

HCP ENGINEERING

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STRUCTURAL CALCULATIONS

FOR

HOME2 SUITES by HILTON

550 Gateway Blvd
South San Francisco, CA
(Project 2017-06)



FOR

ARRIS STUDIO ARCHITECTS

1306 Johnson Ave
San Luis Obispo, CA 93401
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HOME2 SUITES by HILTON

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South San Francisco, CA
(Project 2017-06)

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COMMERCIAL PROJECT LOADS (psf)

(ASCE 7-10)

Project: _____

Date: _____

By: _____

Page No.: _____

ROOF LOADS:

D.L. ROOFING
SHEATHING
FRAMING
MISC. (incl. F.Spklr / Solar*)

CEILING (2 layers)

L.L. (Reducible)
TOTAL

psf			
BUILT UP w/ Mech.	BUILT UP w/ Solar	CONC. TILES	ASPH. SHINGLES
4.0	4.0	10.0	4.0
3.0	3.0	3.0	2.5
3.5	3.5	3.5	3.5
6.0	16.0	2.5	2.5
16.5	26.5	19.0 x 13/12 = 20.8	12.5 x 13/12 = 13.5
5.5	5.5	5.5	5.5
22.0	32.0	26.0	19.0
20.0	20.0	20.0	20.0
42.0	52.0	46.0	39.0

*Incl. 12.0 psf for Solar

CEILING LOADS (Unhabitable)

D.L. FRAMING 2.0
CEILING 2.2
MISC. 0.8
5.0 psf

L.L. Attic without Storage 10.0 psf
TOTAL 15.0 psf
Attic with Storage 20.0

FLOOR LOADS:

D.L. FLOORING 2.0
SHEATHING 2.5
FRAMING 4.0
CEILING (2 layers) 5.6
MISC. (incl. 1.5psf F.Spklr) 2.4
16.5 psf

L.L. Typical 40.0 psf
Private Balcony 60.0
Public Bacony 100.0
Corridors w/ Public Functions 100.0
Corridors to Private Areas 40.0
Stairs 100.0
Partition 15.0

Other D.L. 1" Gypcrete 9.0 psf
1 1/2" Lt. Wt. Conc. 15.0
Walls: Interior Drywall 10.0
Walls: Exterior Stucco 15.0
Stucco (ceiling) 8.0
Stone veneer (psf/1" thk) 10.0

DESIGN CRITERIA: CBC 2016

CONCRETE: ACI 318-14

F'c = 2,500 psi, u.n.o.

Fy = 40 ksi, #3 & #4

Fy = 60 ksi, #5 to #11

WOOD: NDS (ASD) 2012

Studs, Joist, Plates

No. 2, D.F.

Fb = 900 psi

Beams 4x

No. 2, D.F.

Fb = 900 psi

Beams 6x or larger

No. 1, D.F.

Fb = 1200 psi

GLB

24F-V8 (V10)

STEEL: AISC - 360-10

Bolts

A307, u.n.o.

Fy - Sections

50 ksi

Fy - Plates

36 ksi

USE 7.1.

SEE DCI ENGINEERS
CALCULATIONS FOR
SUPPORT STRUCTURE
DESIGN LOADS

SEISMIC PARAMETERS.

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TABLE 1- 2013 CBC SEISMIC PARAMETERS *

Parameter	Value
Site Class/Soil Profile Type	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Mapped MCE Spectral Acceleration (0.2 sec), S_s	1.940
Mapped MCE Spectral Acceleration (1.0 sec), S_1	0.909
MCE Spectral Acceleration (0.2 sec), S_{MS}	1.940
MCE Spectral Acceleration (1.0 sec), S_{M1}	1.363
Design Spectral Acceleration (0.2 sec), S_{DS}	1.294
Design Spectral Acceleration (0.2 sec), S_{D1}	0.909

* Foundation Geotechnical Report By:
 GEO ENGINEERING CONSULTANTS.
 4125 BLACKHORN AVE #145.
 AMADOR, CA 95117
 P: 925-321-5530
 Report #: P16-122
 DATE: FEBRUARY 12, 2016.

SEISMIC WT: Podium.

$$\begin{aligned}
 \text{Slab: } 15" \text{ THICK: } & 150 (15/12) (19500) & = 3,656,250 \\
 10" \text{ Ponto Co: } & 150 (10/12) (1100) & = 137,500 \\
 \text{Conc. Walls: } & 150 (7) (720) & = 756,000 \\
 \text{Conc. Cols: } & 150 (1.5 \times 1.5 \times 7) (17) & = 46,163 \\
 W_{\text{podium}} = & & = 4,589,913 \\
 W_{\text{updr}} = & & = 2,950,000 \\
 & & \hline
 & & 7,539,913
 \end{aligned}$$

Two Stage Load Amplification Factor (ASCE 12.2.3.2)
 Super Structure: $R=6.5$ $C=1.0$
 Podium: $R=5.0$ $C=1.3$.

$$A = \frac{(6.5/1)}{(5/1.3)} = 1.69$$

Load Factor Applied
To EQX & EQY
Loads

2017-06
3

SEISMIC BASE SHEAR

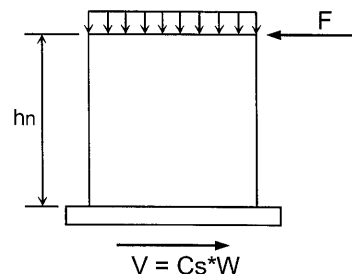
Per CBC 2016 and ASCE 7-10 Specifications

Using Equivalent Lateral Force Procedure for Regular Single-Level Building/Structural Systems

Job Name:	HOME2 - SSF (PODIUM)	Subject:	
Job Number:		Originator:	Checker:

Input Data:

Occupancy Category =	II	ASCE 7-10 Table 1.5-1
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2
Soil Site Class =	D	ASCE 7-10, Chp. 20
Latitude =		
Longitude =		
Spectral Accel., S _s =	1.940	ASCE 7-10 Figures 22-1, 3, 5, 6
Spectral Accel., S ₁ =	0.909	ASCE 7-10 Figures 22-2, 4, 5, 6
Long. Trans. Period, T _L =	8.000	sec. ASCE 7 Fig's. 22-12 to 22-16
Structure Height, h _n =	14.000	ft.
Total Seismic Weight, W =	7540.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, T _c =	0.000	sec. from Analysis or "0" if T _a (below (Regular Bldg. Configurations Only)
Seismic Resist. System =	A1	A1. Special reinforced concrete shear walls (ASCE 7-10, Table 12.2-1)



Results:

Site Coefficients:

F _a =	1.000	ASCE 7-10, Table 11.4-1
F _v =	1.500	ASCE 7-10, Table 11.4-2

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS =	1.940	SMS = F _a *S _s , ASCE 7-10, Eqn 11.4-1
SM1 =	1.364	SM1 = F _v *S ₁ , ASCE 7-10, Eqn 11.4-2

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS =	1.293	SDS = 2*SMS/3, ASCE -10, Eqn 11.4-3
SD1 =	0.909	SD1 = 2*SM1/3, ASCE 7-10 Eqn 11.4-4

Seismic Design Category:

Category(for SDS) =	E	ASCE 7-10, Table 11.6-1
Category(for SD1) =	E	ASCE 7-10, Table 11.6-2
Use Category =	E	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C _T =	0.020	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2
Approx. Period, T _a =	0.145	sec., T _a = C _T *h _n ^x , ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C _u =	1.400	ASCE 7-10 Table 12.8-1
Period max., T _(max) =	0.203	sec., T _(max) = C _u *T _a , ASCE 7-10 Section 12.8.2
Fundamental Period, T =	0.145	sec., T = T _a <= C _u *T _a , ASCE 7-10 Section 12.8.2

Seismic Design Coefficients and Factors:

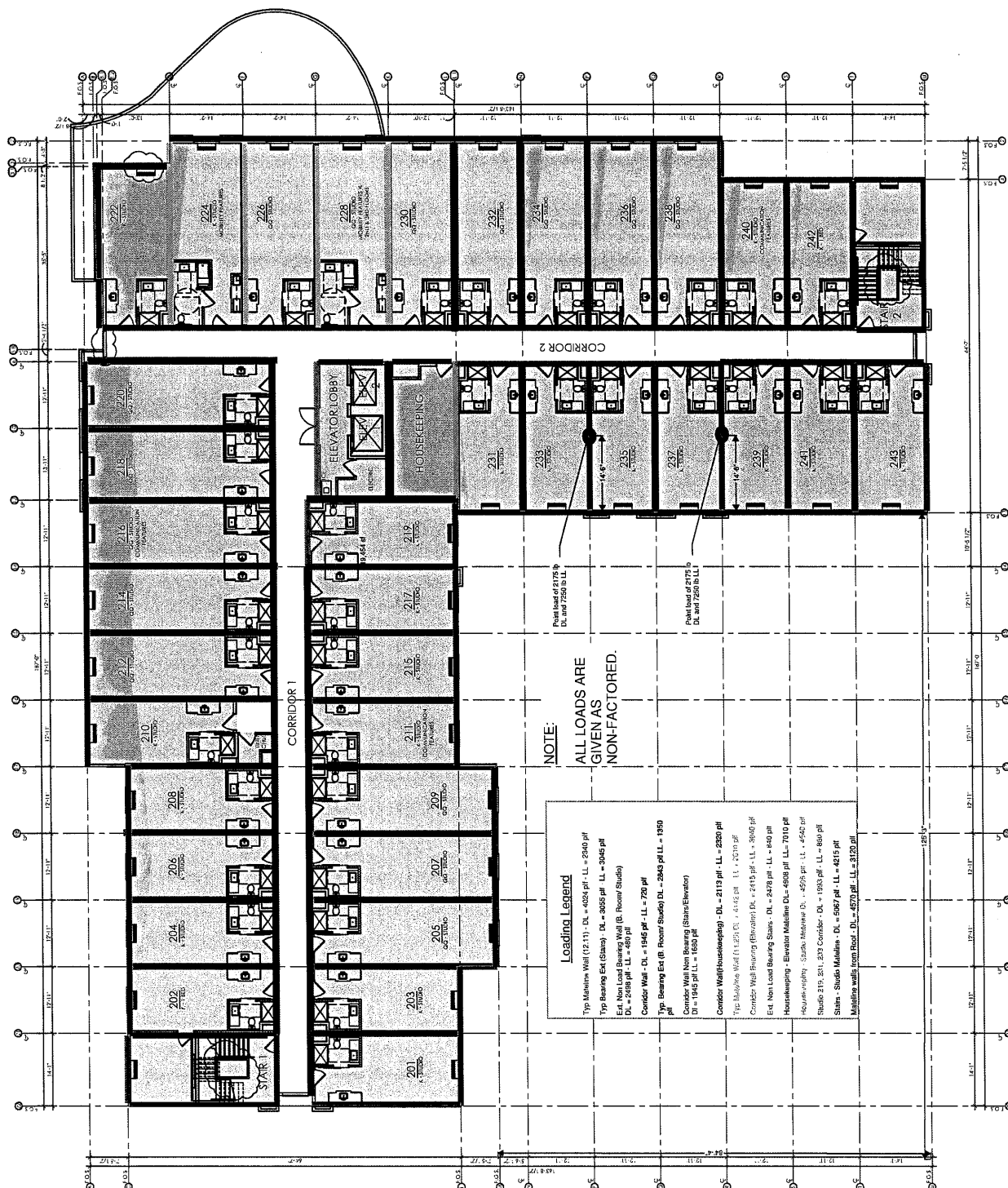
Response Mod. Coef., R =	5	ASCE 7-10 Table 12.2-1
Overstrength Factor, Ω _o =	2.5	ASCE 7-10 Table 12.2-1
Defl. Amplif. Factor, C _d =	5	ASCE 7-10 Table 12.2-1
C _s =	0.259	C _s = SDS/(R/I), ASCE 7-10 Section 12.8.1.1, Eqn. 12.8-2
C _s (max) =	1.256	For T <= T _L , C _s (max) = SD1/(T*(R/I)), ASCE 7-10 Eqn. 12.8-3
C _s (min) =	0.091	C _s (min) = 0.5*S ₁ /(R/I), ASCE 7-10 Eqn. 12.8-6
Use: C _s =	0.259	C _s (min) <= C _s <= C _s (max)
C _{ASD} =	0.181	C _{ASD} = 0.7 x C _s

Seismic Base Shear:

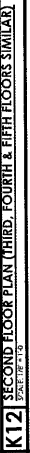
Redundancy, Rho ρ =	1.3	ASCE 12.3.4.1
V =	2535.45	kips, V = C _s *W, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1
V _{ASD} =	1774.82	kips, V _{ASD} = 0.7 x V

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SUPERSTRUCTURE DEAD & LIVE LOADS. (SUPPLIED BY BUILDING MAINT - 4/14/17)



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Project Name: HOME2 - SSF
Date of execution: May 28, 2017
2016

Specific Data: MESH
File Name: h2ssf-Adapt-170526-1.adm

FLOOR-PRO 2016-06
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Podium

117 MATERIALS

117.20 CONCRETE MATERIAL PROPERTIES

ID	Label	F'c	Unit Weight	Type	Ec	Creep coefficient
		psi	pcf		ksi	
1	WALLS	4000	150.00	Normal	3835	2.00
2	SLAB	5000	150.00	Normal	4287	2.00
3	COLUMN	5000	150.00	Normal	4287	2.00
4	WALL4K	4000	150.00	Normal	3835	2.00
5	COL5K	5000	150.00	Normal	3835	2.00
6	SLAB5K	5000	150.00	Normal	3835	2.00

F'c = strength at 28 days

Ec = modulus of elasticity at 28 days

117.40 REINFORCEMENT (NONPRESTRESSED) MATERIAL PROPERTIES

ID	Label	fy	fvy	Es
		ksi	ksi	ksi
1	MildSteel 1	60.000	60.000	30000

fy = yield stress of longitudinal reinforcement

fvy = yield stress of one-way shear reinforcement

Es = modulus of elasticity

117.60 PRESTRESSING MATERIAL PROPERTIES

ID	Label	fpu	fpv	Eps
		ksi	ksi	ksi
1	Prestressing 1	270.000	240.000	29007

fpu = ultimate stress

fpv = yield stress

Eps = modulus of elasticity

117.80 GENERIC MATERIAL PROPERTIES

ID	Label	Unit Weight	E	G	Poisson's Ratio
		pcf	ksi	ksi	

E = modulus of elasticity

G = shear modulus

142 CODES AND ASSUMPTIONS

142.10 DESIGN CODE SPECIFIED: ACI 2014/IBC 2015

142.15 TORSIONAL STIFFNESS OF BEAMS ACCOUNTED FOR

142.16 TORSIONAL STIFFNESS OF LOWER COLUMNS ACCOUNTED FOR

142.17 TORSIONAL STIFFNESS OF UPPER COLUMNS ACCOUNTED FOR

142.20 MATERIAL AND STRENGTH REDUCTION FACTORS

Two-way Slabs

ACI 2014/IBC 2015 strength reduction factors used:

For bending = 0.90
For shear two-way = 0.75

One-way Slabs

ACI 2014/IBC 2015 strength reduction factors used:

For bending = 0.90
For shear one-way = 0.75

Beams

ACI 2014/IBC 2015 strength reduction factors used:

For bending = 0.90
For shear one-way = 0.75

142.30 COVER TO REINFORCEMENT

Two-way Slabs

Prestressing Tendons (CGS)

At top = 2.50 in
At bottom = 2.50 in

Nonprestressing reinforcement (cover)

At top = 1.00 in
At bottom = 1.50 in

One-way Slabs

Prestressing Tendons (CGS)

At top = 2.50 in
At bottom = 2.50 in

Nonprestressing reinforcement (cover)

At top = 1.00 in
At bottom = 1.00 in

Beams

Prestressing Tendons (CGS)

At top = 2.50 in
At bottom = 2.50 in

Nonprestressing reinforcement (cover)

At top = 1.50 in
At bottom = 1.50 in

142.40 MINIMUM BAR LENGTH

Two-way systems

Cut off length of minimum steel over support (length/span) = 0.17

Cut off length of minimum steel in span (length/span) = 0.33

Extension of rebar beyond where required for strength:

Top bars = 18.00 in
Bottom bars = 24.00 in

One-way slabs

Cut off length of minimum steel over support (length/span) = 0.17

Cut off length of minimum steel in span (length/span) = 0.33

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2016

Specific Data: MESH
File Name: h2ssf-Adapt-170526-1.adm

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Extension of rebar beyond where required for strength:

Top bars
Bottom bars

= 12.00 in
= 12.00 in

Beams

Cut off length of minimum steel over support (length/span) = 0.17
Cut off length of minimum steel in span (length/span) = 0.33

Extension of rebar beyond where required for strength:

Top bars
Bottom bars

= 12.00 in
= 12.00 in

143 DESIGN CRITERIA

143.10 ONE-WAY SLABS

Sustain load stresses

Tension stress as multiple of $(f'c)^{1/2}$

At top fibers = 7.50

At bottom fibers = 7.50

Compression stress as multiple of $f'c$

Extreme fiber = 0.45

Initial load stresses

Tension stress as multiple of $(f'c)^{1/2}$

At top fibers = 3.00

At bottom fibers = 3.00

Compression stress as multiple of $f'c$

Extreme fiber = 0.60

Total load stresses

Tension stress as multiple of $(f'c)^{1/2}$

At top fibers = 7.50

At bottom fibers = 7.50

Compression stress as multiple of $f'c$

Extreme fiber = 0.60

143.20 TWO-WAY SLABS

Sustain load stresses

Tension stress as multiple of $(f'c)^{1/2}$

At top fibers = 6.00

At bottom fibers = 6.00

Compression stress as multiple of $f'c$

Extreme fiber = 0.45

Initial load stresses

Tension stress as multiple of $(f'c)^{1/2}$

At top fibers = 3.00

At bottom fibers = 3.00

Compression stress as multiple of $f'c$

Extreme fiber = 0.60

Total load stresses

Tension stress as multiple of $(f'c)^{1/2}$

At top fibers = 6.00

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At bottom fibers = 6.00
Compression stress as multiple of f'_c
Extreme fiber = 0.60

143.30 BEAMS

Sustain load stresses

Tension stress as multiple of $(f'_c)^{1/2}$
At top fibers = 7.50
At bottom fibers = 7.50
Compression stress as multiple of f'_c
Extreme fiber = 0.45

Initial load stresses

Tension stress as multiple of $(f'_c)^{1/2}$
At top fibers = 3.00
At bottom fibers = 3.00
Compression stress as multiple of f'_c
Extreme fiber = 0.60

Total load stresses

Tension stress as multiple of $(f'_c)^{1/2}$
At top fibers = 7.50
At bottom fibers = 7.50
Compression stress as multiple of f'_c
Extreme fiber = 0.60

144.40 USER DEFINED STIFFNESS MODIFIERS SUMMARY

Columns

usage	#	M11	M22	M33	F11
-------	---	-----	-----	-----	-----

Beams

usage	#	M11	M33	F11
-------	---	-----	-----	-----

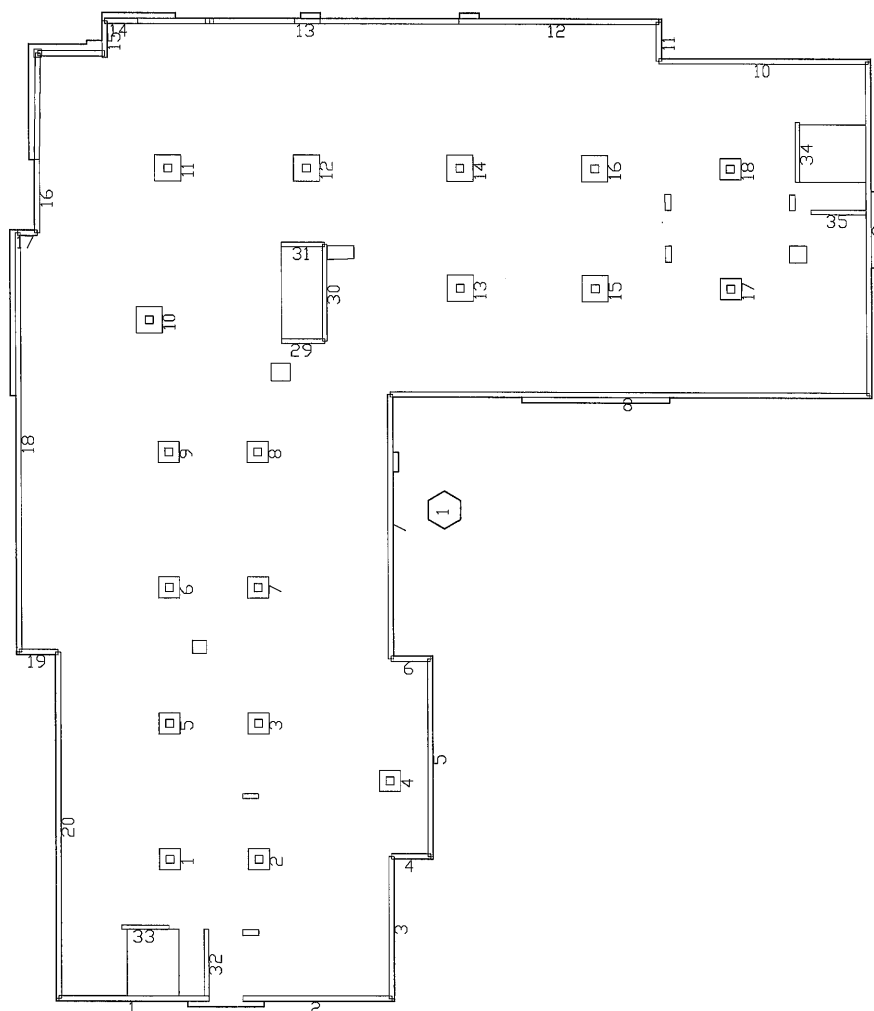
Walls

usage	#	M11	M22	F11	F22
-------	---	-----	-----	-----	-----

Slabs

usage	#	M11	M22	F11	F22
-------	---	-----	-----	-----	-----

10

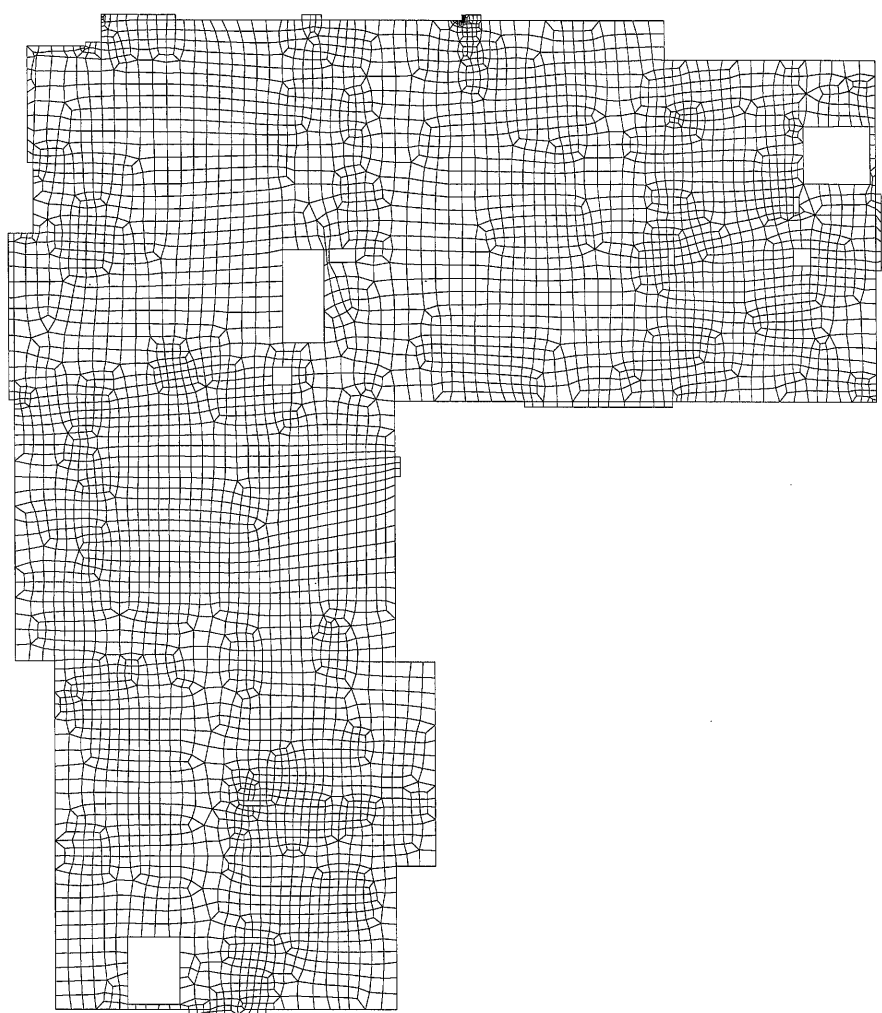


HOME2 - SSF MODEL

11/02/17
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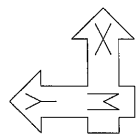
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Mesh Plan

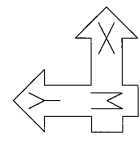
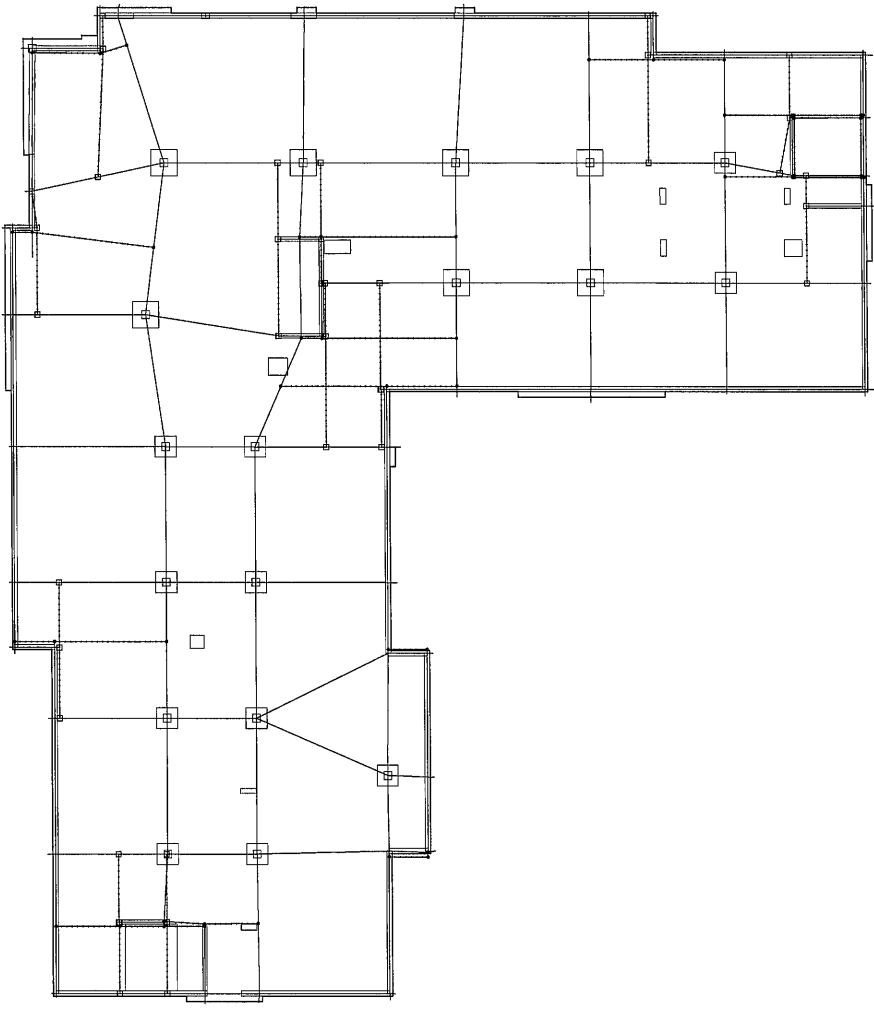
HOME2 - SSF



11/02/17
15:33:48

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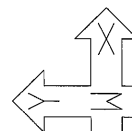
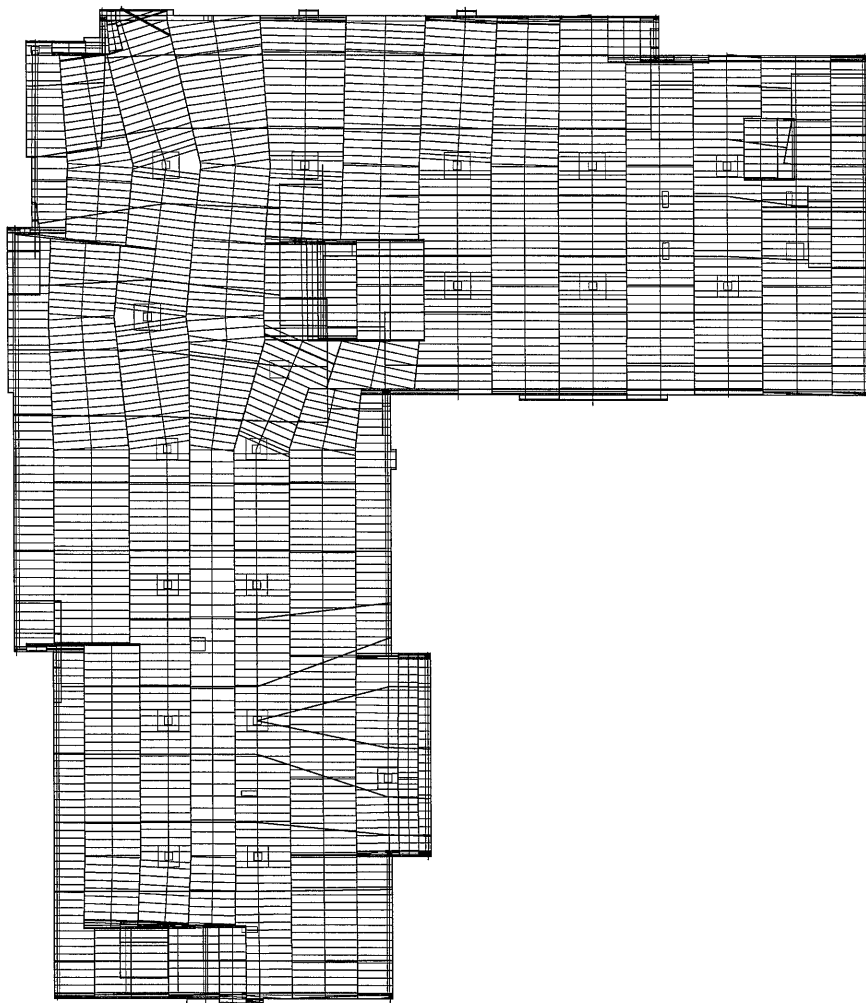


HOME2 - SSF SUPPORT LINES

11/02/17
15:36:29

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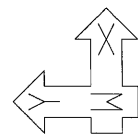
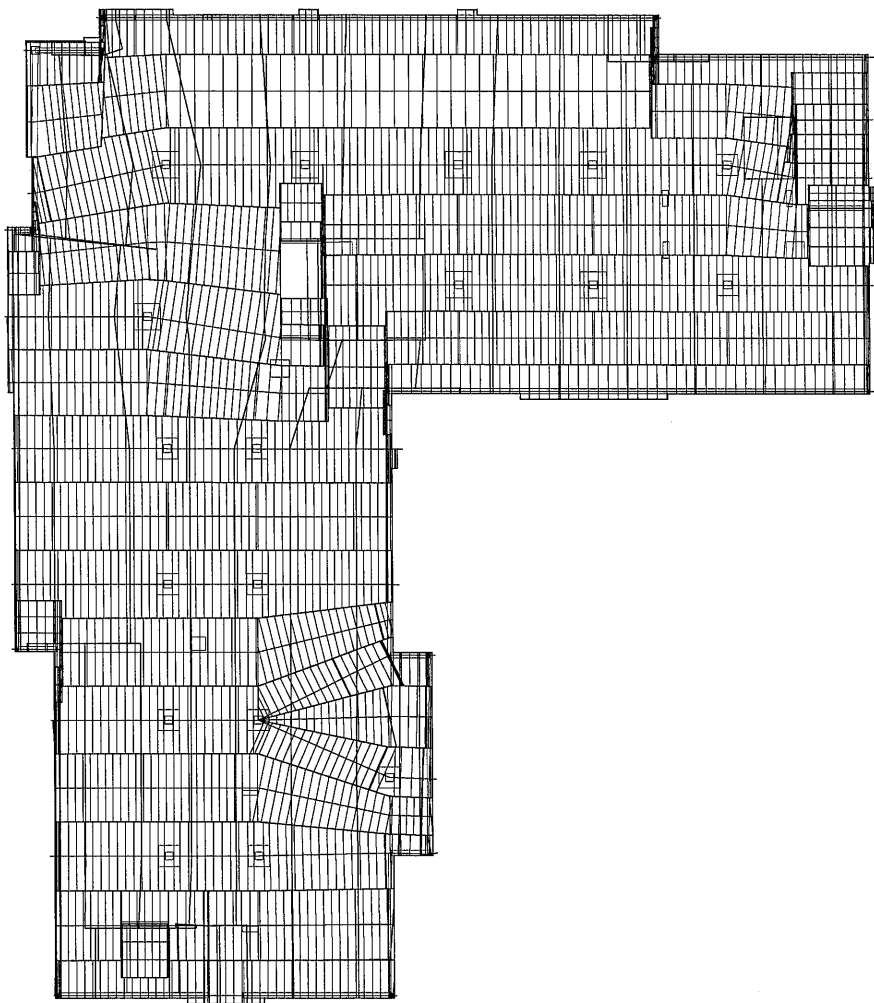
11/02/17
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HOME2 - SSF DESIGN STRIPS - X

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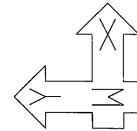
HOME2 - SSF DESIGN STRIPS - Y

11/02/17
15:38:26

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Two copies

The table contains a large number of rows and columns of handwritten text, which appears to be a detailed design criteria or specification document. The text is too small to read accurately but follows a structured grid format.

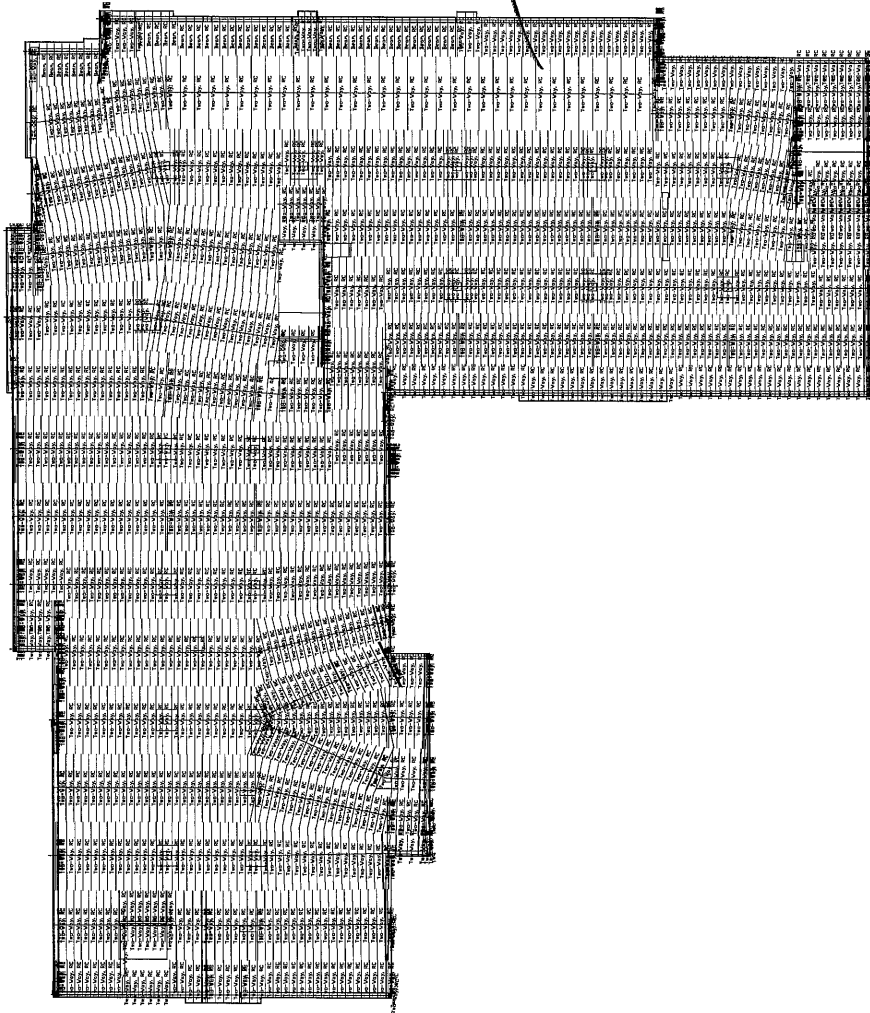


11/03/17
09:33:46

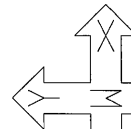
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HOME2 - SSF DESIGN CRITERIA - X DIR

2017.06
14.1



TWO ONLY



11/03/17
09:34:40

h2ssf-Adapt-171020-NoPortCo.adm

HOME2 - SSF DESIGN CRITERIA - Y DIR

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146 LOAD CASES AND COMBINATIONS

146.20 LOAD CASES

Dead load
Live load
Selfweight
Wind-X
Wind-Y
EQX
EQY
Wind_P0
Wind_P90
Seis-Podium

146.40 LOAD COMBINATIONS

Name: Service(Total Load)

Evaluation: SERVICE TOTAL LOAD

Combination detail: $1.00 \times \text{Selfweight} + 1.00 \times \text{Dead load} + 1.00 \times \text{Live load}$

Name: Service(Sustained Load)

Evaluation: SERVICE SUSTAINED LOAD

Combination detail: $1.00 \times \text{Selfweight} + 1.00 \times \text{Dead load} + 0.30 \times \text{Live load}$

Name: Strength(Dead and Live)

Evaluation: STRENGTH

Combination detail: $1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.60 \times \text{Live load}$

Name: Strength(Dead Load Only)

Evaluation: STRENGTH

Combination detail: $1.40 \times \text{Selfweight} + 1.40 \times \text{Dead load}$

Name: Strenght (D+L-Skip-Max)

Evaluation: STRENGTH

Combination detail: $1.60 \times \text{Skip_Live load_Max} + 1.00 \times \text{Hyperstatic} + 1.20 \times \text{Selfweight} +$

$1.20 \times \text{Dead load}$

Name: Strenght (D+L-Skip-Min)

Evaluation: STRENGTH

Combination detail: $1.00 \times \text{Hyperstatic} + 1.60 \times \text{Skip_Live load_Min} + 1.20 \times \text{Selfweight} +$

$1.20 \times \text{Dead load}$

Name: Initial

Evaluation: INITIAL

Combination detail: $1.15 \times \text{Prestressing} + 1.00 \times \text{Selfweight}$

Name: 4: 1.2D+WX+L

Evaluation: STRENGTH

Combination detail: $1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.00 \times \text{Live load} + 1.00 \times$

$\text{Wind-X} + 1.00 \times \text{Wind_P0}$

Name: 4: 1.2D-WX+L

Evaluation: STRENGTH

Combination detail: $1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.00 \times \text{Live load} - 1.00 \times$

$\text{Wind-X} - 1.00 \times \text{Wind_P0}$

Name: 4: 1.2D+WY+L

Evaluation: STRENGTH

Combination detail: $1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.00 \times \text{Live load} + 1.00 \times$

$\text{Wind-Y} + 1.00 \times \text{Wind_P90}$

Name: 4: 1.2D-WY+L

Evaluation: STRENGTH

Combination detail: $1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.00 \times \text{Live load} - 1.00 \times$

$\text{Wind-Y} - 1.00 \times \text{Wind_P90}$

Name: 5: 1.2D+EQX+L

Evaluation: STRENGTH

Combination detail: $1.69 \times \text{SEIS-X} + 1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.00 \times \text{Live load} + 1.69 \times \text{EQX} + 1.00 \times \text{Seis-Podium}$

Name: 5: 1.2D-EQX+L

Evaluation: STRENGTH

Combination detail: $-1.69 \times \text{SEIS-X} + 1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.00 \times \text{Live load} - 1.69 \times \text{EQX} - 1.00 \times \text{Seis-Podium}$

Name: 5: 1.2D+EQY+L

Evaluation: STRENGTH

Combination detail: $1.69 \times \text{SEIS-Y} + 1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.00 \times \text{Live load} + 1.69 \times \text{EQY} + 1.00 \times \text{Seis-Podium}$

Name: 5: 1.2D-EQY+L

Evaluation: STRENGTH

Combination detail: $-1.69 \times \text{SEIS-Y} + 1.20 \times \text{Selfweight} + 1.20 \times \text{Dead load} + 1.00 \times \text{Live load} - 1.69 \times \text{EQY} - 1.00 \times \text{Seis-Podium}$

Name: 6: 0.9D+WX

Evaluation: STRENGTH

Combination detail: $0.90 \times \text{Selfweight} + 0.09 \times \text{Dead load} + 1.00 \times \text{Wind-X} + 1.00 \times \text{Wind_P0}$

Name: 6: 0.9D-WX

Evaluation: STRENGTH

Combination detail: $0.90 \times \text{Selfweight} + 0.09 \times \text{Dead load} - 1.00 \times \text{Wind-X} - 1.00 \times \text{Wind_P0}$

Name: 6: 0.9D+WY

Evaluation: STRENGTH

Combination detail: $0.90 \times \text{Selfweight} + 0.09 \times \text{Dead load} + 1.00 \times \text{Wind-Y} + 1.00 \times \text{Wind_P90}$

Name: 6: 0.9D-WY

Evaluation: STRENGTH

Combination detail: $0.90 \times \text{Selfweight} + 0.09 \times \text{Dead load} - 1.00 \times \text{Wind-Y} - 1.00 \times \text{Wind_P90}$

Name: 7: 0.9D+EQX

Evaluation: STRENGTH

Combination detail: $1.69 \times \text{SEIS-X} + 0.90 \times \text{Selfweight} + 0.09 \times \text{Dead load} + 1.69 \times \text{EQX} + 1.00 \times \text{Seis-Podium}$

Name: 7: 0.9D-EQX

Evaluation: STRENGTH

Combination detail: $-1.69 \times \text{SEIS-X} + 0.90 \times \text{Selfweight} + 0.09 \times \text{Dead load} - 1.69 \times \text{EQX} - 1.00 \times \text{Seis-Podium}$

Name: 7: 0.9D+EQY

Evaluation: STRENGTH

Combination detail: $1.69 \times \text{SEIS-Y} + 0.90 \times \text{Selfweight} + 0.09 \times \text{Dead load} + 1.69 \times \text{EQY} + 1.00 \times \text{Seis-Podium}$

Name: 7: 0.9D-EQY

Evaluation: STRENGTH

Combination detail: $-1.69 \times \text{SEIS-Y} + 0.90 \times \text{Selfweight} + 0.09 \times \text{Dead load} - 1.69 \times \text{EQY} - 1.00 \times \text{Seis-Podium}$

Name: Sustained_Load

Evaluation: CRACKED DEFLECTION

Combination detail: $1.00 \times \text{Prestressing} + 1.00 \times \text{Selfweight} + 1.00 \times \text{Dead load} + 0.30 \times \text{Live load}$

Name: Service - DEAD ONLY

Evaluation: SERVICE TOTAL LOAD

Combination detail: $1.00 \times \text{Selfweight} + 1.00 \times \text{Dead load}$

Name: Service - LIVE ONLY

Evaluation: SERVICE TOTAL LOAD

Project Name: HOME2 - SSF

Specific Data: DESIGN STRIPS - Y

Date of execution: November 02, 2017

File Name: h2ssf-Adapt-171020-NoPorteCo.adm

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Combination detail: 1.00 x Live load

Name: Service - Seismic Only- X Dir

Evaluation: SERVICE TOTAL LOAD

Combination detail: 1.00 x SEIS-X + 1.00 x EQX + 1.00 x Seis-Podium

Name: Service - Seismic Only- Y Dir

Evaluation: SERVICE TOTAL LOAD

Combination detail: 1.00 x SEIS-Y + 1.00 x EQY + 1.00 x Seis-Podium

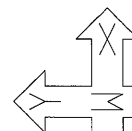
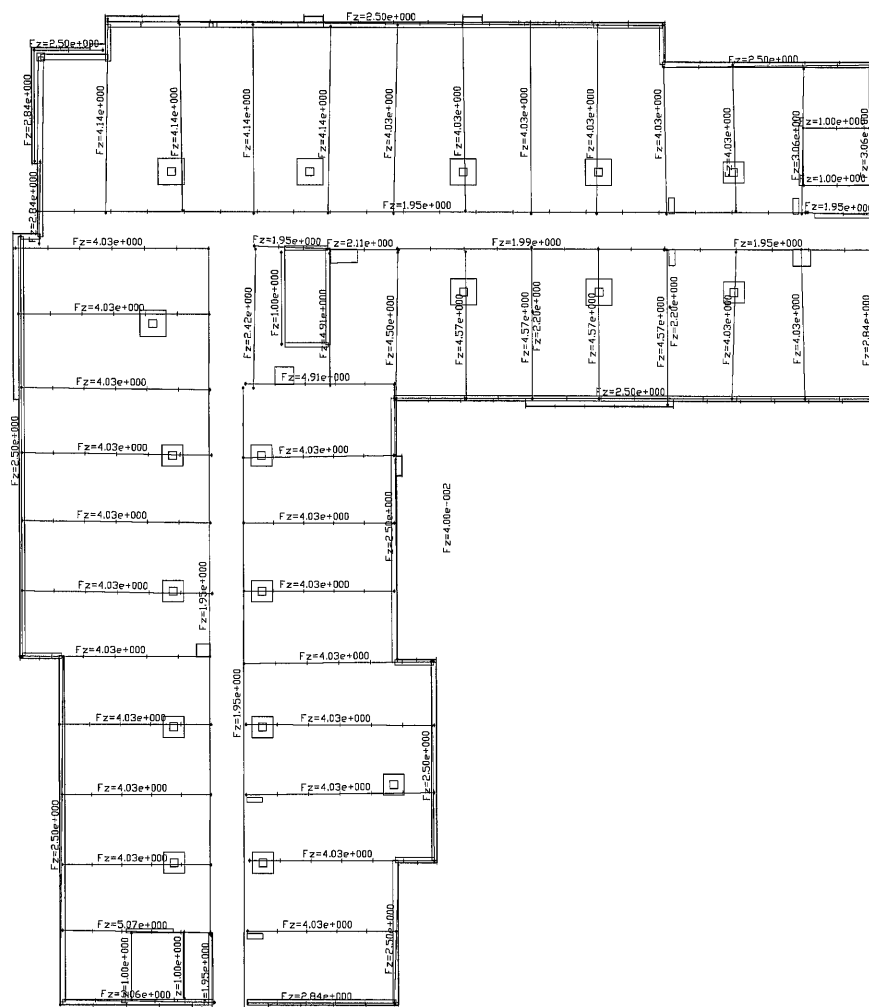
Name: Long_Term

Evaluation: LONG TERM DEFLECTION

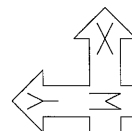
Combination detail: 4.00 x Sustained_Load

11/02/17
15:44:52

h2ssf-Adapt-171020-NoPorteCo.adm



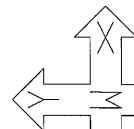
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11/02/17
15:46:25

h2ssf-Adapt-171020-NoPorteCo.adm

20



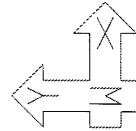
HOME2 - SSF LOADS - EQX

11/02/17

15:47:27

h2ssf-Adapt-171020-NoPortCo.adm

Max demand/capacity ratio: 1.01



h2ssf-Adapt-171020-NoPorteCo.adm

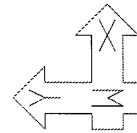
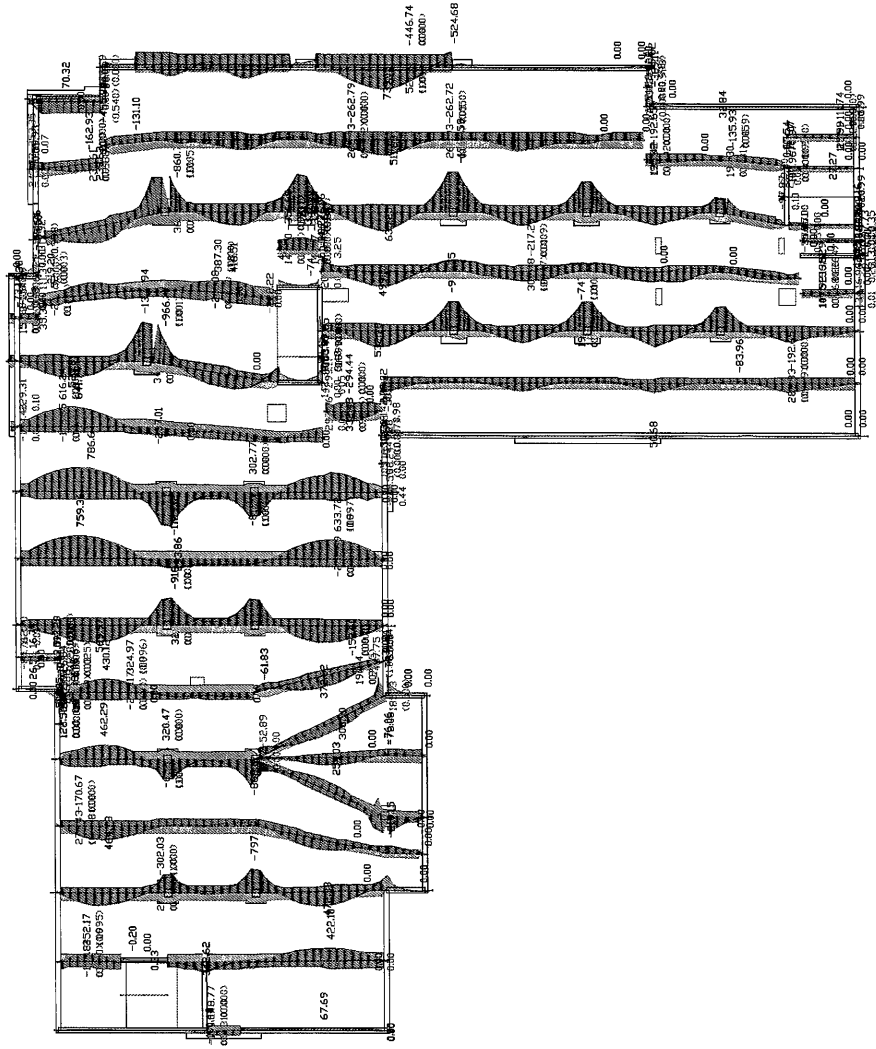
2017-06
(22)

Design Sections, Investigation, Moment Capacity with Demand

Max: 975.80

Min: -1573.23

Max demand/capacity ratio: 1.01



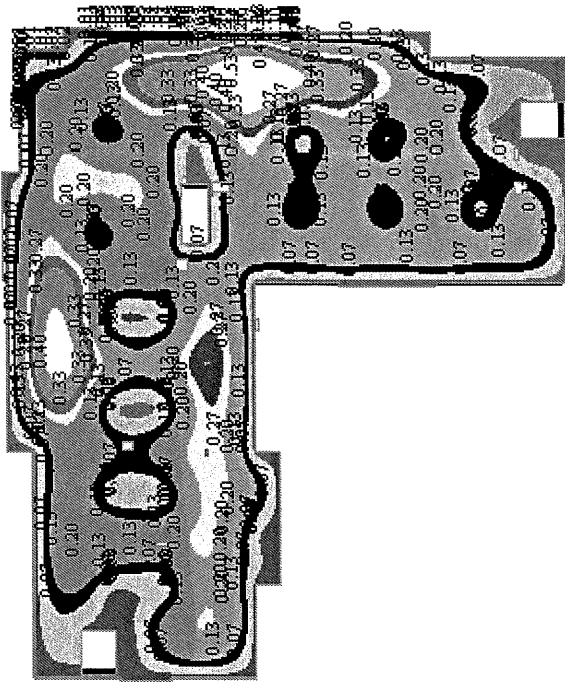
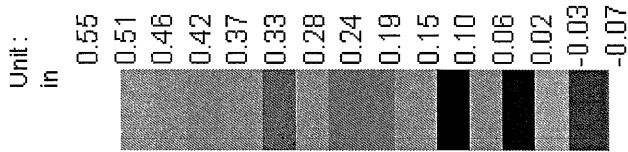
2017-06
23

11/03/17

10:20:01

h2ssf-Adapt-171020-NoPorteCo.adm

HOME2 - SSF MOMENT CAPACITY w/ DEMAND



Z-Translation

General
Specifi

11/02/17
17:12:41

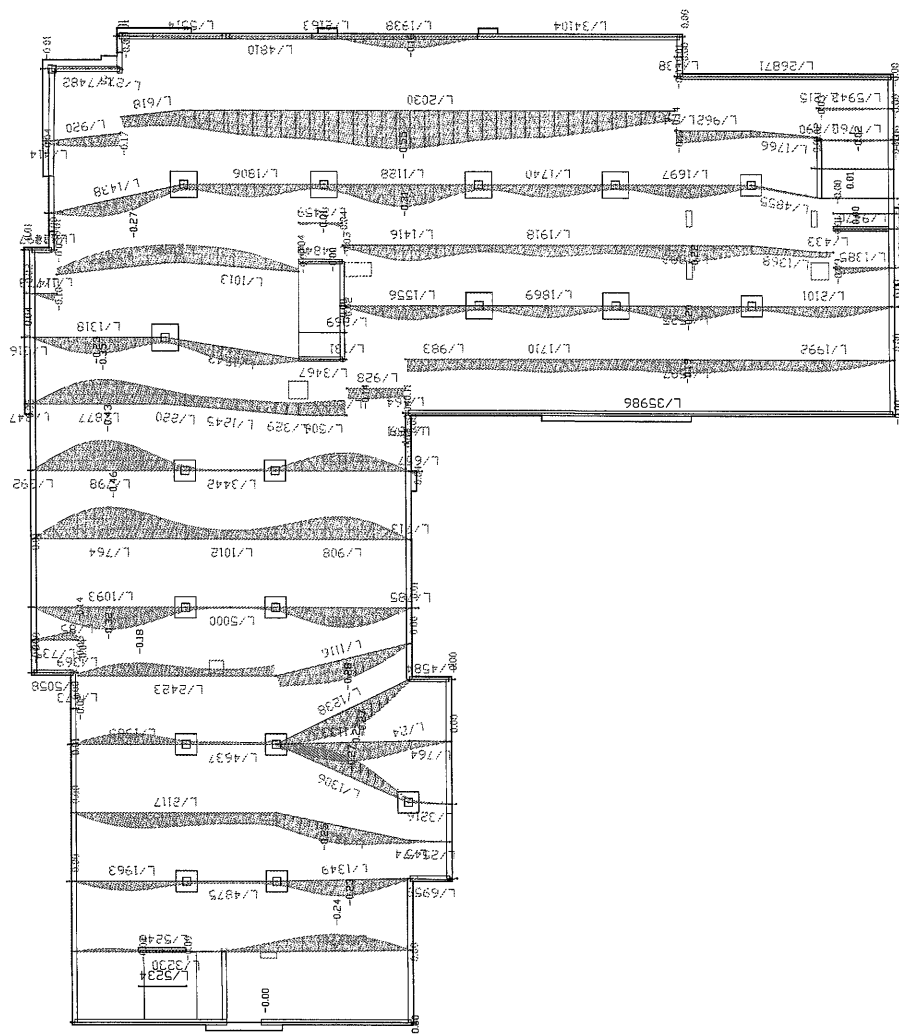
h2ssf-Adapt-171020-NoPorteCo.adv

Units: in

Deformation

25

A diagram showing a cross-section of a road with a central divider and arrows indicating traffic flow.



