HCP ENGINEERING

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STRUCTURAL CALCULATIONS

FOR

HOME2 SUITES by HILTON

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

REGISTERED PROFESSIONAL ENGINEER
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Exp. 6-30-2019
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08/25/2017

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HOME2 SUITES by HILTON

550 Gateway Blvd
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:		psf					
		BUILT UP w/ Mech.	BUILT UP w/ Solar	CONC. TILES		ASPH. SHINGLES	
D.L.	ROOFING	4.0	4.0	10.0		4.0	
	SHEATHING	3.0	3.0	3.0		2.5	
	FRAMING	3.5	3.5	3.5		3.5	
	MISC. (incl.F.Spklr / Solar*)	6.0	16.0	2.5		2.5	
		16.5	26.5	19.0 x 13/12 =	20.6	12.5 x 13/12 =	13.5
	CEILING (2 layers)	5.5	5.5	5.5	5.5		5.5
		22.0	32.0		26.0		19.0
L.L	*(Reducible)*	20.0	20.0		20.0		20.0
	TOTAL	42.0	52.0		46.0		39.0

*Incl. 12.0 psf for Solar

1 Use Typ.

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

+15 psf (Lt. Wt. Conc) = 31.5 psf ≈ 32.0 psf
– Reduce to 30 psf at Area > 450 sf (R=0.75).

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

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SEISMIC PARAMETERS.

TABLE 1- 2013 CBC SESIMIC PARAMETERS *

Parameter	Value
Site Class/Soil Profile Type	D
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.5
Mapped MCE Spectral Acceleration (0.2 sec), S_S	1.506
Mapped MCE Spectral Acceleration (1.0 sec), S_1	0.600
MCE Spectral Acceleration (0.2 sec), S_{MS}	1.506
MCE Spectral Acceleration (1.0 sec), S_{M1}	0.900
Design Spectral Acceleration (0.2 sec), S_{DS}	1.004
Design Spectral Acceleration (0.2 sec), S_{D1}	0.600

*FROM GEOTECHNICAL REPORT BY:
GEO ENGINEERING CONSULTANTS.
4125 BLACKFORD AVE #145.
ALAMEDA, CA 95117
P: 925-321-5530
REPORT #: P16-152
DATE: MARCH 31, 2016.

SEISMIC WT: PODIUM.

SLAB: 15" THK: 150(15/12)(19500) = 3,656,250 #
10" PONTO CO: 150(10/12)(1100) = 137,500 #
CONC. WALLS: 150(7)(720) = 756,000 #
CONC. COLS: 150(1.5)(1.5)(7)(17) = 46,163 #
4,589,913 #

TWO STAGE LOAD AMPLIFICATION FACTOR (ASCE 12.2.3.2)

SUPER STRUCTURE: R = 6.5 ρ = 1.0
PODIUM R = 5.0 ρ = 1.3.

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

LOAD FACTOR APPLIED TO EQX & EQY LOADS

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SEISMIC BASE SHEAR

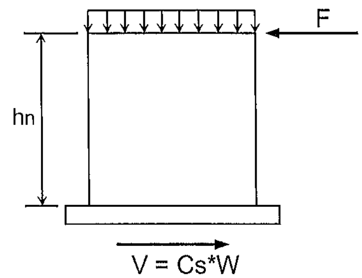
Per CBC 2016 and ASCE 7-10 Specifications

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

Job Name:	HOME2 - SSF (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., Ss =	1.506	ASCE 7-10 Figures 22-1, 3, 5, 6
Spectral Accel., S1 =	0.600	ASCE 7-10 Figures 22-2, 4, 5, 6
Long. Trans. Period, TL =	8.000	sec. ASCE 7 Fig's. 22-12 to 22-16
Structure Height, hn =	14.000	ft.
Total Seismic Weight, W =	4590.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, Tc =	0.000	sec. from Analysis or "0" if T_a (below)
Seismic Resist. System =	A1	A1. Special reinforced concrete shear walls (ASCE 7-10, Table 12.2-1)



Seismic Base Shear
(Regular Bldg. Configurations Only)

Results:

Site Coefficients:

Fa =	1.000	ASCE 7-10, Table 11.4.-1
Fv =	1.500	ASCE 7-10, Table 11.4-2

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

SMS =	1.506	SMS = Fa*SS, ASCE 7-10, Eqn 11.4-1
SM1 =	0.900	SM1 = Fv*S1, ASCE 7-10, Eqn 11.4-2

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

SDS =	1.004	SDS = 2*SMS/3, ASCE -10, Eqn 11.4-3
SD1 =	0.600	SD1 = 2*SM1/3, ASCE 7-10 Eqn 11.4-4

Seismic Design Category:

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

Fundamental Period:

Period Coefficient, CT =	0.020	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2
Approx. Period, Ta =	0.145	sec., Ta = CT*hn^(x), ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., Cu =	1.400	ASCE 7-10 Table 12.8-1
Period max., T(max) =	0.203	sec., T(max) = Cu*Ta, ASCE 7-10 Section 12.8.2
Fundamental Period, T =	0.145	sec., T = Ta <= Cu*Ta, 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, Ωo =	2.5	ASCE 7-10 Table 12.2-1
Defl. Amplif. Factor, Cd =	5	ASCE 7-10 Table 12.2-1
Cs =	0.201	CS = SDS/(R/I), ASCE 7-10 Section 12.8.1.1, Eqn. 12.8-2
CS(max) =	0.829	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-10 Eqn. 12.8-3
CS(min) =	0.060	CS(min) = 0.5*S1/(R/I), ASCE 7-10 Eqn. 12.8-6
Use: Cs =	0.201	CS(min) <= CS <= CS(max)
C_{ASD} =	0.141	CASD = 0.7 x CS

Seismic Base Shear:

Redundancy, Rho ρ =	1.3	ASCE 12.3.4.1
V =	1198.17	kips, V = CS*W, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1
V_{ASD} =	838.72	kips, V_{ASD} = 0.7 x V

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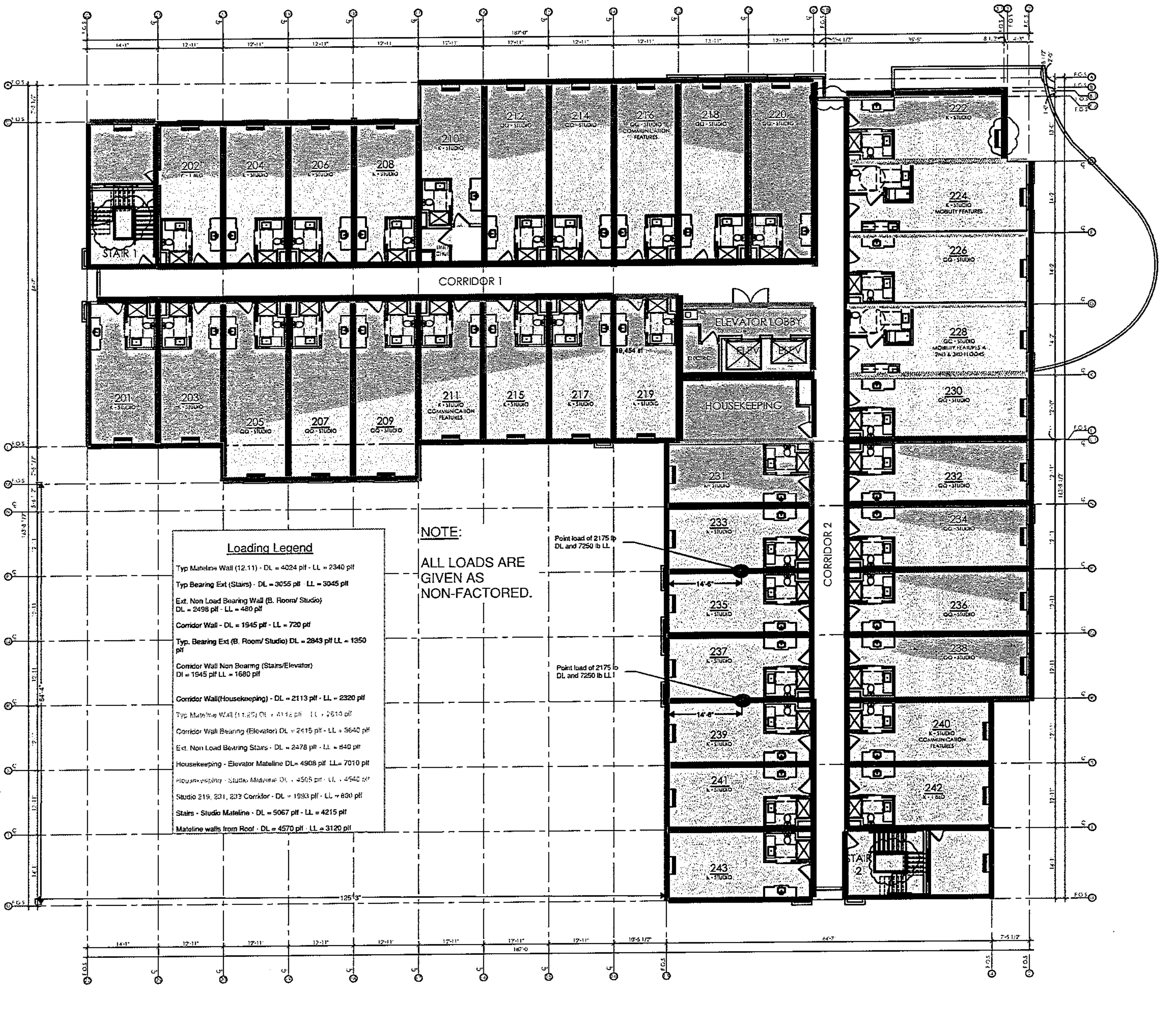
SUPERSTRUCTURE
DEAD & LIVE LOADS.
(SUPPLIED BY GULDMAN MANUS - 4/14/17)



K12 SECOND FLOOR PLAN (THIRD, FOURTH & FIFTH FLOORS SIMILAR)
SCALE 1/8" = 1'-0"

Superstructure
Seismic Load
(Supplied By Guendon Manuel - 4/14/17)

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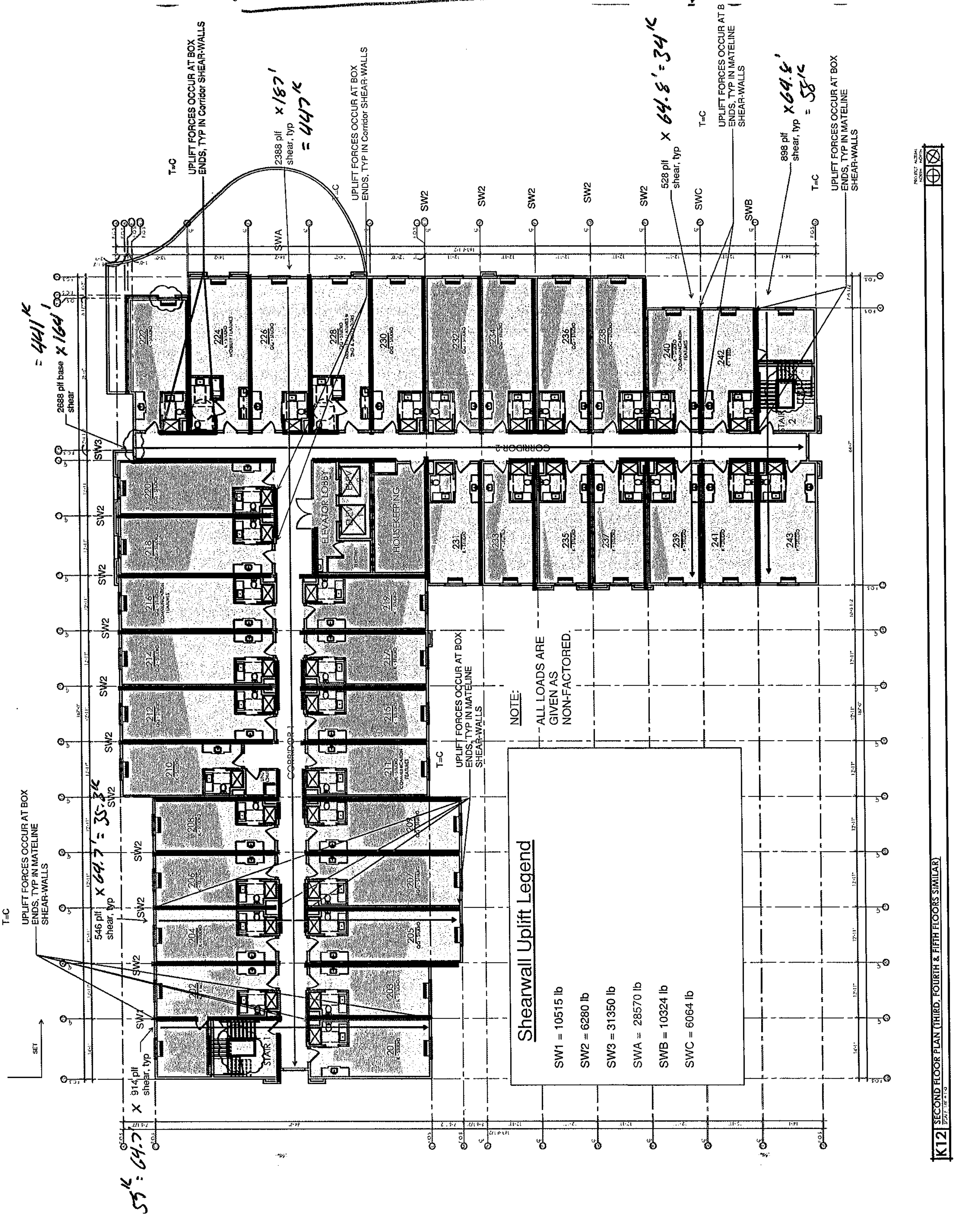

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Podium

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	fpy	Eps
		ksi	ksi	ksi
1	Prestressing 1	270.000	240.000	29007

fpu = ultimate stress
fpy = 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

Extension of rebar beyond where required for strength:
Top bars = 12.00 in
Bottom bars = 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 = 12.00 in
Bottom bars = 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

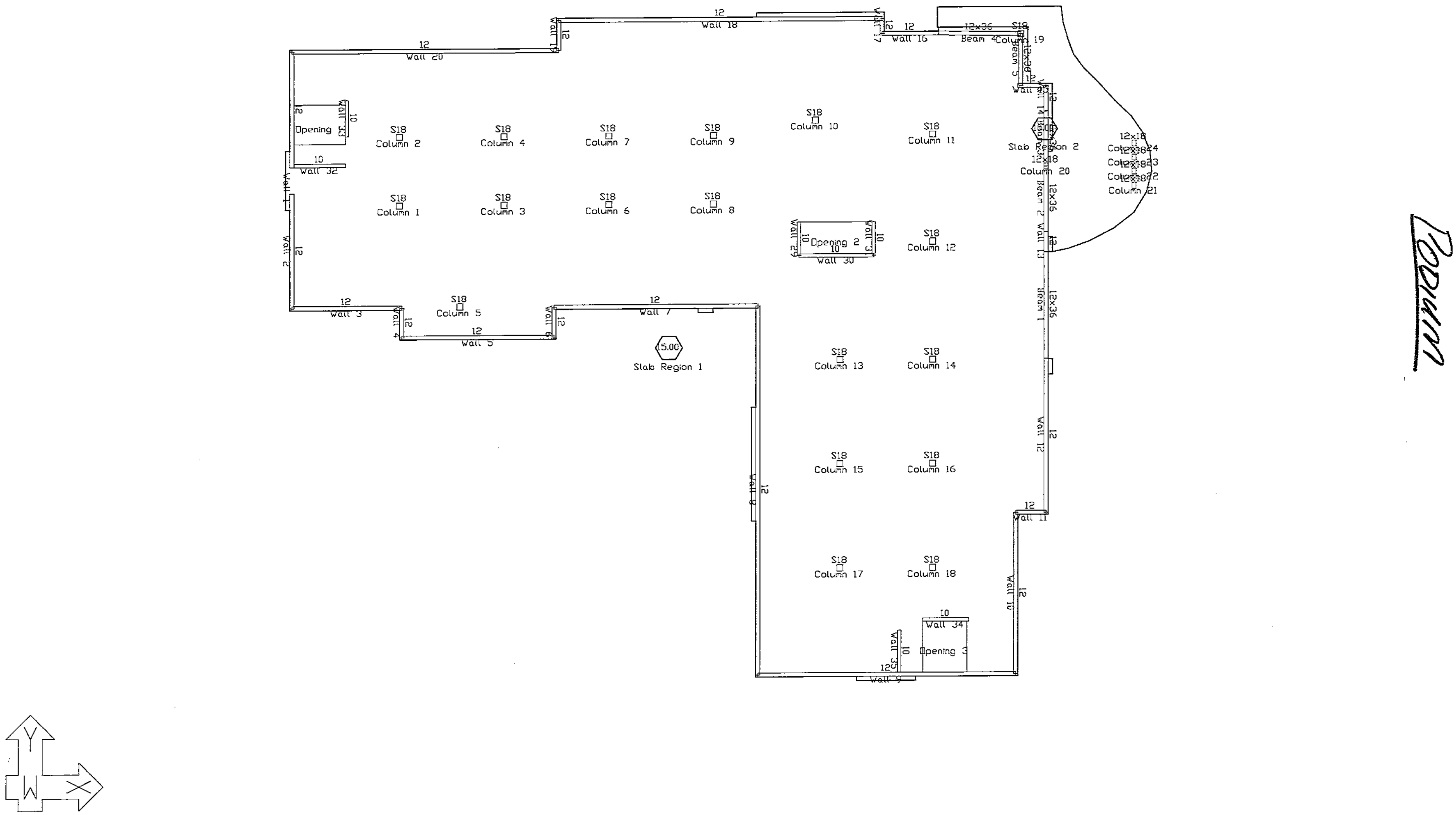
usage	#	M11	M33	F11

Walls

usage	#	M11	M22	F11	F22

Slabs

usage	#	M11	M22	F11	F22



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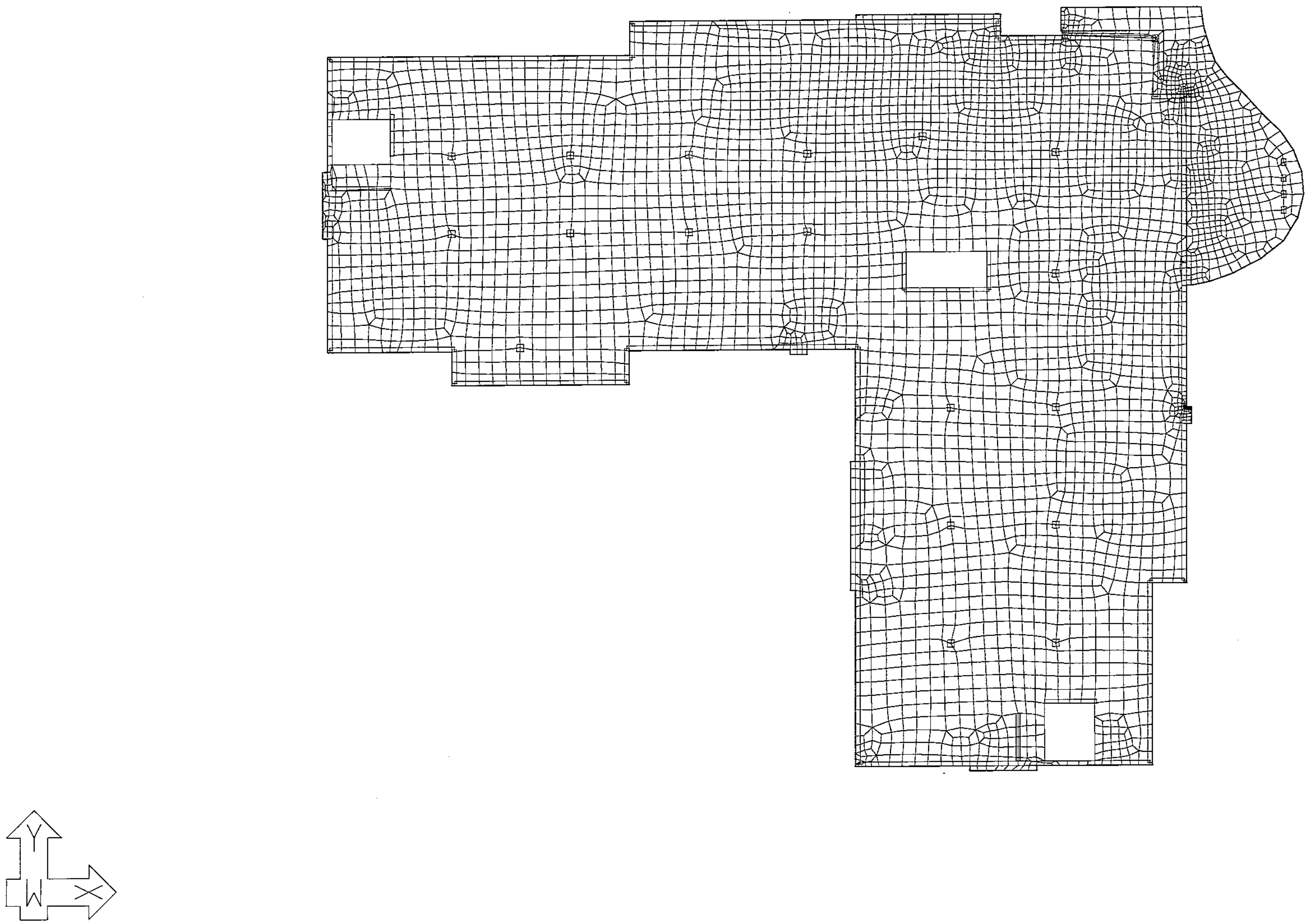

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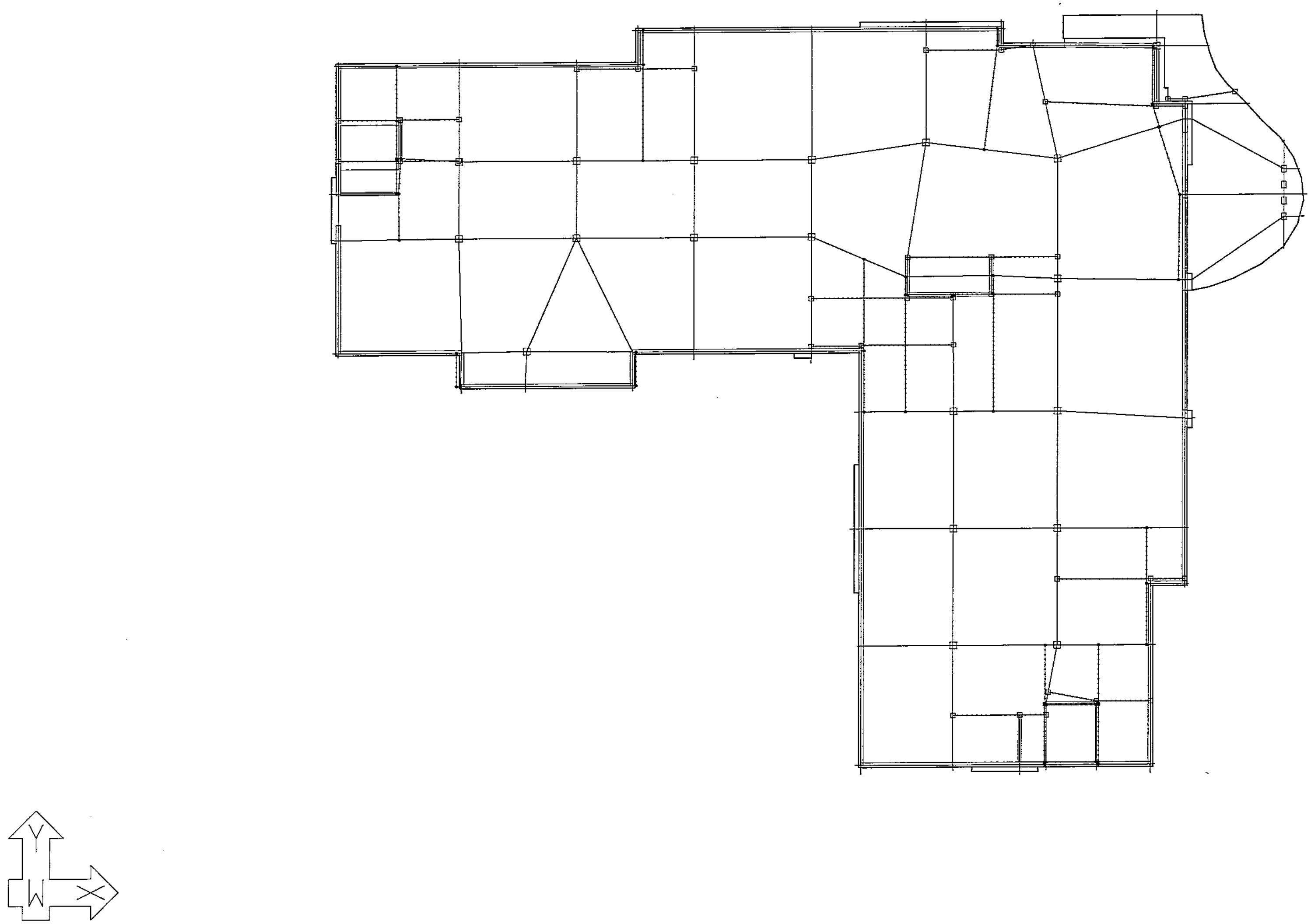

05/28/17 10:39:55	HOME2 - SSF SUPPORT LINES	
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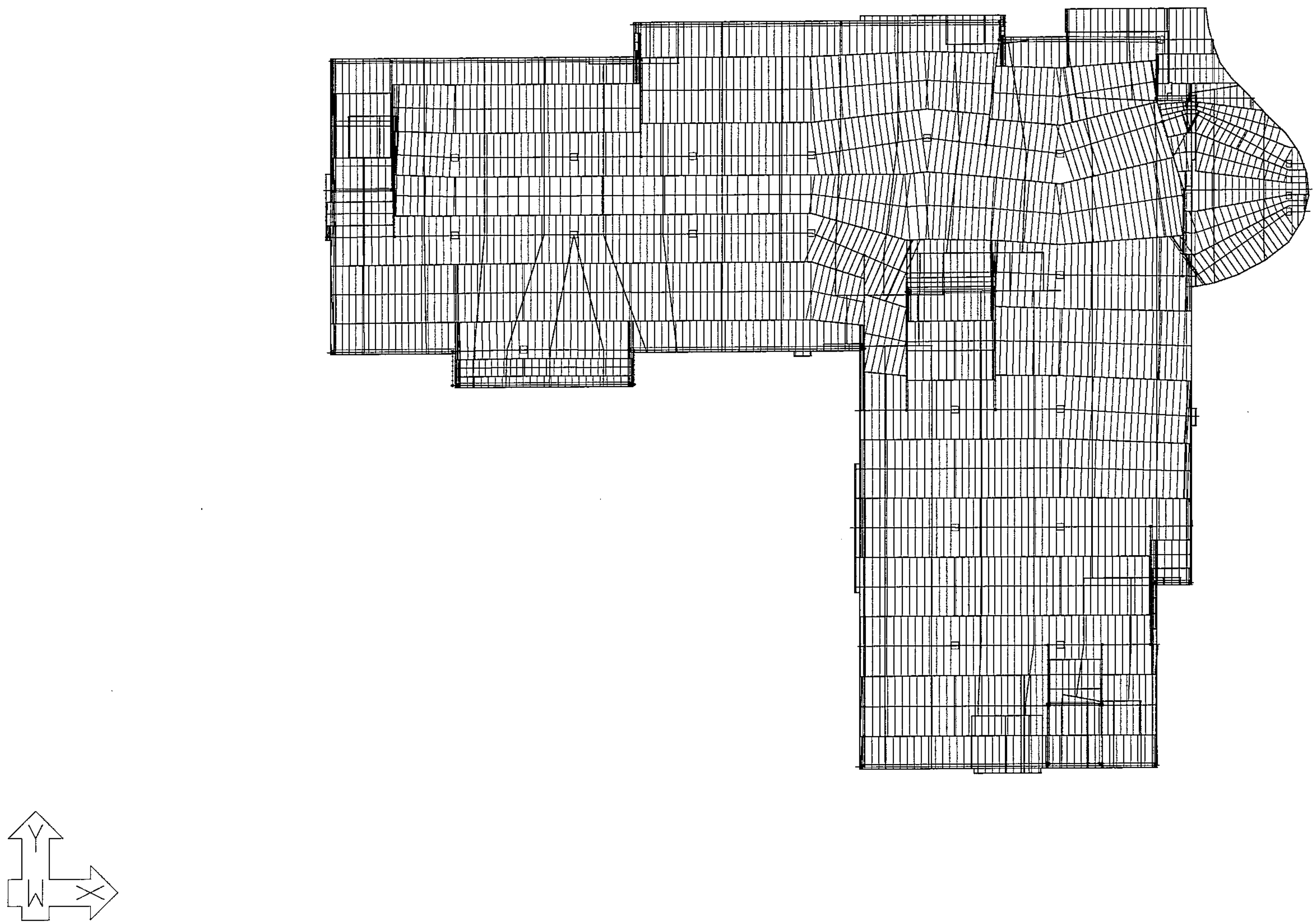

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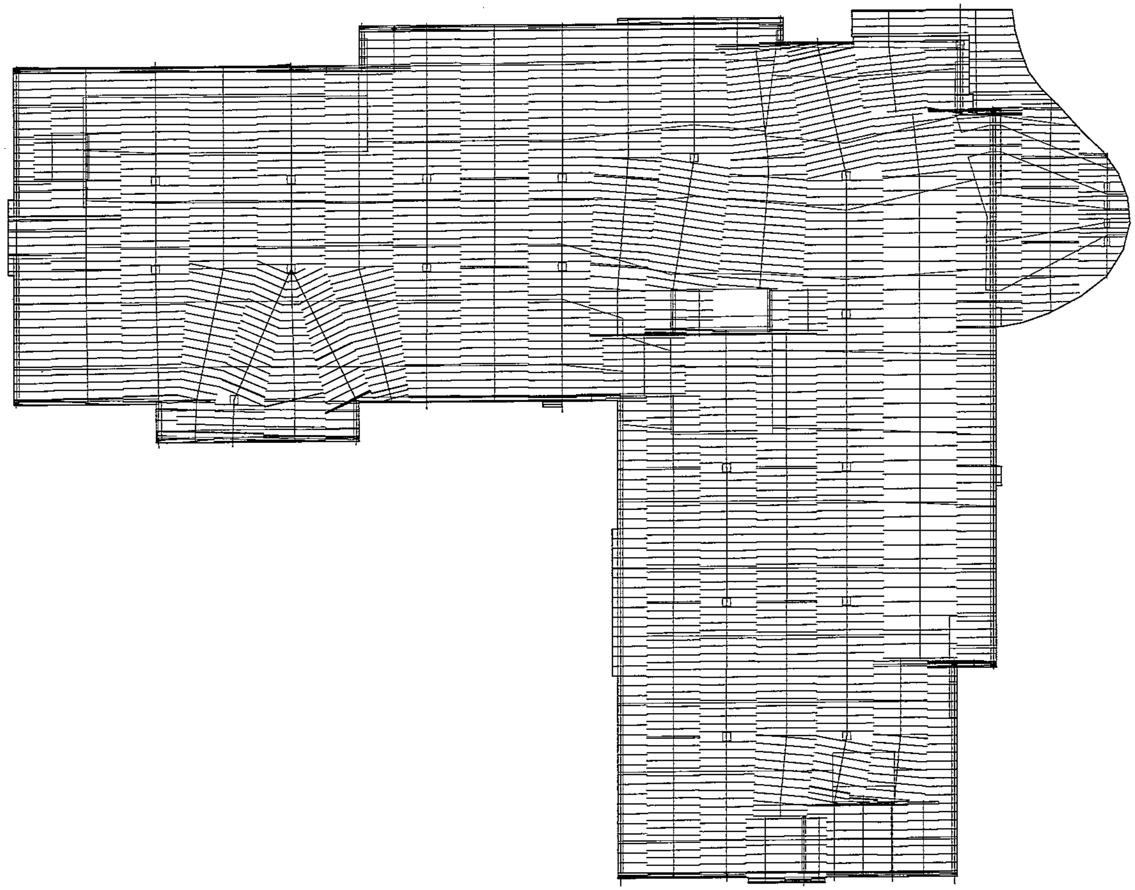

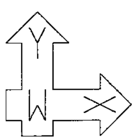

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HOME2 - SSF
DESIGN STRIP - Y

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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 x Selfweight + 1.00 x Dead load + 1.00 x Live load
Name: Service(Sustained Load)
Evaluation: SERVICE SUSTAINED LOAD
Combination detail: 1.00 x Selfweight + 1.00 x Dead load + 0.30 x Live load
Name: Strength(Dead and Live)
Evaluation: STRENGTH
Combination detail: 1.20 x Selfweight + 1.20 x Dead load + 1.60 x Live load
Name: Strength(Dead Load Only)
Evaluation: STRENGTH
Combination detail: 1.40 x Selfweight + 1.40 x Dead load
Name: Strenght (D+L-Skip-Max)
Evaluation: STRENGTH
Combination detail: 1.60 x Skip_Live load_Max + 1.00 x Hyperstatic + 1.20 x Selfweight + 1.20 x Dead load
Name: Strenght (D+L-Skip-Min)
Evaluation: STRENGTH
Combination detail: 1.00 x Hyperstatic + 1.60 x Skip_Live load_Min + 1.20 x Selfweight + 1.20 x Dead load
Name: Initial
Evaluation: INITIAL
Combination detail: 1.15 x Prestressing + 1.00 x Selfweight
Name: 4: 1.2D+WX+L
Evaluation: STRENGTH
Combination detail: 1.20 x Selfweight + 1.20 x Dead load + 1.00 x Live load + 1.00 x Wind-X + 1.00 x Wind_P0
Name: 4: 1.2D-WX+L
Evaluation: STRENGTH
Combination detail: 1.20 x Selfweight + 1.20 x Dead load + 1.00 x Live load -1.00 x Wind-X -1.00 x Wind_P0
Name: 4: 1.2D+WY+L
Evaluation: STRENGTH
Combination detail: 1.20 x Selfweight + 1.20 x Dead load + 1.00 x Live load + 1.00 x Wind-Y + 1.00 x Wind_P90
Name: 4: 1.2D-WY+L
Evaluation: STRENGTH
Combination detail: 1.20 x Selfweight + 1.20 x Dead load + 1.00 x Live load -1.00 x Wind-Y -1.00 x Wind_P90

