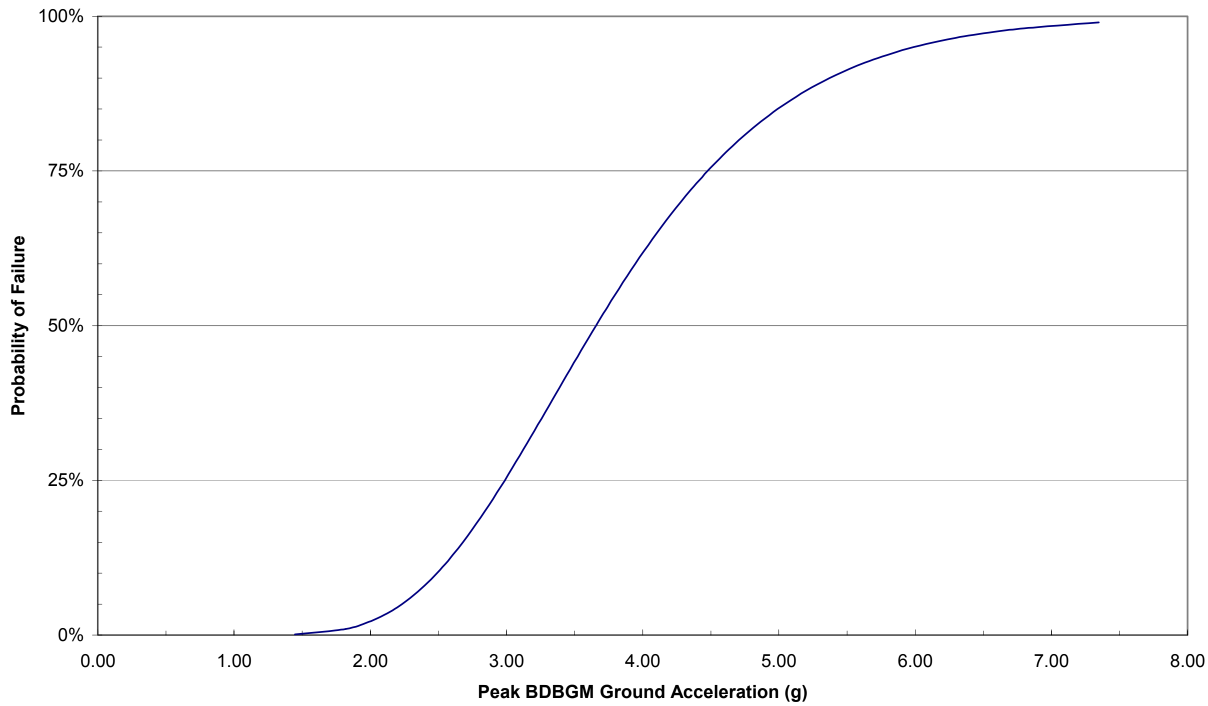


Table 6.9.1 Mean Fragility Curve for CRCF at Limit State A - Digitized Values (HCLPF = 1.82g, β = 0.3)

Probabiltiy of Failure	Peak Horizontal Ground Acceleration	Probabiltiy of Failure	Peak Horizontal Ground Acceleration
0.10%	1.45	50%	3.66
1%	1.82	51%	3.68
2%	1.97	52%	3.71
3%	2.08	53%	3.74
4%	2.16	54%	3.77
5%	2.23	55%	3.80
6%	2.29	56%	3.83
7%	2.35	57%	3.86
8%	2.40	58%	3.89
9%	2.45	59%	3.92
10%	2.49	60%	3.95
11%	2.53	61%	3.98
12%	2.57	62%	4.01
13%	2.61	63%	4.04
14%	2.64	64%	4.07
15%	2.68	65%	4.11
16%	2.71	66%	4.14
17%	2.75	67%	4.17
18%	2.78	68%	4.21
19%	2.81	69%	4.24
20%	2.84	70%	4.28
21%	2.87	71%	4.32
22%	2.90	72%	4.36
23%	2.93	73%	4.40
24%	2.96	74%	4.44
25%	2.99	75%	4.48
26%	3.02	76%	4.52
27%	3.04	77%	4.56
28%	3.07	78%	4.61
29%	3.10	79%	4.66
30%	3.12	80%	4.71
31%	3.15	81%	4.76
32%	3.18	82%	4.81
33%	3.20	83%	4.87
34%	3.23	84%	4.93
35%	3.26	85%	4.99
36%	3.28	86%	5.06
37%	3.31	87%	5.13
38%	3.34	88%	5.20
39%	3.36	89%	5.28
40%	3.39	90%	5.37
41%	3.42	91%	5.47
42%	3.44	92%	5.57
43%	3.47	93%	5.69
44%	3.50	94%	5.83
45%	3.52	95%	5.99
46%	3.55	96%	6.18
47%	3.58	97%	6.43
48%	3.60	98%	6.77
49%	3.63	99%	7.35

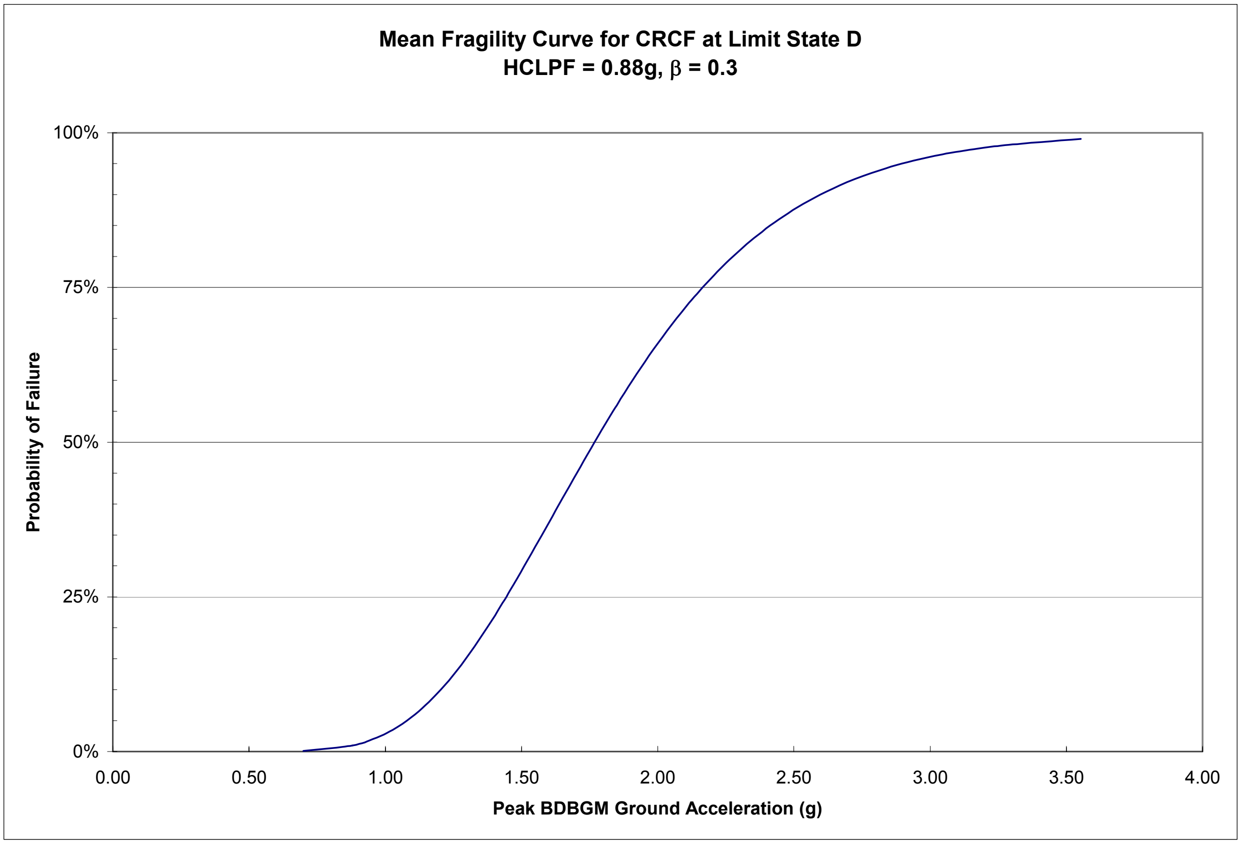

Mean Fragility Curve for CRCF at Limit State D
HCLPF = 0.88g, β = 0.3
100%
75%
50%
25%
0%
Probability of Failure
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
Peak BDBGM Ground Acceleration (g)