Structural Plan

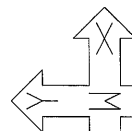
HOME2 - SSF

DEFLECTION - Z TRANSLATION

11/02/17

17:16:03

h2ssf-Adapt-171020-NoPorteCo.adm



Structural Plan

HOME2 - SSF

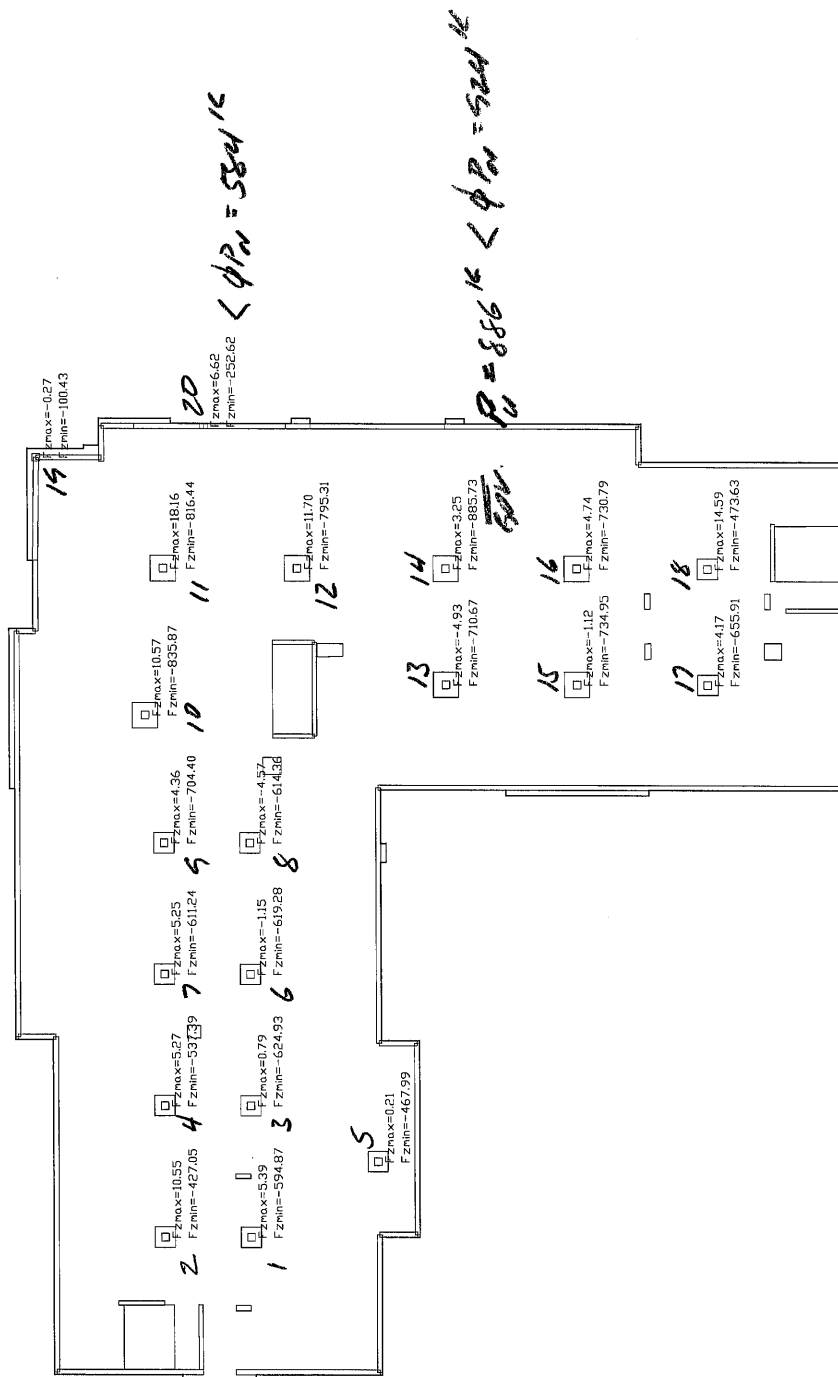
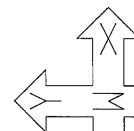
DEFLECTION - Z TRANSLATION

11/02/17

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h2ssf-Adapt-171020-NoPortCo.adm

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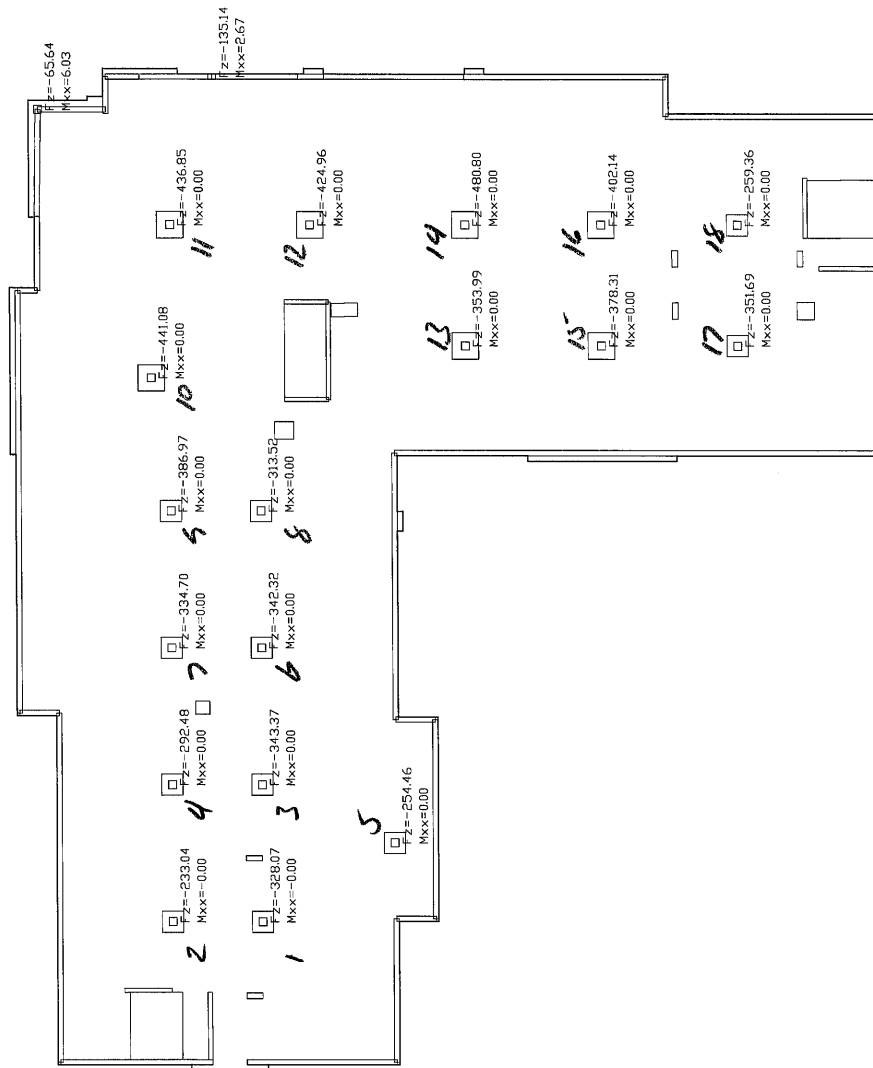
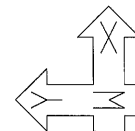
Column Reactions Plan_Envelope

HOME2 - SSF COLUMN REACTIONS

11/02/17
17:25:26

h2ssf-Adapt-171020-NoPorteCo.adm

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Column Reactions Plan_Service - DEAD ONLY COLUMN REACTION - Fz - SERVICE = DEAD

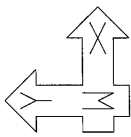
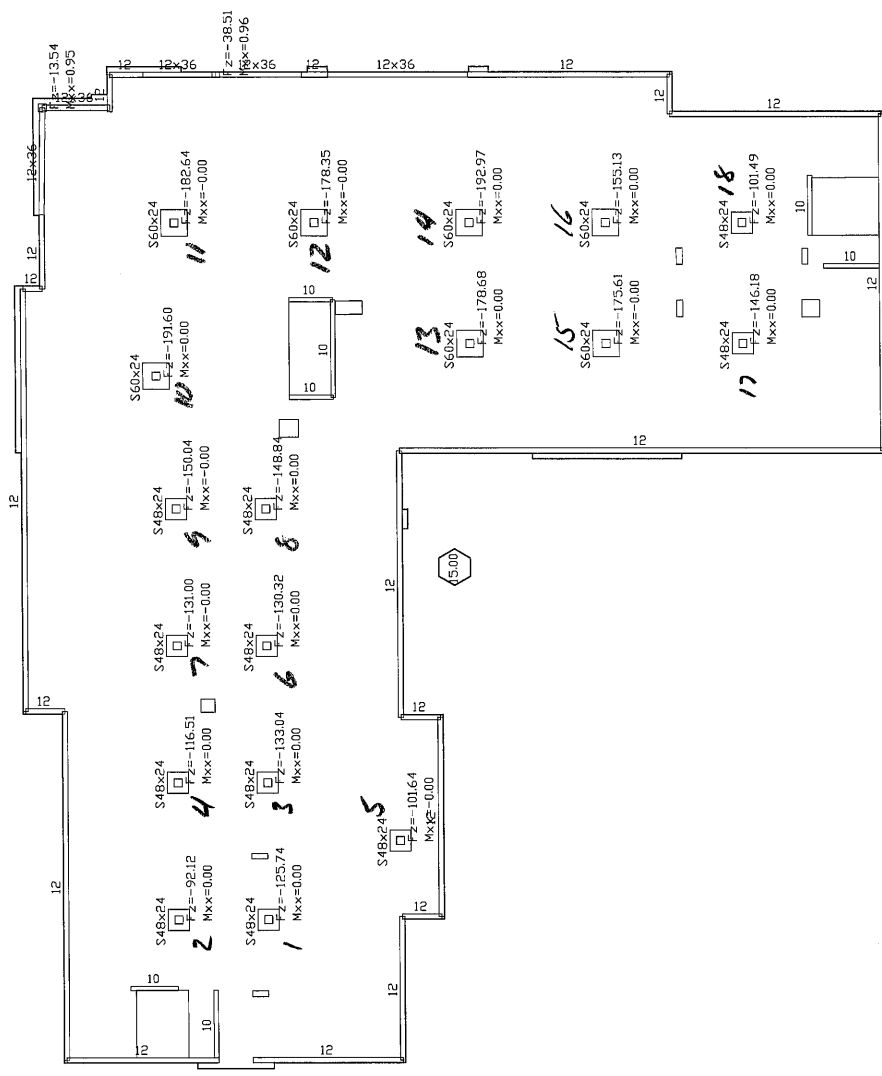
11/02/17
17:29:57

HOME2 - SSF

COLUMN REACTION - Fz - SERVICE = DEAD

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Column Reactions Plan_Service - LIVE ONLY COLUMN REACTION - Fz - SERVICE = LIVE

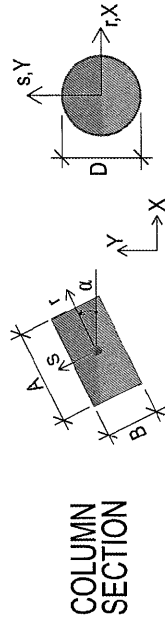
HOME2 - SSF

COLUMN REACTION - Fz - SERVICE = LIVE

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17:31:33

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160.20/40 LOWER/UPPER COLUMNS



COLUMN SECTION

Service/Total Load											
#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft			
1	Column 1	top	-453.802	0.000	-0.000	-0.000	0.000	0.000			
		bottom	455.960	-0.000	0.000	-0.000	0.000	0.000			
2	Column 2	top	-325.165	0.000	0.000	-0.000	0.000	0.000			
		bottom	327.323	0.000	0.000	-0.000	0.000	0.000			
3	Column 3	top	-476.417	-0.000	0.000	0.000	0.000	0.000			
		bottom	478.576	0.000	-0.000	0.000	0.000	0.000			
4	Column 4	top	-408.974	0.000	0.000	0.000	-0.000	0.000			
		bottom	411.133	0.000	-0.000	0.000	0.000	0.000			
5	Column 5	top	-358.099	-0.000	0.000	0.000	0.000	0.000			
		bottom	358.257	-0.000	0.000	0.000	-0.000	0.000			
6	Column 6	top	-462.367	0.000	0.000	0.000	0.000	0.000			
		bottom	464.525	0.000	0.000	0.000	0.000	0.000			
7	Column 7	top	-463.694	-0.000	0.000	-0.000	0.000	0.000			
		bottom	467.852	0.000	0.000	-0.000	0.000	0.000			
8	Column 8	top	-462.367	0.000	0.000	0.000	-0.000	0.000			
		bottom	464.525	0.000	0.000	0.000	-0.000	0.000			
9	Column 9	top	-537.004	0.000	-0.000	-0.000	0.000	0.000			
		bottom	539.139	-0.000	0.000	-0.000	0.000	0.000			
10	Column 10	top	-632.683	0.000	0.000	0.000	0.000	0.000			
		bottom	634.841	-0.000	0.000	0.000	-0.000	0.000			
11	Column 11	top	-619.487	0.000	-0.000	-0.000	0.000	0.000			
		bottom	621.645	0.000	0.000	-0.000	0.000	0.000			
12	Column 12	top	-603.300	0.000	0.000	0.000	-0.000	0.000			
		bottom	605.458	0.000	0.000	0.000	0.000	0.000			
13	Column 13	top	-532.665	-0.000	0.000	0.000	0.000	0.000			
		bottom	534.823	0.000	-0.000	0.000	0.000	0.000			
14	Column 14	top	-673.778	-0.000	0.000	0.000	0.000	0.000			
		bottom	675.936	0.000	-0.000	0.000	0.000	0.000			
15	Column 15	top	-553.909	-0.000	-0.000	-0.000	0.000	0.000			
		bottom	556.068	0.000	0.000	0.000	0.000	0.000			
16	Column 16	top	-557.282	-0.000	0.000	0.000	0.000	0.000			

		bottom	559.440	0.000	0.000	0.000	0.000	0.000			
17	Column 17	top	-497.864	0.000	0.000	0.000	0.000	0.000			
		bottom	500.022	-0.000	0.000	-0.000	0.000	0.000			
18	Column 18	top	-360.865	0.000	0.000	0.000	0.000	0.000			
		bottom	363.023	0.000	0.000	-0.000	0.000	0.000			
19	Column 19	top	-79.176	3.001	0.548	6.987	-38.262	0.000			
		bottom	81.327	-3.001	-0.548	0.000	-0.000	0.000			

20	Column 20	top	-173.648	4.004	0.284	3.624	-51.052	-0.000			
		bottom	175.082	-4.004	-0.284	0.000	0.000	0.000			

Service/Sustained Load

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft			
1	Column 1	top	-365.788	0.000	-0.000	-0.000	0.000	0.000			
		bottom	367.947	-0.000	0.000	-0.000	0.000	0.000			
2	Column 2	top	-260.690	0.000	0.000	-0.000	0.000	0.000			
		bottom	262.825	0.000	0.000	0.000	0.000	0.000			
3	Column 3	top	-383.301	-0.000	0.000	0.000	0.000	0.000			
		bottom	385.437	0.000	-0.000	0.000	0.000	0.000			
4	Column 4	top	-327.413	0.000	0.000	0.000	-0.000	0.000			
		bottom	329.572	0.000	-0.000	0.000	0.000	0.000			
5	Column 5	top	-284.969	-0.000	0.000	0.000	0.000	0.000			
		bottom	287.105	0.000	-0.000	0.000	0.000	0.000			
6	Column 6	top	-381.413	0.000	0.000	0.000	0.000	0.000			
		bottom	383.571	0.000	0.000	0.000	0.000	0.000			
7	Column 7	top	-374.016	-0.000	0.000	-0.000	0.000	0.000			
		bottom	376.152	0.000	0.000	-0.000	0.000	0.000			
8	Column 8	top	-358.167	0.000	0.000	0.000	-0.000	0.000			
		bottom	360.326	0.000	0.000	0.000	-0.000	0.000			
9	Column 9	top	-431.972	0.000	-0.000	-0.000	0.000	0.000			
		bottom	434.131	-0.000	0.000	-0.000	0.000	0.000			
10	Column 10	top	-498.561	0.000	0.000	0.000	0.000	0.000			
		bottom	500.719	-0.000	0.000	0.000	-0.000	0.000			
11	Column 11	top	-491.637	0.000	0.000	0.000	0.000	0.000			
		bottom	493.795	0.000	0.000	-0.000	0.000	0.000			
12	Column 12	top	-478.463	0.000	0.000	0.000	-0.000	0.000			
		bottom	480.621	0.000	0.000	0.000	0.000	0.000			
13	Column 13	top	-407.581	-0.000	0.000	0.000	0.000	0.000			
		bottom	409.739	0.000	-0.000	0.000	0.000	0.000			
14	Column 14	top	-538.712	-0.000	0.000	0.000	0.000	0.000			
		bottom	540.848	0.000	-0.000	0.000	0.000	0.000			
15	Column 15	top	-430.983	-0.000	0.000	0.000	0.000	0.000			
		bottom	433.141	0.000	0.000	0.000	0.000	0.000			
16	Column 16	top	-448.676	-0.000	0.000	0.000	0.000	0.000			
		bottom	450.834	0.000	0.000	0.000	0.000	0.000			
17	Column 17	top	-395.531	0.000	0.000	0.000	0.000	0.000			
		bottom	397.689	-0.000	0.000	-0.000	0.000	0.000			
18	Column 18	top	-289.825	0.000	0.000	0.000	0.000	0.000			

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Project Name: HOME2 - SSF Specific Data: MOMENT CAPACITY w/ DEMAND
Date of execution: November 03, 2017 File Name: h2ssf-Adapt-171020-NoPorteCo.adm
FLOOR-PRO 2016

		bottom	291.961	0.000	0.000	-0.000	0.000	0.000	0.000
19	Column 19	top	-69.700	2.392	0.496	6.320	-30.497	0.000	0.000
		bottom	71.852	-2.392	-0.496	0.000	-0.000	0.000	0.000

20	Column 20	top	-148.689	3.139	0.232	2.953	-40.018	-0.000	0.000
		bottom	148.123	-3.139	-0.232	0.000	0.000	0.000	0.000

Strength(Dead and Live)

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-594.870	0.000	-0.000	-0.000	0.000	0.000
		bottom	597.433	-0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-427.049	0.000	0.000	-0.000	0.000	0.000
		bottom	429.634	0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-624.927	-0.000	0.000	0.000	0.000	0.000
		bottom	627.512	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-537.386	0.000	0.000	0.000	-0.000	0.000
		bottom	539.971	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-467.987	-0.000	0.000	-0.000	0.000	0.000
		bottom	470.572	-0.000	0.000	-0.000	0.000	0.000
6	Column 6	top	-619.284	0.000	0.000	0.000	0.000	0.000
		bottom	621.870	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-611.236	-0.000	-0.000	-0.000	0.000	0.000
		bottom	613.821	0.000	0.000	0.000	-0.000	0.000
8	Column 8	top	-614.361	0.000	0.000	0.000	-0.000	0.000
		bottom	616.946	0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-704.397	0.000	-0.000	-0.000	0.000	0.000
		bottom	706.983	-0.000	-0.000	-0.000	0.000	0.000
10	Column 10	top	-835.866	0.000	0.000	0.000	0.000	0.000
		bottom	838.429	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-816.443	0.000	-0.000	-0.000	0.000	0.000
		bottom	819.028	0.000	0.000	-0.000	0.000	0.000
12	Column 12	top	-795.310	0.000	0.000	-0.000	-0.000	0.000
		bottom	797.896	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-717.669	-0.000	0.000	0.000	0.000	0.000
		bottom	713.255	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	-885.729	-0.000	0.000	0.000	0.000	0.000
		bottom	888.314	-0.000	-0.000	0.000	0.000	0.000
15	Column 15	top	-734.949	-0.000	-0.000	-0.000	0.000	0.000
		bottom	737.534	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-730.790	-0.000	0.000	0.000	0.000	0.000
		bottom	733.375	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-655.906	0.000	0.000	0.000	0.000	0.000
		bottom	658.491	0.000	0.000	-0.000	0.000	0.000
18	Column 18	top	-473.630	0.000	0.000	0.000	0.000	0.000
		bottom	476.215	0.000	0.000	-0.000	0.000	0.000
19	Column 19	top	-100.427	3.949	0.887	8.765	-50.351	0.000
		bottom	103.008	-3.949	-0.687	0.000	-0.000	0.000

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ADAPT International Pvt. Ltd, Kolkata, India Tel: +91-33-302 86580 Fax: +91-33-224 67281

Project Name: HOME2 - SSF Specific Data: MOMENT CAPACITY w/ DEMAND
Date of execution: November 03, 2017 File Name: h2ssf-Adapt-171020-NoPorteCo.adm
FLOOR-PRO 2016

20	Column 20	top	-223.783	5.299	0.371	4.732	-67.569	-0.000	0.000
		bottom	225.507	-5.299	-0.371	0.000	0.000	0.000	0.000

Strength(Dead Load Only)

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	459.287	0.000	-0.000	-0.000	0.000	0.000
		bottom	462.299	-0.000	0.000	0.000	0.000	0.000
2	Column 2	top	326.267	0.000	0.000	-0.000	0.000	0.000
		bottom	329.279	0.000	0.000	-0.000	0.000	0.000
3	Column 3	top	480.734	0.000	0.000	0.000	0.000	0.000
		bottom	483.746	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	409.447	0.000	0.000	0.000	-0.000	0.000
		bottom	412.459	0.000	0.000	0.000	0.000	0.000
5	Column 5	top	356.256	-0.000	0.000	0.000	0.000	0.000
		bottom	359.269	0.000	-0.000	0.000	0.000	0.000
6	Column 6	top	479.228	0.000	0.000	0.000	0.000	0.000
		bottom	482.240	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	468.594	0.000	0.000	0.000	0.000	0.000
		bottom	471.606	0.000	0.000	-0.000	0.000	0.000
8	Column 8	top	438.942	0.000	0.000	0.000	-0.000	0.000
		bottom	441.954	0.000	0.000	0.000	-0.000	0.000
9	Column 9	top	541.747	0.000	0.000	0.000	0.000	0.000
		bottom	544.760	-0.000	0.000	-0.000	0.000	0.000
10	Column 10	top	617.508	0.000	0.000	0.000	0.000	0.000
		bottom	620.521	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	611.596	0.000	0.000	0.000	-0.000	0.000
		bottom	614.608	0.000	0.000	0.000	0.000	0.000
12	Column 12	top	594.937	0.000	0.000	0.000	-0.000	0.000
		bottom	597.950	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	495.571	-0.000	0.000	0.000	0.000	0.000
		bottom	498.584	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	673.126	-0.000	0.000	0.000	0.000	0.000
		bottom	676.139	0.000	-0.000	0.000	0.000	0.000
15	Column 15	top	529.630	-0.000	0.000	0.000	0.000	0.000
		bottom	532.642	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	562.992	-0.000	0.000	0.000	0.000	0.000
		bottom	566.004	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	492.356	0.000	0.000	0.000	0.000	0.000
		bottom	495.369	0.000	0.000	-0.000	0.000	0.000
18	Column 18	top	363.113	0.000	0.000	0.000	0.000	0.000
		bottom	366.126	0.000	0.000	-0.000	0.000	0.000
19	Column 19	top	91.896	2.983	0.663	8.449	-38.036	0.000
		bottom	94.906	-2.983	-0.663	0.000	-0.000	0.000
20	Column 20	top	189.189	3.875	0.293	3.732	-49.404	-0.000
		bottom	191.199	-3.875	-0.293	0.000	0.000	0.000

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Project Name: HOME2 - SSF Specific Data: MOMENT CAPACITY w/ DEMAND
Date of execution: November 03, 2017 File Name: h2ssf-Adapt-171020-NoPorteCo.adm
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Strength (D+L-Skip-Max)									
#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft	
1	Column 1	top	-393.687	0.000	-0.000	-0.000	0.000	0.000	
		bottom	396.273	-0.000	0.000	0.000	0.000	0.000	
2	Column 2	top	-279.664	0.000	-0.000	-0.000	0.000	0.000	
		bottom	282.249	0.000	0.000	0.000	0.000	0.000	
3	Column 3	top	-412.054	0.000	-0.000	-0.000	0.000	0.000	
		bottom	414.640	0.000	0.000	0.000	0.000	0.000	
4	Column 4	top	-350.973	0.000	-0.000	-0.000	0.000	0.000	
		bottom	353.536	0.000	0.000	0.000	0.000	0.000	
5	Column 5	top	-305.359	0.000	-0.000	-0.000	0.000	0.000	
		bottom	307.945	0.000	0.000	0.000	0.000	0.000	
6	Column 6	top	-410.773	0.000	-0.000	-0.000	0.000	0.000	
		bottom	413.358	0.000	0.000	0.000	0.000	0.000	
7	Column 7	top	-401.646	0.000	-0.000	-0.000	0.000	0.000	
		bottom	404.231	0.000	0.000	0.000	0.000	0.000	
8	Column 8	top	-376.242	0.000	-0.000	-0.000	0.000	0.000	
		bottom	378.805	0.000	0.000	0.000	0.000	0.000	
9	Column 9	top	-464.345	0.000	-0.000	-0.000	0.000	0.000	
		bottom	466.930	0.000	0.000	0.000	0.000	0.000	
10	Column 10	top	-529.293	0.000	-0.000	-0.000	0.000	0.000	
		bottom	531.878	0.000	0.000	0.000	0.000	0.000	
11	Column 11	top	-524.212	0.000	-0.000	-0.000	0.000	0.000	
		bottom	526.797	0.000	0.000	0.000	0.000	0.000	
12	Column 12	top	-509.937	0.000	-0.000	-0.000	0.000	0.000	
		bottom	512.522	0.000	0.000	0.000	0.000	0.000	
13	Column 13	top	-424.779	0.000	-0.000	-0.000	0.000	0.000	
		bottom	427.364	0.000	0.000	0.000	0.000	0.000	
14	Column 14	top	-576.975	0.000	-0.000	-0.000	0.000	0.000	
		bottom	579.560	0.000	0.000	0.000	0.000	0.000	
15	Column 15	top	-453.959	0.000	-0.000	-0.000	0.000	0.000	
		bottom	456.544	0.000	0.000	0.000	0.000	0.000	
16	Column 16	top	-482.577	0.000	-0.000	-0.000	0.000	0.000	
		bottom	485.163	0.000	0.000	0.000	0.000	0.000	
17	Column 17	top	-422.013	0.000	-0.000	-0.000	0.000	0.000	
		bottom	424.599	0.000	0.000	0.000	0.000	0.000	
18	Column 18	top	-311.249	0.000	-0.000	-0.000	0.000	0.000	
		bottom	313.835	0.000	0.000	0.000	0.000	0.000	
19	Column 19	top	-78.767	2.557	0.568	7.242	-32.602	0.000	
		bottom	81.350	-2.557	-0.568	0.000	-0.000	0.000	

20	Column 20	top	-162.162	3.321	0.251	3.199	-42.346	-0.000	
		bottom	163.884	-3.321	-0.251	0.000	0.000	0.000	

Strength (D+L-Skip-Min)									
#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft	
1	Column 1	top	-393.687	0.000	-0.000	-0.000	0.000	0.000	

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Date of execution: November 03, 2017 File Name: h2ssf-Adapt-171020-NoPorteCo.adm
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2	Column 2	bottom	396.273	-0.000	0.000	0.000	0.000	0.000	
		top	-279.664	0.000	-0.000	0.000	0.000	0.000	
3	Column 3	top	-412.054	0.000	-0.000	-0.000	0.000	0.000	
		bottom	414.640	0.000	0.000	0.000	0.000	0.000	
4	Column 4	top	-350.973	0.000	-0.000	-0.000	0.000	0.000	
		bottom	353.536	0.000	0.000	0.000	0.000	0.000	
5	Column 5	top	-305.359	-0.000	0.000	0.000	0.000	0.000	
		bottom	307.945	0.000	-0.000	0.000	0.000	0.000	
6	Column 6	top	-410.773	0.000	-0.000	0.000	0.000	0.000	
		bottom	413.358	0.000	0.000	0.000	0.000	0.000	
7	Column 7	top	-401.646	0.000	-0.000	0.000	0.000	0.000	
		bottom	404.231	0.000	0.000	-0.000	0.000	0.000	
8	Column 8	top	-376.242	0.000	-0.000	-0.000	0.000	0.000	
		bottom	378.805	0.000	0.000	0.000	-0.000	0.000	
9	Column 9	top	-464.345	0.000	-0.000	-0.000	0.000	0.000	
		bottom	466.930	0.000	0.000	-0.000	-0.000	0.000	
10	Column 10	top	-529.293	0.000	-0.000	-0.000	0.000	0.000	
		bottom	531.878	-0.000	0.000	0.000	-0.000	0.000	
11	Column 11	top	-524.212	0.000	-0.000	-0.000	0.000	0.000	
		bottom	526.797	0.000	0.000	0.000	0.000	0.000	
12	Column 12	top	-509.937	0.000	-0.000	-0.000	0.000	0.000	
		bottom	512.522	0.000	0.000	0.000	0.000	0.000	
13	Column 13	top	-424.779	-0.000	0.000	0.000	0.000	0.000	
		bottom	427.364	0.000	-0.000	0.000	0.000	0.000	
14	Column 14	top	-576.975	-0.000	-0.000	-0.000	0.000	0.000	
		bottom	579.560	0.000	0.000	0.000	0.000	0.000	
15	Column 15	top	-453.959	-0.000	-0.000	-0.000	0.000	0.000	
		bottom	456.544	0.000	0.000	0.000	0.000	0.000	
16	Column 16	top	-482.577	-0.000	-0.000	-0.000	0.000	0.000	
		bottom	485.163	0.000	0.000	0.000	0.000	0.000	
17	Column 17	top	-422.013	0.000	-0.000	0.000	0.000	0.000	
		bottom	424.599	0.000	0.000	-0.000	0.000	0.000	
18	Column 18	top	-311.249	0.000	-0.000	0.000	0.000	0.000	
		bottom	313.835	0.000	0.000	-0.000	0.000	0.000	
19	Column 19	top	-78.767	2.557	0.568	7.242	-32.602	0.000	
		bottom	81.350	-2.557	-0.568	0.000	-0.000	0.000	

20	Column 20	top	-162.162	3.321	0.251	3.199	-42.346	-0.000	
		bottom	163.884	-3.321	-0.251	0.000	0.000	0.000	

Initial									
#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft	
1	Column 1	top	-109.916	0.000	0.000	0.000	0.000	0.000	
		bottom	112.068	0.000	0.000	0.000	0.000	0.000	
2	Column 2	top	-78.241	0.000	-0.000	-0.000	0.000	0.000	
		bottom	80.392	0.000	-0.000	0.000	0.000	0.000	
3	Column 3	top	-110.656	0.000	0.000	0.000	0.000	0.000	

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		bottom	112.807	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4	Column 4	top	-97.300	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	99.451	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5	Column 5	top	-89.627	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	91.779	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6	Column 6	top	-112.263	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	114.415	0.000	0.000	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-111.513	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	113.664	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	Column 8	top	-98.139	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	101.290	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-129.237	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	131.388	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	Column 10	top	-149.166	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	151.317	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	Column 11	top	-148.532	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	150.683	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12	Column 12	top	-149.002	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	151.153	0.000	0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-109.073	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	111.225	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-160.748	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	162.900	0.000	0.000	0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-115.343	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	117.495	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-132.080	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	134.232	0.000	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-118.221	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	120.372	0.000	0.000	0.000	0.000	0.000	0.000	0.000
18	Column 18	top	-87.435	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	89.587	0.000	0.000	0.000	0.000	0.000	0.000	0.000
19	Column 19	top	-17.980	0.812	0.238	3.037	-10.359	0.000	0.000	0.000
		bottom	20.131	-0.812	-0.238	0.000	0.000	0.000	0.000	0.000

20	Column 20	top	-40.954	1.075	0.069	0.882	-13.708	0.000	0.000	0.000
		bottom	42.388	-1.075	-0.069	0.000	0.000	0.000	0.000	0.000

4: 1.2D-WX+L

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-519.424	0.000	-0.000	-0.000	0.000	0.000
		bottom	522.009	0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-371.768	0.000	0.000	-0.000	0.000	0.000
		bottom	374.354	0.000	0.000	-0.000	0.000	0.000
3	Column 3	top	-545.097	0.000	-0.000	0.000	0.000	0.000
		bottom	547.682	0.000	0.000	0.000	0.000	0.000
4	Column 4	top	-467.470	0.000	0.000	-0.000	0.000	0.000
		bottom	470.055	0.000	0.000	-0.000	0.000	0.000
5	Column 5	top	-406.996	0.000	0.000	-0.000	0.000	0.000
		bottom	409.581	0.000	0.000	-0.000	0.000	0.000

		bottom	409.581	-0.000	0.000	-0.000	0.000	0.000	0.000	0.000
6	Column 6	top	-541.095	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	543.681	0.000	0.000	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-532.642	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	535.228	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8	Column 8	top	-525.066	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	527.652	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-614.383	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	616.969	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	Column 10	top	-720.898	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	723.484	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	Column 11	top	-706.870	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	709.433	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12	Column 12	top	-688.301	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	690.886	0.000	0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-603.458	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	606.043	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-769.952	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	772.515	0.000	0.000	0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-629.581	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	632.166	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-637.719	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	640.281	0.000	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-568.207	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	570.793	0.000	0.000	0.000	0.000	0.000	0.000	0.000
18	Column 18	top	-412.729	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	415.314	0.000	0.000	0.000	0.000	0.000	0.000	0.000
19	Column 19	top	-92.305	3.427	0.643	8.194	-43.695	0.000	0.000	0.000
		bottom	94.886	-3.427	-0.643	0.000	0.000	0.000	0.000	0.000

20	Column 20	top	-200.674	4.558	0.326	4.157	-58.110	0.000	0.000	0.000
		bottom	202.396	-4.558	-0.326	0.000	0.000	0.000	0.000	0.000

4: 1.2D-WX+L

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-519.424	0.000	-0.000	-0.000	0.000	0.000
		bottom	522.009	0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-371.768	0.000	0.000	-0.000	0.000	0.000
		bottom	374.354	0.000	0.000	-0.000	0.000	0.000
3	Column 3	top	-545.097	0.000	0.000	0.000	0.000	0.000
		bottom	547.682	0.000	0.000	0.000	0.000	0.000
4	Column 4	top	-467.470	0.000	0.000	-0.000	0.000	0.000
		bottom	470.055	0.000	0.000	-0.000	0.000	0.000
5	Column 5	top	-406.996	0.000	0.000	-0.000	0.000	0.000
		bottom	409.581	0.000	0.000	-0.000	0.000	0.000
6	Column 6	top	-541.095	0.000	0.000	0.000	0.000	0.000
		bottom	543.681	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-532.642	0.000	0.000	-0.000	0.000	0.000

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		bottom	535.228	0.000	0.000	-0.000	0.000	0.000	0.000
8	Column 8	top	-525.066	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	527.652	0.000	0.000	0.000	0.000	-0.000	0.000
9	Column 9	top	-614.383	0.000	-0.000	0.000	0.000	0.000	0.000
		bottom	616.969	-0.000	0.000	-0.000	-0.000	0.000	0.000
10	Column 10	top	-720.898	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	723.484	-0.000	0.000	-0.000	-0.000	0.000	0.000
11	Column 11	top	-706.870	0.000	-0.000	0.000	0.000	0.000	0.000
		bottom	709.433	0.000	0.000	-0.000	0.000	0.000	0.000
12	Column 12	top	-688.301	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	690.886	0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-603.458	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	606.043	0.000	-0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-769.952	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	772.515	0.000	0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-629.581	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	632.166	0.000	-0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-637.719	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	640.281	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-568.207	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	570.793	-0.000	0.000	0.000	-0.000	0.000	0.000
18	Column 18	top	-412.729	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	415.314	0.000	0.000	0.000	0.000	0.000	0.000
19	Column 19	top	-92.305	3.427	0.643	8.194	-43.695	0.000	0.000
		bottom	94.886	-3.427	-0.643	0.000	-0.000	0.000	0.000

20	Column 20	top	-200.674	4.558	0.326	4.157	-58.110	-0.000	0.000
		bottom	202.396	-4.558	-0.326	0.000	0.000	0.000	0.000

4: 1.2D+WY+L

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-519.424	0.000	-0.000	-0.000	0.000	0.000
		bottom	522.009	-0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-371.768	0.000	0.000	-0.000	0.000	0.000
		bottom	374.354	0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-545.097	-0.000	0.000	0.000	0.000	0.000
		bottom	547.682	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-467.470	0.000	0.000	0.000	-0.000	0.000
		bottom	470.055	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-406.996	-0.000	0.000	0.000	0.000	0.000
		bottom	409.581	-0.000	-0.000	0.000	-0.000	0.000
6	Column 6	top	-541.095	0.000	0.000	0.000	0.000	0.000
		bottom	543.681	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-532.642	-0.000	0.000	0.000	0.000	0.000
		bottom	535.228	0.000	0.000	0.000	-0.000	0.000
8	Column 8	top	-525.066	0.000	0.000	0.000	0.000	0.000
		bottom	527.652	0.000	0.000	0.000	-0.000	0.000
9	Column 9	top	-614.383	0.000	-0.000	0.000	0.000	0.000
		bottom	616.969	-0.000	0.000	-0.000	0.000	0.000
10	Column 10	top	-720.898	0.000	0.000	0.000	-0.000	0.000
		bottom	723.484	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-706.870	0.000	-0.000	0.000	0.000	0.000
		bottom	709.433	0.000	0.000	0.000	0.000	0.000

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		bottom	616.969	-0.000	0.000	-0.000	-0.000	0.000	0.000
10	Column 10	top	-720.898	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	723.484	-0.000	0.000	0.000	-0.000	0.000	0.000
11	Column 11	top	-706.870	0.000	-0.000	0.000	0.000	0.000	0.000
		bottom	709.433	0.000	0.000	-0.000	0.000	0.000	0.000
12	Column 12	top	-688.301	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	690.886	0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-603.458	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	606.043	0.000	-0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-769.952	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	772.515	0.000	-0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-629.581	-0.000	0.000	-0.000	0.000	0.000	0.000
		bottom	632.166	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-637.719	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	640.281	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-568.207	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	570.793	-0.000	0.000	-0.000	-0.000	0.000	0.000
18	Column 18	top	-412.729	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	415.314	0.000	0.000	0.000	0.000	0.000	0.000
19	Column 19	top	-92.305	3.427	0.643	8.194	-43.695	0.000	0.000
		bottom	94.886	-3.427	-0.643	0.000	-0.000	0.000	0.000

20	Column 20	top	-200.674	4.558	0.326	4.157	-58.110	-0.000	0.000
		bottom	202.396	-4.558	-0.326	0.000	0.000	0.000	0.000

4: 1.2D-WY+L

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-519.424	0.000	-0.000	-0.000	0.000	0.000
		bottom	522.009	-0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-371.768	0.000	0.000	-0.000	0.000	0.000
		bottom	374.354	0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-545.097	-0.000	0.000	0.000	0.000	0.000
		bottom	547.682	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-467.470	0.000	0.000	0.000	-0.000	0.000
		bottom	470.055	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-406.996	-0.000	0.000	0.000	0.000	0.000
		bottom	409.581	-0.000	-0.000	0.000	-0.000	0.000
6	Column 6	top	-541.095	0.000	0.000	0.000	0.000	0.000
		bottom	543.681	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-532.642	-0.000	0.000	0.000	0.000	0.000
		bottom	535.228	0.000	0.000	0.000	-0.000	0.000
8	Column 8	top	-525.066	0.000	0.000	0.000	0.000	0.000
		bottom	527.652	0.000	0.000	0.000	-0.000	0.000
9	Column 9	top	-614.383	0.000	-0.000	0.000	0.000	0.000
		bottom	616.969	-0.000	0.000	-0.000	0.000	0.000
10	Column 10	top	-720.898	0.000	0.000	0.000	-0.000	0.000
		bottom	723.484	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-706.870	0.000	-0.000	0.000	0.000	0.000
		bottom	709.433	0.000	0.000	0.000	0.000	0.000

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29.5

Project Name: HOME2 - SSF Specific Data: MOMENT CAPACITY w/ DEMAND
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		bottom	709.433	0.000	0.000	0.000	-0.000	0.000	0.000	0.000
12	Column 12	top	-688.301	0.000	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	690.886	0.000	0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-603.458	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	606.043	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-769.952	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	772.515	0.000	0.000	0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-629.581	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	632.166	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-637.719	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	640.281	0.000	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-568.207	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	570.793	0.000	0.000	0.000	0.000	0.000	0.000	0.000
18	Column 18	top	-412.729	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	415.314	0.000	0.000	0.000	0.000	0.000	0.000	0.000
19	Column 19	top	-92.305	3.427	0.643	8.194	-43.695	0.000	0.000	0.000
		bottom	94.886	-3.427	-0.643	0.000	-0.000	0.000	0.000	0.000

20	Column 20	top	-200.674	4.558	0.326	4.157	-58.110	-0.000	0.000	0.000
		bottom	202.396	-4.558	-0.326	0.000	0.000	0.000	0.000	0.000

5: 1,2D-EQX+L

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-510.296	0.000	-0.000	-0.000	-0.000	0.000
		bottom	512.882	-0.000	0.000	-0.000	-0.000	0.000
2	Column 2	top	-353.941	0.000	0.000	-0.000	-0.000	0.000
		bottom	356.526	0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-543.748	-0.000	0.000	0.000	-0.000	0.000
		bottom	546.333	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-468.639	0.000	0.000	0.000	-0.000	0.000
		bottom	471.202	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-407.850	-0.000	0.000	0.000	0.000	0.000
		bottom	410.436	0.000	-0.000	0.000	-0.000	0.000
6	Column 6	top	-543.029	0.000	0.000	0.000	-0.000	0.000
		bottom	545.614	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-534.216	-0.000	0.000	0.000	-0.000	0.000
		bottom	536.779	0.000	0.000	0.000	0.000	0.000
8	Column 8	top	-535.857	0.000	0.000	0.000	-0.000	0.000
		bottom	538.443	0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-625.534	0.000	0.000	0.000	-0.000	0.000
		bottom	628.119	-0.000	0.000	0.000	0.000	0.000
10	Column 10	top	-708.871	0.000	-0.000	0.000	-0.000	0.000
		bottom	711.434	-0.000	0.000	0.000	0.000	0.000
11	Column 11	top	-676.184	0.000	-0.000	0.000	-0.000	0.000
		bottom	678.769	0.000	0.000	0.000	0.000	0.000
12	Column 12	top	-868.518	0.000	0.000	0.000	-0.000	0.000
		bottom	871.103	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-611.798	-0.000	0.000	0.000	0.000	0.000

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		bottom	614.383	0.000	-0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-764.467	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	767.029	0.000	-0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-636.595	-0.000	0.000	-0.000	0.000	0.000	0.000
		bottom	639.180	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-629.693	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	632.278	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-572.029	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	574.614	-0.000	0.000	-0.000	-0.000	0.000	0.000
18	Column 18	top	-405.332	0.000	-0.000	0.000	-0.000	0.000	0.000
		bottom	407.918	0.000	0.000	0.000	-0.000	0.000	0.000
19	Column 19	top	-93.161	3.562	0.855	8.348	-45.417	0.000	0.000
		bottom	95.742	-3.562	-0.855	0.000	-0.000	0.000	0.000

20	Column 20	top	-252.619	4.514	0.420	5.357	-57.550	-0.000	0.000
		bottom	254.350	-4.514	-0.420	0.000	0.000	0.000	0.000

5: 1,2D-EQX+L

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-528.528	-0.000	-0.000	-0.000	0.000	0.000
		bottom	531.114	0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-389.596	-0.000	0.000	-0.000	0.000	0.000
		bottom	392.181	0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-546.446	-0.000	0.000	0.000	-0.000	0.000
		bottom	549.009	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-466.323	0.000	0.000	0.000	-0.000	0.000
		bottom	468.909	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-406.164	-0.000	0.000	0.000	0.000	0.000
		bottom	408.727	-0.000	-0.000	0.000	-0.000	0.000
6	Column 6	top	-539.162	-0.000	0.000	0.000	0.000	0.000
		bottom	541.725	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-531.091	-0.000	0.000	0.000	0.000	0.000
		bottom	533.654	0.000	0.000	0.000	0.000	0.000
8	Column 8	top	-514.275	0.000	0.000	0.000	-0.000	0.000
		bottom	516.861	0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-603.255	-0.000	-0.000	-0.000	0.000	0.000
		bottom	605.818	0.000	0.000	-0.000	-0.000	0.000
10	Column 10	top	-732.926	-0.000	0.000	0.000	0.000	0.000
		bottom	735.511	0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-737.534	0.000	-0.000	-0.000	-0.000	0.000
		bottom	740.120	0.000	0.000	-0.000	0.000	0.000
12	Column 12	top	-708.084	0.000	0.000	0.000	-0.000	0.000
		bottom	710.669	0.000	0.000	-0.000	0.000	0.000
13	Column 13	top	-595.117	-0.000	0.000	0.000	0.000	0.000
		bottom	597.702	0.000	-0.000	-0.000	0.000	0.000
14	Column 14	top	-775.437	-0.000	0.000	0.000	0.000	0.000
		bottom	778.000	0.000	-0.000	-0.000	0.000	0.000
15	Column 15	top	-622.566	-0.000	-0.000	-0.000	0.000	0.000

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Project Name: HOME2 - SSF Specific Data: MOMENT CAPACITY w/ DEMAND
Date of execution: November 03, 2017 File Name: h2ssf-Adapt-171020-NoPorteCo.adm
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		bottom	625.129	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-645.722	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	0.000
		bottom	648.307	0.000	0.000	0.000	-0.000	0.000	0.000	0.000
17	Column 17	top	-564.386	-0.000	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	566.971	-0.000	0.000	0.000	-0.000	-0.000	0.000	0.000
18	Column 18	top	-420.147	-0.000	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	422.710	0.000	0.000	0.000	-0.000	0.000	0.000	0.000
19	Column 19	top	-91.448	3.292	0.631	8.040	-41.974	0.000	0.000	0.000
		bottom	94.029	-3.292	-0.631	0.000	-0.000	-0.000	0.000	0.000

20	Column 20	top	-148.719	4.601	0.232	2.957	-58.670	-0.000	-0.000	0.000
		bottom	150.441	-4.601	-0.232	-0.000	0.000	0.000	0.000	0.000

5: 1.2D-EQY+L

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-530.732	0.000	-0.000	-0.000	0.000	0.000
		bottom	533.317	-0.000	0.000	-0.000	-0.000	0.000
2	Column 2	top	-363.698	0.000	0.000	-0.000	0.000	0.000
		bottom	366.193	-0.000	0.000	-0.000	0.000	0.000
3	Column 3	top	-553.078	-0.000	0.000	0.000	0.000	0.000
		bottom	555.640	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-458.568	0.000	0.000	0.000	-0.000	0.000
		bottom	461.153	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-406.659	-0.000	0.000	0.000	0.000	0.000
		bottom	408.244	-0.000	-0.000	0.000	-0.000	0.000
6	Column 6	top	-549.076	0.000	0.000	0.000	0.000	0.000
		bottom	551.661	-0.000	0.000	0.000	-0.000	0.000
7	Column 7	top	-523.785	-0.000	0.000	-0.000	0.000	0.000
		bottom	526.370	0.000	-0.000	0.000	-0.000	0.000
8	Column 8	top	-532.777	0.000	0.000	0.000	-0.000	0.000
		bottom	535.363	0.000	0.000	0.000	-0.000	0.000
9	Column 9	top	-607.010	0.000	-0.000	-0.000	0.000	0.000
		bottom	609.595	-0.000	0.000	-0.000	-0.000	0.000
10	Column 10	top	-703.026	0.000	0.000	0.000	0.000	0.000
		bottom	705.611	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-707.837	0.000	0.000	0.000	-0.000	0.000
		bottom	710.422	0.000	0.000	0.000	-0.000	0.000
12	Column 12	top	-672.856	0.000	0.000	-0.000	0.000	0.000
		bottom	675.442	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-627.332	-0.000	0.000	0.000	0.000	0.000
		bottom	629.918	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	-794.906	-0.000	0.000	0.000	0.000	0.000
		bottom	797.469	0.000	-0.000	0.000	0.000	0.000
15	Column 15	top	-631.469	-0.000	-0.000	-0.000	0.000	0.000
		bottom	634.054	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-637.831	-0.000	0.000	0.000	0.000	0.000
		bottom	640.394	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-561.171	0.000	0.000	0.000	-0.000	0.000

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Project Name: HOME2 - SSF Specific Data: MOMENT CAPACITY w/ DEMAND
Date of execution: November 03, 2017 File Name: h2ssf-Adapt-171020-NoPorteCo.adm
FLOOR-PRO 2016

		bottom	563.734	-0.000	0.000	-0.000	-0.000	0.000	0.000
18	Column 18	top	-388.090	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	390.652	0.000	0.000	-0.000	0.000	0.000	0.000
19	Column 19	top	-92.757	3.483	0.847	10.796	-44.411	0.000	0.000
		bottom	95.340	-3.483	-0.847	0.000	-0.000	0.000	0.000

20	Column 20	top	-201.212	4.626	0.512	6.532	-58.978	-0.000	0.000
		bottom	202.934	-4.626	-0.512	0.000	0.000	0.000	0.000

5: 1.2D-EQY+L

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-508.093	-0.000	-0.000	-0.000	0.000	0.000
		bottom	510.678	-0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-379.951	0.000	0.000	-0.000	-0.000	0.000
		bottom	382.514	0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-537.116	-0.000	0.000	0.000	0.000	0.000
		bottom	539.701	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-476.372	0.000	0.000	0.000	-0.000	0.000
		bottom	478.958	0.000	-0.000	-0.000	0.000	0.000
5	Column 5	top	-407.356	-0.000	0.000	-0.000	0.000	0.000
		bottom	409.941	-0.000	0.000	0.000	-0.000	0.000
6	Column 6	top	-533.115	-0.000	0.000	-0.000	0.000	0.000
		bottom	535.700	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-541.500	-0.000	-0.000	-0.000	0.000	0.000
		bottom	544.085	0.000	0.000	-0.000	0.000	0.000
8	Column 8	top	-517.355	0.000	0.000	0.000	-0.000	0.000
		bottom	519.941	0.000	0.000	-0.000	-0.000	0.000
9	Column 9	top	-621.757	-0.000	-0.000	-0.000	0.000	0.000
		bottom	624.342	-0.000	0.000	-0.000	-0.000	0.000
10	Column 10	top	-738.771	0.000	-0.000	-0.000	0.000	0.000
		bottom	741.334	0.000	0.000	-0.000	-0.000	0.000
11	Column 11	top	-705.881	0.000	-0.000	-0.000	0.000	0.000
		bottom	708.466	0.000	0.000	0.000	0.000	0.000
12	Column 12	top	-703.745	0.000	0.000	0.000	-0.000	0.000
		bottom	706.331	0.000	0.000	-0.000	0.000	0.000
13	Column 13	top	-579.583	-0.000	0.000	0.000	0.000	0.000
		bottom	582.168	0.000	-0.000	-0.000	0.000	0.000
14	Column 14	top	-744.998	-0.000	0.000	0.000	0.000	0.000
		bottom	747.561	0.000	-0.000	0.000	0.000	0.000
15	Column 15	top	-627.692	-0.000	-0.000	-0.000	0.000	0.000
		bottom	630.277	0.000	0.000	-0.000	0.000	0.000
16	Column 16	top	-637.584	-0.000	-0.000	-0.000	0.000	0.000
		bottom	640.169	0.000	0.000	-0.000	0.000	0.000
17	Column 17	top	-575.244	-0.000	0.000	0.000	0.000	0.000
		bottom	577.829	-0.000	0.000	-0.000	-0.000	0.000
18	Column 18	top	-437.390	0.000	-0.000	-0.000	-0.000	0.000
		bottom	439.976	0.000	0.000	-0.000	-0.000	0.000
19	Column 19	top	-91.851	3.371	0.439	5.591	-42.980	0.000

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2017-06
29.7

Project Name: HOME2 - SSF Specific Data: MOMENT CAPACITY w/ DEMAND
Date of execution: November 03, 2017 File Name: h2ssf-Adapt-171020-NoPorteCo.adm
FLOOR-PRO 2016

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
20	Column 20	top	-200.137	4.489	0.140	1.781	-57.243	-0.000
		bottom	201.859	-4.489	-0.140	-0.000	0.000	0.000
6: 0.9D+WX								
#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-118.558	0.000	0.000	0.000	0.000	0.000
		bottom	120.494	-0.000	0.000	0.000	0.000	0.000
2	Column 2	top	-84.349	0.000	-0.000	-0.000	0.000	0.000
		bottom	86.284	0.000	-0.000	0.000	0.000	0.000
3	Column 3	top	-120.536	0.000	0.000	0.000	0.000	0.000
		bottom	122.472	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-105.135	0.000	0.000	0.000	-0.000	0.000
		bottom	107.073	0.000	0.000	0.000	0.000	0.000
5	Column 5	top	-95.499	-0.000	0.000	0.000	0.000	0.000
		bottom	97.437	0.000	-0.000	0.000	0.000	0.000
6	Column 6	top	-121.741	0.000	0.000	0.000	0.000	0.000
		bottom	123.677	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-120.449	0.000	0.000	0.000	0.000	0.000
		bottom	122.384	0.000	-0.000	0.000	0.000	0.000
8	Column 8	top	-108.520	0.000	0.000	0.000	-0.000	0.000
		bottom	110.456	-0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-139.508	0.000	0.000	0.000	0.000	0.000
		bottom	141.444	-0.000	-0.000	0.000	-0.000	0.000
10	Column 10	top	-160.521	0.000	0.000	0.000	0.000	0.000
		bottom	162.459	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-159.629	0.000	0.000	0.000	-0.000	0.000
		bottom	161.564	0.000	0.000	0.000	0.000	0.000
12	Column 12	top	-158.938	0.000	0.000	0.000	-0.000	0.000
		bottom	160.874	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-120.208	-0.000	0.000	0.000	0.000	0.000
		bottom	122.144	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	-173.479	0.000	0.000	0.000	-0.000	0.000
		bottom	175.415	0.000	-0.000	0.000	0.000	0.000
15	Column 15	top	-127.476	0.000	0.000	0.000	-0.000	0.000
		bottom	129.412	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-143.179	-0.000	0.000	0.000	0.000	0.000
		bottom	145.115	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-127.411	0.000	0.000	0.000	0.000	0.000
		bottom	129.347	0.000	0.000	0.000	0.000	0.000
18	Column 18	top	-94.166	0.000	0.000	0.000	0.000	0.000
		bottom	96.102	0.000	-0.000	0.000	0.000	0.000
19	Column 19	top	-20.471	0.850	0.236	3.003	-10.836	0.000
		bottom	22.407	-0.850	-0.236	0.000	-0.000	0.000

#	Label	Location	Axial	Fr	Fs	Mrr	Mss	Mzz
20	Column 20	top	-45.335	1.120	0.075	0.955	-14.280	-0.000

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Project Name: HOME2 - SSF Specific Data: MOMENT CAPACITY w/ DEMAND
Date of execution: November 03, 2017 File Name: h2ssf-Adapt-171020-NoPorteCo.adm
FLOOR-PRO 2016

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
6: 0.9D-WX								
1	Column 1	top	-118.558	0.000	0.000	-0.000	0.000	0.000
		bottom	120.494	-0.000	0.000	0.000	0.000	0.000
2	Column 2	top	-84.349	0.000	-0.000	-0.000	0.000	0.000
		bottom	86.284	0.000	-0.000	0.000	0.000	0.000
3	Column 3	top	-120.536	0.000	0.000	0.000	0.000	0.000
		bottom	122.472	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-105.135	0.000	0.000	0.000	-0.000	0.000
		bottom	107.073	0.000	0.000	0.000	0.000	0.000
5	Column 5	top	-95.499	-0.000	0.000	0.000	0.000	0.000
		bottom	97.437	0.000	-0.000	0.000	0.000	0.000
6	Column 6	top	-121.741	0.000	0.000	0.000	0.000	0.000
		bottom	123.677	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-120.449	0.000	0.000	0.000	0.000	0.000
		bottom	122.384	0.000	0.000	-0.000	0.000	0.000
8	Column 8	top	-108.520	0.000	0.000	0.000	-0.000	0.000
		bottom	110.456	-0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-139.508	0.000	0.000	0.000	0.000	0.000
		bottom	141.444	-0.000	-0.000	0.000	-0.000	0.000
10	Column 10	top	-160.521	0.000	0.000	0.000	0.000	0.000
		bottom	162.459	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-159.629	0.000	0.000	0.000	-0.000	0.000
		bottom	161.564	0.000	0.000	0.000	0.000	0.000
12	Column 12	top	-158.938	0.000	0.000	0.000	-0.000	0.000
		bottom	160.874	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-120.208	-0.000	0.000	0.000	0.000	0.000
		bottom	122.144	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	-173.479	0.000	0.000	0.000	-0.000	0.000
		bottom	175.415	0.000	-0.000	0.000	0.000	0.000
15	Column 15	top	-127.476	0.000	0.000	0.000	-0.000	0.000
		bottom	129.412	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-143.179	-0.000	0.000	0.000	0.000	0.000
		bottom	145.115	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-127.411	0.000	0.000	0.000	0.000	0.000
		bottom	129.347	0.000	0.000	0.000	0.000	0.000
18	Column 18	top	-94.166	0.000	0.000	0.000	0.000	0.000
		bottom	96.102	0.000	-0.000	0.000	0.000	0.000
19	Column 19	top	-20.471	0.850	0.236	3.003	-10.836	0.000
		bottom	22.407	-0.850	-0.236	0.000	-0.000	0.000

#	Label	Location	Axial	Fr	Fs	Mrr	Mss	Mzz
20	Column 20	top	-45.335	1.120	0.075	0.955	-14.280	-0.000
		bottom	46.626	-1.120	-0.075	0.000	0.000	0.000

6: 0.9D+WX

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2017-06
25.8

Project Name: HOME2 - SSF
Date of execution: November 03, 2017
Specific Data: MOMENT CAPACITY w/ DEMAND
File Name: h2ssf-Adapt-171020-NoPorteCo.adm
FLOOR-PRO 2016

			k	k	k	k	k-ft	k-ft	k-ft
1	Column 1	top	-118.558	0.000	0.000	-0.000	0.000	0.000	0.000
		bottom	120.494	-0.000	0.000	0.000	0.000	0.000	0.000
2	Column 2	top	-84.349	0.000	-0.000	0.000	0.000	0.000	0.000
		bottom	86.284	0.000	-0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-120.536	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	122.472	0.000	-0.000	0.000	0.000	0.000	0.000
4	Column 4	top	-105.135	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	107.073	0.000	0.000	0.000	0.000	0.000	0.000
5	Column 5	top	-95.499	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	97.437	0.000	-0.000	0.000	0.000	0.000	0.000
6	Column 6	top	-121.741	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	123.677	0.000	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-139.508	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	141.444	0.000	-0.000	0.000	0.000	0.000	0.000
8	Column 8	top	-160.521	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	162.459	0.000	0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-159.629	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	161.564	0.000	-0.000	0.000	0.000	0.000	0.000
10	Column 10	top	-158.938	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	160.874	0.000	0.000	0.000	0.000	0.000	0.000
11	Column 11	top	-120.208	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	122.144	0.000	-0.000	0.000	0.000	0.000	0.000
12	Column 12	top	-173.479	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	175.415	0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-127.476	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	129.412	0.000	-0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-143.179	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	145.115	0.000	0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-127.411	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	129.347	0.000	-0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-94.166	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	96.102	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-20.471	0.850	0.236	3.003	-10.836	0.000	0.000
		bottom	22.407	-0.850	-0.236	0.000	-0.000	0.000	0.000

			Fr	Fs	Mrr	Miss	Mzz
20	Column 20	top	-45.335	1.120	0.075	0.955	-14.280
		bottom	46.626	-1.120	-0.075	0.000	0.000

#	Label	Location	Axial	Fr	Fs	Mrr	Miss	Mzz
1	Column 1	top	-118.558	0.000	0.000	-0.000	0.000	0.000
		bottom	120.494	-0.000	0.000	0.000	0.000	0.000
2	Column 2	top	-84.349	0.000	-0.000	0.000	0.000	0.000
		bottom	86.284	0.000	-0.000	0.000	0.000	0.000

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Project Name: HOME2 - SSF
Date of execution: November 03, 2017
Specific Data: MOMENT CAPACITY w/ DEMAND
File Name: h2ssf-Adapt-171020-NoPorteCo.adm
FLOOR-PRO 2016

			k	k	k	k	k-ft	k-ft	k-ft
3	Column 3	bottom	86.284	0.000	0.000	-0.000	0.000	0.000	0.000
		top	-120.536	0.000	0.000	0.000	0.000	0.000	0.000
4	Column 4	bottom	122.472	0.000	-0.000	0.000	0.000	0.000	0.000
		top	-105.135	0.000	0.000	0.000	0.000	0.000	0.000
5	Column 5	bottom	107.073	0.000	0.000	0.000	0.000	0.000	0.000
		top	-95.499	0.000	0.000	0.000	0.000	0.000	0.000
6	Column 6	bottom	97.437	0.000	-0.000	0.000	0.000	0.000	0.000
		top	-121.741	0.000	0.000	0.000	0.000	0.000	0.000
7	Column 7	bottom	123.677	0.000	0.000	0.000	0.000	0.000	0.000
		top	-120.449	0.000	0.000	0.000	0.000	0.000	0.000
8	Column 8	bottom	122.384	0.000	0.000	-0.000	0.000	0.000	0.000
		top	-108.520	0.000	0.000	0.000	0.000	0.000	0.000
9	Column 9	bottom	110.456	0.000	0.000	0.000	0.000	0.000	0.000
		top	-139.508	0.000	0.000	0.000	0.000	0.000	0.000
10	Column 10	bottom	141.444	0.000	-0.000	0.000	0.000	0.000	0.000
		top	-160.521	0.000	0.000	0.000	0.000	0.000	0.000
11	Column 11	bottom	162.459	0.000	0.000	0.000	0.000	0.000	0.000
		top	-159.629	0.000	0.000	0.000	0.000	0.000	0.000
12	Column 12	bottom	161.564	0.000	0.000	0.000	0.000	0.000	0.000
		top	-158.938	0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	bottom	160.874	0.000	0.000	0.000	0.000	0.000	0.000
		top	-120.208	0.000	0.000	0.000	0.000	0.000	0.000
14	Column 14	bottom	122.144	0.000	-0.000	0.000	0.000	0.000	0.000
		top	-173.479	0.000	0.000	0.000	0.000	0.000	0.000
15	Column 15	bottom	175.415	0.000	0.000	0.000	0.000	0.000	0.000
		top	-127.476	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	bottom	129.412	0.000	0.000	0.000	0.000	0.000	0.000
		top	-143.179	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	bottom	145.115	0.000	0.000	0.000	0.000	0.000	0.000
		top	-127.411	0.000	0.000	0.000	0.000	0.000	0.000
18	Column 18	bottom	129.347	0.000	0.000	0.000	0.000	0.000	0.000
		top	-94.166	0.000	0.000	0.000	0.000	0.000	0.000
19	Column 19	bottom	96.102	0.000	0.000	0.000	0.000	0.000	0.000
		top	-20.471	0.850	0.236	3.003	-10.836	0.000	0.000
		bottom	22.407	-0.850	-0.236	0.000	-0.000	0.000	0.000

			Fr	Fs	Mrr	Miss	Mzz
20	Column 20	top	-45.335	1.120	0.075	0.955	-14.280
		bottom	46.626	-1.120	-0.075	0.000	0.000

7: 0.9D+EQX

#	Label	Location	Axial	Fr	Fs	Mrr	Miss	Mzz
1	Column 1	top	-109.442	0.000	0.000	-0.000	0.000	0.000
		bottom	111.378	-0.000	0.000	0.000	0.000	0.000
2	Column 2	top	-66.521	0.000	-0.000	0.000	0.000	0.000
		bottom	68.457	0.000	-0.000	0.000	0.000	0.000
3	Column 3	top	-119.197	0.000	0.000	0.000	0.000	0.000
		bottom	121.132	-0.000	0.000	0.000	0.000	0.000
4	Column 4	top	-106.288	0.000	0.000	0.000	0.000	0.000

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2017-06
25.9

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		bottom	108.224	0.000	0.000	-0.000	-0.000	0.000
5	Column 5	top	-96.347	-0.000	0.000	0.000	0.000	0.000
		bottom	98.282	0.000	-0.000	0.000	-0.000	0.000
6	Column 6	top	-123.684	0.000	0.000	-0.000	0.000	0.000
		bottom	125.619	-0.000	0.000	0.000	-0.000	0.000
7	Column 7	top	-122.009	0.000	0.000	-0.000	-0.000	0.000
		bottom	123.947	-0.000	0.000	0.000	-0.000	0.000
8	Column 8	top	-119.302	0.000	0.000	-0.000	-0.000	0.000
		bottom	121.240	-0.000	0.000	0.000	-0.000	0.000
9	Column 9	top	-150.647	0.000	0.000	-0.000	-0.000	0.000
		bottom	152.585	-0.000	-0.000	0.000	-0.000	0.000
10	Column 10	top	-148.487	0.000	-0.000	-0.000	-0.000	0.000
		bottom	150.425	-0.000	-0.000	0.000	-0.000	0.000
11	Column 11	top	-128.944	0.000	0.000	-0.000	-0.000	0.000
		bottom	130.880	-0.000	-0.000	0.000	-0.000	0.000
12	Column 12	top	-139.155	0.000	0.000	-0.000	-0.000	0.000
		bottom	141.093	-0.000	0.000	0.000	-0.000	0.000
13	Column 13	top	-128.542	0.000	-0.000	0.000	0.000	0.000
		bottom	130.480	0.000	0.000	-0.000	0.000	0.000
14	Column 14	top	-167.992	0.000	0.000	-0.000	-0.000	0.000
		bottom	169.929	0.000	-0.000	0.000	0.000	0.000
15	Column 15	top	-134.499	0.000	0.000	-0.000	-0.000	0.000
		bottom	136.435	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-135.169	0.000	0.000	-0.000	-0.000	0.000
		bottom	137.105	0.000	-0.000	0.000	-0.000	0.000
17	Column 17	top	-131.231	0.000	0.000	-0.000	-0.000	0.000
		bottom	133.166	0.000	0.000	-0.000	-0.000	0.000
18	Column 18	top	-86.765	0.000	-0.000	-0.000	-0.000	0.000
		bottom	88.703	-0.000	0.000	-0.000	-0.000	0.000
19	Column 19	top	-21.328	0.985	0.248	3.157	-12.558	0.000
		bottom	23.263	-0.985	-0.248	0.000	-0.000	0.000

20	Column 20	top	-97.291	1.076	0.169	2.155	-13.720	-0.000
		bottom	98.581	-1.076	-0.169	0.000	0.000	0.000

7: 0.9D-EQX

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-127.674	-0.000	0.000	-0.000	0.000	0.000
		bottom	129.612	0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-102.178	-0.000	-0.000	-0.000	0.000	0.000
		bottom	104.114	0.000	-0.000	0.000	0.000	0.000
3	Column 3	top	-121.876	0.000	0.000	0.000	0.000	0.000
		bottom	123.812	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-103.981	-0.000	0.000	0.000	0.000	0.000
		bottom	105.919	-0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-94.654	-0.000	0.000	0.000	0.000	0.000
		bottom	96.590	0.000	-0.000	0.000	0.000	0.000
6	Column 6	top	-119.799	-0.000	0.000	0.000	0.000	0.000

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7	Column 7	bottom	121.735	0.000	0.000	0.000	0.000	0.000
		top	-118.889	-0.000	0.000	0.000	0.000	0.000
8	Column 8	bottom	120.824	0.000	0.000	-0.000	0.000	0.000
		top	-97.736	0.000	0.000	0.000	-0.000	0.000
9	Column 9	bottom	99.674	-0.000	0.000	0.000	-0.000	0.000
		top	-128.367	-0.000	-0.000	0.000	0.000	0.000
10	Column 10	bottom	130.304	-0.000	-0.000	-0.000	0.000	0.000
		top	-172.555	-0.000	0.000	0.000	0.000	0.000
11	Column 11	bottom	174.493	-0.000	0.000	0.000	-0.000	0.000
		top	-190.311	-0.000	0.000	0.000	0.000	0.000
12	Column 12	bottom	192.249	0.000	0.000	-0.000	0.000	0.000
		top	-178.719	-0.000	0.000	0.000	-0.000	0.000
13	Column 13	bottom	180.655	0.000	-0.000	-0.000	0.000	0.000
		top	-111.872	-0.000	0.000	0.000	0.000	0.000
14	Column 14	bottom	113.808	0.000	-0.000	-0.000	0.000	0.000
		top	-178.965	0.000	0.000	0.000	-0.000	0.000
15	Column 15	bottom	180.902	0.000	-0.000	0.000	0.000	0.000
		top	-120.451	-0.000	0.000	-0.000	-0.000	0.000
16	Column 16	bottom	122.389	0.000	0.000	0.000	0.000	0.000
		top	-151.187	-0.000	-0.000	0.000	0.000	0.000
17	Column 17	bottom	153.123	0.000	0.000	-0.000	0.000	0.000
		top	-123.592	-0.000	0.000	0.000	0.000	0.000
18	Column 18	bottom	125.527	-0.000	0.000	-0.000	0.000	0.000
		top	-101.565	-0.000	0.000	0.000	0.000	0.000
19	Column 19	bottom	103.503	0.000	0.000	-0.000	0.000	0.000
		top	-19.614	0.715	0.223	2.849	-9.114	0.000
		bottom	21.550	-0.715	-0.223	0.000	-0.000	0.000

20	Column 20	top	6.621	1.164	-0.019	-0.246	-14.840	0.000
		bottom	-5.330	-1.164	0.019	-0.000	-0.000	-0.000

7: 0.9D+EQY

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-129.875	0.000	0.000	-0.000	0.000	0.000
		bottom	131.811	-0.000	0.000	0.000	-0.000	0.000
2	Column 2	top	-76.181	-0.000	0.000	-0.000	0.000	0.000
		bottom	78.117	0.000	-0.000	0.000	0.000	0.000
3	Column 3	top	-128.508	0.000	0.000	0.000	-0.000	0.000
		bottom	130.444	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-96.232	0.000	0.000	0.000	-0.000	0.000
		bottom	98.170	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-95.153	-0.000	0.000	0.000	0.000	0.000
		bottom	97.089	0.000	-0.000	0.000	-0.000	0.000
6	Column 6	top	-129.718	0.000	0.000	0.000	0.000	0.000
		bottom	131.653	-0.000	0.000	0.000	-0.000	0.000
7	Column 7	top	-111.582	0.000	0.000	0.000	-0.000	0.000
		bottom	113.518	0.000	-0.000	-0.000	-0.000	0.000
8	Column 8	top	-116.238	0.000	0.000	0.000	-0.000	0.000

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2017-06
29.10

		bottom	118.174	-0.000	0.000	0.000	-0.000	0.000	0.000
9	Column 9	top	-132.139	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	134.074	-0.000	0.000	0.000	-0.000	0.000	0.000
10	Column 10	top	-142.658	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	144.593	-0.000	0.000	0.000	-0.000	0.000	0.000
11	Column 11	top	-160.604	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	162.542	0.000	0.000	0.000	-0.000	0.000	0.000
12	Column 12	top	-143.494	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	145.430	-0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-144.076	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	146.012	0.000	0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-198.431	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	200.366	0.000	0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-129.369	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	131.305	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-143.294	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	145.230	0.000	0.000	0.000	-0.000	0.000	0.000
17	Column 17	top	-120.366	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	122.301	0.000	0.000	0.000	-0.000	0.000	0.000
18	Column 18	top	-69.505	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	71.442	0.000	0.000	0.000	-0.000	0.000	0.000
19	Column 19	top	-20.924	0.906	0.440	5.606	-11.552	0.000	0.000
		bottom	22.861	-0.906	-0.440	0.000	-0.000	0.000	0.000