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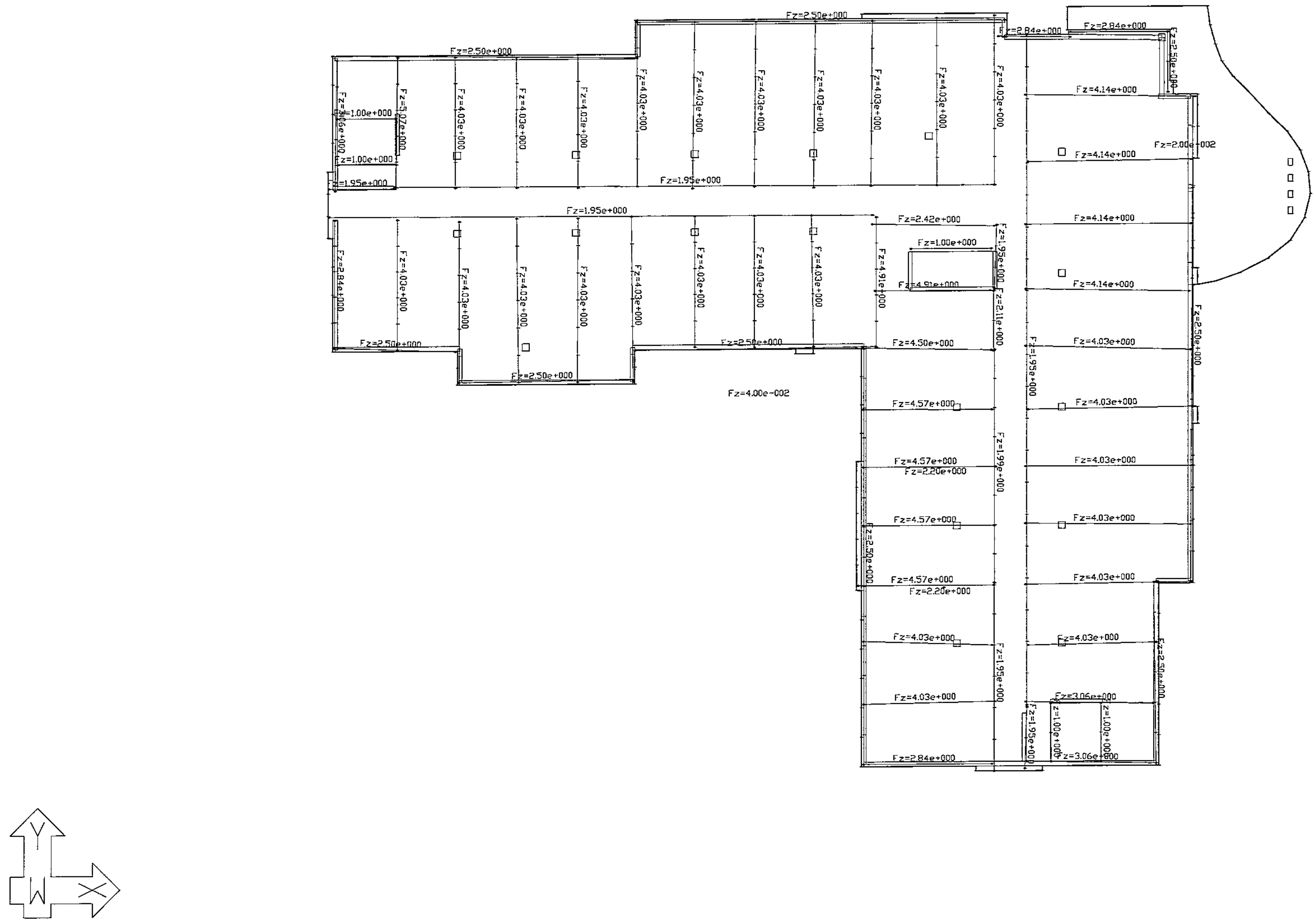
Name: 5: 1.2D+EQX+L
Evaluation: STRENGTH
Combination detail: 1.69 x SEIS-X + 1.20 x Selfweight + 1.20 x Dead load + 1.00 x Live load + 1.69 x EQX + 1.00 x Seis-Podium
Name: 5: 1.2D-EQX+L
Evaluation: STRENGTH
Combination detail: -1.69 x SEIS-X + 1.20 x Selfweight + 1.20 x Dead load + 1.00 x Live load -1.69 x EQX -1.00 x Seis-Podium
Name: 5: 1.2D+EQY+L
Evaluation: STRENGTH
Combination detail: 1.69 x SEIS-Y + 1.20 x Selfweight + 1.20 x Dead load + 1.00 x Live load + 1.69 x EQY + 1.00 x Seis-Podium
Name: 5: 1.2D-EQY+L
Evaluation: STRENGTH
Combination detail: -1.69 x SEIS-Y + 1.20 x Selfweight + 1.20 x Dead load + 1.00 x Live load -1.69 x EQY -1.00 x Seis-Podium
Name: 6: 0.9D+WX
Evaluation: STRENGTH
Combination detail: 0.90 x Selfweight + 0.09 x Dead load + 1.00 x Wind-X + 1.00 x Wind_P0
Name: 6: 0.9D-WX
Evaluation: STRENGTH
Combination detail: 0.90 x Selfweight + 0.09 x Dead load -1.00 x Wind-X -1.00 x Wind_P0
Name: 6: 0.9D+WY
Evaluation: STRENGTH
Combination detail: 0.90 x Selfweight + 0.09 x Dead load + 1.00 x Wind-Y + 1.00 x Wind_P90
Name: 6: 0.9D-WY
Evaluation: STRENGTH
Combination detail: 0.90 x Selfweight + 0.09 x Dead load -1.00 x Wind-Y -1.00 x Wind_P90
Name: 7: 0.9D+EQX
Evaluation: STRENGTH
Combination detail: 1.69 x SEIS-Y + 0.90 x Selfweight + 0.09 x Dead load + 1.69 x EQX + 1.00 x Seis-Podium
Name: 7: 0.9D-EQX
Evaluation: STRENGTH
Combination detail: -1.69 x SEIS-Y + 0.90 x Selfweight + 0.09 x Dead load -1.69 x EQX -1.00 x Seis-Podium
Name: 7: 0.9D+EQY
Evaluation: STRENGTH
Combination detail: 1.69 x SEIS-X + 0.90 x Selfweight + 0.09 x Dead load + 1.69 x EQY + 1.00 x Seis-Podium
Name: 7: 0.9D-EQY
Evaluation: STRENGTH
Combination detail: -1.69 x SEIS-X + 0.90 x Selfweight + 0.09 x Dead load -1.69 x EQY -1.00 x Seis-Podium
Name: Sustained_Load
Evaluation: CRACKED DEFLECTION
Combination detail: 1.00 x Prestressing + 1.00 x Selfweight + 1.00 x Dead load + 0.30 x Live load
Name: Service - DEAD ONLY
Evaluation: SERVICE TOTAL LOAD
Combination detail: 1.00 x Selfweight + 1.00 x Dead load
Name: Service - LIVE ONLY
Evaluation: SERVICE TOTAL LOAD

Combination detail: 1.00 x Live load
Name: Long_Term
Evaluation: LONG TERM DEFLECTION
Combination detail: 4.00 x Sustained_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 SUSTAINED LOAD
Combination detail: 1.00 x SEIS-Y + 1.00 x EQY + 1.00 x Seis-Podium
Name: Service - Wind Only - X Dir
Evaluation: SERVICE SUSTAINED LOAD
Combination detail: 1.00 x Wind-X + 1.00 x Wind_P0
Name: Service - Wind Only - Y Dir
Evaluation: SERVICE SUSTAINED LOAD
Combination detail: 1.00 x Wind-Y + 1.00 x Wind_P90



Loading Plan Dead Load DEAD LOADS

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HOME2 - SSF
DEAD LOADS

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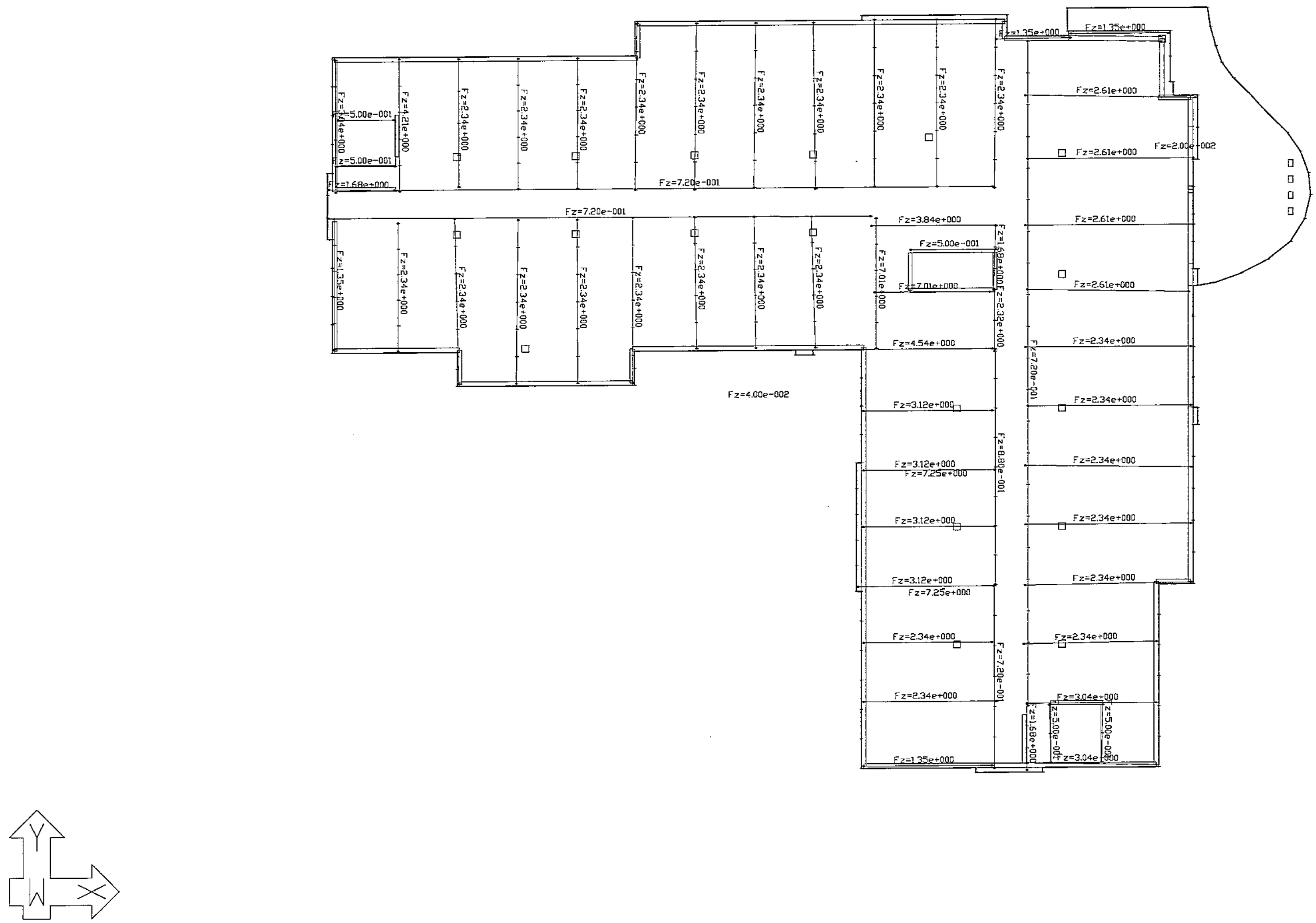


Loading Plan Dead Load DEAD LOADS

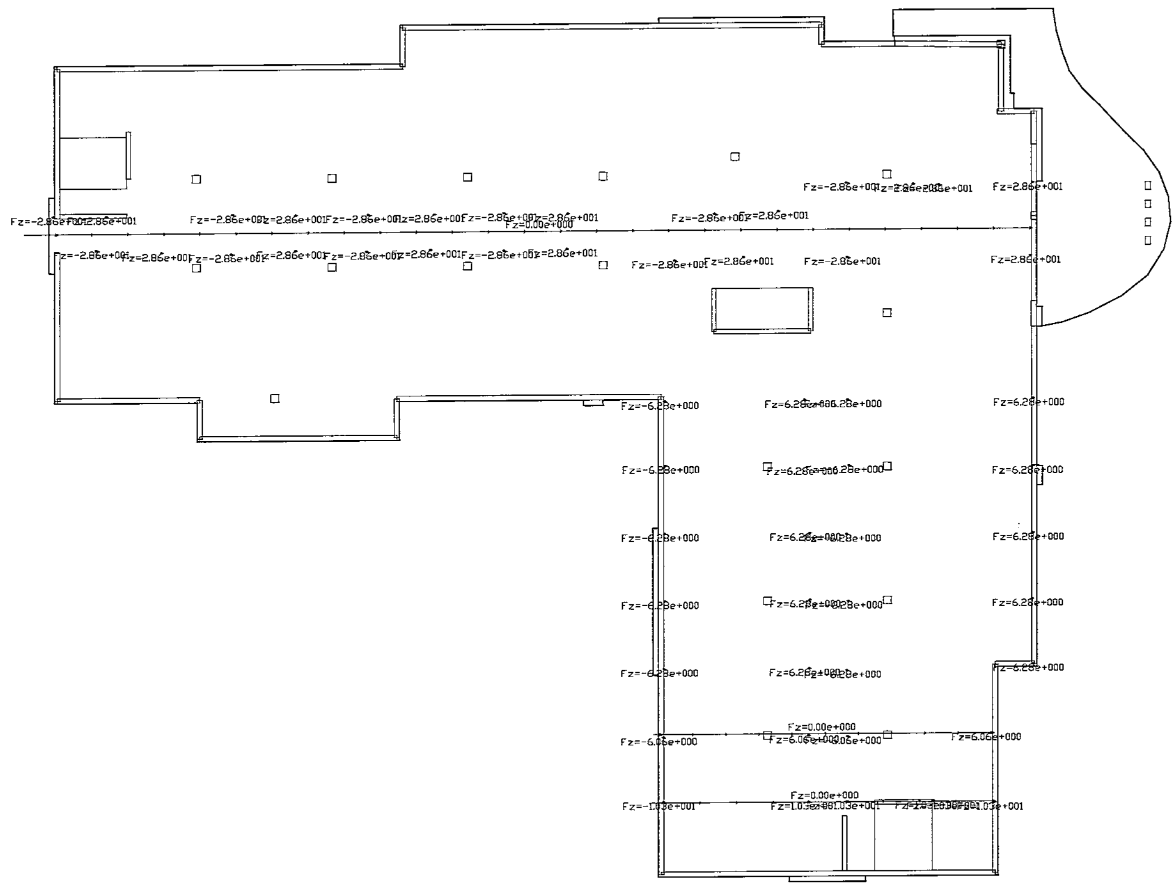
05/28/17
10:56:41

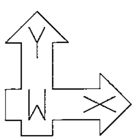
h2ssf-Adapt-170526-1.adm

HOME2 - SSF
LIVE LOADS

(19)

2017-06

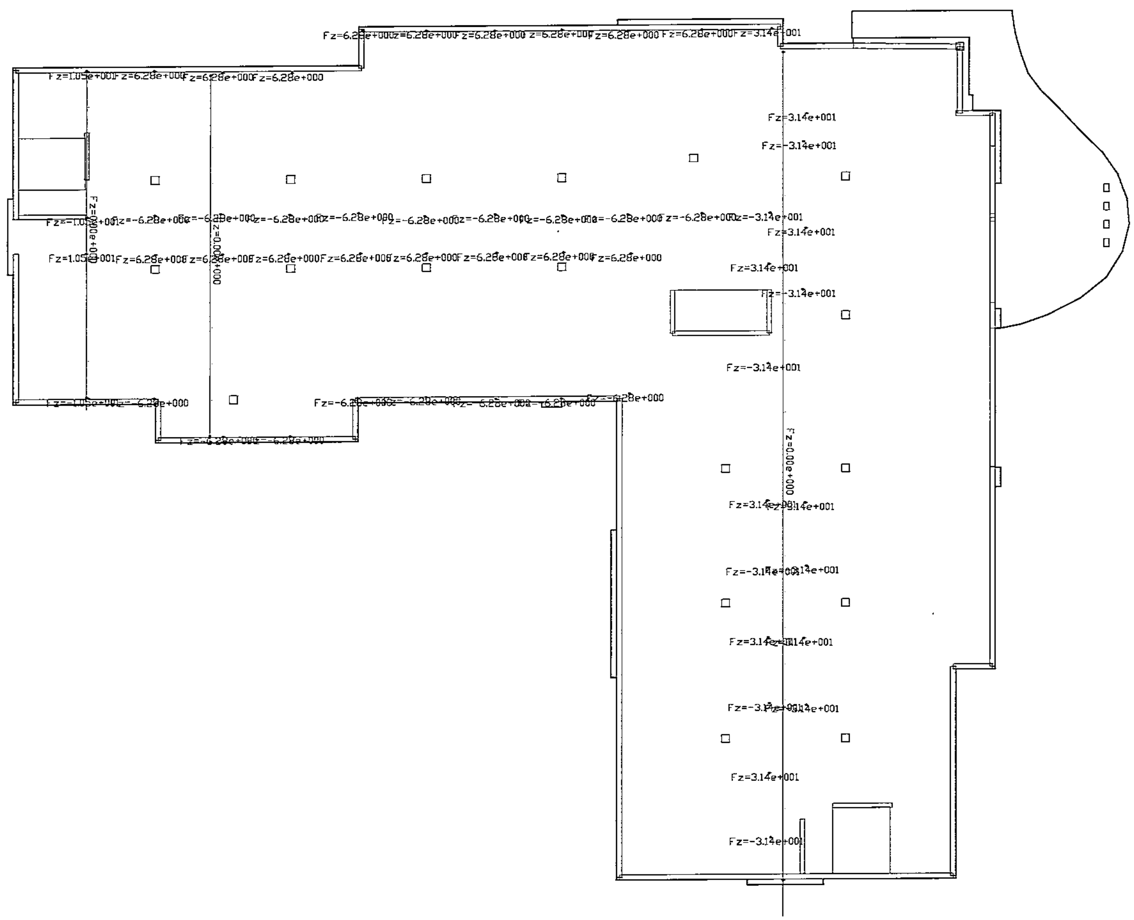


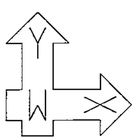


Loading Plan Dead Load DEAD LOADS

05/28/17 10:58:52	HOME2 - SSF LOADS - EQX	
h2ssf-Adapt-170526-1.adm		20

2017-06





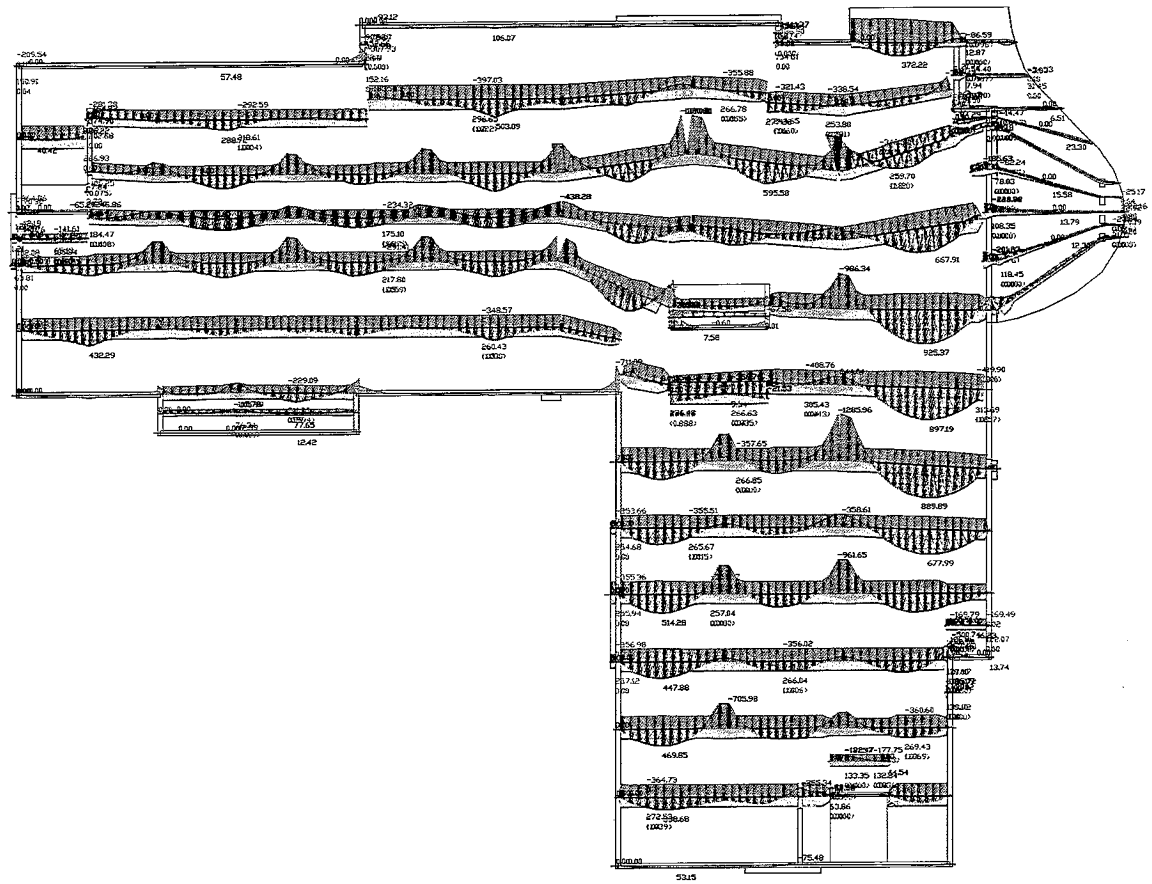
Loading Plan Dead Load DEAD LOADS

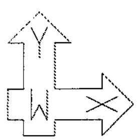
05/28/17 10:59:35	HOME2 - SSF LOADS - EQY
h2ssf-Adapt-170526-1.adm	

(21)

2017-06

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

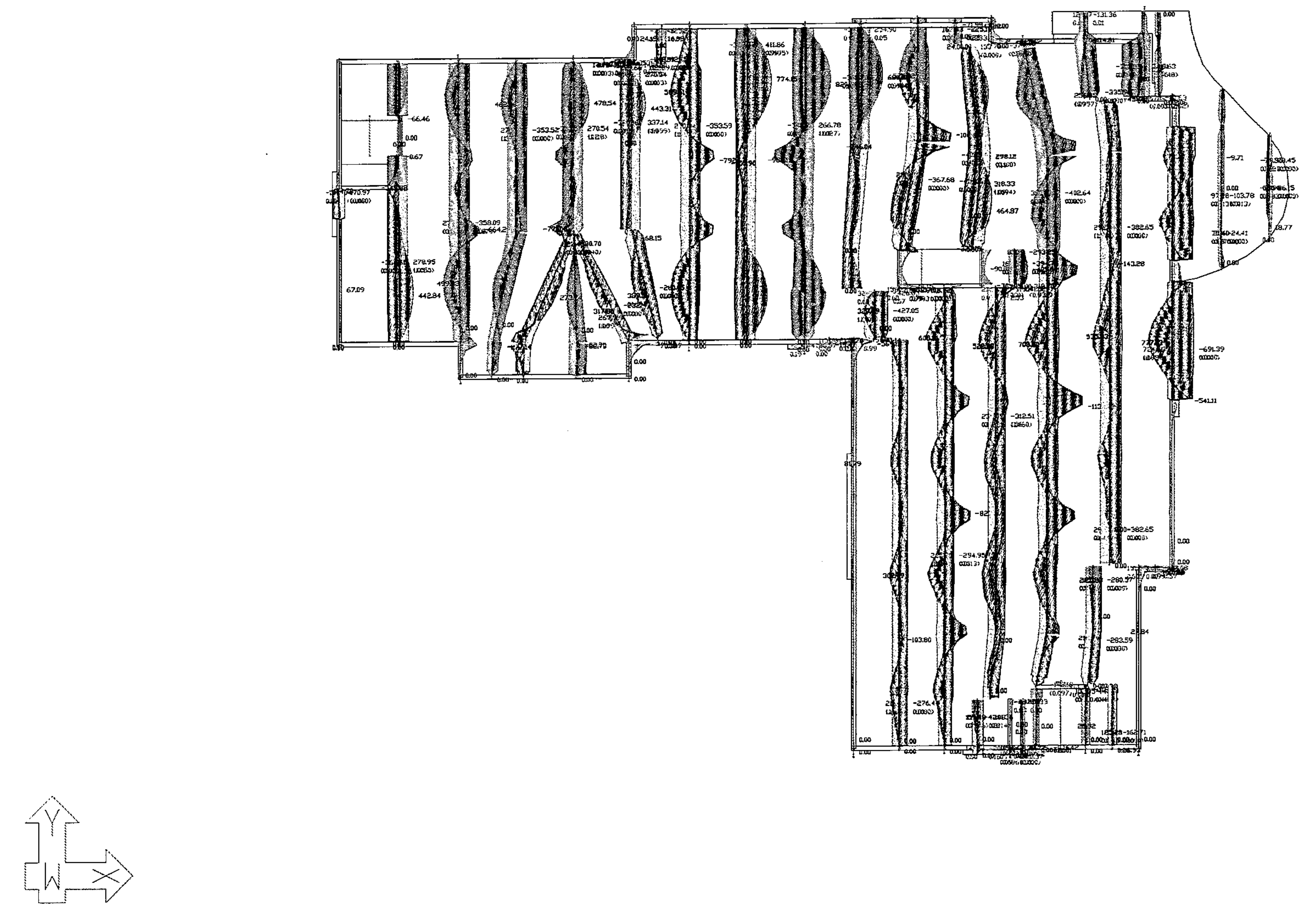




05/28/17 11:26:49	HOME2 - SSF	
h2ssf-Adapt-170526-1.adm	MOMENT CAPACITY w/ DEMAND - X DIR	22

2017-06

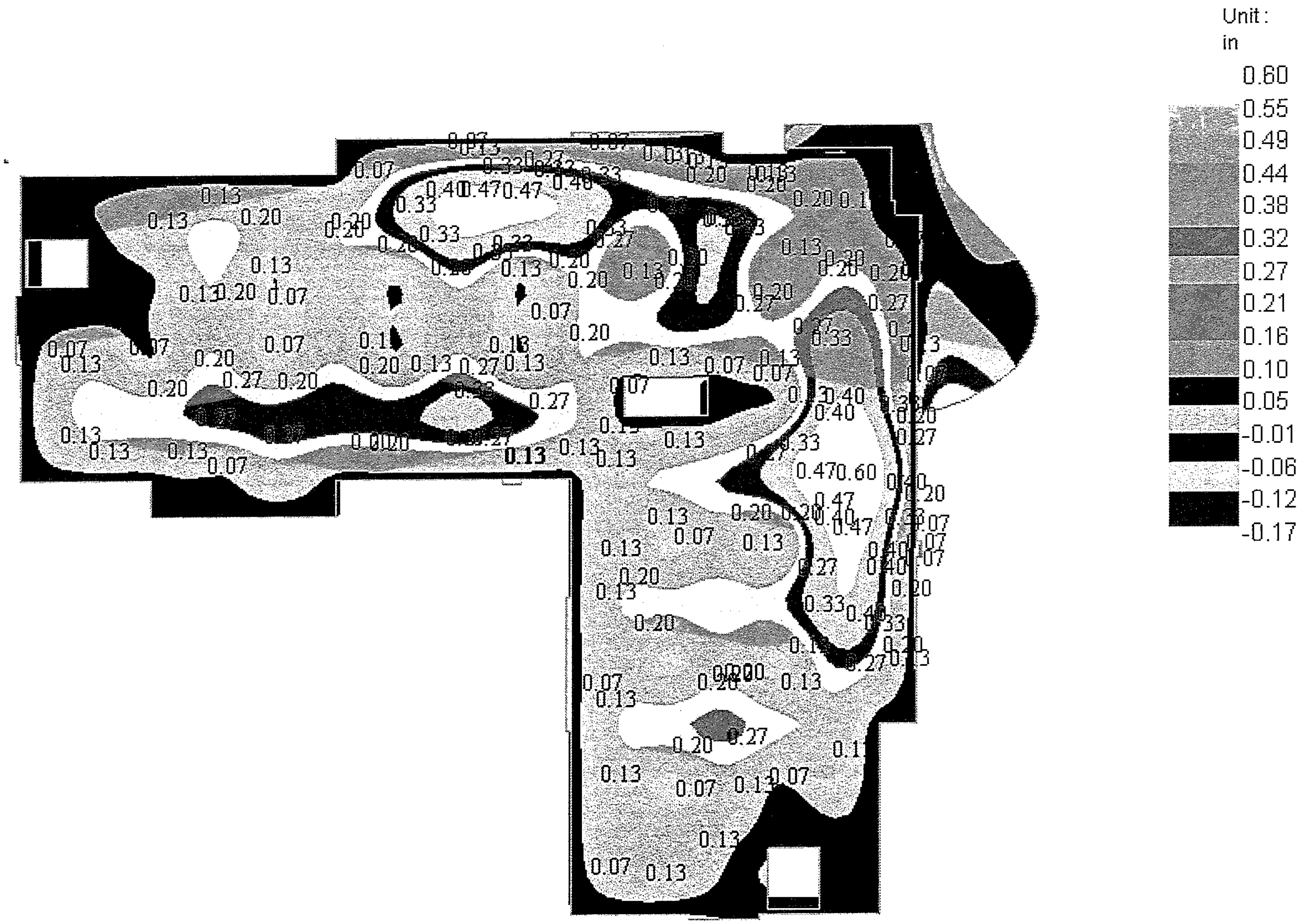
Design Sections,Investigation,Moment Capacity with Demand
Max: 946.30
Min: -1301.80
Max demand/capacity ratio: 1.12



05/28/17 11:28:08	HOME2 - SSF MOMENT CAPACITY w/ DEMAND - Y DIR
h2ssf-Adapt-170526-1.adm	

(22)

2017-06

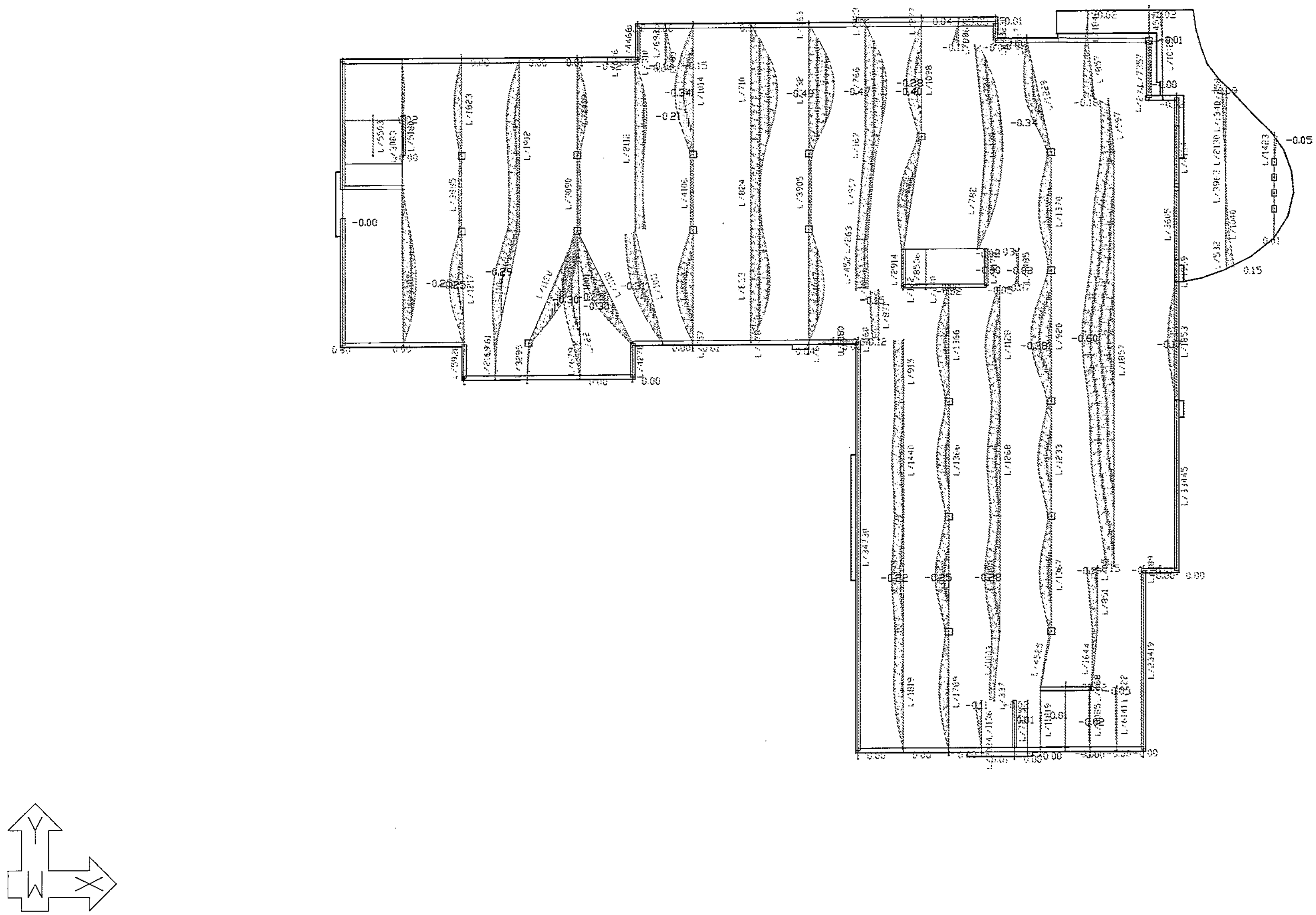


Z-Translation

05/28/17 11:17:35	General	Units: in
h2ssf-Adapt-170526-1.adv	Specifi	Deformation (241)

2017-06

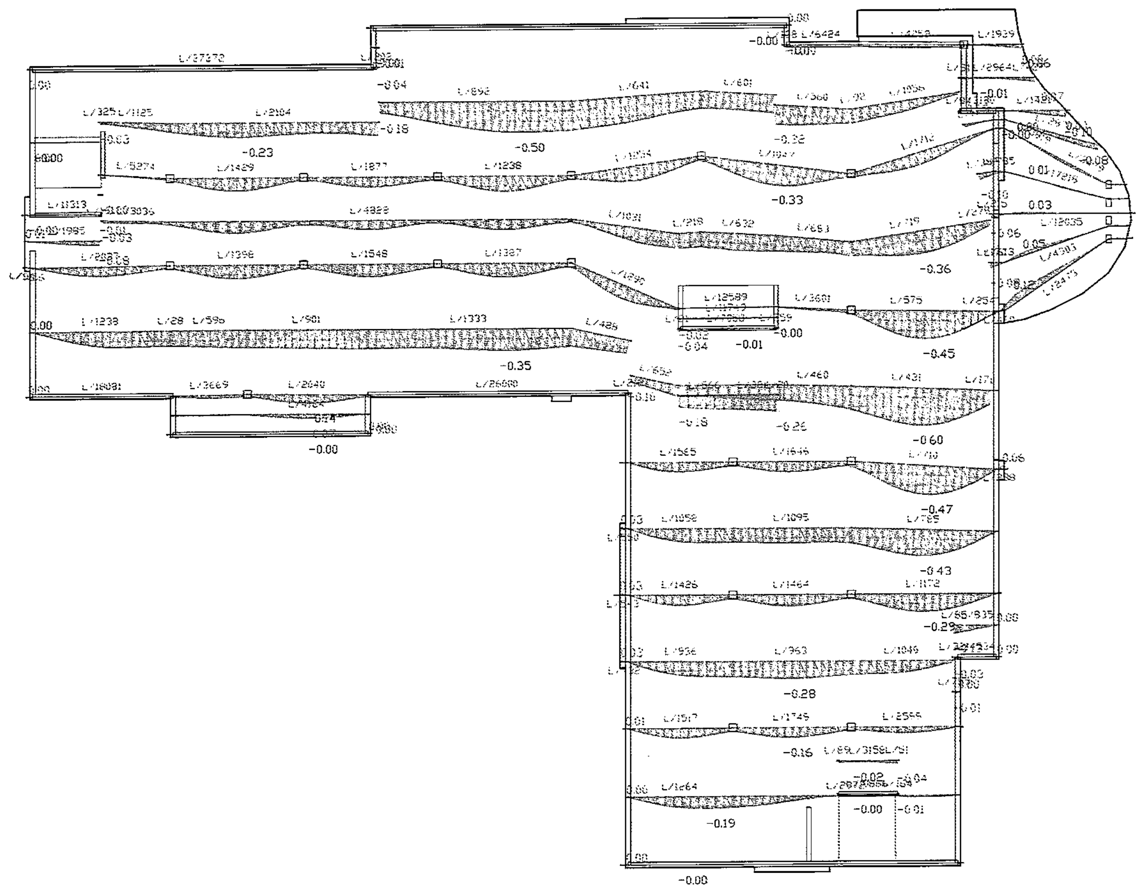
Design Sections,Deformation,Z-Translation(in)
Load Combination: Service(Total Load)(SERVICE_TOTAL_LOAD)
Max allowable: L/360
Max: 0.15
Min: -0.60