Table 6.9.2 Mean Fragility Curve for CRCF at Limit State D - Digitized Values (HCLPF = 0.88g, β = 0.3)

Probabiltiy of Failure	Peak Horizontal Ground Acceleration	Probabiltiy of Failure	Peak Horizontal Ground Acceleration
0.10%	0.70	50%	1.77
1%	0.88	51%	1.78
2%	0.95	52%	1.80
3%	1.01	53%	1.81
4%	1.05	54%	1.82
5%	1.08	55%	1.84
6%	1.11	56%	1.85
7%	1.14	57%	1.86
8%	1.16	58%	1.88
9%	1.18	59%	1.89
10%	1.20	60%	1.91
11%	1.22	61%	1.92
12%	1.24	62%	1.94
13%	1.26	63%	1.95
14%	1.28	64%	1.97
15%	1.30	65%	1.98
16%	1.31	66%	2.00
17%	1.33	67%	2.02
18%	1.34	68%	2.03
19%	1.36	69%	2.05
20%	1.37	70%	2.07
21%	1.39	71%	2.09
22%	1.40	72%	2.11
23%	1.42	73%	2.13
24%	1.43	74%	2.14
25%	1.44	75%	2.16
26%	1.46	76%	2.19
27%	1.47	77%	2.21
28%	1.48	78%	2.23
29%	1.50	79%	2.25
30%	1.51	80%	2.28
31%	1.52	81%	2.30
32%	1.54	82%	2.33
33%	1.55	83%	2.35
34%	1.56	84%	2.38
35%	1.58	85%	2.41
36%	1.59	86%	2.45
37%	1.60	87%	2.48
38%	1.61	88%	2.52
39%	1.63	89%	2.55
40%	1.64	90%	2.60
41%	1.65	91%	2.64
42%	1.66	92%	2.70
43%	1.68	93%	2.75
44%	1.69	94%	2.82
45%	1.70	95%	2.90
46%	1.72	96%	2.99
47%	1.73	97%	3.11
48%	1.74	98%	3.27
49%	1.75	99%	3.55

7 RESULTS AND CONCLUSIONS

7.1 RESULTS

Figure 6.9.1 and Table 6.9.1 give the mean fragility curve data for the CRCF that should be used when the CRCF structure is just short of collapse (i.e. excessive cracking in the shear walls is permitted). This condition corresponds to Limit State A as defined in ASCE 43-05 (Ref. 2.2.6).

Figure 6.9.2 and Table 6.9.2 give the mean fragility curve data for the CRCF that should be used when essentially elastic building response is required (i.e. cracking in the shear walls is limited). This condition corresponds to Limit State D as defined in ASCE 43-05 (Ref. 2.2.6).

The following changes to the CRCF structural layout shown in Ref. 2.2.7 to Ref. 2.2.26 are required if the mean seismic fragility curves described above are to be used in a probabilistic seismic risk assessment of the CRCF.

18" roof slabs at EL. 32' between col. line 12-13/E-G and between col. line 1-2/E-G

- Chord Reinforcement
  Provide 6 - #9 bars for N-S seismic excitation chord reinforcement
  Provide 22 - #9 bars for E-W seismic excitation chord reinforcement

- Slab Reinforcement
  Provide #7 @ 9" on centers, both ways, top and bottom

Structural Steel Columns

MARK	Size given in Ref. 2.2.38	Size Required by HCLPF Evaluation	Grid Location
C1	W14x257	W14x398	D.3-2.5, D.7-2.5, G.3-2.5, G.7-2.5
C2	W14x145	W14x233	D.3-6.8, D.7-6.8, G.3-6.8, G.7-6.8
C3	W14x233	W14x370	D.3-10, D.7-10, G.3-10, G.7-10
C4	W14x311	W14x426	E.3-10, F-10, F.7-10
C5	W14x176	W14x257	D.3-11, D.7-11, G.3-11, G.7-11

Reinforcing around openings at EL. 0’-0”
For openings in the structural shear walls at EL. 0-0”, Figure 6.5.6 shows the detail required to meet the HCLPF capacity requirements for in-plane bending and axial force on the walls of the CRCF.

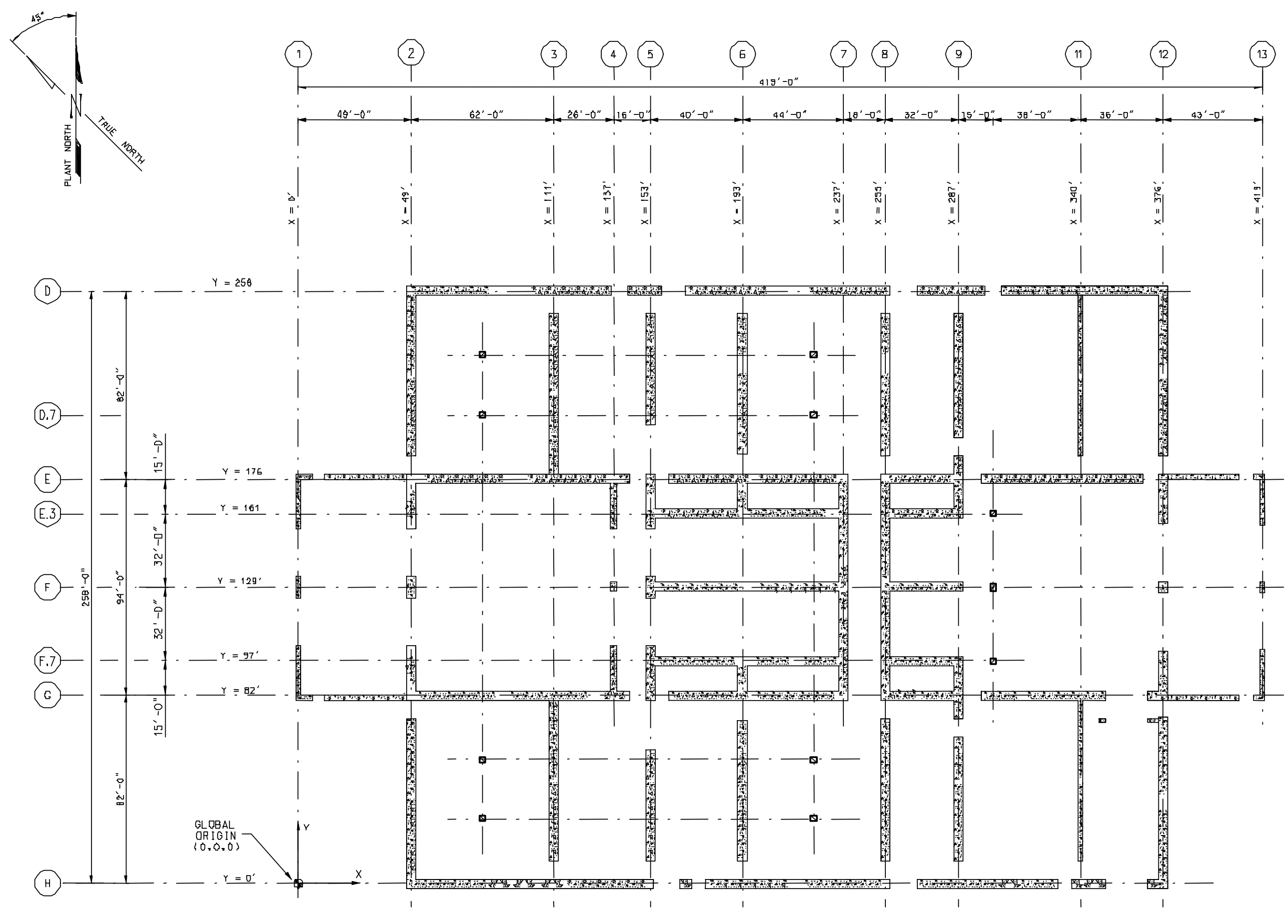
7.2 CONCLUSIONS

Based on the results of this calculation, the mean seismic fragility curves of the CRCF are as discussed in Section 7.1. These seismic fragility curves are to be used in a seismic probabilistic risk assessment of the CRCF.