20	Column 20	top	-45.872	1.188	0.261	3.330	-15.147	-0.000	0.000
		bottom	47.163	-1.188	-0.261	0.000	0.000	0.000	0.000

7: 0.9D-EQY

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Miss k-ft	Mzz k-ft
1	Column 1	top	-107.241	0.000	0.000	-0.000	0.000	0.000
		bottom	109.179	-0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-92.518	0.000	-0.000	-0.000	-0.000	0.000
		bottom	94.454	0.000	-0.000	0.000	0.000	0.000
3	Column 3	top	-112.565	-0.000	-0.000	0.000	0.000	0.000
		bottom	114.500	-0.000	0.000	0.000	0.000	0.000
4	Column 4	top	-114.037	0.000	0.000	0.000	-0.000	0.000
		bottom	115.975	0.000	0.000	-0.000	0.000	0.000
5	Column 5	top	-95.848	-0.000	-0.000	0.000	0.000	0.000
		bottom	97.783	0.000	-0.000	0.000	0.000	0.000
6	Column 6	top	-113.765	-0.000	-0.000	-0.000	0.000	0.000
		bottom	115.703	0.000	-0.000	0.000	0.000	0.000
7	Column 7	top	-129.315	-0.000	0.000	-0.000	0.000	0.000
		bottom	131.251	0.000	-0.000	0.000	0.000	0.000
8	Column 8	top	-100.803	0.000	-0.000	-0.000	-0.000	0.000
		bottom	102.738	-0.000	0.000	-0.000	0.000	0.000
9	Column 9	top	-146.877	-0.000	-0.000	-0.000	0.000	0.000
		bottom	148.813	-0.000	-0.000	-0.000	0.000	0.000
10	Column 10	top	-178.387	0.000	-0.000	-0.000	0.000	0.000

		bottom	180.322	-0.000	0.000	-0.000	-0.000	0.000	0.000
11	Column 11	top	-158.651	0.000	-0.000	-0.000	-0.000	0.000	0.000
		bottom	160.586	0.000	0.000	-0.000	0.000	0.000	0.000
12	Column 12	top	-174.383	0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	176.319	0.000	-0.000	-0.000	0.000	0.000	0.000
13	Column 13	top	-96.340	0.000	0.000	-0.000	-0.000	0.000	0.000
		bottom	98.276	0.000	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	-148.527	-0.000	-0.000	0.000	-0.000	0.000	0.000
		bottom	150.463	0.000	-0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-125.581	0.000	-0.000	0.000	-0.000	0.000	0.000
		bottom	127.519	0.000	0.000	-0.000	0.000	0.000	0.000
16	Column 16	top	-143.062	-0.000	-0.000	-0.000	0.000	0.000	0.000
		bottom	145.000	0.000	0.000	-0.000	0.000	0.000	0.000
17	Column 17	top	-134.457	-0.000	0.000	0.000	-0.000	0.000	0.000
		bottom	136.392	-0.000	0.000	-0.000	0.000	0.000	0.000
18	Column 18	top	-118.826	0.000	-0.000	-0.000	-0.000	0.000	0.000
		bottom	120.763	0.000	0.000	-0.000	-0.000	0.000	0.000
19	Column 19	top	-20.018	0.794	0.031	-10.120	0.000	0.000	0.000
		bottom	21.954	-0.794	-0.031	-0.000	-0.000	0.000	0.000

20	Column 20	top	-44.798	1.052	-0.111	-1.421	-13.413	0.000	0.000
		bottom	46.088	-1.052	0.111	-0.000	0.000	-0.000	-0.000

Sustained Load

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Miss k-ft	Mzz k-ft
1	Column 1	top	-365.788	0.000	-0.000	-0.000	0.000	0.000
		bottom	367.947	-0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-260.690	0.000	0.000	-0.000	0.000	0.000
		bottom	262.825	0.000	0.000	-0.000	0.000	0.000
3	Column 3	top	-383.301	-0.000	0.000	0.000	0.000	0.000
		bottom	385.437	0.000	-0.000	0.000	0.000	0.000
4	Column 4	top	-327.413	0.000	0.000	0.000	-0.000	0.000
		bottom	329.572	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	-284.969	-0.000	0.000	0.000	-0.000	0.000
		bottom	287.105	0.000	-0.000	0.000	0.000	0.000
6	Column 6	top	-381.413	0.000	0.000	0.000	0.000	0.000
		bottom	383.571	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-374.016	-0.000	0.000	-0.000	0.000	0.000
		bottom	376.152	0.000	0.000	-0.000	0.000	0.000
8	Column 8	top	-358.167	0.000	0.000	0.000	-0.000	0.000
		bottom	360.326	0.000	-0.000	0.000	-0.000	0.000
9	Column 9	top	-431.972	0.000	-0.000	-0.000	-0.000	0.000
		bottom	434.131	-0.000	0.000	0.000	0.000	0.000
10	Column 10	top	-498.561	0.000	0.000	0.000	-0.000	0.000
		bottom	500.719	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-491.637	0.000	0.000	-0.000	-0.000	0.000
		bottom	493.795	0.000	0.000	-0.000	0.000	0.000
12	Column 12	top	-478.463	0.000	0.000	0.000	-0.000	0.000

2017-06-29.11

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		bottom	480.621	0.000	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-407.581	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	409.739	0.000	-0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-538.712	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	540.848	0.000	-0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-430.963	0.000	-0.000	-0.000	0.000	0.000	0.000
		bottom	433.141	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-448.676	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	450.834	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-395.531	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	397.689	-0.000	0.000	-0.000	-0.000	0.000	0.000
18	Column 18	top	-289.825	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	291.961	0.000	0.000	-0.000	0.000	0.000	0.000
19	Column 19	top	-69.700	2.392	0.496	6.320	-30.497	0.000	0.000
		bottom	71.852	-2.392	-0.496	0.000	-0.000	0.000	0.000

20	Column 20	top	-146.689	3.139	0.232	2.953	-40.018	-0.000	0.000
		bottom	148.123	-3.139	-0.232	0.000	0.000	0.000	0.000

Service - DEAD ONLY

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-328.065	0.000	-0.000	-0.000	0.000	0.000
		bottom	330.223	-0.000	0.000	0.000	0.000	0.000
2	Column 2	top	-233.038	0.000	0.000	-0.000	0.000	0.000
		bottom	235.196	0.000	0.000	-0.000	0.000	0.000
3	Column 3	top	-343.375	0.000	0.000	0.000	0.000	0.000
		bottom	345.533	0.000	-0.000	0.000	-0.000	0.000
4	Column 4	top	-292.478	0.000	0.000	0.000	0.000	0.000
		bottom	294.614	0.000	0.000	0.000	0.000	0.000
5	Column 5	top	-254.462	-0.000	0.000	0.000	0.000	0.000
		bottom	256.621	0.000	-0.000	0.000	0.000	0.000
6	Column 6	top	-342.318	0.000	0.000	0.000	0.000	0.000
		bottom	344.454	0.000	0.000	0.000	0.000	0.000
7	Column 7	top	-334.697	0.000	0.000	0.000	0.000	0.000
		bottom	336.855	0.000	-0.000	0.000	0.000	0.000
8	Column 8	top	-313.520	0.000	0.000	0.000	-0.000	0.000
		bottom	315.678	0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-386.966	0.000	0.000	0.000	0.000	0.000
		bottom	389.101	-0.000	0.000	0.000	-0.000	0.000
10	Column 10	top	-441.077	0.000	0.000	0.000	0.000	0.000
		bottom	443.235	-0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-436.851	0.000	0.000	0.000	0.000	0.000
		bottom	439.009	0.000	0.000	0.000	-0.000	0.000
12	Column 12	top	-424.958	0.000	0.000	0.000	0.000	0.000
		bottom	427.117	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-353.966	-0.000	0.000	0.000	0.000	0.000
		bottom	356.144	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	-480.801	-0.000	0.000	0.000	0.000	0.000

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		bottom	482.959	0.000	-0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-378.310	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	380.446	0.000	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-402.140	-0.000	0.000	0.000	0.000	0.000	0.000
		bottom	404.298	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-351.693	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	353.829	0.000	0.000	0.000	0.000	0.000	0.000
18	Column 18	top	-259.363	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	261.522	0.000	0.000	0.000	0.000	0.000	0.000
19	Column 19	top	-65.640	2.131	0.473	6.035	-27.169	0.000	0.000
		bottom	67.791	-2.131	-0.473	0.000	-0.000	0.000	0.000

20	Column 20	top	-135.136	2.768	0.209	2.666	-35.289	-0.000	0.000
		bottom	136.570	-2.768	-0.209	0.000	0.000	0.000	0.000

Service - LIVE ONLY

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-125.736	0.000	0.000	0.000	0.000	0.000
		bottom	125.736	0.000	0.000	-0.000	0.000	0.000
2	Column 2	top	-92.116	0.000	0.000	0.000	0.000	0.000
		bottom	92.116	0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-133.040	-0.000	0.000	0.000	0.000	0.000
		bottom	133.040	0.000	0.000	0.000	0.000	0.000
4	Column 4	top	-116.512	0.000	0.000	0.000	0.000	0.000
		bottom	116.512	0.000	-0.000	0.000	-0.000	0.000
5	Column 5	top	-101.641	0.000	0.000	0.000	0.000	0.000
		bottom	101.641	-0.000	0.000	0.000	-0.000	0.000
6	Column 6	top	-130.322	0.000	0.000	0.000	0.000	0.000
		bottom	130.322	0.000	-0.000	0.000	0.000	0.000
7	Column 7	top	-130.997	-0.000	0.000	0.000	0.000	0.000
		bottom	130.997	0.000	0.000	0.000	0.000	0.000
8	Column 8	top	-148.835	-0.000	0.000	0.000	0.000	0.000
		bottom	148.835	0.000	0.000	0.000	0.000	0.000
9	Column 9	top	-150.036	0.000	-0.000	0.000	0.000	0.000
		bottom	150.036	0.000	0.000	0.000	0.000	0.000
10	Column 10	top	-191.599	0.000	0.000	0.000	0.000	0.000
		bottom	191.599	0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	-182.640	0.000	-0.000	0.000	0.000	0.000
		bottom	182.640	0.000	0.000	0.000	0.000	0.000
12	Column 12	top	-178.353	0.000	-0.000	0.000	-0.000	0.000
		bottom	178.353	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-178.684	0.000	0.000	0.000	0.000	0.000
		bottom	178.684	0.000	0.000	0.000	0.000	0.000
14	Column 14	top	-192.970	0.000	0.000	0.000	0.000	0.000
		bottom	192.970	-0.000	0.000	0.000	-0.000	0.000
15	Column 15	top	-175.615	0.000	-0.000	0.000	0.000	0.000
		bottom	175.615	0.000	0.000	0.000	0.000	0.000
16	Column 16	top	-155.135	0.000	0.000	0.000	0.000	0.000

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		bottom	155.135	0.000	0.000	0.000	0.000	0.000	0.000
17	Column 17	top	-146.183	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	146.183	-0.000	0.000	0.000	-0.000	0.000	0.000
18	Column 18	top	-101.495	0.000	0.000	0.000	0.000	0.000	0.000
		bottom	101.495	0.000	0.000	0.000	0.000	0.000	0.000
19	Column 19	top	-13.537	0.870	0.075	0.952	-11.093	0.000	0.000
		bottom	13.537	-0.870	-0.075	0.000	-0.000	0.000	0.000

20	Column 20	top	-38.512	1.236	0.075	0.958	-15.764	-0.000	0.000
		bottom	38.512	-1.236	-0.075	0.000	0.000	0.000	0.000

Service - Seismic Only- X Dir

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	5.394	0.000	0.000	0.000	-0.000	0.000
		bottom	-5.394	-0.000	0.000	0.000	-0.000	0.000
2	Column 2	top	10.549	0.000	-0.000	-0.000	0.000	0.000
		bottom	-10.549	0.000	0.000	-0.000	-0.000	0.000
3	Column 3	top	0.793	0.000	-0.000	0.000	0.000	0.000
		bottom	-0.793	-0.000	0.000	-0.000	-0.000	0.000
4	Column 4	top	-0.682	0.000	-0.000	0.000	-0.000	0.000
		bottom	0.682	0.000	0.000	-0.000	0.000	0.000
5	Column 5	top	-0.501	0.000	-0.000	0.000	-0.000	0.000
		bottom	0.501	-0.000	0.000	-0.000	0.000	0.000
6	Column 6	top	-1.150	0.000	-0.000	0.000	0.000	0.000
		bottom	1.150	-0.000	0.000	-0.000	-0.000	0.000
7	Column 7	top	-0.924	0.000	-0.000	0.000	-0.000	0.000
		bottom	0.924	-0.000	0.000	-0.000	0.000	0.000
8	Column 8	top	-6.381	0.000	-0.000	-0.000	0.000	0.000
		bottom	6.381	0.000	0.000	-0.000	-0.000	0.000
9	Column 9	top	-6.592	0.000	0.000	0.000	-0.000	0.000
		bottom	6.592	0.000	0.000	0.000	-0.000	0.000
10	Column 10	top	7.121	0.000	-0.000	-0.000	0.000	0.000
		bottom	-7.121	0.000	0.000	0.000	-0.000	0.000
11	Column 11	top	18.156	0.000	0.000	0.000	-0.000	0.000
		bottom	-18.156	-0.000	0.000	0.000	0.000	0.000
12	Column 12	top	11.705	0.000	-0.000	0.000	0.000	0.000
		bottom	-11.705	0.000	0.000	0.000	-0.000	0.000
13	Column 13	top	-4.932	0.000	0.000	0.000	-0.000	0.000
		bottom	4.932	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	3.246	0.000	0.000	0.000	-0.000	0.000
		bottom	-3.246	0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-4.156	0.000	0.000	0.000	-0.000	0.000
		bottom	4.156	-0.000	0.000	0.000	0.000	0.000
16	Column 16	top	4.739	0.000	0.000	0.000	-0.000	0.000
		bottom	-4.739	0.000	-0.000	0.000	0.000	0.000
17	Column 17	top	-2.260	0.000	-0.000	-0.000	-0.000	0.000
		bottom	2.260	0.000	0.000	-0.000	0.000	0.000
18	Column 18	top	4.378	0.000	-0.000	-0.000	-0.000	0.000

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		bottom	-4.378	-0.000	0.000	0.000	-0.000	0.000	0.000
19	Column 19	top	-0.507	0.080	0.007	0.091	-1.019	0.000	0.000
		bottom	0.507	-0.080	-0.007	0.000	-0.000	0.000	0.000

20	Column 20	top	-30.743	-0.026	0.056	0.710	0.331	-0.000	0.000
		bottom	30.743	0.026	-0.056	0.000	0.000	0.000	0.000

Service - Seismic Only- Y Dir

#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
1	Column 1	top	-6.696	0.000	0.000	0.000	0.000	0.000
		bottom	6.696	-0.000	0.000	0.000	-0.000	0.000
2	Column 2	top	4.833	-0.000	0.000	0.000	0.000	0.000
		bottom	-4.833	0.000	0.000	0.000	0.000	0.000
3	Column 3	top	-4.717	0.000	0.000	0.000	-0.000	0.000
		bottom	4.717	-0.000	0.000	0.000	0.000	0.000
4	Column 4	top	5.268	0.000	0.000	0.000	0.000	0.000
		bottom	-5.268	0.000	-0.000	0.000	0.000	0.000
5	Column 5	top	0.206	0.000	0.000	0.000	0.000	0.000
		bottom	-0.206	0.000	-0.000	0.000	-0.000	0.000
6	Column 6	top	-4.719	0.000	0.000	0.000	0.000	0.000
		bottom	4.719	-0.000	0.000	0.000	-0.000	0.000
7	Column 7	top	5.246	0.000	0.000	0.000	-0.000	0.000
		bottom	-5.246	0.000	0.000	0.000	0.000	0.000
8	Column 8	top	-4.567	0.000	0.000	0.000	0.000	0.000
		bottom	4.567	-0.000	-0.000	0.000	0.000	0.000
9	Column 9	top	4.361	0.000	0.000	0.000	-0.000	0.000
		bottom	-4.361	-0.000	-0.000	0.000	0.000	0.000
10	Column 10	top	10.571	0.000	0.000	0.000	0.000	0.000
		bottom	-10.571	0.000	-0.000	0.000	-0.000	0.000
11	Column 11	top	-0.578	-0.000	0.000	0.000	0.000	0.000
		bottom	0.578	0.000	-0.000	0.000	0.000	0.000
12	Column 12	top	9.139	0.000	-0.000	-0.000	0.000	0.000
		bottom	-9.139	0.000	0.000	0.000	0.000	0.000
13	Column 13	top	-14.123	-0.000	0.000	0.000	0.000	0.000
		bottom	14.123	0.000	-0.000	0.000	0.000	0.000
14	Column 14	top	-14.765	0.000	0.000	0.000	-0.000	0.000
		bottom	14.765	-0.000	0.000	0.000	0.000	0.000
15	Column 15	top	-1.120	-0.000	0.000	0.000	0.000	0.000
		bottom	1.120	0.000	-0.000	0.000	0.000	0.000
16	Column 16	top	-0.068	-0.000	0.000	0.000	0.000	0.000
		bottom	0.068	0.000	-0.000	0.000	-0.000	0.000
17	Column 17	top	4.169	0.000	0.000	0.000	-0.000	0.000
		bottom	-4.169	0.000	0.000	0.000	0.000	0.000
18	Column 18	top	14.592	-0.000	0.000	0.000	0.000	0.000
		bottom	-14.592	0.000	0.000	0.000	0.000	0.000
19	Column 19	top	-0.268	0.033	0.121	1.540	-0.424	0.000
		bottom	0.268	-0.033	-0.121	0.000	-0.000	0.000

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20	Column 20	top	-0.318	0.040	0.110	1.406	-0.513	-0.000
		bottom	0.318	-0.040	-0.110	0.000	0.000	0.000

Long Term	#	Label	Location	Axial k	Fr k	Fs k	Mrr k-ft	Mss k-ft	Mzz k-ft
	1	Column 1	top	-1463.154	0.000	-0.000	-0.000	0.000	0.000
			bottom	1471.764	-0.000	0.000	0.000	0.000	0.000
	2	Column 2	top	-1042.736	0.000	0.000	-0.000	0.000	0.000
			bottom	1051.324	0.000	0.000	-0.000	0.000	0.000
	3	Column 3	top	-1533.159	-0.000	0.000	0.000	0.000	0.000
			bottom	1541.770	0.000	-0.000	0.000	0.000	0.000
	4	Column 4	top	-1309.698	0.000	0.000	0.000	-0.000	0.000
			bottom	1318.286	0.000	-0.000	0.000	0.000	0.000
	5	Column 5	top	-1139.832	-0.000	0.000	0.000	0.000	0.000
			bottom	1148.442	0.000	-0.000	0.000	-0.000	0.000
	6	Column 6	top	-1525.628	0.000	0.000	0.000	0.000	0.000
			bottom	1534.239	0.000	0.000	0.000	0.000	0.000
	7	Column 7	top	-1496.021	-0.000	0.000	-0.000	0.000	0.000
			bottom	1504.631	0.000	0.000	-0.000	0.000	0.000
	8	Column 8	top	-1432.714	0.000	0.000	0.000	-0.000	0.000
			bottom	1441.325	0.000	0.000	0.000	0.000	0.000
	9	Column 9	top	-1727.867	0.000	-0.000	-0.000	0.000	0.000
			bottom	1736.478	-0.000	0.000	-0.000	-0.000	0.000
	10	Column 10	top	-1994.245	0.000	0.000	0.000	0.000	0.000
			bottom	2002.855	-0.000	0.000	0.000	-0.000	0.000
	11	Column 11	top	-1966.571	0.000	0.000	0.000	-0.000	0.000
			bottom	1975.181	0.000	0.000	-0.000	0.000	0.000
	12	Column 12	top	-1913.853	0.000	0.000	0.000	-0.000	0.000
			bottom	1922.463	0.000	0.000	0.000	0.000	0.000
	13	Column 13	top	-1630.345	-0.000	0.000	0.000	0.000	0.000
			bottom	1638.955	0.000	-0.000	0.000	0.000	0.000
	14	Column 14	top	-2154.804	-0.000	0.000	0.000	0.000	0.000
			bottom	2163.414	0.000	-0.000	0.000	0.000	0.000
	15	Column 15	top	-1723.956	-0.000	-0.000	-0.000	0.000	0.000
			bottom	1732.566	0.000	0.000	0.000	0.000	0.000
	16	Column 16	top	-1794.748	-0.000	0.000	0.000	0.000	0.000
			bottom	1803.336	0.000	0.000	0.000	0.000	0.000
	17	Column 17	top	-1582.168	0.000	0.000	0.000	0.000	0.000
			bottom	1590.778	-0.000	0.000	-0.000	-0.000	0.000
	18	Column 18	top	-1159.278	0.000	0.000	0.000	0.000	0.000
			bottom	1167.888	0.000	0.000	-0.000	0.000	0.000
	19	Column 19	top	-2778.809	9.568	1.983	25.281	-121.985	0.000
			bottom	287.397	-9.568	-1.983	0.000	-0.000	0.000

20	Column 20	top	-586.754	12.555	0.926	11.812	-160.073	-0.000
		bottom	592.487	-12.555	-0.926	0.000	0.000	0.000

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29.14

Concrete Column

File = F:\HCPENG~1\ENGINE~1\ENERCA~1\2017-0-4.EC6
ENERCALC, INC. 1983-2017, Build:10.17.9.7, Ver:10.17.9.7

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: TYP. CONC. COL: 18" x 18"



Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

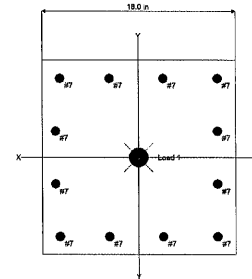
f'_c : Concrete 28 day strength = 5.0 ksi
 E = 3,122.0 ksi
Density = 150.0 pcf
 β = 0.80
 f_y - Main Rebar = 60.0 ksi
 E - Main Rebar = 29,000.0 ksi
Allow. Reinforcing Limits *ASTM A615 Bars Used*
Min. Reinf. = 1.0 %
Max. Reinf. = 8.0 %

Overall Column Height = 14.0 ft
End Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns:
X-X (width) axis:
Unbraced Length for X-X Axis buckling = 14.0 ft, $K = 1.0$
Y-Y (depth) axis:
Unbraced Length for X-X Axis buckling = 14.0 ft, $K = 1.0$

Column Cross Section

Column Dimensions: 18.0in Square Column, Column Edge to Rebar Edge Cover = 1.250in

Column Reinforcing: 4 - #7 bars @ corners,, 2.0 - #7 bars top & bottom between corner bars, 2.0 - #7 bars left & right between corner bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included: 4,725.0 lbs * Dead Load Factor

AXIAL LOADS...

Axial Load at 14.0 ft above base, $D = 468.0$, $L = 189.0$ k

DESIGN SUMMARY

Load Combination +1.40D
Location of max. above base 13.906 ft
Maximum Stress Ratio **0.716 : 1**
Ratio = $(P_u^2 + M_u^2)^{.5} / (\Phi P_n^2 + \Phi M_n^2)^{.5}$
 $P_u = 661.82$ k $\Phi * P_n = 924.77$ k
 $M_u - x = 0.0$ k-ft $\Phi * M_n - x = 0.0$ k-ft
 $M_u - y = 0.0$ k-ft $\Phi * M_n - y = 0.0$ k-ft
 M_u Angle = 0.0 deg
 M_u at Angle = 0.0 k-ft ΦM_n at Angle = 0.0 k-ft
 P_n & M_n values located at P_u - M_u vector intersection with capacity curve

Maximum SERVICE Load Reactions ..

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Deflections ...

Along Y-Y	0.0 in at	0.0 ft above base
for load combination :		
Along X-X	0.0 in at	0.0 ft above base
for load combination :		

General Section Information . $\phi = 0.650$ $\beta = 0.80$ $\theta = 0.80$

ρ : % Reinforcing	2.222 %	Rebar % Ok
Reinforcing Area	7.20 in ²	
Concrete Area	324.0 in ²	

Column Capacities ...

P_{nmax} : Nominal Max. Compressive Axial Capacity	1,778.40 k
P_{nmin} : Nominal Min. Tension Axial Capacity	k
ϕP_n , max : Usable Compressive Axial Capacity	924.77 k
ϕP_n , min : Usable Tension Axial Capacity	k

Governing Load Combination Results

Governing Factored Load Combination	Moment		Dist. from base ft	Axial Load		Bending Analysis k-ft						Utilization Ratio	
	X-X	Y-Y		P_u	$\phi * P_n$	δx	$\delta x * M_{ux}$	δy	$\delta y * M_{uy}$	Alpha (deg)	δM_u	ϕM_n	Ratio
+1.40D			13.91	661.82	924.77					0.000			0.716

Concrete Column

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: TYP. CONC. COL: 18" x 18"

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only						472.725					
+D+L						661.725					
L Only						189.000					

Maximum Moment Reactions

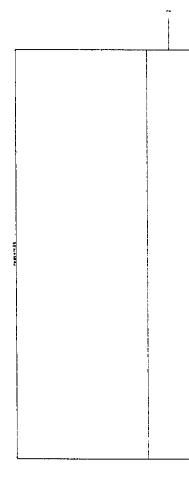
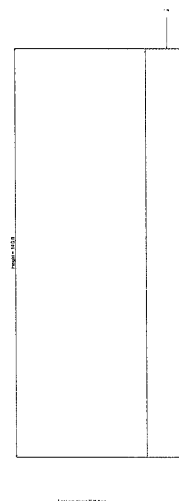
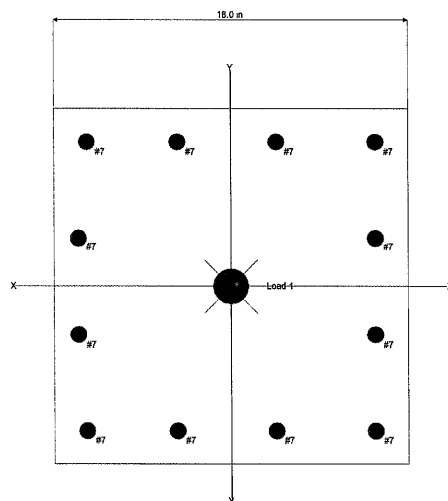
Note: Only non-zero reactions are listed.

Load Combination	Moment About X-X Axis			Moment About Y-Y Axis		
	@ Base	@ Top		@ Base	@ Top	
D Only			k-ft			k-ft
+D+L			k-ft			k-ft
L Only			k-ft			k-ft

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance	Max. Y-Y Deflection		Distance
D Only	0.0000	in	0.000	0.000	in	0.000
+D+L	0.0000	in	0.000	0.000	in	0.000
L Only	0.0000	in	0.000	0.000	in	0.000

Sketches



Interaction Diagrams

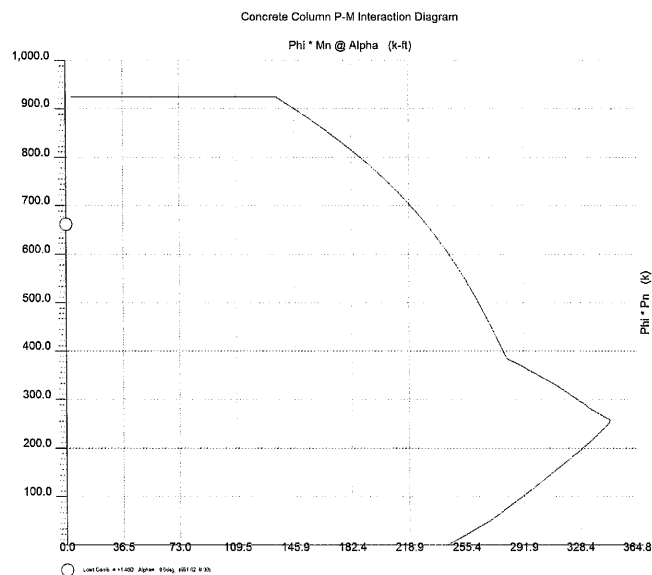
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ENERCALC, INC. 1983-2017, Build:10.17.9.7, Ver:10.17.9.7

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: TYP. CONC. COL: 18" x 18"



Concrete Column

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ENERCALC, INC. 1983-2017, Build:10.17.9.7, Ver:10.17.9.7

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: 12" x 18" COL



Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

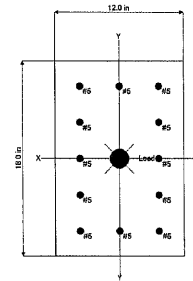
f'_c : Concrete 28 day strength = 5.0 ksi
E = 3,122.0 ksi
Density = 150.0 pcf
 β = 0.80
 f_y - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi
Allow. Reinforcing Limits *ASTM A615 Bars Used*
Min. Reinf. = 1.0 %
Max. Reinf. = 8.0 %

Overall Column Height = 14.0 ft
End Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns:
X-X (width) axis:
Unbraced Length for X-X Axis buckling = 14.0 ft, K = 1.0
Y-Y (depth) axis:
Unbraced Length for X-X Axis buckling = 14.0 ft, K = 1.0

Column Cross Section

Column Dimensions: 18.0in high x 12.0in Wide, Column Edge to Rebar Edge Cover = 2.0in

Column Reinforcing: 4 - #5 bars @ corners,, 1.0 - #5 bars top & bottom between corner bars, 3.0 - #5 bars left & right between corner bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included: 3,150.0 lbs * Dead Load Factor

AXIAL LOADS...

Axial Load at 14.0 ft above base, D = 181.0, L = 50.0 k

DESIGN SUMMARY

Load Combination +1.40D
Location of max. above base 13.906 ft
Maximum Stress Ratio 0.442 : 1
Ratio = $(P_u^2 + M_u^2)^{.5} / (\Phi P_n^2 + \Phi M_n^2)^{.5}$
 $P_u = 257.810$ k $\Phi * P_n = 584.38$ k
 $M_u - x = 0.0$ k-ft $\Phi * M_n - x = 0.0$ k-ft
 $M_u - y = 24.071$ k-ft $\Phi * M_n - y = 54.182$ k-ft
 M_u Angle = 90.0 deg
 M_u at Angle = 24.071 k-ft ΦM_n at Angle = 54.438 k-ft

P_n & M_n values located at P_u-M_u vector intersection with capacity curve

Column Capacities...

P_{nmax} : Nominal Max. Compressive Axial Capacity 1,125.39 k
 P_{nmin} : Nominal Min. Tension Axial Capacity k
 ΦP_n , max: Usable Compressive Axial Capacity 585.20 k
 ΦP_n , min: Usable Tension Axial Capacity k

Maximum SERVICE Load Reactions..

Top along Y-Y 0.0 k Bottom along Y-Y 0.0 k
Top along X-X 0.0 k Bottom along X-X 0.0 k

Maximum SERVICE Load Deflections...

Along Y-Y 0.0 in at 0.0 ft above base
for load combination:
Along X-X 0.0 in at 0.0 ft above base
for load combination:

General Section Information . $\phi = 0.650$ $\beta = 0.80$ $\theta = 0.80$

ρ : % Reinforcing 1.722 % Rebar % Ok
Reinforcing Area 3.720 in²
Concrete Area 216.0 in²

Governing Load Combination Results

Governing Factored Load Combination	Moment		Dist. from base ft	Axial Load k		Bending Analysis k-ft					Utilization		
	X-X	Y-Y		P_u	$\phi * P_n$	$\delta \times$	$\delta x * M_{ux}$	δy	$\delta y * M_{uy}$	Alpha (deg)	δM_u	ϕM_n	Ratio
+1.40D		M2,min	13.91	257.81	584.38			1.167	24.07	90.000	24.07	54.44	0.442

Concrete Column

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: 12" x 18" COL

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction		k	Y-Y Axis Reaction		Axial Reaction	My - End Moments		k-ft	Mx - End Moments	
	@ Base	@ Top		@ Base	@ Top	@ Base	@ Base	@ Top		@ Base	@ Top
D Only						184.150					
+D+L						234.150					
L Only						50.000					

Maximum Moment Reactions

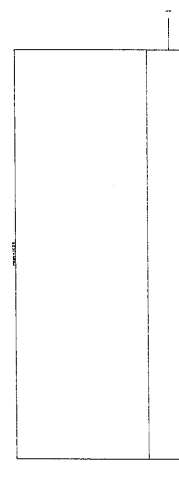
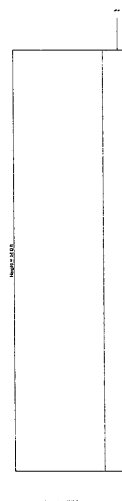
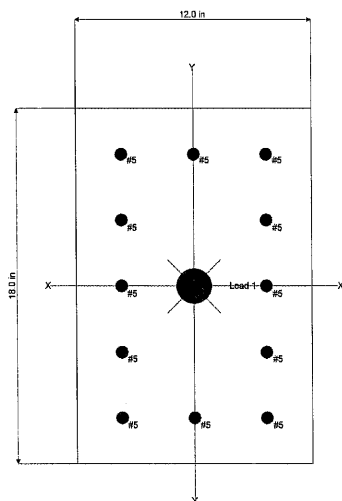
Note: Only non-zero reactions are listed.

Load Combination	Moment About X-X Axis			Moment About Y-Y Axis		
	@ Base	@ Top		@ Base	@ Top	
D Only			k-ft			k-ft
+D+L			k-ft			k-ft
L Only			k-ft			k-ft

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance	Max. Y-Y Deflection		Distance
D Only	0.0000	in	0.000 ft	0.000	in	0.000 ft
+D+L	0.0000	in	0.000 ft	0.000	in	0.000 ft
L Only	0.0000	in	0.000 ft	0.000	in	0.000 ft

Sketches



Interaction Diagrams

33.1

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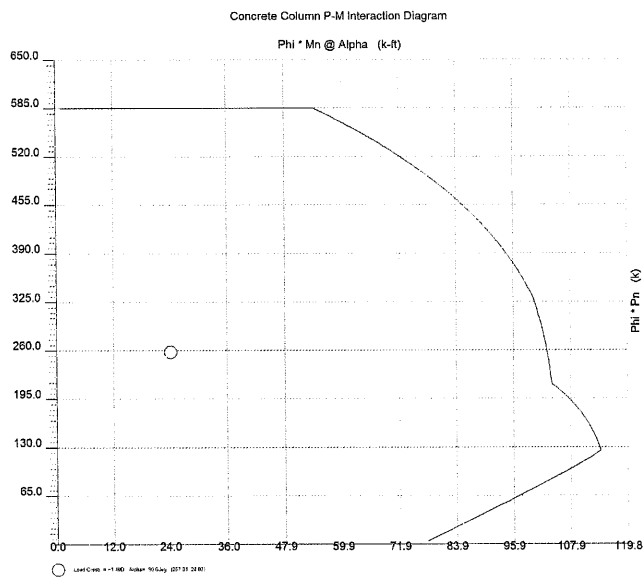
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Concrete Column

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: 12" x 18" COL



2017-06

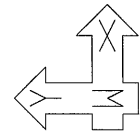
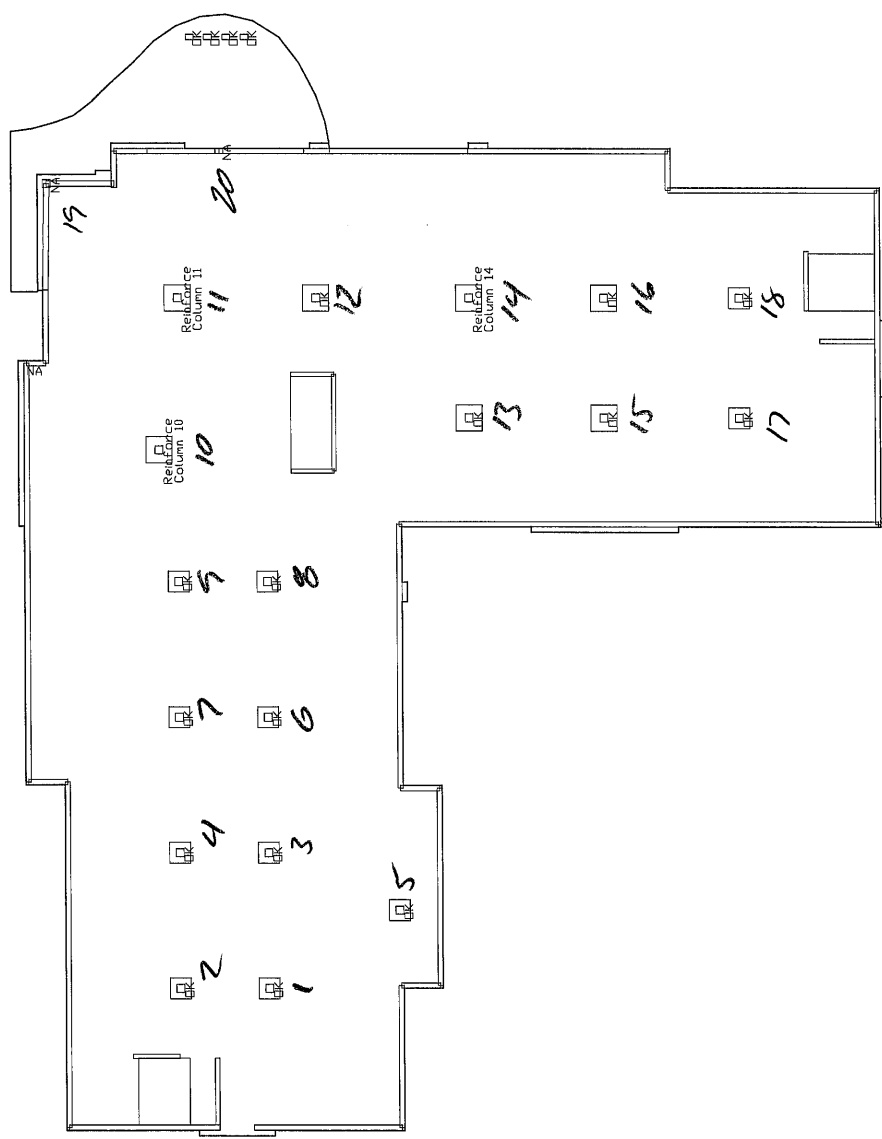
(39)

Wall Reactions Plan_Strength(Dead Load Only)

HOME2 - SSF PUNCHING SHEAR

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2017-06
(35)

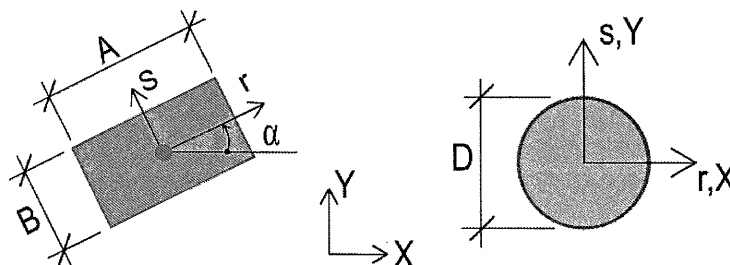
Project Name: HOME2 - SSF
Date of execution: May 28, 2017
2016

Specific Data: PUNCHING SHEAR
File Name: h2ssf-Adapt-170526-1.adm

FLOOR-PRO

180.60 PUNCHING SHEAR STRESS CHECK PARAMETERS

COLUMN
SECTION



Label	Condition	Axis	Effective depth	Design length rr	Design length ss	Design area	Section constant	Gamma
			in	in	in	in2	in4	
Column 1	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 1	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 2	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 2	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 3	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 3	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 4	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 4	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 5	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 5	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 6	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 6	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 7	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 7	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 8	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 8	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 9	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 9	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 10	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 10	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 11	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 11	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 12	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 12	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 13	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 13	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 14	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 14	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 15	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 15	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 16	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 16	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 17	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 17	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 18	Interior	rr	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 18	Interior	ss	13.25	31.25	31.25	1.66E+003	2.82E+005	0.40
Column 21	Edge	rr	8.25	16.13	26.25	4.83E+002	5.95E+004	0.46
Column 21	End	ss	8.25	16.13	26.25	4.83E+002	1.50E+004	0.34

180.90 SCHEDULE OF STUD RAILS FOR PUNCHING SHEAR REINFORCEMENT

Load Combination: Strength(Dead and Live)

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6
10	7	0.63	Distance	5.50	11.00	6.50*	13.00*	19.75*	26.25*
				(7)33.00*					
11	12	0.63	Distance	5.50	11.00	3.25*	6.50*	9.75*	13.00*
				(7)16.50*	(8)19.75*	(9)23.00*	(10)26.25*	(11)29.75*	(12)33.00*
14	8	0.63	Distance	5.50	11.00	6.50*	13.00*	19.75*	26.25*
				(7)33.00*	(8)39.50*				

Load Combination: Strength(Dead Load Only)

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: Strength (D+L-Skip-Max)

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: Strength (D+L-Skip-Min)

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 4: 1.2D+WX+L

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

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Load Combination: 4: 1.2D-WX+L

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 4: 1.2D+WY+L

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 4: 1.2D-WY+L

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 5: 1.2D+EQX+L

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 5: 1.2D-EQX+L

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 5: 1.2D+EQY+L

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6
14	12	0.63	Distance	5.50	11.00	3.25*	6.50*	9.75*	13.00*
				(7)16.50*	(8)19.75*	(9)23.00*	(10)26.25*	(11)29.75*	(12)33.00*

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2017-10-10
36

Load Combination: 5: 1.2D-EQY+L

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 6: 0.9D+WX

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 6: 0.9D+WX

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 6: 0.9D+WY

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 6: 0.9D-WY

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 7: 0.9D+EQX

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 7: 0.9D-EQX

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 7: 0.9D+EQY

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Load Combination: 7: 0.9D-EQY

Number of rails per side: 3

Column ID	Number of studs/rail	Stud diameter (in)	Studs	1	2	3	4	5	6

Notes:

Dimensions are in (in)

Distance = distance of the vertical bars from the respective column face

(7) 16.5 indicates layer 7 at distance 16.50 in from face of support.

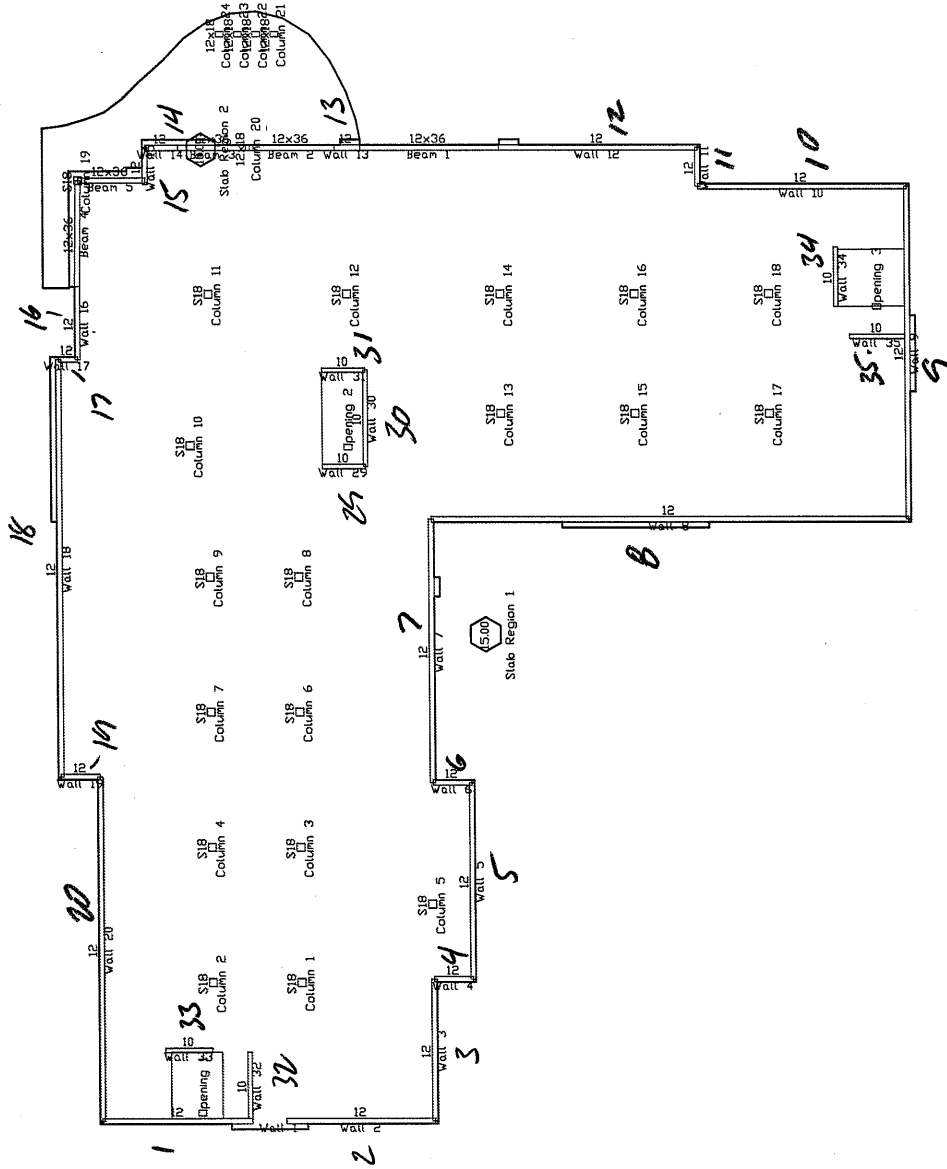
(7) 16.5" indicates layer 7 at distance 16.50 in from face of drop caps.

*** = exceeds maximum allowable by code

Refer to details for arrangement.

2017-06
(37)

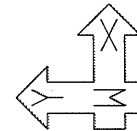
WALLS



HOME2 - SSF MODEL

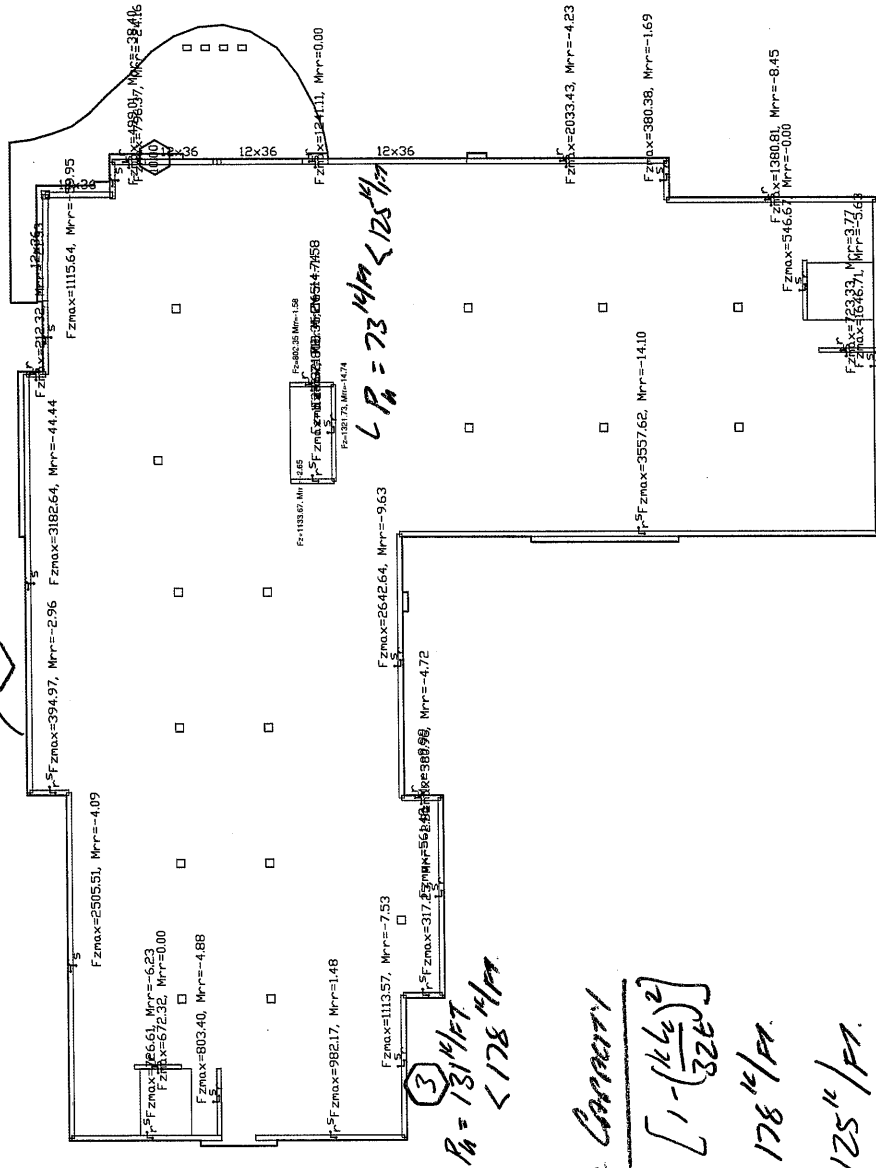
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Design Sections, Investigation, Moment Capacity with Demand
 Max: 946.30
 Min: -1301.80
 Max demand/capacity ratio: 1.12

$P_h = 112 \text{ k/ft} < 178 \text{ k/ft}$
 (18)



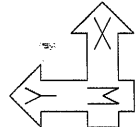
$P_h = 131 \text{ k/ft} < 178 \text{ k/ft}$
 (3)

Bearing Wall Capacity

$$P_h' = 0.55 f'_c A_g \left[1 - \left(\frac{k l_u}{32 h} \right)^2 \right]$$

12" Wall: $P_h = 178 \text{ k/ft}$

10" Wall: $P_h = 125 \text{ k/ft}$



CHECK BEARING WALL CAPACITY

Wall Reactions Plan Envelope WALL REACTIONS

HOME2 - SSF

WALL REACTION - Fz_Mrr

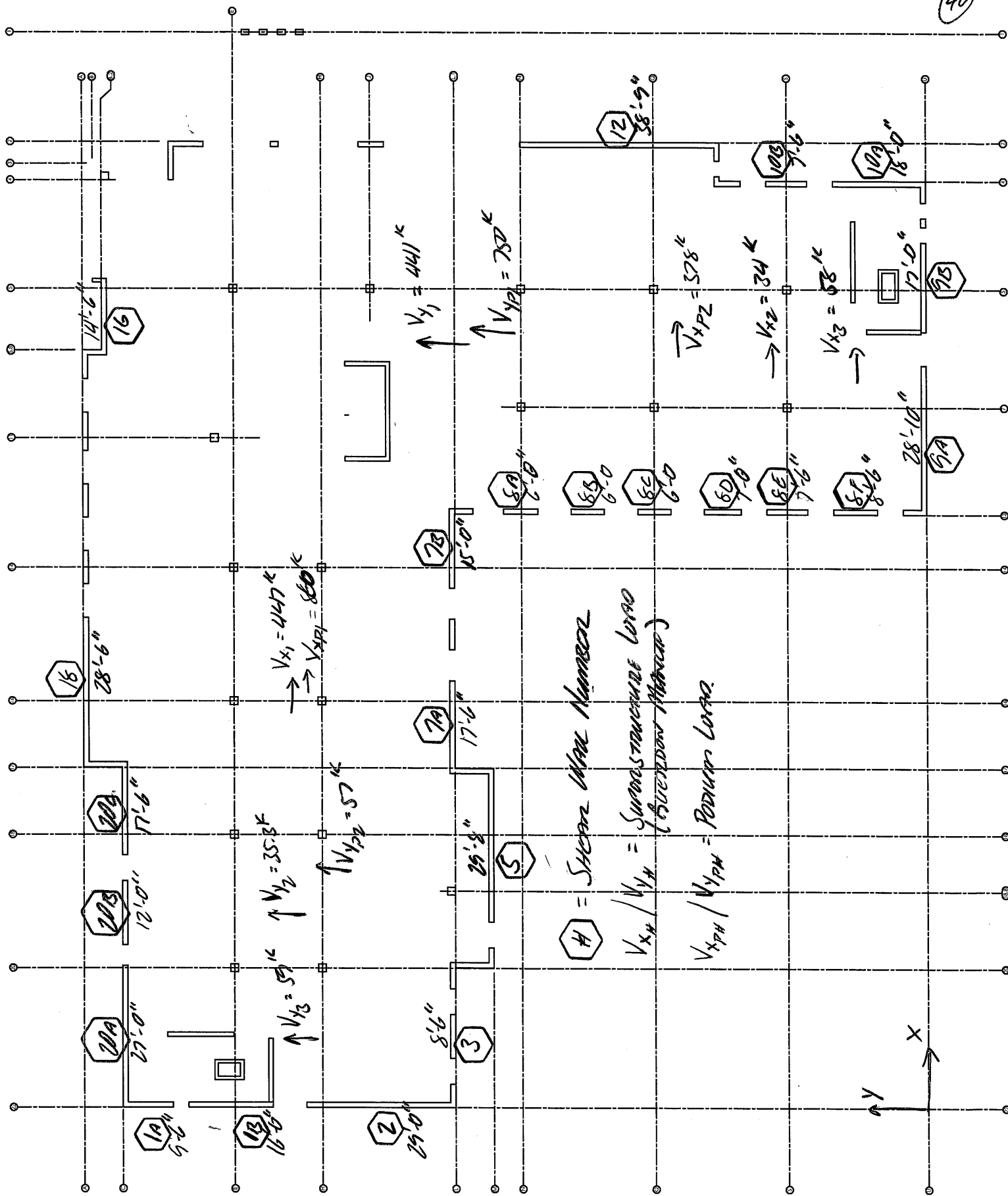
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(39)

7/27/06



2017-06
(41)

SEISMIS LOAD
SUPERSTRUCTURE

	Vx	Vy	x	y	Vx * y	Vy * x
Vx1	447				126	56322
Vx2	34				29	986
Vx3	58				13	754
Vy1			441	147		64827
Vy2			35.8	29		1038.2
Vy3			59	13		767
TOTAL	539	535.8			58062	66632.2

X 124.4

Y 107.7

PODIUM

Vx1	860				126	108360
Vx2	378				44	16632
Vy1			750	158		118500
Vy2			488	57		27816
TOTAL	1238	1238	215	170	124992	146316

X 118.2

Y 101.0

$$V_x = 539(1.65) + 1238 = 2148^k$$

$$\bar{y} = \frac{539(1.65)(107.7) + 1238(101)}{539(1.65) + 1238} = 103.5'$$

$$V_y = 535.8(1.65) + 1238 = 2143^k$$

$$\bar{x} = \frac{535.8(1.65)(124.4) + 1238(118.2)}{535.8(1.65) + 1238} = 120.8'$$



Torsional Analysis of Rigid Diaphragm

File = HCPENG-ENGINE-ENERCA-12017-0-4.E06
ENERCALC, INC, 1983-2017, Build: 6-17-3-17, Ver: 6-17-3-17

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: Shear Walls

General Information

Calculations per IBC 2012, CBC 2013, ASCE 7-1

Applied Lateral Force in "X" Direction	2,148.0 k	Center of Shear Application :	
Applied Lateral Force in "Y" Direction	2,143.0 k	Distance from "X" datum point	120.80 ft
		Distance from "Y" datum point	103.90 ft
Note: These loads are resolved into X & Y components when applied to the system of elements at angular increments.		Accidental Torsion values per ASCE 7-05 12.8.4.2	
		Ecc. as % of Maximum Dimension	5.00 %
Load Orientation Angular Increment	15.0 deg	Maximum Dimensions :	
Load Location Angular Increment	15.0 deg	Along "X" Axis	187.0 ft
		Along "Y" Axis	164.0 ft
Center of Rigidity Location (calculated) . . .			
"X" dist. from Datum	111.816 ft		
"Y" dist. from Datum	102.988 ft		
		Accidental Eccentricity +/- from "Y" Coord. of Center of Load Application :	9.350 ft
		Accidental Eccentricity +/- from "X" Coord. of Center of Load Application :	8.20 ft

Wall Information

Label: 10A	X Wall C.G. Location	179 ft	Length	18 ft
	Y Wall C.G. Location	8.7 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "y" Dir	2.3461E-004 in	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	5.0893E-002 in		E - Shear	1 Mpsi
Label: 10B	X Wall C.G. Location	179 ft	Length	7.5 ft
	Y Wall C.G. Location	26.75 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "y" Dir	2.3548E-003 in	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	1.2214E-001 in		E - Shear	1 Mpsi
Label: 12	X Wall C.G. Location	186.5 ft	Length	38.75 ft
	Y Wall C.G. Location	59 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "y" Dir	5.1849E-005 in	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	2.3640E-002 in		E - Shear	1 Mpsi
Label: 16	X Wall C.G. Location	153.33 ft	Length	14.5 ft
	Y Wall C.G. Location	159 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "y" Dir	3.9658E-004 in	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	6.3177E-002 in		E - Shear	1 Mpsi
Label: 18	X Wall C.G. Location	80.7 ft	Length	28.5 ft
	Y Wall C.G. Location	164 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "y" Dir	8.8635E-005 in	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	3.2143E-002 in		E - Shear	1 Mpsi
Label: 1A	X Wall C.G. Location	0 ft	Length	9.5 ft
	Y Wall C.G. Location	150.8 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "y" Dir	1.2142E-003 in	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	9.6428E-002 in		E - Shear	1 Mpsi
Label: 1B	X Wall C.G. Location	0 ft	Length	16 ft
	Y Wall C.G. Location	135.2 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "y" Dir	3.1081E-004 in	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	5.7254E-002 in		E - Shear	1 Mpsi
Label: 2	X Wall C.G. Location	0 ft	Length	29 ft
	Y Wall C.G. Location	106.58 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "y" Dir	8.5779E-005 in	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	3.1589E-002 in		E - Shear	1 Mpsi

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Printed: 3 JUN 2017, 4:08PM

Torsional Analysis of Rigid Diaphragm

File = f:\HCPENG-1\ENGINE-1\ENERCA-1\2017-0-4.EC6
ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: Shear Walls

Wall Information

Label: 20A	X Wall C.G. Location	14.08 ft	Length	27 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	155.75 ft	Height	14 ft
Along Wall "y" Dir 9.8322E-005 in	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "x" Dir 3.3928E-002 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 20B	X Wall C.G. Location	38 ft	Length	12 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	155.75 ft	Height	14 ft
Along Wall "y" Dir 6.4599E-004 in	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "x" Dir 7.6339E-002 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 20C	X Wall C.G. Location	58.17 ft	Length	17.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	155.75 ft	Height	14 ft
Along Wall "y" Dir 1.2267E-004 in	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "x" Dir 5.2347E-002 in	Wall Fixity	Fix-Fix	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 3	X Wall C.G. Location	13.75 ft	Length	8.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	92 ft	Height	14 ft
Along Wall "y" Dir 1.6541E-003 in	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "x" Dir 1.0777E-001 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 5	X Wall C.G. Location	50.25 ft	Length	29.7 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	84.7 ft	Height	14 ft
Along Wall "y" Dir 8.2052E-005 in	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "x" Dir 3.0844E-002 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 7A	X Wall C.G. Location	73.25 ft	Length	17.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	92 ft	Height	14 ft
Along Wall "y" Dir 2.5067E-004 in	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "x" Dir 5.2347E-002 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 7B	X Wall C.G. Location	107.8 ft	Length	15 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	92 ft	Height	14 ft
Along Wall "y" Dir 3.6435E-004 in	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "x" Dir 6.1071E-002 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 8A	X Wall C.G. Location	115.5 ft	Length	6 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	78.7 ft	Height	14 ft
Along Wall "y" Dir 4.4679E-003 in	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "x" Dir 1.5268E-001 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 8B	X Wall C.G. Location	115.5 ft	Length	6 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	66 ft	Height	14 ft
Along Wall "y" Dir 4.4679E-003 in	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "x" Dir 1.5268E-001 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 8C	X Wall C.G. Location	115.5 ft	Length	6 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	52.83 ft	Height	14 ft
Along Wall "y" Dir 4.4679E-003 in	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "x" Dir 1.5268E-001 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 8D	X Wall C.G. Location	115.5 ft	Length	7 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	40 ft	Height	14 ft
Along Wall "y" Dir 2.8667E-003 in	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "x" Dir 1.3087E-001 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi
Label: 8E	X Wall C.G. Location	115.5 ft	Length	7.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	27 ft	Height	14 ft
Along Wall "y" Dir 2.3548E-003 in	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "x" Dir 1.2214E-001 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
			E - Shear	1 Mpsi

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Torsional Analysis of Rigid Diaphragm

File = f:\HCPENG-1\ENGINE-1\ENERCA-1\2017-0-4\EC6
ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: Shear Walls

Wall Information

Label: 8F	X Wall C.G. Location	115.5 ft	Length	8.5 ft
	Y Wall C.G. Location	14.17 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "y" Dir	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir			E - Shear	1 Mpsi
Label: 9A	X Wall C.G. Location	129 ft	Length	28.8 ft
	Y Wall C.G. Location	0 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "y" Dir	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir			E - Shear	1 Mpsi
Label: 9B	X Wall C.G. Location	158.5 ft	Length	17 ft
	Y Wall C.G. Location	0 ft	Height	14 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "y" Dir	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir			E - Shear	1 Mpsi

ANALYSIS SUMMARY

Maximum shear forces applied to resisting elements. Eccentricity with respect to Center of Rigidity

Resisting Element	Load Angle	Max Shear along Member Local "y-y" Axis			Load Angle	Max Shear along Member Local "x-x" Axis		
		X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)		X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)
10A	90	-18.33	0.91	236.514	0	0.37	0.91	0.615
10B	90	-18.33	0.91	23.564	0	0.37	0.91	0.256
12	90	-18.33	0.91	1,079.700	0	0.37	0.91	1.324
16	0	-8.98	9.11	83.570	90	-18.33	0.91	0.852
18	0	-8.98	9.11	375.764	90	-18.33	0.91	1.653
1A	90	0.37	0.91	42.065	0	-8.98	9.11	0.341
1B	90	0.37	0.91	164.328	0	-8.98	9.11	0.565
2	90	0.37	0.91	595.424	0	-8.98	9.11	0.995
20A	0	-8.98	9.11	335.997	90	0.37	0.91	1.505
20B	0	-8.98	9.11	51.140	90	0.37	0.91	0.669
20C	0	-8.98	9.11	269.307	90	0.37	0.91	0.976
3	0	0.37	0.91	18.928	90	0.37	0.91	0.474
5	0	0.37	0.91	381.578	90	0.37	0.91	1.656
7A	0	0.37	0.91	124.902	90	0.37	0.91	0.976
7B	0	0.37	0.91	85.932	90	-18.33	0.91	0.841
8A	90	-18.33	0.91	11.486	0	0.37	0.91	0.205
8B	90	-18.33	0.91	11.486	0	0.37	0.91	0.205
8C	90	-18.33	0.91	11.486	0	0.37	0.91	0.205
8D	90	-18.33	0.91	17.901	0	0.37	0.91	0.239
8E	90	-18.33	0.91	21.792	0	0.37	0.91	0.256
8F	90	-18.33	0.91	31.024	0	0.37	0.91	0.291
9A	0	0.37	0.91	360.286	90	-18.33	0.91	1.641
9B	0	0.37	0.91	116.595	90	-18.33	0.91	1.005

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Torsional Analysis of Rigid Diaphragm

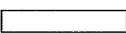
File = F:\HCPENG-1\ENGINE-1\ENERCA-1\2017-0-4.EC6
ENERCALC, INC. 1983-2017, Build: 6.17.3.17, Ver: 6.17.3.17

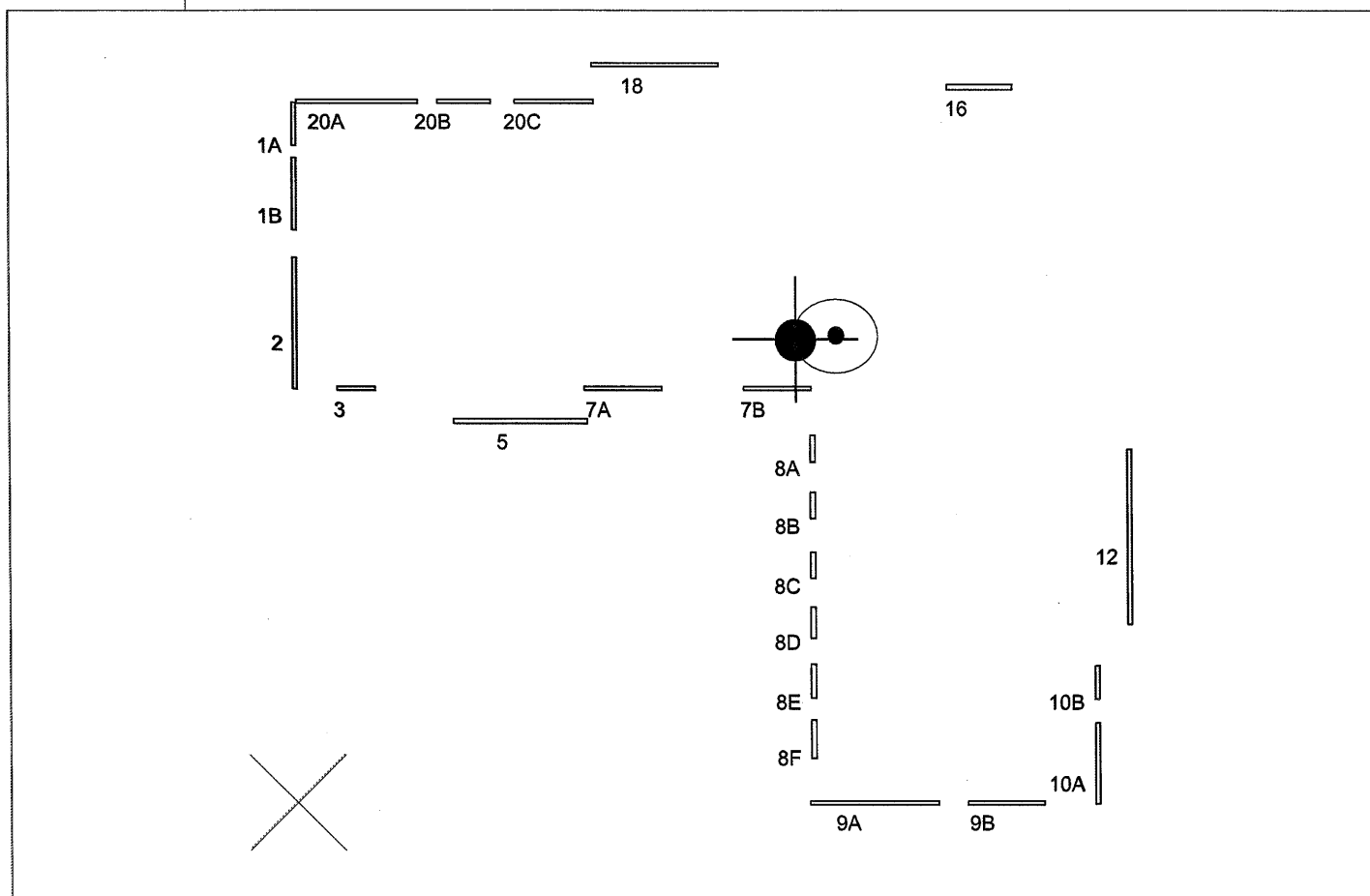
Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: Shear Walls

Layout of Resisting Elements

Legend:  Defined Wall  Center of Rigidity  Center of Mass  Accidental eccentricity application boundary  Datum



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Printed: 3 JUN 2017, 4:08PM

Torsional Analysis of Rigid Diaphragm

File = f:\HCPENG\1\ENGINE\1\ENERCA\1\2017-0-4.EC6
ENERCALC, INC. 1983-2017, Build:6.17.3.17, Ver:6.17.3.17

Lic. # : KW-06007967

Licensee : HCP ENGINEERING

Description : Shear Walls

Analysis Notes

This program is designed to distribute an applied shear load to a set of resisting elements.