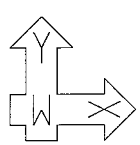


05/28/17 11:14:48	HOME2 - SSF DEFLECTION - Z TRANSLATIONS	
h2ssf-Adapt-170526-1.adm		

2017-06 (18)

Design Sections,Deformation,Z-Translation(in)
Load Combination: Service(Total Load)(SERVICE_TOTAL_LOAD)
Max allowable: L/360
Max: 0.15
Min: -0.60





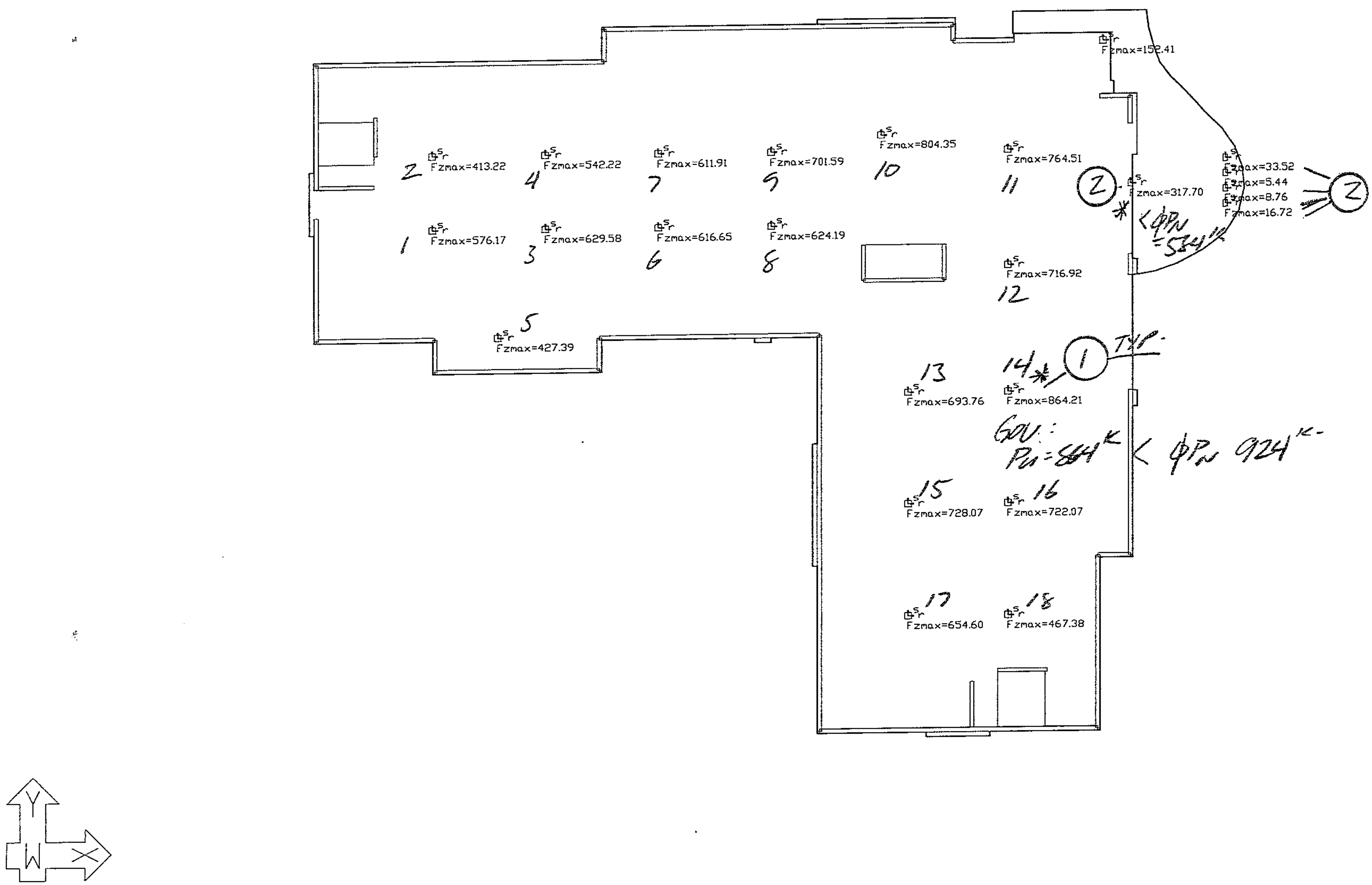
05/28/17
11:14:06

h2ssf-Adapt-170526-1.adm

HOME2 - SSF
DEFLECTION - Z TRANSLATIONS

2017-06
26

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

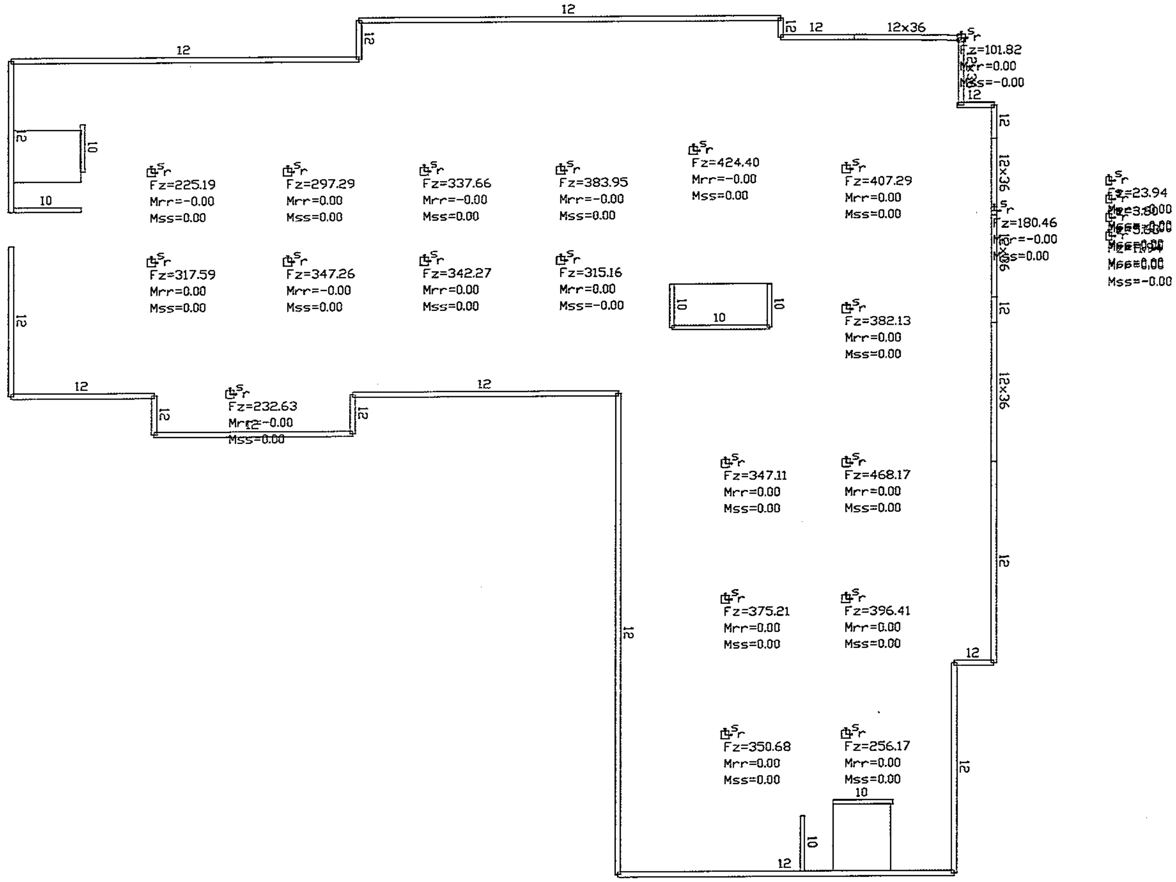


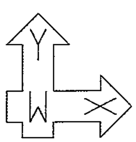
Column Reactions Plan_Envelope COLUMN REACTIONS

05/28/17 11:37:11	HOME2 - SSF COLUMN REACTIONS	27
h2ssf-Adapt-170526-1.adm		

2017-06

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

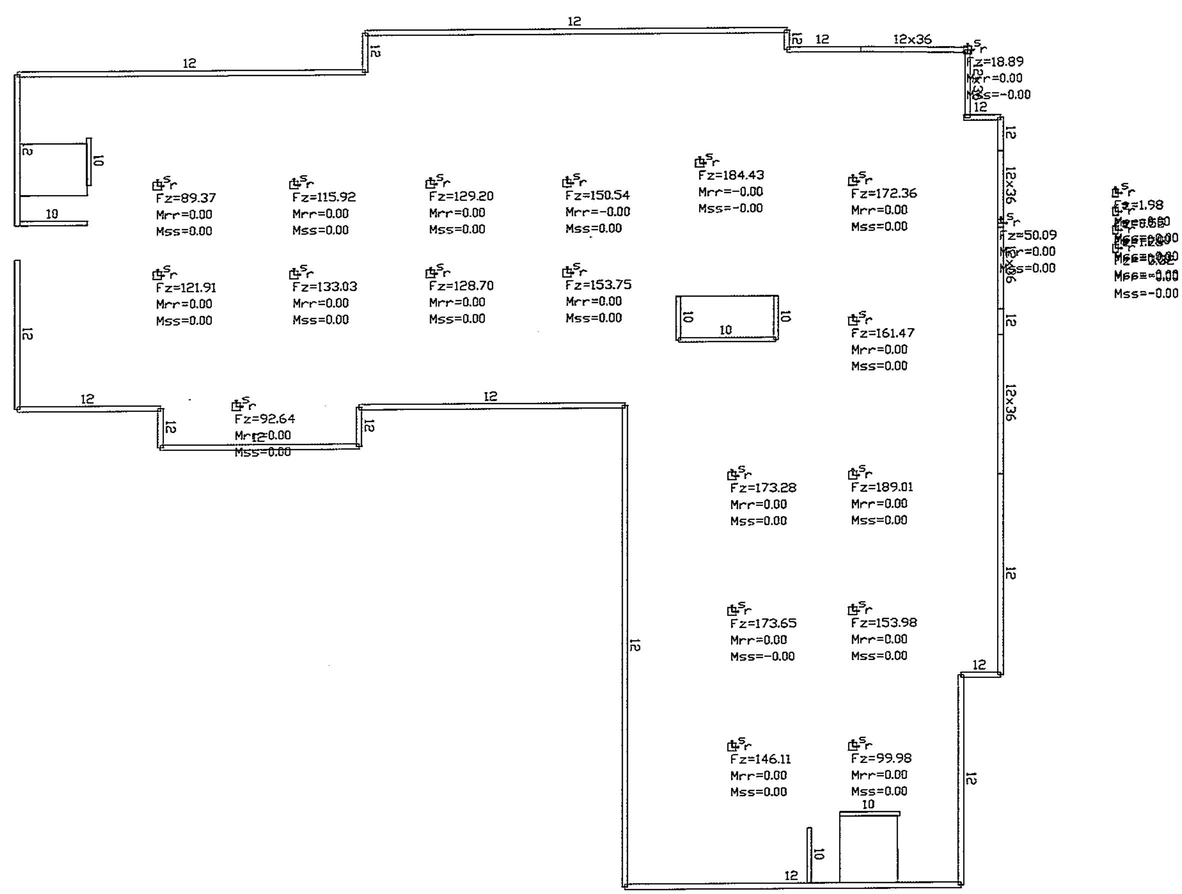


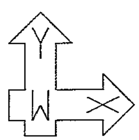


Column Reactions Plan_Service - DEAD ONLY COLUMN REACTION - SERVICE - DEAD LOAD

05/28/17 11:50:23	HOME2 - SSF	
h2ssf-Adapt-170526-1.adm	COLUMN REACTION - Fz - SERVICE = DEAD	28 2017-06

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





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

05/28/17 11:51:44	HOME2 - SSF	
h2ssf-Adapt-170526-1.adm	COLUMN REACTION - Fz - SERVICE = LIVE	2017-06

HCP ENGINEERING
Corona, CA

Project Title: Home2, SSF
Engineer:
Project Descr:

Project ID: 2017-06

30

Printed: 3 JUN 2017, 4:59PM

Concrete Column

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 : TYP. CONC. COL: 18" x 18" 1

Code References

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

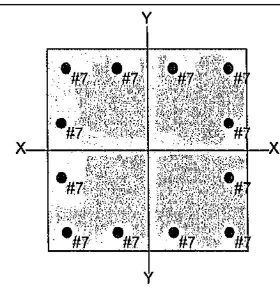
f'c : Concrete 28 day strength = 5.0 ksi
E = = 3,122.0 ksi
Density = 150.0 pcf
β = 0.80
fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi
Allow. Reinforcing Limits *ASTM A615 Bars Used*
Min. Reinf. = 1.0 %
Max. Reinf. = 8.0 %

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 - #7 bars top & bottom between corner bars, 2 - #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 = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu = 661.82 k φ * Pn = 924.77 k
Mu-x = 0.0 k-ft φ * Mn-x = 0.0 k-ft
Mu-y = 0.0 k-ft φ * Mn-y = 0.0 k-ft

Mu Angle = 0.0 deg
Mu at Angle = 0.0 k-ft φMn at Angle = 0.0 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .

Pnmax : Nominal Max. Compressive Axial Capacity 1,778.40 k
Pnmin : Nominal Min. Tension Axial Capacity -432.0 k
φ Pn, max : Usable Compressive Axial Capacity 924.77 k
φ Pn, min : Usable Tension Axial Capacity -280.80 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 . φ = 0.650 β = 0.80 θ = 0.80

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

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	Axial Load k φ * Pn	Bending Analysis k-ft δ x	δ x * Mux	δ y	δ y * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.40D			13.91	661.82	924.77					0.000			**0.716**

Concrete Column

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

Maximum Reactions

Note: Only non-zero reactions are listed.

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

Maximum Moment Reactions

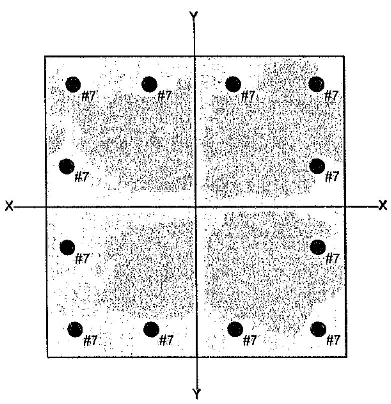
Note: Only non-zero reactions are listed.

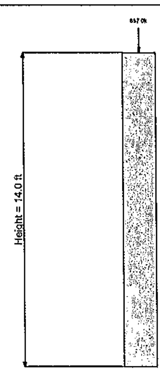
Load Combination	Moment About X-X Axis @ Base	Moment About X-X Axis @ Top		Moment About Y-Y Axis @ Base	Moment About Y-Y Axis @ Top	
D Only			k-ft			k-ft
+D+L			k-ft			k-ft
L Only			k-ft			k-ft

Maximum Deflections for Load Combinations

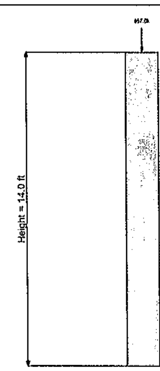
Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Sketches





Looking along X-X Axis



Looking along Y-Y Axis

Interaction Diagrams

Concrete Column

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 : 12" x 18" COL (2)

Code References

Calculations per ACI 318-11, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

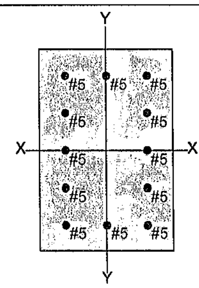
f'c : Concrete 28 day strength	=	5.0 ksi
E =	=	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.80
fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		*ASTM A615 Bars Used*
Min. Reinf.	=	1.0 %
Max. Reinf.	=	8.0 %

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 = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu = 257.810 k φ * Pn = 584.38 k

Mu-x = 0.0 k-ft φ * Mn-x = 0.0 k-ft
Mu-y = 24.071 k-ft φ * Mn-y = 54.182 k-ft

Mu Angle = 90.0 deg
Mu at Angle = 24.071 k-ft φMn at Angle = 54.438 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .

Pnmax : Nominal Max. Compressive Axial Capacity	1,125.39 k
Pnmin : Nominal Min. Tension Axial Capacity	-223.20 k
φ Pn, max : Usable Compressive Axial Capacity	585.20 k
φ Pn, min : Usable Tension Axial Capacity	-145.080 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 . φ = 0.650 β = 0.80 θ = 0.80

ρ : % Reinforcing 1.722 % Rebar % Ok
Reinforcing Area 3.720 in^2
Concrete Area 216.0 in^2

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	Axial Load k φ * Pn	Bending Analysis k-ft δ x	δ x * Mux	δ y	δ y * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization 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

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 : 12" x 18" COL

Maximum Reactions

Note: Only non-zero reactions are listed.

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

Maximum Moment Reactions

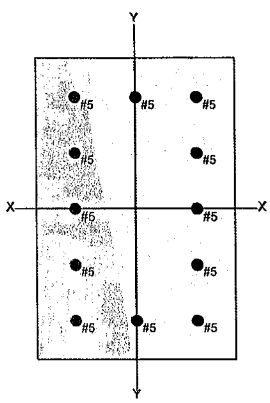
Note: Only non-zero reactions are listed.

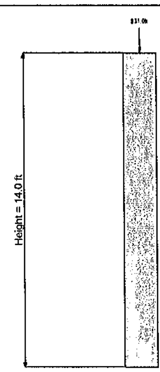
Load Combination	Moment About X-X Axis @ Base	Moment About X-X Axis @ Top		Moment About Y-Y Axis @ Base	Moment About Y-Y Axis @ Top	
D Only			k-ft			k-ft
+D+L			k-ft			k-ft
L Only			k-ft			k-ft

Maximum Deflections for Load Combinations

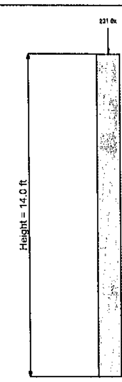
Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Sketches



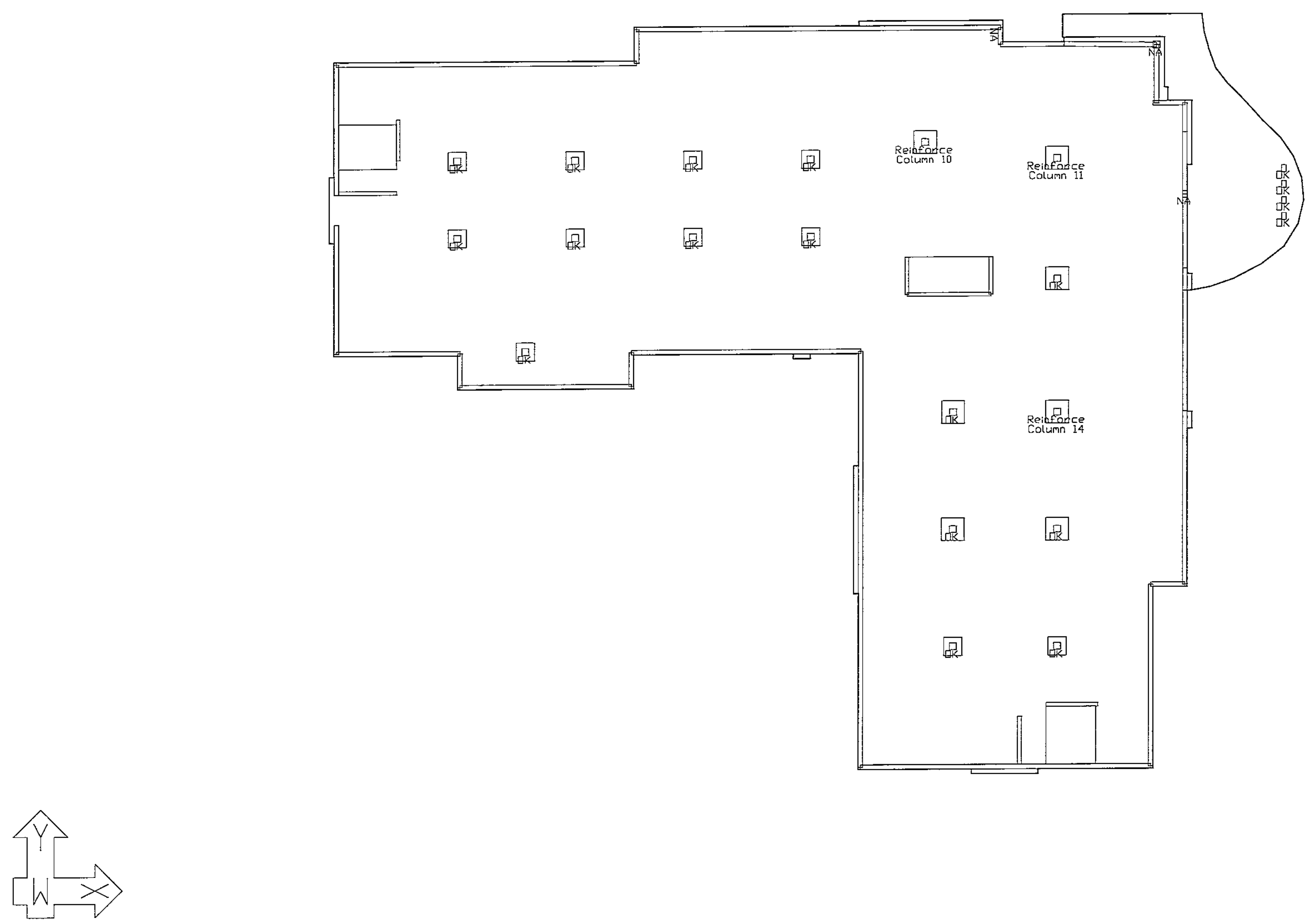


Looking along X-X Axis



Looking along Y-Y Axis

Interaction Diagrams

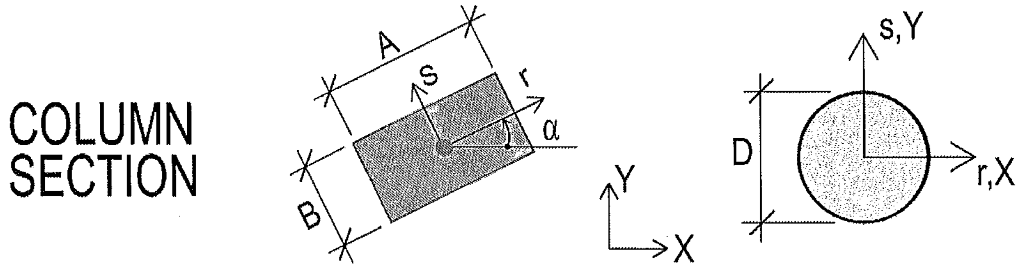


Wall Reactions Plan_Strength(Dead Load Only)

08/10/17 17:54:26	HOME2 - SSF PUNCHING SHEAR	
h2ssf-Adapt-170526-1.adm		(34) 2017-06

Project Name: HOME2 - SSF **Specific Data: PUNCHING SHEAR**
Date of execution: May 28, 2017 **File Name: h2ssf-Adapt-170526-1.adm** **FLOOR-PRO 2016**

180.60 PUNCHING SHEAR STRESS CHECK PARAMETERS



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: Strenght (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: Strenght (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

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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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.

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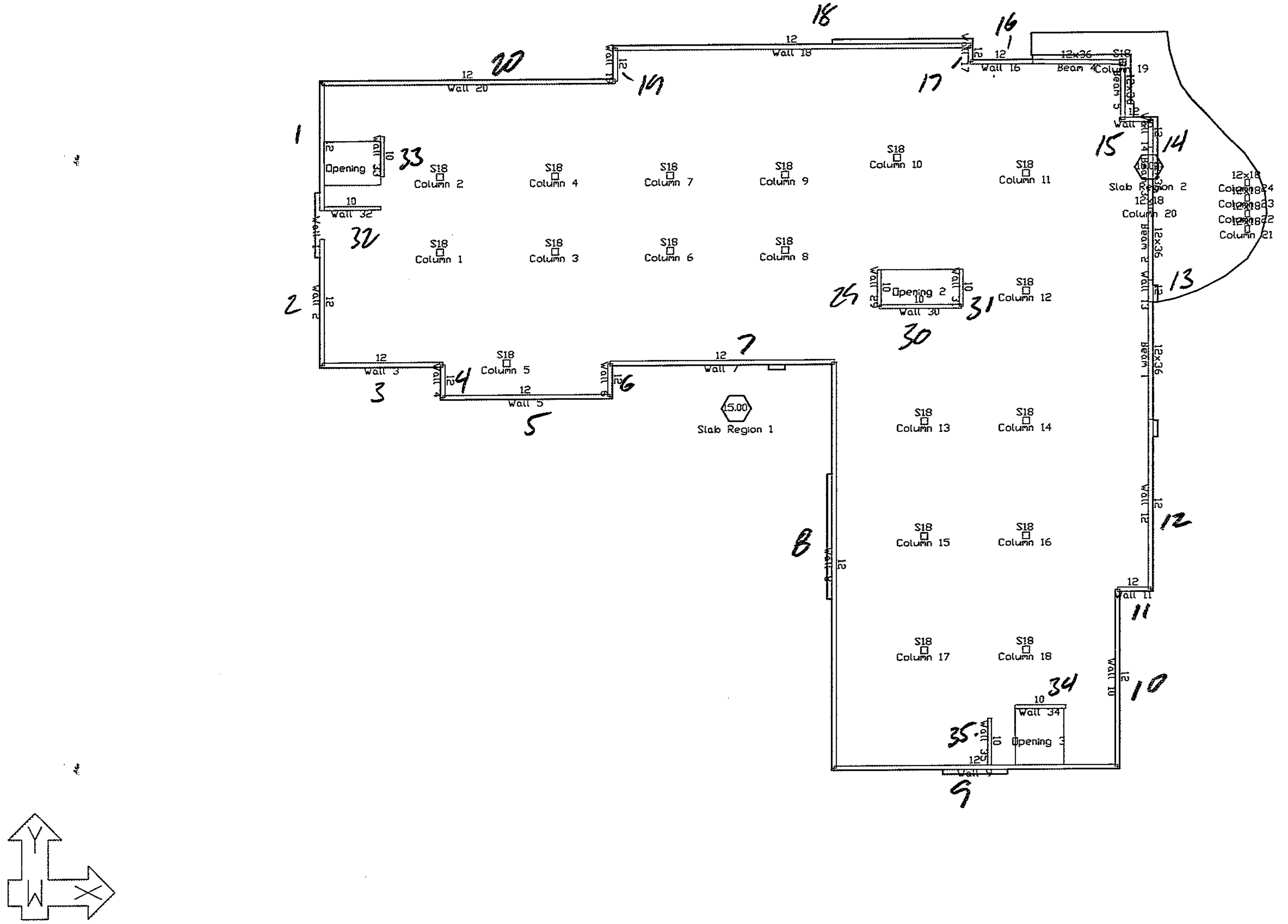
Walls



05/28/17 10:43:03	HOME2 - SSF MODEL
h2ssf-Adapt-170526-1.adm	

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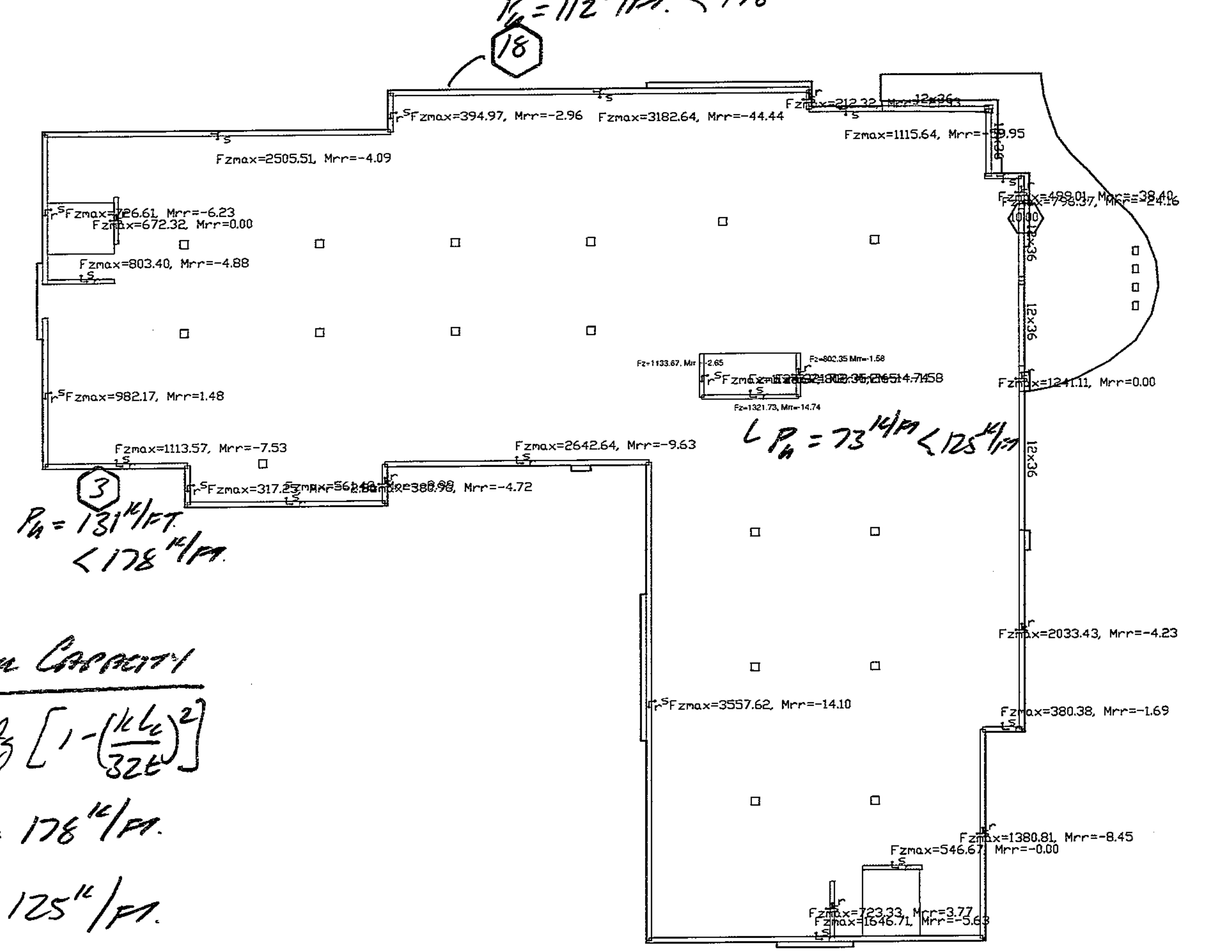
CHECK BEARING WALL CAPACITY

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

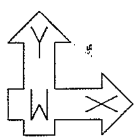


BEARING WALL CAPACITY

$$P_a' = 0.55 f'_c A_g \left[1 - \left(\frac{k l_c}{32t}\right)^2\right]$$

12" WALL : $P_A = 178^{k}/ft.$

10" WALL : $P_A = 125^{k}/ft.$



Wall Reactions Plan_Envelope WALL REACTIONS

05/28/17 11:39:26	HOME2 - SSF
h2ssf-Adapt-170526-1.adm	WALL REACTION - Fz _Mrr

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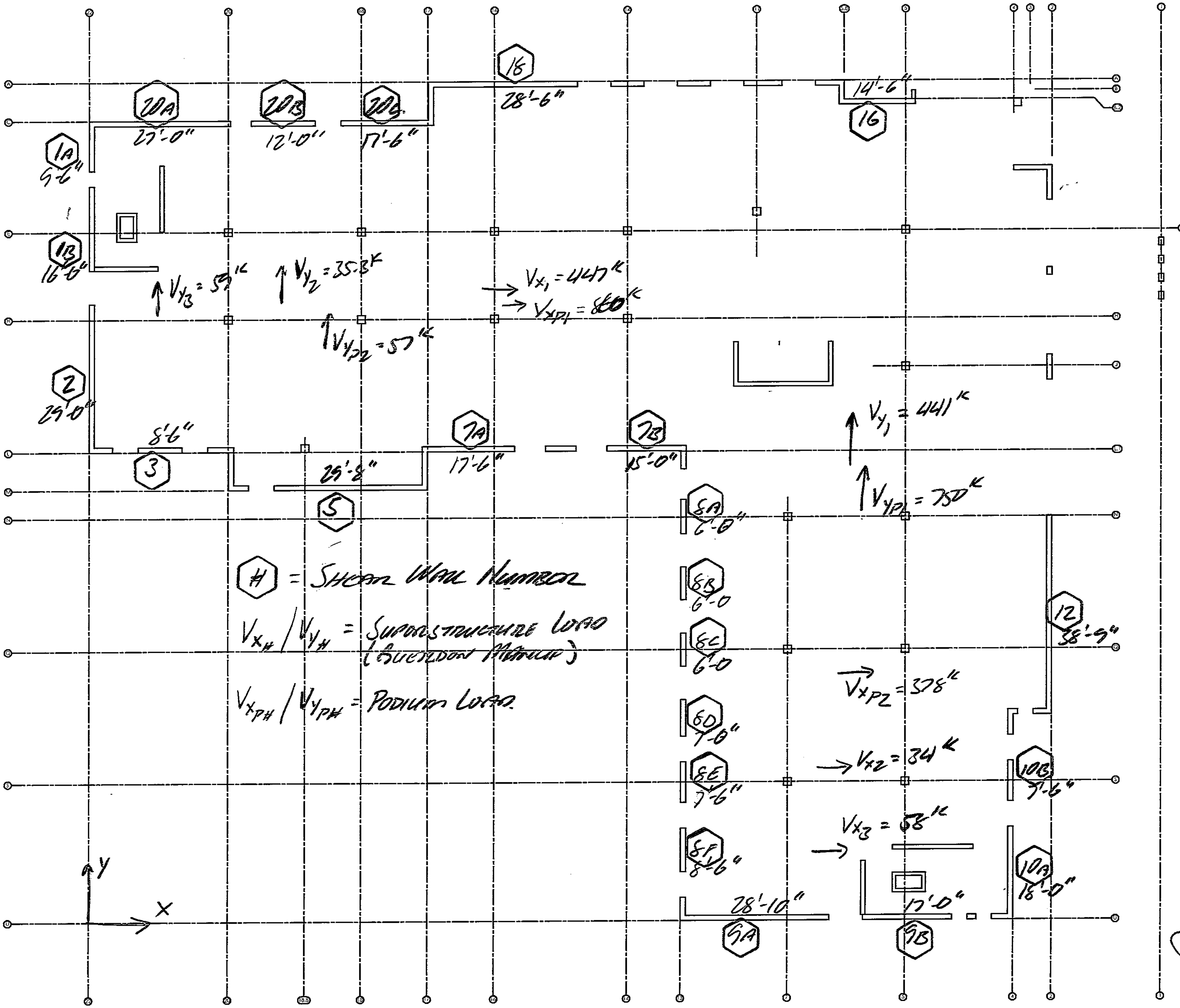

(#) = Shear Wall Number
$V_{x\#}$ / $V_{y\#}$ = Superstructure Load (Buildown Manual)
$V_{xP\#}$ / $V_{yP\#}$ = Podium Load
V_{x_1} = 447k
V_{xP1} = 880k
V_{y_1} = 441k
V_{yP1} = 750k
V_{y_2} = 35.3k
V_{yP2} = 57k
V_{y_3} = 57k
V_{xP2} = 378k
V_{x_2} = 34k
V_{x_3} = 58k
1A 9'-6"
1B 16'-0"
2 29'-0"
3 8'-6"
5 25'-8"
7A 17'-6"
7B 15'-0"
8A 6'-0"
8B 6'-0
8C 6'-0
8D 7'-0"
8E 7'-6"
8F 8'-6"
9A 28'-10"
9B 17'-0"
10A 18'-0"
10B 7'-6"
12 38'-9"
16 14'-6"
18 28'-6"
20A 27'-0"
20B 12'-0"
20C 17'-6"
X
Y

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(4D)

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

Description : Shear Walls

General Information

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

Applied Lateral Force in "X" Direction	2,148.0 k
Applied Lateral Force in "Y" Direction	2,143.0 k

Note: These loads are resolved into X & Y components when applied to the system of elements at angular increments.