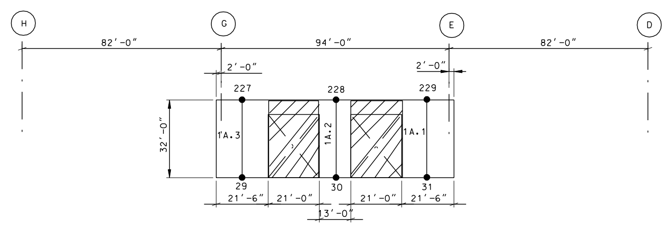
ATTACHMENT A: FLOOR PLAN AND WALL ELEVATIONS

Page No.

Ground Floor Plan at EL. 0’-0” A-2
Elevation Along Column Line 1 ..... A-3
Elevation Along Column Line 2 ..... A-4
Elevation Along Column Line 3 ..... A-5
Elevation Along Column Line 4 ..... A-6
Elevation Along Column Line 5 ..... A-7
Elevation Along Column Line 6 ..... A-8
Elevation Along Column Line 7 ..... A-9
Elevation Along Column Line 8 ..... A-10
Elevation Along Column Line 9 ..... A-11
Elevation Along Column Line 11 ..... A-12
Elevation Along Column Line 12 ..... A-13
Elevation Along Column Line 13 ..... A-14
Elevation Along Column Line D A-15
Elevation Along Column Line E. A-16
Elevation Along Column Line E.3. ..... A-17
Elevation Along Column Line F. A-18
Elevation Along Column Line F.7. ..... A-19
Elevation Along Column Line G. A-20
Elevation Along Column Line H. A-21

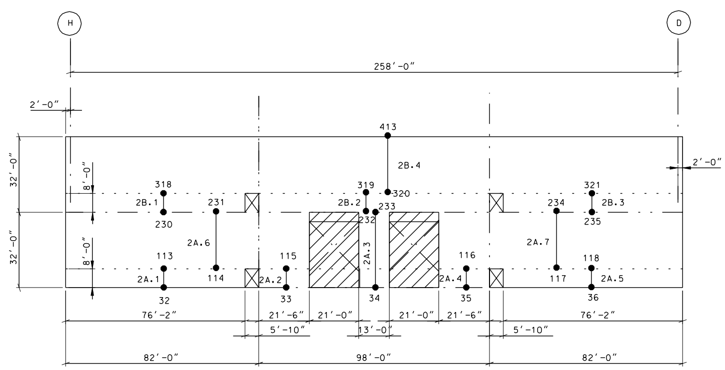


GROUND FLOOR PLAN AT EL 0'-0"
(All Dimensions +/- 6")



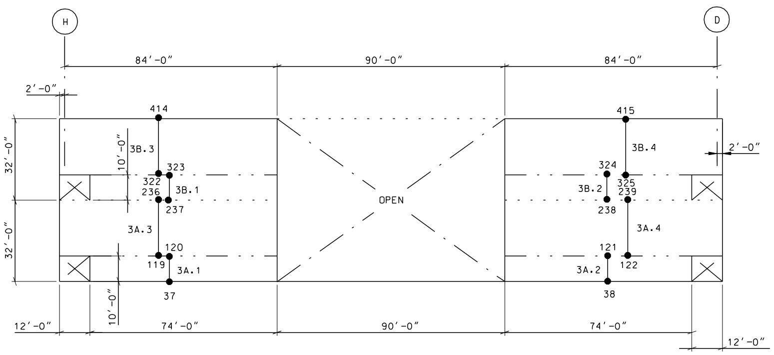
ELEVATION ALONG COLUMN LINE 1
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



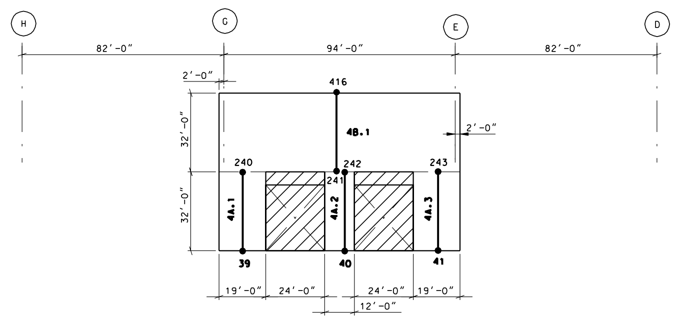
ELEVATION ALONG COLUMN LINE 2
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



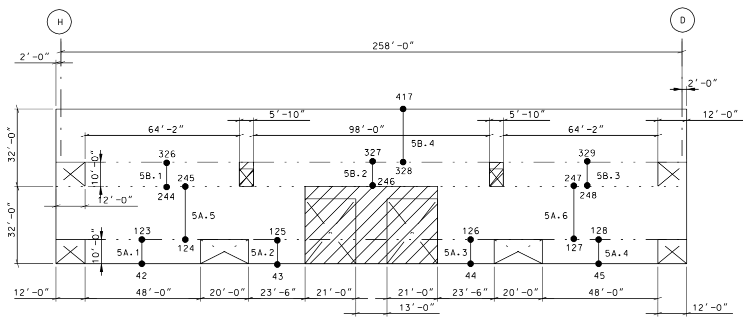
ELEVATION ALONG COLUMN LINE 3
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



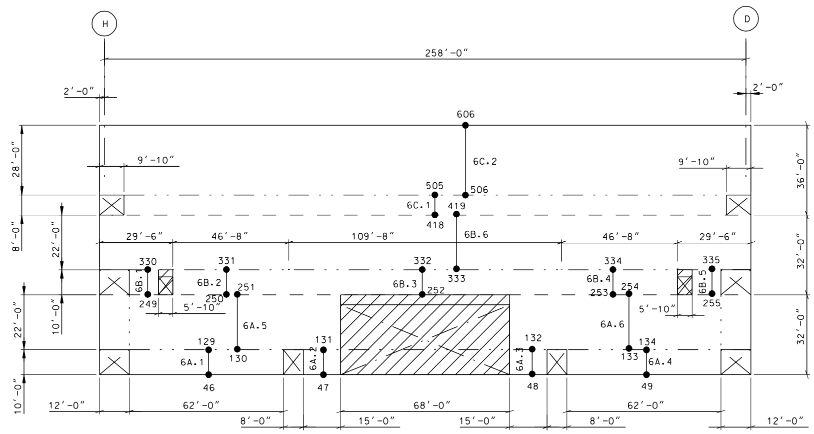
ELEVATION ALONG COLUMN LINE 4
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



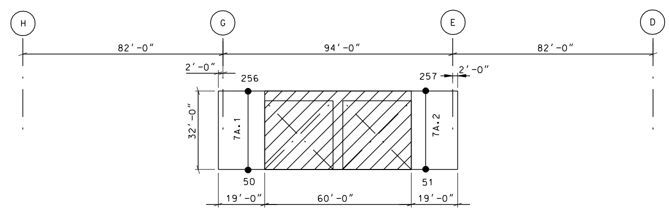
ELEVATION ALONG COLUMN LINE 5
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