Each resisting element data entry specifies a deflection along a "major" and "minor" axis due to a 1,000 lb load. Each resisting element may be entered as a wall or a column (whereby the deflection is calculated), or as a generic resisting element with specified deflection. The deflections define the stiffness of each resisting element.

Each resisting element is defined at an (X,Y) location from a datum the user has previously defined. A counter-clockwise rotation of the element can be entered with respect to a traditional "+X" axis line.

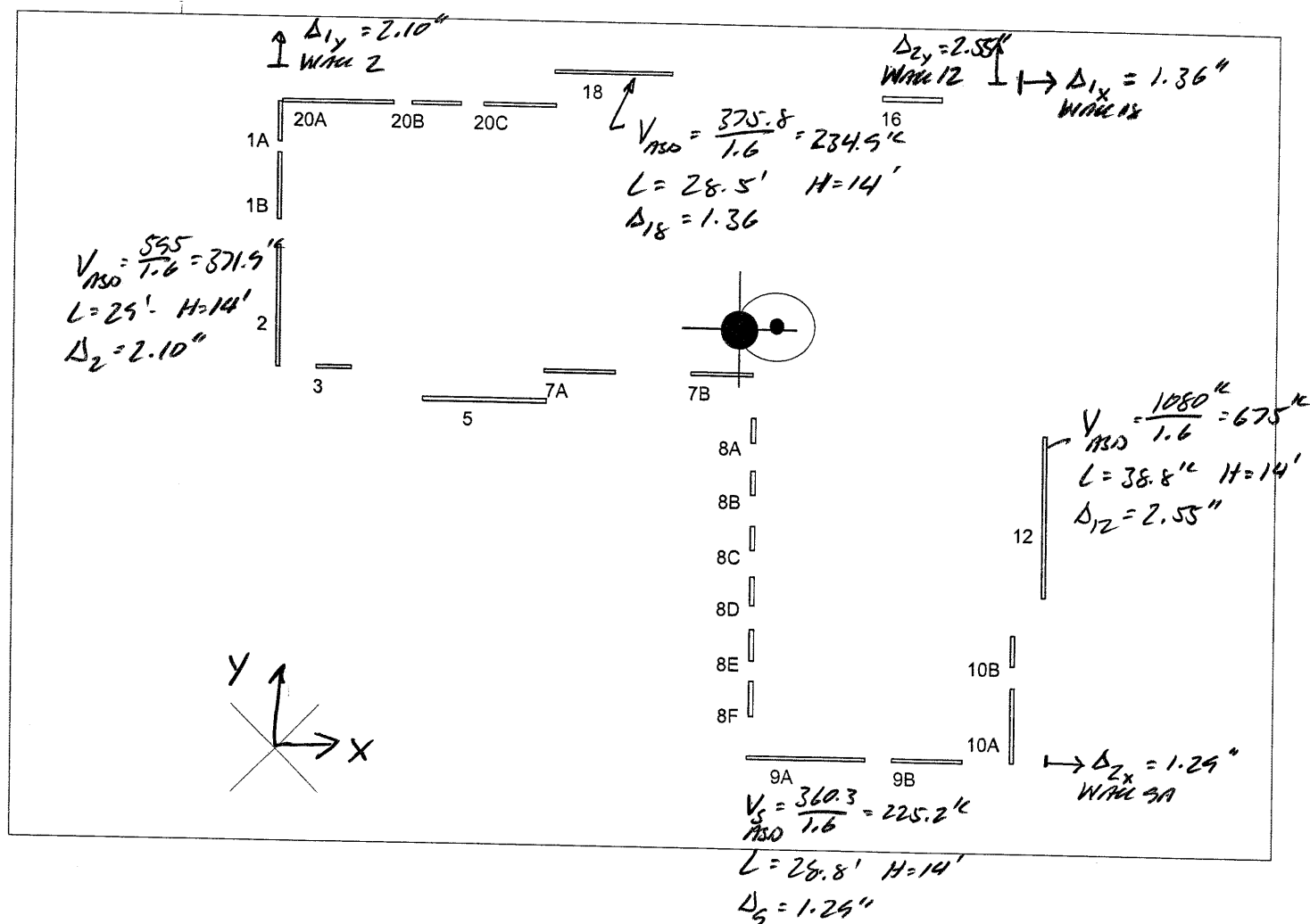
A main "shear" load and an optional orthogonal shear load are specified for distribution to the system of resisting elements. In addition the maximum orthogonal dimensions of the structure and minimum accidental eccentricity percentage are specified.

From the entered loads the program calculates resultant force vectors for each angular orientation that is requested. The force is applied to the resisting elements in angular increments to generate a series of resulting direct and torsional shear loads on each element. This application of force is then repeated at angular intervals along an elliptical path defined by the minimum accidental eccentricity.

The end result is a table of direct shear and torsional shear values for each element from the iterated angles of load application and accidental eccentricity. These values are then searched to find the maximum major and minor axis shears applied to each resisting element.

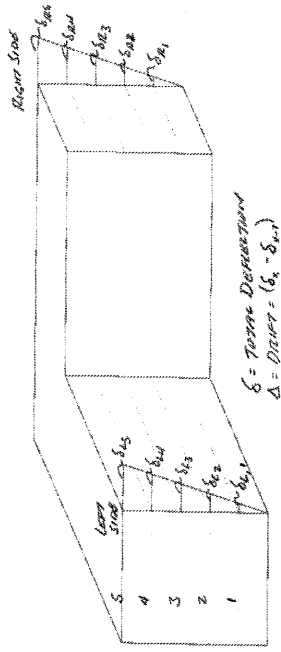
2017-06
46.1

TORSIONAL ANALYSIS



(SEE p. 46.2 TO 46.6 FOR DEFLECTION CALCULATION)

TORSIONAL ANALYSIS



X Direction

LEVEL	FLOOR DRIFT						CHECK TORSIONAL IRREGULARITY TYPE 1a $\Delta_{\text{MAX}} / \Delta_{\text{AVG}} < 1.2$	CHECK EXTREME TORSIONAL IRREGULARITY TYPE 1b $\Delta_{\text{MAX}} / \Delta_{\text{AVG}} < 1.4$	AMPLIFICATION FACTOR $A_x = (\delta_{\text{MAX}} / 1.2 \delta_{\text{AVG}})^2$
	Δ_1	δ_1	Δ_2	δ_2	Δ_{AVG}	Δ_{MAX}			
1	1.36	1.36	1.29	1.29	1.33	1.36	OKAY	OKAY	0.73
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-

Y - Direction

LEVEL	FLOOR DRIFT						CHECK TORSIONAL IRREGULARITY TYPE 1a $\Delta_{\text{MAX}} / \Delta_{\text{AVG}} < 1.2$	CHECK EXTREME TORSIONAL IRREGULARITY TYPE 1b $\Delta_{\text{MAX}} / \Delta_{\text{AVG}} < 1.4$	AMPLIFICATION FACTOR $A_x = (\delta_{\text{MAX}} / 1.2 \delta_{\text{AVG}})^2$
	Δ_1	δ_1	Δ_2	δ_2	Δ_{AVG}	Δ_{MAX}			
1	2.10	2.10	2.55	2.55	2.33	2.55	OKAY	OKAY	0.84
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-

2017-06
46.2

2017-06
46.3

WALL NO: 18

Wall Geometry:

Height, H = 14.0 ft
 Thickness, t = 1.0 ft
 Length, L = 28.5 ft
 Fix at Top = Yes CANTILEVERED WALL

Concrete Strength, F'c = 5.0 ksi
 Modulus of Elasticity, E = 4,031 ksi

Loading:

Uniform Lateral Load, w = 0 klf
 Concentrated Load at Top, P = 234.9 kips

Wall Deflection, D = 1.36 in

Cantilevered Wall

Uniform Lateral Load

$$\delta = \frac{1.5wH}{Et} \left[\left(\frac{H}{L} \right)^3 + \frac{H}{L} \right]$$

Point Load at Top

$$\delta_c = \frac{4P}{Et} \left[\left(\frac{H}{L} \right)^3 + 0.75 \frac{H}{L} \right]$$

Rigid Connection at Top

$$\delta_f = \frac{P}{Et} \left[\left(\frac{H}{L} \right)^3 + 3 \frac{H}{L} \right]$$

2017-06
(46.4)

WALL NO: 9A

Wall Geometry:

Height, H = 14.0 ft
Thickness, t = 1.0 ft
Length, L = 28.8 ft
Fix at Top = Yes CANTILEVERED WALL

Concrete Strength, F_c = 5.0 ksi
Modulus of Elasticity, E = 4,031 ksi

Loading:

Uniform Lateral Load, w = 0 klf
Concentrated Load at Top, P = 225.2 kips

Wall Deflection, D = 1.29 in

Cantilevered Wall

Uniform Lateral Load

$$\delta = \frac{1.5wH}{Et} \left[\left(\frac{H}{L} \right)^3 + \frac{H}{L} \right]$$

Point Load at Top

$$\delta_c = \frac{4P}{Et} \left[\left(\frac{H}{L} \right)^3 + 0.75 \frac{H}{L} \right]$$

Rigid Connection at Top

$$\delta_f = \frac{P}{Et} \left[\left(\frac{H}{L} \right)^3 + 3 \frac{H}{L} \right]$$

2017-06
46.5

WALL NO: 2

Wall Geometry:

Height, H = 14.0 ft
 Thickness, t = 1.0 ft
 Length, L = 29.0 ft
 Fix at Top = Yes CANTILEVERED WALL

Concrete Strength, F'c = 5.0 ksi
 Modulus of Elasticity, E = 4,031 ksi

Loading:

Uniform Lateral Load, w = 0 klf
 Concentrated Load at Top, P = 371.9 kips

Wall Deflection, D = 2.10 in

Cantilevered Wall

Uniform Lateral Load

$$\delta = \frac{1.5wH}{Et} \left[\left(\frac{H}{L} \right)^3 + \frac{H}{L} \right]$$

Point Load at Top

$$\delta_c = \frac{4P}{Et} \left[\left(\frac{H}{L} \right)^3 + 0.75 \frac{H}{L} \right]$$

Rigid Connection at Top

$$\delta_f = \frac{P}{Et} \left[\left(\frac{H}{L} \right)^3 + 3 \frac{H}{L} \right]$$

WALL NO: 12

Wall Geometry:

Height, H = 14.0 ft
 Thickness, t = 1.0 ft
 Length, L = 38.8 ft
 Fix at Top = Yes CANTILEVERED WALL

Concrete Strength, F'c = 5.0 ksi
 Modulus of Elasticity, E = 4,031 ksi

Loading:

Uniform Lateral Load, w = 0 klf
 Concentrated Load at Top, P = 675.0 kips

Wall Deflection, D = 2.55 in

Cantilevered Wall

Uniform Lateral Load

$$\delta = \frac{1.5wH}{Et} \left[\left(\frac{H}{L} \right)^3 + \frac{H}{L} \right]$$

Point Load at Top

$$\delta_c = \frac{4P}{Et} \left[\left(\frac{H}{L} \right)^3 + 0.75 \frac{H}{L} \right]$$

Rigid Connection at Top

$$\delta_f = \frac{P}{Et} \left[\left(\frac{H}{L} \right)^3 + 3 \frac{H}{L} \right]$$

REG-Entrance Corridor / DRAG CALCULATIONLINE A-C

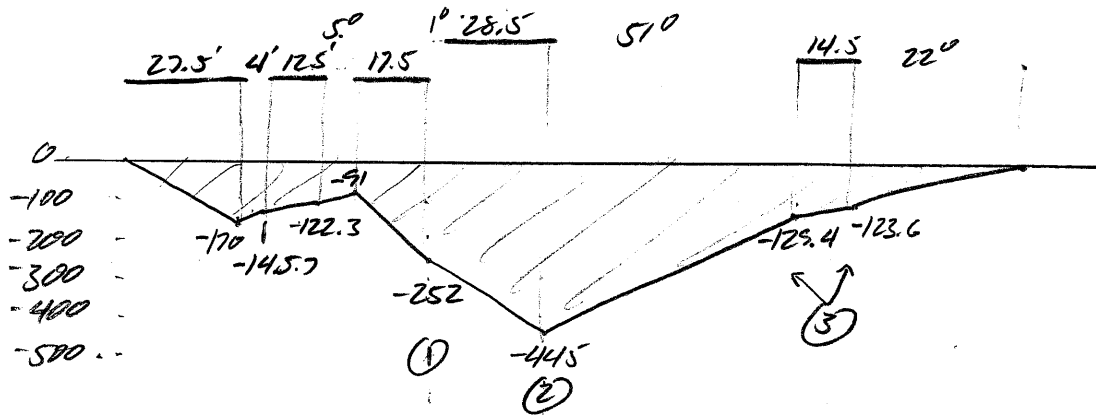
Wave	V_u (ft)	L (ft)	v_{SW} (ft/s)	v_{SD} (ft/s)
20A	336	27'	12.4	-6.2
20B	51	12'	4.25'	+1.55
20C	269	17.5'	15.4	-5.2
18	376	28.5'	13.2	-7.0
16	84	14.5'	5.8	+0.40

$$1116''$$

$$L_{Diam} = 181'$$

$$v_D = 6.2 \text{ ft/s}$$

$$4 \sim \#6 \text{ TYP. - Capacity} = 4(0.44)(60) \times 0.9 = 95''$$



$$(1) \quad A_R = \frac{252}{0.9(60)} = 4.67 \text{ in}^2 - 12 \sim \#6$$

$$L = \frac{252 - 95}{6.2} \times 2 = 50.6' \approx 51'$$

$$\therefore \underline{12 \sim \#6 \times 51' - 0}$$

$$(2) \quad A_R = \frac{445}{0.9(60)} = 8.2 \text{ in}^2 - 20 \sim \#6$$

$$L = \frac{445 - 95}{6.2} = 56'$$

$$\therefore \underline{20 \sim \#6 \times 56' - 0}$$

$$(3) \quad A_R = \frac{129}{0.9(60)} = 2.4 \text{ in}^2 - 6 \sim \#6$$

$$L = \frac{129 - 95}{6.2} \times 2 = 11'$$

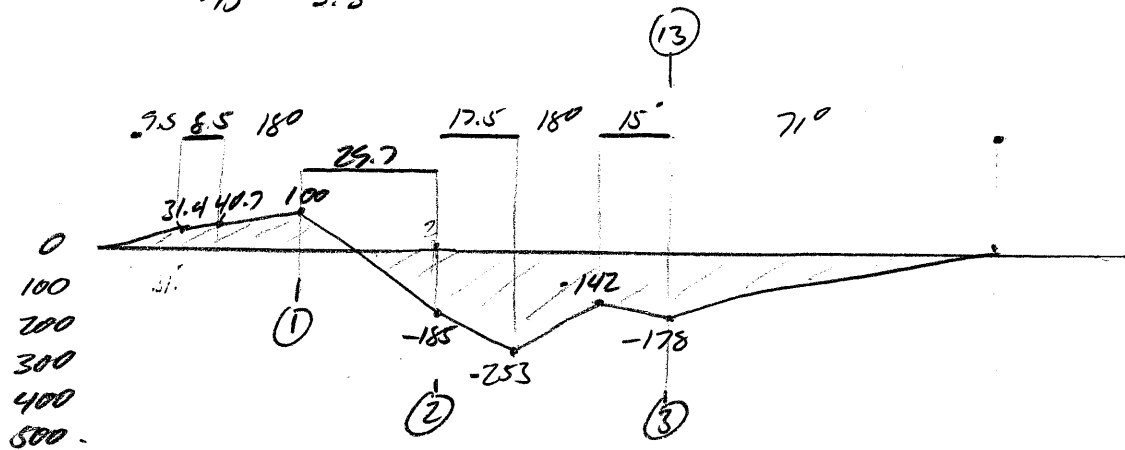
$$\therefore \underline{6 \sim \#6 \times 11' - 0}$$

LINE L-M

Water	$V_h (ft)$	$L (ft)$	$v_{SW} (ft^3/ft)$	$v_{SD} (ft^3/ft)$
3	18.5	8.5	2.22	
5	381.6	29.7	12.9	
7A	124.9	17.5	7.2	
7B	85.9	15	5.7	
	<u>611</u>			

$$L_{DIAP} = 186'$$

$$v_D = 3.3 \text{ ft/ft}$$



$$(1) A_s = \frac{100}{0.9(60)} = 1.85 \text{ in}^2 \quad 6 \#6$$

$$L = \frac{100 - 95}{3.3} \times 2 = 3.0 < 3.7' \quad \text{EXTEND } 4' \text{ min.}$$

$$(2) A_s = \frac{185}{0.9(60)} = 3.4 \text{ in}^2 \quad 8 \#6$$

$$L = \frac{185 - 95}{3.3} = 27'$$

$$(3) A_s = \frac{178}{0.9(60)} = 3.3 \text{ in}^2 \quad 8 \#6$$

$$L = \frac{178 - 95}{3.3} = 25' \quad (\text{Past Line 13})$$

COMBINED
USE 8 #6
L = 117'
(25' FROM
LINE 13)

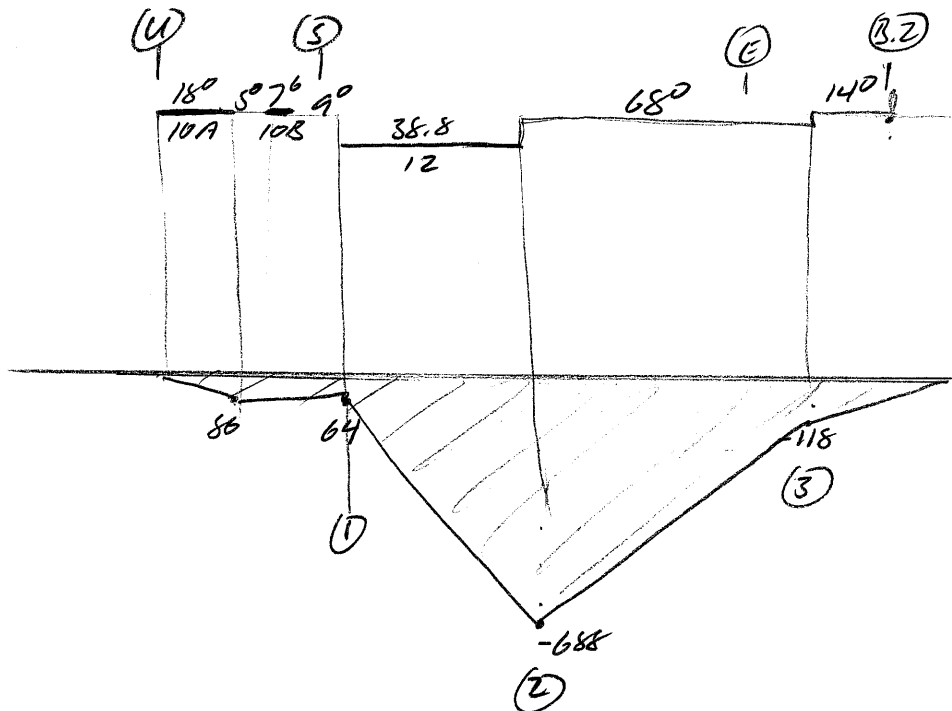
2017-06
(46.9)

LINE 2-4

W/m	$V_d (in)$	$L (ft)$	$v_{30} (in/ft)$	$v_{50} (in/ft)$
10A	236.5	18'	13.2	-4.8
10B	23.6	7.5'	3.2	+5.2
12	1079.7	38.8	27.8	-15.4
	<u>1339.1</u>			

$$L_{D180} = 160'$$

$$v_D = 8.4 in/ft$$



(1) $T_d < 95'' \therefore$ Use 4n #6 (Extend 6'-0 min Past Column)

(2) $A_s = \frac{688}{0.9(60)} = 12.7 in^2 \quad 30n \#6$

$$L = \frac{688-95}{8.4} = 70.6 + 38.8 = 109'$$

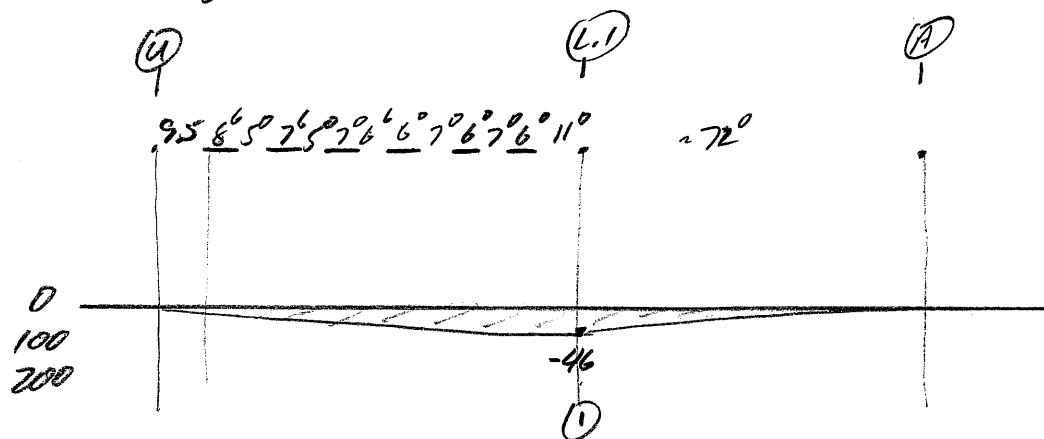
$\therefore 30n \#6 \times 109'$
(70.6' Past W/m)

LINE 13

WAVE	V_u (k)	L (ft)	r_{sum} (1"/ft)	r_{SD} (1"/ft)
8A	11.5	6°	1.9	-1.26
8B	11.5	6°	1.9	-1.26
8C	11.5	6°	1.9	-1.26
8D	18.0	7°	2.6	-1.96
8E	21.8	7.5	2.9	-2.26
8F	31.0	8.5	3.6	-2.46
105.3'				

$$L_{DIAP} = 164'$$

$$r_D = 0.64 \text{ 1"/ft}$$

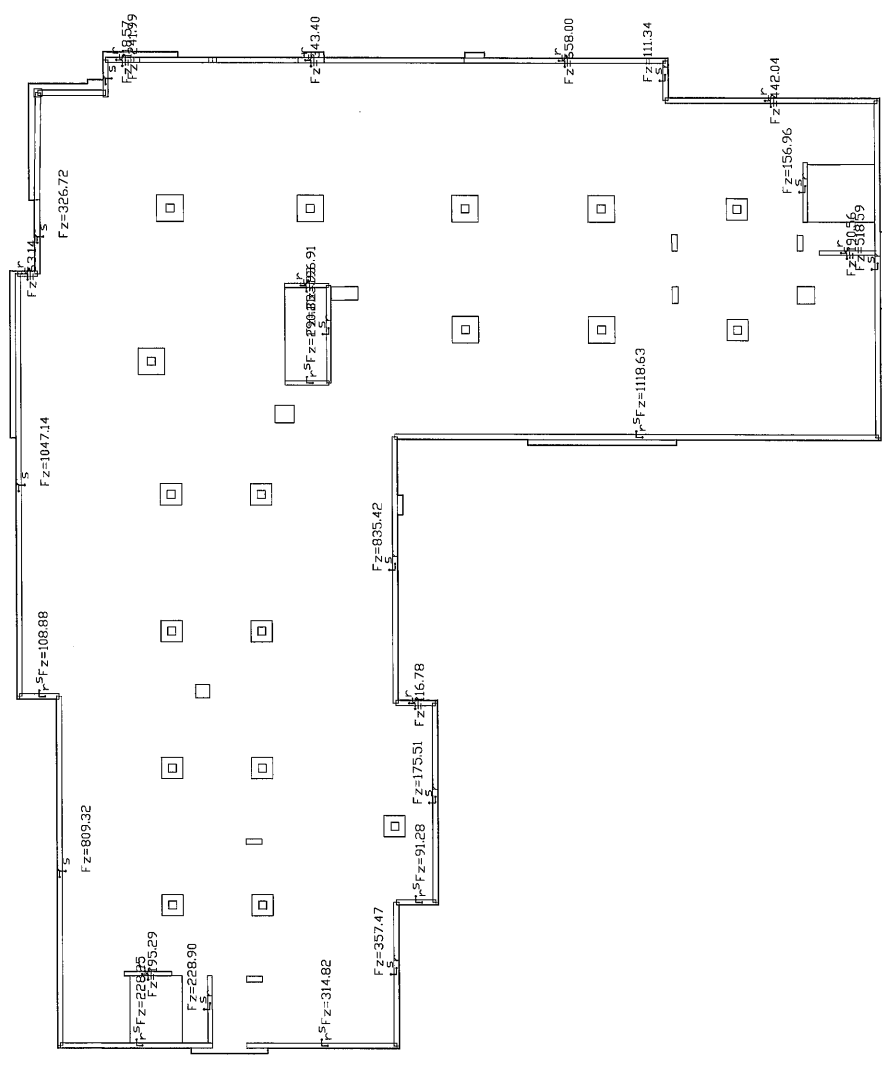
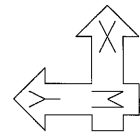


$$(1) A_{SP} = \frac{46}{0.5(60)} = 0.85 \text{ in}^2 < 1.76 \text{ in}^2 - 4\#6 \text{ MIN.}$$

$$L_{MIN} = 44'' \approx \underline{\text{USE 6'-0 PAST L.1}}$$

$$\therefore \underline{4\#6 (6'-0 \text{ MIN PAST L.1})}$$

2017-06
(47)



Wall Reactions Plan_Strength(Dead Load Only) WALL REACTION - FZ_Mr - SERVICE LOADS

HOME2 - SSF

COLUMN REACTION - Fz - SERVICE = LIVE

11/02/17

17:45:54

h2ssf-Adapt-171020-NoPorteCo.adm

2017-06
(48)

SHEAR WALL SUMMARY

WALL	LENGTH	HEIGHT	THICK	Pu (DEAD LOAD)		Vu		Mu		BOUNDARY ELEMENT				ZONE TYPE	
				k	k/ft	k	k/ft	k	k/ft	D	B	BULB R/F	HOOP - B	HOOP - D	ZONE TYPE
1A	9.5	14	12	74.2	7.8	42.0	4.42	399.0	399.0	12	12	6-#5	2-#4	3-#4	6
1B	16.5	14	12	128.8	7.8	164.0	9.94	2,706.0	2,706.0	24	12	10-#5	2-#4	3-#4	10
2	29	14	12	311.0	10.7	595.0	20.52	17,255.0	17,255.0	24	12	10-#5	2-#4	3-#4	10
3	8.5	14	12	361.0	42.5	19.0	2.24	161.5	161.5	12	12	6-#5	2-#4	3-#4	6
5	29.7	14	12	215.0	7.2	382.0	12.86	11,345.4	11,345.4	24	12	10-#5	2-#4	3-#4	10
7A	17.5	14	12	468.5	26.8	125.0	7.14	2,187.5	2,187.5	24	12	10-#5	2-#4	3-#4	10
7B	15	14	12	401.5	26.8	86.0	5.73	1,290.0	1,290.0	24	12	10-#5	2-#4	3-#4	10
8A	6	14	12	179.3	29.9	11.0	1.83	66.0	66.0	12	12	6-#5	2-#4	3-#4	6
8B	6	14	12	179.3	29.9	11.0	1.83	66.0	66.0	12	12	6-#5	2-#4	3-#4	6
8C	6	14	12	179.3	29.9	11.0	1.83	66.0	66.0	12	12	6-#5	2-#4	3-#4	6
8D	7	14	12	209.1	29.9	18.0	2.57	126.0	126.0	12	12	6-#5	2-#4	3-#4	6
8E	7.5	14	12	224.1	29.9	22.0	2.93	165.0	165.0	12	12	6-#5	2-#4	3-#4	6
8F	7.5	14	12	224.1	29.9	31.0	4.13	232.5	232.5	12	12	6-#5	2-#4	3-#4	6
9A	28.8	14	12	345.9	12.0	360.0	12.50	10,368.0	10,368.0	24	12	10-#5	2-#4	3-#4	10
9B	17	14	12	204.1	12.0	117.0	6.88	1,989.0	1,989.0	24	12	10-#5	2-#4	3-#4	10
10A	18	14	12	324.7	18.0	237.0	13.17	4,266.0	4,266.0	24	12	10-#5	2-#4	3-#4	10
10B	7.5	14	12	135.3	18.0	24.0	3.20	180.0	180.0	12	12	6-#5	2-#4	3-#4	6
12	38.75	14	12	693.0	17.9	1,080.0	27.87	41,850.0	41,850.0	36	12	18-#5	2-#4	3-#4	18
16	14.5	14	12	147.0	10.1	84.0	5.79	1,218.0	1,218.0	24	12	10-#5	2-#4	3-#4	10
18	28.5	14	12	1,064.0	37.3	376.0	13.19	10,716.0	10,716.0	24	12	10-#5	2-#4	3-#4	10
20A	17.5	14	12	255.2	14.6	336.0	19.20	5,880.0	5,880.0	24	12	10-#5	2-#4	3-#4	10
20B	12	14	12	175.0	14.6	51.0	4.25	612.0	612.0	12	12	6-#5	2-#4	3-#4	6
20C	27	14	12	393.8	14.6	200.0	7.41	1,500.0	1,500.0	24	12	8-#5	2-#4	3-#4	10
29	8	19	12	1,038.0	129.8	270.0	33.75	2,160.0	2,160.0	24	12	8-#5	2-#4	4-#6	10
30	19	19	12	1,202.0	63.3	271.0	14.26	5,149.0	5,149.0	24	12	8-#5	2-#4	4-#6	10
31	8	19	12	667.0	83.4	272.0	34.00	2,176.0	2,176.0	24	12	8-#5	2-#4	4-#6	10



(45)

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

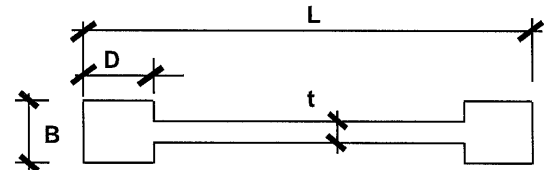
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 74.2$ k
FACTORED MOMENT LOAD $M_u = 399$ ft-k
FACTORED SHEAR LOAD $V_u = 42$ k
LENGTH OF SHEAR WALL $L = 9.5$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 12$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 6 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 9.50$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 12.00$ in
BULB REINFORCING 6 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

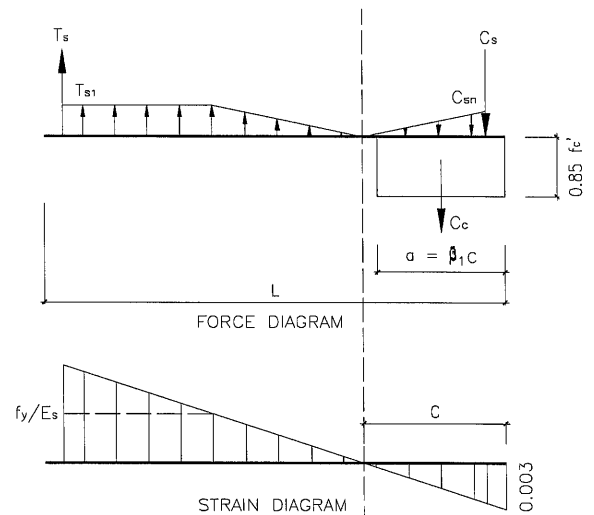
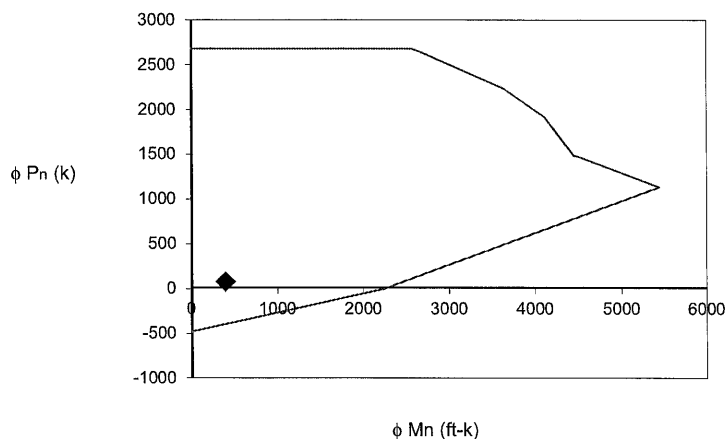
THE WALL DESIGN IS ADEQUATE



ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$P_u = 74.2$ k $< 0.35 A_g f'_c = 1915$ k [Satisfactory]
where $A_g = 1368$ in².



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$(\rho_t)_{min.} = 0.0020$ [for $A_{cv} (f'_c)^{0.5} = 86.52$ kips $> V_u$, and bar size # 5 horizontal]
 $(\rho_l)_{min.} = 0.0012$ [for $A_{cv} (f'_c)^{0.5} = 86.52$ kips $> V_u$, and bar size # 5 vertical]
 $(\rho_t)_{prov.} = 0.0043 > (\rho_t)_{min.}$ [Satisfactory]
 $(\rho_l)_{prov.} = 0.0043 > (\rho_l)_{min.}$ [Satisfactory]

where $A_{cv} = 1368$ in² (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 367.78$ kips $> V_u$ [Satisfactory]

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$\alpha_c = 3.0$ (for $h_w / L = 1.47 < 1.5$)

$\rho_l > \rho_t$ [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

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CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 2683.2 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 8.99 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 2568 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 64 \text{ in} \quad a = C_b \beta_1 = 54 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 108 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 1476 kips AND 4493 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 2234 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -485 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 2683	0
AT MAXIMUM LOAD	= 2683	2568
AT 0 % TENSION	= 2640	2690
AT 25 % TENSION	= 2237	3636
AT 50 % TENSION	= 1925	4106
AT $\epsilon_t = 0.002$	= 1489	4443
AT BALANCED CONDITION	= 1476	4493
AT $\epsilon_t = 0.005$	= 1136	5443
AT FLEXURE ONLY	= 0	2234
AT TENSION ONLY	= -485	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 2522 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 13 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)
$$\bar{\epsilon} = (P_u / A) + (M_u y / I) = 0.226 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)
$$y = 57 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)
$$A = 1440 \text{ in}^2.$$
 (area of transformed section.)
$$I = 1559866 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.013 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 6.44 \text{ in.} \quad (\text{ACI 318-14 18.10.6.4})$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.} \quad (\text{ACI 318-14 18.10.6.2 \& 18.10.6.5})$$

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$



Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

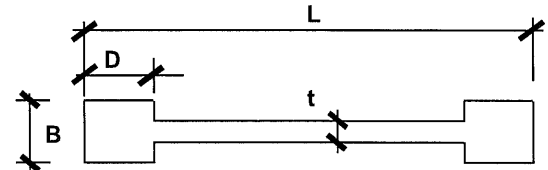
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 128.8$ k
FACTORED MOMENT LOAD $M_u = 2706$ ft-k
FACTORED SHEAR LOAD $V_u = 164$ k
LENGTH OF SHEAR WALL $L = 16.5$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 24$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 16.50$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 24.00$ in
BULB REINFORCING 10 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

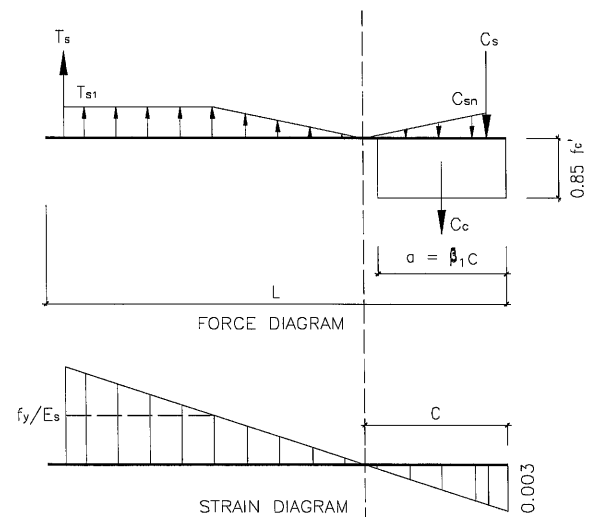
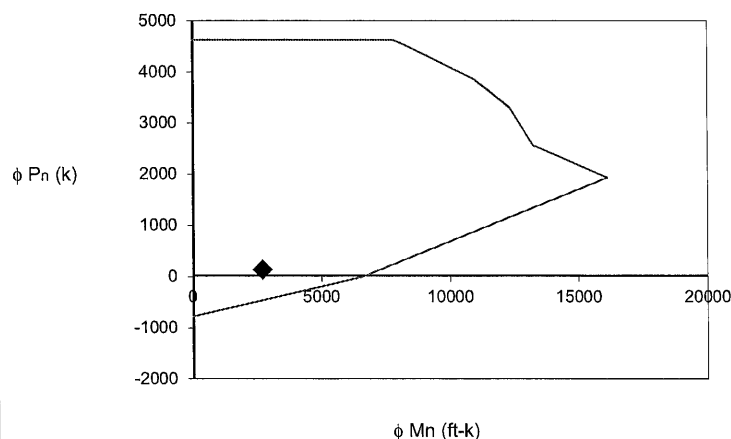


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 128.8 \text{ k} < 0.35 A_g f'_c = 3326 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 2376 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 150.27 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 150.27 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{prov.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{prov.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

where $A_{cv} = 2376 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 638.77 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.85 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

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CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 4629.6 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 14.57 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 7804 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 110 \text{ in} \quad a = C_b \beta_1 = 94 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 186 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 2540 kips AND 13389 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 6600 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -787 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 4630	0
AT MAXIMUM LOAC	= 4630	7804
AT 0 % TENSION	= 4543	8217
AT 25 % TENSION	= 3850	10967
AT 50 % TENSION	= 3313	12312
AT $\epsilon_t = 0.002$	= 2563	13244
AT BALANCED CONDITION	= 2540	13389
AT $\epsilon_t = 0.005$	= 1942	16118
AT FLEXURE ONLY	= 0	6600
AT TENSION ONLY	= -787	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 7459 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 23 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)
$$\bar{\epsilon} = (P_u / A) + (M_u y / I) = 0.446 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)
$$y = 99 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)
$$A = 2493 \text{ in}^2.$$
 (area of transformed section.)
$$I = 8145307 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 11.48 \text{ in.} \quad (\text{ACI 318-14 18.10.6.4})$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.} \quad (\text{ACI 318-14 18.10.6.2 \& 18.10.6.5})$$

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

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Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

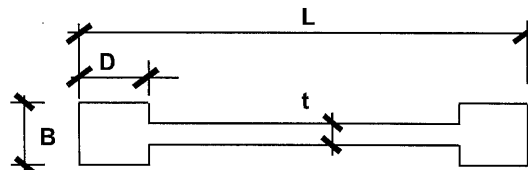
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 311$ k
FACTORED MOMENT LOAD $M_u = 17255$ ft-k
FACTORED SHEAR LOAD $V_u = 595$ k
LENGTH OF SHEAR WALL $L = 29$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 24$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 29.00$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 24.00$ in
BULB REINFORCING 10 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

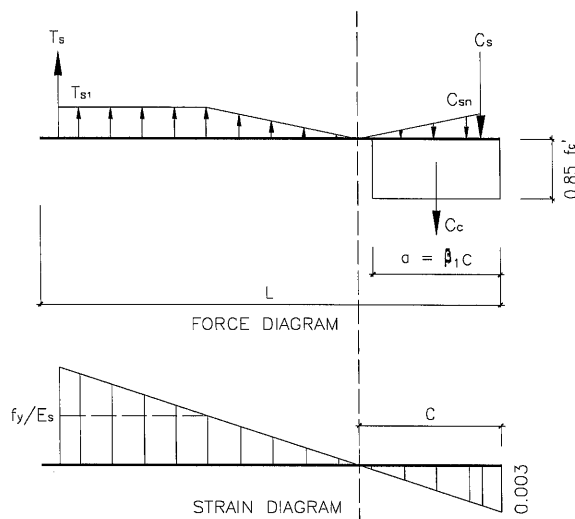
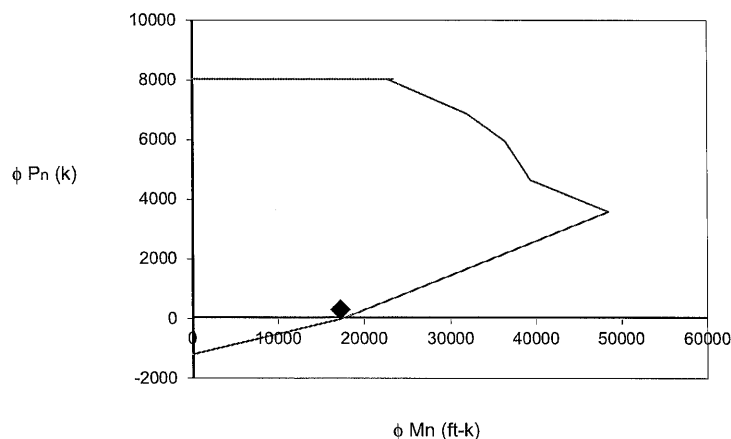


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 311 \text{ k} < 0.35 A_g f'_c = 5846 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 4176 \text{ in}^2$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$\begin{aligned} (\rho_t)_{\min.} &= 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 264.11 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}] \\ (\rho_l)_{\min.} &= 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 264.11 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}] \\ (\rho_t)_{\text{prov.}} &= 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}] \\ (\rho_l)_{\text{prov.}} &= 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}] \end{aligned}$$

where $A_{cv} = 4176 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u > 2 A_{cv} (f'_c)^{0.5}$, two curtains reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 1122.68 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.48 < 1.5)$$

$$\rho_t > \rho_l \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 8040.1 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)
 $A_{st} = 22.32 \text{ in}^2$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 23382 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 199 \text{ in} \quad a = C_b \beta_1 = 169 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 336 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 4608 kips AND 39863 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 17432 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -1205 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 8040	0
AT MAXIMUM LOAD	= 8040	23382
AT 0 % TENSION	= 8040	22779
AT 25 % TENSION	= 6883	31939
AT 50 % TENSION	= 5943	36416
AT $\epsilon_t = 0.002$	= 4645	39432
AT BALANCED CONDITION	= 4608	39863
AT $\epsilon_t = 0.005$	= 3599	48502
AT FLEXURE ONLY	= 0	17432
AT TENSION ONLY	= -1205	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.

THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 21183 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 38 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$\delta_u = 0.8 \text{ in.}$ (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$\xi_c = (P_u / A) + (M_u y / I) = 0.891 \text{ ksi.}$ (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$y = 174 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$A = 4356 \text{ in}^2$. (area of transformed section.)

$I = 43956219 \text{ in}^4$. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = $0.011 > 400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = $\text{MAX}(c - 0.1L, 0.5c) = 19.18 \text{ in.}$ (ACI 318-14 18.10.6.4)

The maximum hoop spacing = $\text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.}$ (ACI 318-14 18.10.6.2 & 18.10.6.5)

$A_{sh, \text{B DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2$

$A_{sh, \text{L DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2$

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Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

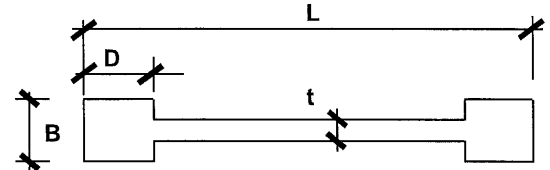
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 361$ k
FACTORED MOMENT LOAD $M_u = 161.5$ ft-k
FACTORED SHEAR LOAD $V_u = 19$ k
LENGTH OF SHEAR WALL $L = 8.5$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 12$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 6 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 8.50$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 12.00$ in
BULB REINFORCING 6 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

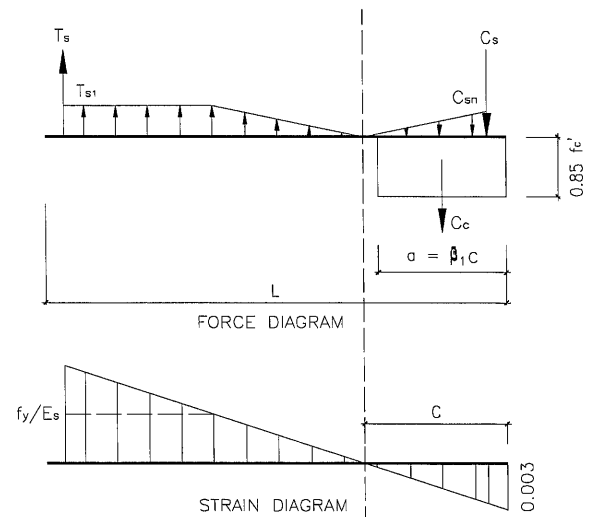
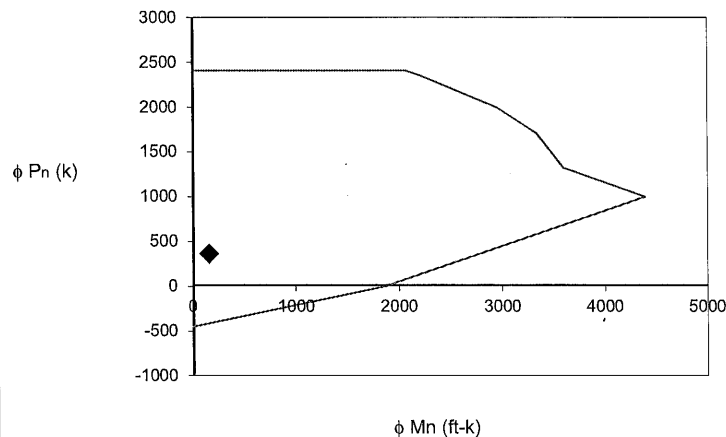


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 361 \text{ k} < 0.35 A_g f'_c = 1714 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 1224 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0020 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 77.41 \text{ kips} > V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0012 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 77.41 \text{ kips} > V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{provd.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{provd.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

where $A_{cv} = 1224 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c f'_c)^{0.5} + \rho_t f_y], \phi 8 A_{cv} (f'_c)^{0.5}] = 315.40 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 2.7 \quad (\text{for } h_w / L = 1.65 \text{ @ } [1.5, 2])$$

$$\rho_t > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

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CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 2410.4 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 8.37 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 2068 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 57 \text{ in} \quad a = C_b \beta_1 = 48 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 96 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 1311 kips AND 3634 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 1858 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -452 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 2410	0
AT MAXIMUM LOAD	= 2410	2068
AT 0 % TENSION	= 2354	2209
AT 25 % TENSION	= 1994	2954
AT 50 % TENSION	= 1714	3324
AT $\epsilon_t = 0.002$	= 1323	3593
AT BALANCED CONDITION	= 1311	3634
AT $\epsilon_t = 0.005$	= 1003	4393
AT FLEXURE ONLY	= 0	1858
AT TENSION ONLY	= -452	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 3009 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 19 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$$f_c = (P_u / A) + (M_u y / I) = 0.368 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$$y = 51 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$$A = 1291 \text{ in}^2.$$
 (area of transformed section.)

$$I = 1119584 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.013 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 9.57 \text{ in.}$$
 (ACI 318-14 18.10.6.4)

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_w)/3] = 8 \text{ in.o.c.}$$
 (ACI 318-14 18.10.6.2 & 18.10.6.5)

$$A_{sh, \text{B DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, \text{L DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

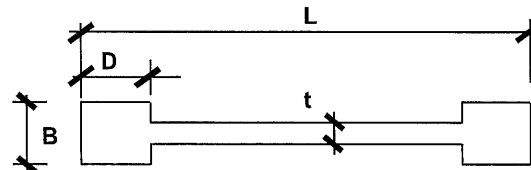
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
 REBAR YIELD STRESS $f_y = 60$ ksi
 FACTORED AXIAL LOAD $P_u = 215$ k
 FACTORED MOMENT LOAD $M_u = 11345$ ft-k
 FACTORED SHEAR LOAD $V_u = 382$ k
 LENGTH OF SHEAR WALL $L = 29.7$ ft
 THICKNESS OF WALL $t = 12$ in
 DEPTH AT FLANGE $D = 24$ in
 WIDTH AT FLANGE $B = 12$ in
 TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
 REINF. BARS AT BULB 10 # 5
 WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
 WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
 HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
 HOOP REINF - LENGTH DIR. 3 legs of # 4
 WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 29.70$ ft
 SHEAR WALL THICKNESS $t = 12.00$ in
 BULB END WIDTH $B = 12.00$ in
 BULB END DEPTH $D = 24.00$ in
 BULB REINFORCING 10 # 5
 WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
 WALL VERT. REINF 2 # 5 @ 12 in o.c.
 HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
 HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

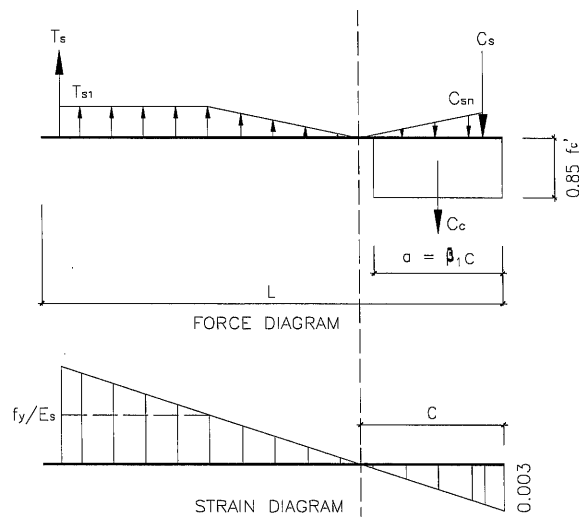
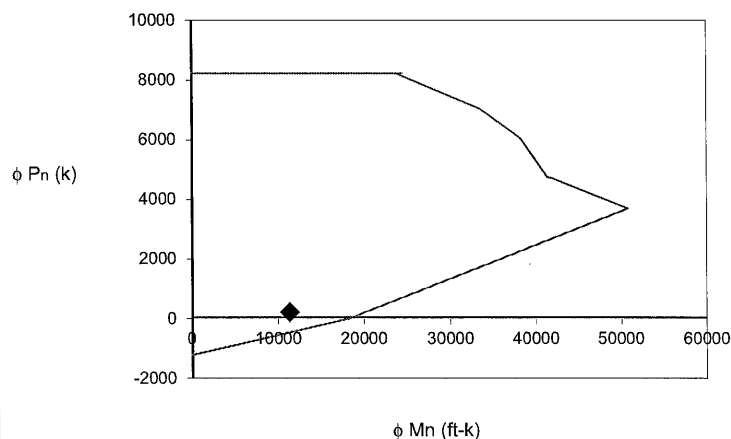


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 215 \text{ k} < 0.35 A_g f'_c = 5988 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 4277 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$\begin{aligned} (\rho_t)_{\min} &= 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 270.49 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}] \\ (\rho_l)_{\min} &= 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 270.49 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}] \\ (\rho_t)_{\text{prov.}} &= 0.0043 > (\rho_t)_{\min} \quad [\text{Satisfactory}] \\ (\rho_l)_{\text{prov.}} &= 0.0043 > (\rho_l)_{\min} \quad [\text{Satisfactory}] \end{aligned}$$

where $A_{cv} = 4277 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 1149.78 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.47 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 8231.1 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 22.75 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 24500 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 204 \text{ in} \quad a = C_b \beta_1 = 173 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 344.4 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 4724 kips AND 41757 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 18183 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -1229 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 8231	0
AT MAXIMUM LOAD	= 8231	24500
AT 0 % TENSION	= 8231	23802
AT 25 % TENSION	= 7053	33432
AT 50 % TENSION	= 6091	38137
AT $\epsilon_t = 0.002$	= 4762	41306
AT BALANCED CONDITION	= 4724	41757
AT $\epsilon_t = 0.005$	= 3691	50818
AT FLEXURE ONLY	= 0	18183
AT TENSION ONLY	= -1229	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 20858 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 36 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$$\xi = (P_u / A) + (M_u y / I) = 0.562 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$$y = 178 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$$A = 4460 \text{ in}^2.$$
 (area of transformed section.)

$$I = 47207785 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 18.20 \text{ in.}$$
 (ACI 318-14 18.10.6.4)

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.}$$
 (ACI 318-14 18.10.6.2 & 18.10.6.5)

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

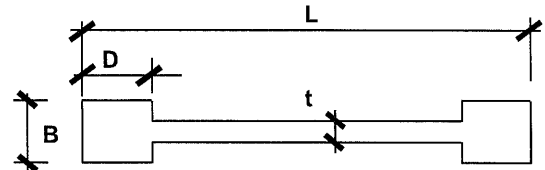
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
 REBAR YIELD STRESS $f_y = 60$ ksi
 FACTORED AXIAL LOAD $P_u = 468.5$ k
 FACTORED MOMENT LOAD $M_u = 2187.5$ ft-k
 FACTORED SHEAR LOAD $V_u = 125$ k
 LENGTH OF SHEAR WALL $L = 17.5$ ft
 THICKNESS OF WALL $t = 12$ in
 DEPTH AT FLANGE $D = 24$ in
 WIDTH AT FLANGE $B = 12$ in
 TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
 REINF. BARS AT BULB 10 # 5
 WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
 WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
 HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
 HOOP REINF - LENGTH DIR. 3 legs of # 4
 WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 17.50$ ft
 SHEAR WALL THICKNESS $t = 12.00$ in
 BULB END WIDTH $B = 12.00$ in
 BULB END DEPTH $D = 24.00$ in
 BULB REINFORCING 10 # 5
 WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
 WALL VERT. REINF 2 # 5 @ 12 in o.c.
 HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
 HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

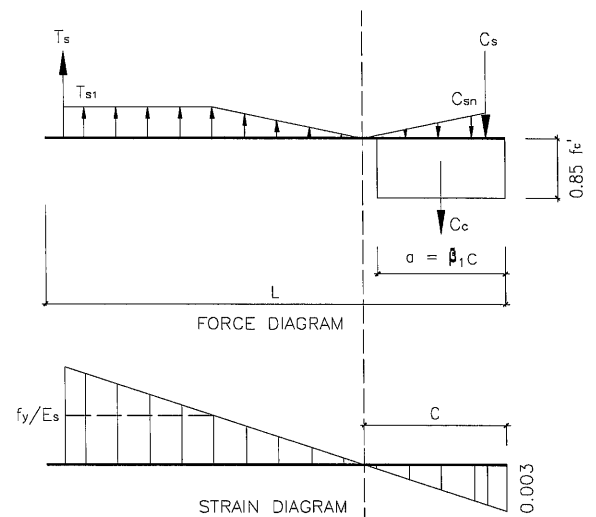
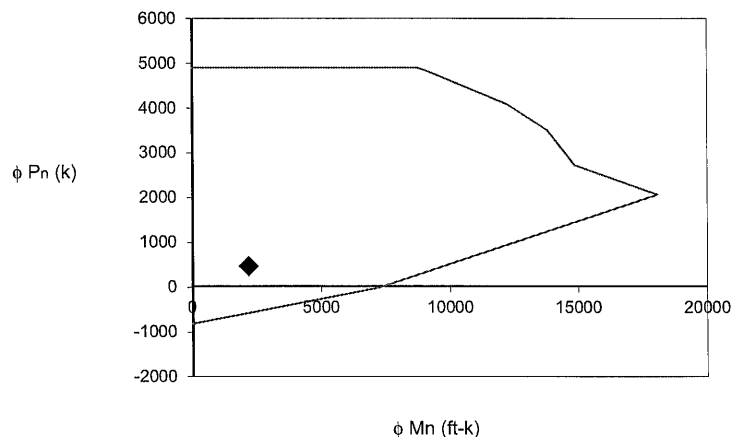


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 468.5 \text{ k} < 0.35 A_g f'_c = 3528 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 2520 \text{ in}^2$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0020 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 159.38 \text{ kips} > V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0012 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 159.38 \text{ kips} > V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{provd.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{provd.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

$$\text{where } A_{cv} = 2520 \text{ in}^2 \quad (\text{gross area of concrete section bounded by web thickness and length in the shear direction})$$

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 677.48 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

$$\text{where } \alpha_c = 0.60 \quad (\text{conservatively, ACI 318-14 21.2})$$

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.80 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 4902.4 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 15.19 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 8749 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 117 \text{ in} \quad a = C_b \beta_1 = 100 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 198 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 2706 kips AND 14995 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 7286 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -820 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 4902	0
AT MAXIMUM LOAD	= 4902	8749
AT 0 % TENSION	= 4828	9124
AT 25 % TENSION	= 4092	12248
AT 50 % TENSION	= 3524	13778
AT $\epsilon_t = 0.002$	= 2730	14832
AT BALANCED CONDITION	= 2706	14995
AT $\epsilon_t = 0.005$	= 2075	18076
AT FLEXURE ONLY	= 0	7286
AT TENSION ONLY	= -820	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 10480 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 32 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$$\xi = (P_u / A) + (M_u y / I) = 0.461 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$$y = 105 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$$A = 2642 \text{ in}^2.$$
 (area of transformed section.)

$$I = 9710064 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 16.13 \text{ in.}$$
 (ACI 318-14 18.10.6.4)

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.}$$
 (ACI 318-14 18.10.6.2 & 18.10.6.5)

$$A_{sh, \text{B DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, \text{L DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

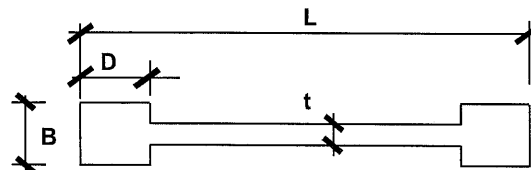
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
 REBAR YIELD STRESS $f_y = 60$ ksi
 FACTORED AXIAL LOAD $P_u = 401.5$ k
 FACTORED MOMENT LOAD $M_u = 1290$ ft-k
 FACTORED SHEAR LOAD $V_u = 86$ k
 LENGTH OF SHEAR WALL $L = 15$ ft
 THICKNESS OF WALL $t = 12$ in
 DEPTH AT FLANGE $D = 24$ in
 WIDTH AT FLANGE $B = 12$ in
 TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
 REINF. BARS AT BULB 10 # 5
 WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
 WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
 HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
 HOOP REINF - LENGTH DIR. 3 legs of # 4
 WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 15.00$ ft
 SHEAR WALL THICKNESS $t = 12.00$ in
 BULB END WIDTH $B = 12.00$ in
 BULB END DEPTH $D = 24.00$ in
 BULB REINFORCING 10 # 5
 WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
 WALL VERT. REINF 2 # 5 @ 12 in o.c.
 HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
 HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

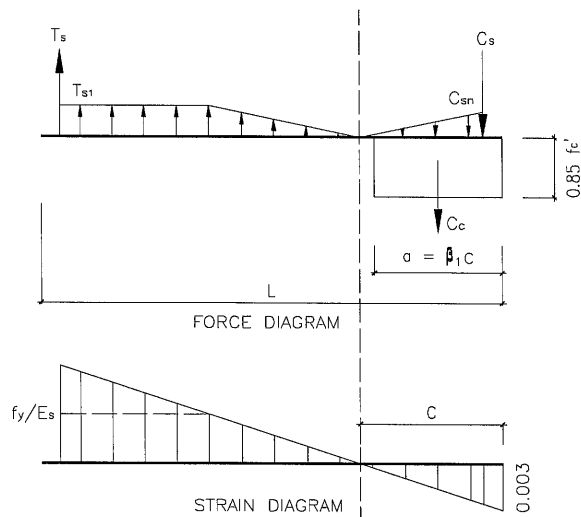
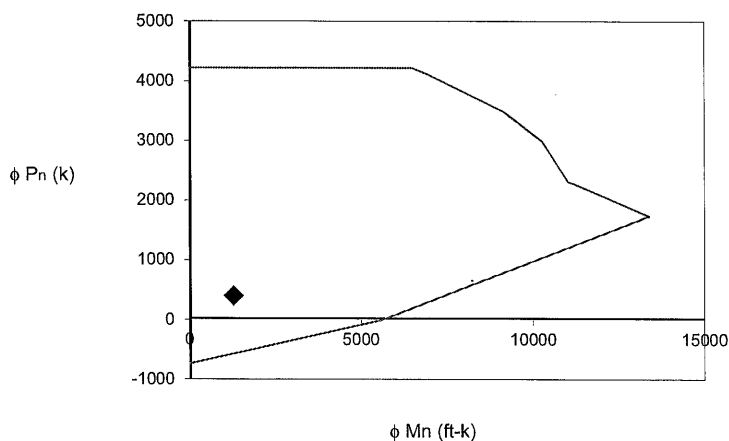


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 401.5 \text{ k} < 0.35 A_g f'_c = 3024 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 2160 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$\begin{aligned} (\rho_t)_{\min.} &= 0.0020 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 136.61 \text{ kips} > V_u, \text{ and bar size \# 5 horizontal}] \\ (\rho_l)_{\min.} &= 0.0012 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 136.61 \text{ kips} > V_u, \text{ and bar size \# 5 vertical}] \\ (\rho_t)_{\text{prov.}} &= 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}] \\ (\rho_l)_{\text{prov.}} &= 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}] \end{aligned}$$

$$\text{where } A_{cv} = 2160 \text{ in}^2 \quad (\text{gross area of concrete section bounded by web thickness and length in the shear direction})$$

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 580.70 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.60 \quad (\text{conservatively, ACI 318-14 21.2})$$

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.93 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 4220.3 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 13.64 \text{ in}^2.$$

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DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 6487 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 99 \text{ in} \quad a = C_b \beta_1 = 85 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 168 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 2292 kips AND 11148 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 5630 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -737 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 4220	0
AT MAXIMUM LOAD	= 4220	6487
AT 0 % TENSION	= 4115	6943
AT 25 % TENSION	= 3486	9174
AT 50 % TENSION	= 2997	10264
AT $\epsilon_t = 0.002$	= 2313	11027
AT BALANCED CONDITION	= 2292	11148
AT $\epsilon_t = 0.005$	= 1743	13389
AT FLEXURE ONLY	= 0	5630
AT TENSION ONLY	= -737	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 7949 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 28 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$$\xi = (P_u / A) + (M_u y / I) = 0.404 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$$y = 90 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$$A = 2270 \text{ in}^2.$$
 (area of transformed section.)

$$I = 6128259 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 14.18 \text{ in.}$$
 (ACI 318-14 18.10.6.4)

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_w)/3] = 8 \text{ in.o.c.}$$
 (ACI 318-14 18.10.6.2 & 18.10.6.5)

$$A_{sh, B \text{ DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, L \text{ DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

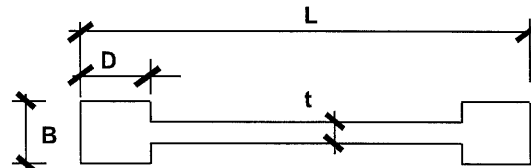
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 224$ k
FACTORED MOMENT LOAD $M_u = 232.5$ ft-k
FACTORED SHEAR LOAD $V_u = 31$ k
LENGTH OF SHEAR WALL $L = 7.5$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 12$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 6 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 7.50$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 12.00$ in
BULB REINFORCING 6 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

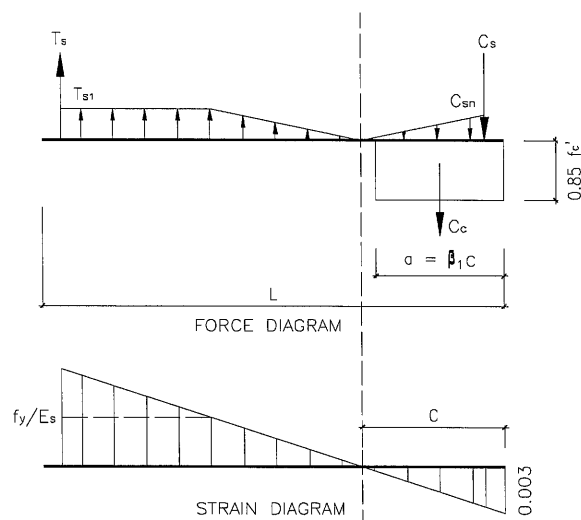
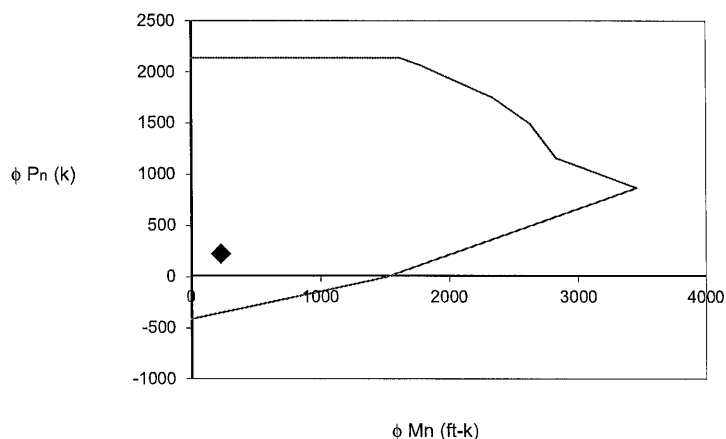


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 224 \text{ k} < 0.35 A_g f'_c = 1512 \text{ k} \quad [\text{Satisfactory}]$$

where $A_g = 1080 \text{ in}^2$.