Load Orientation Angular Increment	15.0 deg
Load Location Angular Increment	15.0 deg

Center of Rigidity Location (calculated) . . .

"X" dist. from Datum	111.816 ft
"Y" dist. from Datum	102.988 ft

Center of Shear Application :

Distance from "X" datum point	120.80 ft
Distance from "Y" datum point	103.90 ft

Accidental Torsion values per ASCE 7-05 12.8.4.2

Ecc. as % of Maximum Dimension	5.00 %

Maximum Dimensions :

Along "X" Axis	187.0 ft
Along "Y" Axis	164.0 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	Wall Fixity	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	Wall Fixity	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	Wall Fixity	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	Wall Fixity	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	Wall Fixity	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	Wall Fixity	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	Wall Fixity	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	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	3.1589E-002 in			E - Shear	1 Mpsi

Torsional Analysis of Rigid Diaphragm

Description : Shear Walls

Wall Information

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

Torsional Analysis of Rigid Diaphragm

Description : Shear Walls

Wall Information

Label : 8F		X Wall C.G. Location	115.5 ft	Length	8.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :		Y Wall C.G. Location	14.17 ft	Height	14 ft
		Wall Angle CCW	90 deg	Thickness	12 in
Along Wall "y" Dir	1.6541E-003 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	1.0777E-001 in			E - Shear	1 Mpsi
Label : 9A		X Wall C.G. Location	129 ft	Length	28.8 ft
Wall Deflections (Stiffness) for 1.0 kip load :		Y Wall C.G. Location	0 ft	Height	14 ft
		Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "y" Dir	8.6901E-005 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	3.1808E-002 in			E - Shear	1 Mpsi
Label : 9B		X Wall C.G. Location	158.5 ft	Length	17 ft
Wall Deflections (Stiffness) for 1.0 kip load :		Y Wall C.G. Location	0 ft	Height	14 ft
		Wall Angle CCW	0 deg	Thickness	12 in
Along Wall "y" Dir	2.6853E-004 in	Wall Fixity	Fix-Pin	E - Bending	1 Mpsi
Along Wall "x" Dir	5.3886E-002 in			E - Shear	1 Mpsi

ANALYSIS SUMMARY

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

	Max Shear along Member Local "y-y" Axis				Max Shear along Member Local "x-x" Axis			
Resisting Element	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	Load Angle	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**

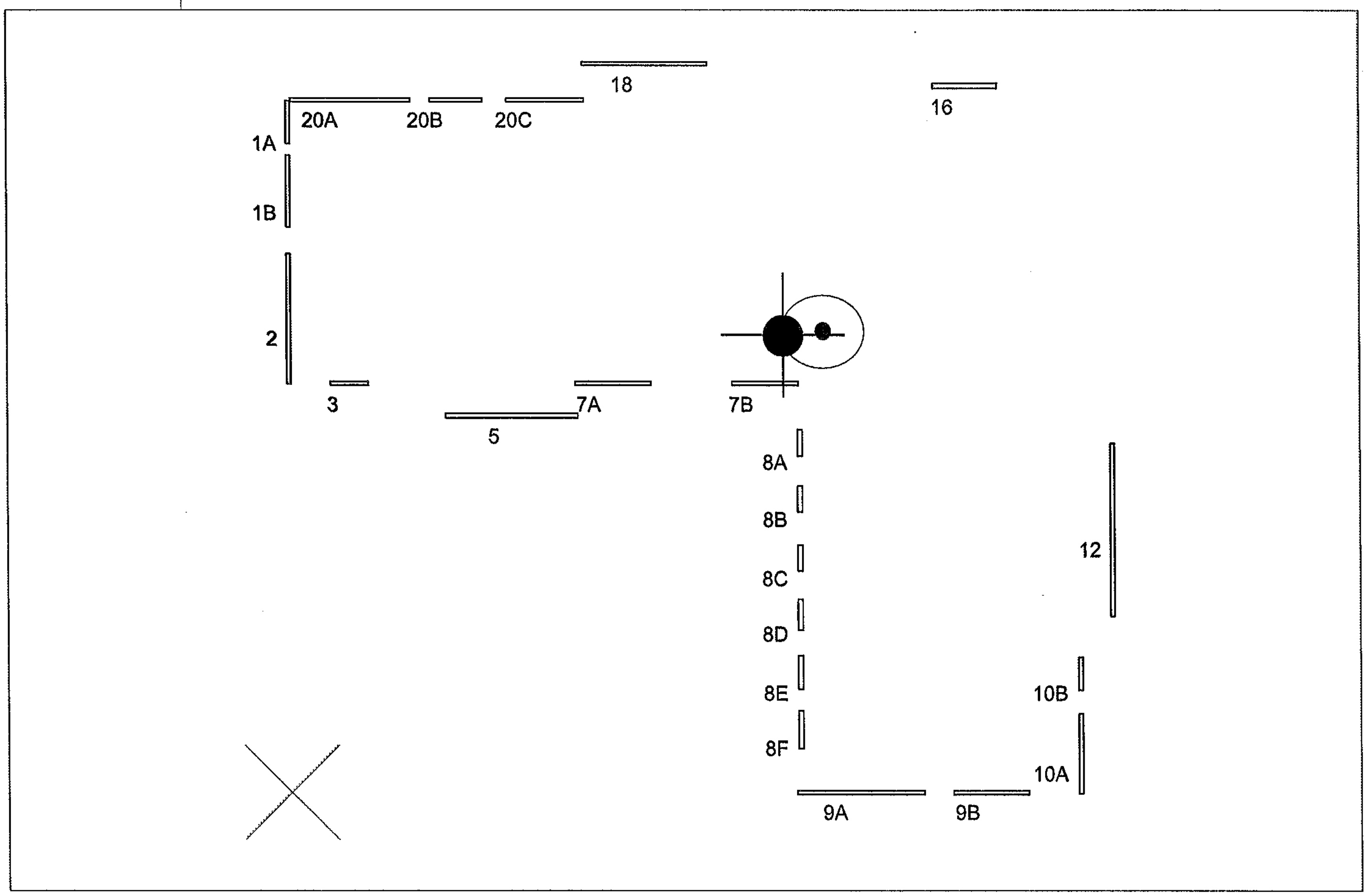
Torsional Analysis of Rigid Diaphragm

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 — X Datum



Torsional Analysis of Rigid Diaphragm

Description : Shear Walls

Analysis Notes

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

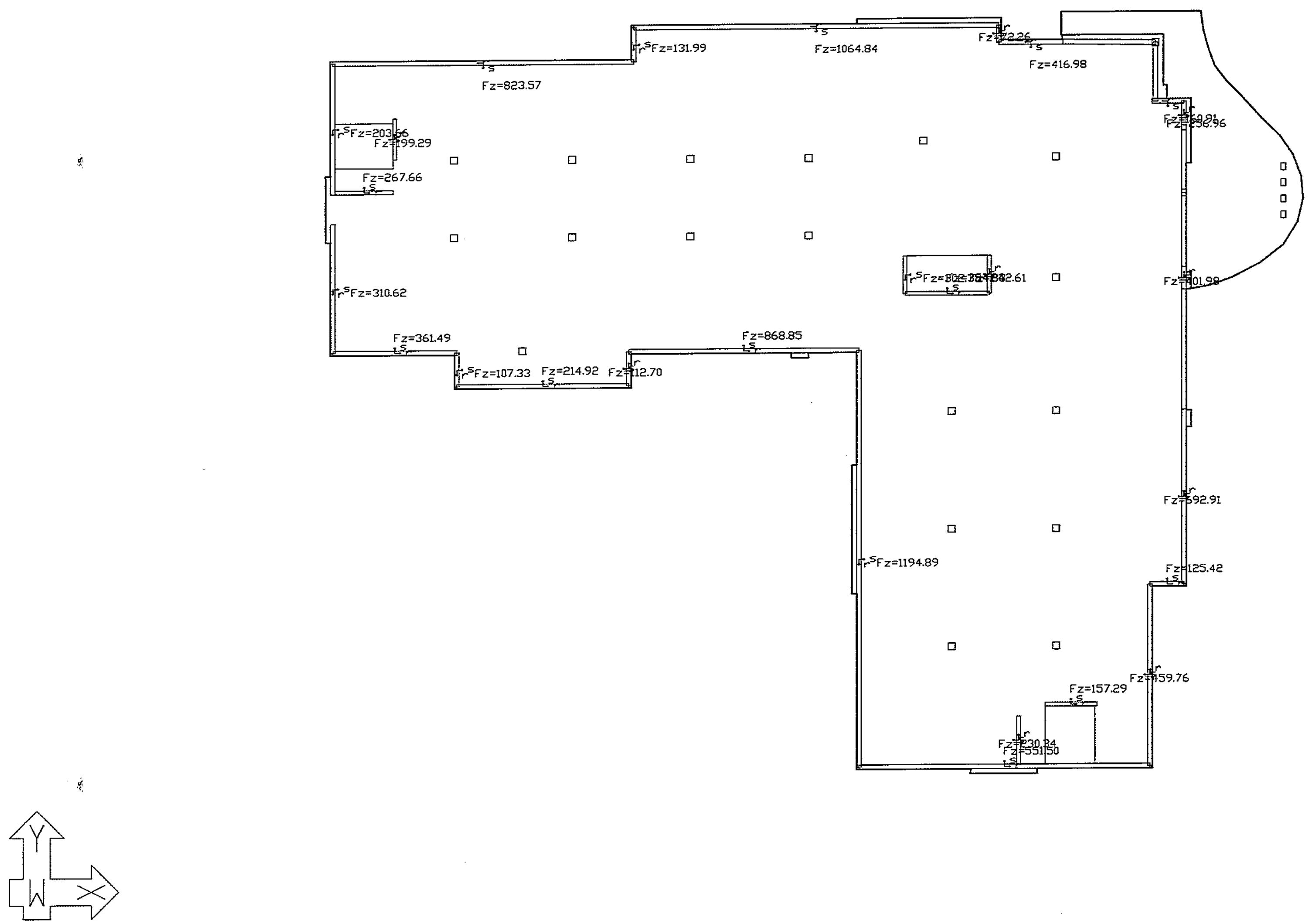
Each resisting element data entry specifies a deflection along a "major" and "minor" axis due to a 1,000 lb load. Each resisting element may be entered as a wall or a column (whereby the deflection is calculated), or as a generic resisting element with specified deflection. The deflections define the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



Wall Reactions Plan_Strength(Dead Load Only)

06/03/17 15:27:06	HOME2 - SSF
h2ssf-Adapt-170526-1.adm	WALL REACTIONS - Fz _Mrr - SERVICE LOADS

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SHEAR WALL SUMMARY

				Pu (DEAD LOAD)		Vu			BOUNDRY ELEMENT					ZONE TYPE
WALL	LENGTH	HEIGHT	THICK	k	k/ft	k	k/ft	Mu	D	B	BULB R/F	HOOP - B	HOOP - D	
1A	9.5	14	12	74.2	7.8	42.0	4.42	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	24	24	10-#5	2-#4	3-#4	10
2	29	14	12	311.0	10.7	595.0	20.52	17,255.0	24	24	10-#5	2-#4	3-#4	10
3	8.5	14	12	361.0	42.5	19.0	2.24	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	24	24	10-#5	2-#4	3-#4	10
7A	17.5	14	12	468.5	26.8	125.0	7.14	2,187.5	24	24	10-#5	2-#4	3-#4	10
7B	15	14	12	401.5	26.8	86.0	5.73	1,290.0	24	24	10-#5	2-#4	3-#4	10
8A	6	14	12	179.3	29.9	11.0	1.83	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	12	12	6-#5	2-#4	3-#4	6
8C	6	14	12	179.3	29.9	11.0	1.83	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	12	12	6-#5	2-#4	3-#4	6
8E	7.5	14	12	224.1	29.9	22.0	2.93	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	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	24	24	10-#5	2-#4	3-#4	10
9B	17	14	12	204.1	12.0	117.0	6.88	1,989.0	24	24	10-#5	2-#4	3-#4	10
10A	18	14	12	324.7	18.0	237.0	13.17	4,266.0	24	24	10-#5	2-#4	3-#4	10
10B	7.5	14	12	135.3	18.0	24.0	3.20	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	36	24	18-#5	2-#4	3-#4	18
16	14.5	14	12	147.0	10.1	84.0	5.79	1,218.0	24	24	10-#5	2-#4	3-#4	10
18	28.5	14	12	1,064.0	37.3	376.0	13.19	10,716.0	24	24	10-#5	2-#4	3-#4	10
20A	17.5	14	12	255.2	14.6	336.0	19.20	5,880.0	24	24	10-#5	2-#4	3-#4	10
20B	12	14	12	175.0	14.6	51.0	4.25	612.0	12	12	6-#5	2-#4	3-#4	6
20C	27	14	12	393.8	14.6	269.0	9.96	7,263.0	24	24	10-#5	2-#4	3-#4	10

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PROJECT : WALL 1A
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REVIEW BY :

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Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

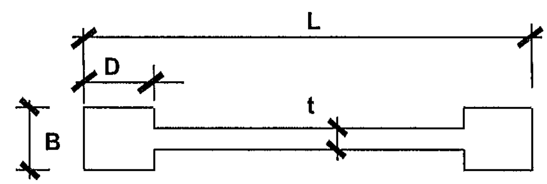
INPUT DATA

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

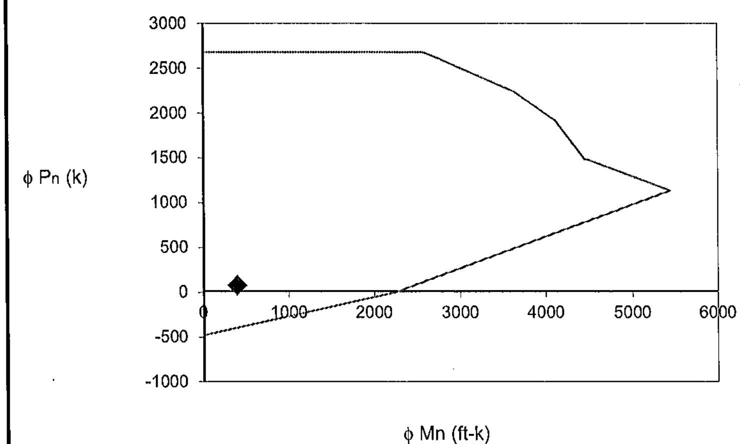


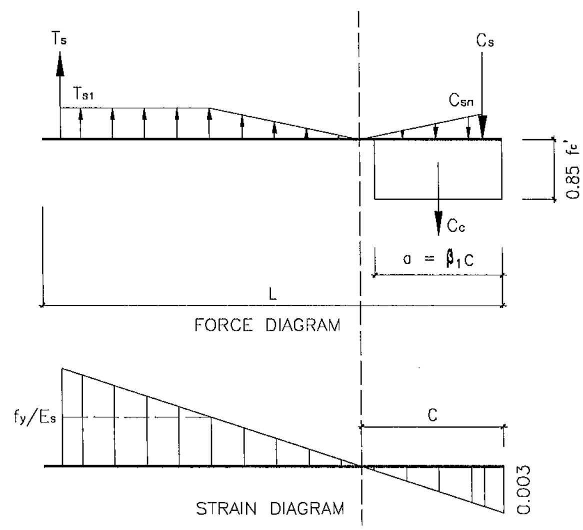
ANALYSIS

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

Pu = 74.2 k < $0.35 A_g f_c'$ = 1915 k [Satisfactory]

where A_g = 1368 in^2.





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 > Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0012 [for $A_{cv} (f_c')^{0.5}$ = 86.52 kips > Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 1368 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)

ϕV_n =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 ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 1.47 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

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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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 8.99 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 64 in $a = C_b \beta_1$ = 54 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 108 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y , 3.3 f_c'^{0.5} 4 L t)$ = -485 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 ε_t = 0.002	=	1489	4443
AT BALANCED CONDITION	=	1476	4493
AT ε_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

ϕM_n = 2522 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

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

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\, y / I)$ = 0.226 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 57 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 1440 in^2. (area of transformed section.)

I = 1559866 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.013 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = MAX(c - 0.1L , 0.5 c) = 6.44 in. (ACI 318-14 18.10.6.4)

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

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

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

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

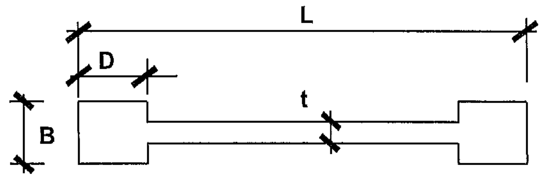
INPUT DATA

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

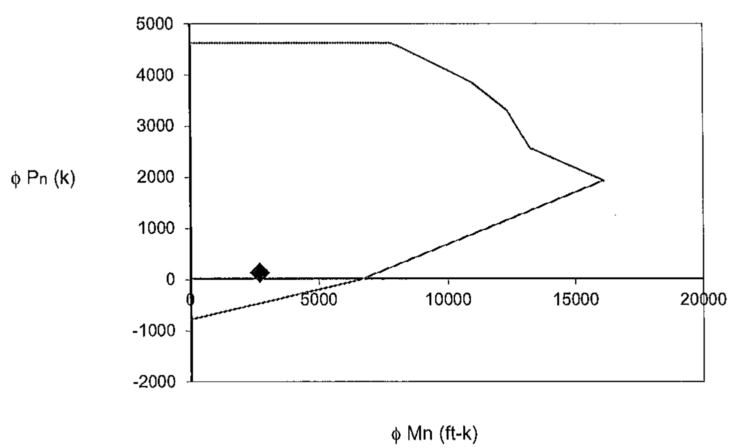


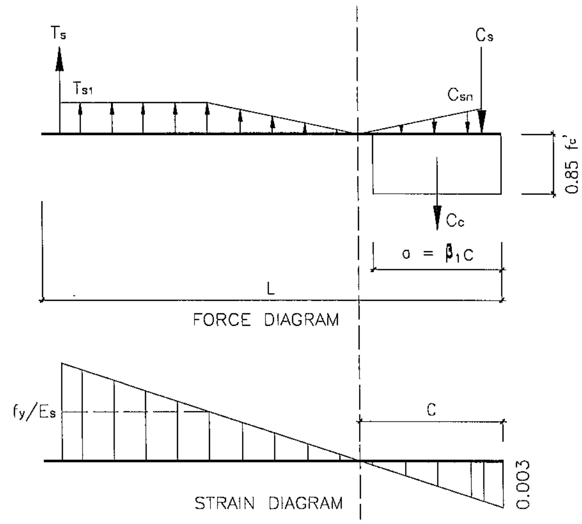
ANALYSIS

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

P_u = 128.8 k **<** $0.35 A_g f_c'$ = 3326 k [Satisfactory]

where A_g = 2376 in^2.





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

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

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

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

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

where A_{cv} = 2376 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)

ϕV_n =MIN [$\phi A_{cv} (\alpha_c (f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8 A_{cv} (f_c')^{0.5}$] = 638.77 kips **>** V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.85 < 1.5)

ρ_l **>** ρ_t [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

(cont'd)

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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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 14.57 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\ \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 110 in $a = C_b\ \beta_1$ = 94 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 186 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y , 3.3 f_c'^{0.5} 4 L t)$ = -787 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 LOAD	=	4630	7804
AT 0 % TENSION	=	4543	8217
AT 25 % TENSION	=	3850	10967
AT 50 % TENSION	=	3313	12312
AT ε_t = 0.002	=	2563	13244
AT BALANCED CONDITION	=	2540	13389
AT ε_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

ϕM_n = 7459 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\ h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

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

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\ y / I)$ = 0.446 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 99 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 2493 in^2. (area of transformed section.)

I = 8145307 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 = MAX(c - 0.1L , 0.5 c) = 11.48 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING
STRUCTURAL ENGINEERING & DESIGN
LIC.: C49389

PROJECT : WALL 2
CLIENT :
JOB NO. :
DATE :

PAGE : 2017-06
DESIGN BY :
REVIEW BY : (53)

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 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

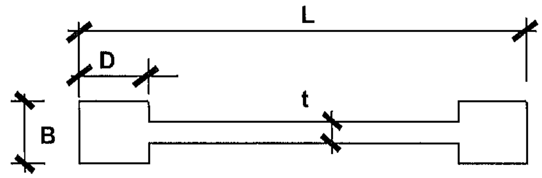
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

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

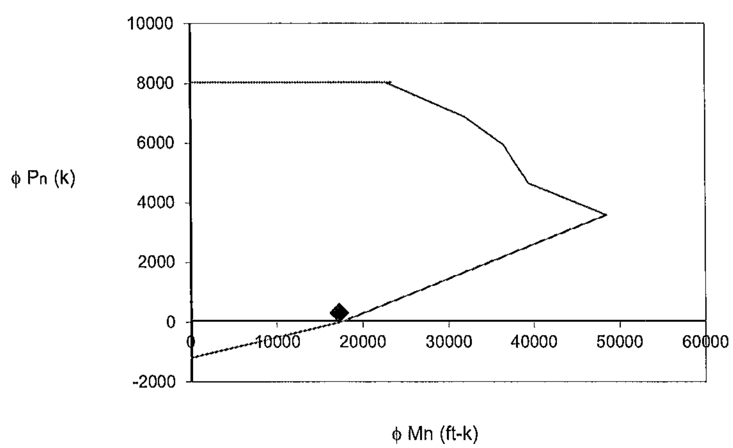


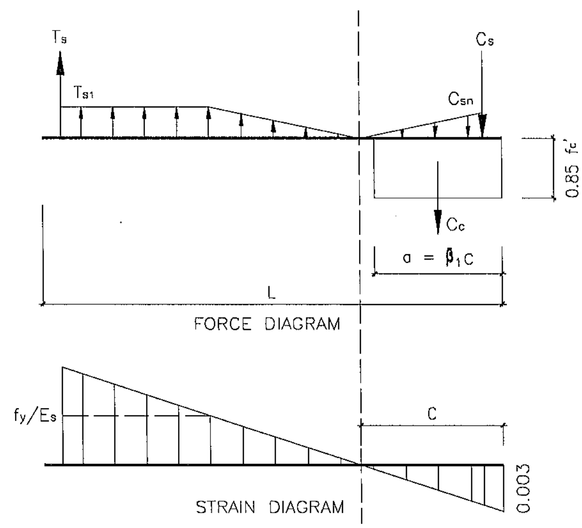
ANALYSIS

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

Pu = 311 k < $0.35\,A_g\,f_c'$ = 5846 k [Satisfactory]

where A_g = 4176 in^2.





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

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv}\,(f_c')^{0.5}$ = 264.11 kips < Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv}\,(f_c')^{0.5}$ = 264.11 kips < Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 4176 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)

ϕV_n =MIN [$\phi\,A_{cv}\,(\alpha_c\,(f_c')^{0.5} + \rho_t\,f_y)$, $\phi\,8\,A_{cv}\,(f_c')^{0.5}$] = 1122.68 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.48 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2019.06 (54)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 22.32 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 $\varepsilon_c = 0.003$ AND $\varepsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 199 in $a = C_b \beta_1$ = 169 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 336 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\,,\ 3.3 f_c'^{0.5}\, 4\, L\, t)$ = -1205 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 $\varepsilon_t = 0.002$	=	4645	39432
AT BALANCED CONDITION	=	4608	39863
AT $\varepsilon_t = 0.005$	=	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

ϕM_n = 21183 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\, h_w) / (600\, \delta_u)$ for ACI 21.9.6.2 apply   or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply   [Satisfactory]

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

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

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

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

A = 4356 in^2. (area of transformed section.)

I = 43956219 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 = MAX(c - 0.1L , 0.5 c) = 19.18 in. (ACI 318-14 18.10.6.4)

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

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

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

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STRUCTURAL ENGINEERING & DESIGN
Lic.: C50349

PROJECT : WALL 3 | PAGE : 2017-06
CLIENT : | DESIGN BY : ES
JOB NO. : | DATE : | REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

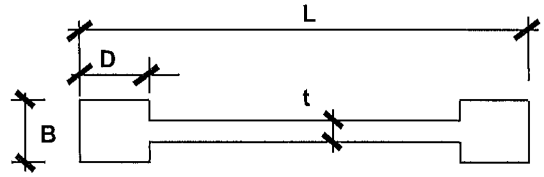
INPUT DATA

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

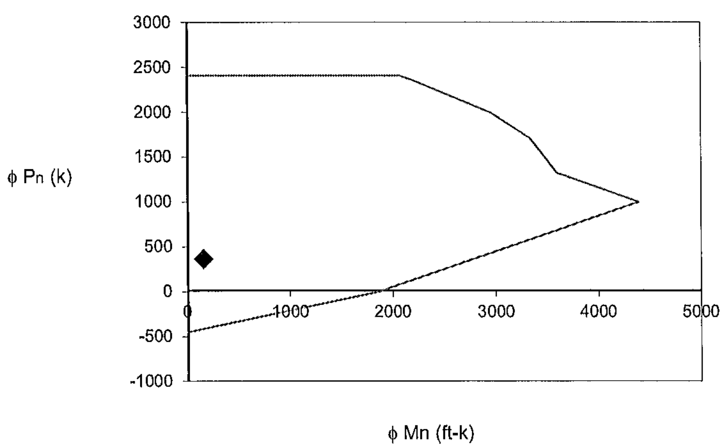


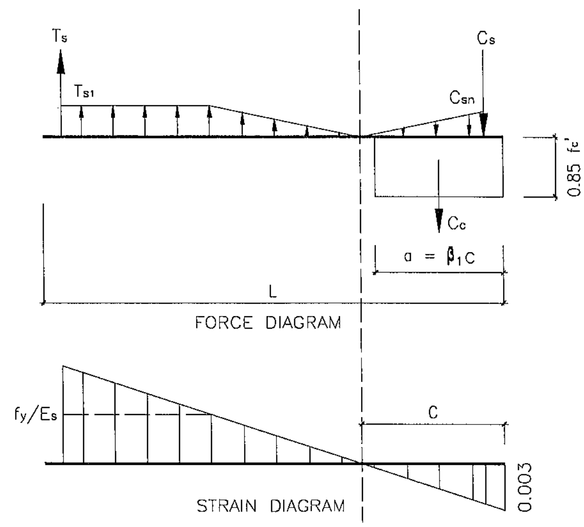
ANALYSIS

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

P_u = 361 k < $0.35 A_g f_c'$ = 1714 k [Satisfactory]

where A_g = 1224 in^2.





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

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

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

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

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

where A_{cv} = 1224 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)

ϕV_n = MIN [$\phi A_{cv} (\alpha_c (f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8 A_{cv} (f_c')^{0.5}$] = 315.40 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 2.7 (for h_w / L = 1.65 @ [1.5, 2])

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2012-06 (58)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 8.37 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 57 in $a = C_b \beta_1$ = 48 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 96 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\ ,\ 3.3 f_c'^{0.5}\, 4\, L\, t)$ = -452 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 ε_t = 0.002	=	1323	3593
AT BALANCED CONDITION	=	1311	3634
AT ε_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

ϕM_n = 3009 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\, h_w) / (600\, \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

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

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

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

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

A = 1291 in^2. (area of transformed section.)

I = 1119584 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.013 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

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

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

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

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

HCP ENGINEERING
STRUCTURAL ENGINEERING & DESIGN
LIC.: C19389

PROJECT : WALL 5
CLIENT :
JOB NO. : DATE :

PAGE : 2017/06
DESIGN BY : 57
REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

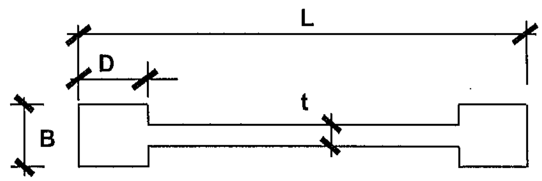
INPUT DATA

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

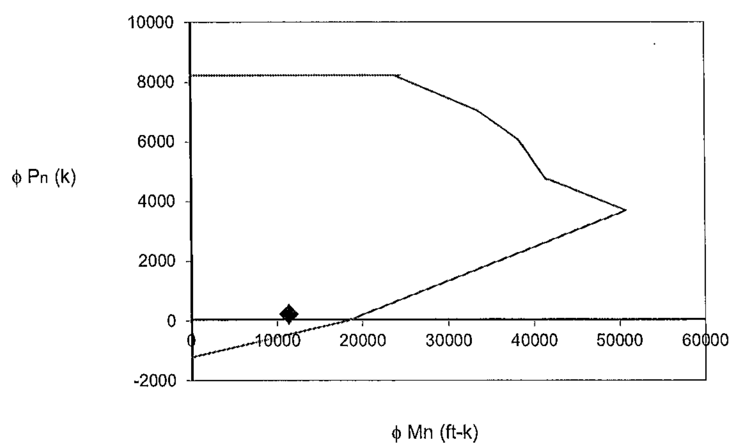


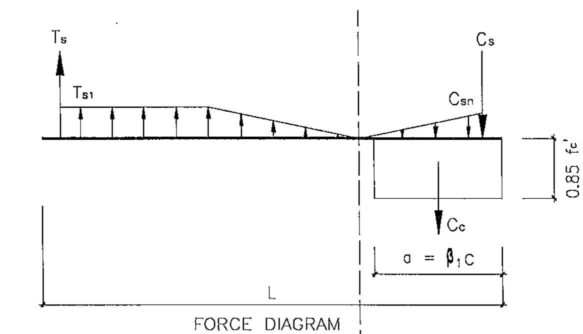
ANALYSIS

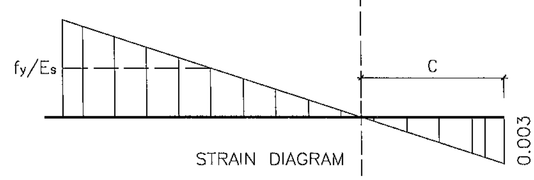
DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

Pu = 215 k **<** $0.35\ A_g\ f_c'$ = 5988 k [Satisfactory]

where A_g = 4277 in^2.







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

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv}\,(f_c')^{0.5}$ = 270.49 kips < Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv}\,(f_c')^{0.5}$ = 270.49 kips < Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 4277 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)

ϕV_n =MIN [$\phi\ A_{cv}\,(\alpha_c\,(f_c')^{0.5} + \rho_t\,f_y)$, $\phi\ 8\ A_{cv}\,(f_c')^{0.5}$] = 1149.78 kips **>** V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.47 < 1.5)

ρ_l **>** ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2017-06 (58)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 22.75 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 204 in $a = C_b\,\beta_1$ = 173 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) d = (L - 0.5D) = 344.4 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 kips.

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

$-\phi P_n = -0.9\, \mathrm{MIN}(A_{st} F_y\,,\ 3.3\, f_c'^{0.5}\, 4\, L\, t)$ = -1229 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 ε_t = 0.002	=	4762	41306
AT BALANCED CONDITION	=	4724	41757
AT ε_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

ϕM_n = 20858 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\,h_w) / (600\,\delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2\, f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

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

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

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

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

A = 4460 in^2. (area of transformed section.)

I = 47207785 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 = MAX(c - 0.1L , 0.5 c) = 18.20 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING

STRUCTURAL ENGINEERING & DESIGN

Lic.: C59369

PROJECT : WALL 7A

CLIENT :

JOB NO. : DATE :

PAGE : 2017-126 (55)

DESIGN BY :

REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

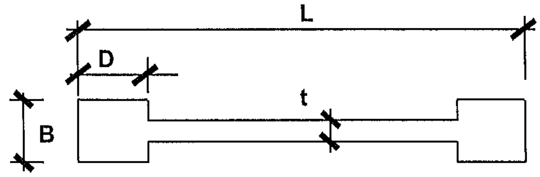
INPUT DATA

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

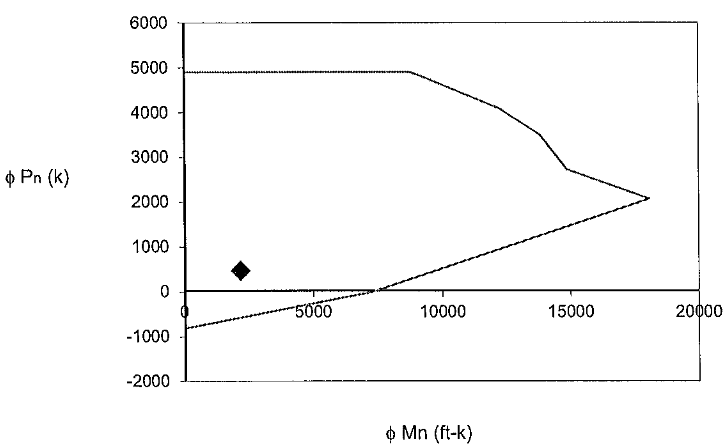


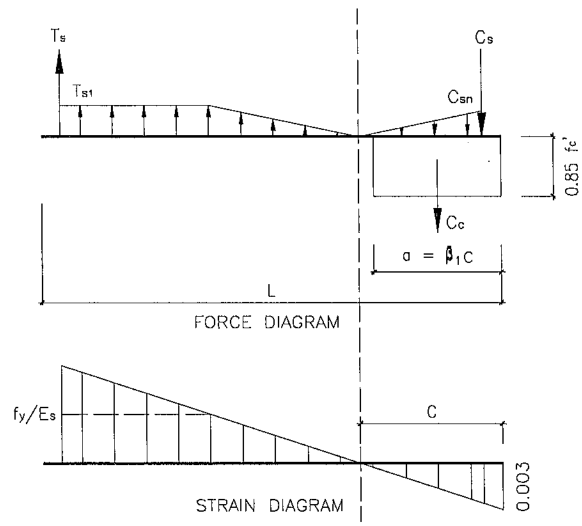
ANALYSIS

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

Pu = 468.5 k < $0.35 A_g f_c'$ = 3528 k [Satisfactory]

where A_g = 2520 in^2.