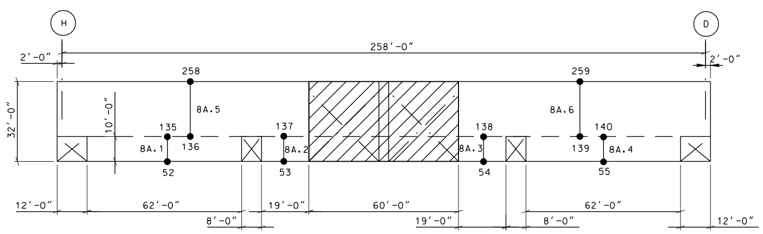
ELEVATION ALONG COLUMN LINE 6
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



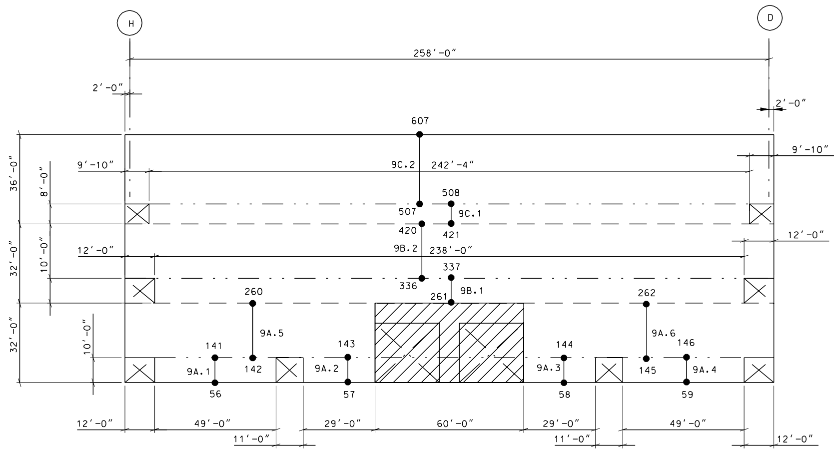
ELEVATION ALONG COLUMN LINE 7
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



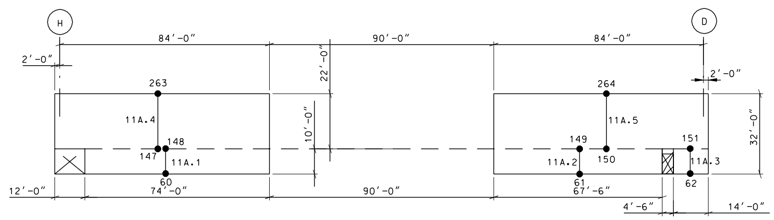
ELEVATION ALONG COLUMN LINE 8
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



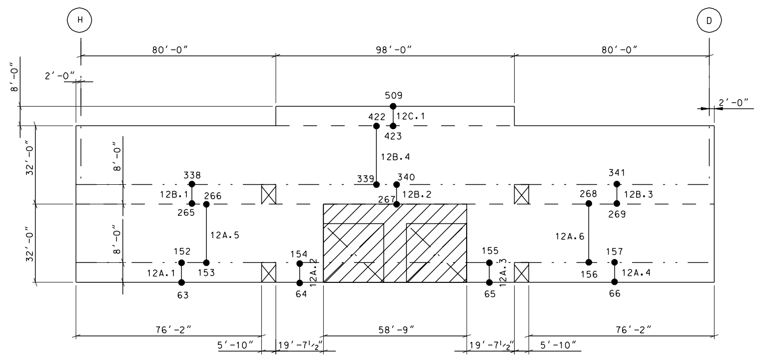
ELEVATION ALONG COLUMN LINE 9
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



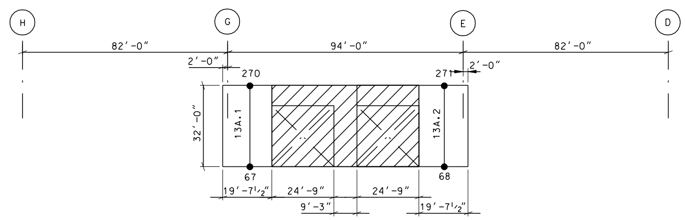
ELEVATION ALONG COLUMN LINE 11
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")



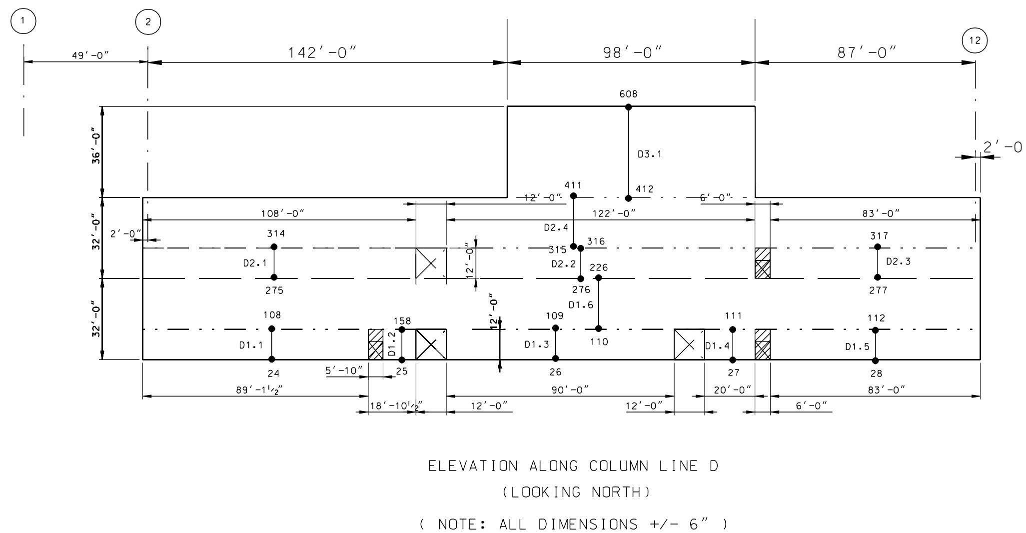
ELEVATION ALONG COLUMN LINE 12
(LOOKING WEST)

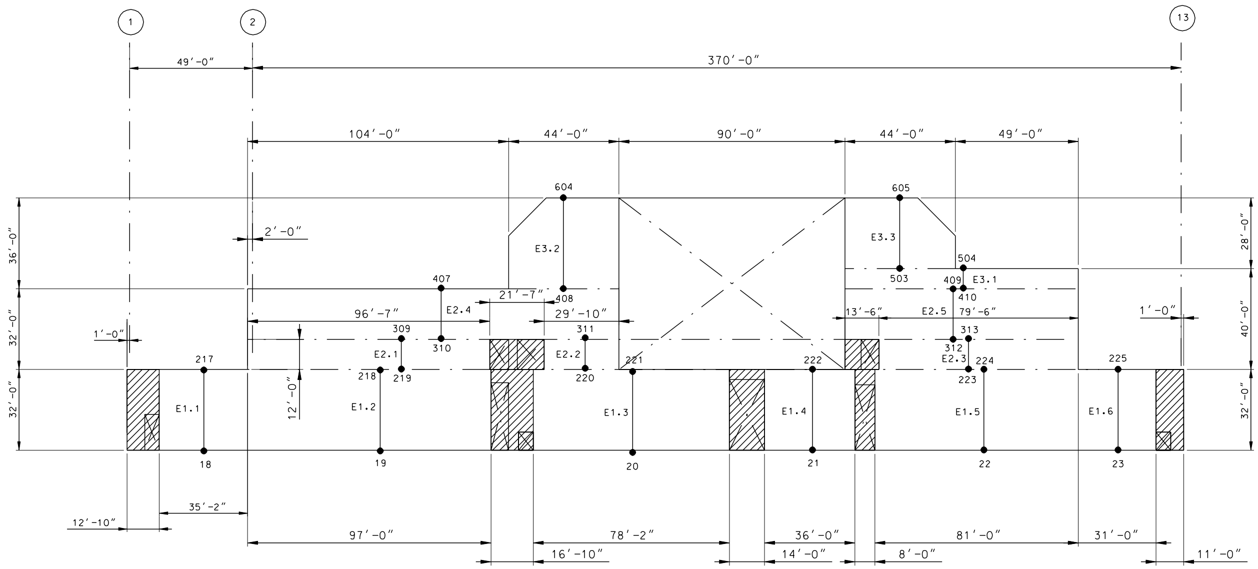
(NOTE: ALL DIMENSIONS +/- 6")