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$\begin{aligned} (\rho_t)_{\min} &= 0.0020 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 68.31 \text{ kips} > V_u, \text{ and bar size \# 5 horizontal}] \\ (\rho_l)_{\min} &= 0.0012 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 68.31 \text{ kips} > V_u, \text{ and bar size \# 5 vertical}] \\ (\rho_t)_{\text{prov.}} &= 0.0043 > (\rho_t)_{\min} \quad [\text{Satisfactory}] \\ (\rho_l)_{\text{prov.}} &= 0.0043 > (\rho_l)_{\min} \quad [\text{Satisfactory}] \end{aligned}$$

where $A_{cv} = 1080 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 260.30 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 2.3 \quad (\text{for } h_w / L = 1.87 \text{ @ } [1.5, 2])$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 2137.5 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.65 \quad (\text{ACI 318-14 21.2})$$

$$A_{st} = 7.75 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 1622 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 50 \text{ in} \quad a = C_b \beta_1 = 42 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 84 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 1145 kips AND 2864 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 1514 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -419 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 2138	0
AT MAXIMUM LOAD	= 2138	1622
AT 0 % TENSION	= 2069	1773
AT 25 % TENSION	= 1751	2340
AT 50 % TENSION	= 1504	2623
AT $\epsilon_t = 0.002$	= 1156	2832
AT BALANCED CONDITION	= 1145	2864
AT $\epsilon_t = 0.005$	= 871	3453
AT FLEXURE ONLY	= 0	1514
AT TENSION ONLY	= -419	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 2156 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900 \quad (\text{ACI 318-14 21.2})$$

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

$$\text{where } c = 15 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_n \text{ loads.)}$$

$$\delta_u = 0.8 \text{ in. (design displacement, assume } 0.005 h_w \text{ as a conservative short cut, see ACI 318-14 18.10.6.2.)}$$

$$\epsilon_c = (P_u / A) + (M_u y / I) = 0.359 \text{ ksi. (the maximum extreme fiber compressive stress at } P_u \text{ \& } M_u \text{ loads.)}$$

$$y = 45 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_u \text{ loads.)}$$

$$A = 1142 \text{ in}^2. \text{ (area of transformed section.)}$$

$$I = 771082 \text{ in}^4. \text{ (moment of inertia of transformed section.)}$$

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.013 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED!

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 7.41 \text{ in. (ACI 318-14 18.10.6.4)}$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c. (ACI 318-14 18.10.6.2 \& 18.10.6.5)}$$

$$A_{sh, \text{B DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$

$$A_{sh, \text{L DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$



65

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

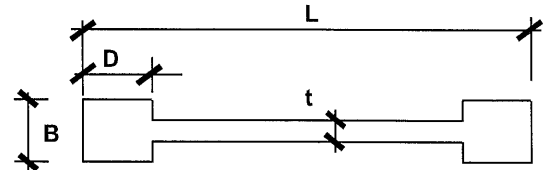
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
 REBAR YIELD STRESS $f_y = 60$ ksi
 FACTORED AXIAL LOAD $P_u = 345.9$ k
 FACTORED MOMENT LOAD $M_u = 10368$ ft-k
 FACTORED SHEAR LOAD $V_u = 360$ k
 LENGTH OF SHEAR WALL $L = 28.8$ ft
 THICKNESS OF WALL $t = 12$ in
 DEPTH AT FLANGE $D = 24$ in
 WIDTH AT FLANGE $B = 12$ in
 TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
 REINF. BARS AT BULB 10 # 5
 WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
 WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
 HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
 HOOP REINF - LENGTH DIR. 3 legs of # 4
 WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 28.80$ ft
 SHEAR WALL THICKNESS $t = 12.00$ in
 BULB END WIDTH $B = 12.00$ in
 BULB END DEPTH $D = 24.00$ in
 BULB REINFORCING 10 # 5
 WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
 WALL VERT. REINF 2 # 5 @ 12 in o.c.
 HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
 HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

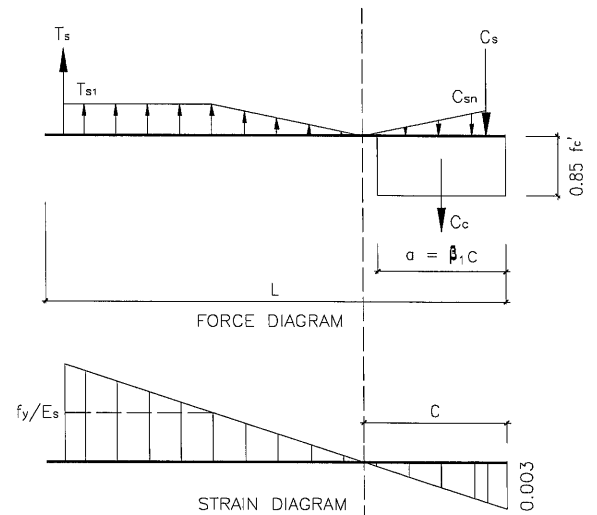
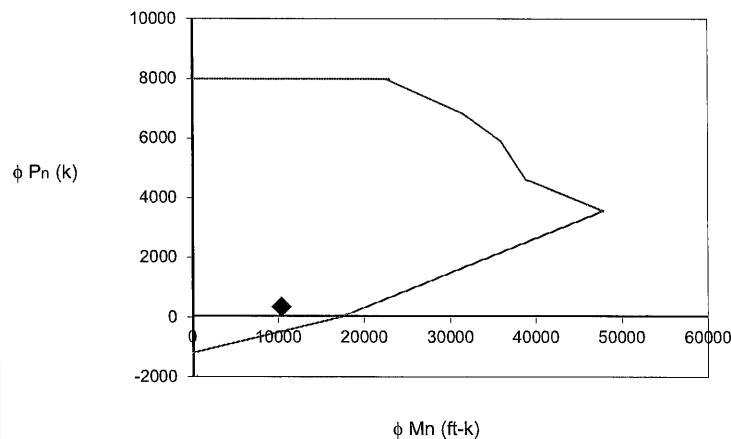


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 345.9 \text{ k} < 0.35 A_g f'_c = 5806 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 4147 \text{ in}^2$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 262.29 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 262.29 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{prov.}} = 0.0043 > (\rho_t)_{\min} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{prov.}} = 0.0043 > (\rho_l)_{\min} \quad [\text{Satisfactory}]$$

where $A_{cv} = 4147 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 1114.94 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.49 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 7985.5 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)
 $A_{st} = 22.20 \text{ in}^2$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 23068 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 197 \text{ in} \quad a = C_b \beta_1 = 168 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 333.6 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 4575 kips AND 39330 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 17220 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -1199 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 7986	0
AT MAXIMUM LOAD	= 7986	23068
AT 0 % TENSION	= 7986	22491
AT 25 % TENSION	= 6834	31519
AT 50 % TENSION	= 5901	35931
AT $\epsilon_t = 0.002$	= 4612	38905
AT BALANCED CONDITION	= 4575	39330
AT $\epsilon_t = 0.005$	= 3572	47850
AT FLEXURE ONLY	= 0	17220
AT TENSION ONLY	= -1199	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 21353 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 39 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$\delta_u = 0.8 \text{ in.}$ (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u y / I) = 0.579 \text{ ksi.}$ (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$y = 173 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$A = 4326 \text{ in}^2$ (area of transformed section.)

$I = 43055434 \text{ in}^4$ (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.011 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = $\text{MAX}(c - 0.1L, 0.5c) = 19.66 \text{ in.}$ (ACI 318-14 18.10.6.4)

The maximum hoop spacing = $\text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.}$ (ACI 318-14 18.10.6.2 & 18.10.6.5)

$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2$

$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2$



Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

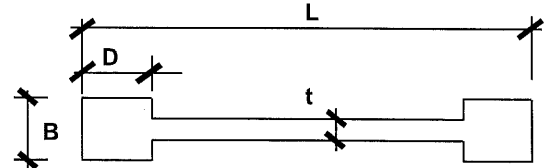
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
 REBAR YIELD STRESS $f_y = 60$ ksi
 FACTORED AXIAL LOAD $P_u = 204$ k
 FACTORED MOMENT LOAD $M_u = 1989$ ft-k
 FACTORED SHEAR LOAD $V_u = 117$ k
 LENGTH OF SHEAR WALL $L = 17$ ft
 THICKNESS OF WALL $t = 12$ in
 DEPTH AT FLANGE $D = 24$ in
 WIDTH AT FLANGE $B = 12$ in
 TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
 REINF. BARS AT BULB 10 # 5
 WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
 WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
 HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
 HOOP REINF - LENGTH DIR. 3 legs of # 4
 WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 17.00$ ft
 SHEAR WALL THICKNESS $t = 12.00$ in
 BULB END WIDTH $B = 12.00$ in
 BULB END DEPTH $D = 24.00$ in
 BULB REINFORCING 10 # 5
 WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
 WALL VERT. REINF 2 # 5 @ 12 in o.c.
 HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
 HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

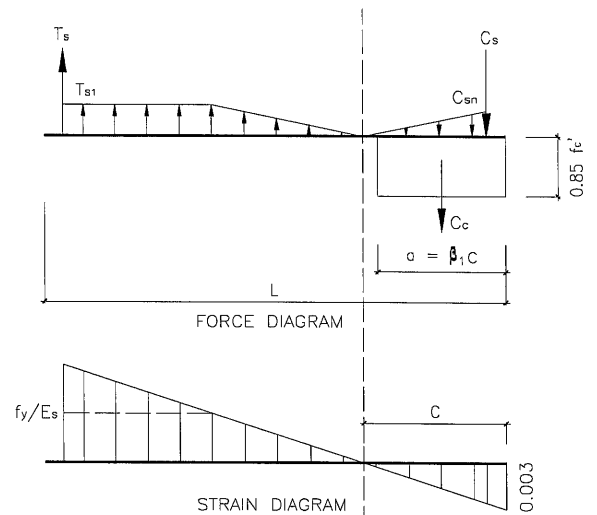
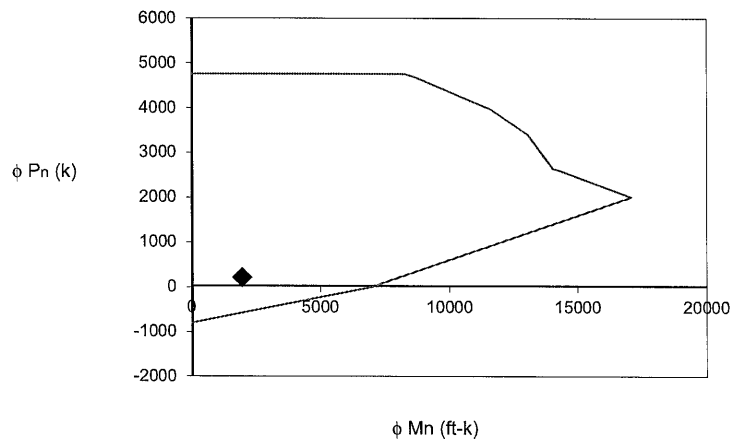


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 204 \text{ k} < 0.35 A_g f'_c = 3427 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 2448 \text{ in}^2$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0020 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 154.83 \text{ kips} > V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0012 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 154.83 \text{ kips} > V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{prov.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{prov.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

$$\text{where } A_{cv} = 2448 \text{ in}^2 \quad (\text{gross area of concrete section bounded by web thickness and length in the shear direction})$$

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 658.13 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.60 \quad (\text{conservatively, ACI 318-14 21.2})$$

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.82 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 4766 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.65 \quad (\text{ACI 318-14 21.2})$$

$$A_{st} = 14.88 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 8270 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE

STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 114 \text{ in} \quad a = C_b \beta_1 = 97 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 192 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 2623 kips AND 14181 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 6939 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c {}^{0.5} 4 L t) = -804 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 4766	0
AT MAXIMUM LOAD	= 4766	8270
AT 0 % TENSION	= 4686	8665
AT 25 % TENSION	= 3971	11598
AT 50 % TENSION	= 3418	13035
AT $\epsilon_t = 0.002$	= 2646	14027
AT BALANCED CONDITION	= 2623	14181
AT $\epsilon_t = 0.005$	= 2009	17083
AT FLEXURE ONLY	= 0	6939
AT TENSION ONLY	= -804	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 8335 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900 \quad (\text{ACI 318-14 21.2})$$

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

$$\text{where } c = 25 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_n \text{ loads.)}$$

$$\delta_u = 0.8 \text{ in. (design displacement, assume } 0.005 h_w \text{ as a conservative short cut, see ACI 318-14 18.10.6.2.)}$$

$$f_c = (P_u / A) + (M_u y / I) = 0.353 \text{ ksi. (the maximum extreme fiber compressive stress at } P_u \text{ \& } M_u \text{ loads.)}$$

$$y = 102 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_u \text{ loads.)}$$

$$A = 2568 \text{ in}^2. \text{ (area of transformed section.)}$$

$$I = 8904785 \text{ in}^4. \text{ (moment of inertia of transformed section.)}$$

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 12.61 \text{ in. (ACI 318-14 18.10.6.4)}$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_w)/3] = 8 \text{ in.o.c. (ACI 318-14 18.10.6.2 \& 18.10.6.5)}$$

$$A_{sh, \text{B DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$

$$A_{sh, \text{L DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

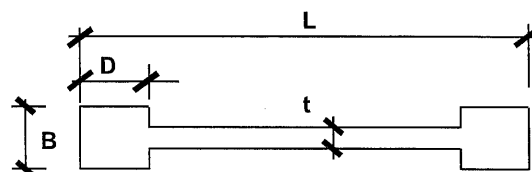
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 324.7$ k
FACTORED MOMENT LOAD $M_u = 4266$ ft-k
FACTORED SHEAR LOAD $V_u = 237$ k
LENGTH OF SHEAR WALL $L = 18$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 24$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 18.00$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 24.00$ in
BULB REINFORCING 10 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

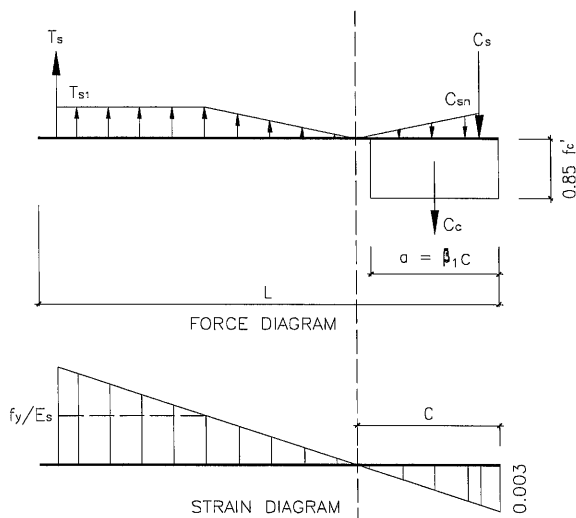
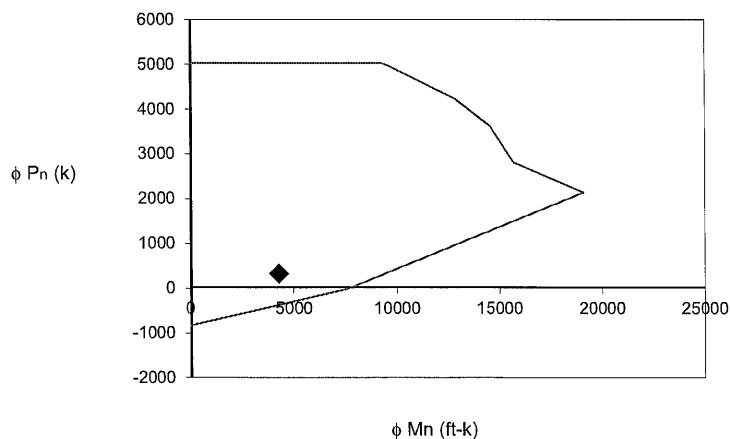


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 324.7 \text{ k} < 0.35 A_g f'_c = 3629 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 2592 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 163.93 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 163.93 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{provd.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{provd.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

$$\text{where } A_{cv} = 2592 \text{ in}^2 \quad (\text{gross area of concrete section bounded by web thickness and length in the shear direction})$$

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c f'_c)^{0.5} + \rho_t f_y, \phi 8 A_{cv} (f'_c)^{0.5}] = 696.84 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.60 \quad (\text{conservatively, ACI 318-14 21.2})$$

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.78 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 5038.9 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 15.50 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 9241 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 121 \text{ in} \quad a = C_b \beta_1 = 103 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 204 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 2788 kips AND 15831 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 7641 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -837 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 5039	0
AT MAXIMUM LOAD	= 5039	9241
AT 0 % TENSION	= 4971	9594
AT 25 % TENSION	= 4214	12914
AT 50 % TENSION	= 3629	14541
AT $\epsilon_t = 0.002$	= 2813	15660
AT BALANCED CONDITION	= 2788	15831
AT $\epsilon_t = 0.005$	= 2141	19095
AT FLEXURE ONLY	= 0	7641
AT TENSION ONLY	= -837	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 9980 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 29 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$$f_c = (P_u / A) + (M_u y / I) = 0.643 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$$y = 108 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$$A = 2717 \text{ in}^2.$$
 (area of transformed section.)

$$I = 10562483 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 14.59 \text{ in.}$$
 (ACI 318-14 18.10.6.4)

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_w)/3] = 8 \text{ in.o.c.}$$
 (ACI 318-14 18.10.6.2 & 18.10.6.5)

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

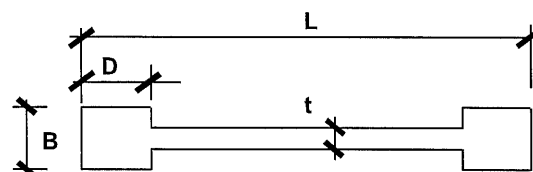
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 135.3$ k
FACTORED MOMENT LOAD $M_u = 180$ ft-k
FACTORED SHEAR LOAD $V_u = 24$ k
LENGTH OF SHEAR WALL $L = 7.5$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 12$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 6 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 7.50$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 12.00$ in
BULB REINFORCING 6 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

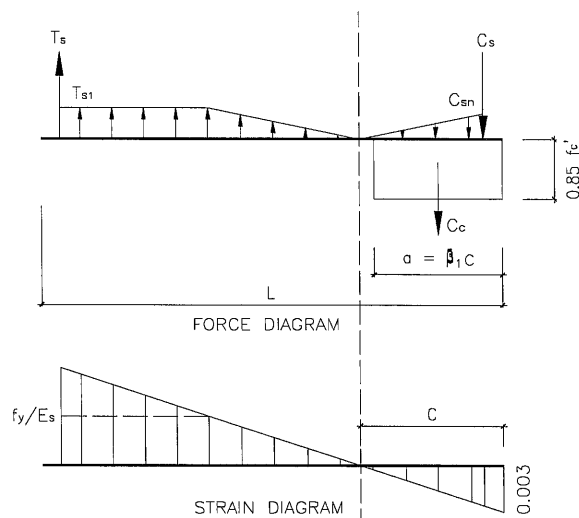
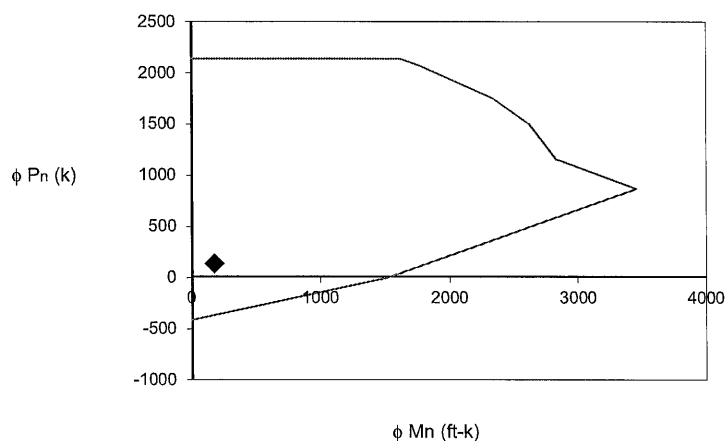
THE WALL DESIGN IS ADEQUATE



ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$P_u = 135.3$ k $< 0.35 A_g f'_c = 1512$ k [Satisfactory]
where $A_g = 1080$ in².



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$(\rho_t)_{min.} = 0.0020$ [for $A_{cv} (f'_c)^{0.5} = 68.31$ kips $> V_u$, and bar size # 5 horizontal]
 $(\rho_l)_{min.} = 0.0012$ [for $A_{cv} (f'_c)^{0.5} = 68.31$ kips $> V_u$, and bar size # 5 vertical]
 $(\rho_t)_{prov.} = 0.0043 > (\rho_t)_{min.}$ [Satisfactory]
 $(\rho_l)_{prov.} = 0.0043 > (\rho_l)_{min.}$ [Satisfactory]

where $A_{cv} = 1080$ in² (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 260.30$ kips $> V_u$ [Satisfactory]

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$\alpha_c = 2.3$ (for $h_w / L = 1.87$ @ [1.5, 2])

$\rho_l > \rho_t$ [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 2137.5 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 7.75 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 1622 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 50 \text{ in} \quad a = C_b \beta_1 = 42 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 84 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 1145 kips AND 2864 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 1514 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -419 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 2138	0
AT MAXIMUM LOAD	= 2138	1622
AT 0 % TENSION	= 2069	1773
AT 25 % TENSION	= 1751	2340
AT 50 % TENSION	= 1504	2623
AT $\epsilon_t = 0.002$	= 1156	2832
AT BALANCED CONDITION	= 1145	2864
AT $\epsilon_t = 0.005$	= 871	3453
AT FLEXURE ONLY	= 0	1514
AT TENSION ONLY	= -419	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 1914 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 13 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)
$$f_c = (P_u / A) + (M_u y / I) = 0.244 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)
$$y = 45 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)
$$A = 1142 \text{ in}^2.$$
 (area of transformed section.)
$$I = 771082 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.013 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 6.37 \text{ in.}$$
 (ACI 318-14 18.10.6.4)

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.}$$
 (ACI 318-14 18.10.6.2 & 18.10.6.5)

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

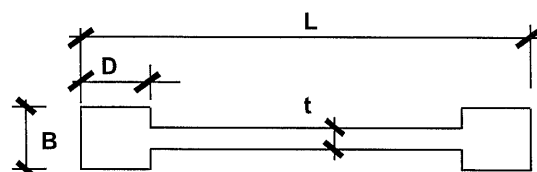
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 693$ k
FACTORED MOMENT LOAD $M_u = 41850$ ft-k
FACTORED SHEAR LOAD $V_u = 1080$ k
LENGTH OF SHEAR WALL $L = 38.75$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 36$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 18 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 38.75$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 36.00$ in
BULB REINFORCING 18 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

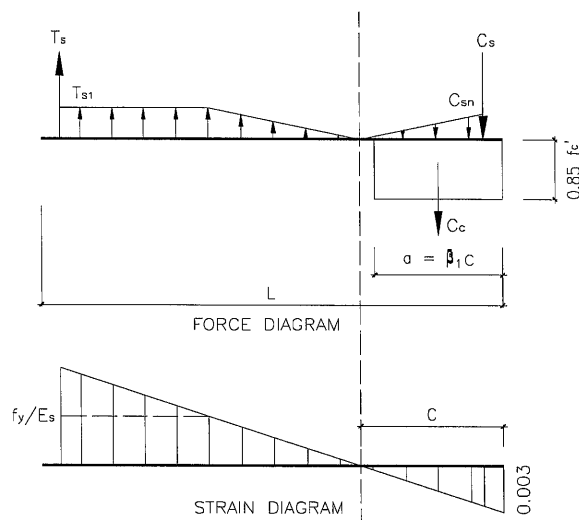
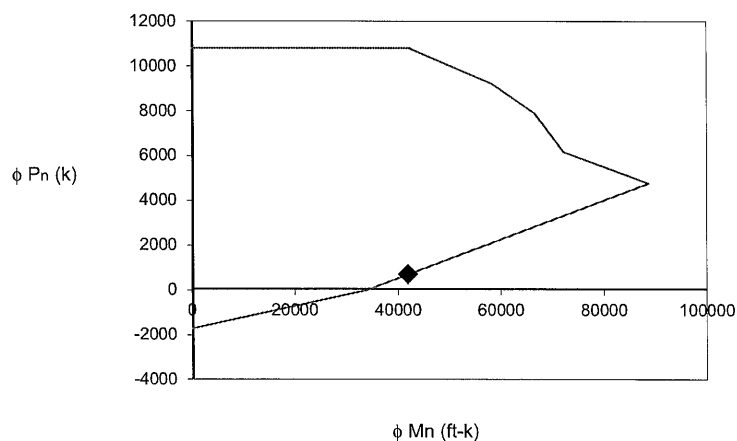
THE WALL DESIGN IS ADEQUATE



ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$P_u = 693$ k $< 0.35 A_g f'_c = 7812$ k [Satisfactory]
where $A_g = 5580$ in².



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$(\rho_t)_{min.} = 0.0025$ [for $A_{cv} (f'_c)^{0.5} = 352.91$ kips $< V_u$, and bar size # 5 horizontal]
 $(\rho_l)_{min.} = 0.0025$ [for $A_{cv} (f'_c)^{0.5} = 352.91$ kips $< V_u$, and bar size # 5 vertical]
 $(\rho_t)_{prov.} = 0.0043 > (\rho_t)_{min.}$ [Satisfactory]
 $(\rho_l)_{prov.} = 0.0043 > (\rho_l)_{min.}$ [Satisfactory]

where $A_{cv} = 5580$ in² (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u > 2 A_{cv} (f'_c)^{0.5}$, two curtains reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 1500.14$ kips $> V_u$ [Satisfactory]

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$\alpha_c = 3.0$ (for $h_w / L = 0.36 < 1.5$)

$\rho_l > \rho_t$ [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 10810 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.65 \quad (\text{ACI 318-14 21.2})$$

$$A_{st} = 32.09 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 42441 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 265 \text{ in} \quad a = C_b \beta_1 = 225 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 447 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 6124 kips AND 72911 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 33784 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -1733 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 10810	0
AT MAXIMUM LOAD	= 10810	42441
AT 0 % TENSION	= 10810	42069
AT 25 % TENSION	= 9197	58389
AT 50 % TENSION	= 7932	66458
AT $\epsilon_t = 0.002$	= 6176	72104
AT BALANCED CONDITION	= 6124	72911
AT $\epsilon_t = 0.005$	= 4766	88754
AT FLEXURE ONLY	= 0	33784
AT TENSION ONLY	= -1733	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 44693 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900 \quad (\text{ACI 318-14 21.2})$$

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

$$\text{where } c = 59 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_n \text{ loads.)}$$

$$\delta_u = 0.8 \text{ in. (design displacement, assume } 0.005 h_w \text{ as a conservative short cut, see ACI 318-14 18.10.6.2.)}$$

$$f_c = (P_u / A) + (M_u y / I) = 1.229 \text{ ksi. (the maximum extreme fiber compressive stress at } P_u \text{ \& } M_u \text{ loads.)}$$

$$y = 233 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_u \text{ loads.)}$$

$$A = 5838 \text{ in}^2. \text{ (area of transformed section.)}$$

$$I = 105195341 \text{ in}^4. \text{ (moment of inertia of transformed section.)}$$

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.013 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}\{c - 0.1L, 0.5c\} = 29.73 \text{ in. (ACI 318-14 18.10.6.4)}$$

$$\text{The maximum hoop spacing} = \text{MIN}\{B/4, 6d_b, 6, 4 + (14 - h_x)/3\} = 8 \text{ in.o.c. (ACI 318-14 18.10.6.2 \& 18.10.6.5)}$$

$$A_{sh, \text{B DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, \text{L DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

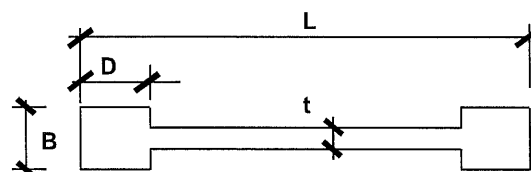
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 147$ k
FACTORED MOMENT LOAD $M_u = 1218$ ft-k
FACTORED SHEAR LOAD $V_u = 84$ k
LENGTH OF SHEAR WALL $L = 14.5$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 24$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 14.50$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 24.00$ in
BULB REINFORCING 10 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

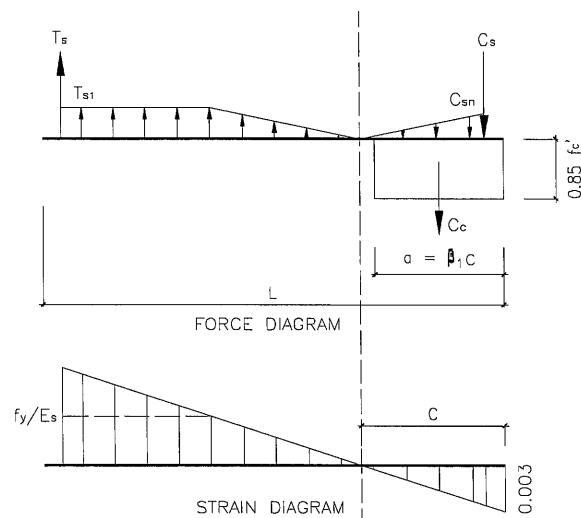
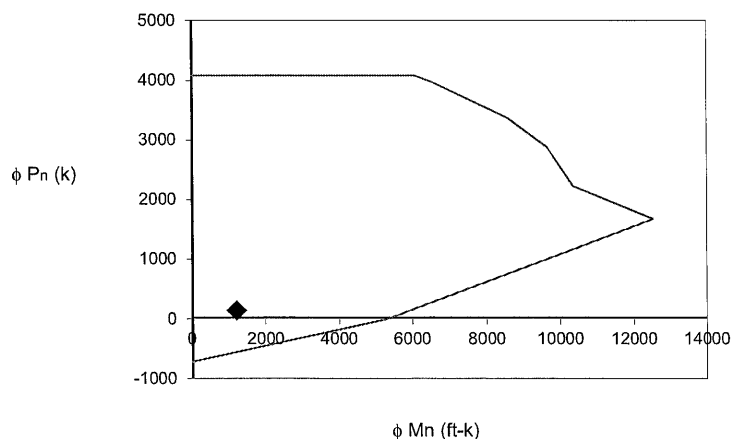


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 147 \text{ k} < 0.35 A_g f'_c = 2923 \text{ k} \quad [\text{Satisfactory}]$$

where $A_g = 2088 \text{ in}^2$.



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min} = 0.0020 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 132.06 \text{ kips} > V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min} = 0.0012 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 132.06 \text{ kips} > V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{prov.}} = 0.0043 > (\rho_t)_{\min} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{prov.}} = 0.0043 > (\rho_l)_{\min} \quad [\text{Satisfactory}]$$

where $A_{cv} = 2088 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 561.34 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.97 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 4083.9 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.65 \quad (\text{ACI 318-14 21.2})$$

$$A_{st} = 13.33 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 6075 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 96 \text{ in} \quad a = C_b \beta_1 = 81 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 162 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 2209 kips AND 10444 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 5322 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -720 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 4084	0
AT MAXIMUM LOAD	= 4084	6075
AT 0 % TENSION	= 3973	6541
AT 25 % TENSION	= 3364	8611
AT 50 % TENSION	= 2891	9622
AT $\epsilon_t = 0.002$	= 2230	10332
AT BALANCED CONDITION	= 2209	10444
AT $\epsilon_t = 0.005$	= 1677	12535
AT FLEXURE ONLY	= 0	5322
AT TENSION ONLY	= -720	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 6169 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900 \quad (\text{ACI 318-14 21.2})$$

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

$$\text{where } c = 22 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_n \text{ loads.)}$$

$$\delta_u = 0.8 \text{ in. (design displacement, assume } 0.005 h_w \text{ as a conservative short cut, see ACI 318-14 18.10.6.2.)}$$

$$\epsilon_c = (P_u / A) + (M_u y / I) = 0.297 \text{ ksi. (the maximum extreme fiber compressive stress at } P_u \text{ \& } M_u \text{ loads.)}$$

$$y = 87 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_u \text{ loads.)}$$

$$A = 2195 \text{ in}^2. \text{ (area of transformed section.)}$$

$$I = 5538570 \text{ in}^4. \text{ (moment of inertia of transformed section.)}$$

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 10.88 \text{ in. (ACI 318-14 18.10.6.4)}$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c. (ACI 318-14 18.10.6.2 \& 18.10.6.5)}$$

$$A_{sh, \text{B DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$

$$A_{sh, \text{L DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$



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Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

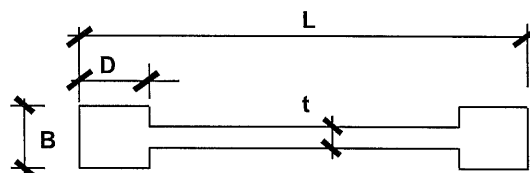
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 1064$ k
FACTORED MOMENT LOAD $M_u = 10716$ ft-k
FACTORED SHEAR LOAD $V_u = 376$ k
LENGTH OF SHEAR WALL $L = 28.5$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 24$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 28.50$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D + 5.11 = 29.11$ in
BULB REINFORCING 10 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

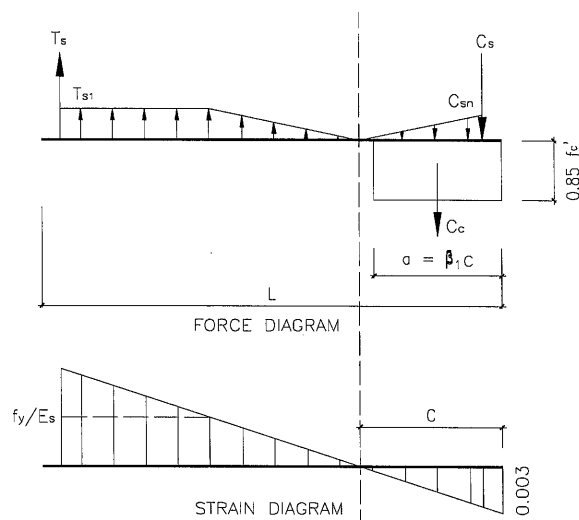
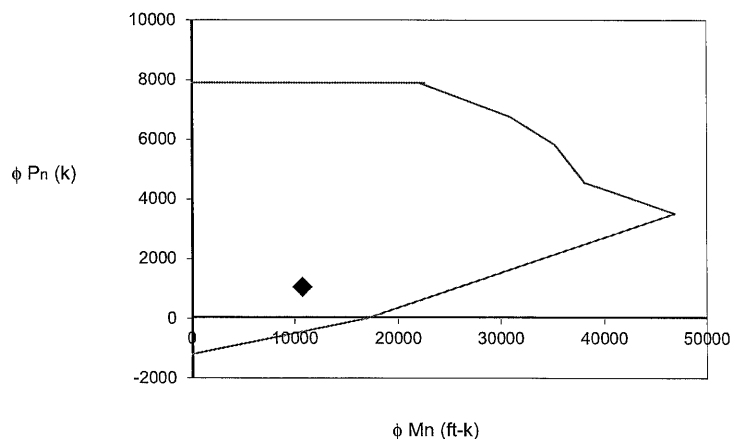


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 1064 \text{ k} < 0.35 A_g f'_c = 5746 \text{ k} \quad [\text{Satisfactory}]$$

where $A_g = 4104 \text{ in}^2$.



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 259.56 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 259.56 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{prov.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{prov.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

where $A_{cv} = 4104 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 1103.33 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.49 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 7903.7 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)
 $A_{st} = 22.01 \text{ in}^2$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 22600 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 195 \text{ in} \quad a = C_b \beta_1 = 166 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 330 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 4525 kips AND 38537 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 16905 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -1189 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 7904	0
AT MAXIMUM LOAD	= 7904	22600
AT 0 % TENSION	= 7904	22062
AT 25 % TENSION	= 6762	30893
AT 50 % TENSION	= 5838	35210
AT $\epsilon_t = 0.002$	= 4562	38121
AT BALANCED CONDITION	= 4525	38537
AT $\epsilon_t = 0.005$	= 3533	46877
AT FLEXURE ONLY	= 0	16905
AT TENSION ONLY	= -1189	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 28592 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 58 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)
 $\delta_u = 0.8 \text{ in.}$ (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)
 $f_c = (P_u / A) + (M_u y / I) = 0.776 \text{ ksi.}$ (the maximum extreme fiber compressive stress at P_u & M_u loads.)
 $y = 171 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)
 $A = 4281 \text{ in}^2$ (area of transformed section.)
 $I = 41727463 \text{ in}^4$ (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 29.11 \text{ in.} \quad (\text{ACI 318-14 18.10.6.4})$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.} \quad (\text{ACI 318-14 18.10.6.2 \& 18.10.6.5})$$

$$A_{sh, B \text{ DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2$$

$$A_{sh, L \text{ DIR}} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2$$



Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

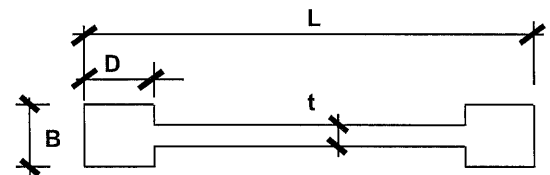
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 255.2$ k
FACTORED MOMENT LOAD $M_u = 5880$ ft-k
FACTORED SHEAR LOAD $V_u = 336$ k
LENGTH OF SHEAR WALL $L = 17.5$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 24$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 17.50$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 24.00$ in
BULB REINFORCING 10 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

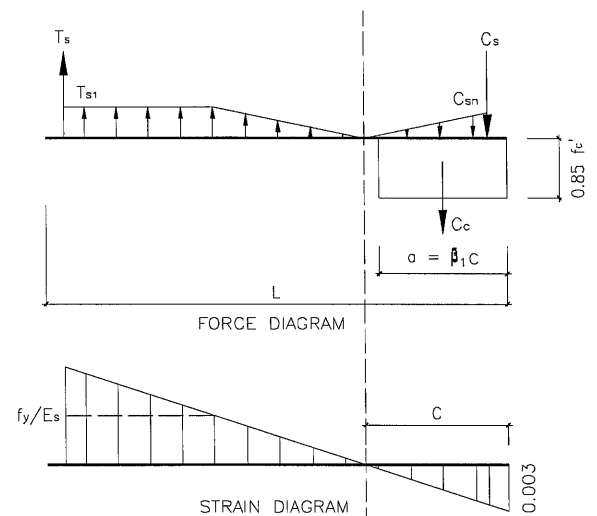
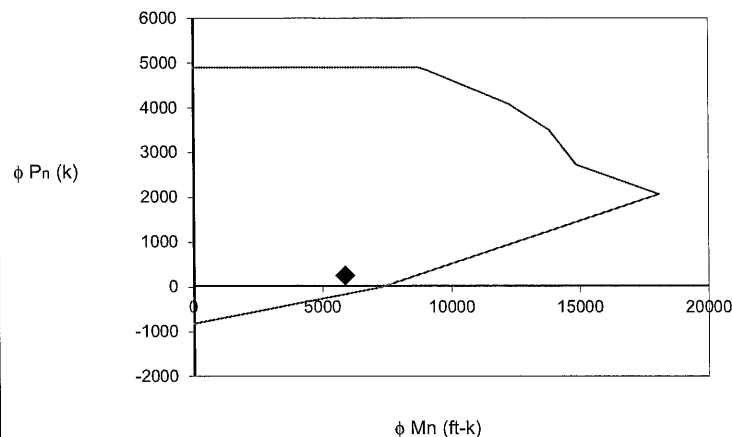


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 255.2 \text{ k} < 0.35 A_g f'_c = 3528 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 2520 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$\begin{aligned} (\rho_t)_{\min.} &= 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 159.38 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}] \\ (\rho_l)_{\min.} &= 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 159.38 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}] \\ (\rho_t)_{\text{provd.}} &= 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}] \\ (\rho_l)_{\text{provd.}} &= 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}] \end{aligned}$$

$$\text{where } A_{cv} = 2520 \text{ in}^2 \quad (\text{gross area of concrete section bounded by web thickness and length in the shear direction})$$

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u > 2 A_{cv} (f'_c)^{0.5}$, two curtains reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 677.48 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.60 \quad (\text{conservatively, ACI 318-14 21.2})$$

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.80 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

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CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 4902.4 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 15.19 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 8749 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 117 \text{ in} \quad a = C_b \beta_1 = 100 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 198 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 2706 kips AND 14995 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 7286 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -820 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 4902	0
AT MAXIMUM LOAD	= 4902	8749
AT 0 % TENSION	= 4828	9124
AT 25 % TENSION	= 4092	12248
AT 50 % TENSION	= 3524	13778
AT $\epsilon_t = 0.002$	= 2730	14832
AT BALANCED CONDITION	= 2706	14995
AT $\epsilon_t = 0.005$	= 2075	18076
AT FLEXURE ONLY	= 0	7286
AT TENSION ONLY	= -820	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 9081 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 27 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$$f_c = (P_u / A) + (M_u y / I) = 0.860 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$$y = 105 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$$A = 2642 \text{ in}^2.$$
 (area of transformed section.)

$$I = 9710064 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 13.47 \text{ in.}$$
 (ACI 318-14 18.10.6.4)

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.}$$
 (ACI 318-14 18.10.6.2 & 18.10.6.5)

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$



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Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

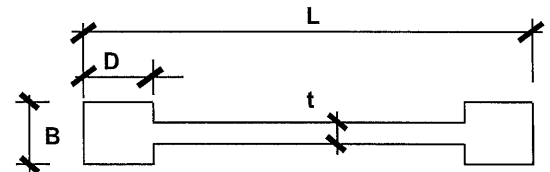
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 175$ k
FACTORED MOMENT LOAD $M_u = 612$ ft-k
FACTORED SHEAR LOAD $V_u = 51$ k
LENGTH OF SHEAR WALL $L = 12$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 12$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 6 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 12.00$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 12.00$ in
BULB REINFORCING 6 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

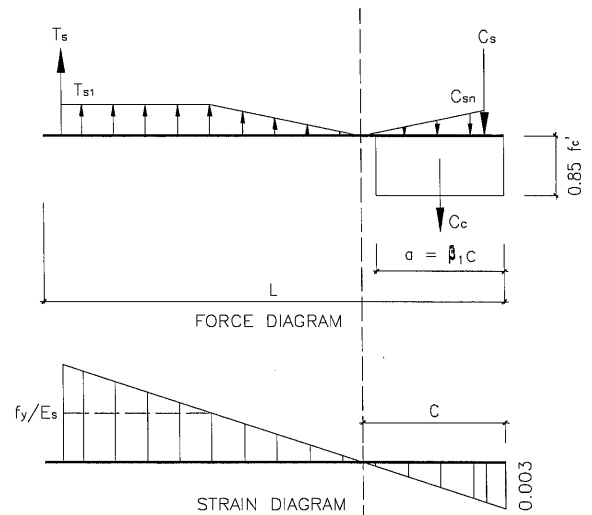
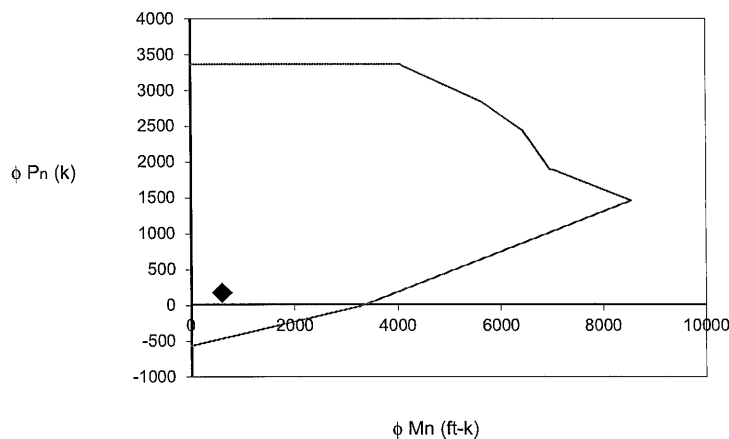


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 175 \text{ k} < 0.35 A_g f'_c = 2419 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 1728 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0020 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 109.29 \text{ kips} > V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0012 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 109.29 \text{ kips} > V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{prov.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{prov.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

$$\text{where } A_{cv} = 1728 \text{ in}^2 \quad (\text{gross area of concrete section bounded by web thickness and length in the shear direction})$$

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 464.56 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.60 \quad (\text{conservatively, ACI 318-14 21.2})$$

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 1.17 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 3365.3 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.65 \quad (\text{ACI 318-14 21.2})$$

$$A_{st} = 10.54 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 4049 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE

STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 82 \text{ in} \quad a = C_b \beta_1 = 69 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 138 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 1890 kips AND 7031 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 3310 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -569 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 3365	0
AT MAXIMUM LOAD	= 3365	4049
AT 0 % TENSION	= 3352	4091
AT 25 % TENSION	= 2843	5644
AT 50 % TENSION	= 2451	6413
AT $\epsilon_t = 0.002$	= 1906	6953
AT BALANCED CONDITION	= 1890	7031
AT $\epsilon_t = 0.005$	= 1467	8550
AT FLEXURE ONLY	= 0	3310
AT TENSION ONLY	= -569	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 4171 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900 \quad (\text{ACI 318-14 21.2})$$

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

$$\text{where } c = 18 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_n \text{ loads.)}$$

$$\delta_u = 0.8 \text{ in. (design displacement, assume } 0.005 h_w \text{ as a conservative short cut, see ACI 318-14 18.10.6.2.)}$$

$$f_c = (P_u / A) + (M_u y / I) = 0.265 \text{ ksi. (the maximum extreme fiber compressive stress at } P_u \text{ \& } M_u \text{ loads.)}$$

$$y = 72 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_u \text{ loads.)}$$

$$A = 1813 \text{ in}^2. \text{ (area of transformed section.)}$$

$$I = 3132497 \text{ in}^4. \text{ (moment of inertia of transformed section.)}$$

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.013 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 8.78 \text{ in. (ACI 318-14 18.10.6.4)}$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c. (ACI 318-14 18.10.6.2 \& 18.10.6.5)}$$

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$

$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$

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Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

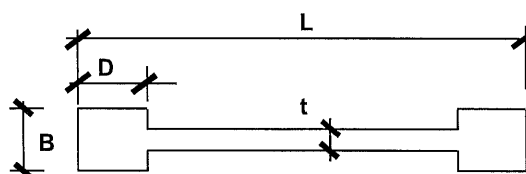
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 393.8$ k
FACTORED MOMENT LOAD $M_u = 7263$ ft-k
FACTORED SHEAR LOAD $V_u = 269$ k
LENGTH OF SHEAR WALL $L = 27$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 24$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 14$ ft
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 27.00$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D = 24.00$ in
BULB REINFORCING 10 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
HOOP REINF - LENGTH DIR. 3 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

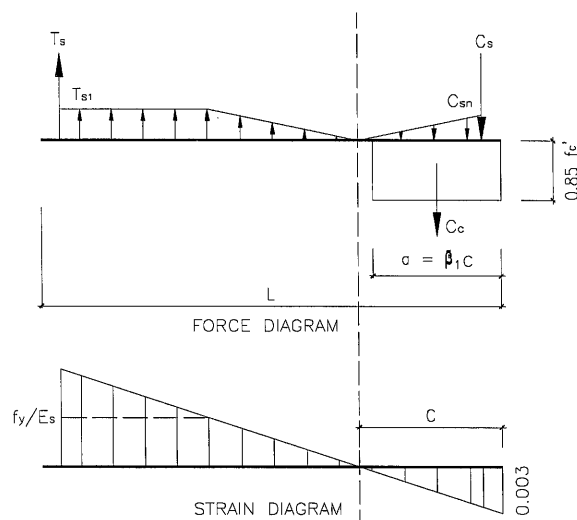
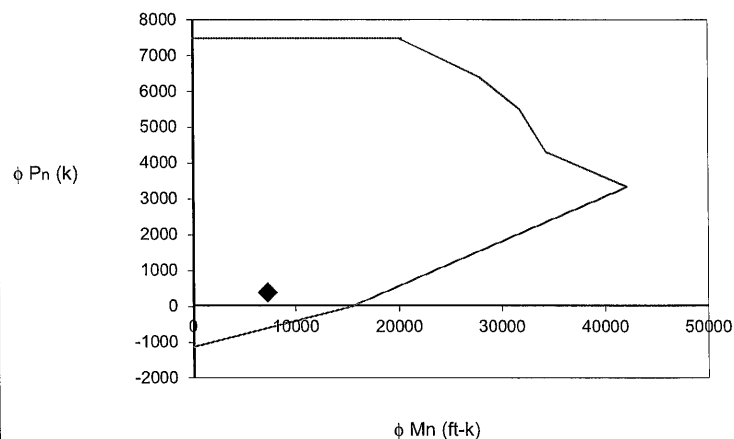


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 393.8 \text{ k} < 0.35 A_g f'_c = 5443 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 3888 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 245.90 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 245.90 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{provd.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{provd.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

where $A_{cv} = 3888 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 1045.26 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.60 \quad (\text{conservatively, ACI 318-14 21.2})$$

$$\alpha_c = 3.0 \quad (\text{for } h_w / L = 0.52 < 1.5)$$

$$\rho_l > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 7494.4 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)

$$A_{st} = 21.08 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 20332 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 185 \text{ in} \quad a = C_b \beta_1 = 157 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 312 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 4277 kips AND 34692 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 15370 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} f_y, 3.3 f'_c {}^{0.5} 4 L t) = -1138 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 7494	0
AT MAXIMUM LOAC	= 7494	20332
AT 0 % TENSION	= 7494	19977
AT 25 % TENSION	= 6398	27859
AT 50 % TENSION	= 5523	31714
AT $\epsilon_t = 0.002$	= 4312	34317
AT BALANCED CONDITION	= 4277	34692
AT $\epsilon_t = 0.005$	= 3334	42165
AT FLEXURE ONLY	= 0	15370
AT TENSION ONLY	= -1138	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 19746 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

where $c = 39 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

$$\delta_u = 0.8 \text{ in.}$$
 (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$$f_c = (P_u / A) + (M_u y / I) = 0.495 \text{ ksi.}$$
 (the maximum extreme fiber compressive stress at P_u & M_u loads.)

$$y = 162 \text{ in.}$$
 (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

$$A = 4058 \text{ in}^2.$$
 (area of transformed section.)

$$I = 35495673 \text{ in}^4.$$
 (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.011 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 19.49 \text{ in.}$$
 (ACI 318-14 18.10.6.4)

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c.}$$
 (ACI 318-14 18.10.6.2 & 18.10.6.5)

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A} \text{ in}^2.$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

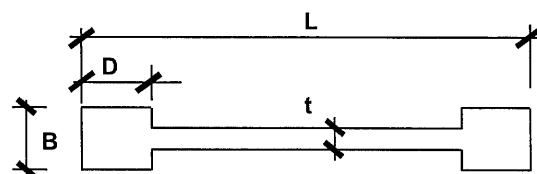
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
REBAR YIELD STRESS $f_y = 60$ ksi
FACTORED AXIAL LOAD $P_u = 1038$ k
FACTORED MOMENT LOAD $M_u = 1500$ ft-k
FACTORED SHEAR LOAD $V_u = 200$ k
LENGTH OF SHEAR WALL $L = 8$ ft
THICKNESS OF WALL $t = 12$ in
DEPTH AT FLANGE $D = 24$ in
WIDTH AT FLANGE $B = 12$ in
TOTAL WALL HEIGHT TO TOP $h_w = 19$ ft
REINF. BARS AT BULB 8 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 4 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 8.00$ ft
SHEAR WALL THICKNESS $t = 12.00$ in
BULB END WIDTH $B = 12.00$ in
BULB END DEPTH $D + 9.48 = 33.48$ in
BULB REINFORCING 8 # 5
WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
WALL VERT. REINF 2 # 5 @ 12 in o.c.
HOOP REINF - WIDTH, B, DIR 2 # 4 @ 3 in o.c.
HOOP REINF - LENGTH DIR. 4 # 4 @ 3 in o.c.

THE WALL DESIGN IS ADEQUATE

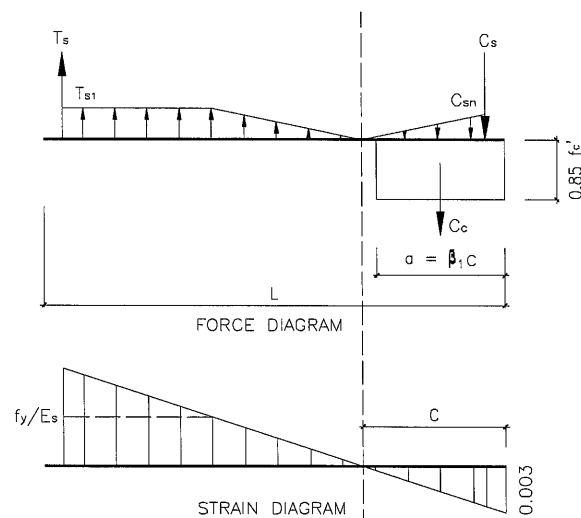
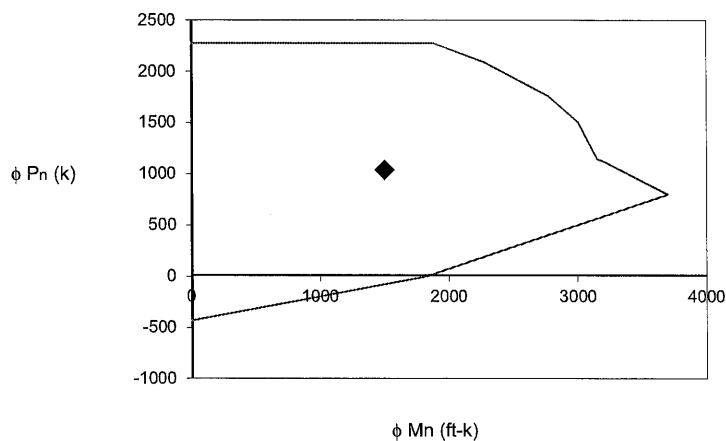


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 1038 \text{ k} < 0.35 A_g f'_c = 1613 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 1152 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 72.86 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 72.86 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{provd.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{provd.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

where $A_{cv} = 1152 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u > 2 A_{cv} (f'_c)^{0.5}$, two curtains reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 265.99 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$$\alpha_c = 2.0 \quad (\text{for } h_w / L = 2.38 > 2)$$

$$\rho_t > \rho_t \quad [\text{Satisfactory}] \quad (\text{only for } h_w / L > 2.0, \text{ ACI 318-14 18.10.4.3})$$

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 2274 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

where $\phi = 0.65$ (ACI 318-14 21.2)
 $A_{st} = 8.06 \text{ in}^2$

2017-06
(84.2)

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 1888 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 50 \text{ in} \quad a = C_b \beta_1 = 42 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 84 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 1129 kips AND 3185 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 1825 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c{}^{0.5} 4 L t) = -435 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 2274	0
AT MAXIMUM LOAD	= 2274	1888
AT 0 % TENSION	= 2094	2267
AT 25 % TENSION	= 1764	2770
AT 50 % TENSION	= 1507	3004
AT $\epsilon_t = 0.002$	= 1142	3153
AT BALANCED CONDITION	= 1129	3185
AT $\epsilon_t = 0.005$	= 801	3700
AT FLEXURE ONLY	= 0	1825
AT TENSION ONLY	= -435	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 3423 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

where $\phi = \text{Min}\{0.9, \text{Max}\{0.65 + (\epsilon_t - 0.002)(250/3), 0.65\}\} = 0.726$ (ACI 318-14 21.2)

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Unsatisfactory}]$$

where $c = 43 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)
 $\delta_u = 1.1 \text{ in.}$ (design displacement, assume $0.005 h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)
 $f_c = (P_u / A) + (M_u y / I) = 1.778 \text{ ksi.}$ (the maximum extreme fiber compressive stress at P_u & M_u loads.)
 $y = 48 \text{ in.}$ (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)
 $A = 1217 \text{ in}^2$. (area of transformed section.)
 $I = 934531 \text{ in}^4$. (moment of inertia of transformed section.)

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.009 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 33.48 \text{ in.} \quad (\text{ACI 318-14 18.10.6.4})$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 3 \text{ in.o.c.} \quad (\text{ACI 318-14 18.10.6.2 \& 18.10.6.5})$$

$$\text{The required hoop reinforcement} \quad A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = 0.153 \text{ in}^2 < \#4 \text{ provided} \quad [\text{Satisfactory}]$$

$$(\text{ACI 318-14 18}) \quad A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = 0.189 \text{ in}^2 < \#4 \text{ provided} \quad [\text{Satisfactory}]$$

Special Reinforced Concrete Shear Wall Design Based on ACI 318-14 & 2016 CBC Chapter A

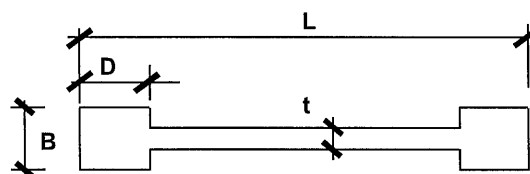
INPUT DATA

CONCRETE STRENGTH (ACI 318 19.2.1.1) $f'_c = 4$ ksi
 REBAR YIELD STRESS $f_y = 60$ ksi
 FACTORED AXIAL LOAD $P_u = 1202$ k
 FACTORED MOMENT LOAD $M_u = 1500$ ft-k
 FACTORED SHEAR LOAD $V_u = 200$ k
 LENGTH OF SHEAR WALL $L = 19$ ft
 THICKNESS OF WALL $t = 12$ in
 DEPTH AT FLANGE $D = 24$ in
 WIDTH AT FLANGE $B = 12$ in
 TOTAL WALL HEIGHT TO TOP $h_w = 19$ ft
 REINF. BARS AT BULB 8 # 5
 WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
 WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
 HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
 HOOP REINF - LENGTH DIR. 4 legs of # 4
 WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

DESIGN SUMMARY

SHEAR WALL LENGTH $L = 19.00$ ft
 SHEAR WALL THICKNESS $t = 12.00$ in
 BULB END WIDTH $B = 12.00$ in
 BULB END DEPTH $D + 6.02 = 30.02$ in
 BULB REINFORCING 8 # 5
 WALL HORIZ. REINF 2 # 5 @ 12 in o.c.
 WALL VERT. REINF 2 # 5 @ 12 in o.c.
 HOOP REINF - WIDTH, B, DIR 2 # 4 @ 8 in o.c.
 HOOP REINF - LENGTH DIR. 4 # 4 @ 8 in o.c.

THE WALL DESIGN IS ADEQUATE

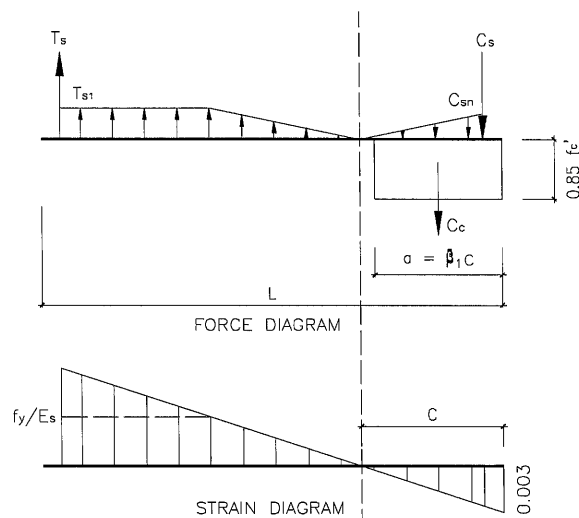
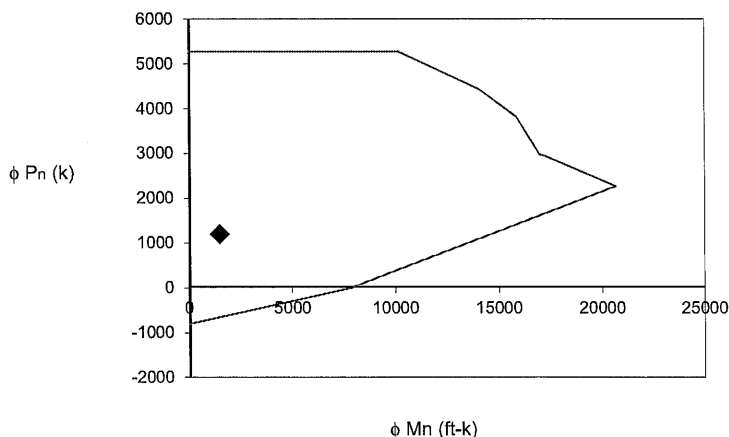


ANALYSIS

DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

$$P_u = 1202 \text{ k} < 0.35 A_g f'_c = 3830 \text{ k} \quad [\text{Satisfactory}]$$

$$\text{where } A_g = 2736 \text{ in}^2.$$



CHECK MINIMUM REINFORCEMENT RATIOS AND SPACING (ACI 318-14 18.10.2.1 & 11.6.1)

$$(\rho_t)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 173.04 \text{ kips} < V_u, \text{ and bar size \# 5 horizontal}]$$

$$(\rho_l)_{\min.} = 0.0025 \quad [\text{for } A_{cv} (f'_c)^{0.5} = 173.04 \text{ kips} < V_u, \text{ and bar size \# 5 vertical}]$$

$$(\rho_t)_{\text{prov.}} = 0.0043 > (\rho_t)_{\min.} \quad [\text{Satisfactory}]$$

$$(\rho_l)_{\text{prov.}} = 0.0043 > (\rho_l)_{\min.} \quad [\text{Satisfactory}]$$

where $A_{cv} = 2736 \text{ in}^2$ (gross area of concrete section bounded by web thickness and length in the shear direction)

The proposed spacing is less than the maximum permissible value of 18 in and is satisfactory. Since wall $V_u < 2 A_{cv} (f'_c)^{0.5}$, one curtain reinforcement required. (ACI 318-14 18.10.2.2)

CHECK SHEAR CAPACITY (ACI 318-14 18.10.4.1 & 18.10.4.4)

$$\phi V_n = \text{MIN} [\phi A_{cv} (\alpha_c (f'_c)^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f'_c)^{0.5}] = 735.55 \text{ kips} > V_u \quad [\text{Satisfactory}]$$

where $\phi = 0.60$ (conservatively, ACI 318-14 21.2)

$\alpha_c = 3.0$ (for $h_w / L = 1.00 < 1.5$)

$\rho_l > \rho_t$ [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

CHECK FLEXURAL & AXIAL CAPACITY

MAXIMUM DESIGN AXIAL LOAD STRENGTH (ACI 318-14 18 & 22.4)

$$\phi P_{\max} = 0.8 \phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}] = 5275.2 \text{ kips.} > P_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = 0.65 \quad (\text{ACI 318-14 21.2})$$

$$A_{st} = 14.88 \text{ in}^2.$$

DESIGN MOMENT CAPACITY AT MAXIMUM AXIAL LOAD STRENGTH ARE FROM 0 TO 10149 ft-kips.