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

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

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

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

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

where A_{cv} = 2520 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)

ϕV_n =MIN [$\phi A_{cv} (\alpha_c (f_c')^{0.5} + \rho_t f_y), \phi 8 A_{cv} (f_c')^{0.5}$] = 677.48 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.80 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2012-06
60

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 15.19 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 $\mathcal{E}_c$ = 0.003 AND $\mathcal{E}_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\, \mathcal{E}_c / (\mathcal{E}_c + \mathcal{E}_s)$ = 117 in $a = C_b \beta_1$ = 100 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\mathcal{E}_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 198 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 kips.

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

$-\phi P_n = -0.9\, MIN(A_{st} F_y , 3.3 f_c'^{0.5} 4 L t)$ = -820 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 ε_t = 0.002	=	2730	14832
AT BALANCED CONDITION	=	2706	14995
AT ε_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

ϕM_n = 10480 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\, h_w) / (600\, \delta_u)$ for ACI 21.9.6.2 apply   or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply   [Satisfactory]

where c = 32 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

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

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

A = 2642 in^2. (area of transformed section.)

I = 9710064 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 = MAX(c - 0.1L , 0.5 c) = 16.13 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING
STRUCTURAL ENGINEERING & DESIGN
Lic.: C50389

PROJECT : WALL 7B
CLIENT :
JOB NO. :
DATE :
PAGE : 2017-06
DESIGN BY : (67)
REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

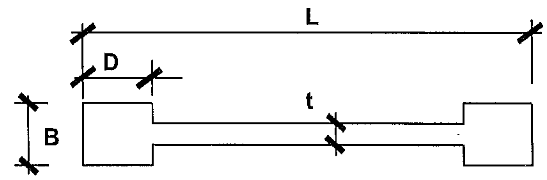
INPUT DATA

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

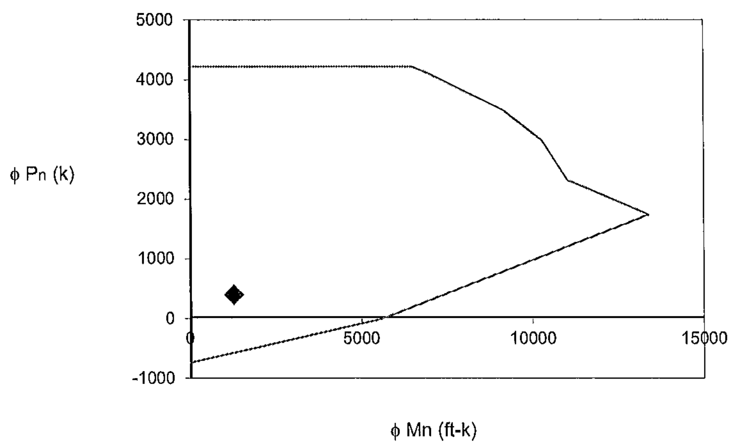


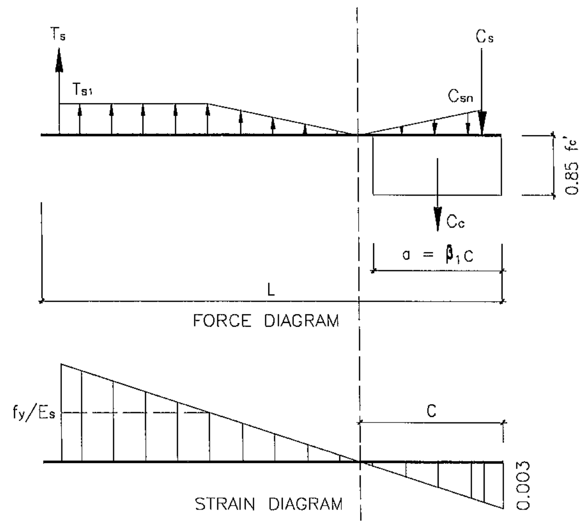
ANALYSIS

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

Pu = 401.5 k < $0.35 A_g f_c'$ = 3024 k [Satisfactory]

where A_g = 2160 in^2.





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

$(\rho_t)_{min.}$ = 0.0020 [for $A_{cv} (f_c')^{0.5}$ = 136.61 kips > Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0012 [for $A_{cv} (f_c')^{0.5}$ = 136.61 kips > Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 2160 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)

ϕV_n =MIN [$\phi A_{cv} (\alpha_c (f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8 A_{cv} (f_c')^{0.5}$] = 580.70 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.93 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2017-06
(62)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 13.64 in^2.

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

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 99 in $a = C_b\,\beta_1$ = 85 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) d = (L - 0.5D) = 168 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 kips.

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

$-\phi P_n = -0.9\,\mathrm{MIN}(A_{st} F_y\,,\ 3.3 f_c'^{0.5}\,4\,L\,t\,)$ = -737 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 ε_t = 0.002	=	2313	11027
AT BALANCED CONDITION	=	2292	11148
AT ε_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

ϕM_n = 7949 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\,h_w) / (600\,\delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

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

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

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

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

A = 2270 in^2. (area of transformed section.)

I = 6128259 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 = MAX(c - 0.1L , 0.5 c) = 14.18 in. (ACI 318-14 18.10.6.4)

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

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

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

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

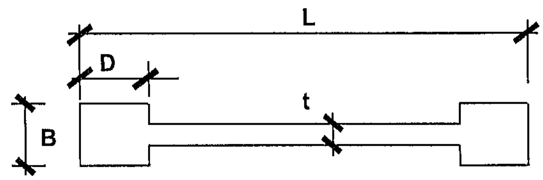
INPUT DATA

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

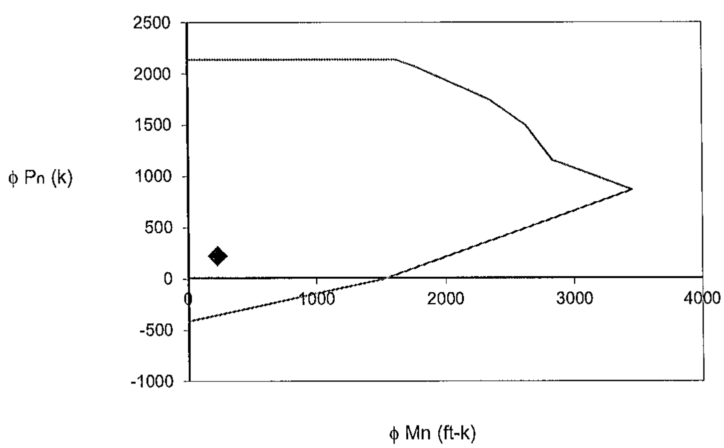


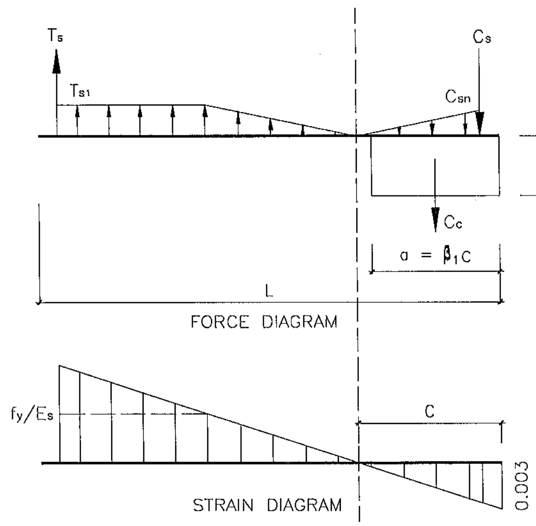
ANALYSIS

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

P_u = 224 k < $0.35\ A_g\ f_c'$ = 1512 k [Satisfactory]

where A_g = 1080 in^2.





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 > Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0012 [for $A_{cv}(f_c')^{0.5}$ = 68.31 kips > Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 1080 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)

ϕV_n = 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 ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 2.3 (for h_w / L = 1.87 @ [1.5, 2])

ρ_l > ρ_t [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

(cont'd)

2017-06 (64)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 7.75 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 $\varepsilon_c = 0.003$ AND $\varepsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 50 in $a = C_b\,\beta_1$ = 42 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 84 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\,,\ 3.3 f_c'^{0.5}\, 4\, L\, t)$ = -419 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 $\varepsilon_t = 0.002$	=	1156	2832
AT BALANCED CONDITION	=	1145	2864
AT $\varepsilon_t = 0.005$	=	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

ϕM_n = 2156 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\, h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

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

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

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

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

A = 1142 in^2. (area of transformed section.)

I = 771082 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.013 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

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

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

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

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

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 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

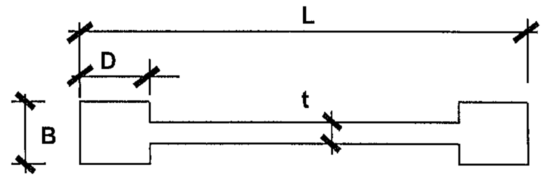
REINF. BARS AT BULB 10 # 5
WALL DIST. HORIZ. REINF. 2 # 5 @ 12 in. o.c.
WALL DIST. VERT. REINF. 2 # 5 @ 12 in. o.c.
HOOP REINF - WIDTH, B, DIR. 2 legs of # 4
HOOP REINF - LENGTH DIR. 3 legs of # 4
WALL EFFECTIVELY CONTINUOUS ? Yes (ACI 18.10.6.2 apply)

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

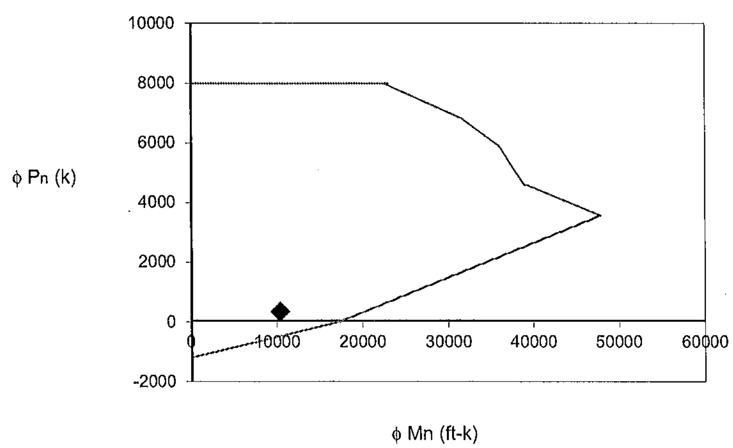


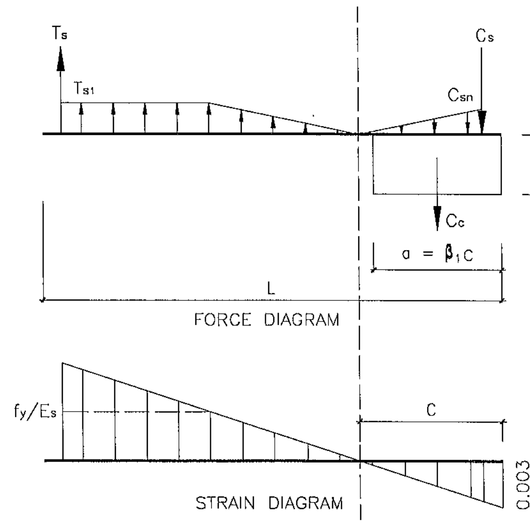
ANALYSIS

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

Pu = 345.9 k < $0.35 A_g f_c'$ = 5806 k [Satisfactory]

where A_g = 4147 in^2.





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

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv} (f_c')^{0.5}$ = 262.29 kips < Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv} (f_c')^{0.5}$ = 262.29 kips < Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 4147 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)

ϕV_n =MIN [$\phi A_{cv} (\alpha_c (f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8 A_{cv} (f_c')^{0.5}$] = 1114.94 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.49 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

W7-06
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CHECK FLEXURAL & AXIAL CAPACITY

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

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

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 22.20 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 197 in $a = C_b \beta_1$ = 168 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 333.6 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\ , 3.3 f_c'^{0.5} 4 L t)$ = -1199 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 ε_t = 0.002	=	4612	38905
AT BALANCED CONDITION	=	4575	39330
AT ε_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

ϕM_n = 21353 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\, h_w) / (600\, \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

where c = 39 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

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

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

A = 4326 in^2. (area of transformed section.)

I = 43055434 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 = MAX(c - 0.1L , 0.5 c) = 19.66 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING
STRUCTURAL ENGINEERING & DESIGN
Lic.: C50359

PROJECT : WALL 9B
CLIENT :
JOB NO. :
DATE :
PAGE : 2017-06 (68)
DESIGN BY :
REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

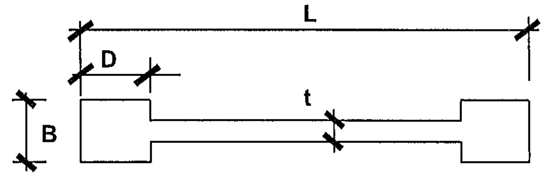
INPUT DATA

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

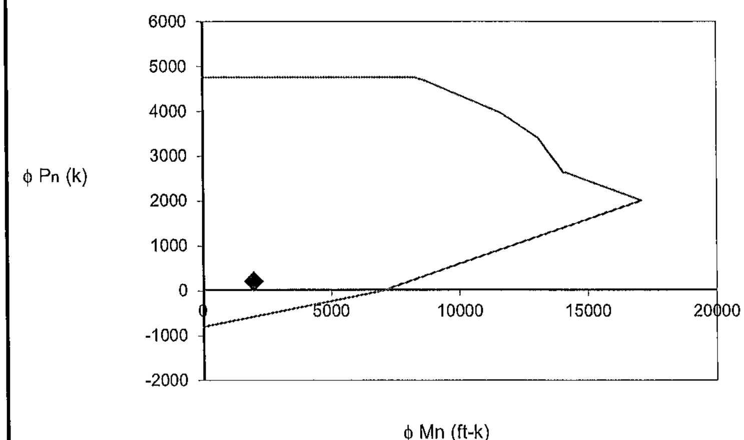


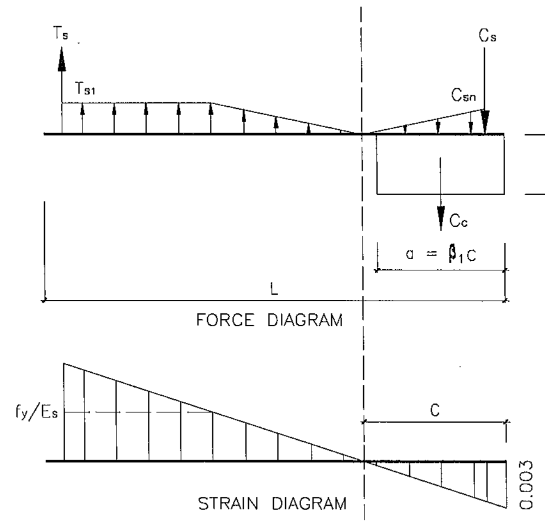
ANALYSIS

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

P_u = 204 k < $0.35\, A_g\, f_c'$ = 3427 k [Satisfactory]

where A_g = 2448 in^2.





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

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

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

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

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

where A_{cv} = 2448 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)

ϕV_n =MIN [$\phi\, A_{cv}\,(\alpha_c\,(f_c')^{0.5} + \rho_t\, f_y)$, $\phi\, 8\, A_{cv}\,(f_c')^{0.5}$] = 658.13 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.82 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

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CHECK FLEXURAL & AXIAL CAPACITY

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

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

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 14.88 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 114 in $a = C_b\,\beta_1$ = 97 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 192 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\ ,\ 3.3\, f_c'^{0.5}\, 4\, L\, t)$ = -804 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 ε_t = 0.002	=	2646	14027
AT BALANCED CONDITION	=	2623	14181
AT ε_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

ϕM_n = 8335 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\, h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2\, f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

where c = 25 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\, y / I)$ = 0.353 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 102 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 2568 in^2. (area of transformed section.)

I = 8904785 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 = MAX(c - 0.1L , 0.5 c) = 12.61 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING
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PROJECT :	WALL 10A	PAGE :	2017-06 10
CLIENT :		DESIGN BY :	
JOB NO. :	DATE :	REVIEW BY :	

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

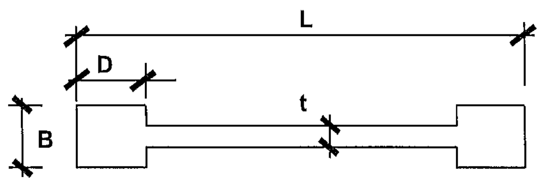
INPUT DATA

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

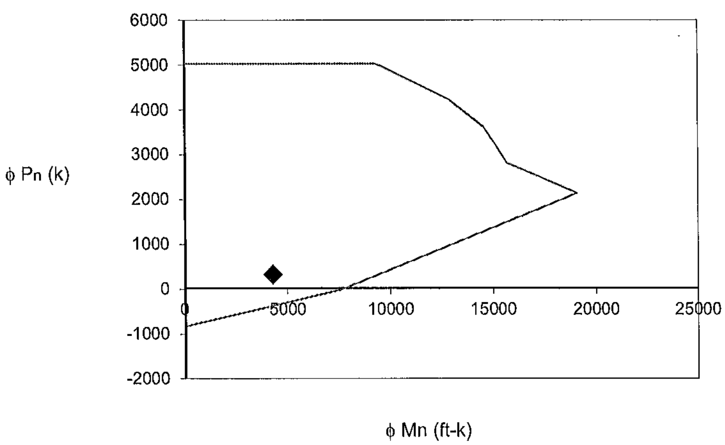


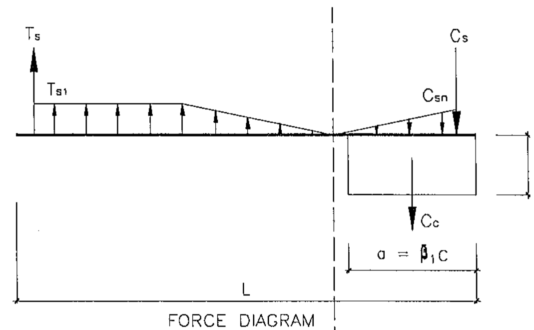
ANALYSIS

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

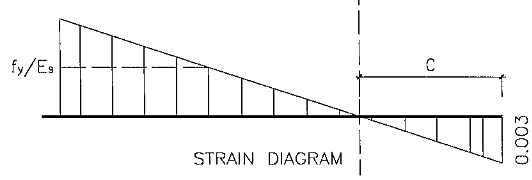
Pu = 324.7 k < $0.35 A_g f_c'$ = 3629 k [Satisfactory]

where A_g = 2592 in^2.





FORCE DIAGRAM



STRAIN DIAGRAM

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

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv} (f_c')^{0.5}$ = 163.93 kips < Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv} (f_c')^{0.5}$ = 163.93 kips < Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 2592 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)

ϕV_n = MIN [$\phi A_{cv} (\alpha_c (f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8 A_{cv} (f_c')^{0.5}$] = 696.84 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.78 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2017-06
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CHECK FLEXURAL & AXIAL CAPACITY

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

$\phi P_{max} = 0.8\ \phi\,[\,0.85\, f_c'\,(A_g - A_{st}) + f_y A_{st}]$ = 5038.9 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 15.50 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 121 in $a = C_b\,\beta_1$ = 103 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2)   d = (L - 0.5D) = 204 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 kips.

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

$-\phi P_n = -0.9\,\mathrm{MIN}(A_{st} F_y\,,\ 3.3\, f_c'^{0.5}\, 4\, L\, t\,)$ = -837 kips.

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

		ϕ P_n (kips)	**ϕ M_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 ε_t = 0.002	=	2813	15660
AT BALANCED CONDITION	=	2788	15831
AT ε_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

ϕM_n = 9980 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\, h_w) / (600\, \delta_u)$ for ACI 21.9.6.2 apply   or $f_c < 0.2\, f_c'$ for ACI 18.10.6.3 apply   [Satisfactory]

where c = 29 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\, y / I)$ = 0.643 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 108 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 2717 in^2. (area of transformed section.)

I = 10562483 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 = MAX(c - 0.1L , 0.5 c) = 14.59 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING PROJECT : WALL 10B PAGE : 2017-06

CLIENT : DESIGN BY :

JOB NO. : DATE : REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

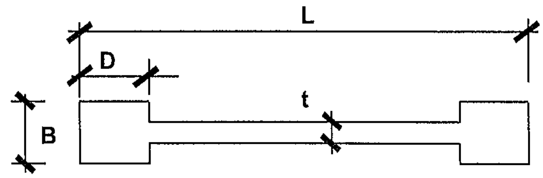
INPUT DATA

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

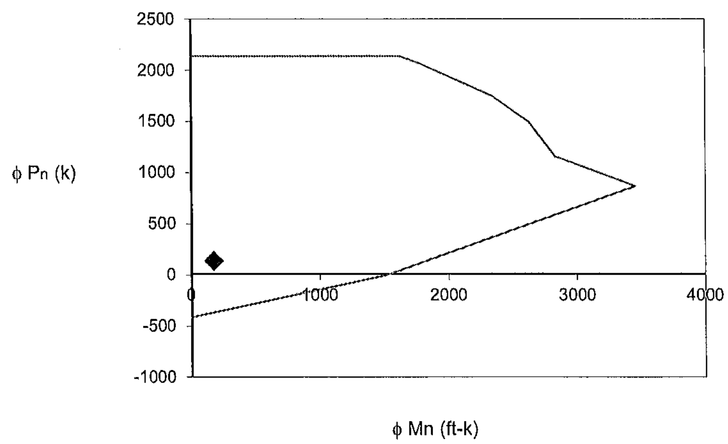


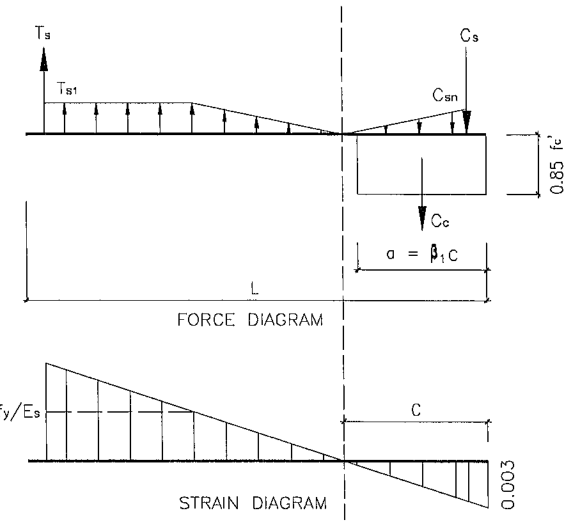
ANALYSIS

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

Pu = 135.3 k < 0.35 A_g f_c' = 1512 k [Satisfactory]

where A_g = 1080 in^2.





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 > Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0012 [for $A_{cv} (f_c')^{0.5}$ = 68.31 kips > Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 1080 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)

ϕV_n =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 ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 2.3 (for h_w / L = 1.87 @ [1.5, 2])

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2017-06

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CHECK FLEXURAL & AXIAL CAPACITY

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

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

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 7.75 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 $\varepsilon_c = 0.003$ AND $\varepsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 50 in $a = C_b\,\beta_1$ = 42 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 84 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\ ,\ 3.3 f_c'^{0.5}\ 4\ L\ t)$ = -419 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 $\varepsilon_t = 0.002$	=	1156	2832
AT BALANCED CONDITION	=	1145	2864
AT $\varepsilon_t = 0.005$	=	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

ϕM_n = 1914 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\,h_w) / (600\,\delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

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

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\,y / I)$ = 0.244 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

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

A = 1142 in^2. (area of transformed section.)

I = 771082 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.013 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = $\mathrm{MAX}(c - 0.1L\ ,\ 0.5\,c)$ = 6.37 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING
STRUCTURAL ENGINEERING & DESIGN
Lic.: C10349

PROJECT : WALL 12
CLIENT :
JOB NO. :
DATE :

PAGE : 2017-06
DESIGN BY : (74)
REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

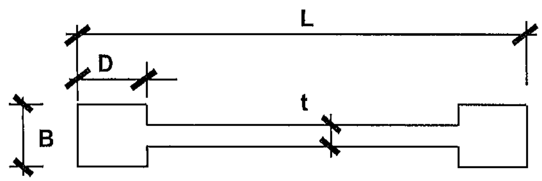
INPUT DATA

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

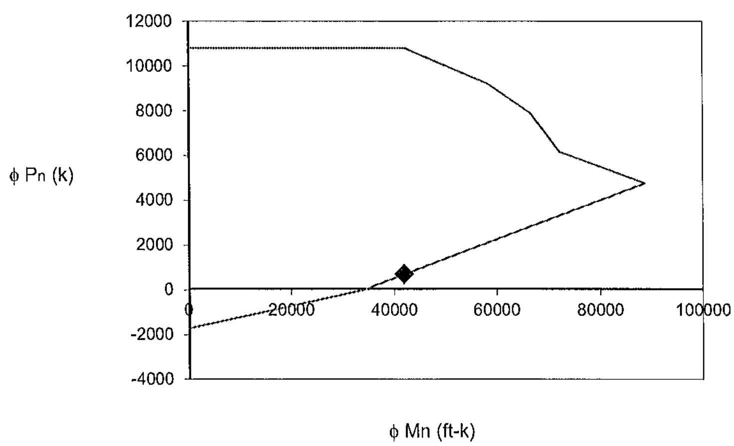


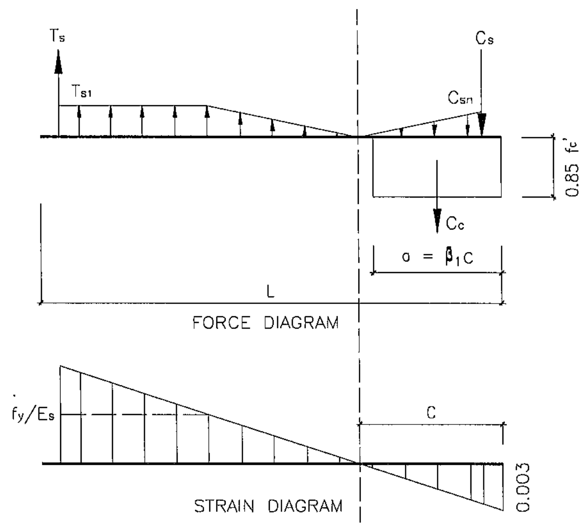
ANALYSIS

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

Pu = 693 k < $0.35\ A_g\ f_c'$ = 7812 k [Satisfactory]

where A_g = 5580 in^2.





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 < Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv}\,(f_c')^{0.5}$ = 352.91 kips < Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 5580 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)

ϕV_n =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 ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.36 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

(cont'd)

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CHECK FLEXURAL & AXIAL CAPACITY

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

$\phi P_{max} = 0.8\ \phi [0.85 f_c' (A_g - A_{st}) + f_y A_{st}]$ = 10810 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 32.09 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 $\varepsilon_c = 0.003$ AND $\varepsilon_t = f_y / E_s = 0.002069$. THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\, \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 265 in $a = C_b \beta_1$ = 225 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) d = (L - 0.5D) = 447 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\ , 3.3 f_c'^{0.5} 4 L t)$ = -1733 kips.

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

		ϕ Pn (kips)	**ϕ Mn (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 $\varepsilon_t = 0.002$	=	6176	72104
AT BALANCED CONDITION	=	6124	72911
AT $\varepsilon_t = 0.005$	=	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

ϕM_n = 44693 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\ h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

where c = 59 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\, y / I)$ = 1.229 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 233 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 5838 in^2. (area of transformed section.)

I = 105195341 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.013 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = MAX(c - 0.1L , 0.5 c) = 29.73 in. (ACI 318-14 18.10.6.4)

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

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

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

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STRUCTURAL ENGINEERING & DESIGN
LIC.: C50589

PROJECT : WALL 16
CLIENT :
JOB NO. :
DATE :

PAGE : 2017-06
DESIGN BY : (76)
REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

INPUT DATA

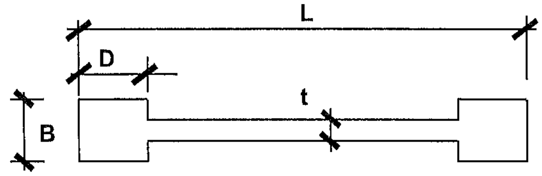
CONCRETE STRENGTH (ACI 318 19.2.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
FACTORED AXIAL LOAD	P_u	=	147	k
FACTORED MOMENT LOAD	M_u	=	1218	ft-k
FACTORED SHEAR LOAD	V_u	=	84	k
LENGTH OF SHEAR WALL	L	=	14.5	ft
THICKNESS OF WALL	t	=	12	in
DEPTH AT FLANGE	D	=	24	in
WIDTH AT FLANGE	B	=	12	in
TOTAL 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

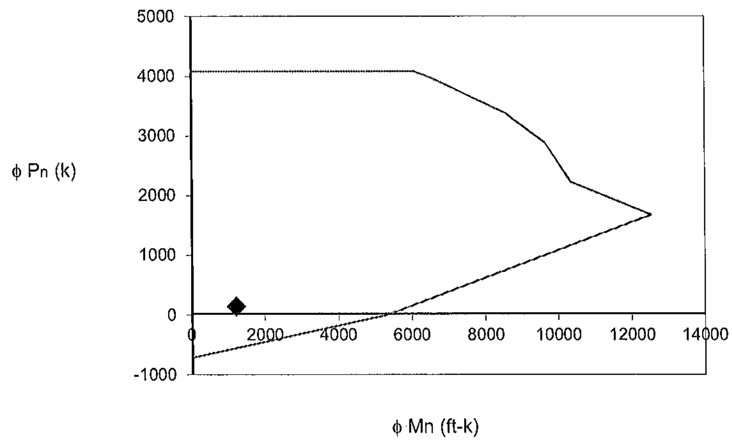


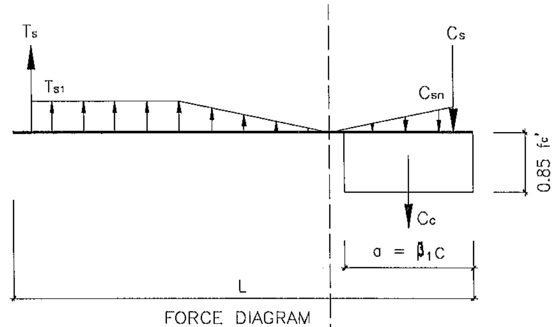
ANALYSIS

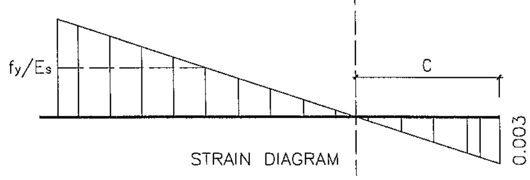
DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

P_u = 147 k < $0.35 A_g f_c'$ = 2923 k [Satisfactory]

where A_g = 2088 in^2.







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

$(\rho_t)_{min.}$ = 0.0020 [for $A_{cv}(f_c')^{0.5}$ = 132.06 kips > Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0012 [for $A_{cv}(f_c')^{0.5}$ = 132.06 kips > Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 2088 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)

ϕV_n = MIN [$\phi A_{cv}(\alpha_c (f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8 A_{cv}(f_c')^{0.5}$] = 561.34 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.97 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

(cont'd)

2017.06
(77)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 13.33 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\ \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 96 in $a = C_b\ \beta_1$ = 81 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 162 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\ , 3.3\ f_c'^{0.5}\ 4\ L\ t)$ = -720 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 ε_t = 0.002	=	2230	10332
AT BALANCED CONDITION	=	2209	10444
AT ε_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

ϕM_n = 6169 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\ h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2\ f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

where c = 22 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\ y / I)$ = 0.297 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 87 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 2195 in^2. (area of transformed section.)

I = 5538570 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 = MAX(c - 0.1L , 0.5 c) = 10.88 in. (ACI 318-14 18.10.6.4)

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

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

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

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STRUCTURAL ENGINEERING & DESIGN
Lic.: C50389

PROJECT : WALL 18
CLIENT :
JOB NO. : DATE :

PAGE : 2017-06
DESIGN BY : 68
REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

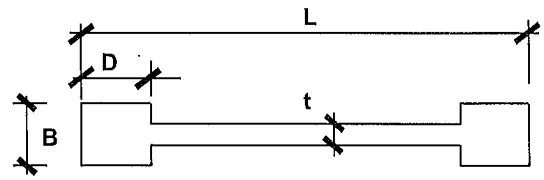
INPUT DATA

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

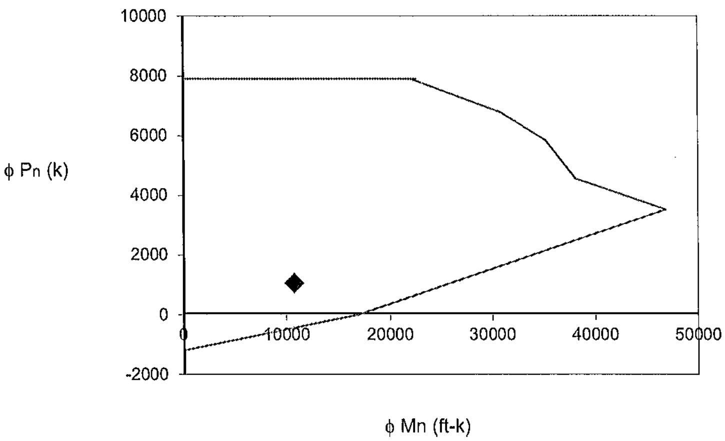


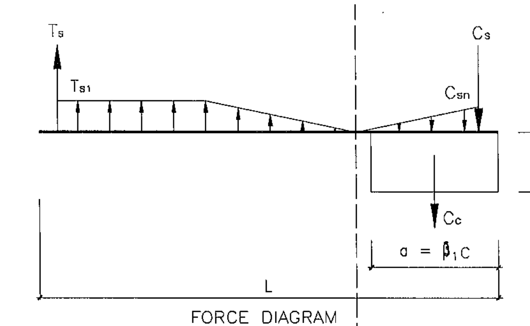
ANALYSIS

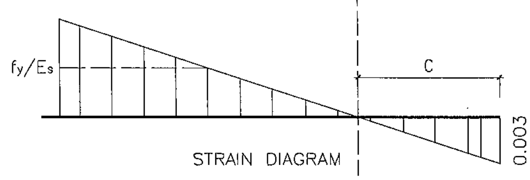
DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

P_u = 1064 k < $0.35\ A_g\ f_c'$ = 5746 k [Satisfactory]

where A_g = 4104 in^2.







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

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv}\ (f_c')^{0.5}$ = 259.56 kips < Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv}\ (f_c')^{0.5}$ = 259.56 kips < Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 4104 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)

ϕV_n =MIN [$\phi\ A_{cv}\ (\alpha_c\ (f_c')^{0.5} + \rho_t\ f_y)$, $\phi\ 8\ A_{cv}\ (f_c')^{0.5}$] = 1103.33 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.49 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2017-06

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CHECK FLEXURAL & AXIAL CAPACITY

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

$\phi P_{max} = 0.8\ \phi [0.85 f_c' (A_g - A_{st}) + f_y A_{st}]$ = 7903.7 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 22.01 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\ \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 195 in $a = C_b \beta_1$ = 166 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 330 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y , 3.3 f_c'^{0.5} 4 L t)$ = -1189 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 ε_t = 0.002	=	4562	38121
AT BALANCED CONDITION	=	4525	38537
AT ε_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

ϕM_n = 28592 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\ h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

where c = 58 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\ y / I)$ = 0.776 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 171 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 4281 in^2. (area of transformed section.)