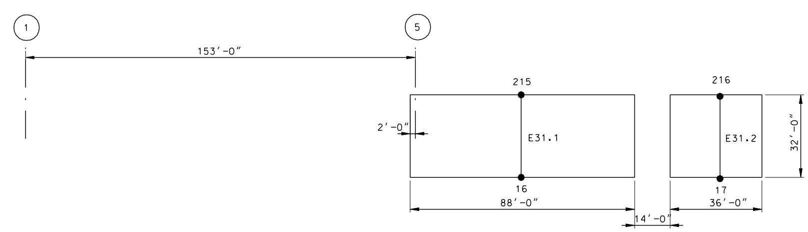
ELEVATION ALONG COLUMN LINE 13
(LOOKING WEST)

(NOTE: ALL DIMENSIONS +/- 6")


ELEVATION ALONG COLUMN LINE D
(LOOKING NORTH)
(NOTE: ALL DIMENSIONS +/- 6")

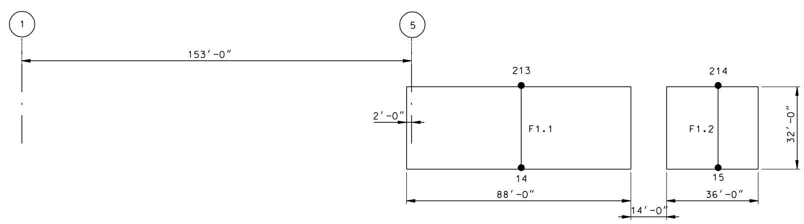


ELEVATION ALONG COLUMN LINE E
(LOOKING NORTH)
(NOTE: ALL DIMENSIONS +/- 6")



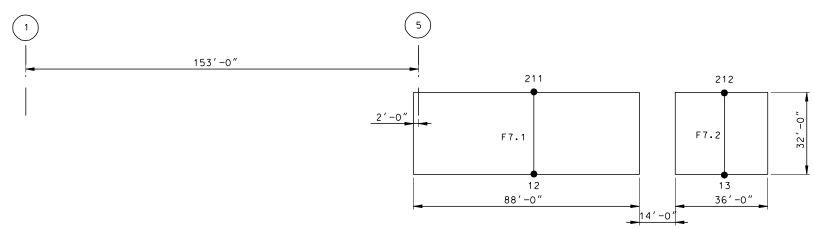
ELEVATION ALONG COLUMN LINE E.3
(LOOKING NORTH)

(NOTE: ALL DIMENSIONS +/- 6")



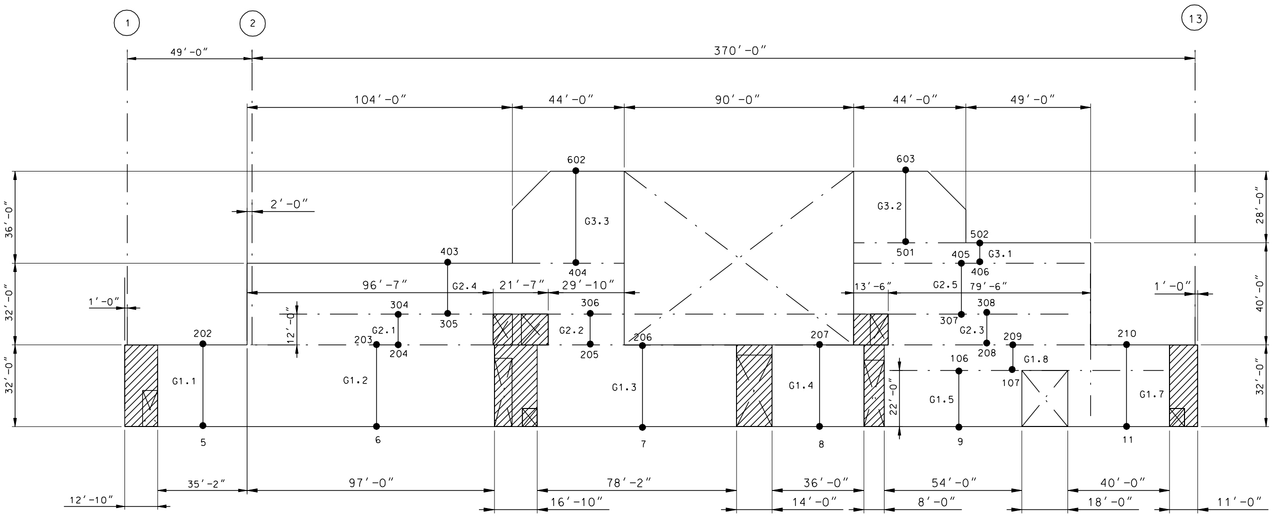
ELEVATION ALONG COLUMN LINE F
(LOOKING NORTH)

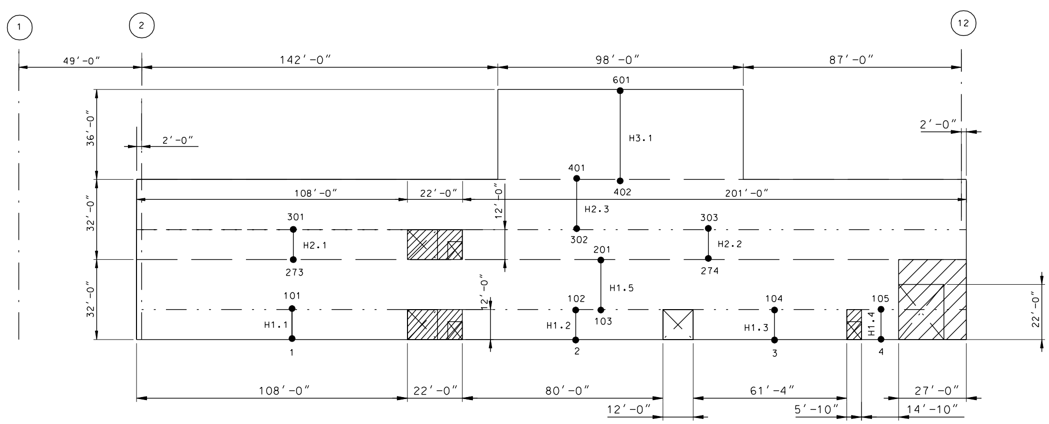
(NOTE: ALL DIMENSIONS +/- 6")



ELEVATION ALONG COLUMN LINE F.7
(LOOKING NORTH)

(NOTE: ALL DIMENSIONS +/- 6")


ELEVATION ALONG COLUMN LINE G
(LOOKING NORTH)
(NOTE: ALL DIMENSIONS +/- 6")



ELEVATION ALONG COLUMN LINE H
(LOOKING NORTH)

(NOTE: ALL DIMENSIONS +/- 6")

ATTACHMENT B: SHEAR WALL DESIGN SUMMARY TABLE
(Source: Ref. 2.2.29 Table 7.1.1)

SHEAR WALL DESIGN SUMMARY							
WALL Line	**Segment**	**Horizontal Reinforcement**	**Vertical Reinforcement**	**Shear on Gross Section: D/C (MAX)**	**In-Plane Shear: D/C (MAX) (Horizontal Reinforcing)**	**Out-of-Plane Shear: D/C (MAX)**	**Bending + axial Loads D/C (MAX) (Vertical Reinforcing)**
1	1A.1						
	1A.2	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.34**	**0.23**	**0.12**	**0.49**
	1A.3						
2	2A.1						
	2A.2						
	2A.3						
	2A.4						
	2A.5	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.59**	**0.66**	**0.14**	**0.68**
	2A.6						
	2A.7						
	2B.1						
	2B.2						
	2B.3						
	2B.4						
3	3A.1						
	3A.2						
	3A.3						
	3A.4	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.58**	**0.64**	**0.14**	**0.43**
	3B.1						
	3B.2						
	3B.3						
	3B.4						
4	4A.1						
	4A.2	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.27**	**0.3**	**0.14**	**0.68**
	4A.3						