FOR THE BALANCED STRAIN CONDITION UNDER COMBINED FLEXURE AND AXIAL LOAD, THE MAXIMUM STRAIN IN THE CONCRETE AND IN THE TENSION REINFORCEMENT MUST SIMULTANEOUSLY REACH THE VALUES SPECIFIED IN ACI 318-14 21.2.2

AS $\epsilon_c = 0.003$ AND $\epsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE

STRESS BLOCK ARE GIVEN BY

$$C_b = d \epsilon_c / (\epsilon_c + \epsilon_s) = 128 \text{ in} \quad a = C_b \beta_1 = 109 \text{ in} \quad \beta_1 = 0.85 \quad (\text{ACI 318-14 22.2.2.4.3})$$

$$\phi = 0.65 + (\epsilon_t - 0.002)(250/3) = 0.656 \quad (\text{ACI 318-14 21.2}) \quad d = (L - 0.5D) = 216 \text{ in}$$

DESIGN AXIAL AND MOMENT CAPACITIES AT THE BALANCED STRAIN CONDITION ARE 2955 kips AND 17168 ft-kips.

IN ACCORDANCE WITH ACI 318-14 21.2 THE DESIGN MOMENT CAPACITY WITHOUT AXIAL LOAD IS

$$\phi M_n = 0.9 M_n = 7815 \text{ kips.}$$

TO KEEP TENSION SECTION WITH SHEAR CAPACITY PER ACI 11.5.4.6, THE PURE AXIAL TENSION CAPACITY IS

$$-\phi P_n = -0.9 \text{ MIN}(A_{st} F_y, 3.3 f'_c A_g) = -804 \text{ kips.}$$

SUMMARY OF LOAD VERSUS MOMENT CAPACITIES ARE SHOWN IN THE TABLE BELOW, AND THEY ARE PLOTTED ON THE INTERACTION DIAGRAM AT FRONT PAGE.

	ϕP_n (kips)	ϕM_n (ft-kips)
AT AXIAL LOAD ONLY	= 5275	0
AT MAXIMUM LOAD	= 5275	10149
AT 0 % TENSION	= 5233	10373
AT 25 % TENSION	= 4440	14053
AT 50 % TENSION	= 3829	15832
AT $\epsilon_t = 0.002$	= 2980	16988
AT BALANCED CONDITION	= 2955	17168
AT $\epsilon_t = 0.005$	= 2276	20665
AT FLEXURE ONLY	= 0	7815
AT TENSION ONLY	= -804	0

DESIGN FORCES P_u & M_u ARE ALSO PLOTTED ON THE INTERACTION DIAGRAM. FROM THE INTERACTION DIAGRAM.THE ALLOWABLE MOMENT AT AN AXIAL LOAD P_u IS GIVEN BY

$$\phi M_n = 15902 \text{ kips.} > M_u \quad [\text{Satisfactory}]$$

$$\text{where } \phi = \text{Min}\{0.9, \text{Max}[0.65 + (\epsilon_t - 0.002)(250/3), 0.65]\} = 0.900 \quad (\text{ACI 318-14 21.2})$$

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI 318-14 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$$c < (L h_w) / 1.5(600 \delta_u) \text{ for ACI 21.9.6.2 apply} \quad \text{or} \quad f_c < 0.2 f'_c \text{ for ACI 18.10.6.3 apply} \quad [\text{Satisfactory}]$$

$$\text{where } c = 53 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_n \text{ loads.)}$$

$$\delta_u = 1.1 \text{ in. (design displacement, assume } 0.005 h_w \text{ as a conservative short cut, see ACI 318-14 18.10.6.2.)}$$

$$\xi = (P_u / A) + (M_u y / I) = 0.587 \text{ ksi. (the maximum extreme fiber compressive stress at } P_u \text{ \& } M_u \text{ loads.)}$$

$$y = 114 \text{ in. (distance from the extreme compression fiber to neutral axis at } P_u \text{ \& } M_u \text{ loads.)}$$

$$A = 2856 \text{ in}^2. \text{ (area of transformed section.)}$$

$$I = 12370895 \text{ in}^4. \text{ (moment of inertia of transformed section.)}$$

$$\text{Or the longitudinal reinforcement ratio at the wall end} = 0.009 > 400 / f_y \quad [\text{Unsatisfactory}]$$

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

$$\text{The boundary element length} = \text{MAX}(c - 0.1L, 0.5c) = 30.02 \text{ in. (ACI 318-14 18.10.6.4)}$$

$$\text{The maximum hoop spacing} = \text{MIN}[B/4, 6d_b, 6, 4 + (14 - h_x)/3] = 8 \text{ in.o.c. (ACI 318-14 18.10.6.2 \& 18.10.6.5)}$$

$$A_{sh, BDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$

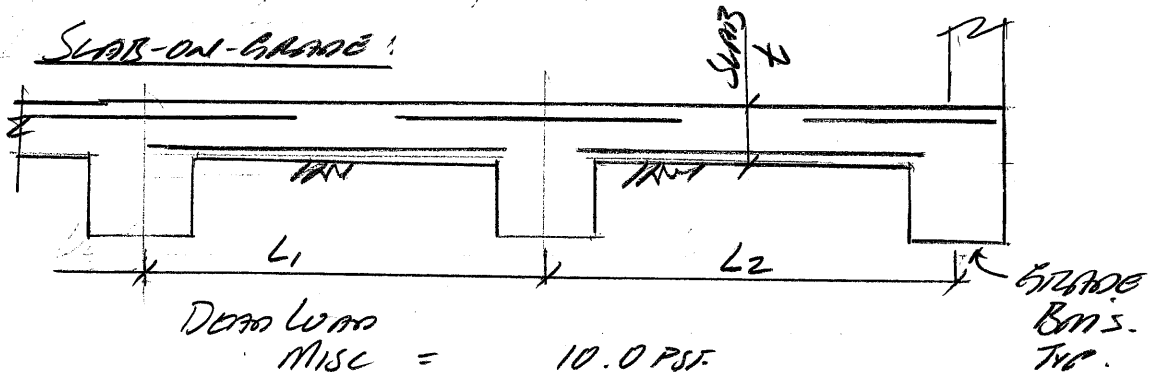
$$A_{sh, LDIR} = (0.09 s h_c f'_c) / f_{yh} = \text{N/A in}^2.$$

FOUNDATION DESIGN

SOILS REPORT : GEO ENGINEERING CONSULTANTS
4125 BRADFORD AVE #145
ARADORA, CA 95717
925-321-5550

PROJ : P16-122
DATE : 2/12/16

* STRUCTURE SUPPORTED ON DISPLACEMENT PILES -
DESIGNED BY OTHERS.



DEAD LOAD
MISC = 10.0 PSF
PARTITION = 15.0
25.0 PSF

LIVE LOAD = 100 PSF

ONE WAY SLAB w/ BEAMS

$$L_{min} = 27 - 2 = 25'$$

$$d_{min} = \frac{25(12)}{24} = 12.5"$$

USE 12 1/2" MIN. THK

$$W_u = \left[\frac{12.5}{12} (150) + 25 \right] (1.2) + 1.6 (100) = 376 \text{ PSF}$$

$$M_u = \frac{376(25)^2}{16} = 23500 \text{ #-1} = 23.5 \text{ #-1}$$

$$b = 12" \quad d = 12.5 - 1 = 11.5"$$

$$A_{sR} = 0.50 \text{ in}^2 \quad \#6 @ 10" \text{ O.C. TOP. R/F}$$

O/ EA BEAM

2017-06
(87)

$$M_u^+ = \frac{376(25)^2}{12} = 19882 \text{ ft-lb} = 19.6 \text{ k-ft}$$

$$b = 12" \quad d = 12.5 - 3.5 = 9"$$

$$A_s = 0.54 \text{ in}^2 - \underline{\#5 @ 6\frac{1}{2}" o.c. \text{ BOT. R/F E.M.}}$$

Grade Beams - GBI

$$b = 24" \quad h = 30" \quad d = 26"$$

$$W_u = 376(25) = 9400 \text{ lb}$$

$$M_u^- = \frac{9.4(25)^2}{10} = 588 \text{ k-ft} \quad A_s = 5.7 \text{ in}^2 - \underline{8-\#8 \text{ TOP R/F}}$$

$$M_u^+ = \frac{9.4(25)^2}{12} = 485 \text{ k-ft} \quad A_s = 4.7 \text{ in}^2 - \underline{6-\#8 \text{ BOT. R/F}}$$

$$V_u = \frac{9.4(25)}{2} - 9.4(2.5) = 98.7 \text{ k}$$

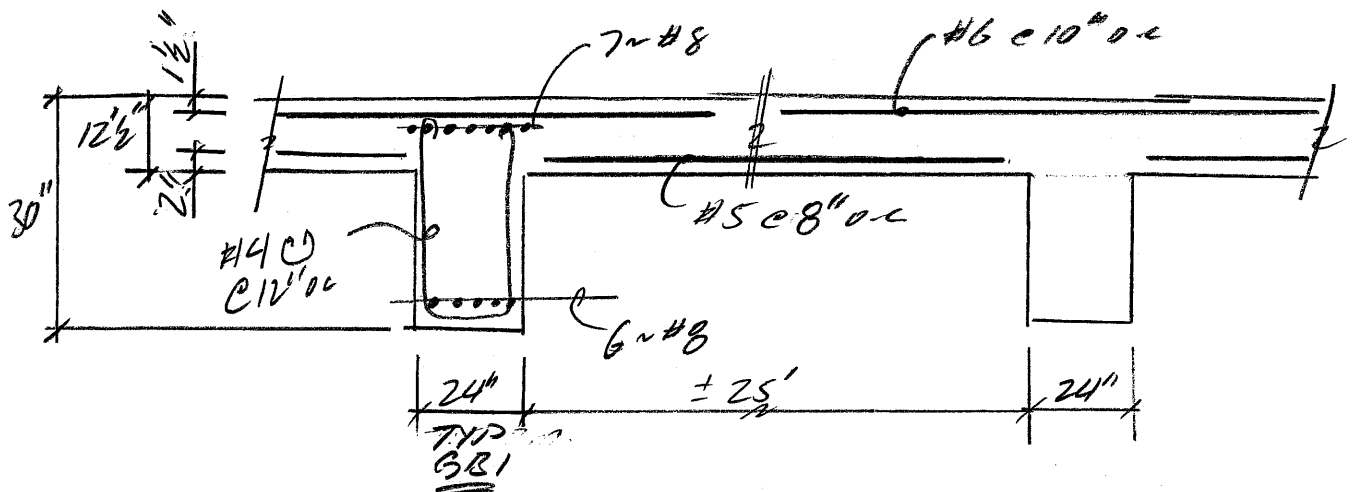
$$\phi V_c = 78 \text{ k}$$

$$\frac{\phi V_c}{2} = 39 \text{ k} < 98.7 \text{ k} \therefore \underline{\text{SHOR. R/F REQ'D}}$$

$$\#4 @ 12" o.c.$$

$$\phi V_s = 46 \text{ k}$$

$$\phi V_c + \phi V_s = 124 \text{ k} > 98.7 \text{ k} \therefore \underline{\#4 @ 12" o.c.}$$



GRADE BEAMS AT WAUS. - G82

USE 30" W x 24" TH.

PILE SPACING 8'-0" O.C. MIN.
10'-0" O.C. MAX ← ROW'S

GRADE BEAMS

LINE B - WAUS7 CASE

$$\begin{aligned} W_u (\text{SLAB}) &= 0.376(12.5) = 4.7 \text{ k/ft} \\ W_{\text{WALL B}} &= 28.7 \\ W_u (\text{FTG}) &= 1.2(750) = 0.9 \\ W_u &= 34.3 \text{ k-1} \end{aligned}$$

PILE DIAMETER = 18" MIN.

$$\therefore L_u = 10' - 1.5' = 8.5'$$

$$M_u^- = \frac{34.3(8.5)^2}{10} = 248 \text{ k-1} \quad A_{s_u} = 2.4 \text{ in}^2 \quad \underline{\text{5-}\#8 \text{ TOP}}$$

$$M_u^+ = \frac{34.3(8.5)^2}{12} = 208 \text{ k-1} \quad A_{s_u} = 2.0 \text{ in}^2 < A_{s_{\text{MIN}}} = 2.1 \text{ in}^2$$

USE 5-#8 BOTTOM

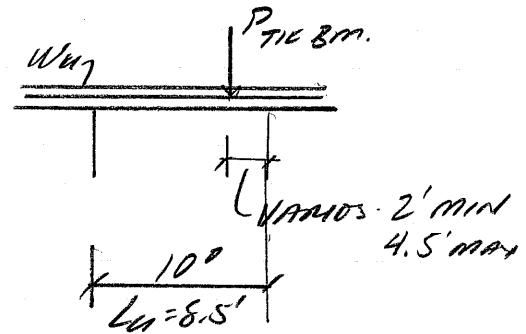
$$V_u = \frac{34.3(8.5)}{2} - 34.3(2) = 128.6 \text{ k}$$

$$\#4 \text{ } \square \text{ } @ 12" \text{ O.C. } \phi V_s = 71 \text{ k}$$

$$\phi V_c + \phi V_s = 61 + 71 = 132 \text{ k} > 128.6 \text{ k} \quad \therefore \underline{\#4 \text{ } \square \text{ } @ 12" \text{ O.C.}}$$

2017-06
(85)

Grade Beam @ Tie Beam.



$$W_u W_{DM} = 34.3 \text{ k/ft}$$

$$P_{u \text{ TIE BM}} = 9.4(12.5) = 117.5 \text{ k}$$

$$M_u^- = \frac{34.3(8.5)^2}{10} + \frac{117.5(8.5)}{8} = 375 \text{ k-ft} - A_{sT} = 3.47 \text{ in}^2$$

5 #8 TOP

$$M_u^+ = \frac{34.3(6.5)^2}{12} + \frac{117.5(6.5)}{8} = 331 \text{ k-ft} - A_{sB} = 3.07 \text{ in}^2$$

5 #8 BOTTOM

$$V_u \text{ max} = \frac{34.3(8.5)}{2} + 117.5 = 264 \text{ k} < \phi V_u \text{ S}$$

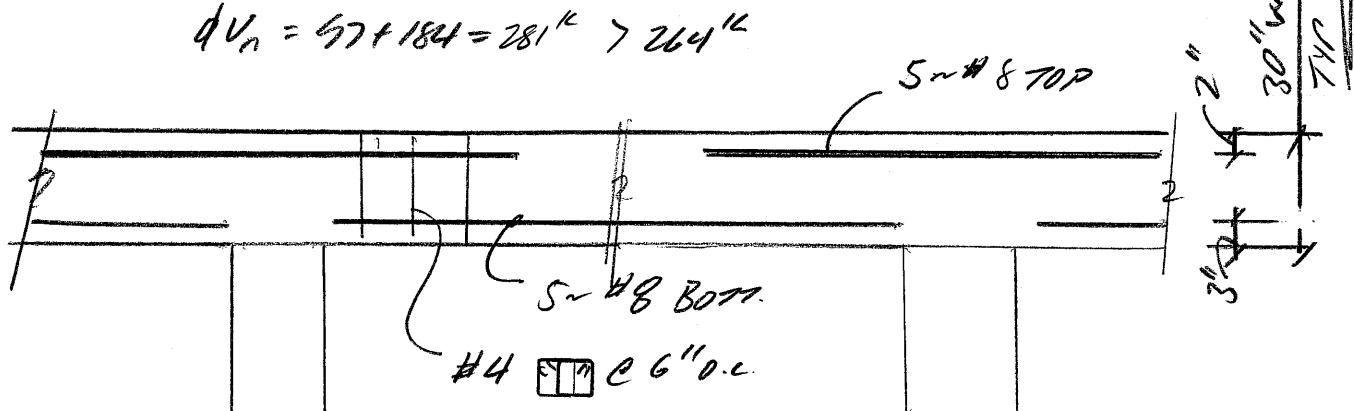
$$\text{Use } b = 30 \text{ in min } h = 30 \text{ in } d = 21 \text{ in}$$

$$\phi V_c = 97 \text{ k}$$

$$\#4 \text{ @ } 6 \text{ in o.c.}$$

$$\phi V_s = 184 \text{ k}$$

$$\phi V_u = 97 + 184 = 281 \text{ k} > 264 \text{ k}$$



1ST FLOOR SLAB & FOUNDATION

2012.06
 (90)

TYP Slab
 or grade

26'

TYP T16 Bm

25'

26'

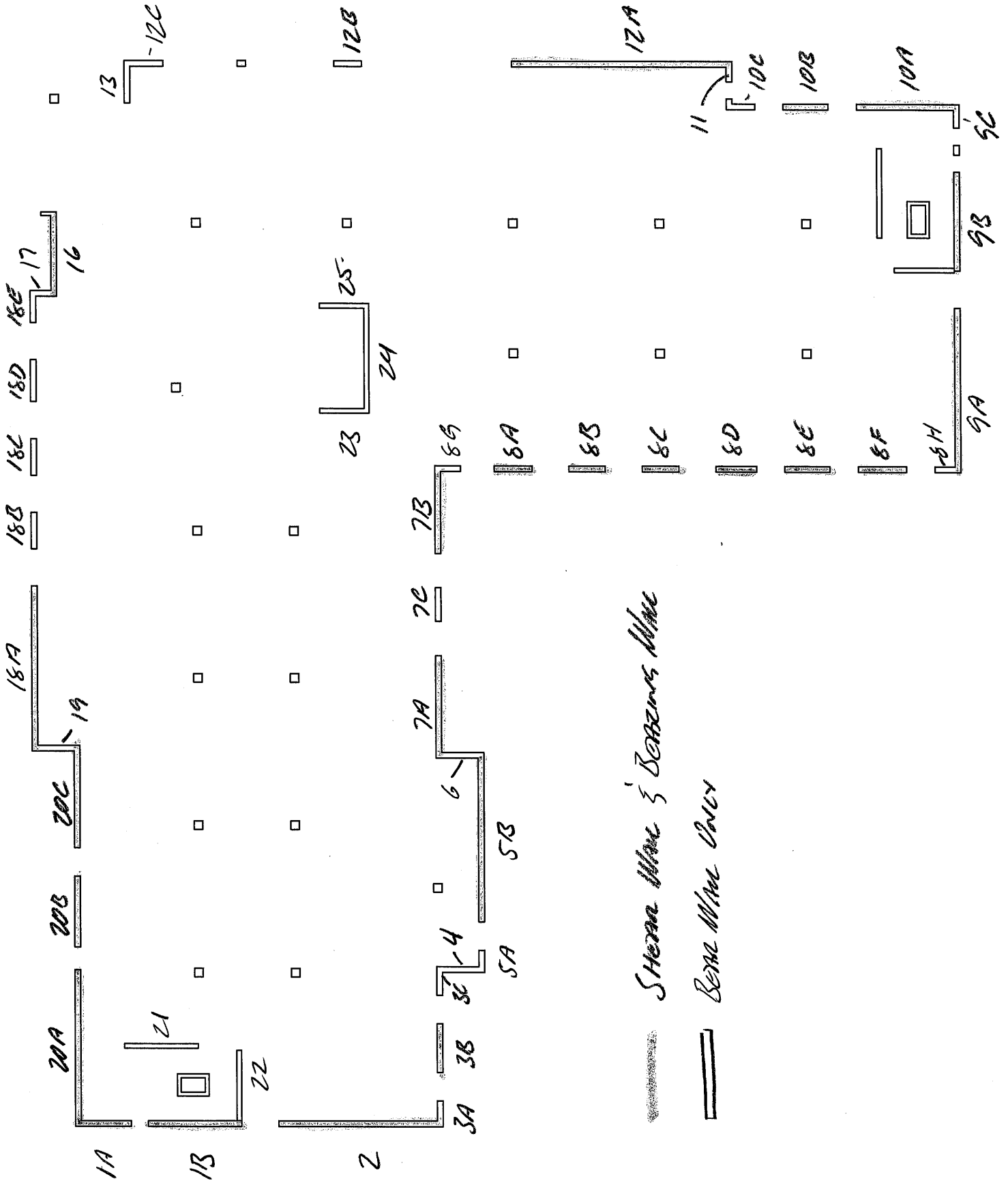
24'
 MIN
 5-#
 TYP.

Piles By Others

FOUNDATION PLAN
 1/2" = 1'-0"

1

Wm Plan



Shenan Wm & Benzene Wm

Bene Wm Only

WAM Loans

W0057
CASE
S.B.

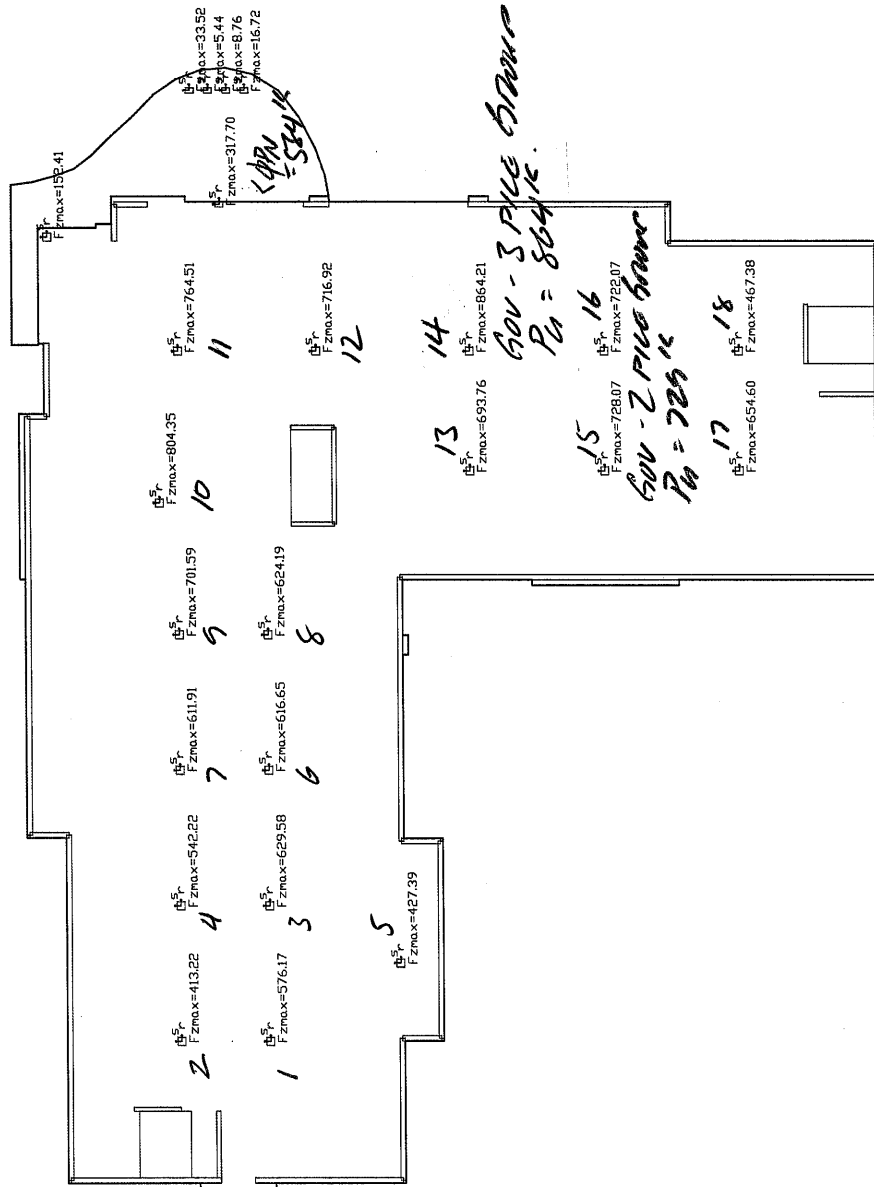
-PILE-
-PILE-
-PILE-
-PILE-

2017.06
(52)

-PILE-

SHEAR WALL SUMMARY										BEARING WALLS									
WALL	LENGTH	HEIGHT	THICK	Pu (DEAD LOAD)				Vu				Service Level Loads				Strength Level			
				k	k/ft	Mu	E	k	k/ft	Mu	E	Dead	Live	k	k/ft	1.2D + 1.6L	wu (k/ft)	1.2D + 1.6L	wu (k/ft)
1A	9.5	14	12	74.2	7.8	42.0	4.42	588.0	61.9			47.8	5.0	28.8	3.0	10.9			
1B	16.5	14	12	128.8	7.8	164.0	9.94	2,296.0	139.2			83.1	5.0	50.1	3.0	10.9			
2	29	14	12	311.0	10.7	595.0	20.52	8,330.0	287.2			221.87	7.7	67.1	2.3	12.9			
3A	4.5											64.6	14.4	16.6	3.7	23.1			
3B	8.5	14	12	361.0	42.5	19.0	2.24	266.0	31.3			122	14.4	31.4	3.7	23.1			
3C	5											71.8	14.4	18.5	3.7	23.2			
4	8.5											76.66	9.0	24.6	2.9	15.5			
5A	3.8											17.4	4.6	0.5	0.1	5.7			
5B	29.7	14	12	215.0	7.2	382.0	12.86	5,348.0	180.1			136	4.6	3.3	0.1	5.7			
6	8.5											80.5	9.5	27.6	3.2	16.6			
7A	17.5	14	12	468.5	26.8	125.0	7.14	1,750.0	100.0			282.1	16.1	92.8	5.3	27.8			
7B	15	14	12	401.5	26.8	86.0	5.73	1,204.0	80.3			241.65	16.1	79.5	5.3	27.8			
7C	6											96.7	16.1	31.8	5.3	27.8			
8A	6	14	12	179.3	29.9	11.0	1.83	154.0	25.7			104.5	17.4	29.3	4.9	28.7			
8B	6	14	12	179.3	29.9	11.0	1.83	154.0	25.7			104.5	17.4	29.3	4.9	28.7			
8C	6	14	12	179.3	29.9	11.0	1.83	154.0	25.7			104.5	17.4	29.3	4.9	28.7			
8D	7	14	12	209.1	29.9	18.0	2.57	252.0	36.0			121.94	17.4	34.1	4.9	28.7			
8E	7.5	14	12	224.1	29.9	22.0	2.93	308.0	41.1			130.65	17.4	36.6	4.9	28.7			
8F	7.5	14	12	224.1	29.9	31.0	4.13	434.0	57.9			130.65	17.4	36.6	4.9	28.7			
8G	4.5											78.4	17.4	21.9	4.9	28.7			
8H	4.5											78.4	17.4	21.9	4.9	28.7			
9A	28.8	14	12	345.9	12.0	360.0	12.50	5,040.0	175.0			227.9	7.9	83.5	2.9	14.1			
9B	17	14	12	204.1	12.0	117.0	6.88	1,638.0	96.4			134.3	7.9	49.3	2.9	14.1			
9C	4											31.6	7.9	11.6	2.9	14.1			
10A	18	14	12	324.7	18.0	237.0	13.17	3,318.0	184.3			193.8	10.8	47.9	2.7	17.2			
10B	7.5	14	12	135.3	18.0	24.0	3.20	336.0	44.8			80.78	10.8	19.9	2.7	17.2			
10C	5											53.85	10.8	13.3	2.7	17.2			
11	3.58											89.6	25.0	27.8	7.8	42.5			
12A	38.75	14	12	693.0	17.9	1,080.0	27.87	15,120.0	390.2			494.92	12.8	118.2	3.1	20.2			
12B	5											287.1	57.4	80.15	16.0	94.6			
12C	6.75											183.54	27.2	70.8	10.5	49.4			
13	7.5											114.94	15.3	19.3	2.6	22.5			
16	14.5	14	12	147.0	10.1	84.0	5.79	1,176.0	81.1			297.85	20.5	104.8	7.2	36.2			
17	4.8											51.2	10.7	20	4.2	19.5			
18A	28.5	14	12	1,064.0	37.3	376.0	13.19	5,264.0	184.7			397.75	14.0	91.3	3.2	21.9			
18B	6.5											90.7	14.0	20.8	3.2	21.9			
18C	6.5											90.7	14.0	20.8	3.2	21.9			
18D	7.4											103.3	14.0	23.7	3.2	21.9			
18E	5.6											78.17	14.0	17.9	3.2	21.9			
19	8.5											94.3	11.1	29.6	3.5	18.9			
20A	17.5	14	12	255.2	14.6	336.0	19.20	4,704.0	268.8			182.2	10.4	44	2.5	16.5			
20B	12	14	12	175.0	14.6	51.0	4.25	714.0	59.5			124.9	10.4	30	2.5	16.5			
20C	27	14	12	393.8	14.6	269.0	9.96	3,766.0	139.5			280.8	10.4	68	2.5	16.5			
21	13											142.4	11.0	82.3	6.3	23.3			
22	12.5											191.2	15.3	82.6	6.6	28.9			
23	8.7											216	24.8	190	21.8	64.7			
24	19.3											275	14.2	222	11.5	35.5			
25	8.7											174	20.0	190	21.8	58.9			

93



Column Reactions Plan_Envelope COLUMN REACTIONS

HOME2 - SSF COLUMN REACTIONS

05/28/17

11:37:11

h2ssf-Adapt-170526-1.adm

PILE CAPS

LOADS FROM TIE BEAMS

DL

$$\begin{aligned} \text{SLAB} &= 156 \times 25 = 3900 \text{ #/ft} \\ \text{TIE BM} &= 150(2.5)(2) = 750 \text{ #/ft} \end{aligned}$$

$$W_{DL} = \underline{4650 \text{ #/ft}}$$

LL $\text{SLAB} = 100 \times 25 = 2500 \text{ #/ft}$

$$W_u = 1.2(4650) + 1.6(2500) = 9580 \text{ #/ft}$$

ADD'L LOAD
PER PILE GROUP $= \phi P_u = 9580(25) = 239500 \text{ #} \sim 240 \text{ K}$

PILE CAP LOADS

2-PILE GROUP

$$\phi P_u = 729 + 240 = 969 \text{ K}$$

3-PILE GROUP

$$\phi P_u = 864 + 240 = 1104 \text{ K}$$



2017-06
(95)

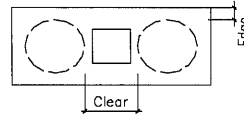
Pile Cap Design for 2-Piles Pattern Based on ACI 318-14

DESIGN CRITERIA

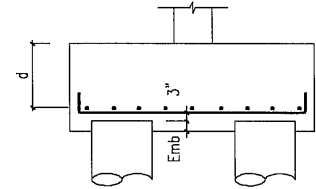
- FROM PILE DESIGN & SOIL REPORT, DETERMINE SINGLE PILE MAX LOADS OR CAPACITY AT CAP BOTTOM FACE, ϕP_n , ϕM_n , & ϕV_n .
- THE MAXIMUM COLUMN CAPACITY AT COLUMN BASE, $\phi P_{n,col}$, $\phi M_{n,col}$, $\phi V_{n,col}$, MAY BE BASED ON PILE CAP BALANCED LOADS. USER CAN CHANGE THE YELLOW CELLS TO MODIFY THEM FOR DIFFERENT CASES.
- PILE CAPS SHALL BE INTERCONNECTED BY TIES WITH $\text{Min}(0.25, S_{DS}/10)$ TIMES AXIAL VERT COLUMN LOADING. (IBC 1810.3.13)

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f'_c	=	5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	16	in
COLUMN SIZE (SHORT SIDE)	C	=	18	in
SINGLE PILE MAX LOADS OR CAPACITY	ϕP_n	=	325.5	k
(at the section of cap bottom face)	ϕM_n	=	0	ft-k
	ϕV_n	=	15	k
PILE CLEAR DIST. (24" min, 2D reqd)	Clear	=	32	in
EDGE DISTANCE (9" min)	Edge	=	16	in
EFFECTIVE DEPTH	d	=	40	in
CAP BOTTOM REINFORCING	#	8	@	12 in o.c., each way



PILE PATTERN



SECTION

The Column Can Support Max Loads:

$P_{u,col} \leq \phi P_{n,col}$	=	969	kips
$M_{u,col} \leq \phi M_{n,col}$	=	0.0	ft-kips
$V_{u,col} \leq \phi V_{n,col}$	=	0	kips

THE PILE CAP DESIGN IS ADEQUATE.

ANALYSIS

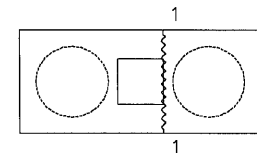
CHECK FLEXURE CAPACITY AT COLUMN FACE (ACI 318 21.22, & 24.4.3.1)

Pile Spacing =	48	in
Cap Edge Length =	96.0	in
L_{1-1} =	48.0	in, length of section 1-1
$M_{u,1-1}$ =	0.0	ft-kips / ft, (to middle of cap elevation)
ρ_{provD} =	0.0016	

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,max}}{0.383 b d^2 f'_c}} \right)}{f_y} = 0.0000 < \rho_{provD} \quad \text{[Satisfactory]}$$

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c \epsilon_u}{f_y \epsilon_u + \epsilon_t} = 0.0243 > \rho_{provD} \quad \text{[Satisfactory]}$$

$$\rho_{MIN} = \text{MIN} \left(0.0018 \frac{T}{d}, \frac{4}{3} \rho \right) = 0.0000 < \rho_{provD} \quad \text{[Satisfactory]}$$



FLEXURE CRITICAL SECTION

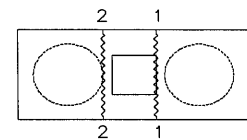
CHECK ONE WAY SHEAR CAPACITY AT THE FACE OF COLUMN & PILE

(ACI 318 22.5)

L_{2-2} =	48.0	in, length of section 2-2
$V_{u,1-1} = (\phi P_n) / L_{1-1}$	0.0	kips / ft (No shear at "d" offset.)
$V_{u,2-2} = (\phi P_n) / L_{2-2}$	0.0	kips / ft (No shear at "d" offset.)

$$\phi V_c = 2 \phi b d (f'_c)^{0.5} = 50.9 \text{ kips / ft} > V_{u,max} \quad \text{[Satisfactory]}$$

where $\phi = 0.75$ (ACI 318 21.2)



ONE WAY SHEAR CRITICAL SECTION

CHECK COLUMN PUNCHING CAPACITY (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$$\phi v_c (psi) = \phi (2 + y) \sqrt{f'_c} = 212 \text{ ksi} > v_u (psi) = \frac{P_{u,col}}{A_P} + \frac{0.5 \gamma_v M_{u,col} b_1}{J} = 132 \text{ ksi} \quad \text{[Satisfactory]}$$

where $\beta_c = 1.00$

$$b_1 = (C + d) = 58 \text{ in}$$

$$b_2 = [(C + \text{min}(\text{Edge}, d))] = 34 \text{ in}$$

$$b_0 = 2b_1 + 2b_2 = 184 \text{ in}$$

$$A_P = b_0 d = 7360 \text{ in}^2$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} = 0.4654$$

$$y = \text{MIN} \left(2, \frac{4}{\beta_c}, 40 \frac{d}{b_0} \right) = 2.0$$

$$J = \left(\frac{d b_1^3}{6} \right) \left[1 + \left(\frac{d}{b_1} \right)^2 + 3 \left(\frac{b_2}{b_1} \right) \right] = 203 \text{ ft}^4$$

CHECK SINGLE PILE PUNCHING CAPACITY (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$$\phi v_c(ksi) = \phi(2 + \gamma) \sqrt{f'_c} = 106 \text{ ksi} > v_u(ksi) = \frac{P_{u,col}}{A_p} + \frac{0.5 \gamma_v M_{u,col} b_1}{J} = 93 \text{ ksi} \quad \text{[Satisfactory]}$$

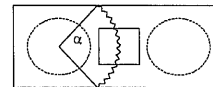
where

$$\begin{aligned} \alpha &= 180.0 \text{ deg} \\ b_1 &= (\alpha / 360) (D \pi / 4 + d) = 26 \text{ in} \\ b_2 &= (\alpha / 360) (D \pi / 4 + d) = 26 \text{ in} \\ b_0 &= (\alpha / 360) (D + d) \pi = 88 \text{ in, conservative value} \\ A_p &= b_0 d = 3519 \text{ in}^2 \end{aligned}$$

$$J = \left(\frac{d b_1^3}{6} \right) \left[1 + \left(\frac{d}{b_1} \right)^2 + 3 \left(\frac{b_2}{b_1} \right) \right] = 37 \text{ ft}^4$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} = 0.4$$

$$\gamma = 0, \text{ conservative value as one way shear}$$



PILE PUNCHING CRITICAL SECTION

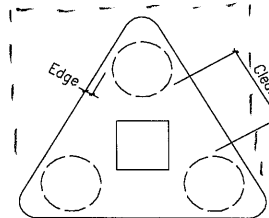
Pile Cap Design for 3-Piles Pattern Based on ACI 318-14

DESIGN CRITERIA

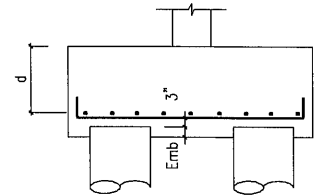
- FROM PILE DESIGN & SOIL REPORT, DETERMINE SINGLE PILE MAX LOADS OR CAPACITY AT CAP BOTTOM FACE, ϕP_n , ϕM_n , & ϕV_n .
- THE MAXIMUM COLUMN CAPACITY AT COLUMN BASE, $\phi P_{n,col}$, $\phi M_{n,col}$, $\phi V_{n,col}$, MAY BE BASED ON PILE CAP BALANCED LOADS. USER CAN CHANGE THE YELLOW CELLS TO MODIFY THEM FOR DIFFERENT CASES.
- PILE CAPS SHALL BE INTERCONNECTED BY TIES WITH $\text{Min}(0.25, S_{DS}/10)$ TIMES AXIAL VERT COLUMN LOADING. (IBC 1810.3.13)

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f'_c	=	5	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
PILE DIAMETER	D	=	16	in
COLUMN SIZE (SHORT SIDE)	C	=	18	in
SINGLE PILE MAX LOADS OR CAPACITY	ϕP_n	=	325.5	k
(at the section of cap bottom face)	ϕM_n	=	0	ft-k
	ϕV_n	=	15	k
PILE CLEAR DIST. (24" min, 2D reqd)	Clear	=	32	in
EDGE DISTANCE (9" min)	Edge	=	16	in
EFFECTIVE DEPTH	d	=	40	in
CAP BOTTOM REINFORCING	#	8	@	12 in o.c., each way



PILE PATTERN



SECTION

The Column Can Support Max Loads:

$$\begin{aligned} P_{u,col} &\leq \phi P_{n,col} = 1104 \text{ kips} \\ M_{u,col} &\leq \phi M_{n,col} = 0.0 \text{ ft-kips} \\ V_{u,col} &\leq \phi V_{n,col} = 0 \text{ kips} \end{aligned}$$

THE PILE CAP DESIGN IS ADEQUATE.

ANALYSIS

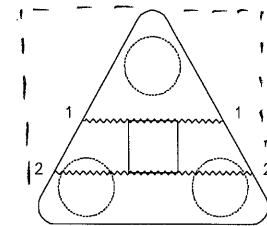
CHECK FLEXURE CAPACITY AT COLUMN FACE (ACI 318 21.22, & 24.4.3.1)

Pile Spacing =	48	in
Cap Edge Length =	131.1	in
L_{1-1} =	77.0	in, length of section 1-1
L_{2-2} =	97.8	in, length of section 2-2
$M_{u,1-1}$ =	83.3	ft-kips / ft, (to middle of cap elevation)
$M_{u,2-2}$ =	38.9	ft-kips / ft, (to middle of cap elevation)
ρ_{provD} =	0.0016	

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,max}}{0.383 b d^2 f'_c}} \right)}{f_y} = 0.0010 < \rho_{provD} \quad \text{[Satisfactory]}$$

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c \epsilon_u}{f_y \epsilon_u + \epsilon_t} = 0.0243 > \rho_{provD} \quad \text{[Satisfactory]}$$

$$\rho_{MIN} = \text{MIN} \left(0.0018 \frac{T}{d}, \frac{4}{3} \rho \right) = 0.0013 < \rho_{provD} \quad \text{[Satisfactory]}$$



FLEXURE CRITICAL SECTION

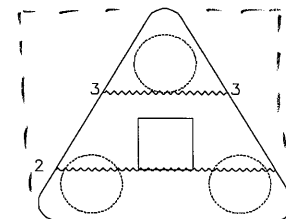
CHECK ONE WAY SHEAR CAPACITY AT THE FACE OF COLUMN & PILE

(ACI 318 22.5)

L_{3-3} =	64.7	in, length of section 3-3
$V_{u,2-2} = 2 (\phi P_n) / L_{2-2}$ =	0.0	kips / ft (No shear at "d" offset.)
$V_{u,3-3} = (\phi P_n) / L_{3-3}$ =	0.0	kips / ft (No shear at "d" offset.)

$$\phi V_c = 2 \phi b d (f'_c)^{0.5} = 50.9 \text{ kips / ft} > V_{u,max} \quad \text{[Satisfactory]}$$

where $\phi = 0.75$ (ACI 318 21.2)



ONE WAY SHEAR CRITICAL SECTION

CHECK COLUMN PUNCHING CAPACITY (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$$\phi v_c(psi) = \phi (2 + \gamma) \sqrt{f'_c} = 212 \text{ ksi} > v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5 \gamma_v M_{u,col} b_1}{J} = 119 \text{ ksi} \quad \text{[Satisfactory]}$$

where	β_c	=	1.00
	b_1	=	(C + d) = 58 in
	b_2	=	(C + d) = 58 in
	b_0	=	$2b_1 + 2b_2 = 232$ in
	A_P	=	$b_0 d = 9280$ in ²

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} = 0.4 \quad \gamma = \text{MIN} \left(2, \frac{4}{\beta_c}, 40 \frac{d}{b_0} \right) = 2.0$$

$$J = \left(\frac{d b_1^3}{6} \right) \left[1 + \left(\frac{d}{b_1} \right)^2 + 3 \left(\frac{b_2}{b_1} \right) \right] = 281 \text{ ft}^4$$

CHECK SINGLE PILE PUNCHING CAPACITY (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$$\phi v_c(ksi) = \phi(2 + y)\sqrt{f'_c} = 106 \text{ ksi} > v_u(ksi) = \frac{P_{u,col}}{A_p} + \frac{0.5\gamma_v M_{u,col} b_1}{J} = 94 \text{ ksi} \quad \text{[Satisfactory]}$$

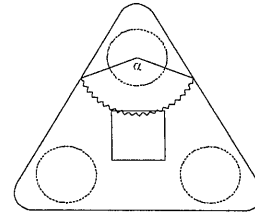
where

$$\begin{aligned} \alpha &= 178.0 \text{ deg} \\ b_1 &= (\alpha / 360) (D\pi / 4 + d) = 26 \text{ in} \\ b_2 &= (\alpha / 360) (D\pi / 4 + d) = 26 \text{ in} \\ b_0 &= (\alpha / 360) (D + d) \pi = 87 \text{ in, conservative value} \\ A_p &= b_0 d = 3479 \text{ in}^2 \end{aligned}$$

$$J = \left(\frac{db_1^3}{6} \right) \left[1 + \left(\frac{d}{b_1} \right)^2 + 3 \left(\frac{b_2}{b_1} \right) \right] = 36 \text{ ft}^4$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} = 0.4$$

$$y = 0, \text{ conservative value as one way shear}$$



PILE PUNCHING CRITICAL SECTION

2017-06
(99)

SHEAR ON PILES

$$V_s = 2148^k$$

STRUCTURE WEIGHT

ROOF =	22(19400)	=	426,800
5TH =	(22+10)(19400)	=	620,800
4TH =		=	620,800
3RD =		=	620,800
PODIUM		=	<u>4585,913</u>
			6,875,113 [#]

$$\text{Friction} = 0.35$$

$$V_{\text{FAIL}} = 0.35(6875) = 2407^k > V_s = 2148^k$$

PASSIVE

$$\text{PASSIVE} = 300 \text{ PSF}$$

MIN. LENGTH B2, B3 OR F14 PERPENDICULAR TO V_s

$$L_{\text{min}} = 163(4) + 63(5) = 967'$$

$$H = 30" = 2'-6" \quad (12" \text{ BELOW B2.})$$

$$V_{\text{PASSIVE}} = 300(967)(2.5) = 725,250^{\#}$$

$$V_{\text{R TOTAL}} = V_{\text{TENS}} + V_{\text{PASS}} = 2407 + 725 = 3132^k > V_s = 2148^k$$

∴ SHEAR TO PILES
FROM SEISMIC
15 ϕ

PORTO COCHERE

2017-06
(100)

LOADS

ROOF

DL	ROOFING - TPO	4.0 PSF.
	DECKING.	3.0 PSF.
	INSULATION	2.0
	FRAMING	4.0
	CEILING	5.0
	MISC	2.0
		20.0 PSF.
LL		20.0
		40.0 PSF

DECKING

$$L_u = 6^0$$

$$W_{DL} = 20.0 \text{ PSF}$$

$$W_{LL} = 20.0 \text{ PSF}$$

$$W_{TL} = 40.0 \text{ PSF}$$

ASC STEEL DECK

B-36 x 20 GA.

$$W_d = 66 \text{ PSF} > 40 \text{ PSF}$$

DIAPHRAGM

$$(V_s \text{ w/ } R=2.5)$$

$$V_s = \frac{11300}{2} = 5650 \text{ \#}$$

$$L_{eff} = 19^0 \text{ MIN}$$

$$D_s = 257 \text{ \#} < q = 1047 \text{ \#}$$

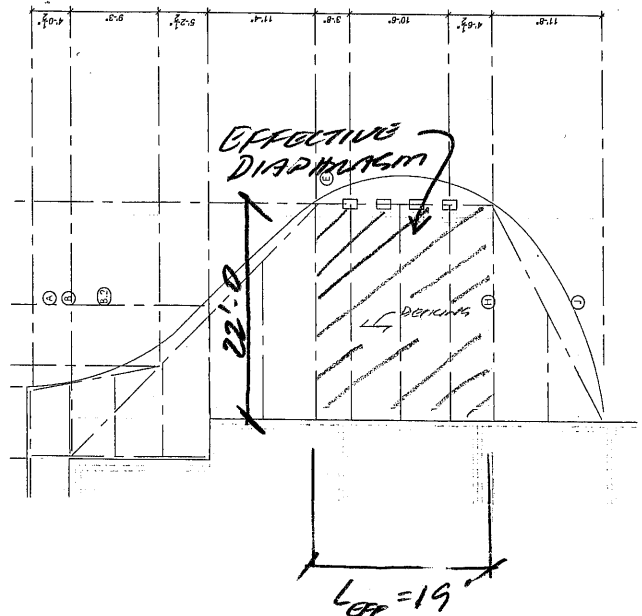
DGB-36 x 20 GA.

w/ 36/4 PLATING.

12" O.C.

(SEE P 100.2)

(100-ES-ES12-1414) (SEE P. 100.1)



Inward Allowable (f_b/Ω) and Factored (Φf_b) Distributed Load (lbs/ft²)

Gage	Span	Limit Condition	Panel Span (Support Spacing)								
			4' - 0"	5' - 0"	6' - 0"	7' - 0"	8' - 0"	9' - 0"	10' - 0"	11' - 0"	12' - 0"
22	SS	f_b / Ω	177	113	79	58	44	35	28	23	20
		Φf_b	281	180	125	92	70	56	45	37	31
		L/360	120	61	35	22	15	11	8	6	4
		L/240	-	92	53	34	22	16	11	9	7
		L/180	-	-	71	45	30	21	15	12	9
		L/120	-	-	-	-	-	32	23	17	13
	DS	f_b / Ω	185	118	82	60	46	37	30	24	21
		Φf_b	293	188	130	96	73	58	47	39	33
		L/360	-	-	-	54	36	25	18	14	11
		L/240	-	-	-	-	-	-	28	21	16
		L/180	-	-	-	-	-	-	-	-	-
		L/120	-	-	-	-	-	-	-	-	-
	TS	f_b / Ω	231	148	103	75	58	46	37	31	26
		Φf_b	367	235	163	120	92	72	59	48	41
		L/360	-	135	78	49	33	23	17	13	10
		L/240	-	-	-	74	50	35	25	19	15
		L/180	-	-	-	-	-	-	34	25	20
		L/120	-	-	-	-	-	-	-	-	-
20	SS	f_b / Ω	221	141	98	72	55	44	35	29	25
		Φf_b	350	224	156	114	88	69	56	46	39
		L/360	149	76	44	28	19	13	10	7	6
		L/240	-	114	66	42	28	20	14	11	8
		L/180	-	-	88	56	37	26	19	14	11
		L/120	-	-	-	-	-	39	29	21	17
	DS	f_b / Ω	233	149	104	76	58	46	37	31	26
		Φf_b	370	237	164	121	93	73	59	49	41
		L/360	-	-	-	67	45	31	23	17	13
		L/240	-	-	-	-	-	-	34	26	20
		L/180	-	-	-	-	-	-	-	-	-
		L/120	-	-	-	-	-	-	-	-	-
	TS	f_b / Ω	292	187	130	95	73	58	47	39	32
		Φf_b	463	296	206	151	116	91	74	61	51
		L/360	-	168	97	61	41	29	21	16	12
		L/240	-	-	-	92	62	43	32	24	18
		L/180	-	-	-	-	-	-	42	32	24
		L/120	-	-	-	-	-	-	-	-	-
18	SS	f_b / Ω	300	192	133	98	75	59	48	40	33
		Φf_b	475	304	211	155	119	94	76	63	53
		L/360	208	106	62	39	26	18	13	10	8
		L/240	-	160	92	58	39	27	20	15	12
		L/180	-	-	123	78	52	36	27	20	15
		L/120	-	-	-	-	-	55	40	30	23
	DS	f_b / Ω	312	200	139	102	78	62	50	41	35
		Φf_b	495	317	220	162	124	98	79	65	55
		L/360	-	-	-	93	63	44	32	24	19
		L/240	-	-	-	-	-	-	48	36	28
		L/180	-	-	-	-	-	-	-	-	-
		L/120	-	-	-	-	-	-	-	-	-
	TS	f_b / Ω	390	250	173	127	97	77	62	52	43
		Φf_b	619	396	275	202	155	122	99	82	69
		L/360	-	235	136	86	57	40	29	22	17
		L/240	-	-	-	-	86	60	44	33	25
		L/180	-	-	-	-	-	-	59	44	34
		L/120	-	-	-	-	-	-	-	-	-
16	SS	f_b / Ω	376	241	167	123	94	74	60	50	42
		Φf_b	597	382	265	195	149	118	96	79	66
		L/360	262	134	78	49	33	23	17	13	10
		L/240	-	201	116	73	49	34	25	19	15
		L/180	-	-	155	98	65	46	33	25	19
		L/120	-	-	-	-	-	69	50	38	29
	DS	f_b / Ω	383	245	170	125	96	76	61	51	43
		Φf_b	608	389	270	198	152	120	97	80	68
		L/360	-	-	-	118	79	55	40	30	23
		L/240	-	-	-	-	-	-	61	45	35
		L/180	-	-	-	-	-	-	-	-	-
		L/120	-	-	-	-	-	-	-	-	-
	TS	f_b / Ω	479	306	213	156	120	95	77	63	53
		Φf_b	760	486	338	248	190	150	122	100	84
		L/360	-	296	171	108	72	51	37	28	21
		L/240	-	-	-	-	108	76	55	42	32
		L/180	-	-	-	-	-	-	74	56	43
		L/120	-	-	-	-	-	-	-	-	-

B PANELS

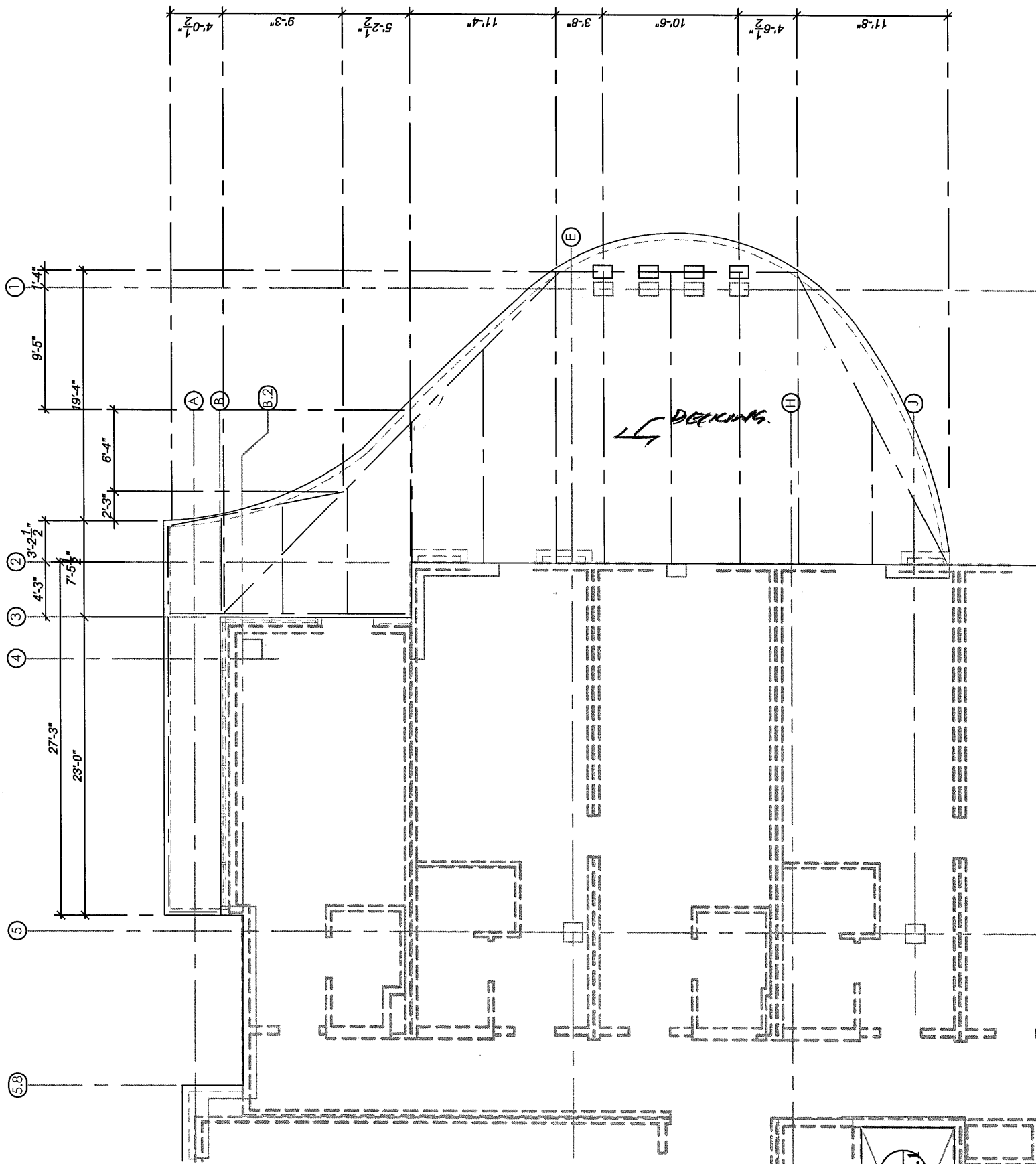
Allowable Diaphragm Shear, q_a (plf)

Flexibility Factor, F (10^{-6} in/lbs)

Gage	Welds	Spacing	Span									
			4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	
36/7/4	4"	q _a	2332	2261	2211	2173	1916	1514	1226	1013	852	
		F, lap	5.9-0.6R	6-0.6R	6.1-0.5R	6.2-0.5R	6.3-0.4R	6.4-0.4R	6.4-0.4R	6.4-0.4R	6.4-0.4R	6.5-0.3R
	F, no lap	4.1+22.9R	4.6+18.3R	4.9+15.2R	5.1+13.1R	5.3+11.5R	5.5+10.2R	5.6+9.2R	5.7+8.4R	6.2+6.6R		
	6"	q _a	2025	1924	1851	1796	1754	1514	1226	1013	852	
		F, lap	6.6-1R	6.9-0.9R	7.1-0.9R	7.2-0.9R	7.4-0.8R	7.5-0.8R	7.6-0.7R	7.6-0.7R	7.7-0.6R	
	F, no lap	4.8+22.5R	5.4+17.9R	5.9+14.8R	6.2+12.7R	6.4+11.2R	6.6+9.9R	6.8+8.9R	6.9+8R	7.5+6.3R		
	8"	q _a	1825	1751	1618	1590	1493	1226	1013	852		
		F, lap	7.1-1.3R	7.4-1.2R	7.8-1.3R	7.9-1.2R	8.3-1.2R	8.3-1.1R	8.6-1.1R	8.6-1R	8.8-0.9R	
	F, no lap	5.3+22.2R	5.9+17.6R	6.6+14.4R	6.9+12.4R	7.3+10.8R	7.4+9.5R	7.8+8.5R	7.9+7.7R	8.5+6R		
	12"	q _a	1587	1445	1344	1269	1211	1160	1114	1013	852	
		F, lap	7.9-1.8R	8.5-1.9R	9-2R	9.4-1.9R	9.7-1.9R	10-1.9R	10.2-1.8R	10.5-1.8R	10.6-1.5R	
	F, no lap	6.1+21.7R	7.1+16.9R	7.8+13.7R	8.3+11.6R	8.7+10.1R	9.1+8.8R	9.4+7.8R	9.7+6.9R	10.3+5.4R		
18"	q _a	1452	1329	1128	1066	1020	897	877	861	781		
	F, lap	8.5-2.2R	9.1-2.3R	10.2-2.8R	10.5-2.7R	10.8-2.5R	11.7-2.9R	11.9-2.7R	12-2.6R	12.6-2.3R		
F, no lap	6.7+21.3R	7.6+16.5R	9+12.9R	9.4+10.9R	9.8+9.4R	10.9+7.8R	11.1+6.9R	11.3+6.2R	12.4+4.6R			
24"	q _a	1308	1207	996	954	823	809	719	717	650		
	F, lap	9.1-2.8R	9.7-2.8R	11-3.4R	11.3-3.2R	12.4-3.6R	12.5-3.4R	13.5-3.7R	13.5-3.4R	14.2-3R		
F, no lap	7.3+20.7R	8.3+16R	9.8+12.3R	10.2+10.4R	11.4+8.4R	11.6+7.3R	12.7+6R	12.8+5.3R	13.9+3.9R			
36"	q _a	1308	1056	865	841	724	634	640	574	519		
	F, lap	9.1-2.8R	10.6-3.5R	12-4.2R	12.2-3.9R	13.4-4.3R	14.7-4.9R	14.5-4.3R	15.7-4.8R	16.3-4.1R		
F, no lap	7.3+20.7R	9.1+15.3R	10.8+11.5R	11.1+9.7R	12.4+7.6R	13.8+5.8R	13.7+5.3R	14.9+3.9R	16.1+2.9R			
48"	q _a	1145	1056	865	728	626	634	561	502	453		
	F, lap	10-3.6R	10.6-3.5R	12-4.2R	13.4-4.8R	14.7-5.4R	14.7-4.9R	15.8-5.3R	17.1-5.8R	17.7-4.8R		
F, no lap	8.2+19.9R	9.1+15.3R	10.8+11.5R	12.3+8.8R	13.8+6.6R	13.8+5.8R	15+4.3R	16.4+2.9R	17.5+2.1R			
60"	q _a	1145	898	865	728	626	546	483	502	453		
	F, lap	10-3.6R	11.7-4.5R	12-4.2R	13.4-4.8R	14.7-5.4R	16.2-6R	17.5-6.5R	17.1-5.8R	17.7-4.8R		
F, no lap	8.2+19.9R	10.2+14.3R	10.8+11.5R	12.3+8.8R	13.8+6.6R	15.3+4.6R	16.7+3.1R	16.4+2.9R	17.5+2.1R			
36/5	4"	q _a	1884	1847	1820	1799	1782	1514	1226	1013	852	
		F, lap	6-0.6R	6.1-0.5R	6.2-0.5R	6.3-0.4R	6.4-0.4R	6.4-0.4R	6.5-0.3R	6.5-0.3R	6.5-0.3R	
	F, no lap	-4.7+139.3R	-2.4+111.4R	-0.9+92.8R	-0.1+80.2R	0.6+70.7R	1.2+62.8R	1.7+56.8R	2.2+51.6R	5.2+40.9R		
	6"	q _a	1699	1637	1592	1557	1530	1507	1226	1013	852	
		F, lap	6.8-0.9R	7.1-0.9R	7.3-0.8R	7.4-0.8R	7.5-0.7R	7.6-0.7R	7.7-0.7R	7.8-0.6R	7.8-0.5R	
	F, no lap	-3.9+138.9R	-1.5+111R	0.1+92.4R	1+79.8R	1.7+70.4R	2.4+62.5R	3+56.5R	3.5+51.3R	6.5+40.6R		
	8"	q _a	1564	1518	1426	1409	1345	1340	1226	1013	852	
		F, lap	7.5-1.3R	7.7-1.2R	8.1-1.2R	8.2-1.1R	8.5-1.1R	8.5-1R	8.8-1R	8.8-0.9R	9-0.8R	
	F, no lap	-3.3+138.5R	-0.9+110.7R	1+92R	1.8+79.5R	2.7+70R	3.4+62.2R	4.1+56.1R	4.5+51R	7.6+40.4R		
	12"	q _a	1390	1288	1212	1155	1110	1044	1074	1013	852	
		F, lap	8.4-1.9R	9-2R	9.5-2R	9.8-1.9R	10.2-1.9R	10.4-1.8R	10.7-1.7R	10.9-1.7R	11-1.4R	
	F, no lap	-2.3+137.9R	0.4+109.9R	2.3+91.3R	3.4+78.7R	4.3+69.3R	5.3+61.4R	5.9+55.4R	6.5+50.2R	9.6+39.7R		
18"	q _a	1285	1194	1039	1001	971	863	846	833	756		
	F, lap	9.1-2.4R	9.7-2.4R	10.9-2.9R	11.2-2.7R	11.4-2.5R	12.4-2.9R	12.6-2.7R	12.7-2.5R	13.3-2.3R		
F, no lap	-1.6+137.5R	1.1+109.5R	3.7+90.4R	4.8+77.9R	5.6+68.6R	7.3+60.3R	7.8+54.5R	8.3+49.4R	11.9+38.9R			
24"	q _a	1166	1092	943	910	785	775	689	690	625		
	F, lap	9.9-3.1R	10.5-3R	11.9-3.6R	12.1-3.3R	13.3-3.7R	13.4-3.4R	14.5-3.7R	14.4-3.4R	15.1-3.1R		
F, no lap	-0.8+136.8R	1.9+108.9R	4.7+89.6R	5.7+77.3R	7.5+67.4R	8.2+59.8R	9.7+53.4R	10.1+48.5R	13.8+38.1R			
36"	q _a	1166	980	814	797	686	600	610	546	493		
	F, lap	9.9-3.1R	11.6-3.8R	13.2-4.6R	13.3-4.1R	14.6-4.6R	16-5.2R	17-5R	17.7-4.3R	17.7-4.3R		
F, no lap	-0.8+136.8R	3+108R	6+88.6R	6.9+76.5R	8.8+66.5R	10.9+58R	11+52.6R	12.7+46.9R	16.4+36.9R			
48"	q _a	1034	980	814	685	588	531	474	427			
	F, lap	11.1-4.1R	11.6-3.8R	13.2-4.6R	14.8-5.3R	16.3-5.9R	16-5.2R	17.4-5.6R	18.8-6.2R	19.5-5.2R		
F, no lap	0.4+135.8R	3+108R	6+88.6R	8.4+75.3R	10.5+65.2R	10.9+58R	12.6+51.5R	14.4+45.7R	18.1+36R			
60"	q _a	1034	837	814	685	588	512	452	474			
	F, lap	11.1-4.1R	13-5.1R	13.2-4.6R	14.8-5.3R	16.3-5.9R	18-6.6R	19.5-7.2R	18.8-6.2R	19.5-5.2R		
F, no lap	0.4+135.8R	4.5+106.8R	6+88.6R	8.4+75.3R	10.5+65.2R	12.8+56.6R	14.7+49.9R	14.4+45.7R	18.1+36R			
36/4	4"	q _a	1575	1555	1540	1528	1520	1513	1226	1013	852	
		F, lap	6.1-0.5R	6.3-0.5R	6.4-0.4R	6.4-0.4R	6.5-0.3R	6.5-0.3R	6.6-0.3R	6.6-0.3R	6.6-0.2R	
	F, no lap	-9+197.5R	-5.9+157.9R	-3.8+131.6R	-2.6+113.8R	-1.8+100.3R	-0.8+89.2R	-0.2+80.6R	0.5+73.2R	4.7+58.1R		
	6"	q _a	1434	1397	1369	1348	1332	1319	1226	1013	852	
		F, lap	7.1-0.9R	7.3-0.8R	7.5-0.8R	7.6-0.7R	7.7-0.7R	7.8-0.6R	7.9-0.6R	7.9-0.5R	8-0.5R	
	F, no lap	-8.1+197.1R	-4.8+157.6R	-2.6+131.2R	-1.5+113.4R	-0.5+100R	0.5+88.9R	1.1+80.3R	1.8+73R	6+57.8R		
	8"	q _a	1323	1300	1234	1228	1182	1184	1149	1013	852	
		F, lap	7.8-1.3R	8-1.1R	8.5-1.2R	8.5-1R	8.8-1R	8.8-0.9R	9.1-0.9R	9-0.8R	9.2-0.7R	
	F, no lap	-7.3+196.7R	-4.2+157.3R	-1.7+130.8R	-0.6+111.3R	0.6+99.7R	1.5+88.6R	2.3+80R	2.9+72.7R	7.3+57.6R		
	12"	q _a	1170	1099	1047	1008	977	952	931	914	852	
		F, lap	9-2R	9.6-2R	10-1.9R	10.4-1.8R	10.7-1.8R	10.9-1.7R	11.1-1.6R	11.3-1.6R	11.4-1.3R	
	F, no lap	-6.2+196R	-2.6+156.4R	-0.1+130.1R	1.3+112.3R	2.4+98.9R	3.6+87.8R	4.4+79.2R	5.2+71.9R	9.5+57R		
18"	q _a	1073	1014	887	865	849	761	755	750	680		
	F, lap	9.9-2.5R	10.4-2.4R	11.8-2.9R	12-2.7R	12.2-2.5R	13.3-2.8R	13.3-2.6R	13.4-2.4R	14.1-2.2R		
F, no lap	-5.3+195.5R	-1.8+156R	1.7+129.1R	3+111.4R	3.9+98.2R	6+86.7R	6.6+78.3R	7.3+71.1R	12.2+56.1R			
24"	q _a	960	917	793	779	671	674	597	606	548		
	F, lap	11-3.4R	11.5-3.2R	13.1-3.8R	13.1-3.4R	14.5-3.8R	14.4-3.4R	15.6-3.7R	15.5-3.4R	16.3-3.1R		
F, no lap	-4.2+194.6R	-0.7+155.2R	2.9+128.2R	4.1+110.8R	6.3+96.9R	7.1+86.1R	8.9+77.1R	9.3+70.1R	14.4+55.2R			
36"	q _a	960	809	661	667	572	498	518	463	417		
	F, lap	11-3.4R	12.9-4.2R	14.8-5.1R	14.6-4.4R	16.2-4.9R	17.8-5.5R	17.2-4.7R	18.6-5.1R	19.5-4.5R		
F, no lap	-4.2+194.6R	0.7+154.2R	4.7+126.9R	5.6+109.8R	7.9+95.8R	10.4+84R	10.5+76.2R	12.5+68.4R	17.6+53.8R			
48"	q _a	829	809	661	554	473	498	440	391	351		
	F, lap	12.6-4.7R	12.9-4.2R	14.8-5.1R	16.6-5.8R	18.5-6.5R	17.8-5.5R	19.3-6R	20.9-6.6R	21.9-5.6R		
F, no lap	-2.5+193.3R	0.7+154.2R	4.7+126.9R	7.6+108.3R	10.2+94.2R	10.4+84R	12.6+74.9R	14.8+66.9R	19.9+52.7R			
60"	q _a	829	654	661	554	473	411	361	391	351		
	F, lap	12.6-4.7R	14.9-5.9R	14.8-5.1R	16.6-5.8R	18.5-6.5R	20.3-7.3R	22.1-7.9R	20.9-6.6R	21.9-5.6R		
F, no lap	-2.5+193.3R	2.8+152.5R	4.7+126.9R	7.6+108.3R	10.2+94.2R	13+82.2R	15.4+72.9R	14.8+66.9R	19.9+52.7R			