I = 41727463 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 = MAX(c - 0.1L , 0.5 c) = 29.11 in. (ACI 318-14 18.10.6.4)

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

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

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

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Lic.: C50339

PROJECT : WALL 20A		PAGE :	10n-06
CLIENT :		DESIGN BY :	86
JOB NO. :	DATE :	REVIEW BY :	

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

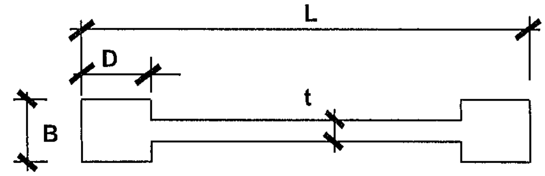
INPUT DATA

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

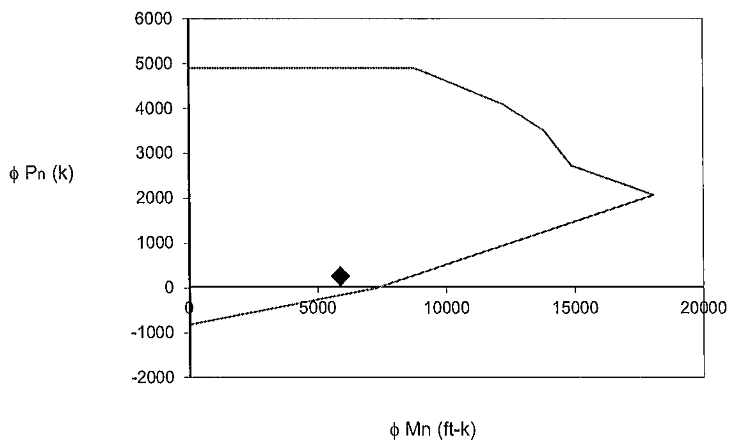


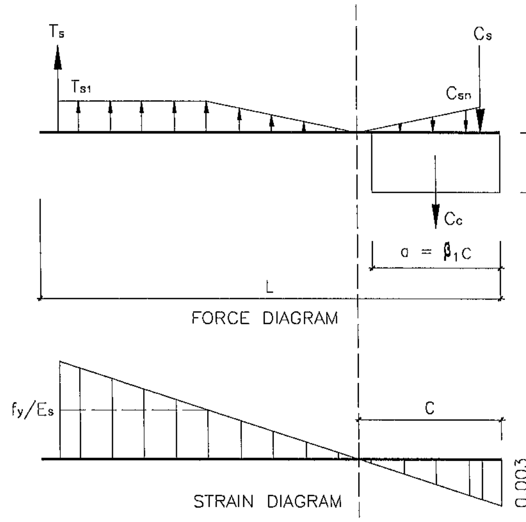
ANALYSIS

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

Pu = 255.2 k < 0.35 A_g f_c' = 3528 k [Satisfactory]

where A_g = 2520 in^2.





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

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv} (f_c')^{0.5}$ = 159.38 kips < Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv} (f_c')^{0.5}$ = 159.38 kips < Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 2520 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)

ϕV_n =MIN [$\phi A_{cv} (\alpha_c (f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8 A_{cv} (f_c')^{0.5}$] = 677.48 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.80 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

(cont'd)

2017-06 (87)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 15.19 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\, \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 117 in $a = C_b\, \beta_1$ = 100 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) d = (L - 0.5D) = 198 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 kips.

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

$-\phi P_n = -0.9\ MIN(A_{st} F_y\ , 3.3 f_c'^{0.5}\, 4\, L\, t\ )$ = -820 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 ε_t = 0.002	=	2730	14832
AT BALANCED CONDITION	=	2706	14995
AT ε_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

ϕM_n = 9081 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\, h_w) / (600\, \delta_u)$ for ACI 21.9.6.2 apply or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply [Satisfactory]

where c = 27 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\, y / I)$ = 0.860 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

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

A = 2642 in^2. (area of transformed section.)

I = 9710064 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 = MAX(c - 0.1L , 0.5 c) = 13.47 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING
STRUCTURAL ENGINEERING & DESIGN
Lic.: C50389

PROJECT : WALL 20B
CLIENT :
JOB NO. :
DATE :

PAGE : 2017-06 (82)
DESIGN BY :
REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

INPUT DATA

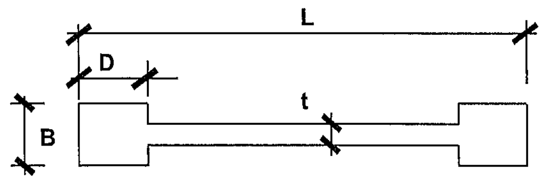
CONCRETE STRENGTH (ACI 318 19.2.1.1)	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
FACTORED AXIAL LOAD	P_u	=	175	k
FACTORED MOMENT LOAD	M_u	=	612	ft-k
FACTORED SHEAR LOAD	V_u	=	51	k
LENGTH OF SHEAR WALL	L	=	12	ft
THICKNESS OF WALL	t	=	12	in
DEPTH AT FLANGE	D	=	12	in
WIDTH AT FLANGE	B	=	12	in
TOTAL WALL 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

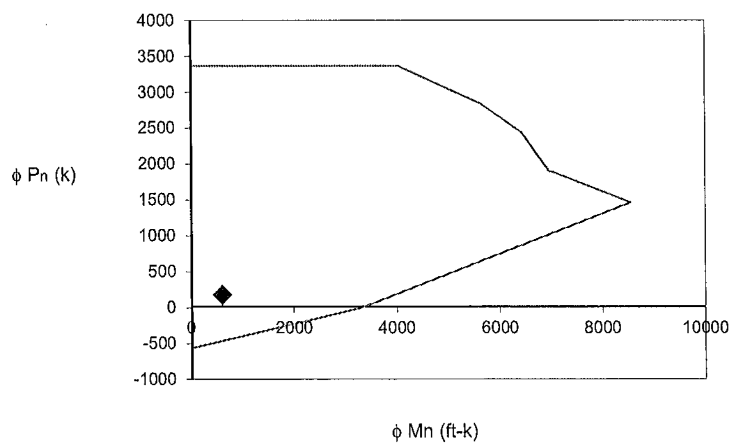


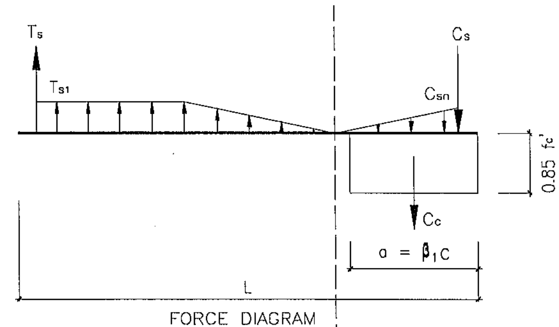
ANALYSIS

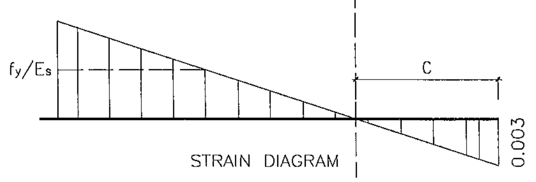
DETERMINE WHETHER THE WALL CAN RESIST SEISMIC LOADS (2016 CBC 1905A)

Pu = 175 k < $0.35\ A_g\ f_c'$ = 2419 k [Satisfactory]

where A_g = 1728 in^2.







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

$(\rho_t)_{min.}$ = 0.0020 [for $A_{cv}\ (f_c')^{0.5}$ = 109.29 kips > Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0012 [for $A_{cv}\ (f_c')^{0.5}$ = 109.29 kips > Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 1728 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)

ϕV_n =MIN [$\phi\ A_{cv}\ (\alpha_c\ (f_c')^{0.5} + \rho_t\ f_y)$, $\phi\ 8\ A_{cv}\ (f_c')^{0.5}$] = 464.56 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 1.17 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for h_w / L > 2.0, ACI 318-14 18.10.4.3)

(cont'd)

2017-06

(63)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 10.54 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\ \varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 82 in $a = C_b \beta_1$ = 69 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2) $d = (L - 0.5D)$ = 138 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 kips.

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

$-\phi P_n = -0.9\ MIN(A_{st} F_y , 3.3 f_c'^{0.5} 4 L t)$ = -569 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 ε_t = 0.002	=	1906	6953
AT BALANCED CONDITION	=	1890	7031
AT ε_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

ϕM_n = 4171 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\ h_w) / (600\ \delta_u)$ for ACI 21.9.6.2 apply   or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply   [Satisfactory]

where c = 18 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\ y / I)$ = 0.265 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 72 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 1813 in^2. (area of transformed section.)

I = 3132497 in^4. (moment of inertia of transformed section.)

Or the longitudinal reinforcement ratio at the wall end = 0.013 > $400 / f_y$ [Unsatisfactory]

HENCE SPECIAL BOUNDARY ZONE DETAILING REQUIRED !

The boundary element length = MAX(c - 0.1L , 0.5 c) = 8.78 in. (ACI 318-14 18.10.6.4)

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

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

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

HCP ENGINEERING PROJECT : WALL 20C PAGE : 2017-06 (84)

CLIENT : DESIGN BY :

JOB NO. : DATE : REVIEW BY :

Special Reinforced Concrete Shear Wall Design Based on ACI 318-11 & 2016 CBC Chapter A

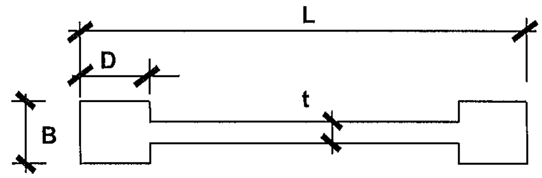
INPUT DATA

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

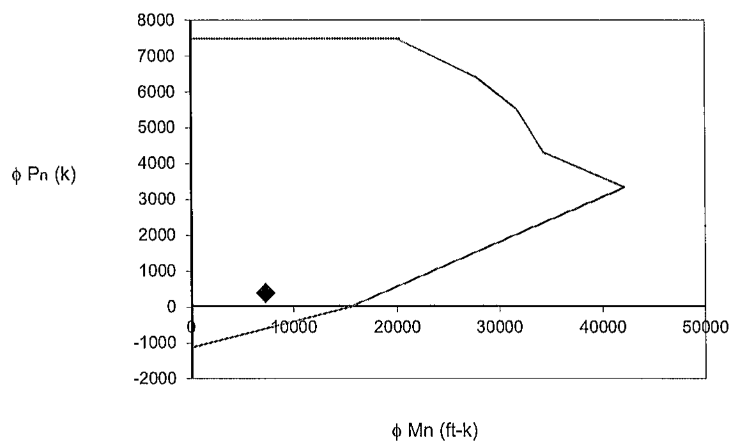


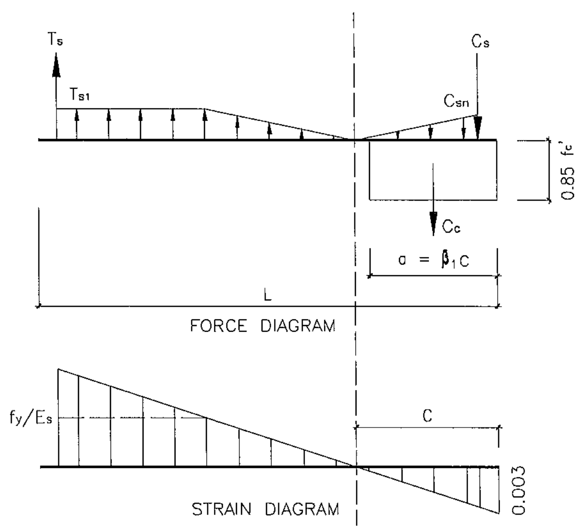
ANALYSIS

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

Pu = 393.8 k < $0.35 A_g f_c'$ = 5443 k [Satisfactory]

where A_g = 3888 in^2.





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

$(\rho_t)_{min.}$ = 0.0025 [for $A_{cv}(f_c')^{0.5}$ = 245.90 kips < Vu , and bar size # 5 horizontal]

$(\rho_l)_{min.}$ = 0.0025 [for $A_{cv}(f_c')^{0.5}$ = 245.90 kips < Vu , and bar size # 5 vertical]

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

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

where A_{cv} = 3888 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)

ϕV_n =MIN [$\phi A_{cv} (\alpha_c (f_c')^{0.5} + \rho_t f_y)$, $\phi\, 8 A_{cv} (f_c')^{0.5}$] = 1045.26 kips > V_u [Satisfactory]

where ϕ = 0.60 (conservatively, ACI 318-14 21.2)

α_c = 3.0 (for h_w / L = 0.52 < 1.5)

ρ_l > ρ_t [Satisfactory] (only for $h_w / L > 2.0$, ACI 318-14 18.10.4.3)

(cont'd)

2012-06 (85)

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 kips. > P_u [Satisfactory]

where ϕ = 0.65 (ACI 318-14 21.2)

A_{st} = 21.08 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 ε_c = 0.003 AND $\varepsilon_t = f_y / E_s$ = 0.002069 . THE DEPTH TO THE NEUTRAL AXIS AND EQUIVALENT RECTANGULAR CONCRETE STRESS BLOCK ARE GIVEN BY

$C_b = d\,\varepsilon_c / (\varepsilon_c + \varepsilon_s)$ = 185 in $a = C_b\,\beta_1$ = 157 in β_1 = 0.85 (ACI 318-14 22.2.2.4.3)

$\phi = 0.65 + (\varepsilon_t - 0.002)(250/3)$ = 0.656 (ACI 318-14 21.2)   d = (L - 0.5D) = 312 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 kips.

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

$-\phi P_n = -0.9\ \mathrm{MIN}(A_{st} F_y\,,\ 3.3\,f_c'^{0.5}\,4\,L\,t)$ = -1138 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 LOAD	=	7494	20332
AT 0 % TENSION	=	7494	19977
AT 25 % TENSION	=	6398	27859
AT 50 % TENSION	=	5523	31714
AT ε_t = 0.002	=	4312	34317
AT BALANCED CONDITION	=	4277	34692
AT ε_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

ϕM_n = 19746 kips. > M_u [Satisfactory]

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

CHECK BOUNDARY ZONE REQUIREMENTS

AN EXEMPTION FROM THE PROVISION OF BOUNDARY ZONE CONFINEMENT REINFORCEMENT IS GIVEN BY ACI318-11 18.10.6.2, 18.10.6.3, and 18.10.6.5(a) PROVIDED THAT

$c < (L\,h_w) / (600\,\delta_u)$ for ACI 21.9.6.2 apply   or $f_c < 0.2 f_c'$ for ACI 18.10.6.3 apply   [Satisfactory]

where c = 39 in. (distance from the extreme compression fiber to neutral axis at P_u & M_n loads.)

δ_u = 1.2 in. (design displacement, assume $0.007h_w$ as a conservative short cut, see ACI 318-14 18.10.6.2.)

$f_c = (P_u / A) + (M_u\,y / I)$ = 0.495 ksi. (the maximum extreme fiber compressive stress at P_u & M_u loads.)

y = 162 in. (distance from the extreme compression fiber to neutral axis at P_u & M_u loads.)

A = 4058 in^2. (area of transformed section.)

I = 35495673 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 = MAX(c - 0.1L , 0.5 c) = 19.49 in. (ACI 318-14 18.10.6.4)

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

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

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

2017.06

(66)

FOUNDATION DESIGN

SOILS REPORT : GEO ENGINEERING CONSULTANTS
4125 BLACKFORD AVE #145
ALAMEDA, CA 55117
925-321-5550

PROJ : P16-152
DATE : 3/31/16.

* STRUCTURE SUPPORT BY DISPLACEMENT PILE - DESIGNED BY OTHERS.

SLAB-ON-GRADE

DEAD LOAD : MISC = 10.0 PSF
PARTITION = 15.0
25.0 PSF

LIVE LOAD 100 PSF

ONE WAY SLAB w/ BEAMS

$L_{u\,max} : 27 - 2 = 25'$

$t_{min} : \frac{25(12)}{24} = 12.5''$

$w_u : \left[\frac{12.5}{12}(150) + 25\right](1.2) + 1.6(100) = 376 \text{ PSF}$

$M_u^- = \frac{376(25)^2}{16} = 23500^{\#-\prime} = 23.5^{k-\prime}$

$b = 12''$ $d = 12.5 - 1 = 11.5''$

$A_{s_R} = 0.50 \text{ in}^2$ — #6 @ 10" o.c. TOP R/F O/ EA. BEAM

$$M_u^+ = \frac{376(25)^2}{12} = 19582^{\#\text{-}'} = 19.6^{k\text{-}'}$$

$b = 12''$ $d = 12.5 - 2 = 10.5''$

$A_{sr} = 0.46 \text{ in}^2$ - #5 @ 8" o.c BOTT. R/F E.W

GRADE BEAMS - GB1

$b = 24''$ $h = 30''$ $d = 27''$

$w_u = 376(25) = 9400^{\#/'}$

$$M_u^- = \frac{9.4(25)^2}{10} = 588^{k\text{-}'}$$ - $A_{sr} = 5.5 \text{ in}^2$ - 7~#8 TOP R/F

$$M_u^+ = \frac{9.4(25)^2}{12} = 489^{k\text{-}'}$$ - $A_{sr} = 4.5 \text{ in}^2$ - 6~#8 BOTT R/F

$$V_u = \frac{9.4(25)}{2} - 9.4(2.5) = 98.7^k$$

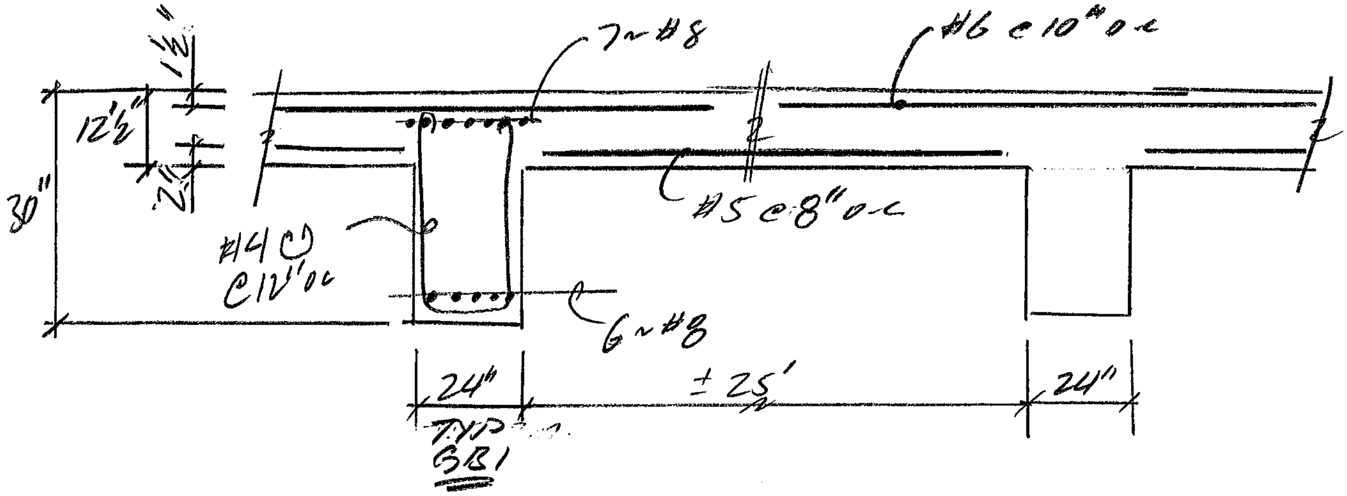
$\phi V_c = 78^k$

$\frac{\phi V_c}{2} = 39^k < 98.7^k$ ∴ SHEAR R/F REQ'D

#4 [] @ 12" o.c

$\phi V_s = 46^k$

$\phi V_c + \phi V_s = 124^k > 98.7^k$ ∴ #4 [] @ 12" o.c



GRADE BEAMS AT WALLS. - GB2

USE 24"W × 24" TYP.

PILES SPACING 8'-0" o.c. MIN.
10'-0" o.c. MAX ← BOVA'S

GRADE BEAMS

LINE 8 - WORST CASE

w_u (SLAB) = 0.376(12.5) = $4.7^{k}/FT$
$w_{WALL\ 8}$ = 28.7
w_u (FTG) = 1.2(750) = 0.9
w_u = $34.3^{k\text{-}1}$

PILE DIAMETER = 18" MIN.

$\therefore L_u = 10' - 1.5 = 8.5'$

$M_u^- = \frac{34.3(8.5)^2}{10} = 248^{k\text{-}1}$ · $A_{s_R} = 3.0 in^2$ · **4-#8 TOP**

$M_u^+ = \frac{34.3(8.5)^2}{12} = 208^{k\text{-}1}$ · $A_{s_R} = 1.9 in^2 < A_{s_{MIN}} = 2.1 in^2$

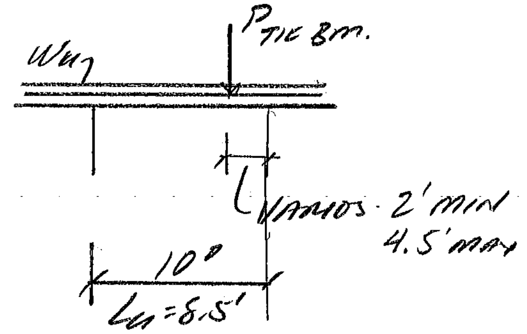
USE 4-#8 BOTT

$V_u = \frac{34.3(8.5)}{2} - 34.3(2) = 128.6^k$

#4 [] @12" o.c. $\phi V_s = 71^k$

$\phi V_c + \phi V_s = 61 + 71 = 132^k > 128.6^k$ $\therefore$ **#4 [] @12" o.c.**

Grade Beam @ Tie Beam.



$W_{u_{MAX}} = 34.3^{k}/ft$

$P_{u_{TIE\ BM}} = 9.4(12.5) = 117.5^{k}$

$M_{u_{max}}^{-} = \frac{34.3(8.5)^2}{10} + \frac{117.5(8.5)}{8} = 375^{k\text{-}ft}$ - $A_{s_R} = 3.47 in^2$
5~#8 Top

$M_{u_{max}}^{+} = \frac{34.3(8.5)^2}{12} + \frac{117.5(8.5)}{8} = 331^{k\text{-}ft}$ - $A_{s_R} = 3.07 in^2$
5~#8 Bott

$V_{u_{max}} = \frac{34.3(8.5)}{2} + 117.5 = 264^{k}$ ← Govn's

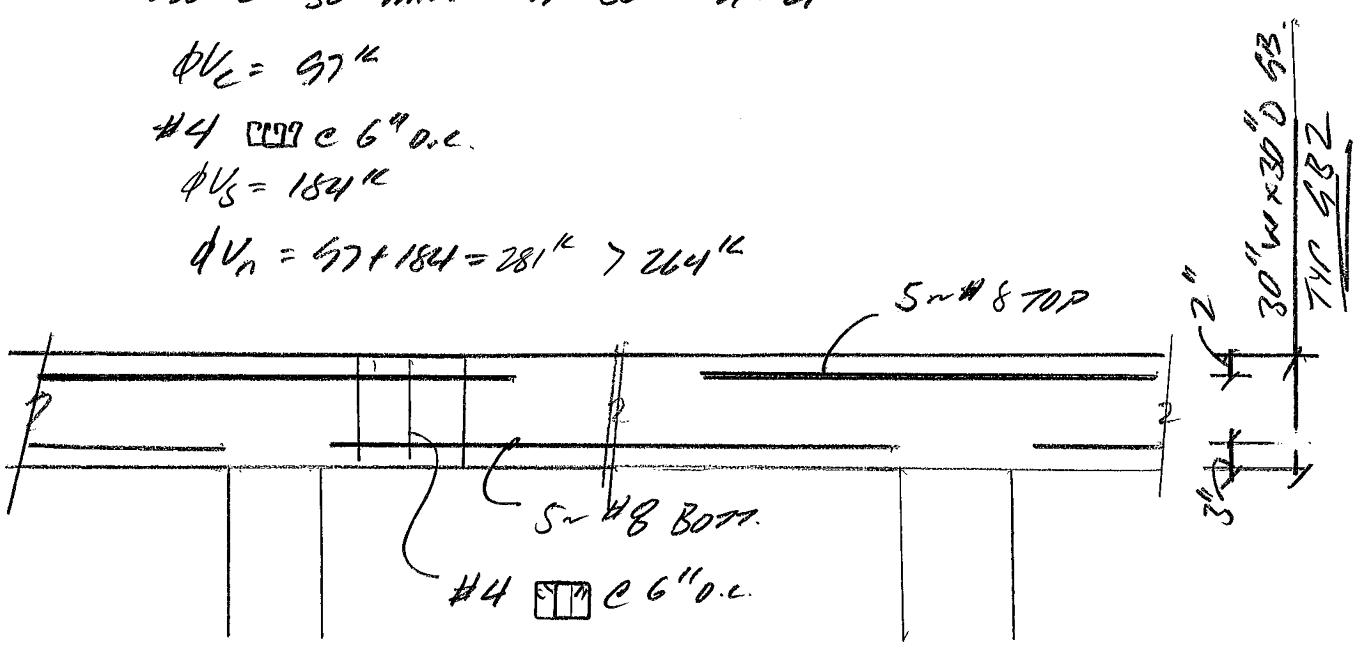
Use b = 30" min   h = 30"   d = 21"

$\phi V_c = 97^{k}$

#4 [stirrup] @ 6" o.c.

$\phi V_s = 184^{k}$

$\phi V_n = 97 + 184 = 281^{k} > 264^{k}$



1st Floor Slab & Foundation

2017-06
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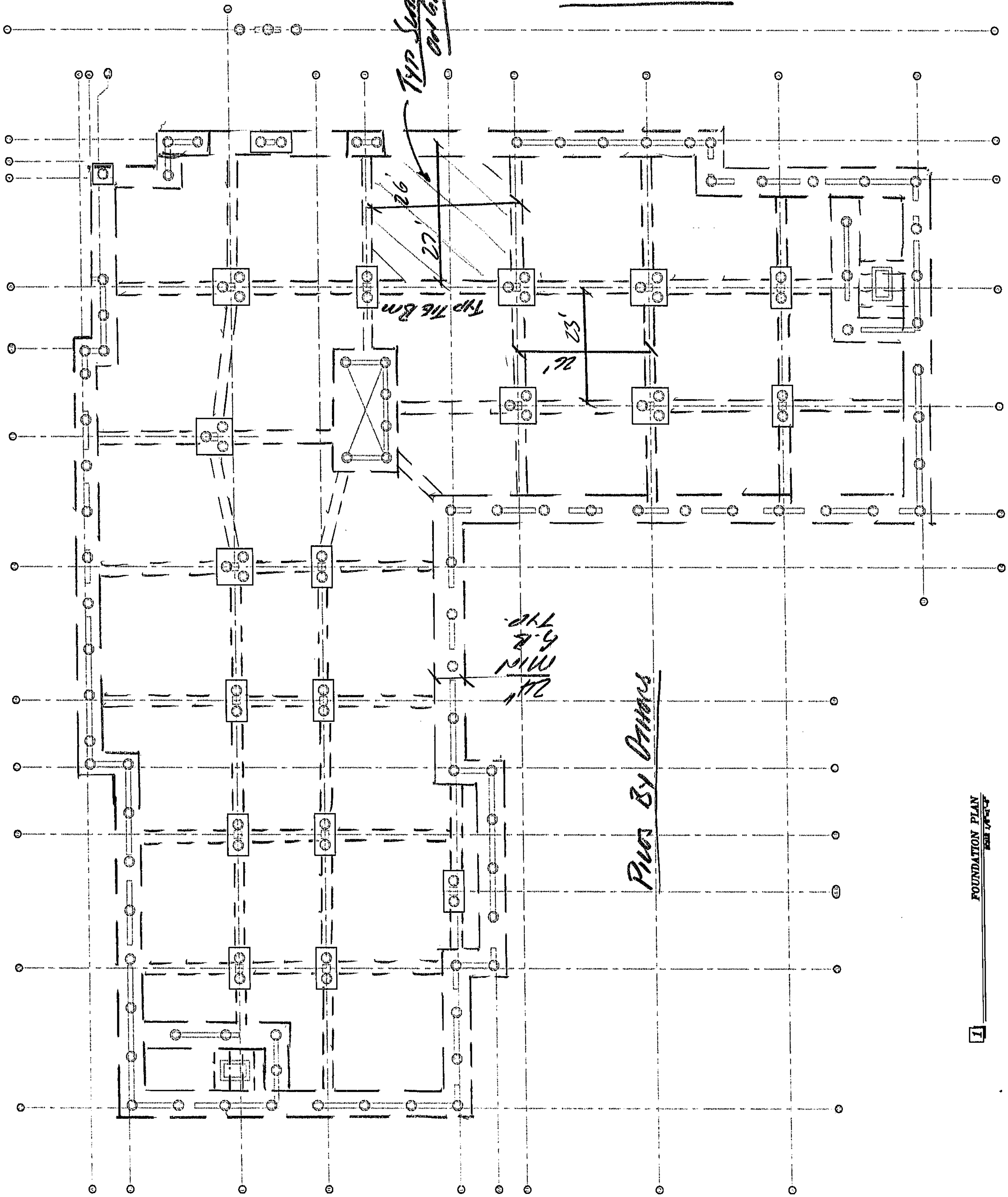


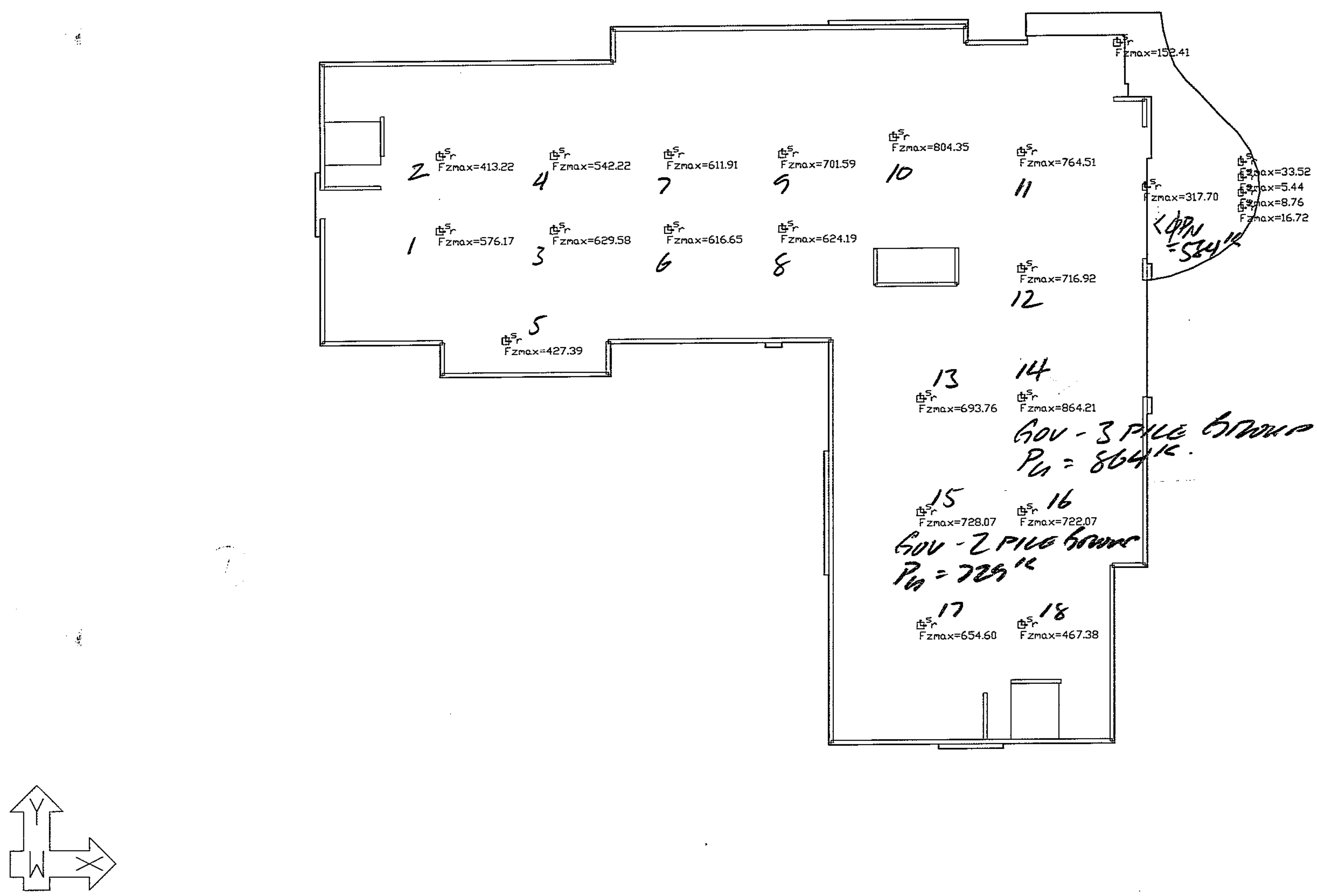
WALL PLAN


12C
12B
12A
13
11
10C
10B
10A
9C
9B
9A
16
17
18E
18D
18C
18B
18A
19
20C
20B
20A
21
22
1A
1B
2
3A
3B
3C
4
5A
5B
6
7A
7C
7B
23
24
25
8G
8A
8B
8C
8D
8E
8F
8H
SHEAR WALL & BEARING WALL
BEARING WALL ONLY

Wall Loads

2017-06 (52)

	SHEAR WALL SUMMARY									BEARING WALLS					
				Strenght Level Loads						Service Level Loads				Strength Level	
				Pu (DEAD LOAD)		Vu			E	Dead		Live		1.2D + 1.6L	
WALL	LENGTH	HEIGHT	THICK	k	k/ft	k	k/ft	Mu	k	k	k/ft	k	k/ft	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	Worst Case G.B.
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	- PILE.
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	- PILE
12C	6.75									183.54	27.2	70.8	10.5	49.4	- PILE
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	- PILE
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	PILES.
24	19.3									275	14.2	222	11.5	35.5	
25	8.7									174	20.0	190	21.8	58.9	



PILE CAPS

Column Reactions Plan_Envelope COLUMN REACTIONS

05/28/17 11:37:11	HOME2 - SSF COLUMN REACTIONS
h2ssf-Adapt-170526-1.adm	

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PILE CAPS

LOAD FROM TIE BEAMS

DL

$SLAB = 156 \times 25 = 3900\ ^{\#}/'$

$TIE\ BM = 150(2.5)(2) = 750\ ^{\#}/'$

$w_{DL} = 4650\ ^{\#}/'$

LL $\quad SLAB = 100 \times 25 = 2500\ ^{\#}/'$

$w_u = 1.2(4650) + 1.6(2500) = 9580\ ^{\#}/'$

ADD'L LOAD PER PILE GROUP $= \phi P_u = 9580(25) = 239500^{\#} \approx 240^{K}$

PILE CAP LOADS

2-PILE GROUP

$\phi P_u = 725 + 240 = 965^{K}$

3-PILE GROUP

$\phi P_u = 864 + 240 = 1104^{K}$

HCP ENGINEERING
STRUCTURAL ENGINEERING & DESIGN
Lic.: C38199

PROJECT : 2-Pile Cap
CLIENT : Home2 - SSF
JOB NO. :
DATE :
PAGE :
DESIGN BY :
REVIEW BY :

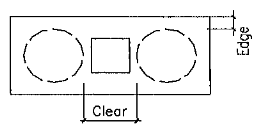
Pile Cap Design for 2-Piles Pattern Based on ACI 318-14

DESIGN CRITERIA

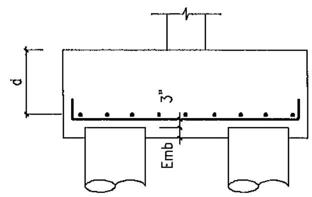
1. FROM PILE DESIGN & SOIL REPORT, DETERMINE SINGLE PILE MAX LOADS OR CAPACITY AT CAP BOTTOM FACE, ϕP_n, ϕM_n, & ϕV_n.
2. THE MAXIMUM COLUMN CAPACITY AT COLUMN BASE, $\phi P_{n,col}$, $\phi M_{n,col}$, $\phi V_{n,col}$, MAY BE BASED ON PILE CAP BALANCED LOADS. USER CAN CHANGE THE YELLOW CELLS TO MODIFY THEM FOR DIFFERENT CASES.
3. PILE CAPS SHALL BE INTERCONNECTED BY TIES WITH 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:

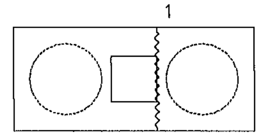
$P_{u,col} \le \phi P_{n,col}$ =	969	kips
$M_{u,col} \le \phi M_{n,col}$ =	0.0	ft-kips
$V_{u,col} \le \phi V_{n,col}$ =	0	kips

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



FLEXURE CRITICAL SECTION

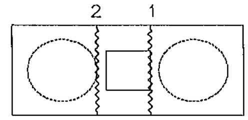
$$\rho = \frac{0.85 f_c'\left(1-\sqrt{1-\frac{M_{u,\max}}{0.383 b d^2 f_c'}}\right)}{f_y} = \quad 0.0000 \quad < \quad \rho_{provD} \qquad \textbf{[Satisfactory]}$$

$$\rho_{MAX} = \frac{0.85\beta_1 f_c'}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = \quad 0.0243 \quad > \quad \rho_{provD} \qquad \textbf{[Satisfactory]}$$

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) = \quad 0.0000 \quad < \quad \rho_{provD} \qquad \textbf{[Satisfactory]}$$

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.)