SHEAR WALL DESIGN SUMMARY							
WALL Line	**Segment**	**Horizontal Reinforcement**	**Vertical Reinforcement**	**Shear on Gross Section: D/C (MAX)**	**In-Plane Shear: D/C (MAX) (Horizontal Reinforcing)**	**Out-of-Plane Shear: D/C (MAX)**	**Bending + axial Loads D/C (MAX) (Vertical Reinforcing)**
	4B.1						
5	5A.1						
	5A.2						
	5A.3						
	5A.4						
	5A.5	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.66**	**0.74**	**0.14**	**0.49**
	5A.6						
	5B.1						
	5B.2						
	5B.3						
	5B.4						
6	6A.1						
	6A.2						
	6A.3						
	6A.4						
	6A.5						
	6A.6						
	6B.1	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.59**	**0.65**	**0.25**	**0.6**
	6B.2						
	6B.3						
	6B.4						
	6B.5						
	6B.6						
	6C.1						
	6C.2						
7	7A.1	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.26**	**0.29**	**0.11**	**0.66**
	7A.2						
8	8A.1						
	8A.2						
	8A.3	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.61**	**0.68**	**0.11**	**0.54**
	8A.4						
	8A.5						
	8A.6						

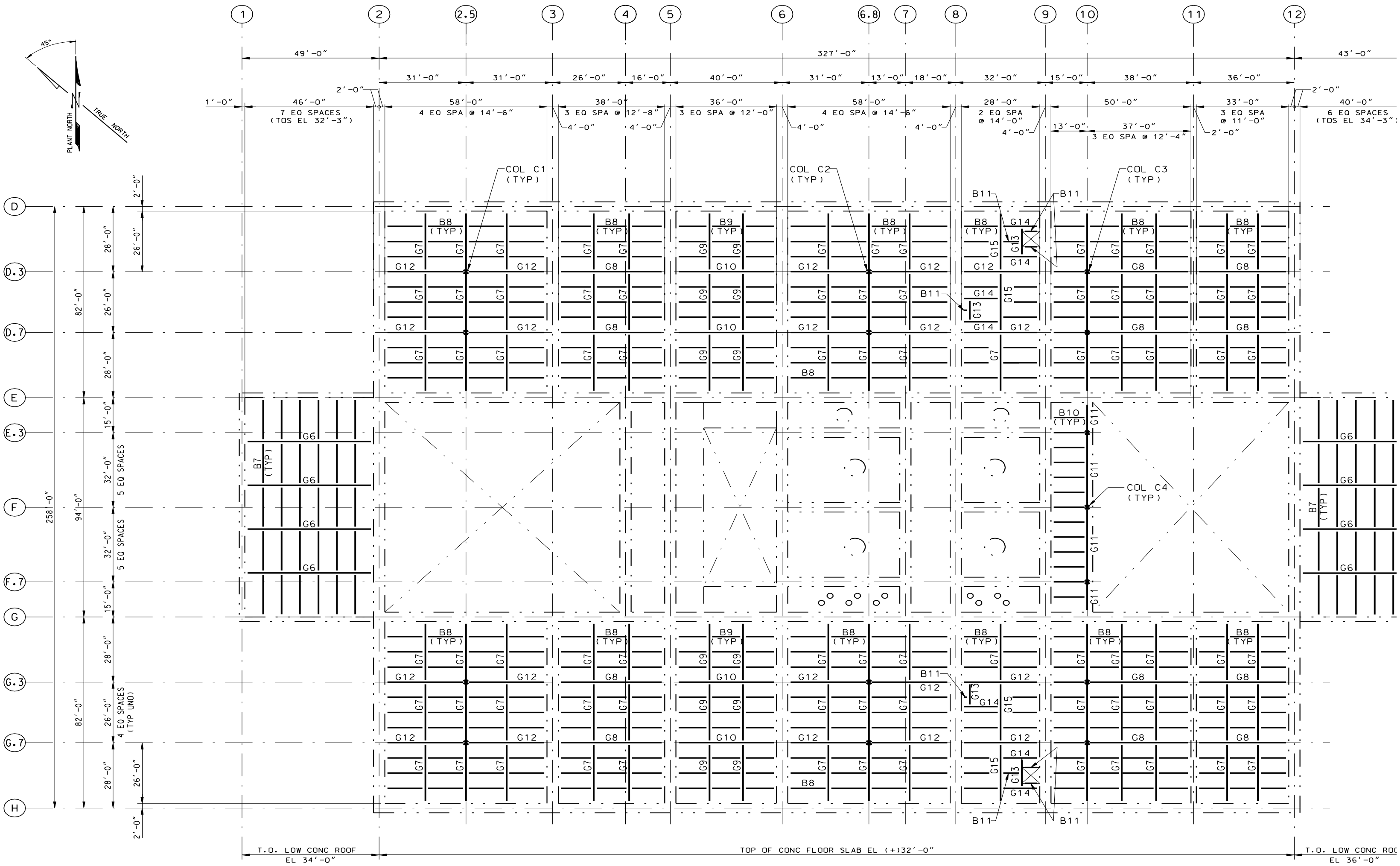
SHEAR WALL DESIGN SUMMARY							
WALL Line	Segment	Horizontal Reinforcement	Vertical Reinforcement	Shear on Gross Section: D/C (MAX)	In-Plane Shear: D/C (MAX) (Horizontal Reinforcing)	Out-of-Plane Shear: D/C (MAX)	Bending + axial Loads D/C (MAX) (Vertical Reinforcing)
9	9A.1						
	9A.2						
	9A.3						
	9A.4						
	9A.5	1 # 11@ 12" E.F	1 # 11@ 12" E.F				
	9A.6			**0.65**	**0.72**	**0.25**	**0.54**
	9B.1						
	9B.2						
	9C.1						
	9C.2						
11	11A.1						
	11A.2						
	11A.3	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.63**	**0.42**	**0.12**	**0.33**
	11A.4						
	11A.5						
12	12A.1						
	12A.2						
	12A.3	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.64**	**0.72**	**0.17**	**0.5**
	12A.4						
	12A.5						
	12A.6						
	12B.1						
	12B.2						
	12B.3						
	12B.4						
	12C.1						
13	13A.1	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.34**	**0.23**	**0.12**	**0.52**
	13A.2						
D	D1.1	1 # 11@ 12" E.F (Typical U.N.O)	1 # 11@ 12" E.F (Typical U.N.O)	**0.57**	**0.63**	**0.16**	**0.64**
	D1.2						

SHEAR WALL DESIGN SUMMARY							
WALL Line	**Segment**	**Horizontal Reinforcement**	**Vertical Reinforcement**	**Shear on Gross Section: D/C (MAX)**	**In-Plane Shear: D/C (MAX) (Horizontal Reinforcing)**	**Out-of-Plane Shear: D/C (MAX)**	**Bending + axial Loads D/C (MAX) (Vertical Reinforcing)**
	D1.3						
	D1.4						
	D1.5						
	D1.6	1 # 11@ 9" E.F	1 # 11@ 9" E.F	**Between col. lines 6 and 9 only**			
	D2.1						
	D2.2	1 # 11@ 9" E.F	1 # 11@ 9" E.F				
	D2.3						
	D2.4	1 # 11@ 9" E.F	1 # 11@ 9" E.F	**0.45**	**0.41**	**0.34**	**0.83**
	D3.1	1 # 11@ 9" E.F	1 # 11@ 9" E.F				
E	E1.1						
	E1.2						
	E1.3						
	E1.4						
	E1.5						
	E1.6	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.47**	**0.52**	**0.27**	**0.64**
	E2.1						
	E2.2						
	E2.3						
	E2.4						
	E2.5						
	E3.1						
	E3.2						
	E3.3						
E.3	E31.1	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.46**	**0.51**	**0.21**	**0.53**
	E31.2						
F	F1.1	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.45**	**0.5**	**0.11**	**0.52**
	F1.2						
F.7	F7.1	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.46**	**0.51**	**0.11**	**0.53**
	F7.2						