Porte Cochere

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SEISMIC BASE SHEAR

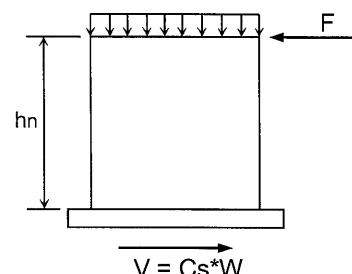
Per CBC 2016 and ASCE 7-10 Specifications

Using Equivalent Lateral Force Procedure for Regular Single-Level Building/Structural Systems

Job Name:	HOME2 - SSF (Portico)	Subject:	
Job Number:		Originator:	Checker:

Input Data:

Occupancy Category =	II	ASCE 7-10 Table 1.5-1
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2
Soil Site Class =	D	ASCE 7-10, Chp. 20
Latitude =		
Longitude =		
Spectral Accel., S_s =	1.940	ASCE 7-10 Figures 22-1, 3, 5, 6
Spectral Accel., S_1 =	0.909	ASCE 7-10 Figures 22-2, 4, 5, 6
Long. Trans. Period, T_L =	8.000	sec. ASCE 7 Fig's. 22-12 to 22-16
Structure Height, h_n =	14.000	ft.
Total Seismic Weight, W =	24.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, T_c =	0.000	sec. from Analysis or "0" if T_a (below (Regular Bldg. Configurations Only)
Seismic Resist. System =	G1	G1. Steel special cantilevered column system (ASCE 7-10, Table 12.2-1)



Seismic Base Shear

Results:

Site Coefficients:

F_a =	1.000	ASCE 7-10, Table 11.4.-1
F_v =	1.500	ASCE 7-10, Table 11.4-2

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS =	1.940	SMS = $F_a * S_s$, ASCE 7-10, Eqn 11.4-1
SM1 =	1.364	SM1 = $F_v * S_1$, ASCE 7-10, Eqn 11.4-2

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS =	1.293	SDS = $2 * SMS / 3$, ASCE -10, Eqn 11.4-3
SD1 =	0.909	SD1 = $2 * SM1 / 3$, ASCE 7-10 Eqn 11.4-4

Seismic Design Category:

Category(for SDS) =	E	ASCE 7-10, Table 11.6-1
Category(for SD1) =	E	ASCE 7-10, Table 11.6-2
Use Category =	E	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2
Approx. Period, T_a =	0.145	sec., $T_a = C_T * h_n^x$, ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.400	ASCE 7-10 Table 12.8-1
Period max., $T_{(max)}$ =	0.203	sec., $T_{(max)} = C_u * T_a$, ASCE 7-10 Section 12.8.2
Fundamental Period, T =	0.145	sec., $T = T_a \leq C_u * T_a$, ASCE 7-10 Section 12.8.2

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	2.5	ASCE 7-10 Table 12.2-1
Overstrength Factor, Ω_o =	1.25	ASCE 7-10 Table 12.2-1
Defl. Amplif. Factor, C_d =	2.5	ASCE 7-10 Table 12.2-1
C_s =	0.517	$C_s = SDS / (R/I)$, ASCE 7-10 Section 12.8.1.1, Eqn. 12.8-2
$C_s(max)$ =	2.512	For $T \leq T_L$, $C_s(max) = SD1 / (T * (R/I))$, ASCE 7-10 Eqn. 12.8-3
$C_s(min)$ =	0.182	$C_s(min) = 0.5 * S_1 / (R/I)$, ASCE 7-10 Eqn. 12.8-6
Use: C_s =	0.517	$C_s(min) \leq C_s \leq C_s(max)$
C_{ASD} =	0.362	$C_{ASD} = 0.7 * C_s$

Seismic Base Shear:

Redundancy, ρ =	1.3	ASCE 12.3.4.1
V =	16.14	kips, $V = C_s * W$, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1
V_{ASD} =	11.30	kips, $V_{ASD} = 0.7 * V$

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MecaWind Pro v2.2.7.5 per ASCE 7-10

Developed by MECA Enterprises, Inc. Copyright www.mecaenterprises.com

Date : 8/11/2017 Project No. :
 Company Name : HCP ENGINEERING Designed By :
 Address : 2466 Redrock Dr Description :
 City : Corona Customer Name :
 State : CA Proj Location :
 File Location: C:\Users\HCP ENGINEERING\AppData\Roaming\MecaWind\Default.wnd

Input Parameters: Directional Procedure Open Building (Ch 27 Part 1)

Basic Wind Speed(V)	=	110.00 mph	Exposure Category	=	C
Structural Category	=	II	Flexible Structure	=	No
Natural Frequency	=	N/A	Kd Directional Factor	=	0.85
Importance Factor	=	1.00			
Damping Ratio (beta)	=	0.01			
Alpha	=	9.50	Zg	=	900.00 ft
At	=	0.11	Bt	=	1.00
Am	=	0.15	Bm	=	0.65
Cc	=	0.20	l	=	500.00 ft
Epsilon	=	0.20	Zmin	=	15.00 ft
Pitch of Roof	=	0.25 : 12	Slope of Roof(Theta)	=	1.19 Deg
D: Roof Len along Ridge	=	60.00 ft	L: Horizontal Width	=	25.00 ft
h: Mean Roof Ht	=	14.00 ft	Type of Roof	=	MONOSLOPE

Gust Factor Calculations

Gust Factor Category I Rigid Structures - Simplified Method
 Gust1: For Rigid Structures (Nat. Freq.>1 Hz) use 0.85 = 0.85

Gust Factor Category II Rigid Structures - Complete Analysis
 Zm: $0.6 \cdot H_t$ = 15.00 ft
 lzm: $C_c \cdot (33/Z_m)^{0.167}$ = 0.23
 Lzm: $1 \cdot (Z_m/33)^{\text{Epsilon}}$ = 427.06 ft
 Q: $(1/(1+0.63 \cdot ((B+H_t)/Lzm)^{0.63}))^{0.5}$ = 0.94
 Gust2: $0.925 \cdot ((1+1.7 \cdot lzm \cdot 3.4 \cdot Q)/(1+1.7 \cdot 3.4 \cdot lzm))$ = 0.89

Gust Factor Summary
 Not a Flexible Structure use the Lessor of Gust1 or Gust2 = 0.85

Open Building-Monoslope Roof per Figure 27.4-4:

Normal to Ridge - Open Building - Monoslope Roof per Figure 27.4-4:

Gamma = 0 degrees, Clear Wind Flow

Load Case	Cnw	Cnl	Windward Wind Pres. psf	Leeward Wind Pres. psf
A	1.200	0.300	13.679	3.420
B	-1.100	-0.100	-12.539	-1.140

Normal to Eave - Open Building - Monoslope Roof per Figure 27.4-4:

Gamma = 180 degrees, Clear Wind Flow

Load Case	Cnw	Cnl	Windward Wind Pres. psf	Leeward Wind Pres. psf
A	1.200	0.300	13.679	3.420
B	-1.100	-0.100	-12.539	-1.140

Along Ridge - Open Building - Monoslope Roof per Figure 27.4-4:

Gamma = 90 degrees, Clear Wind Flow

Length Along Ridge of Roof	Roof Angle (Theta)	Load Case	Cn psf	Wind Press Along Ridge psf
<= 14.0	Theta<=45 deg	A	-0.800	-9.119
		B	0.800	9.119
>14.0 & <= 2*14.0	Theta<=45 deg	A	-0.600	-6.839
		B	0.500	5.699
>2*14	Theta<=45 deg	A	-0.300	-3.420
		B	0.300	3.420

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Wind Pressure on Components and Cladding (Ch 30 Part 5)

All pressures shown are based upon ASD Design, with a Load Factor of .6

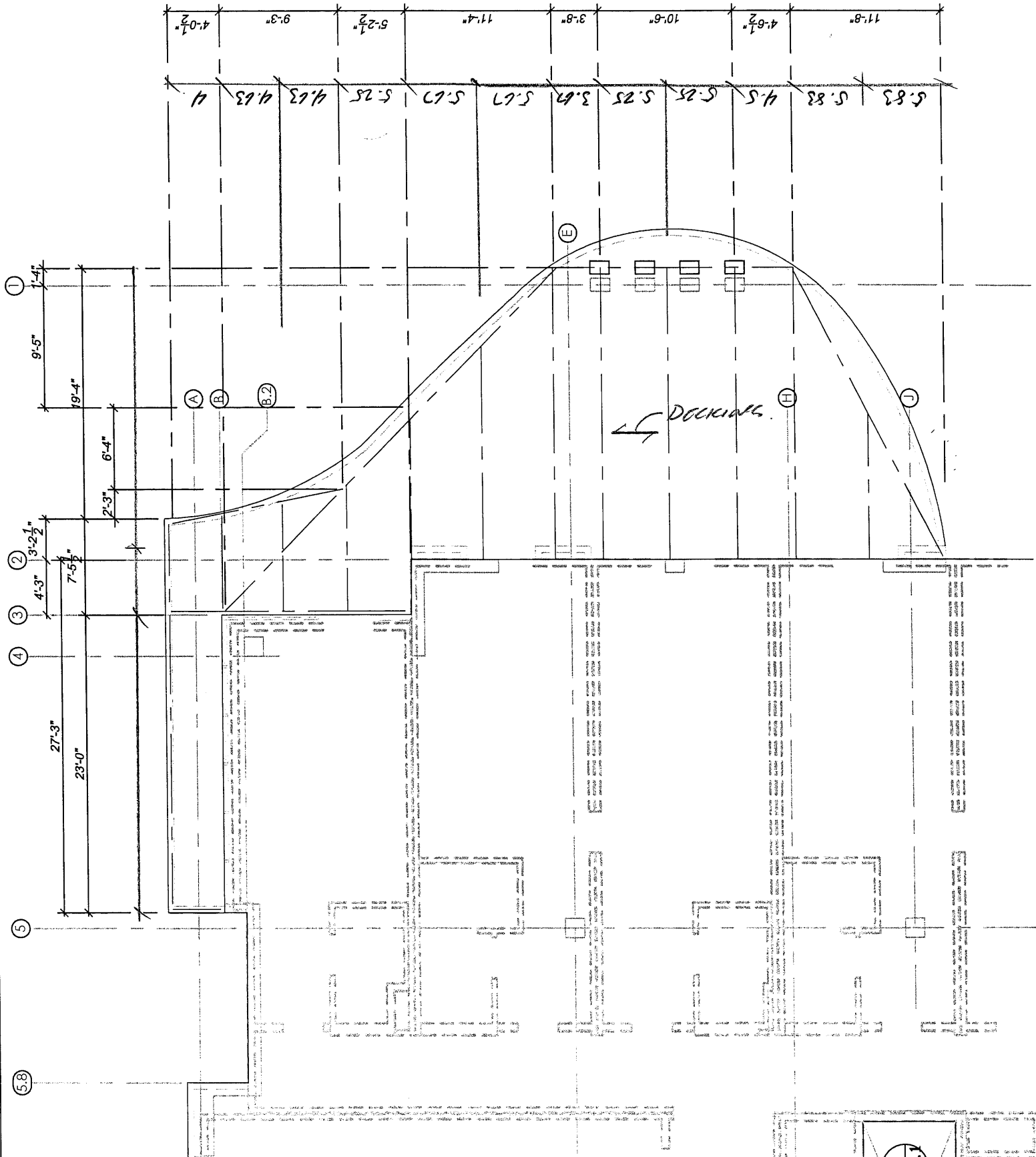
Width of Pressure Coefficient Zone "a" = 3.00 ft

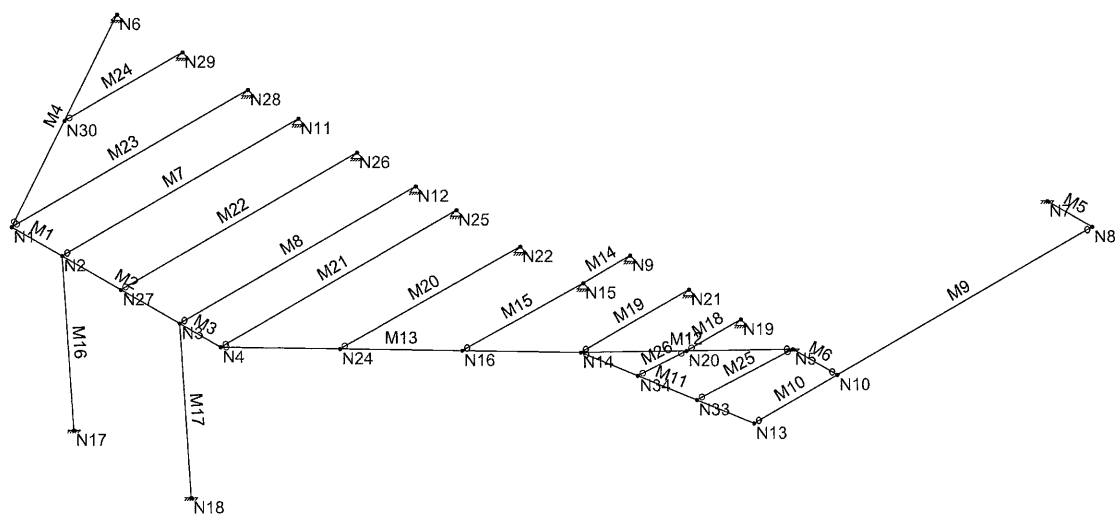
Description	Width ft	Span ft	Area ft^2	Zone	Cn Max	Cn Min	Max P psf	Min P psf
Zone 1	1.00	1.00	1.0	1	1.26	-1.15	14.40	-13.08
Zone 2	1.00	1.00	1.0	2	1.90	-1.76	21.61	-20.10
Zone 3	1.00	1.00	1.0	3	2.53	-3.44	28.81	-39.25
Zone 4	1.00	1.00	1.0	1	1.26	-1.15	14.40	-13.08
Zone 5	1.00	1.00	1.0	1	1.26	-1.15	14.40	-13.08

} Avg = -26.07 psf

Porte Cochonne

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Envelope Only Solution

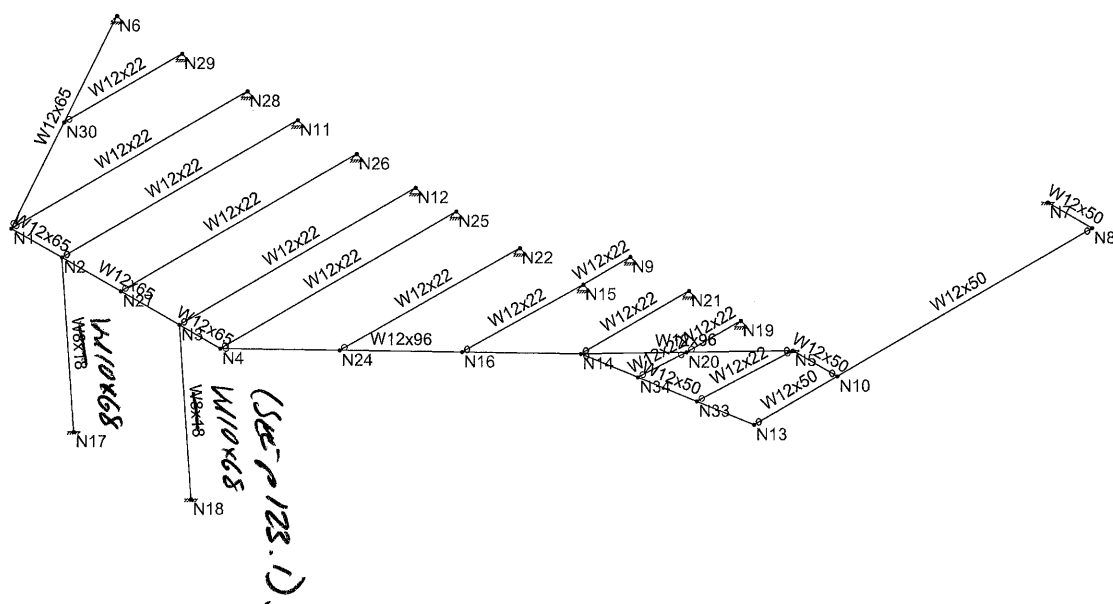
HCP ENGINEERING

SK - 1

Oct 21, 2017 at 8:08 AM

2017-06-h2ssf-PorteCo-rev-17102...

MODEL



Envelope Only Solution

HCP ENGINEERING

SK - 2

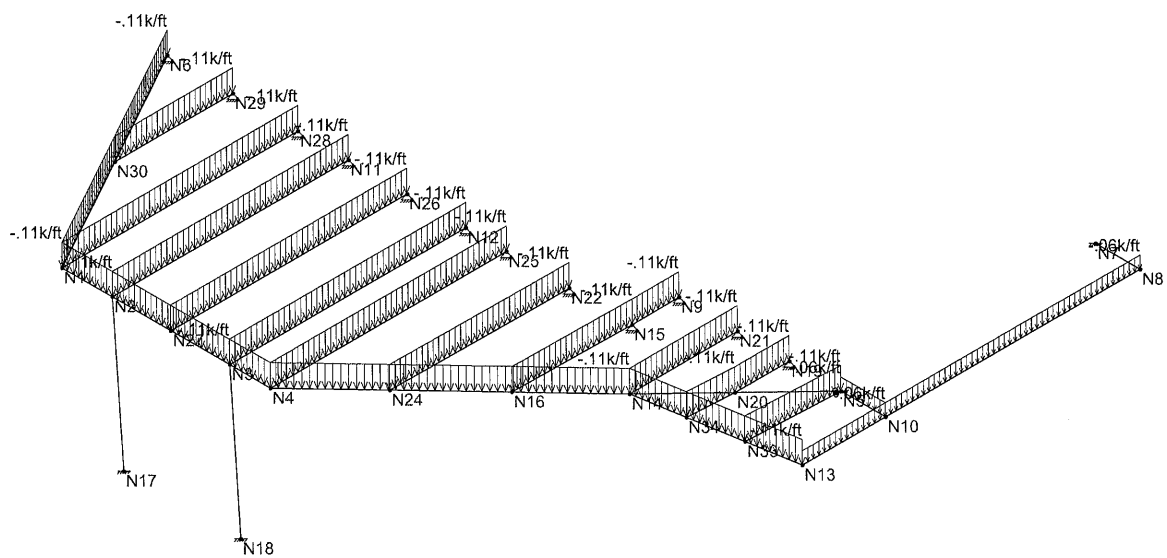
Oct 21, 2017 at 8:09 AM

2017-06-h2ssf-PorteCo-rev-17102...

MODEL



2017-06
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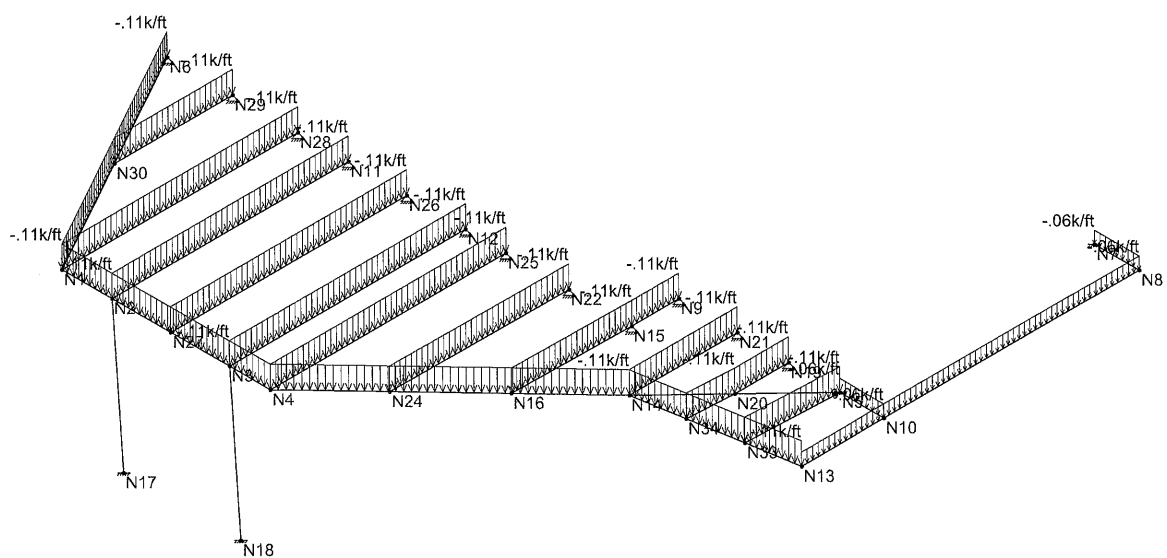


Loads: BLC 1, D
Envelope Only Solution

HCP ENGINEERING	DEAL LOAD	SK - 2
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		2017-06-h2ssf-PorteCo.r3d



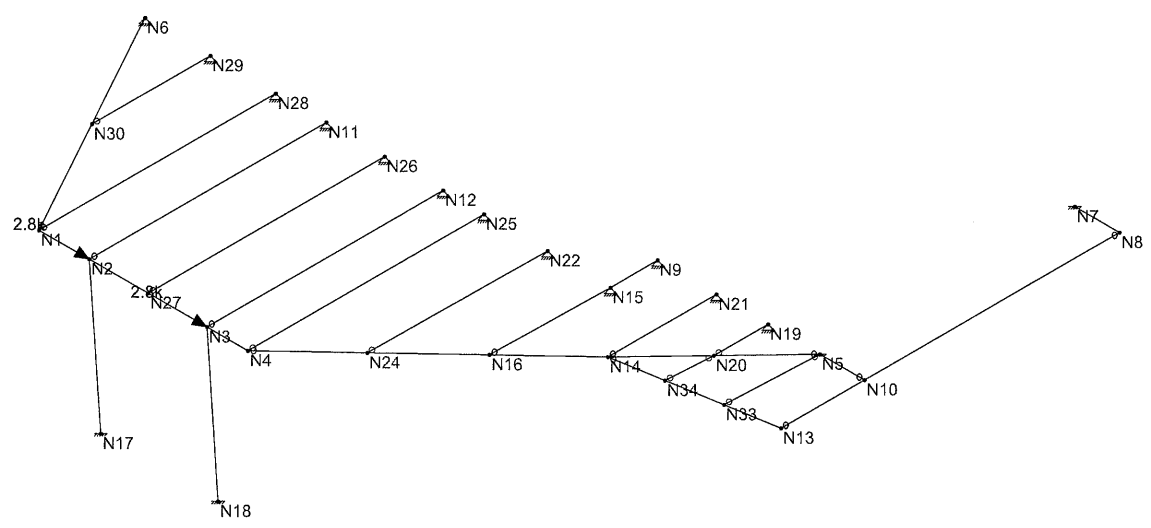
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Loads: BLC 2, L
Envelope Only Solution

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		2017-06-h2ssf-PorteCo.r3d

2017-06
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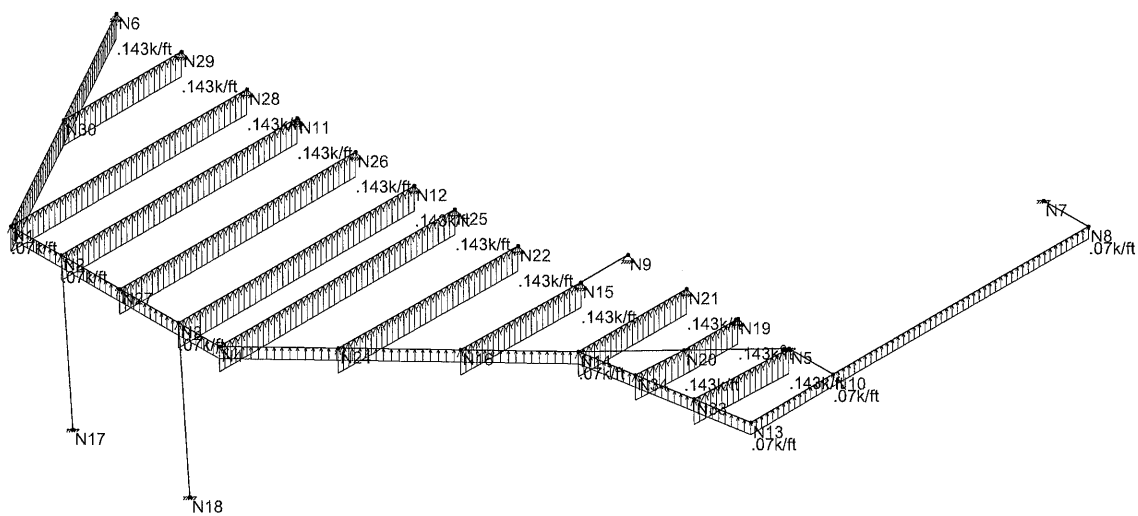


Loads: BLC 6, EY
Envelope Only Solution

HCP ENGINEERING	SEISMIC - EQX	SK - 3
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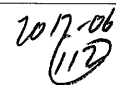


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Loads: BLC 3, WX

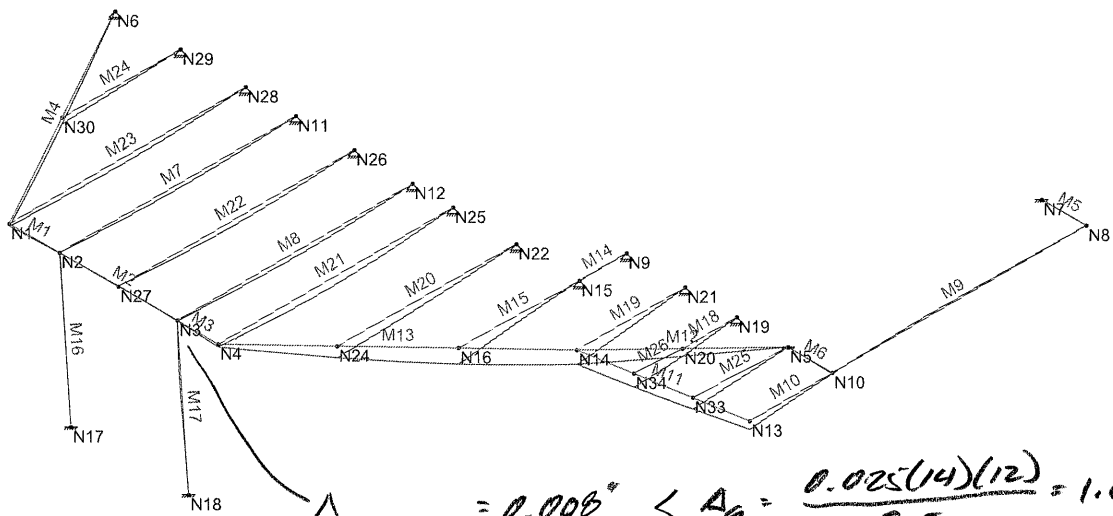
HCP ENGINEERING	WIND-WX	SK - 5
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		2017-06-h2ssf-PorteCo.r3d



HCP ENGINEERING	UNITY	SK - 14
		Aug 11, 2017 at 3:55 PM
		2017-06-h2ssf-PorteCo.r3d



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$$\Delta_{N2 \& N3} = 0.008'' < \Delta_a = \frac{0.025(14)(12)}{2.5} = 1.67$$

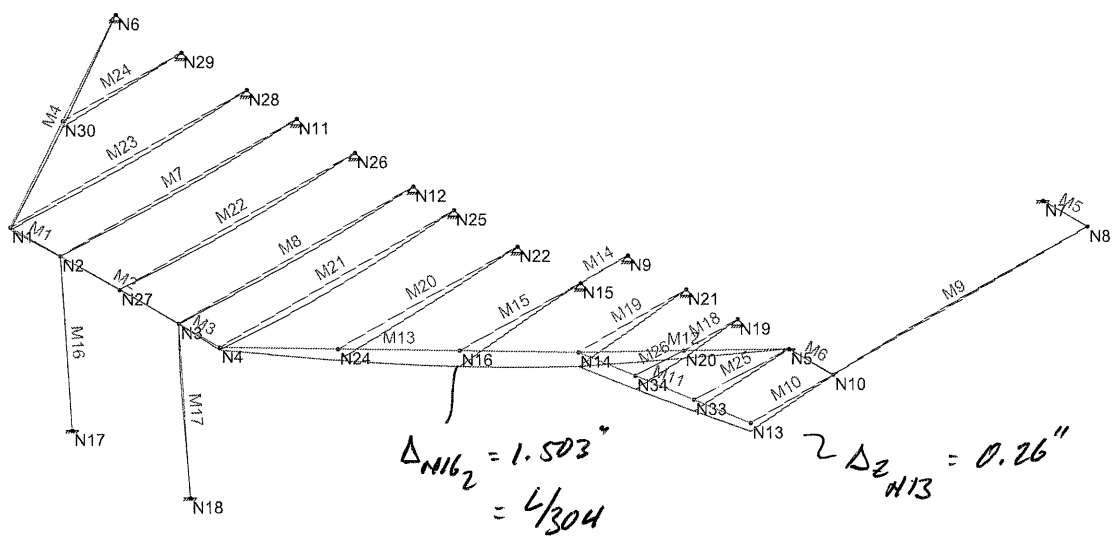
O.K.

Results for LC 19, 8) D + 0.7EY

HCP ENGINEERING		SK - 10
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	DRIFT	2017-06-h2ssf-PorteCo.r3d



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Results for LC 3, 2a) D + L

HCP ENGINEERING

SK - 11

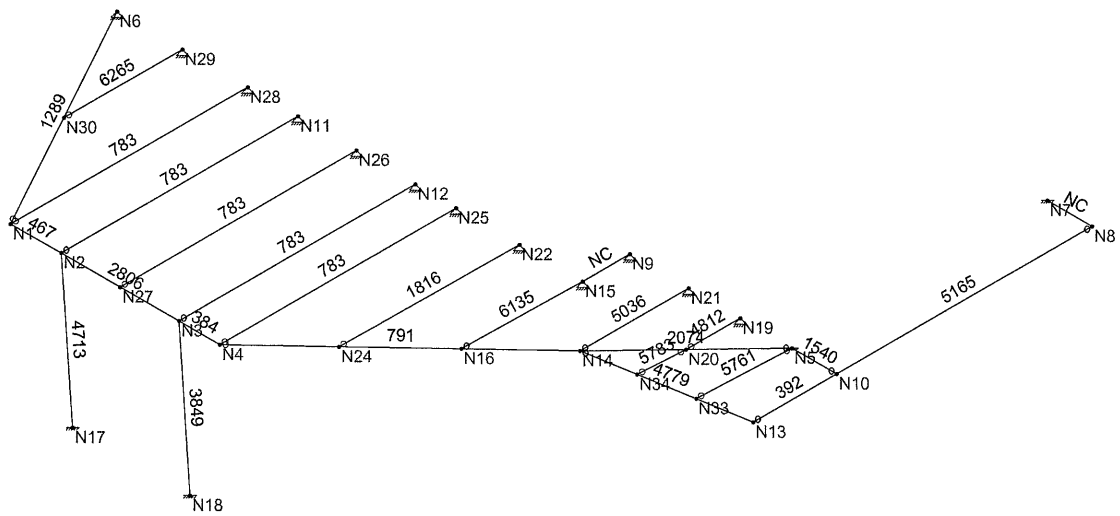
Aug 11, 2017 at 2:54 PM

DELECTION

2017-06-h2ssf-PorteCo.r3d



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L/y Deflection Ratio (Enveloped)
Envelope Only Solution

HCP ENGINEERING	DEFLECTION RATIO	SK - 15
		Aug 11, 2017 at 3:56 PM
		2017-06-h2ssf-PorteCo.r3d



Company : HCP ENGINEERING
Designer :
Job Number :
Model Name :

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116.1

Oct 22, 2017
12:04 PM
Checked By: _____

(Global) Model Settings

Display Sections for Member Calcs	5
Max Internal Sections for Member Calcs	97
Include Shear Deformation?	Yes
Increase Nailing Capacity for Wind?	Yes
Include Warping?	Yes
Trans Load Btwn Intersecting Wood Wall?	Yes
Area Load Mesh (in^2)	144
Merge Tolerance (in)	.12
P-Delta Analysis Tolerance	0.50%
Include P-Delta for Walls?	Yes
Automatically Iterate Stiffness for Walls?	Yes
Max Iterations for Wall Stiffness	3
Gravity Acceleration (ft/sec^2)	32.2
Wall Mesh Size (in)	24
Eigensolution Convergence Tol. (1.E-)	4
Vertical Axis	Z
Global Member Orientation Plane	XY
Static Solver	Sparse Accelerated
Dynamic Solver	Accelerated Solver

Hot Rolled Steel Code	AISC 14th(360-10): ASD
Adjust Stiffness?	Yes(Iterative)
RISAConnection Code	AISC 14th(360-10): ASD
Cold Formed Steel Code	AISI S100-12: ASD
Wood Code	AWC NDS-12: ASD
Wood Temperature	< 100F
Concrete Code	ACI 318-11
Masonry Code	ACI 530-13: ASD
Aluminum Code	AA ADM1-10: ASD - Building

Number of Shear Regions	4
Region Spacing Increment (in)	4
Biaxial Column Method	Exact Integration
Parame Beta Factor (PCA)	.65
Concrete Stress Block	Rectangular
Use Cracked Sections?	Yes
Use Cracked Sections Slab?	Yes
Bad Framing Warnings?	No
Unused Force Warnings?	Yes
Min 1 Bar Diam. Spacing?	No
Concrete Rebar Set	REBAR SET ASTMA615
Min % Steel for Column	2
Max % Steel for Column	8

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Hot Rolled Steel Properties

Label	Shape	Type	E [ksi]	G [ksi]	Nu	Therm (L...Density)	Yield [ksi]	Rv	Fu [ksi]	Rt
1	A36 Gr.36	Shapes M, S, C MC & L	29000	11154	3	.65	49	36	1.5	58
2	A572 Gr.50	Shapes: HP	29000	11154	3	.65	49	50	1.1	65
3	A992	Gr.50	29000	11154	3	.65	49	50	1.1	65
4	A500 Gr.B	Shapes: HSS RND	29000	11154	3	.65	527	42	1.4	58
5	A500 Gr.B	Shapes: Rect	29000	11154	3	.65	527	46	1.4	58
6	A53 Gr.B	Shapes: Pipes	29000	11154	3	.65	49	35	1.6	60
7	A1085		29000	11154	3	.65	49	50	1.4	65

Hot Rolled Steel Section Sets

Label	Shape	Type	Material	Design Rules	A [in2]	Iy [in4]	Izz [in4]	J [in4]
1	BM1	W12x65	A992- Shapes: W's	Typical	19.1	174	533	2.18
2	BM2	W12x65	A992- Shapes: W's	Typical	19.1	174	533	2.18
3	BM3	W12x65	A992- Shapes: W's	Typical	28.2	270	833	6.85
4	BM4	W12x22	A992- Shapes: W's	Typical	6.48	156	293	1.71
5	BM1	W12x22	A992- Shapes: W's	Typical	14.6	56.3	391	1.71
6	RIM2	W12x50	A992- Shapes: W's	Typical	14.6	56.3	391	1.71
7	CANT1	W12x50	A992- Shapes: W's	Typical	14.6	56.3	391	1.71
8	LDGR	W10x33	A992- Shapes: W's	Typical	9.71	36.6	171	.583
9	COL	W8x18	A992- Shapes: W's	Typical	5.26	7.97	61.9	.172

Joint Coordinates and Temperatures

Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Di...
1	48.5	11.67	0	0	
2	N1	48.5	0	0	
3	N2	16.17	0	0	
4	N3	26.67	0	0	
5	N4	30.34	0	0	
6	N5	23	0	0	
7	N6	27.25	0	0	
8	N7	0	56.17	0	
9	N8	0	60.17	0	
10	N9	23	41.67	0	
11	N10	27.25	16.17	0	
12	N11	27.25	0	0	
13	N12	27.25	0	0	
14	N13	30.34	0	0	
15	N14	32.75	0	0	
16	N15	37.9475	0	0	
17	N16	47.56	0	0	
18	N17	47.56	-14	0	
19	N18	47.56	0	0	
20	N19	23	51.55	0	
21	N20	27.875	0	0	
22	N21	23	46.92	0	
23	N22	27.25	36	0	
24	N23	43.3025	0	0	
25	N24	27.25	30.33	0	
26	N25	27.25	21.41	0	
27	N26	27.25	21.42	0	
28	N27	27.25	11.66	0	
29	N28	27.25	5.83	0	
30	N29	37.875	5.835	0	
31	N30	31.2425	55.7975	0	
32	N31	32.0075	51.2925	0	

Joint Boundary Conditions

Joint Label	X [in]	Y [in]	Z [in]	X Rot. [k-ft/rad]	Y Rot. [k-ft/rad]	Z Rot. [k-ft/rad]
1	N7	Reaction	Reaction	Reaction	Reaction	Reaction
2	N5	Reaction	Reaction	Reaction	Reaction	Reaction
3	N9	Reaction	Reaction	Reaction	Reaction	Reaction
4	N15	Reaction	Reaction	Reaction	Reaction	Reaction
5	N12	Reaction	Reaction	Reaction	Reaction	Reaction
6	N11	Reaction	Reaction	Reaction	Reaction	Reaction
7	N6	Reaction	Reaction	Reaction	Reaction	Reaction
8	N17	Reaction	Reaction	Reaction	Reaction	Reaction
9	N18	Reaction	Reaction	Reaction	Reaction	Reaction
10	N29	Reaction	Reaction	Reaction	Reaction	Reaction
11	N28	Reaction	Reaction	Reaction	Reaction	Reaction
12	N26	Reaction	Reaction	Reaction	Reaction	Reaction
13	N25	Reaction	Reaction	Reaction	Reaction	Reaction
14	N21	Reaction	Reaction	Reaction	Reaction	Reaction
15	N19	Reaction	Reaction	Reaction	Reaction	Reaction
16	N22	Reaction	Reaction	Reaction	Reaction	Reaction

Member Primary Data

Label	I Joint	J Joint	K Joint	Rotate (deg)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2		BM2	Beam	None	A992...	Typical
2	M2	N2	N3		BM2	Beam	None	A992...	Typical
3	M3	N3	N4		BM2	Beam	None	A992...	Typical
4	M4	N6	N1		BM1	Beam	None	A992...	Typical
5	M5	N7	N8		CANT1	Beam	None	A992...	Typical
6	M6	N5	N10		CANT1	Beam	None	A992...	Typical
7	M7	N11	N2		BM4	Beam	None	A992...	Typical
8	M8	N12	N3		BM4	Beam	None	A992...	Typical
9	M9	N8	N10		RIM1	Beam	None	A992...	Typical
10	M10	N10	N13		RIM1	Beam	None	A992...	Typical
11	M11	N13	N14		RIM2	Beam	None	A992...	Typical
12	M12	N5	N14		BM3	Beam	None	A992...	Typical
13	M13	N14	N15		BM4	Beam	None	A992...	Typical
14	M14	N9	N15		BM4	Beam	None	A992...	Typical
15	M15	N15	N16		BM4	Beam	None	A992...	Typical
16	M16	N17	N2	90	COL	Column	Wide Flange	A992...	Typical
17	M17	N18	N3	90	COL	Column	Wide Flange	A992...	Typical
18	M18	N19	N20		BM4	Beam	None	A992...	Typical
19	M19	N21	N14		BM4	Beam	None	A992...	Typical
20	M20	N22	N24		BM4	Beam	None	A992...	Typical
21	M21	N25	N4		BM4	Beam	None	A992...	Typical
22	M22	N26	N27		BM4	Beam	None	A992...	Typical
23	M23	N28	N1		BM4	Beam	None	A992...	Typical
24	M24	N29	N30		BM4	Beam	None	A992...	Typical
25	M25	N5	N33		BM4	Beam	None	A992...	Typical
26	M26	N20	N34		BM4	Beam	None	A992...	Typical

Member Advanced Data

Label	I Release	J Release	K Release	I Offset [in]	J Offset [in]	T/C Only	Physical Analysis	Inactive	Seismic Design
1	M1						Yes		None
2	M2						Yes		None
3	M3						Yes		None
4	M4		BenPIN				Yes		None
5	M5						Yes		None
6	M6		AIIPIN				Yes		None
7	M7		AIIPIN				Yes		None
8	M8		AIIPIN				Yes		None
9	M9		AIIPIN				Yes		None

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Member Advanced Data (Continued)

Label	I Release	J Release	I Offset(in)	J Offset(in)	T/C Only	Physical Analysis	Inactive	Seismic Design
10 M10		ALPIN				Yes		None
11 M11		ALPIN				Yes		None
12 M12		ALPIN				Yes		None
13 M13		BenPIN				Yes		None
14 M14		BenPIN				Yes		None
15 M15		ALPIN				Yes		None
16 M16		BenPIN				Yes		None
17 M17		BenPIN				Yes		None
18 M18		BenPIN				Yes		None
19 M19		ALPIN				Yes		None
20 M20		BenPIN				Yes		None
21 M21		BenPIN				Yes		None
22 M22		BenPIN				Yes		None
23 M23		BenPIN				Yes		None
24 M24		BenPIN				Yes		None
25 M25		BenPIN				Yes		None
26 M26		BenPIN				Yes		None

Joint Loads and Enforced Displacements (BLC 6 : EY)

Joint Label	L.D.M	Direction	Magnitude (k.k-ft)	(n,rad)	(k's'2ft)
1 N2	L	Y		2.8	
2 N3	L	Y		2.8	

Member Point Loads

Member Label	Direction	Magnitude(k.k-ft)	Location(ft.)
No Data to Print			

Member Distributed Loads (BLC 1 : D)

Member Label	Direction	Start Magnitude(k/ft.)	End Magnitude(k/ft.)	Start Location(ft.)	End Location(ft.)
1 M24	Z	-11	-11	0	0
2 M23	Z	-11	-11	0	0
3 M7	Z	-11	-11	0	0
4 M22	Z	-11	-11	0	0
5 M8	Z	-11	-11	0	0
6 M21	Z	-11	-11	0	0
7 M20	Z	-11	-11	0	0
8 M15	Z	-11	-11	0	0
9 M19	Z	-11	-11	0	0
10 M18	Z	-11	-11	0	0
11 M25	Z	-11	-11	0	0
12 M9	Z	-06	-06	0	0
13 M10	Z	-06	-06	0	0
14 M4	Z	-11	-11	0	0
15 M2	Z	-11	-11	0	0
16 M1	Z	-11	-11	0	0
17 M3	Z	-11	-11	0	0
18 M13	Z	-11	-11	0	0
19 M11	Z	-11	-11	0	0
20 M26	Z	-11	-11	0	0
21 M14	Z	-11	-11	0	0
22 M6	Z	-06	-06	0	0

Member Distributed Loads (BLC 2 : L)

Member Label	Direction	Start Magnitude(k/ft.)	End Magnitude(k/ft.)	Start Location(ft.)	End Location(ft.)
1 M4	Z	-11	-11	0	0

Member Distributed Loads (BLC 2 : L) (Continued)

Member Label	Direction	Start Magnitude(k/ft.)	End Magnitude(k/ft.)	Start Location(ft.)	End Location(ft.)
2 M24	Z	-11	-11	0	0
3 M23	Z	-11	-11	0	0
4 M1	Z	-11	-11	0	0
5 M7	Z	-11	-11	0	0
6 M2	Z	-11	-11	0	0
7 M22	Z	-11	-11	0	0
8 M8	Z	-11	-11	0	0
9 M21	Z	-11	-11	0	0
10 M3	Z	-11	-11	0	0
11 M13	Z	-11	-11	0	0
12 M20	Z	-11	-11	0	0
13 M15	Z	-11	-11	0	0
14 M19	Z	-11	-11	0	0
15 M11	Z	-11	-11	0	0
16 M18	Z	-11	-11	0	0
17 M25	Z	-11	-11	0	0
18 M9	Z	-06	-06	0	0
19 M10	Z	-06	-06	0	0
20 M6	Z	-06	-06	0	0
21 M5	Z	-06	-06	0	0
22 M14	Z	-11	-11	0	0

Member Distributed Loads (BLC 3 : WX)

Member Label	Direction	Start Magnitude(k/ft.)	End Magnitude(k/ft.)	Start Location(ft.)	End Location(ft.)
1 M24	Z	-143	-143	0	0
2 M23	Z	-143	-143	0	0
3 M7	Z	-143	-143	0	0
4 M22	Z	-143	-143	0	0
5 M8	Z	-143	-143	0	0
6 M21	Z	-143	-143	0	0
7 M20	Z	-143	-143	0	0
8 M15	Z	-143	-143	0	0
9 M19	Z	-143	-143	0	0
10 M18	Z	-143	-143	0	0
11 M25	Z	-143	-143	0	0
12 M9	Z	-143	-143	0	0
13 M10	Z	-143	-143	0	0
14 M4	Z	-07	-07	0	0
15 M2	Z	-07	-07	0	0
16 M1	Z	-07	-07	0	0
17 M3	Z	-07	-07	0	0
18 M13	Z	-07	-07	0	0
19 M11	Z	-07	-07	0	0
20 M26	Z	-07	-07	0	0

Basic Load Cases

BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed Area(Me...	Surface(P...
1 D	DL						22	
2 L	LL						23	
3 WX	WLX						20	
4 WY	WLY							
5 EX	ELX							
6 EY	ELY				2			

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Oct 21, 2017
8:13 AM
Checked By: _____

Company : HCP ENGINEERING
Designer :
Job Number :
Model Name :
IRISA

Oct 21, 2017
8:13 AM
Checked By: _____

Company : HCP ENGINEERING
Designer :
Job Number :
Model Name :
IRISA

Envelope Joint Displacements (Continued)

Joint	Member	Sec	X [in]	Y [in]	Z [in]	LC X Rotation	LC Y Rotation	LC Z Rotation	LC
50	M1	1	min	0	15	0	13	-3.067e-05	13
51	N27	3	max	0	15	0	13	-3.131e-04	15
52	M2	1	min	0	15	0	13	-3.216e-06	13
53	N28	4	max	0	15	0	15	-3.042e-05	15
54	M3	5	min	0	19	0	13	-2.254e-03	13
55	N29	6	max	0	19	0	13	-1.354e-04	13
56	M4	7	min	0	20	0	13	-2.044e-03	13
57	N30	8	max	0	20	0	13	-8.385e-05	13
58	M5	9	min	0	20	0	13	-6.391e-05	13
59	N33	10	max	0	19	0	13	-7.726e-04	13
60	M6	11	min	0	19	0	13	-5.446e-02	13
61	N34	12	max	0	19	0	13	-7.939e-04	13
62	M7	13	min	0	20	0	13	-5.532e-02	13

Envelope AISC 14th(360-10): ASD Steel Code Checks

Member	Shape	Code Check	Lo...LC Shear...Lo...Dir	LC	Pchlo...Prnto...Mnryl...Mnzzl...Cb	Egn
1	M1	W12x65	4.5 13 080 4.5	V	553.6 57.18 106.9 237.0 1.7	H1-1b
2	M2	W12x65	10.5 13 043 7.2	V	533.44 57.18 106.9 237.0 1.7	H1-1b
3	M3	W12x65	0 13 127 1.5	V	563.0 57.18 106.9 237.0 1.6	H1-1b
4	M4	W12x65	12 13 040 24	V	289.9 57.18 106.9 228.5 1.1	H1-1b
5	M5	W12x65	0 13 009 0	V	418.44 43.71 53.144 179.3 1.7	H1-1b
6	M6	W12x65	0 13 070 0	V	418.44 43.71 53.144 179.3 1.6	H1-1b
7	M7	W12x22	10 13 043 21	V	10.772 194.0 9.132 16 22.11 1.1	H1-1b
8	M8	W12x22	10 13 043 21	V	10.772 194.0 9.132 16 22.11 1.1	H1-1b
9	M9	W12x50	23 13 028 23	V	11.10 437.1 53.144 179.3 1.7	H1-1b
10	M10	W12x50	0 13 038 0	V	374.8 437.1 53.144 179.3 1.7	H1-1b
11	M11	W12x50	5.88 13 026 0	V	266.9 437.1 53.144 176.73 1.1	H1-1b
12	M12	W12x50	13 13 052 6.72	V	69.22 844.3 168.4 366.7 1.5	H1-1b
13	M13	W12x50	5.0 13 327 22	V	475.0 844.3 168.4 366.7 1.1	H1-1b
14	M14	W12x22	2.1 3 007 4.25	V	3 146.0 844.3 9.132 39.885 1.1	H1-1b
15	M15	W12x22	5.35 13 021 0	V	42 254 157.4 1 627 42 415 2.1	H1-1a
16	M16	W8x18	4 13 012 7.0	V	42 254 157.4 1 627 42 415 2.1	H1-1a
17	M17	W8x18	24 13 014 7.0	V	42 254 157.4 1 627 42 415 2.1	H1-1a
18	M18	W12x22	4 13 010 0	V	135.8 194.0 9.132 73 104 1.1	H1-1b
19	M19	W12x22	4.8 13 020 9.75	V	51 168 194.0 9.132 45 805 1.1	H1-1b
20	M20	W12x22	8.0 13 032 0	Y	13 18.874 194.0 9.132 22 887 1.1	H1-1b
21	M21	W12x22	10 13 043 0	Y	10.772 194.0 9.132 16 22.11 1.1	H1-1b
22	M22	W12x22	5.3 13 021 10	Y	43 087 194.0 9.132 40 297 1.1	H1-1b
23	M23	W12x22	4.1 13 190 0	Z	11.45 194.0 9.132 55 224 1.1	H1-1b
24	M24	W12x22	2.07 13 177 0	Z	13 147.7 194.0 9.132 73 104 1.1	H1-1b
25	M25	W12x22	0.90 13 190 0	Z	13 147.7 194.0 9.132 73 104 1.1	H1-1b
26	M26	W12x22	0.08 13 177 0	Z	13 147.7 194.0 9.132 73 104 1.1	H1-1b

Envelope Member Section Forces

Member	Sec	Axis [k]	LC	Y Shear [k]	LC	Torque [k-ft]	LC	Y-y Moment	LC	Z-z Moment	LC
1	M1	1	max	971	20	503	15	0	1	0	1
2	M2	1	min	-1037	19	-6524	13	0	1	0	1
3	M3	2	max	971	20	476	15	0	1	0	1
4	M4	3	min	-1037	19	-6776	13	0	1	0	1
5	M5	4	max	971	20	449	15	0	1	0	1
6	M6	5	min	-1037	19	-7028	13	0	1	0	1
7	M7	6	max	971	20	422	15	0	1	0	1
8	M8	7	min	-1037	19	-728	13	0	1	0	1
9	M9	8	max	971	20	395	15	0	1	0	1
10	M10	9	min	-1037	19	-7532	13	0	1	0	1
11	M11	10	max	971	20	1714	13	0	1	0	1
12	M12	11	min	-1147	20	-161	15	0	1	0	1
13	M13	12	max	971	20	1126	13	0	1	0	1

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Envelope Member Section Forces (Continued)

Member	Sec	Axis [k]	LC	Y Shear [k]	LC	Z Shear [k]	LC	Torque [k-ft]	LC	Y-y Moment	LC	Z-z Moment	LC
14	M14	1	max	-1147	20	-224	15	-455	13	-158	19	-1293	15
15	M15	2	min	71	19	-287	13	-507	13	-365	13	-21714	13
16	M16	3	max	-1147	20	-2192	13	-197	20	-485	13	-5621	15
17	M17	4	min	71	19	-14	15	-507	13	-314	19	-2824	13
18	M18	5	max	-1147	20	-278	13	-197	20	-455	13	-337	15
19	M19	6	min	71	19	-203	15	-507	13	-464	19	-3631	15
20	M20	7	max	-1147	20	-3368	13	-197	20	-455	13	-454	15
21	M21	8	min	2878	19	10985	13	-277	20	-329	19	-412	15
22	M22	9	max	-2846	20	084	15	-09	19	-2519	13	-1015	15
23	M23	10	min	2878	19	1078	13	-277	20	-036	15	-247	15
24	M24	11	max	-2846	20	032	15	-09	19	-2519	13	-761	15
25	M25	12	min	2878	19	10574	13	-277	20	-036	15	-164	15
26	M26	13	max	-2846	20	01	15	-09	19	-2519	13	-2142	15
27	M27	14	min	2878	19	10369	13	-277	20	-036	15	-082	15
28	M28	15	max	-2846	20	-012	15	-09	19	-2519	13	-254	15
29	M29	16	min	2878	19	10163	13	-277	20	-036	15	0	1
30	M30	17	max	-2846	20	-034	15	-09	19	-2519	13	0	1
31	M31	18	min	2846	20	3795	13	011	20	0	1	0	1
32	M32	19	max	-2177	19	-293	15	-012	19	0	1	0	1
33	M33	20	min	2177	19	239	13	011	20	0	1	0	1
34	M34	21	max	-2177	19	-293	15	-012	19	0	1	0	1
35	M35	22	min	2177	19	239	13	011	20	0	1	0	1
36	M36	23	max	-2177	19	-293	15	-012	19	0	1	0	1
37	M37	24	min	2177	19	239	13	011	20	0	1	0	1
38	M38	25	max	-2134	19	-238	15	-012	19	0	1	0	1
39	M39	26	min	1999	20	293	13	011	20	0	1	0	1
40	M40	27	max	-2134	19	-238	15	-012	19	0	1	0	1
41	M41	28	min	1999	20	293	13	011	20	0	1	0	1
42	M42	29	max	-2134	19	-238	15	-012	19	0	1	0	1
43	M43	30	min	1999	20	293	13	011	20	0	1	0	1
44	M44	31	max	-2134	19	-238	15	-012	19	0	1	0	1
45	M45	32	min	1999	20	293	13	011	20	0	1	0	1
46	M46	33	max	-2134	19	-238	15	-012	19	0	1	0	1
47	M47	34	min	1999	20	293	13	011	20	0	1	0	1
48	M48	35	max	-2134	19	-238	15	-012	19	0	1	0	1
49	M49	36	min	1999	20	293	13	011	20	0	1	0	1
50	M50	37	max	-2134	19	-238	15	-012	19	0	1	0	1
51	M51	38	min	1999	20	293	13	011	20	0	1	0	1
52	M52	39	max	-2134	19	-238	15	-012	19	0	1	0	1
53	M53	40	min	1999	20	293	13	011	20	0	1	0	1
54	M54	41	max	-2134	19	-238	15	-012	19	0	1	0	1
55	M55	42	min	1999	20	293	13	011	20	0	1	0	1
56	M56	43	max	-2134	19	-238	15	-012	19	0	1	0	1
57	M57	44	min	1999	20	293	13	011	20	0	1	0	1
58	M58	45	max	-2134	19	-238	15	-012	19	0	1	0	1
59	M59	46	min	1999	20	293	13	011	20	0	1	0	1
60	M60	47	max	-2134	19	-238	15	-012	19	0	1	0	1
61	M61	48	min	1999	20	293	13	011	20	0	1	0	1
62	M62	49	max	-2134	19	-238	15	-012	19	0	1	0	1
63	M63	50	min	1999	20	293	13	011	20	0	1	0	1
64	M64	51	max	-2134	19	-238	15	-012	19	0	1	0	1
65	M65	52	min	1999	20	293	13	011	20	0	1	0	1
66	M66	53	max	-2134	19	-238	15	-012	19	0	1	0	1
67	M67	54	min	1999	20	293	13	011	20	0	1	0	1
68	M68	55	max	-2134	19	-238	15	-012	19	0	1	0	1
69	M69	56	min	1999	20	293	13	011	20	0	1	0	1
70	M70	57	max	-2134	19	-238	15	-012	19	0	1	0	1
71	M71	58	min	1999	20	293	13	011	20	0	1	0	1
72	M72	59	max	-2134	19	-238	15	-012	19	0	1	0	1
73	M73	60	min	1999	20	293	13	011	20	0	1	0	1
74	M74	61	max	-2134	19	-238	15	-012	19	0	1	0	1

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Envelope Member Section Forces (Continued)

Member	Sec	Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	yy Moment	LC	zz Moment	LC
75	3	max	-0.08	15	0	1	0	1	0	1	0	1	1.118
76	3	min	-1.007	13	0	1	0	1	0	1	0	1	-14.498
77	4	max	-0.08	15	0	1	0	1	0	1	0	1	1.118
78	4	min	-1.007	13	0	1	0	1	0	1	0	1	-14.498
79	5	max	-0.08	15	0	1	0	1	0	1	0	1	1.118
80	5	min	-1.007	13	0	1	0	1	0	1	0	1	-14.498
81	1	max	0	20	624	13	0	19	0	1	0	1	0
82	1	min	0	19	-0.92	15	0	20	0	1	0	1	0
83	2	max	0	20	-0.54	16	0	19	0	1	0	1	0
84	2	min	0	19	-1.86	3	0	20	0	1	0	1	0
85	3	max	0	20	-0.23	15	0	19	0	1	0	1	0
86	3	min	0	19	-0.94	13	0	20	0	1	0	1	0
87	4	max	0	19	-1.73	13	0	19	0	1	0	1	0
88	4	min	0	20	0.12	15	0	20	0	1	0	1	0
89	5	max	0	19	-0.46	15	0	19	0	1	0	1	0
90	5	min	0	20	2.516	13	0	20	0	1	0	1	0
91	1	max	0	20	3.413	13	0	19	0	1	0	1	0
92	1	min	0	19	0.48	15	0	20	0	1	0	1	0
93	2	max	0	20	3.157	13	0	19	0	1	0	1	0
94	2	min	0	19	0.89	15	0	20	0	1	0	1	0
95	3	max	0	20	2.902	13	0	19	0	1	0	1	0
96	3	min	0	19	0.71	15	0	20	0	1	0	1	0
97	4	max	0	20	2.846	13	0	19	0	1	0	1	0
98	4	min	0	19	0.62	15	0	20	0	1	0	1	0
99	5	max	0	20	2.39	13	0	19	0	1	0	1	0
100	5	min	0	19	0.93	15	0	20	0	1	0	1	0
101	1	max	-0.02	19	0.93	15	0	19	0	1	0	1	0
102	1	min	-0.02	20	0.93	15	0	20	0	1	0	1	0
103	2	max	-0.02	19	1.637	13	0	19	0	1	0	1	0
104	2	min	-0.02	20	0.12	15	0	20	0	1	0	1	0
105	3	max	-0.03	19	0.13	15	0	19	0	1	0	1	0
106	3	min	-0.03	20	-1.75	13	0	20	0	1	0	1	0
107	4	max	-0.02	19	-0.26	15	0	19	0	1	0	1	0
108	4	min	-0.02	20	-1.46	13	0	20	0	1	0	1	0
109	5	max	-0.02	19	-1.07	15	0	19	0	1	0	1	0
110	5	min	-0.02	20	-2.212	13	0	20	0	1	0	1	0
111	1	max	4.184	19	5.747	13	0	19	0	1	0	1	0
112	1	min	-4.137	20	0.29	15	0	20	0	1	0	1	0
113	2	max	4.184	19	5.747	13	0	19	0	1	0	1	0
114	2	min	-4.137	20	0.29	15	0	20	0	1	0	1	0
115	3	max	4.184	19	5.747	13	0	19	0	1	0	1	0
116	3	min	-4.137	20	0.18	15	0	20	0	1	0	1	0
117	4	max	4.159	19	4.589	13	0	19	0	1	0	1	0
118	4	min	-4.112	20	0.18	15	0	20	0	1	0	1	0
119	5	max	4.159	19	4.589	13	0	19	0	1	0	1	0
120	5	min	-4.112	20	0.18	15	0	20	0	1	0	1	0
121	1	max	3.973	19	1.124	13	0	19	0	1	0	1	0
122	1	min	-3.929	20	0.108	15	0	20	0	1	0	1	0
123	2	max	3.973	19	1.124	13	0	19	0	1	0	1	0
124	2	min	-3.929	20	0.108	15	0	20	0	1	0	1	0
125	3	max	3.973	19	1.124	13	0	19	0	1	0	1	0
126	3	min	-3.929	20	0.108	15	0	20	0	1	0	1	0
127	4	max	3.907	19	1.124	13	0	19	0	1	0	1	0
128	4	min	-3.867	20	0.154	15	0	20	0	1	0	1	0
129	5	max	3.907	19	1.124	13	0	19	0	1	0	1	0
130	5	min	-3.867	20	0.154	15	0	20	0	1	0	1	0
131	1	max	0	1	4.67	3	0	19	0	1	0	1	0
132	1	min	0	1	14	16	0	20	0	1	0	1	0
133	2	max	0	1	234	3	0	19	0	1	0	1	0
134	2	min	0	1	0.7	16	0	20	0	1	0	1	0
135	3	max	0	1	0	1	0	19	0	1	0	1	0