ONE WAY SHEAR CRITICAL SECTION

$\phi V_c = 2\ \phi\ b\ d\ (f_c')^{0.5}$ = 50.9 kips / ft > $V_{u,max}$ **[Satisfactory]**
where ϕ = 0.75 (ACI 318 21.2)

CHECK COLUMN PUNCHING CAPACITY (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$$\phi v_c(psi) = \phi(2+y)\sqrt{f_c'} = \ 212 \text{ ksi} \quad > \quad v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} = \ 132 \text{ ksi} \qquad \textbf{[Satisfactory]}$$

where
β_c = 1.00
b_1 = (C + d) = 58 in
b_2 = [(C + min(Edge , d)] = 34 in
b_0 = $2b_1 + 2b_2$ = 184 in
A_p = b_0 d = 7360 in^2

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = 0.4654 \qquad y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right) = 2.0$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1+\left(\frac{d}{b_1}\right)^2+3\left(\frac{b_2}{b_1}\right)\right] = 203 \ ft^4$$

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CHECK SINGLE PILE PUNCHING CAPACITY (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$\phi v_c(psi) = \phi(2+y)\sqrt{f'_c} =$ 106 ksi **>** $v_u(psi) = \dfrac{P_{u,col}}{A_P} + \dfrac{0.5\gamma_v M_{u,col} b_1}{J} =$ 93 ksi **[Satisfactory]**

where $\alpha =$ 180.0 deg

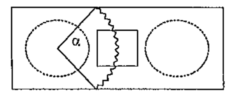
$b_1 =$ $(\alpha / 360)\,(D\pi / 4 + d) =$ 26 in

$b_2 =$ $(\alpha / 360)\,(D\pi / 4 + d) =$ 26 in

$b_0 =$ $(\alpha / 360)\,(D + d)\,\pi =$ 88 in, conservative value

$A_p =$ $b_0\, d$ = 3519 in^2

$$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] = \quad 37 \ ft^4$$



PILE PUNCHING CRITICAL SECTION

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = \quad 0.4$$

$y =$ 0 , conservative value as one way shear

2017-06

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HCP ENGINEERING

PROJECT : 3-Pile Cap
CLIENT : Home2 - SSF
JOB NO. : DATE :

PAGE :
DESIGN BY :
REVIEW BY :

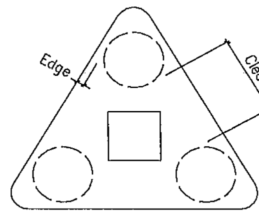
Pile Cap Design for 3-Piles Pattern Based on ACI 318-14

DESIGN CRITERIA

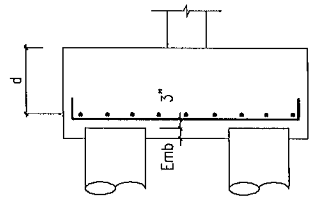
1. FROM PILE DESIGN & SOIL REPORT, DETERMINE SINGLE PILE MAX LOADS OR CAPACITY AT CAP BOTTOM FACE, ϕP_n, ϕM_n, & ϕV_n.
2. THE MAXIMUM COLUMN CAPACITY AT COLUMN BASE, $\phi P_{n,col}$, $\phi M_{n,col}$, $\phi V_{n,col}$, MAY BE BASED ON PILE CAP BALANCED LOADS. USER CAN CHANGE THE YELLOW CELLS TO MODIFY THEM FOR DIFFERENT CASES.
3. PILE CAPS SHALL BE INTERCONNECTED BY TIES WITH 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:

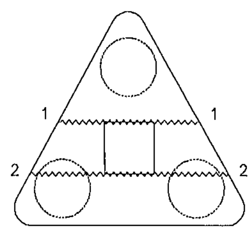
$P_{u,col} \le \phi P_{n,col}$ =	1104	kips
$M_{u,col} \le \phi M_{n,col}$ =	0.0	ft-kips
$V_{u,col} \le \phi V_{n,col}$ =	0	kips

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 = 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



FLEXURE CRITICAL SECTION

$$\rho = \frac{0.85 f_c'\left(1-\sqrt{1-\frac{M_{u,\max}}{0.383 b d^2 f_c'}}\right)}{f_y} = 0.0010 < \rho_{provD}$$ **[Satisfactory]**

$$\rho_{MAX} = \frac{0.85 \beta_1 f_c'}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} = 0.0243 > \rho_{provD}$$ **[Satisfactory]**

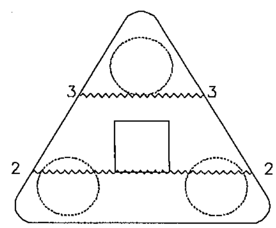
$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right) = 0.0013 < \rho_{provD}$$ **[Satisfactory]**

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 kips / ft > $V_{u,\max}$ **[Satisfactory]**
where ϕ = 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)

$$\phi v_c(psi) = \phi(2+y)\sqrt{f_c'} = 212 \text{ ksi} \quad > \quad v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} = 119 \text{ ksi}$$ **[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

$$\gamma_v = 1 - \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = 0.4$$

$$y = MIN\left(2, \frac{4}{\beta_c}, 40\frac{d}{b_0}\right) = 2.0$$

$$J = \left(\frac{d b_1^3}{6}\right)\left[1+\left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] = 281 \text{ ft}^4$$

(cont'd)

CHECK SINGLE PILE PUNCHING CAPACITY (ACI 318 13.2.7.2, 22.6.4.1, 22.6.4.3, & 8.4.2.3)

$\phi v_c(psi) = \phi(2+y)\sqrt{f'_c} =$ 106 ksi **>** $v_u(psi) = \frac{P_{u,col}}{A_P} + \frac{0.5\gamma_v M_{u,col} b_1}{J} =$ 94 ksi **[Satisfactory]**

where

$\alpha =$ 178.0 deg

$b_1 =$ $(\alpha / 360)(D\pi / 4 + d) =$ 26 in

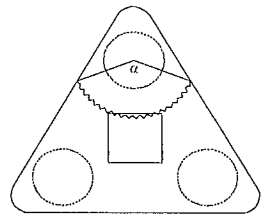
$b_2 =$ $(\alpha / 360)(D\pi / 4 + d) =$ 26 in

$b_0 =$ $(\alpha / 360)(D + d)\pi =$ 87 in, conservative value

$A_p =$ $b_0 d$ = 3479 in^2

$J = \left(\frac{d b_1^3}{6}\right)\left[1 + \left(\frac{d}{b_1}\right)^2 + 3\left(\frac{b_2}{b_1}\right)\right] =$ 36 ft^4

$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} =$ 0.4



PILE PUNCHING CRITICAL SECTION

y = 0 , conservative value as one way shear

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		=	4586,913
			6,875,113#

FRICTION = 0.35

$V_{FRIC} = 0.35(6875) = 2407^K > V_S = 2148^K$

PASSIVE

PASSIVE = 300 PSF

MIN. LENGTH GR. BM OR FTG PERPENDICULAR TO V_S

$L_{min} = 163(4) + 63(5) = 967'$

$H = 30'' = 2'$-$6''$ (12" BELOW GR.)

$V_{PASSIVE} = 300(967)(2.5) = 725,250^{\#}$

$V_{R_{TOTAL}} = V_{FRIC} + V_{PASS} = 2407 + 725 = 3132^K > V_S = 2148^K$

∴ SHEAR TO PILES FROM SEISMIC IS ϕ

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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

$L_u = 6'0"$

$W_{DL} = 20.0$ PSF
$W_{LL} = 20.0$ PSF
$W_{TL} = 40.0$ PSF

ASC STEEL DECK
B-36 x 20 GA
$W_u = 66$ PSF > 40 PSF
(ICC-ES-ESR-1414)

PORTE COCHERE

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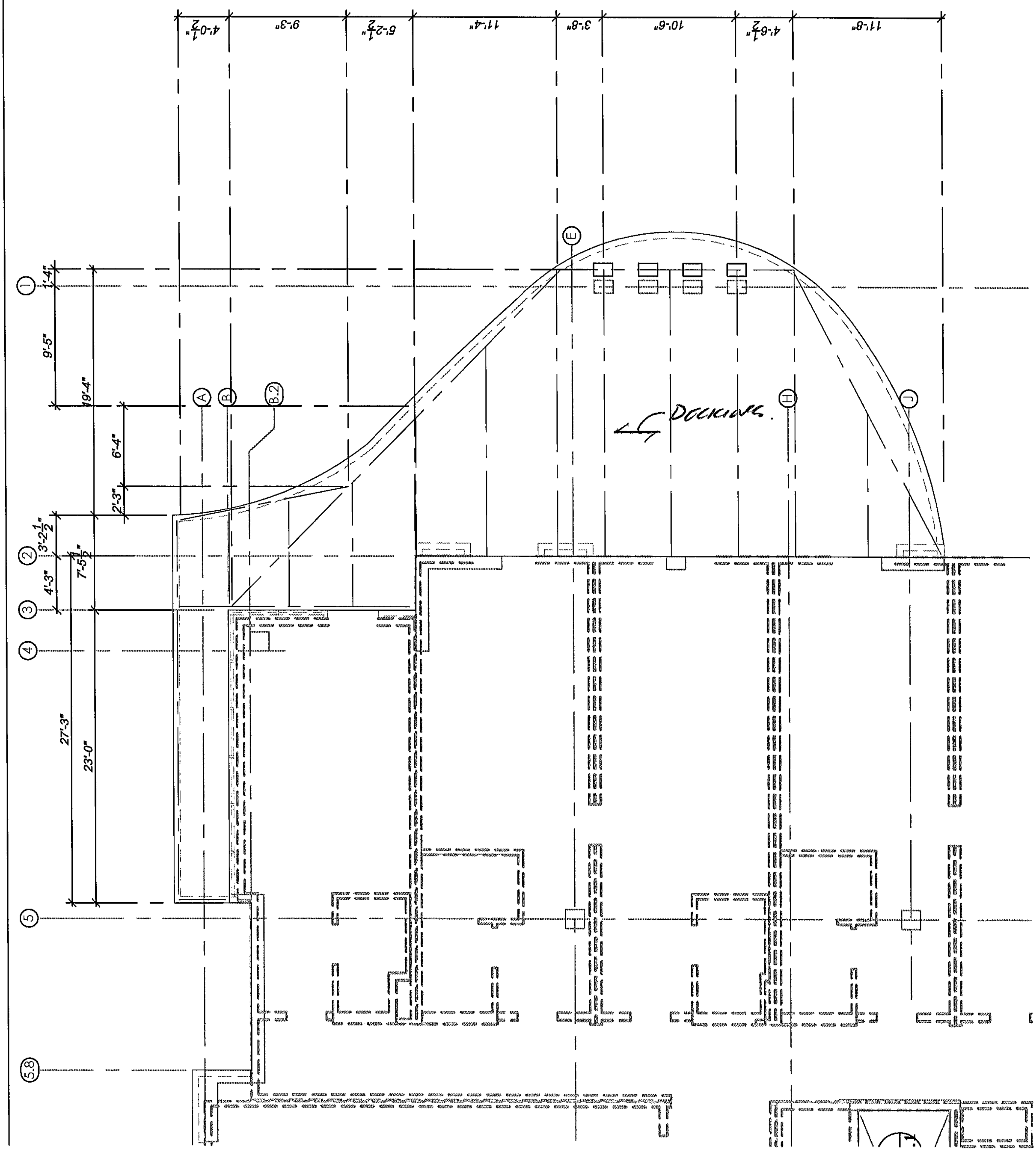

2017-06
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SEISMIC BASE SHEAR

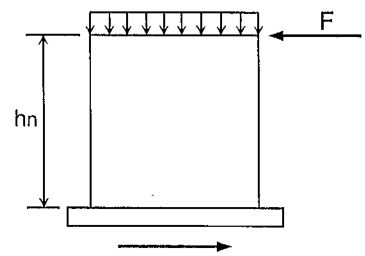
Per CBC 2016 and ASCE 7-10 Specifications

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

Job Name:	HOME2 - SSF (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., Ss =	1.506	ASCE 7-10 Figures 22-1, 3, 5, 6
Spectral Accel., S1 =	0.600	ASCE 7-10 Figures 22-2, 4, 5, 6
Long. Trans. Period, TL =	8.000	sec. ASCE 7 Fig's. 22-12 to 22-16
Structure Height, hn =	14.000	ft.
Total Seismic Weight, W =	24.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, Tc =	0.000	sec. from Analysis or "0" if T_a (below)
Seismic Resist. System =	G1	G1. Steel special cantilevered column system (ASCE 7-10, Table 12.2-1)



Seismic Base Shear
(Regular Bldg. Configurations Only)

Results:

Site Coefficients:

Fa =	1.000	ASCE 7-10, Table 11.4.-1
Fv =	1.500	ASCE 7-10, Table 11.4-2

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

SMS =	1.506	SMS = Fa*SS, ASCE 7-10, Eqn 11.4-1
SM1 =	0.900	SM1 = Fv*S1, ASCE 7-10, Eqn 11.4-2

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

SDS =	1.004	SDS = 2*SMS/3, ASCE -10, Eqn 11.4-3
SD1 =	0.600	SD1 = 2*SM1/3, ASCE 7-10 Eqn 11.4-4

Seismic Design Category:

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

Fundamental Period:

Period Coefficient, CT =	0.020	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2
Approx. Period, Ta =	0.145	sec., Ta = CT*hn^(x), ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., Cu =	1.400	ASCE 7-10 Table 12.8-1
Period max., T(max) =	0.203	sec., T(max) = Cu*Ta, ASCE 7-10 Section 12.8.2
Fundamental Period, T =	0.145	sec., T = Ta <= Cu*Ta, 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, Ωo =	1.25	ASCE 7-10 Table 12.2-1
Defl. Amplif. Factor, Cd =	2.5	ASCE 7-10 Table 12.2-1
CS =	0.402	CS = SDS/(R/I), ASCE 7-10 Section 12.8.1.1, Eqn. 12.8-2
CS(max) =	1.658	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-10 Eqn. 12.8-3
CS(min) =	0.120	CS(min) = 0.5*S1/(R/I), ASCE 7-10 Eqn. 12.8-6
Use: Cs =	0.402	CS(min) <= CS <= CS(max)
C_{ASD} =	0.281	CASD = 0.7 x CS

Seismic Base Shear:

Redundancy, Rho ρ =	1.3	ASCE 12.3.4.1
V =	12.53	kips, V = CS*W, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1
V_{ASD} =	8.77	kips, V_{ASD} = 0.7 x V

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MecaWind Pro v2.2.7.5 per ASCE 7-10

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```
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
Structural Category       =     II        Exposure Category       =       C
Natural Frequency         =   N/A         Flexible Structure      =      No
Importance Factor         =    1.00       Kd Directional Factor   =    0.85
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*Ht                                                =  15.00 ft
lzm:    Cc*(33/Zm)^0.167                                      =   0.23
Lzm:    l*(Zm/33)^Epsilon                                     = 427.06 ft
Q:      (1/(1+0.63*((B+Ht)/Lzm)^0.63))^0.5                    =   0.94
Gust2: 0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*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 = -26.0 PSI

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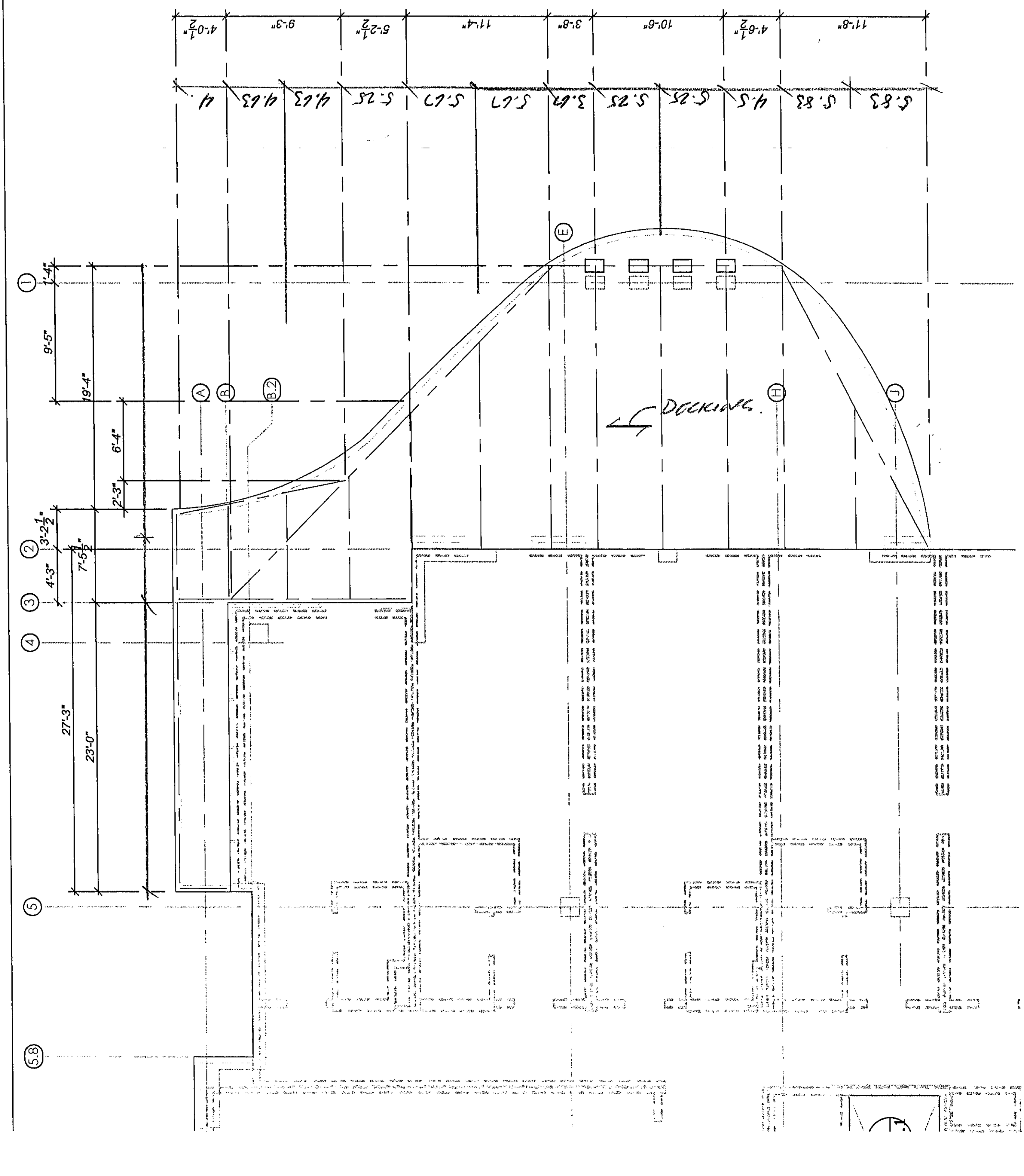
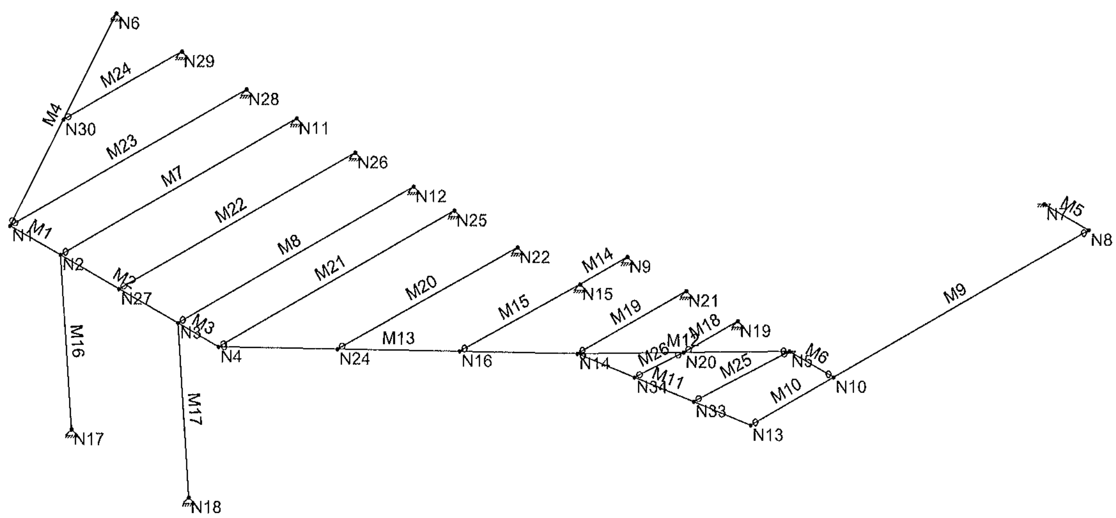
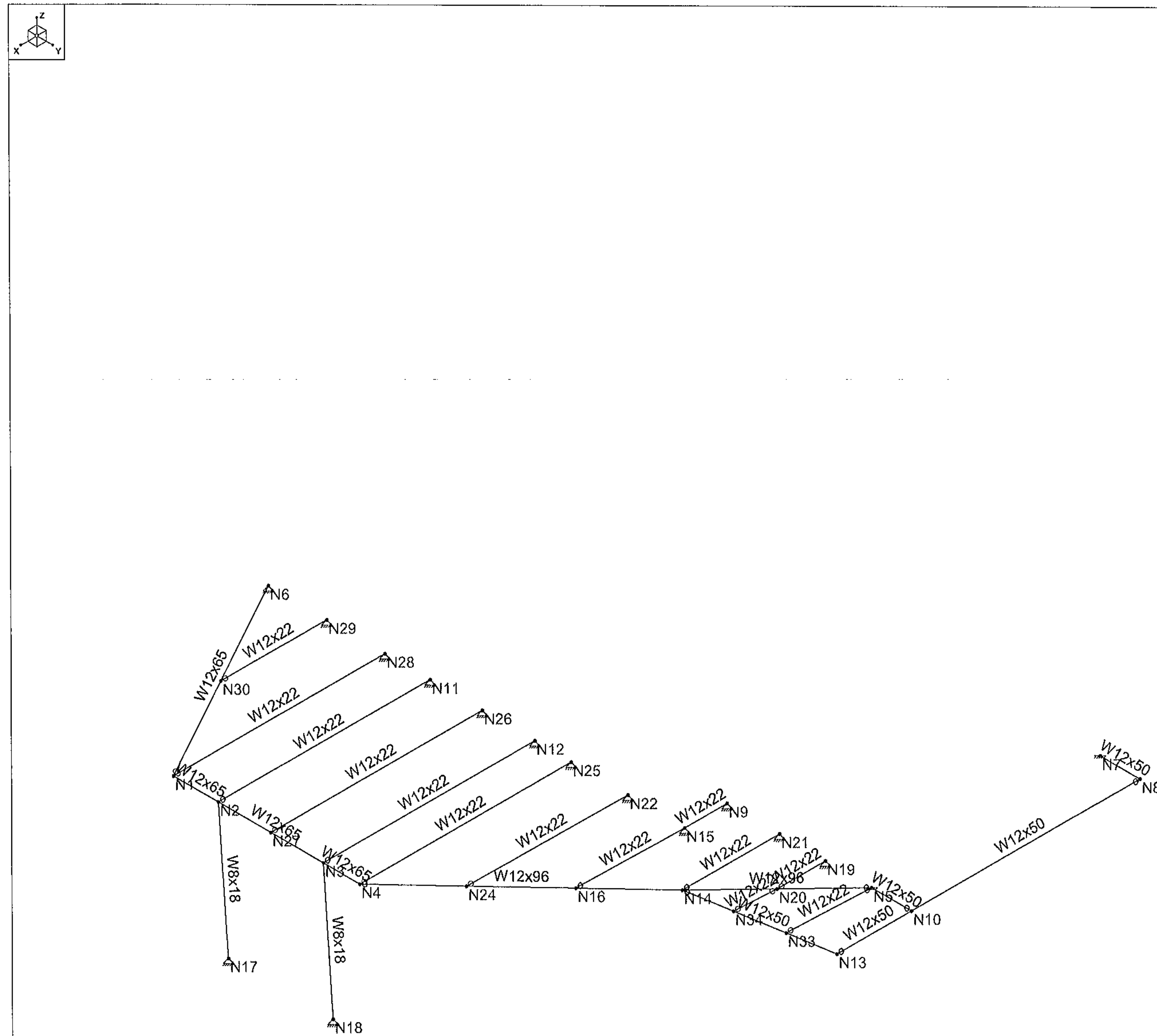


2017-06
(106)



HCP ENGINEERING		SK - 3
		Aug 15, 2017 at 11:12 AM
	MODEL	2017-06-h2ssf-PorteCo.r3d

2017-06 (107)



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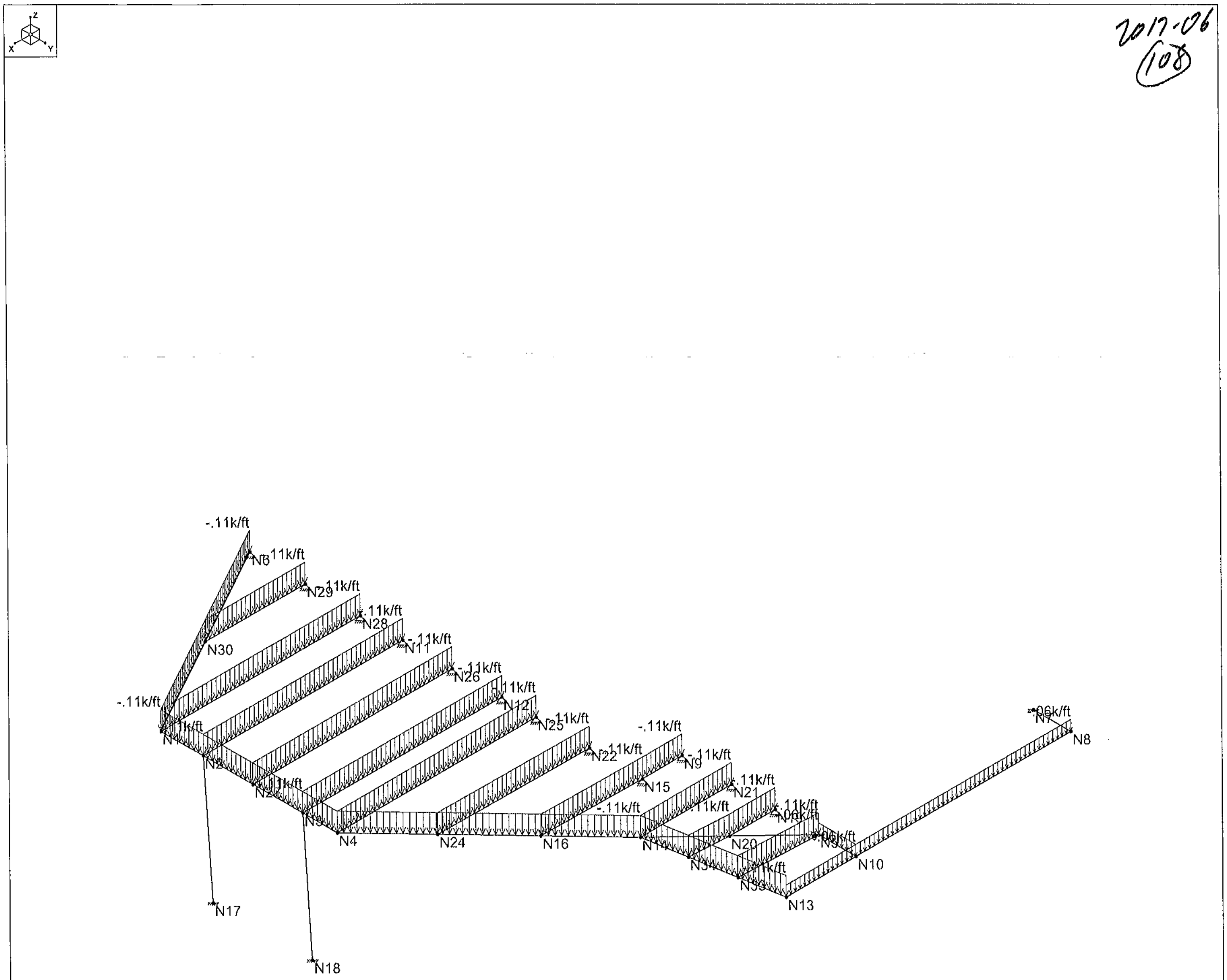
		Aug 15, 2017 at 11:12 AM
	MODEL	2017-06-h2ssf-PorteCo.r3d

2017-06
(108)



Loads: BLC 1, D
Envelope Only Solution

HCP ENGINEERING		SK - 2
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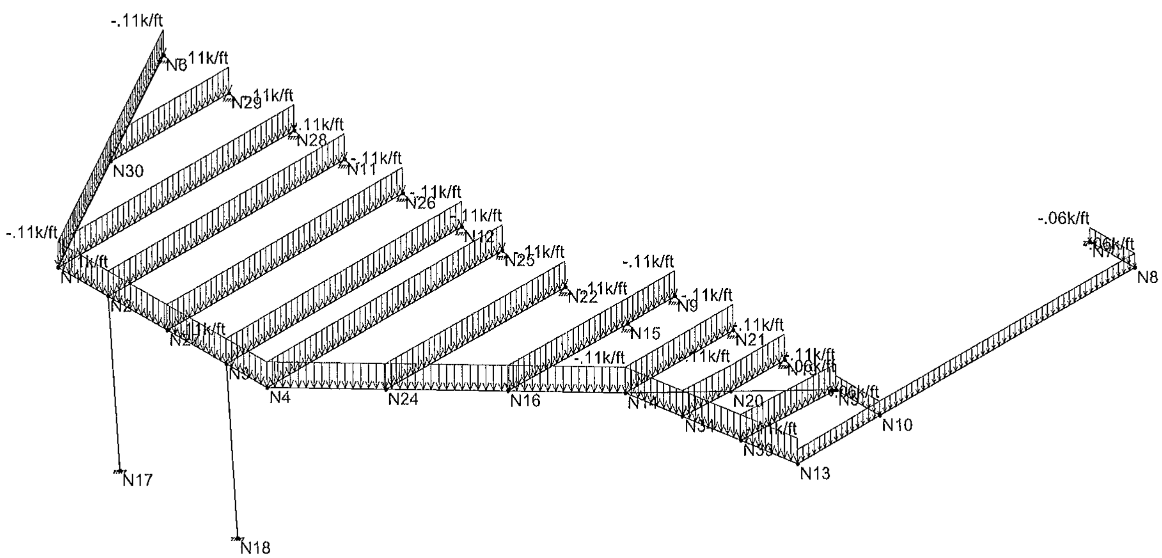
	DEAL LOAD	2017-06-h2ssf-PorteCo.r3d

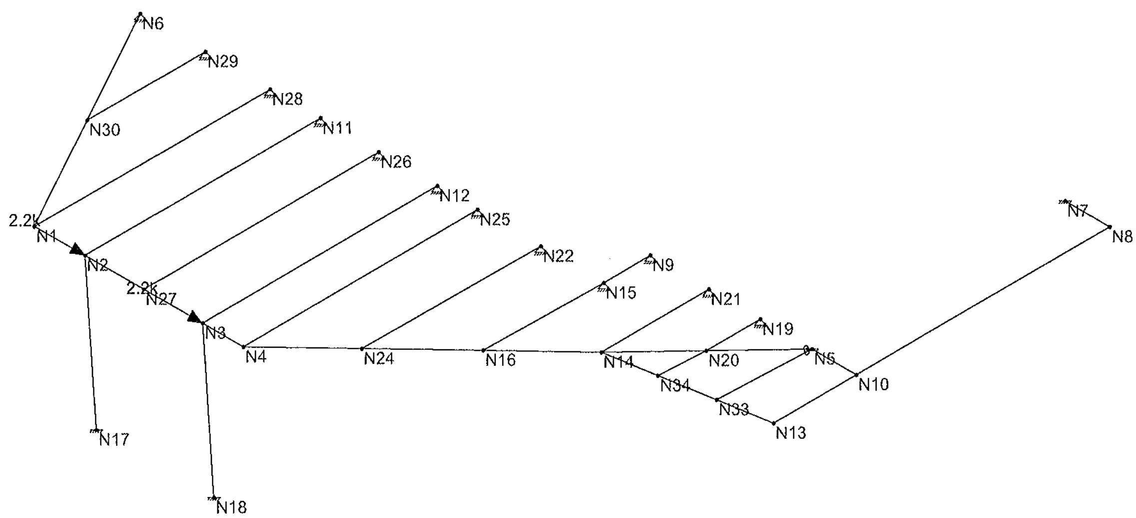
2017-06
(109)



Loads: BLC 2, L
Envelope Only Solution

HCP ENGINEERING		SK - 3
		Aug 11, 2017 at 2:32 PM
	LIVE LOAD	2017-06-h2ssf-PorteCo.r3d