SHEAR WALL DESIGN SUMMARY							
WALL Line	**Segment**	**Horizontal Reinforcement**	**Vertical Reinforcement**	**Shear on Gross Section: D/C (MAX)**	**In-Plane Shear: D/C (MAX) (Horizontal Reinforcing)**	**Out-of-Plane Shear: D/C (MAX)**	**Bending + axial Loads D/C (MAX) (Vertical Reinforcing)**
G	G1.1						
	G1.2						
	G1.3						
	G1.4						
	G1.5						
	G1.7						
	G1.8						
	G2.1	1 # 11@ 12" E.F	1 # 11@ 12" E.F	**0.49**	**0.59**	**0.29**	**0.77**
	G2.2						
	G2.3						
	G2.4						
	G2.5						
	G3.1						
	G3.2						
	G3.3						
H	H1.1	1 # 11@ 12" E.F (Typical U.N.O)	1 # 11@ 12" E.F (Typical U.N.O)	**0.59**	**0.66**	**0.16**	**0.66**
	H1.2						
	H1.3						
	H1.4						
	H1.5	1 # 11@ 9" E.F	1 # 11@ 9" E.F	**Between col. Lines 6 and 9 only**			
	H2.1						
	H2.2	1 # 11@ 9" E.F	1 # 11@ 9" E.F				
	H2.3	1 # 11@ 9" E.F	1 # 11@ 9" E.F	**0.45**	**0.41**	**0.34**	**0.83**
	H3.1	1 # 11@ 9" E.F	1 # 11@ 9" E.F				

Attachment G: Structural Steel Framing Schematics

Figure G.1 Structural Steel Beam and Girder Cases at EL. 32'-0"



SECOND FLOOR FRAMING PLAN AT TOS EL (+)30'-3 " (TYP UNO)
AND LOW ROOF FRAMING AT TOS EL (+)32'-3" & (+)34'-3"

Figure G.2 Structural Steel Beam and Girder Cases at EL. 64'-0"

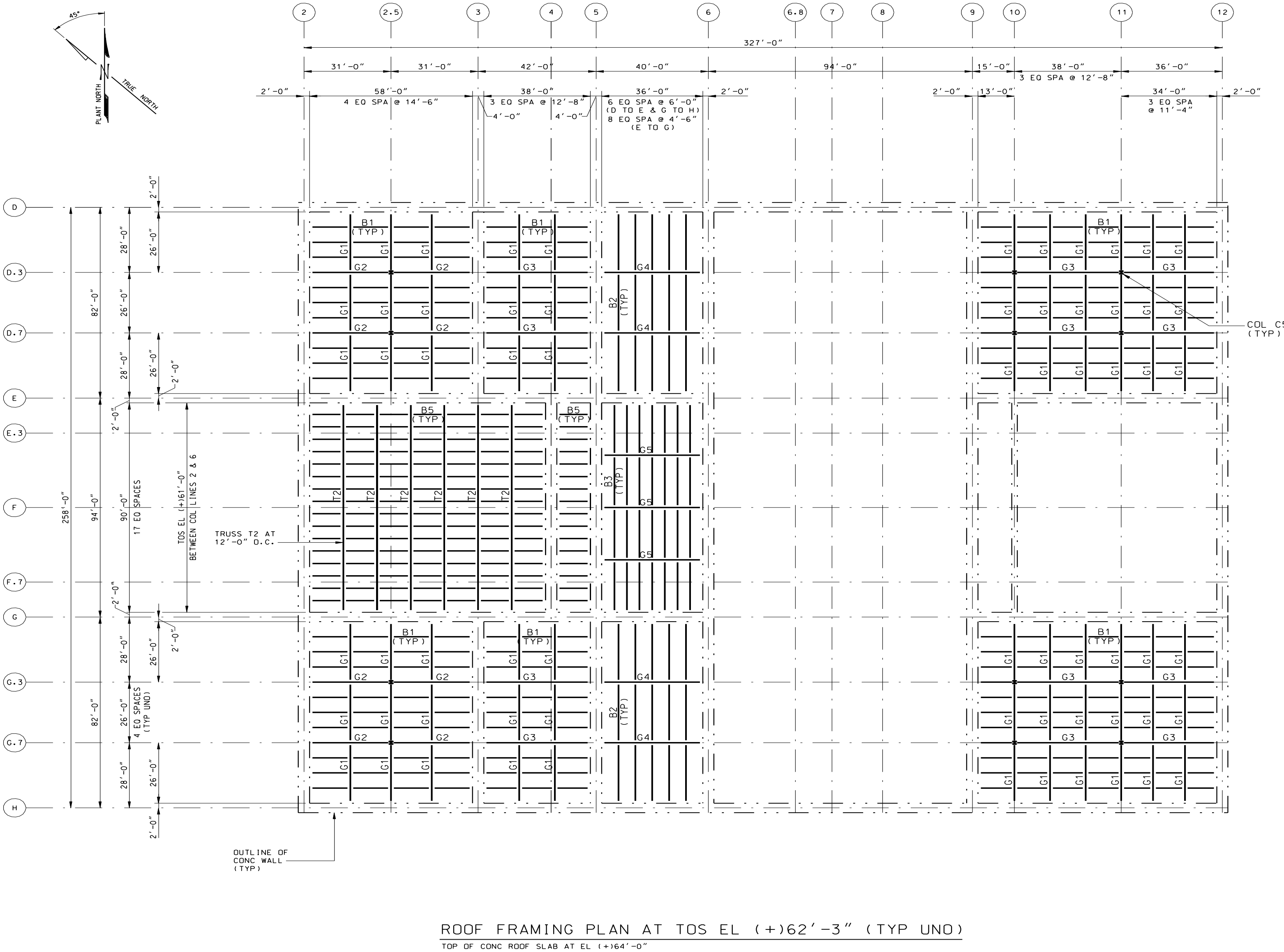
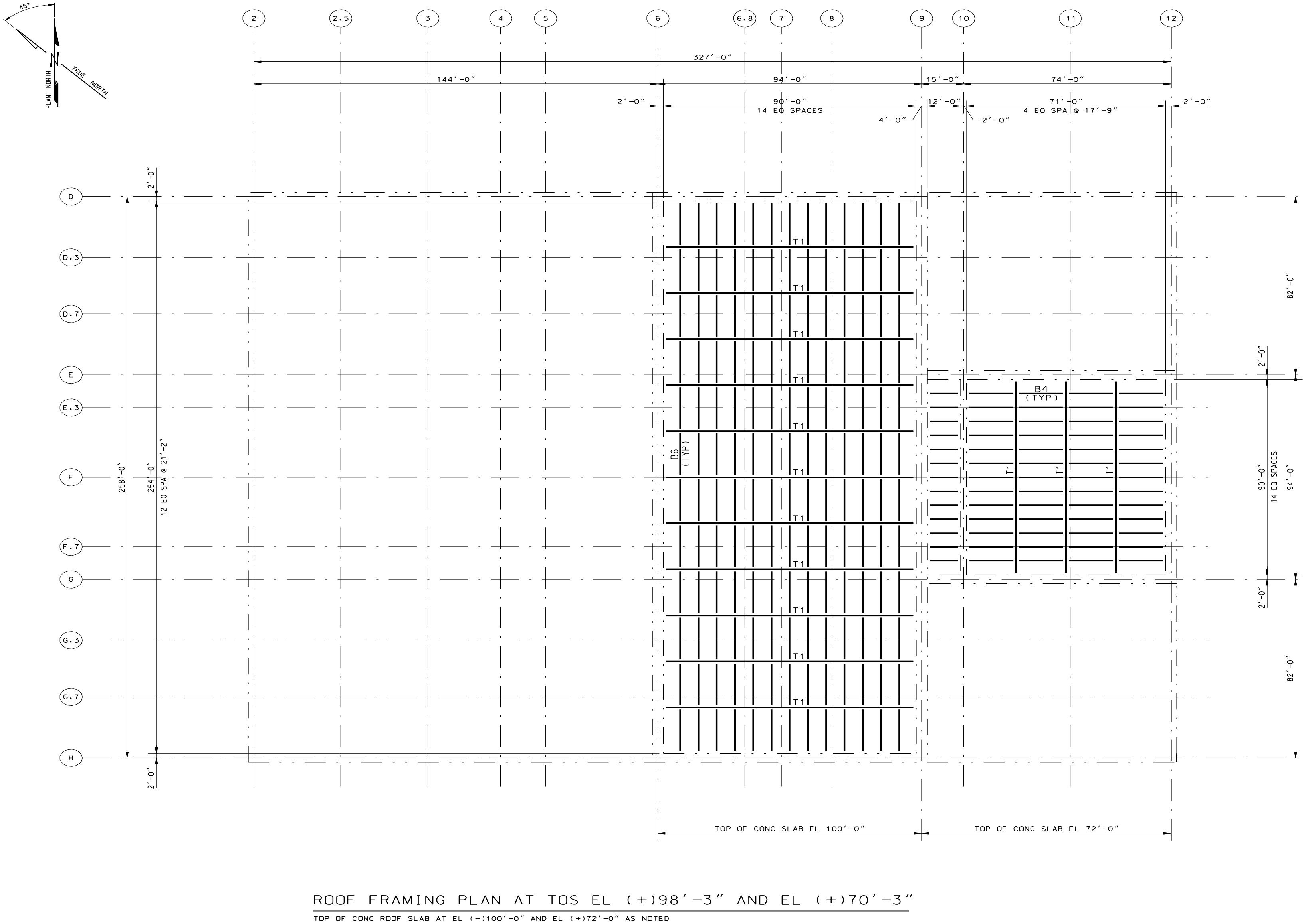
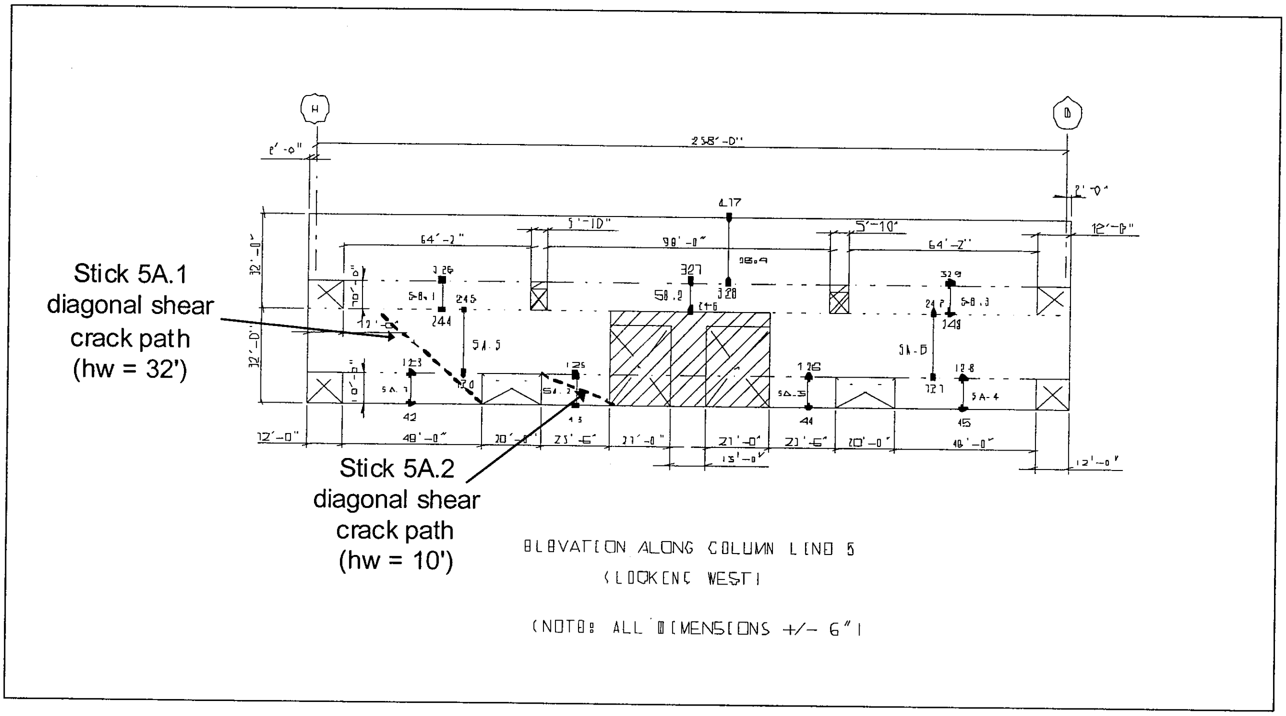