Envelope Member Section Forces (Continued)

Member	Sec	Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	yy Moment	LC	zz Moment	LC	
136	3	min	0	1	0	1	0	19	0	1	-0.01	19	-0.497	
	4	max	0	1	-0.7	15	0	20	0	1	-0.02	20	-1.12	
	5	min	0	1	-2.34	3	0	19	0	1	-0.02	19	-3.73	
	137	3	min	0	1	-1.4	15	0	20	0	1	-0.03	20	0
	140	5	max	0	1	-1.468	3	0	19	0	1	-0.03	19	0
M15	141	1	min	0.88	19	1.374	13	0	19	0	1	-0.03	19	0
	142	2	min	-0.79	20	-1.06	15	0	20	0	1	-0.03	20	0
	143	3	min	0.88	19	1.374	13	0	19	0	1	-0.02	19	15
	144	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.02	20	-2.15
	145	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.02	20	-2.15
M16	146	3	min	0.88	19	1.374	13	0	19	0	1	-0.01	19	15
	147	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.01	20	-3.67
	148	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.01	20	-3.67
	149	3	min	0.88	19	1.374	13	0	19	0	1	-0.01	19	15
	150	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.01	20	-3.67
M17	151	1	min	0.88	19	1.374	13	0	19	0	1	0	1	0
	152	2	min	-0.79	20	-0.53	15	0	20	0	1	0	1	0
	153	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	154	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.224	13	-1.08
	155	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.224	13	-1.08
M18	156	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	157	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	158	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	159	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	160	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
M19	161	1	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	162	2	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	163	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	164	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	165	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
M20	166	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	167	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	168	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	169	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	170	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
M21	171	1	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	172	2	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	173	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	174	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	175	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
M22	176	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	177	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	178	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	179	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	180	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
M23	181	1	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	182	2	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	183	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	184	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	185	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
M24	186	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	187	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	188	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	189	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	190	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
M25	191	1	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	192	2	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	193	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	194	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	195	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
M26	196	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	197	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	198	5	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961
	199	3	min	0.88	19	1.374	13	0	19	0	1	-0.04	15	1.961
	200	4	min	-0.79	20	-0.53	15	0	20	0	1	-0.04	15	1.961

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Envelope Member Section Forces (Continued)

Member	Sec	Axial[k]	LC	v Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	yy Moment...	LC	zz Moment...	LC
197	4	max	-169	19	-0.79	15	0	1	0	1	0	1	47.6
	4	min	-184	19	-1.031	13	0	1	0	1	0	1	-6.206
	5	max	-169	20	-1.59	15	0	1	0	1	0	1	0
	5	min	-184	19	-2.062	13	0	1	0	1	0	1	0
200	1	max	2.343	20	2.729	13	0	1	0	1	0	1	0
	1	min	-2.555	19	-2.1	15	0	1	0	1	0	1	0
	2	max	2.343	20	1.365	13	0	1	0	1	0	1	838
	2	min	-2.555	19	-1.05	15	0	1	0	1	0	1	-10.873
205	3	max	2.343	20	0	1	0	1	0	1	0	1	1.118
	3	min	-2.555	19	0	1	0	1	0	1	0	1	-14.498
	4	max	2.343	20	1.05	15	0	1	0	1	0	1	838
	4	min	-2.555	19	-1.365	13	0	1	0	1	0	1	-10.873
209	5	max	2.343	20	2.1	15	0	1	0	1	0	1	0
	5	min	-2.555	19	-2.729	13	0	1	0	1	0	1	0
	1	max	0.05	15	2.729	13	0	1	0	1	0	1	0
	1	min	-2.71	13	-2.1	15	0	1	0	1	0	1	0
212	2	max	0.05	15	1.365	13	0	1	0	1	0	1	838
	2	min	-2.71	13	-1.05	15	0	1	0	1	0	1	-10.873
	3	max	0.05	15	0	1	0	1	0	1	0	1	1.118
	3	min	-2.71	13	0	1	0	1	0	1	0	1	-14.498
216	4	max	0.05	15	1.05	15	0	1	0	1	0	1	838
	4	min	-2.71	13	-1.365	13	0	1	0	1	0	1	-10.873
	5	max	0.05	15	2.1	15	0	1	0	1	0	1	0
	5	min	-2.71	13	-2.729	13	0	1	0	1	0	1	0
220	1	max	1.729	19	2.729	13	0	1	0	1	0	1	0
	1	min	-1.707	20	-2.1	15	0	1	0	1	0	1	0
	2	max	1.729	19	1.365	13	0	1	0	1	0	1	838
	2	min	-1.707	20	-1.05	15	0	1	0	1	0	1	-10.873
225	3	max	1.729	19	0	1	0	1	0	1	0	1	1.118
	3	min	-1.707	20	0	1	0	1	0	1	0	1	-14.498
	4	max	1.729	19	1.05	15	0	1	0	1	0	1	838
	4	min	-1.707	20	-1.365	13	0	1	0	1	0	1	-10.873
228	5	max	1.729	19	2.1	15	0	1	0	1	0	1	0
	5	min	-1.707	20	-2.729	13	0	1	0	1	0	1	0
	1	max	0.48	19	1.365	13	0	1	0	1	0	1	838
	1	min	-0.47	20	-1.05	15	0	1	0	1	0	1	-10.873
232	2	max	0.48	19	0.682	13	0	1	0	1	0	1	21
	2	min	-0.47	20	-0.53	15	0	1	0	1	0	1	-2.718
	3	max	0.48	19	0	1	0	1	0	1	0	1	2.718
	3	min	-0.47	20	0	1	0	1	0	1	0	1	-3.624
236	4	max	0.48	19	0.53	15	0	1	0	1	0	1	21
	4	min	-0.47	20	-0.82	13	0	1	0	1	0	1	-2.718
	5	max	0.48	19	1.05	15	0	1	0	1	0	1	168
	5	min	-0.47	20	-1.365	13	0	1	0	1	0	1	-2.186
240	1	max	0.04	19	1.06	13	0	1	0	1	0	1	126
	1	min	-0.04	20	-0.82	15	0	1	0	1	0	1	-1.639
	2	max	0.04	19	0.53	13	0	1	0	1	0	1	168
	2	min	-0.04	20	-0.41	15	0	1	0	1	0	1	-2.186
243	3	max	0.04	19	0	1	0	1	0	1	0	1	126
	3	min	-0.04	20	0	1	0	1	0	1	0	1	-1.639
	4	max	0.04	19	0.41	15	0	1	0	1	0	1	0
	4	min	-0.04	20	-0.53	13	0	1	0	1	0	1	0
249	5	max	0.04	19	0.62	13	0	1	0	1	0	1	0
	5	min	-0.04	20	-1.06	15	0	1	0	1	0	1	0
	1	max	0.05	20	0.532	13	0	1	0	1	0	1	0.32
	1	min	-0.05	19	-0.41	15	0	1	0	1	0	1	-4.13
252	2	max	0.05	20	2.66	13	0	1	0	1	0	1	0.42
	2	min	-0.05	19	-0.2	15	0	1	0	1	0	1	-1.83
	3	max	0.05	20	0	1	0	1	0	1	0	1	0
	3	min	-0.05	19	0	1	0	1	0	1	0	1	0
256	4	max	0.05	20	0.2	15	0	1	0	1	0	1	0.32
	4	min	-0.05	19	0	1	0	1	0	1	0	1	0
	5	max	0.05	20	0.2	15	0	1	0	1	0	1	0.32
	5	min	-0.05	19	0	1	0	1	0	1	0	1	0

Envelope Member Section Forces (Continued)

Member	Sec	Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	xx Rotate [r]	LC	yy Moment...	LC	zz Moment...	LC	
258	5	min	-0.05	13	-266	13	0	1	-183	13	0	1	-413	13	1	
259		max	0.05	20	-0.41	15	0	1	-0.03	15	0	1	0	1	1	
260		min	-0.05	19	-532	13	0	1	-183	13	0	1	0	1	1	
		max	0.05	20	-0.41	15	0	1	-0.03	15	0	1	0	1	1	
Envelope Member Section Deflections																
Member	Sec	x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [r]	LC	yy Moment...	LC	zz Moment...	LC	xx Rotate [r]	LC	
M1	1	max	-0.08	19	-0.08	15	-0.03	20	1.264e-03	13	7893.047	15	NC	1	1	
	2	min	-0.08	20	-132	13	-0.03	19	1.688e-05	15	468.834	13	NC	1	1	
	3	max	0.08	19	0.06	15	0.02	20	1.264e-03	13	NC	15	NC	1	1	
	4	min	-0.08	20	-0.02	13	-0.02	19	1.688e-05	15	647.657	13	NC	1	1	
	5	max	0.08	19	0.04	15	0.02	20	1.264e-03	13	NC	15	NC	1	1	
	6	min	-0.08	20	-0.09	13	-0.01	19	1.688e-05	15	1030.344	13	NC	1	1	
	7	4	max	0.08	19	0.02	15	0.01	20	1.264e-03	13	NC	7	NC	1	1
	8	min	-0.08	20	-0.41	13	0	19	1.688e-05	15	2236.701	13	NC	1	1	
	9	5	max	0.08	19	0.01	15	0.01	13	1.264e-03	13	NC	1	NC	1	1
	10	min	-0.08	20	-0.17	13	0	15	1.688e-05	15	NC	1	NC	1	1	
	11	1	max	0.08	19	0.01	15	0.01	13	1.264e-03	13	NC	1	NC	1	1
	12	2	min	-0.08	20	-0.17	13	0	15	1.688e-05	15	NC	7	NC	1	1
M2	13	3	max	0.08	19	0.15	13	0	13	2.367e-03	13	NC	1	NC	1	1
	14	min	-0.08	20	0	15	0	15	3.259e-05	15	3713.406	13	NC	1	1	
	15	3	max	0.08	19	0.24	13	0	13	3.471e-03	13	NC	7	NC	1	1
	16	min	-0.08	20	0	15	0	15	4.85e-05	15	2817.088	13	NC	1	1	
	17	4	max	0.08	19	0.14	13	0.01	13	4.574e-03	13	NC	7	NC	1	1
	18	min	-0.08	20	-0.24	13	0	15	5.644e-05	15	3497.867	13	NC	1	1	
	19	5	max	0.08	19	0	15	0.02	13	5.677e-03	13	NC	1	NC	1	1
	20	min	-0.08	20	-0.24	13	0	15	8.033e-05	15	NC	1	NC	1	1	
	21	1	max	0.08	19	0	15	0.02	13	5.677e-03	13	NC	1	NC	1	1
	22	2	min	-0.08	20	-0.24	13	0	15	8.033e-05	15	NC	1	NC	1	1
	23	3	max	0.08	19	0	15	0.02	19	9.99e-03	13	NC	7	NC	1	1
	M3	24	min	-0.08	20	-0.48	13	0	20	8.477e-05	15	1813.751	13	NC	1	1
25		3	max	0.08	19	0	15	0.03	19	6.302e-03	13	NC	15	NC	1	1
26		min	-0.08	20	-0.76	13	-0.01	20	8.921e-05	15	843.092	13	NC	1	1	
27		4	max	0.08	19	0	15	0.04	19	6.14e-03	13	NC	15	NC	1	1
28		min	-0.08	20	-107	13	-0.03	20	3.66e-05	15	532.444	13	NC	1	1	
29		5	max	0.08	19	0	15	0.04	19	9.322e-03	13	NC	15	NC	1	1
30		min	-0.08	20	-138	13	-0.04	20	9.81e-05	15	384.357	13	NC	1	1	
31		1	max	0	1	0	1	0	1	2.585e-03	13	NC	1	NC	1	1
32		2	min	0	1	0	1	0	1	1.108e-04	15	NC	1	NC	1	1
33		3	max	0	19	0.14	15	0	20	2.585e-03	13	NC	7	NC	1	1
34		min	0	20	-191	13	0	19	1.108e-04	15	1843.796	13	NC	1	1	
M4		35	3	max	0	19	0.21	13	0.01	20	2.585e-03	13	NC	7	NC	1
	36	min	0	20	-292	13	-0.01	19	1.108e-04	15	1288.764	13	NC	1	1	
	37	4	max	0.01	19	0.18	15	0.04	20	2.585e-03	13	NC	7	NC	1	1
	38	min	0	20	-257	13	-0.04	19	1.108e-04	15	1843.796	13	NC	1	1	
	39	5	max	0.01	19	0.08	15	0.08	20	2.385e-03	13	NC	1	NC	1	1
	40	min	-0.01	20	-132	13	-0.09	19	1.108e-04	15	NC	1	NC	1	1	
	41	1	max	0	1	0	1	0	1	0	1	NC	1	NC	1	1
	42	2	min	0	1	0	1	0	1	0	1	NC	1	NC	1	1
	43	3	max	0	1	0	15	0	19	0	1	NC	1	NC	1	1
	44	min	0	1	0	13	0	20	0	1	1	NC	1	NC	1	1
	45	3	max	0	20	0	15	0	19	0	1	NC	1	NC	1	1
	M5	46	min	0	19	-0.01	13	0	20	0	1	NC	1	NC	1	1
47		4	max	0	20	0	15	0	19	0	1	NC	1	NC	1	1
48		min	0	19	-0.02	13	0	20	0	1	NC	1	NC	1	1	
49		5	max	0	20	0	15	0	19	0	1	NC	1	NC	1	1
50		min	0	19	-0.04	13	0	20	0	1	NC	1	NC	1	1	
51		1	max	0	1	0	1	0	1	0	1	NC	1	NC	1	1
52		2	min	0	1	0	1	0	1	0	1	NC	1	NC	1	1
53		3	max	0	19	0	15	0	19	0	1	NC	1	NC	1	1
54		min	0	20	0	13	0	20	0	1	1	NC	1	NC	1	1
55		3	max	0	20	0	15	0	19	0	1	NC	1	NC	1	1
56		min	0	19	-0.01	13	0	20	0	1	1	NC	1	NC	1	1
57		4	max	0	20	0	15	0	19	0	1	NC	1	NC	1	1

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Company : HCP ENGINEERING
Designer :
Job Number :
Model Name :
IRISA

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Company : HCP ENGINEERING
Designer :
Job Number :
Model Name :
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Envelope Member Section Deflections (Continued)

Member	Sec	x [in]	y [in]	z [in]	LC x Rotate [r]	LC	LVRa... LC (m) Lz/Ra... LC
115	3	max	0	20	-0.012	15	NC
116	3	min	0	19	-0.012	15	NC
117	4	max	0	20	-0.018	15	NC
118	4	min	0	19	-0.018	15	NC
119	5	max	0.001	20	-0.023	15	NC
120	5	min	0.001	19	-0.023	15	NC
121	1	max	0.001	20	-0.023	15	NC
122	1	min	0.001	19	-0.023	15	NC
123	2	max	0.001	20	-0.022	15	NC
124	2	min	0.001	19	-0.022	15	NC
125	3	max	0.002	20	-0.017	15	NC
126	3	min	0.002	19	-0.017	15	NC
127	4	max	0.002	20	-0.009	15	NC
128	4	min	0.002	19	-0.009	15	NC
129	5	max	0.003	20	-0.009	15	NC
130	5	min	0.003	19	-0.009	15	NC
131	1	max	0	1	0	1	NC
132	1	min	0	1	0	1	NC
133	2	max	0	1	0	1	NC
134	2	min	0	1	0	1	NC
135	3	max	0	1	0	1	NC
136	3	min	0	1	0	1	NC
137	4	max	0	1	0	1	NC
138	4	min	0	1	0	1	NC
139	5	max	0	1	0	1	NC
140	5	min	0	1	0	1	NC
141	1	max	0	1	0	1	NC
142	1	min	0	1	0	1	NC
143	2	max	0	20	-0.004	15	NC
144	2	min	0	19	-0.004	15	NC
145	3	max	0	20	-0.003	15	NC
146	3	min	0	19	-0.003	15	NC
147	4	max	0	20	-0.014	15	NC
148	4	min	0	19	-0.014	15	NC
149	5	max	0	20	-0.021	15	NC
150	5	min	0	19	-0.021	15	NC
151	1	max	0	1	0	1	NC
152	1	min	0	1	0	1	NC
153	2	max	0	15	0	15	NC
154	2	min	0	15	0	15	NC
155	3	max	0	15	-0.012	13	NC
156	3	min	0	15	-0.012	13	NC
157	4	max	0	15	-0.03	13	NC
158	4	min	0	15	-0.03	13	NC
159	5	max	0.001	15	-0.008	19	NC
160	5	min	0.001	15	-0.008	19	NC
161	1	max	0	1	0	1	NC
162	1	min	0	1	0	1	NC
163	2	max	0	15	-0.014	13	NC
164	2	min	0	15	-0.014	13	NC
165	3	max	0	15	-0.037	13	NC
166	3	min	0	15	-0.037	13	NC
167	4	max	0	15	-0.041	13	NC
168	4	min	0	15	-0.041	13	NC
169	5	max	0	15	-0.008	20	NC
170	5	min	0	15	-0.008	20	NC
171	1	max	0	1	0	1	NC
172	1	min	0	1	0	1	NC
173	2	max	0	19	-0.003	15	NC
174	2	min	0	20	-0.003	15	NC
175	3	max	0	19	-0.006	15	NC

2017-06
(122)

Envelope Member Section Deflections (Continued)

Member	Sec	x [in]	y [in]	z [in]	LC x Rotate [r]	LC	LVRa... LC (m) Lz/Ra... LC
54	3	max	0	20	-0.004	13	NC
55	3	min	0	19	-0.004	13	NC
56	4	max	0	20	-0.011	13	NC
57	4	min	0	19	-0.011	13	NC
58	5	max	0	20	-0.021	13	NC
59	5	min	0	19	-0.021	13	NC
60	1	max	0	20	-0.031	13	NC
61	1	min	0	19	-0.031	13	NC
62	2	max	0	13	0.018	15	NC
63	2	min	0	13	-0.018	15	NC
64	3	max	0	13	0.026	15	NC
65	3	min	0	13	-0.026	15	NC
66	4	max	0	13	0.019	15	NC
67	4	min	0	13	-0.019	15	NC
68	5	max	0.001	13	0.001	15	NC
69	5	min	0.001	13	-0.001	15	NC
70	1	max	0	1	0	1	NC
71	1	min	0	1	0	1	NC
72	2	max	0	13	0.018	15	NC
73	2	min	0	13	-0.018	15	NC
74	3	max	0	13	0.025	15	NC
75	3	min	0	13	-0.025	15	NC
76	4	max	0.001	13	0.006	19	NC
77	4	min	0.001	13	-0.006	19	NC
78	5	max	0	13	-0.024	13	NC
79	5	min	0	13	-0.024	13	NC
80	1	max	0	1	0	1	NC
81	1	min	0	1	0	1	NC
82	2	max	0	20	-0.004	13	NC
83	2	min	0	19	-0.004	13	NC
84	3	max	0	20	-0.011	13	NC
85	3	min	0	19	-0.011	13	NC
86	4	max	0	20	-0.028	13	NC
87	4	min	0	19	-0.028	13	NC
88	5	max	0	20	-0.006	15	NC
89	5	min	0	19	-0.006	15	NC
90	1	max	0	20	-0.031	13	NC
91	1	min	0	19	-0.031	13	NC
92	2	max	0	20	-0.013	13	NC
93	2	min	0	19	-0.013	13	NC
94	3	max	0	20	-0.077	13	NC
95	3	min	0	19	-0.077	13	NC
96	4	max	0	20	-0.132	13	NC
97	4	min	0	19	-0.132	13	NC
98	5	max	0	20	-0.006	15	NC
99	5	min	0	19	-0.006	15	NC
100	1	max	0	20	-0.047	13	NC
101	1	min	0	19	-0.047	13	NC
102	2	max	0	20	-0.016	13	NC
103	2	min	0	19	-0.016	13	NC
104	3	max	0	20	-0.044	13	NC
105	3	min	0	19	-0.044	13	NC
106	4	max	0	20	-0.012	13	NC
107	4	min	0	19	-0.012	13	NC
108	5	max	0	20	-0.021	13	NC
109	5	min	0	19	-0.021	13	NC
110	1	max	0	1	0	1	NC
111	1	min	0	1	0	1	NC
112	2	max	0	20	-0.006	15	NC
113	2	min	0	19	-0.006	15	NC
114	3	max	0	19	-0.012	13	NC

2017-06
(123)

Oct 21, 2017
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Company : HOP ENGINEERING
Designer :
Job Number :
Model Name :



Envelope Member Section Deflections (Continued)

Member	Sec	x [in]	y [in]	z [in]	LC	LC x Rotate [r]	LC	(n) L/Ra..	(n) L/Ra..	
237	4	max	0	20	.017	15	.001	20	2.046e-03	
238		min	0	19	-.233	13	-.001	19	1.835e-06	
239	5	max	0	20	.021	15	.001	20	2.046e-03	
240		min	0	19	-.232	13	-.002	19	3.355e-05	
241	M25	1	max	0	1	0	1	0	0	
242		min	0	1	0	1	0	1	0	
243	2	max	0	20	-.004	15	0	20	2.493e-04	
244		min	0	19	-.196	13	0	19	1.784e-02	
245	3	max	0	20	-.008	15	0	20	4.955e-04	
246		min	0	19	-.389	13	0	19	3.555e-02	
247	4	max	0	20	-.012	15	.001	20	7.475e-04	
248		min	0	19	-.578	13	-.001	19	5.353e-02	
249	5	max	0	20	-.017	15	.002	20	9.975e-04	
250		min	0	19	-.764	13	-.002	19	7.137e-02	
251	M26	1	max	0	20	-.012	15	0	20	1.554e-03
252		min	0	19	-.924	13	0	19	1.115e-01	
253	2	max	0	20	-.014	15	0	20	1.447e-03	
254		min	0	19	-.1006	13	0	19	1.032e-01	
255	3	max	0	20	-.016	15	.001	20	1.331e-03	
256		min	0	19	-.1088	13	-.001	19	9.485e-02	
257	4	max	0	20	-.019	15	.001	20	1.214e-03	
258		min	0	19	-.169	13	-.001	19	8.655e-02	
259	5	max	0	20	-.021	15	.002	20	1.095e-03	
260		min	0	19	-.1251	13	-.002	19	7.821e-02	

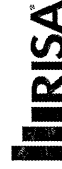
Material Takeoff

Material	Size	Pieces	Length [ft]	Weight [K]
1	Hot Rolled Steel			
2	A992-Shapes: W's	W12x22	13	174.9
3	A992-Shapes: W's	W12x50	5	51.9
4	A992-Shapes: W's	W12x45	4	42.9
5	A992-Shapes: W's	W12x46	2	36.3
6	A992-Shapes: W's	W8x18	2	28.1
7	Total HR Steel		26	334.1

2017-06
(123)

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Company : HOP ENGINEERING
Designer :
Job Number :
Model Name :



Envelope Member Section Deflections (Continued)

Envelope member												
Member	Sec	x [in]	y [in]	z [in]	LC	LC x Rotate [r]	LC	(n) L/Ra..	LC	(n) L/Ra..	LC	
176		min	0	20	-.463	13	0	19	1.049e-01	13	NC	
177	4	max	0	19	-.009	15	0	20	-1.471e-03	15	NC	
178		min	0	20	-.694	13	0	19	-1.049e-01	13	NC	
179	5	max	0	19	-.012	15	0	20	-1.471e-03	15	NC	
180		min	0	20	-.924	13	0	19	-1.049e-01	13	NC	
181	M19	1	max	0	1	0	1	0	0	1	NC	
182		min	0	1	0	1	0	1	0	1	NC	
183	2	max	0	19	-.005	15	0	20	0	1	NC	
184		min	0	20	-.434	13	0	19	0	1	NC	
185	3	max	0	19	-.011	15	0	20	0	1	NC	
186		min	0	20	-.862	13	0	19	0	1	NC	
187	4	max	0	19	-.017	15	.001	20	0	1	NC	
188		min	0	20	-1.282	13	-.001	19	0	1	NC	
189	5	max	0	19	-.023	15	.002	20	0	1	NC	
190		min	0	20	-1.695	13	-.002	19	0	1	NC	
191	M20	1	max	0	1	0	1	0	1	-6.193e-04	15	NC
192		min	0	1	0	1	0	1	-4.479e-02	13	NC	
193	2	max	0	19	.003	15	0	20	-6.193e-04	15	NC	
194		min	0	20	-.309	13	0	19	-4.479e-02	13	NC	
195	3	max	0	19	.002	15	.002	20	-6.193e-04	15	NC	
196		min	0	20	-.574	13	-.002	19	-4.479e-02	13	NC	
197	4	max	0	19	-.003	15	.002	20	-6.193e-04	15	NC	
198		min	0	20	-.777	13	-.002	19	-4.479e-02	13	NC	
199	5	max	0	19	-.012	15	.003	20	-6.193e-04	15	NC	
200		min	0	20	-.935	13	-.003	19	-4.479e-02	13	NC	
201	M21	1	max	0	1	0	1	0	1	1.465e-05	15	NC
202		min	0	1	0	1	0	1	1.465e-05	15	NC	
203	2	max	.001	19	.018	15	.002	20	1.269e-03	13	NC	
204		min	0	20	-.267	13	-.002	19	2.695e-03	13	NC	
205	3	max	.002	19	.025	15	.004	20	1.465e-05	15	NC	
206		min	0	20	-.395	13	-.004	19	2.695e-03	13	NC	
207	4	max	-.003	19	.018	15	.006	20	1.465e-05	15	NC	
208		min	0	20	-.336	13	-.006	19	2.695e-03	13	NC	
209	5	max	.004	19	0	15	.008	20	1.465e-05	15	NC	
210		min	0	20	1.138	13	-.008	19	2.695e-03	13	NC	
211	M22	1	max	0	1	0	1	0	1	3.194e-06	15	NC
212		min	0	1	0	1	0	1	3.194e-06	15	NC	
213	2	max	0	13	.018	15	.002	20	3.194e-06	15	NC	
214		min	0	15	-.226	13	-.002	19	-2.879e-06	13	NC	
215	3	max	0	13	.025	15	.004	20	3.194e-06	15	NC	
216		min	0	15	-.313	13	-.004	19	-2.879e-06	13	NC	
217	4	max	0	13	.017	15	.006	20	3.194e-06	15	NC	
218		min	0	15	-.214	13	-.006	19	-2.879e-06	13	NC	
219	5	max	0	13	.024	15	.008	20	3.194e-06	15	NC	
220		min	0	15	-.008	13	-.008	19	-2.879e-06	13	NC	
221	M23	1	max	0	1	0	1	0	1	2.256e-03	13	NC
222		min	0	1	0	1	0	1	2.256e-03	13	NC	
223	2	max	0	20	.02	15	.002	20	2.256e-03	13	NC	
224		min	0	19	-.265	13	-.002	19	-1.356e-04	15	NC	
225	3	max	.001	20	.029	15	.004	20	2.256e-03	13	NC	
226		min	0	20	-.392	13	-.004	19	-1.356e-04	15	NC	
227	4	max	.002	20	.024	15	.006	20	2.256e-03	13	NC	
228		min	0	20	-.331	13	-.006	19	-1.356e-04	15	NC	
229	5	max	.003	20	.008	15	.008	20	2.256e-03	13	NC	
230		min	0	20	-.132	13	-.008	19	-1.356e-04	15	NC	
231	M24	1	max	0	1	0	1	0	1	2.046e-03	13	NC
232		min	0	1	0	1	0	1	8.395e-05	15	NC	
233	2	max	0	20	.006	15	0	20	2.046e-03	13	NC	
234		min	0	19	-.087	13	0	19	-8.395e-05	15	NC	
235	3	max	0	20	.012	15	0	20	2.046e-03	13	NC	
236		min	0	19	-.166	13	0	19	-8.395e-05	15	NC	

123.1

Steel Column

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: W8X18 Col (Porte Co)

Code References

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used: ASCE 7-10

General Information

Steel Section Name:	W10x68	Overall Column Height	13.0 ft
Analysis Method:	Allowable Strength	Top & Bottom Fixity	Top Free, Bottom Fixed
Steel Stress Grade		Brace condition for deflection (buckling) along columns:	
Fy: Steel Yield	50.0 ksi	X-X (width) axis:	
E: Elastic Bending Modulus	29,000.0 ksi	Unbraced Length for X-X Axis buckling = 13.0 ft, K = 2	
		Y-Y (depth) axis:	
		Unbraced Length for Y-Y Axis buckling = 13.0 ft, K = 2	

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included: 884.0 lbs * Dead Load Factor

AXIAL LOADS ...

Axial Load at 13.0 ft, D = 8.60, LR = 8.60 k

BENDING LOADS ...

Lat. Point Load at 13.0 ft creating Mx-x, E = 1.40 k

Moment acting about X-X axis, E = 2.40 k-ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	0.08773 : 1	Maximum Load Reactions ..	
Load Combination	+D+Lr	Top along X-X	0.0 k
Location of max.above base	0.0 ft	Bottom along X-X	0.0 k
At maximum location values are ...		Top along Y-Y	0.0 k
Pa: Axial	18.084 k	Bottom along Y-Y	1.40 k
Pn / Omega: Allowable	206.122 k	Maximum Load Deflections ...	
Ma-x: Applied	0.0 k-ft	Along Y-Y	0.1238 in at 13.0 ft above base
Mn-x / Omega: Allowable	212.824 k-ft	for load combination: E Only	
Ma-y: Applied	0.0 k-ft	Along X-X	0.0 in at 0.0 ft above base
Mn-y / Omega: Allowable	100.050 k-ft	for load combination:	
PASS Maximum Shear Stress Ratio =	0.01003 : 1		
Load Combination	+D+0.70E		
Location of max.above base	0.0 ft		
At maximum location values are ...			
Va: Applied	0.980 k		
Vn / Omega: Allowable	97.760 k		

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.046	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+Lr	0.088	PASS	0.00 ft	0.000	PASS	0.00 ft
+D+0.70E	0.075	PASS	0.00 ft	0.010	PASS	0.00 ft
+0.60D+0.70E	0.066	PASS	0.00 ft	0.010	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base @ Top	k	Y-Y Axis Reaction @ Base @ Top	Mx - End Moments @ Base @ Top	k-ft	My - End Moments @ Base @ Top
D Only	9.484						
+D+Lr	18.084						
+D+0.70E	9.484			0.980	-11.060		
+0.60D+0.70E	5.690			0.980	-11.060		
Lr Only	8.600						
E Only				1.400	-15.800		

123.2

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ENERCALC, INC. 1983-2017, Build:10.17.9.7, Ver:10.17.9.7

Steel Column

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: W8X18 Col (Porte Co)

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base @ Top	k	Y-Y Axis Reaction @ Base @ Top	Mx - End Moments @ Base @ Top	k-ft	My - End Moments @ Base @ Top
Axial @ Base	Maximum	18.084						
"	Minimum				1.400	-15.800		
Reaction, X-X Axis Base	Maximum	9.484						
"	Minimum	9.484						
Reaction, Y-Y Axis Base	Maximum				1.400	-15.800		
"	Minimum	9.484						
Reaction, X-X Axis Top	Maximum	9.484						
"	Minimum	9.484						
Reaction, Y-Y Axis Top	Maximum	9.484						
"	Minimum	9.484						
Moment, X-X Axis Base	Maximum	9.484						
"	Minimum		-15.800		1.400	-15.800		
Moment, Y-Y Axis Base	Maximum	9.484						
"	Minimum	9.484						
Moment, X-X Axis Top	Maximum	9.484						
"	Minimum	9.484						
Moment, Y-Y Axis Top	Maximum	9.484						
"	Minimum	9.484						

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.087 in	13.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.087 in	13.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.123 in	12.913 ft

Steel Section Properties : W10x68

Depth	=	10.400 in	I xx	=	394.00 in^4	J	=	3.560 in^4
Web Thick	=	0.470 in	S xx	=	75.70 in^3	Cw	=	3,100.00 in^6
Flange Width	=	10.100 in	R xx	=	4.440 in			
Flange Thick	=	0.770 in	Zx	=	85.300 in^3			
Area	=	19.900 in^2	I yy	=	134.000 in^4			
Weight	=	68.000 plf	S yy	=	26.400 in^3	Wno	=	24.300 in^2
Kdesign	=	1.270 in	R yy	=	2.590 in	Sw	=	47.300 in^4
K1	=	0.875 in	Zy	=	40.100 in^3	Qf	=	17.900 in^3
rts	=	2.920 in	rT	=	0.000 in	Qw	=	42.100 in^3
Ycg	=	0.000 in						

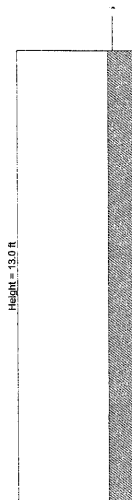
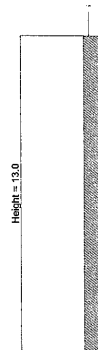
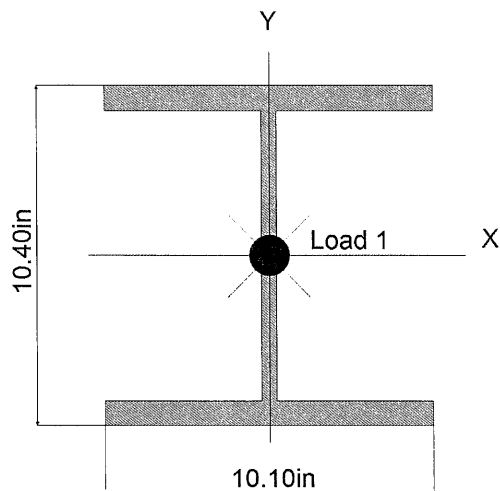
Steel Column

Lic. #: KW-06007967

Licensee: HCP ENGINEERING

Description: W8X18 Col (Porte Co)

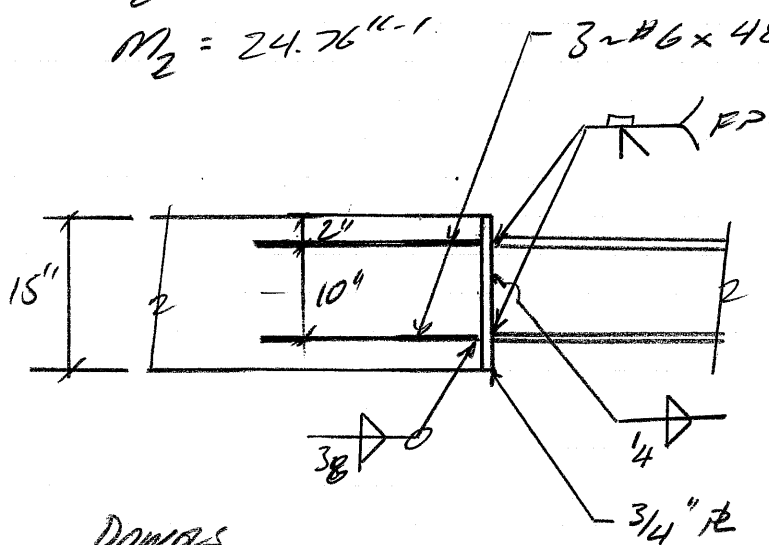
Sketches



Connection At Podium

Joint NS Beam's - Moment Conn

$$\begin{aligned} P_x &= 3.04^k & P_{xu} &= 1.6(3.04) = 4.87^k \\ P_y &= 2.87^k & P_{yu} &= 1.6(2.87) = 4.6^k \\ P_z &= 13.2^k & P_{zu} &= 1.6(13.2) = 21.2^k \\ M_2 &= 24.76^{k-ft} \end{aligned}$$



DWLS

$$T_u = C = \frac{1.6(24.8)(12)}{10} = 47.6^k$$

R/F

3-#6 x 48" (min) DWLS.

$$\phi T_n = 0.9(3)(0.44)(60) = 71.3^k > 47.6^k$$

Weld

$$\therefore \frac{3\text{-}\#6 \times 48''}{\text{DWLS}} \quad (7 \times 3)$$

Weld

$$\frac{3}{8}V - \phi$$

$$T_n = 5.4(2.35)(3) = 38.1^k > 29.5^k$$

$$\therefore \frac{3}{8}V - \phi \quad (\text{As DWLS})$$

$$V = 13.2^k < \frac{V_n}{\phi} = 53.9^k$$

Anchorage To Core

$$N_{uA} = \frac{T_u}{3} + \frac{P_{xu}}{6} = \frac{47.6}{3} + \frac{4.87}{6} = 16.7^k$$

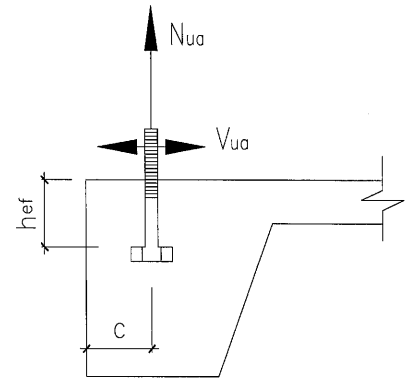
$$V_{uA} = \frac{P_{zu}}{6} = \frac{21.2}{6} = 3.52^k$$

(See p. 124.1)

Single Fastener in Tension and Shear Near an Edge Based on ACI 318-14

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	F'_c	=	5	ksi
SPECIFIED YIELD STRENGTH OF FASTENERS	F_y	=	60	ksi
SPECIFIED STRENGTH OF FASTENER	F_{uta}	=	114	ksi
(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)				
FACTORED DESIGN TENSION LOAD	N_{ua}	=	16.7	k
FACTORED DESIGN SHEAR LOAD	V_{ua}	=	3.52	k
EFFECTIVE EMBEDMENT DEPTH	h_{ef}	=	48	in
FOOTING WIDTH	w	=	15	in
FASTENER DIAMETER	d	=	0.75	in
FASTENER HEAD TYPE			5	Hex
(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)				
DIST. BETWEEN THE FASTENER AND E	C_x	=	6	in
	C_y	=	12	in
SEISMIC LOAD ? (ACI 318-05 D3.3.3)			Yes	



[THE FASTENER DESIGN IS ADEQUATE.]

ANALYSIS

EFFECTIVE AREA OF FASTENER	A_{se}	=	0.334	in ²
BEARING AREA OF HEAD	A_b	=	1.252	in ² , (or determined from manufacture's catalogs.)
CHECK FASTENER TENSILE STRENGTH : (ACI 318-14 17.4.1.2)				

$$\phi N_s = \phi n A_{se} (f_{uta}) = 28.557 \text{ k} > N_{ua} \text{ [Satisfactory]}$$

$$\text{where: } \phi = 0.75 \times 1 = 0.75, \text{ (ACI 318-14.17.3.2.3)}$$

CHECK CONCRETE BREAKOUT STRENGTH : (ACI 318-14 17.4.2.1a)

$$\phi N_{cb} = \phi \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{(9h_{ef}^2)} \left(0.7 + \frac{0.3c}{1.5h_{ef}} \right) \psi_{c,N} \left(24\sqrt{f'_c} h_{ef}^{1.5} \right)$$

$$= 96.96 \text{ k} > N_{ua} \text{ [Satisfactory]}$$

$$\text{where: } \phi = 0.75 \times 1 = 0.75$$

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur = 1.0

CHECK PULLOUT STRENGTH : (ACI 318-14 17.4.3.1)

$$\phi N_{pn} = \phi \psi_{cp,N} (A_b 8 f'_c) = 37.560 \text{ k} > N_{ua} \text{ [Satisfactory]}$$

$$\text{where: } \phi = 0.75 \times 1 = 0.75$$

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur = 1.0

CHECK SIDE-FACE BLOWOUT STRENGTH : (ACI 318-14 17.4.4.1)

$$\phi N_{sb} = \phi \left(160c \sqrt{A_b} \sqrt{f'_c} \right) = 56.967 \text{ k} > N_{ua} \text{ [Satisfactory]}$$

$$\text{where: } \phi = 0.75 \times 1 = 0.75$$

DETERMINE DESIGN TENSILE STRENGTH :

$$\phi N_n = \min(\phi N_s, \phi N_{cb}, \phi N_{pn}, \phi N_{sb}) = \boxed{28.56 \text{ K}}$$

CHECK FASTENER SHEAR STRENGTH : (ACI 318-14 17.5.1.2b)

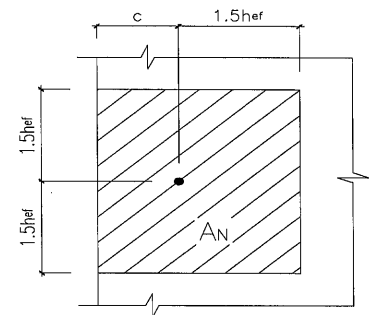
$$\phi V_s = \phi n 0.6 A_{se} f_{ut} = 15.99 \text{ k} > V_{ua} \text{ [Satisfactory]}$$

$$\text{where: } \phi = 0.7 \times 1 = 0.7$$

CHECK CONCRETE BREAKOUT STRENGTH FOR SHEAR LOAD : (ACI 318-14 17.5.2.1a)

$$\phi V_{cb} = \phi \frac{A_v}{A_{Vo}} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{A_v}{A_{Vo}} \psi_{cd,V} \psi_{c,V} \left(7 \left(\frac{l}{d} \right)^{0.2} \sqrt{d} \sqrt{f'_c} c^{1.5} \right) = 7.162 \text{ k} > V_{ua} \text{ [Satisfactory]}$$

$$\text{where: } \phi = 0.75 \times 1 = 0.75, \text{ (ACI 318-14.17.3.2.3)}$$



$\psi_{c,v}$ term is 1.0 for location where concrete cracking is likely to occur.

A_v / A_{v0} and $\psi_{cd,v}$ terms are 1.0 for single shear fastener not influenced by more than one free edge.

l term is load bearing length of the anchor for shear, not to exceed $8d$.

CHECK PRYOUT STRENGTH FOR SHEAR LOAD : (ACI 318-14 17.5.3.1a)

$$\phi V_{cp} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{(9h_{ef}^2)} \left(0.7 + \frac{0.3c}{1.5h_{ef}} \right) \psi_{c,N} \left(24\sqrt{f'_c} h_{ef}^{1.5} \right)$$

$$= 193.926 \text{ k} > V_{ua} \quad [\text{Satisfactory}]$$

where : $\phi = 0.75$ x $1 = 0.75$

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

$k_{cp} = 2.0$ for $h_{ef} > 2.5$ in.

DETERMINE DESIGN SHEAR STRENGTH :

$$\phi V_n = \min(\phi V_s, \phi V_{cb}, \phi V_{cp}) = \boxed{7.162 \text{ K}}$$

CHECK TENSION AND SHEAR INTERACTION : (ACI 318-14 17.6.3)

Since $N_{ua} > 0.2 \phi N_n$ and

$V_{ua} > 0.2 \phi V_n$ the full design strength is not permitted.

The interaction equation must be used

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} = \boxed{1.08} < 1.2 \quad [\text{Satisfactory}]$$

Summary of Dimensional Properties of Fasteners

Fastener Diameter (in)		Gross Area of Fastener (in ²)	Effective Area of Threaded Fastener (in ²)	Bearing Area of Heads, Nuts, and Washers (A _b) (in ²)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Technical Reference:

1. Ronald Cook, "Strength Design of Anchorage to Concrete," PCA, 1999.

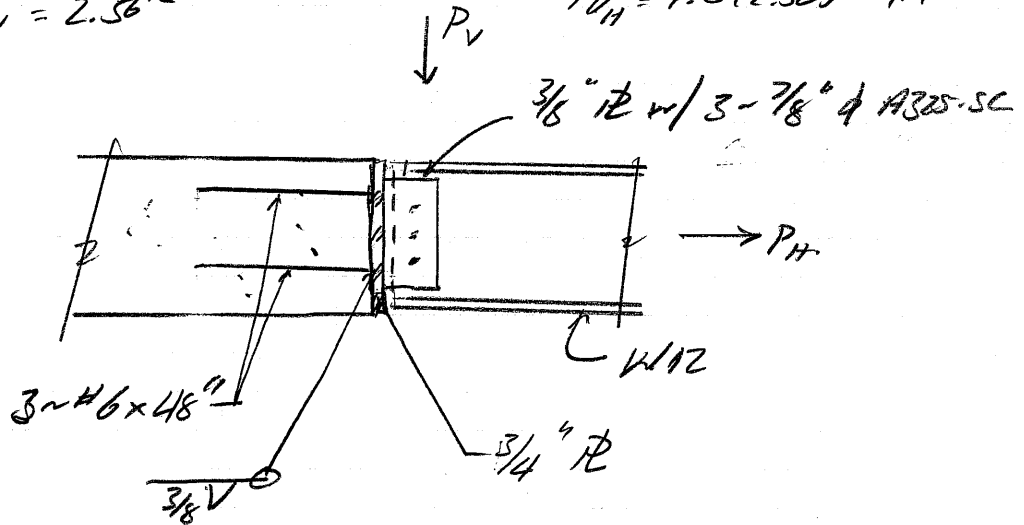
SIMPLE CONJ

$$P_V = 2.73^k \text{ (JOINT N-25)}$$

$$P_H = 2.56^k$$

$$P_{HV} = 1.6(2.73) = 4.4^k$$

$$P_{HH} = 1.6(2.56) = 4.1^k$$



$$\frac{\phi V}{\Sigma} = 21.7^k > 2.73^k \text{ OKAY}$$

USE ARCH. TO CONE
SIMILAR TO JOINT N-5
(See p. 124)

FOUNDATION

$$P_V = 17.1^k \text{ (JOINT N-18)}$$

$$P_{Hx} = 1.37^k$$

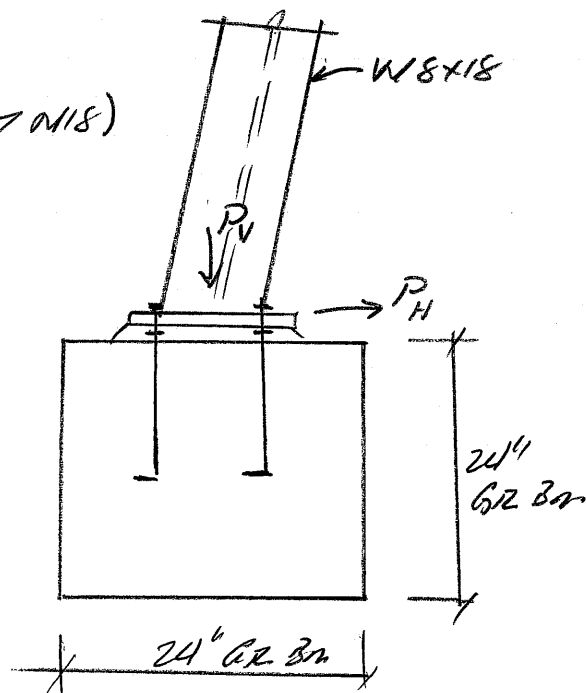
$$P_{Hy} = 0.52^k$$

$$M_x = 2.38^k\text{-ft}$$

$$M_y = 1.03^k\text{-ft}$$

BASE PL

USE 12" x 14" x 1" THK
w/ 4-3/4" diameter A.B.
(See p. 126)



STEEL COLUMN BASE PLATE ANALYSISPer AISC 13th Edition Manual (ASD) and "Design of Welded Structures"
For Axial Load with or without Moment

Job Name:		Subject:	
Job Number:		Originator:	Checker:

Input Data:**Column Size:**Select: **W10x68****Column Loadings:**

Axial Load, $P_{(total)}$ =	-17.10	kips
Axial Load, $P_{(DL)}$ =	-8.00	kips
Shear Load, $V_{(total)}$ =	1.37	kips
Moment @ Base, M =	2.40	ft.-kips

Design Parameters:

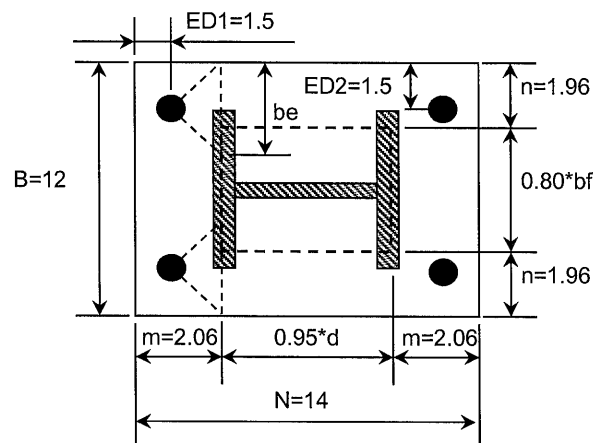
Base Plate Length, N =	14.000	in.
Base Plate Width, B =	12.000	in.
Plate Yield Stress, F_y =	36.00	ksi
Concrete Strength, f'_c =	5.000	ksi
Bearing Area, A_2 =	168.00	in. ²
Shear Coef., C =	1.85	
Coef. of Friction, μ =	0.55	

Anchor Bolt/Rod Data:

Total No. of Bolts, N_b =	4	
Bolt Diameter, d_b =	0.750	in.
Anchor Bolt Material =	A36	
Bolt Edge Dist., ED_1 =	1.500	in.
Bolt Edge Dist., ED_2 =	1.500	in.

Column Properties:

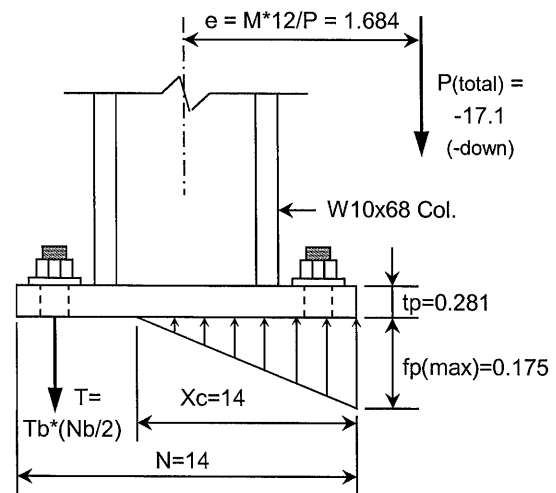
A =	20.00	in. ²
d =	10.400	in.
tw =	0.470	in.
bf =	10.100	in.
tf =	0.770	in.

**Plan****Results:****Eccentricity, Bearing Length, and Bearing Pressures:**

Eccentricity, e =	1.684	in.
Length, X_c =	14.000	in.
F_p =	1.750	ksi
$f_p(max)$ =	0.175	ksi
$f_p(min)$ =	0.028	ksi

 $F_p \geq f_p(max)$, O.K.**Anchor Bolt/Rod Tension and Shear:**

F_t =	19.10	ksi
T_a =	8.44	k/bolt
T_b =	0.00	k/bolt
F_v =	9.90	ksi
V_a =	4.37	k/bolt
$V(bolts)$ =	0.00	= $V_{(total)} - 1/2 * \mu * P_{(DL)}$
V_b =	0.00	k/bolt

(Interaction) S.R. = **N.A.** = $T_b/T_a + (C * V_b)/V_a$ **Elevation****Base Plate Thickness:** $tp(req'd)$ = **0.281** in.Suggested plate thickness for rigidity:
 $tp(min)$ = **0.515** in. $tp(min) \geq \max. \text{ of } m/4 \text{ or } n/4$

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PROJECT : Porte Co Column
CLIENT :
JOB NO. :

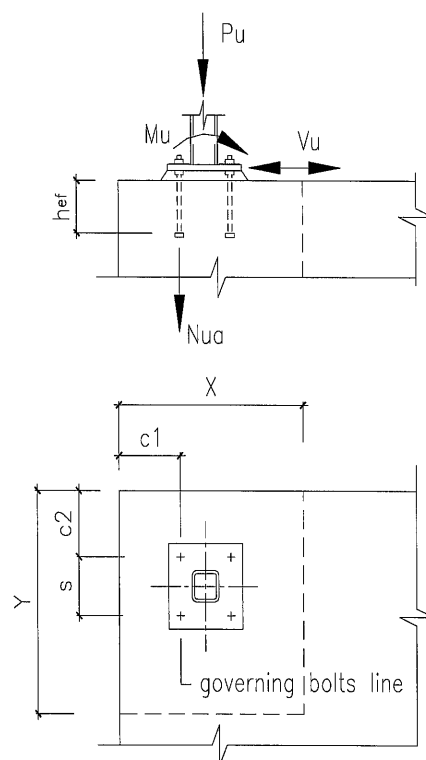
DATE :

PAGE :
DESIGN BY :
REVIEW BY :

Column Base Fasteners Anchorage Based on ACI 318-14

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f'_c	=	5	ksi
SPECIFIED STRENGTH OF FASTENER	f_{uta}	=	58	ksi
YIELD STRENGTH OF FASTENER	f_y	=	36	ksi
(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)				
FACTORED COLUMN AXIAL LOAD	P_u	=	27.4	k, Compression, Do
FACTORED TOTAL BASE SHEAR LOAD	V_u	=	2.2	k
FACTORED COLUMN MOMENT	M_u	=	3.8	ft-k
ECCENTRICITY	e'_N	=	0.1	in
EFFECTIVE EMBEDMENT DEPTH	h_{ef}	=	18	in
FASTENER DIAMETER	d	=	0.75	in
FASTENER HEAD TYPE			4	Heavy Hex
(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers, 6=Bearing Plate)				
TOTAL NUMBER OF BOLTS	n	=	4	
NUMBER OF BOLTS IN TENSION	n_T	=	2	
FASTENER CENTER-TO-CENTER SPACI	s	=	13	in
MIN. EDGE DISTANCE AT X DIRECTION	c_1	=	12	in
MIN. EDGE DISTANCE AT Y DIRECTION	c_2	=	24	in
TOTAL CONCRETE DIM. (999 = no limited	X	=	2.00	ft
	Y	=	12.00	ft
FOOTING / SLAB THICKNESS	h_a	=	24	in
SEISMIC LOAD ? (ACI 318-05 D3.3.3)		=	>	Yes Reduce Concrete Failure Modes by 0.75



[THE FASTENER DESIGN IS ADEQUATE.]

ANALYSIS

MAX TENSION ON GOV'G FASTENERS	$N_{ua} = M_u / (s - 2.0") - P_u / n$	=	-9.55	k
SHEAR FORCE ON GOV'G FASTENERS	$V_{ua} = V_{u, total} / n$	=	1.1	k
EFFECTIVE AREA OF ONE FASTENER	A_{se}	=	0.334	in ²
BEARING AREA OF ONE HEAD	A_b	=	0.911	in ² , (or determined from manufacture's catalogs.)
CHECK THE GOVERNING FASTENERS TENSILE STRENGTH : (ACI 318, D.5.1.2)				

$$\phi N_s = \phi n A_{se} (f_{uta}) = 29.06 \text{ k} > N_{ua} \text{ [Satisfactory]}$$

$$\text{where : } \phi = 1.00 \times 0.75 = 0.75$$

CHECK CONCRETE BREAKOUT STRENGTH OF GOVERNING FASTENERS : (ACI 318, D.5.2.1)

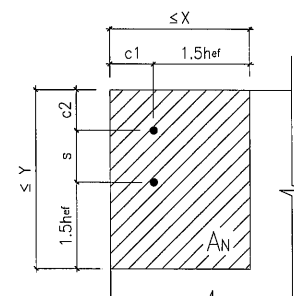
$$\phi N_{cbg} = \phi \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{(9h_{ef}^2)} \psi_{ec,N} \left(0.7 + \frac{0.3c_{min}}{1.5h_{ef}} \right) \psi_{c,N} \left(24\sqrt{f'_c} h_{ef}^{1.5} \right)$$

$$= 34.57 \text{ k} > N_{ua} \text{ [Satisfactory]}$$

$$\text{where : } \phi = 0.75 \times 0.75 = 0.5625, \text{ (ACI 318-05 D.4.4 \& D.}$$

$$\psi_{ec,N} = 1.00 \text{ for to eccentric load}$$

$$\psi_{c,N} \text{ term is 1.0 for location where concrete cracking is likely to occur} = 1.0$$



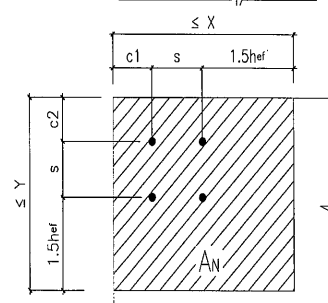
CHECK CONCRETE BREAKOUT STRENGTH OF ALL FASTENERS : (ACI 318, D.5.2.1)

$$\phi N_{cbg} = \phi \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{(9h_{ef}^2)} \psi_{ec,N} \left(0.7 + \frac{0.3c_{min}}{1.5h_{ef}} \right) \psi_{c,N} \left(24\sqrt{f'_c} h_{ef}^{1.5} \right)$$

$$= 34.57 \text{ k} > N_{ua} \text{ [Satisfactory]}$$

$$\text{where : } \phi = 0.75 \times 0.75 = 0.5625$$

$$\psi_{ec,N} = 1.00 \text{ for to eccentric load, (ACI 318 Fig. RD.5.2.4)}$$



$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur = 1.0

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CHECK PULLOUT STRENGTH OF GOVERNING FASTENERS : (ACI 318, D.5.3.1)

$$\phi N_{pn} = \phi n \psi_{cp,N} \left(A_b 8 f'_c \right) = 41.0 \text{ k} > N_{ua} \quad [\text{Satisfactory}]$$

where : $\phi = 0.75 \times 0.75 = 0.5625$

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur. 1.0

CHECK SIDE-FACE BLOWOUT STRENGTH : (ACI 318, D.5.4.1)

$$c_{min} > 0.4 h_{ef}$$

Since this fastener is located far from a free edge of concrete ($c > 0.4 h_{ef}$) this type of failure mode is not applicable.

$$\phi N_{sb} = \phi \left(160 c \sqrt{A_b} \sqrt{f'_c} \right) = 114.73 \text{ k} > N_{ua} \quad [\text{Satisfactory}]$$

where : $\phi = 0.75 \times 1 = 0.75$

DETERMINE DESIGN TENSILE STRENGTH OF GOVERNING FASTENERS :

$$\phi N_n = \min(\phi N_s, \phi N_{cbg}, \phi N_{pn}) = \boxed{29.06 \text{ K}} > N_{ua} \quad [\text{Satisfactory}]$$

CHECK GOVERNING FASTENERS SHEAR STRENGTH : (ACI 318, D.6.1.2b)

$$\phi V_s = \phi n 0.6 A_{se} f_{ut} = 34.87 \text{ k} > V_{ua} \quad [\text{Satisfactory}]$$

where : $\phi = 1.00 \times 0.75 = 0.75$

(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 D.6.1.3)

CHECK CONCRETE BREAKOUT STRENGTH OF GOVERNING FASTENERS FOR SHEAR LOAD : (ACI 318, D.6.2.1b)

$$\phi V_{cbg} = \phi \frac{A_v}{A_{v0}} \psi_{ec,N} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{(1.5cl) \text{Min}(1.5cl + s + c2, Y)}{4.5cl^2} \psi_{ec,V} \left(0.7 + 0.3 \frac{c2}{1.5cl} \right) \psi_{c,V} \left(7 \left(\frac{l}{d} \right)^{0.2} \sqrt{d} \sqrt{f'_c} cl^{1.5} \right)$$

$$= 27.01 \text{ k} > V_{ua} \quad [\text{Satisfactory}]$$

where : $\phi = 0.75 \times 0.75 = 0.5625$

$\psi_{ec,V}$ term is 1.0 for no eccentricity in the connection. 1.0

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur 1.0

l term is load bearing length of the anchor for shear, not to exceed $8d$.

CHECK CONCRETE BREAKOUT STRENGTH OF ALL FASTENERS FOR SHEAR LOAD : (ACI 318, D.6.2.1b)

$$\phi V_{cbg} = \phi \frac{A_v}{A_{v0}} \psi_{ec,N} \psi_{cd,V} \psi_{c,V} V_b = \phi \frac{(1.5cl + s) \text{Min}(1.5cl + s + c2, Y)}{4.5cl^2} \psi_{ec,V} \left(0.7 + 0.3 \frac{c2}{1.5cl} \right) \psi_{c,V} \left(7 \left(\frac{l}{d} \right)^{0.2} \sqrt{d} \sqrt{f'_c} cl^{1.5} \right)$$

$$= 27.01 \text{ k} > V_u \quad [\text{Satisfactory}]$$

where : $\phi = 0.75 \times 0.75 = 0.5625$

$\psi_{ec,V}$ term is 1.0 for no eccentricity in the connection.

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur.

l term is load bearing length of the anchor for shear, not to exceed $8d$.

CHECK PRYOUT STRENGTH FOR SHEAR LOAD ON GOVERNING FASTENERS : (ACI 318, D.6.3.1)

$$\phi V_{cpb} = \phi k_{cp} \frac{A_v}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_v}{(9h_{ef}^2)} \left(0.7 + \frac{0.3c_{min}}{1.5h_{ef}} \right) \psi_{c,N} \left(24 \sqrt{f'_c} h_{ef}^{1.5} \right)$$

$$= 69.138 \text{ k} > V_{ua} \quad [\text{Satisfactory}]$$

where : $\phi = 0.75 \times 0.75 = 0.5625$

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur 1.00

$$k_{cp} = 2.0 \quad \text{for } h_{ef} > 2.5 \text{ in.}$$

CHECK PRYOUT STRENGTH FOR SHEAR LOAD ON ALL FASTENERS : (ACI 318, D.6.3.1)

$$\phi V_{cpb} = \phi k_{cp} \frac{A_v}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_v}{(9h_{ef}^2)} \left(0.7 + \frac{0.3c_{min}}{1.5h_{ef}} \right) \psi_{c,N} \left(24 \sqrt{f'_c} h_{ef}^{1.5} \right)$$

$$= 69.138 \text{ k} > V_u \quad [\text{Satisfactory}]$$

where : $\phi = 0.75 \times 0.75 = 0.5625$
 $\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur 1.00
 $k_{cp} = 2.0$ for $h_{ef} > 2.5$ in.

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DETERMINE DESIGN SHEAR STRENGTH OF GOVERNING FASTENERS :

$$\phi V_n = \min(\phi V_s, \phi V_{cbg}, \phi V_{cp}) = 27.01 \text{ K} > V_{ua} \text{ [Satisfactory]}$$

REQUIRED EDGE DISTANCES AND SPACING TO PRECLUDE SPLITTING FAILURE :

Since headed cast-in-place fasteners are not like to be highly torqued, the minimum cover requirements of ACI 318 Sec. 7.7 apply.

$$\text{Cover}_{\text{Provid}} = 12 > 2 = \text{Cover}_{\text{Reqd}} \text{ [Satisfactory]}$$

CHECK TENSION AND SHEAR INTERACTION OF GOVERNING FASTENERS : (ACI 318, D.7)

Since $N_{ua} > 0.2 \phi N_n$ Check Interaction
 $V_{ua} > 0.2 \phi V_n$ (Fully Tension Permitted, D.7) and
 the full design strength is not permitted.

The interaction equation must be used

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} = 0.37 < 1.2 \text{ [Satisfactory]}$$

Summary of Dimensional Properties of Fasteners

Fastener Diameter (in)		Gross Area of Fastener (in ²)	Effective Area of Threaded Fastener (in ²)	Heads, Nuts, and Washers A_b (in ²)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Technical Reference:

1. Ronald Cook, "Strength Design of Anchorage to Concrete," PCA, 1999.