Figure G.3 Structural Steel Beam and Girder Cases at EL. 72'-0" and EL. 100'-0"



Attachment H: Establishment of hw for In-Plane Shear HCLPF Capacity Evaluations

In order to calculate conservative in-plane shear HCLPF capacities, the wall segment height, hw, for the CRCF stick elements is revised from the hw values given in Ref. 2.2.29. In this calculation, the value of hw is determined by estimating the path of the diagonal shear crack that will develop in the wall segment under in-plane loading. The value of hw is set equal to either the height of the opening on either side of a wall segment or the height of the wall between diaphragms. The former hw value is used for short wall piers with openings on both sides of the pier, while the latter is used for wall segments without openings or long wall piers. The following sketch illustrates typical hw calculations for the conditions described above.



BSC	**Calculation/Analysis Change Notice** *Complete only applicable items.*	1. QA: QA 2. Page 1 of 1

3. Document Identifier: 060-SYC-CR00-01100-000	4. Rev.: 00A	5. CACN: 001
6. Title: Canister Receipt and Closure Facility (CRCF) Seismic Fragility Evaluation		

7. Reason for Change:

The design response spectra shown in Reference 2.2.31 has been qualified with a caveat that indicates points with a period of 3.33 second and above are plotted incorrectly. It has been determined that the highest period that can be qualified is at 2 seconds. This caveat limits the data in ~~DTNs~~ DTN MO0706DSDR1E4A.001.

3/1/2008

8. Supersedes Change Notice:	☐ Yes If, Yes, CACN No.: ____	☒ No

9. Change Impact:

Inputs Changed:	☒ Yes	☐ No	Results Impacted:	☐ Yes	☒ No
Assumptions Changed:	☐ Yes	☒ No	Design Impacted:	☐ Yes	☒ No

10. Description of Change:

Add to the end of Section 7.2 of the calculation:

MO0706DSDR1E4A.001 (Ref. 2.2.31) has been qualified with a caveat that deletes results for SSCs with frequencies below 0.5 hertz (above 2 second period).

The BDBGM seismic analysis results shown in Attachment C are taken from Ref 2.2.5 that has independently been verified as not being affected by responses below 0.5 hertz. Therefore it can be concluded that this calculation is not affected by omitting all responses above 2 seconds.

11. REVIEWS AND APPROVAL

Printed Name	Signature	Date
11a. Originator: Surendra K. Goel	S K Goel	3/1/2008
11b. Checker: T. K. McEwan	T K McEwan	3/1/2008
11c. EGS: Thomas Frankert	Thomas Frankert	3/1/2008
11d. DEM: Raj Rajagopal	R Rajagopal	3/1/2008
11e. Design Authority: Barbara Rusinko	B Rusinko	3/1/08