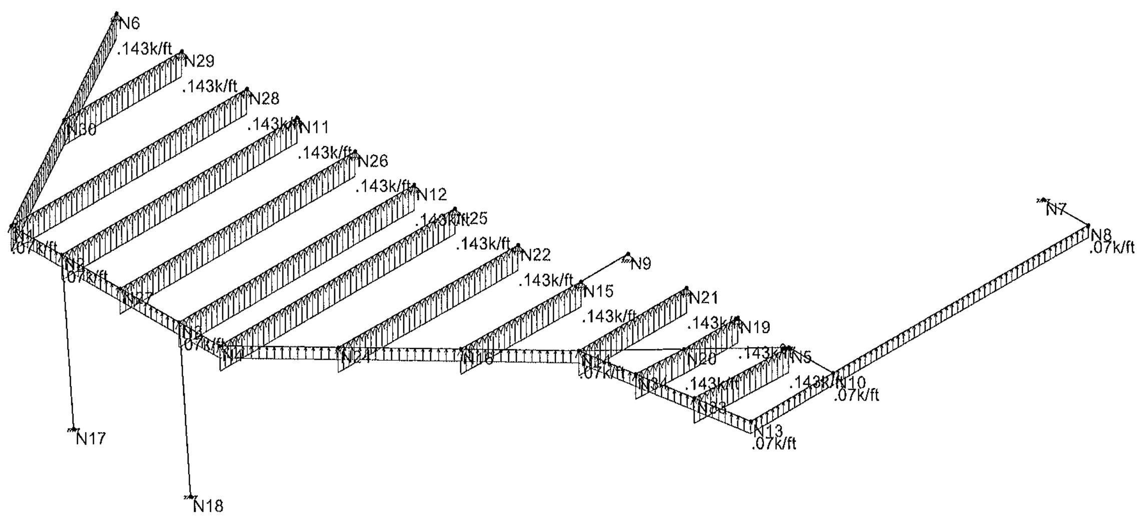
2017-06
(116)



Loads: BLC 6, EY

HCP ENGINEERING		SK - 6
		Aug 11, 2017 at 2:45 PM
	SEISIMIC - EQX	2017-06-h2ssf-PorteCo.r3d

2017-06
(111)



Loads: BLC 3, WX

HCP ENGINEERING		SK - 5
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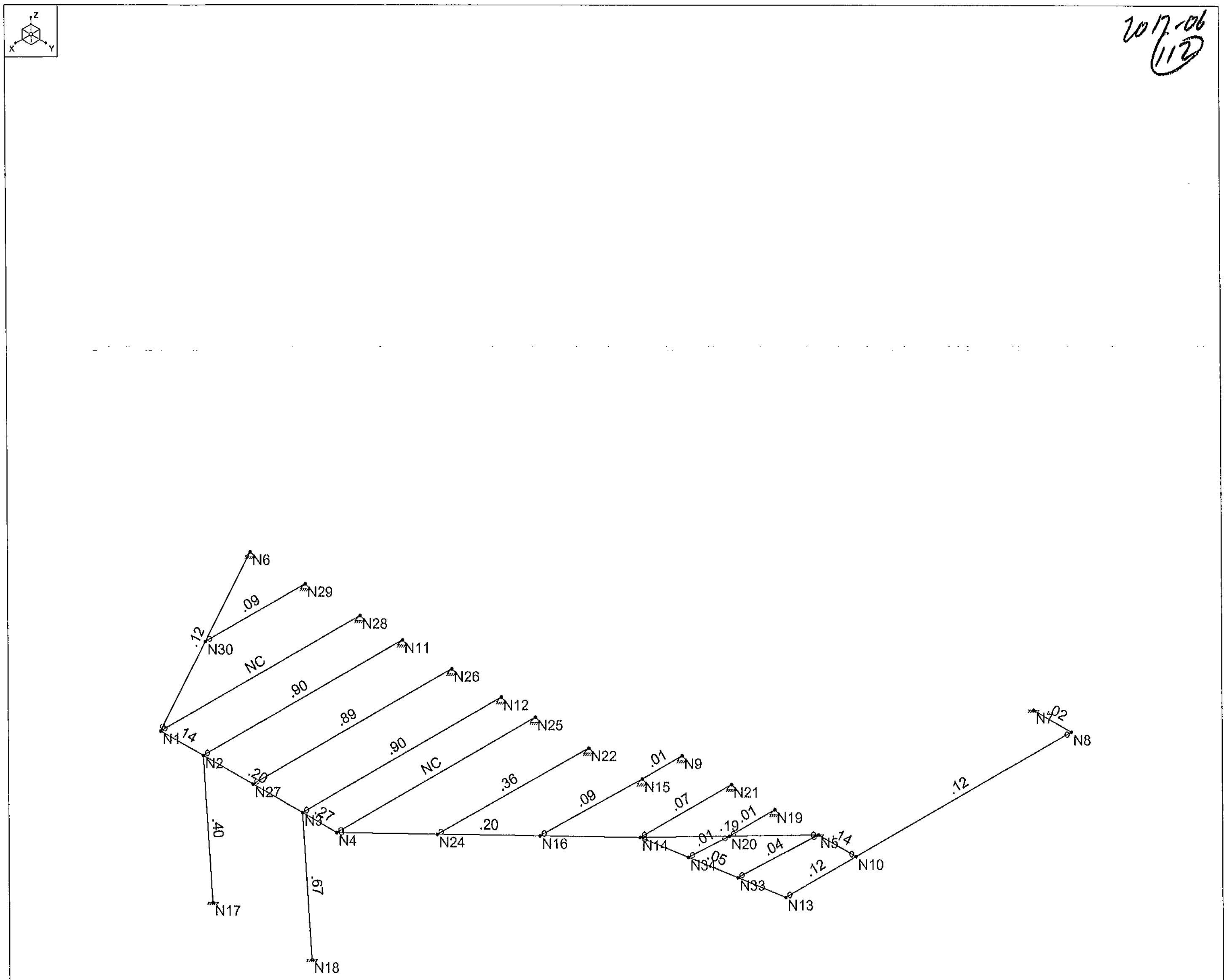
	WIND-WX	2017-06-h2ssf-PorteCo.r3d

2017-06 (112)



Member Code Checks Displayed (Enveloped)
Envelope Only Solution

HCP ENGINEERING		SK - 14
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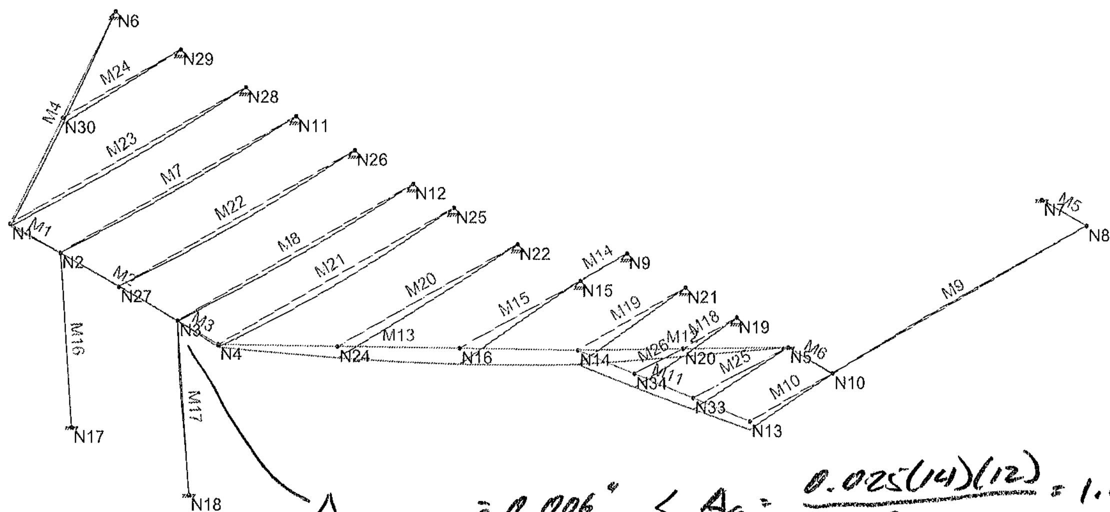
	UNITY	2017-06-h2ssf-PorteCo.r3d

2017-06
(113)



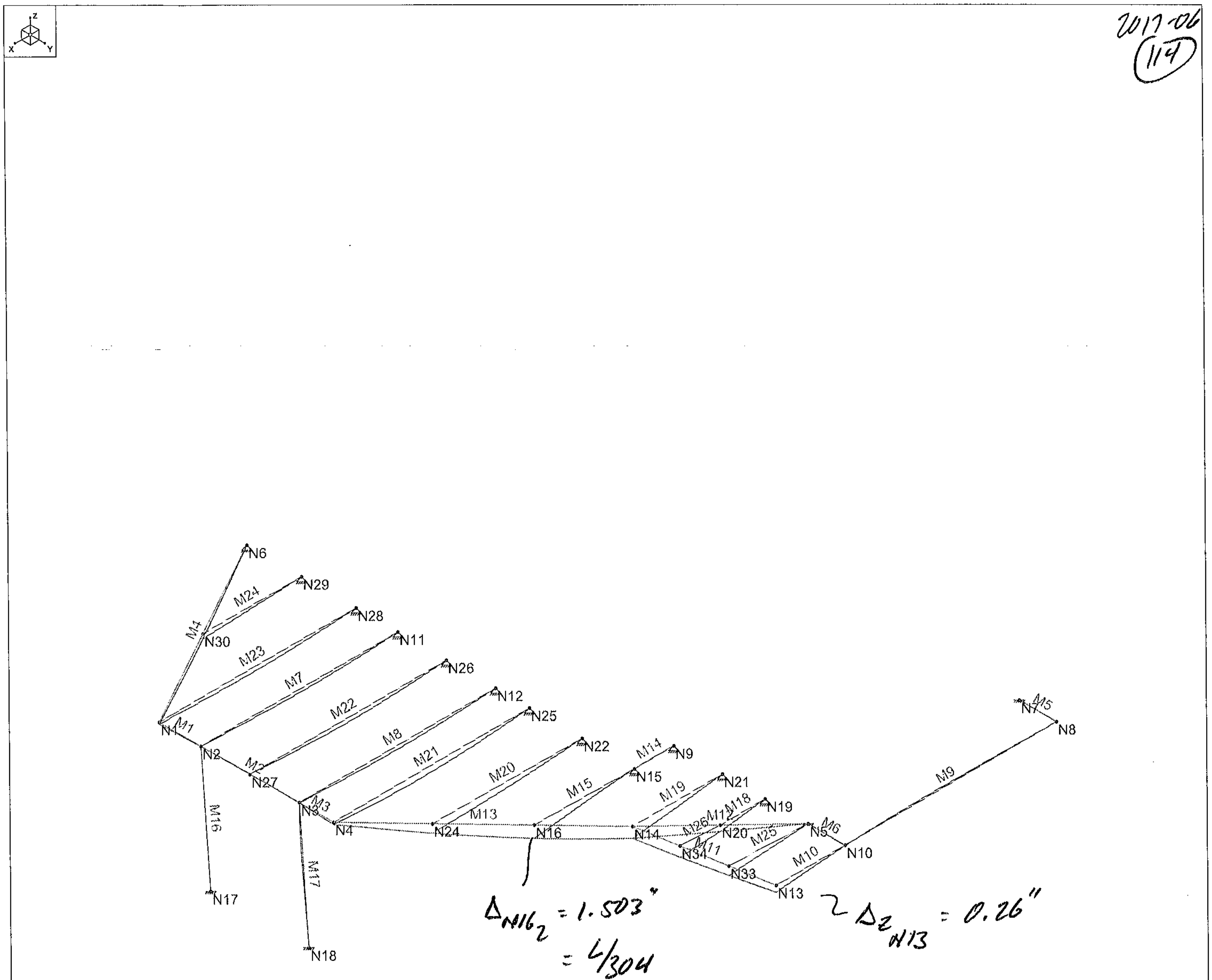
$\Delta_{y_{N2 \,\&\, N3}} = 0.006'' < \Delta_a = \dfrac{0.025(14)(12)}{2.5} = 1.67.$

O/K

Results for LC 19, 8) D + 0.7EY

HCP ENGINEERING		SK - 10
		Aug 11, 2017 at 2:52 PM
	DRIFT	2017-06-h2ssf-PorteCo.r3d

2017-06 (114)



Results for LC 3, 2a) D + L

HCP ENGINEERING		SK - 11
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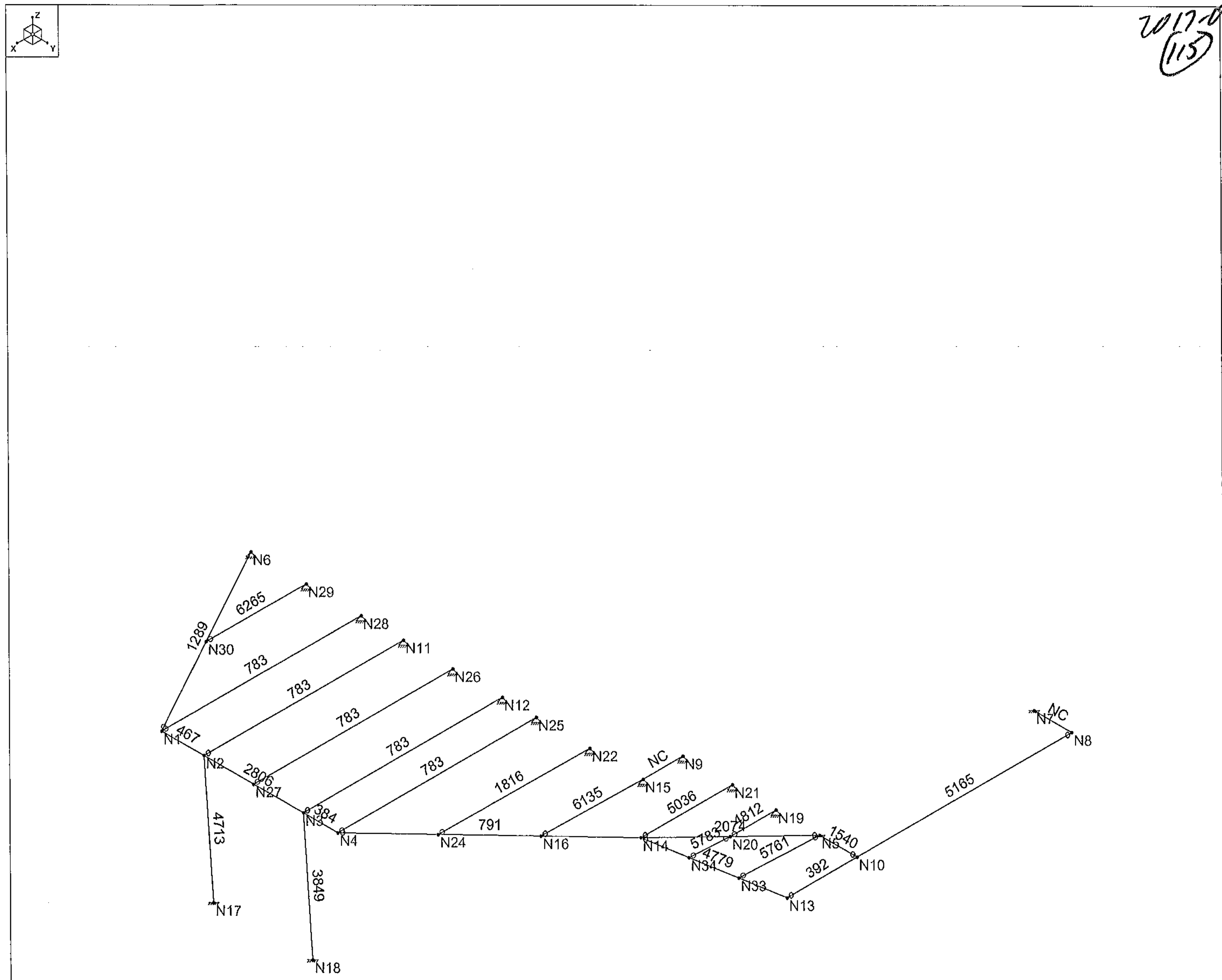
	DELECTION	2017-06-h2ssf-PorteCo.r3d

2017-06
(115)



L/y Deflection Ratio (Enveloped)
Envelope Only Solution

HCP ENGINEERING		SK - 15
		Aug 11, 2017 at 3:56 PM
	DEFLECTION RATIO	2017-06-h2ssf-PorteCo.r3d

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (\...	Density[k...	Yield[ksi]	Ry	Fu[ksi]	Rt
1	A36 Gr.36: Shapes M, S, C MC & L	29000	11154	.3	.65	.49	36	1.5	58	1.2
2	A572 Gr.50, Shapes: HP	29000	11154	.3	.65	.49	50	1.1	65	1.1
3	A992- SHapes: W's	29000	11154	.3	.65	.49	50	1.1	65	1.1
4	A500 Gr.B Shapes: HSS RND	29000	11154	.3	.65	.527	42	1.4	58	1.3
5	A500 Gr.B - Shapes Rect	29000	11154	.3	.65	.527	46	1.4	58	1.3
6	A53 Gr.B - Shapes: Pipes	29000	11154	.3	.65	.49	35	1.6	60	1.2
7	A1085	29000	11154	.3	.65	.49	50	1.4	65	1.3

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	BM1	W12x65	Beam	None	A992- SHapes: W's	Typical	19.1	174	533	2.18
2	BM2	W12x65	Beam	None	A992- SHapes: W's	Typical	19.1	174	533	2.18
3	BM3	W12x96	Beam	None	A992- SHapes: W's	Typical	28.2	270	833	6.85
4	BM4	W12x22	Beam	None	A992- SHapes: W's	Typical	6.48	4.66	156	.293
5	RIM1	W12x50	Beam	None	A992- SHapes: W's	Typical	14.6	56.3	391	1.71
6	RIM2	W12x50	Beam	None	A992- SHapes: W's	Typical	14.6	56.3	391	1.71
7	CANT1	W12x50	Beam	None	A992- SHapes: W's	Typical	14.6	56.3	391	1.71
8	LDGR	W10x33	Beam	None	A992- SHapes: W's	Typical	9.71	36.6	171	.583
9	COL	W8x18	Column	Wide Flange	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	N1	48.5	11.67	0	0	
2	N2	48.5	16.17	0	0	
3	N3	48.5	26.67	0	0	
4	N4	48.5	30.34	0	0	
5	N5	23	56.17	0	0	
6	N6	27.25	0	0	0	
7	N7	0	56.17	0	0	
8	N8	0	60.17	0	0	
9	N9	23	41.67	0	0	
10	N10	23	60.17	0	0	
11	N11	27.25	16.17	0	0	
12	N12	27.25	26.67	0	0	
13	N13	30.5	60.17	0	0	
14	N14	32.75	46.92	0	0	
15	N15	27.25	41.67	0	0	
16	N16	37.9475	41.4486	0	0	
17	N17	47.56	16.17	-14	0	
18	N18	47.56	26.67	-14	0	
19	N19	23	51.55	0	0	
20	N20	27.875	51.545	0	0	
21	N21	23	46.92	0	0	
22	N22	27.25	36	0	0	
23	N24	43.3025	35.8114	0	0	
24	N25	27.25	30.33	0	0	
25	N26	27.25	21.41	0	0	
26	N27	48.5	21.42	0	0	
27	N28	27.25	11.66	0	0	
28	N29	27.25	5.83	0	0	
29	N30	37.875	5.835	0	0	
30	N33	31.2425	55.7975	0	0	
31	N34	32.0075	51.2925	0	0	

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]
1	N7	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
2	N5	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
3	N9	Reaction	Reaction	Reaction			
4	N15	Reaction	Reaction	Reaction			
5	N12	Reaction	Reaction	Reaction			
6	N11	Reaction	Reaction	Reaction			
7	N6	Reaction	Reaction	Reaction			
8	N17	Reaction	Reaction	Reaction			
9	N18	Reaction	Reaction	Reaction			
10	N29	Reaction	Reaction	Reaction			
11	N28	Reaction	Reaction	Reaction			
12	N26	Reaction	Reaction	Reaction			
13	N25	Reaction	Reaction	Reaction			
14	N21	Reaction	Reaction	Reaction			
15	N19	Reaction	Reaction	Reaction			
16	N22	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	N4			BM3	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	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		AllPIN				Yes			None
7	M7		AllPIN				Yes			None
8	M8		AllPIN				Yes			None
9	M9	AllPIN					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		AllPIN				Yes			None
11	M11		AllPIN				Yes			None
12	M12	AllPIN					Yes			None
13	M13		BenPIN				Yes			None
14	M14						Yes			None
15	M15		AllPIN				Yes			None
16	M16						Yes			None
17	M17						Yes			None
18	M18		BenPIN				Yes			None
19	M19		AllPIN				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	BenPIN				Yes			None
26	M26	BenPIN	BenPIN				Yes			None

Joint Loads and Enforced Displacements (BLC 6 : EY)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/f...
1	N2	L	Y	2.2
2	N3	L	Y	2.2

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,F...	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,F...	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,F...	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	M26	Z	-.11	-.11	0	0
19	M10	Z	-.06	-.06	0	0
20	M9	Z	-.06	-.06	0	0
21	M6	Z	-.06	-.06	0	0
22	M5	Z	-.06	-.06	0	0
23	M14	Z	-.11	-.11	0	0

Member Distributed Loads (BLC 3 : WX)

	Member Label	Direction	Start Magnitude[k/ft,...	End Magnitude[k/ft,F...	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	M26	Z	.143	.143	0	0
12	M25	Z	.143	.143	0	0
13	M4	Z	.143	.143	0	0
14	M2	Z	.07	.07	0	0
15	M1	Z	.07	.07	0	0
16	M3	Z	.07	.07	0	0
17	M13	Z	.07	.07	0	0
18	M11	Z	.07	.07	0	0
19	M10	Z	.07	.07	0	0
20	M9	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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Load Combinations

	Description	Solve	PDelta	SRSS	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	1a) D	Yes	Y		DL	1																		
2	1b) L	Yes	Y		LL	1																		
3	2a) D + L	Yes	Y		DL	1	LL	1	L...	1														
4	3a) D + Lr	Yes	Y		DL	1	R...	1																
5	3b) D + S		Y		DL	1	SL	1																
6	4a) D + .75L + .75Lr	Yes	Y		DL	1	LL	.75	R...	.75														
7	5) D + .6WX	Yes	Y		DL	1	W...	.6																
8	5) D - 0.6WX	Yes	Y		DL	1	W...	-.6																
9	5) D + .6WY	Yes	Y		DL	1	W...	.6																
10	5) D - .6WY	Yes	Y		DL	1	W...	-.6																
11	6a) D + .75L + .75Lr + ...	Yes	Y		DL	1	LL	.75	R...	.75	W...	.45												
12	6a) D + .75L + .75Lr + ...	Yes	Y		DL	1	LL	.75	R...	.75	W...	.45												
13	6a) D + .75L + .75Lr - ...	Yes	Y		DL	1	LL	.75	R...	.75	W...	-.45												
14	6a) D + .75L + .75Lr -...	Yes	Y		DL	1	LL	.75	R...	.75	W...	-.45												
15	7) 0.6D + 0.6WX	Yes	Y		DL	.6	W...	.6																
16	7) 0.6D - 0.6WX	Yes	Y		DL	.6	W...	-.6																
17	8) D + 0.7EX	Yes	Y		DL	1	E...	.7																
18	8) D - 0.7EX	Yes	Y		DL	1	E...	-.7																
19	8) D + 0.7EY	Yes	Y		DL	1	E...	.7																
20	8) D - 0.7EY	Yes	Y		DL	1	E...	-.7																
21	6a) D + 0.75L + 0.75(0...	Yes	Y		DL	1	LL	.75	R...	.75	E...	.525												
22	6a) D + 0.75L - 0.75(0...	Yes	Y		DL	1	LL	.75	R...	.75	E...	-.5...												
23	6b) D + .75(.7EX) + .7...		Y		DL	1	E...	.75	SL	.75														
24	6b) D -.75(.7EX) + .75S		Y		DL	1	E...	-.75	SL	.75														
25	6b) D + 0.75L + 0.75(0...	Yes	Y		DL	.6	R...	.75	E...	.525														
26	6b) D + 0.75L - 0.75(0...	Yes	Y		DL	.6	R...	.75	E...	-.5...														
27	6b) D + .75EY + .75S		Y		DL	.6	E...	-.7																
28	6b) D -.75EY + .75S		Y		DL	.6	E...	-.7																

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N7	max	0	20	0	19	.804	13	2.854	13	0	1	.001	19
2		min	0	19	0	20	-.092	15	-.368	15	0	1	-.001	20
3	N5	max	2.402	19	2.273	20	13.155	13	24.755	13	0	15	.001	19
4		min	-2.405	20	-2.271	19	.093	15	.299	15	-.009	13	-.001	20
5	N9	max	0	19	0	19	.467	3	0	1	0	1	0	1
6		min	0	20	0	20	.14	16	0	1	0	1	0	1
7	N15	max	.07	19	.002	20	1.783	13	0	1	0	1	0	1
8		min	-.063	20	-.002	19	.034	15	0	1	0	1	0	1
9	N12	max	-.007	15	0	20	2.729	13	0	1	0	1	0	1
10		min	-.946	13	0	19	-.21	15	0	1	0	1	0	1
11	N11	max	.041	15	0	20	2.729	13	0	1	0	1	0	1
12		min	-.622	13	0	19	-.21	15	0	1	0	1	0	1
13	N6	max	1.422	20	.771	20	3.796	13	0	1	0	1	0	1
14		min	-1.521	19	-.825	19	-.293	15	0	1	0	1	0	1
15	N17	max	.842	13	.012	15	11.957	13	0	1	0	1	0	1
16		min	-.051	15	-.226	13	-.77	15	0	1	0	1	0	1
17	N18	max	1.29	13	.275	13	17.1	13	0	1	0	1	0	1
18		min	.005	15	-.003	15	.05	15	0	1	0	1	0	1
19	N29	max	.038	19	0	19	1.365	13	0	1	0	1	0	1
20		min	-.038	20	0	20	-.105	15	0	1	0	1	0	1
21	N28	max	1.369	19	0	19	2.729	13	0	1	0	1	0	1
22		min	-1.359	20	0	20	-.21	15	0	1	0	1	0	1
23	N26	max	.005	15	0	15	2.729	13	0	1	0	1	0	1
24		min	-.273	13	0	13	-.21	15	0	1	0	1	0	1
25	N25	max	1.862	20	0	20	2.729	13	0	1	0	1	0	1
26		min	-2.027	19	-.001	19	-.21	15	0	1	0	1	0	1
27	N21	max	.207	20	0	3	1.252	13	0	1	0	1	0	1

Envelope Joint Reactions (Continued)

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
28		min	-.208	19	0	19	-.097	15	0	1	0	1	0	1
29	N19	max	.032	20	0	19	.626	13	0	1	0	1	0	1
30		min	-.032	19	0	20	-.048	15	0	1	0	1	0	1
31	N22	max	.135	20	.002	19	2.062	13	0	1	0	1	0	1
32		min	-.146	19	-.002	20	-.159	15	0	1	0	1	0	1
33	Totals:	max	0	19	3.08	20	67.953	13						
34		min	0	20	-3.08	19	-2.298	15						

Envelope Joint Displacements

	Joint		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation ...	LC	Y Rotation ...	LC	Z Rotation ...	LC
1	N1	max	.002	20	.007	19	.008	15	2.31e-03	13	1.982e-03	13	3.319e-05	20
2		min	-.002	19	-.006	20	-.135	13	-1.379e-04	15	2.737e-05	15	-6.045e-05	19
3	N2	max	.001	13	.007	19	.001	15	1.501e-03	13	1.982e-03	13	2.412e-05	20
4		min	0	15	-.006	20	-.017	13	-8.282e-05	15	2.737e-05	15	-1.955e-05	19
5	N3	max	.002	13	.006	19	0	15	1.679e-05	15	7.226e-03	13	4.039e-05	20
6		min	0	15	-.006	20	-.024	13	-1.833e-03	13	1.025e-04	15	-4.2e-05	19
7	N4	max	.003	19	.006	19	0	15	1.571e-05	15	8.474e-03	13	9.58e-05	20
8		min	-.003	20	-.006	20	-.141	13	-2.753e-03	13	1.203e-04	15	-5.616e-05	19
9	N5	max	0	20	0	19	0	15	0	15	0	13	0	20
10		min	0	19	0	20	0	13	0	13	0	15	0	19
11	N6	max	0	19	0	19	0	15	1.249e-03	13	3.913e-03	13	7.824e-07	20
12		min	0	20	0	20	0	13	6.627e-06	15	-2.358e-04	15	-3.667e-07	19
13	N7	max	0	19	0	20	0	15	0	15	0	1	0	20
14		min	0	20	0	19	0	13	0	13	0	1	0	19
15	N8	max	0	19	0	20	0	15	1.168e-05	15	0	1	3.011e-07	20
16		min	0	20	0	19	-.004	13	-8.681e-05	13	0	1	-3.013e-07	19
17	N9	max	0	20	0	20	0	16	5.946e-01	13	2.8e-05	3	1.955e-06	20
18		min	0	19	0	19	0	3	6.208e-03	15	8.399e-06	16	-1.946e-06	19
19	N10	max	0	19	0	19	0	15	0	1	1.662e-03	13	1.107e-05	19
20		min	0	20	0	20	-.031	13	0	1	1.163e-04	15	-1.108e-05	20
21	N11	max	0	13	0	19	0	15	0	1	4.172e-03	13	2.617e-05	19
22		min	0	15	0	20	0	13	0	1	-3.207e-04	15	-2.542e-05	20
23	N12	max	0	13	0	19	0	15	0	1	4.2e-03	13	2.548e-05	19
24		min	0	15	0	20	0	13	0	1	-3.163e-04	15	-2.446e-05	20
25	N13	max	0	19	.001	19	-.013	15	-7.692e-04	15	3.784e-01	13	1.252e-06	19
26		min	0	20	-.001	20	-.261	13	-5.455e-02	13	5.024e-03	15	-1.253e-06	20
27	N14	max	0	19	.001	19	-.023	15	-1.518e-03	15	1.143e-01	13	5.888e-06	19
28		min	0	20	-.001	20	-1.702	13	-1.087e-01	13	1.603e-03	15	-5.882e-06	20
29	N15	max	0	20	0	19	0	15	5.946e-01	13	-8.399e-06	16	3.93e-06	19
30		min	0	19	0	20	0	13	6.208e-03	15	-2.8e-05	3	-3.948e-06	20
31	N16	max	0	20	.002	19	-.021	15	-1.081e-03	15	7.567e-02	13	1.158e-06	20
32		min	0	19	-.002	20	-1.508	13	-7.763e-02	13	1.047e-03	15	-1.517e-06	19
33	N17	max	0	15	0	13	0	15	3.959e-05	15	-1.493e-05	15	1.547e-04	13
34		min	0	13	0	15	0	13	-7.237e-04	13	-9.703e-04	13	-8.557e-06	15
35	N18	max	0	15	0	15	0	15	8.797e-04	13	-5.104e-05	15	1.691e-06	15
36		min	0	13	0	13	0	13	-8.298e-06	15	-3.579e-03	13	-1.839e-04	13
37	N19	max	0	19	0	20	0	15	-1.481e-03	15	1.601e-02	13	1.034e-05	19
38		min	0	20	0	19	0	13	-1.057e-01	13	2.057e-04	15	-1.036e-05	20
39	N20	max	0	19	0	19	-.012	15	-1.48e-03	15	1.15e-01	13	7.482e-06	19
40		min	0	20	0	20	-.927	13	-1.056e-01	13	1.607e-03	15	-7.484e-06	20
41	N21	max	0	19	0	19	0	15	0	1	1.495e-02	13	1.153e-05	19
42		min	0	20	0	3	0	13	0	1	1.679e-04	15	-1.154e-05	20
43	N22	max	0	19	0	20	0	15	-6.309e-04	15	7.186e-03	13	1.365e-05	19
44		min	0	20	0	19	0	13	-4.552e-02	13	-6.815e-05	15	-1.359e-05	20
45	N24	max	0	19	.003	19	-.012	15	-6.244e-04	15	3.631e-02	13	3.076e-05	19
46		min	0	20	-.003	20	-.939	13	-4.517e-02	13	4.821e-04	15	-2.863e-05	20
47	N25	max	0	19	0	19	0	15	1.592e-05	15	4.658e-03	13	2.459e-05	19
48		min	0	20	0	20	0	13	-2.752e-03	13	-3.19e-04	15	-2.357e-05	20
49	N26	max	0	13	0	13	0	15	-3.001e-06	15	4.003e-03	13	2.582e-05	19

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Envelope Joint Displacements (Continued)

	Joint		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation ...	LC	Y Rotation ...	LC	Z Rotation ...	LC
50		min	0	15	0	15	0	13	-2.986e-05	13	-3.129e-04	15	-2.494e-05	20
51	N27	max	0	13	.007	19	.026	13	-3.179e-06	15	4.604e-03	13	3.854e-06	19
52		min	0	15	-.006	20	0	15	-3.014e-05	13	6.495e-05	15	-1.103e-05	20
53	N28	max	0	20	0	20	0	15	2.309e-03	13	4.638e-03	13	2.578e-05	19
54		min	0	19	0	19	0	13	-1.377e-04	15	-3.481e-04	15	-2.506e-05	20
55	N29	max	0	20	0	20	0	15	2.385e-03	13	2.823e-03	13	9.726e-06	19
56		min	0	19	0	19	0	13	-8.087e-05	15	-2.082e-04	15	-9.116e-06	20
57	N30	max	0	20	.001	19	.021	15	2.385e-03	13	1.845e-03	13	2.365e-05	19
58		min	0	19	-.001	20	-.293	13	-8.093e-05	15	-7.633e-05	15	-2.307e-05	20
59	N33	max	0	19	.001	19	-.017	15	-7.781e-04	15	3.783e-01	13	1.943e-07	19
60		min	0	20	-.001	20	-.766	13	-5.487e-02	13	5.022e-03	15	-1.955e-07	20
61	N34	max	0	19	.001	19	-.021	15	-7.994e-04	15	3.796e-01	13	8.275e-07	19
62		min	0	20	-.001	20	-1.255	13	-5.572e-02	13	5.039e-03	15	-8.228e-07	20

Envelope AISC 14th(360-10): ASD Steel Code Checks

	Member	Shape	Code Check	Lo...	LC	Shear...	Lo...	Dir	LC	Pnc/o...	Pnt/o...	Mnyy/...	Mnzz/...	Cb	Eqn
1	M1	W12x65	.138	4.5	13	.080	4.5	y	13	558.6...	571.8...	106.9...	237.0...	1.7...	H1-1b
2	M2	W12x65	.208	10.5	13	.046	6.8...	y	13	503.44	571.8...	106.9...	237.0...	1.3...	H1-1b
3	M3	W12x65	.268	0	13	.127	1.5...	y	13	563.0...	571.8...	106.9...	237.0...	1.6...	H1-1b
4	M4	W12x65	.119	12...	13	.040	24...	y	13	289.9...	571.8...	106.9...	228.5...	1.1...	H1-1b
5	M5	W12x50	.016	0	13	.009	0	y	13	418.44	437.1...	53.144	179.3...	1.7...	H1-1b
6	M6	W12x50	.137	0	13	.070	0	y	13	418.44	437.1...	53.144	179.3...	1.6...	H1-1b
7	M7	W12x22	.895	10...	13	.043	21...	y	13	10.772	194.0...	9.132	16.221	1.1...	H1-1b
8	M8	W12x22	.896	10...	13	.043	21...	y	13	10.772	194.0...	9.132	16.221	1.1...	H1-1b
9	M9	W12x50	.121	23	13	.028	23	y	13	111.0...	437.1...	53.144	179.3...	2.88	H1-1b
10	M10	W12x50	.121	0	13	.038	0	y	13	374.8...	437.1...	53.144	179.3...	1.7...	H1-1b
11	M11	W12x50	.050	5.88	13	.026	0	y	13	266.9...	437.1...	53.144	176.7...	1.1...	H1-1b
12	M12	W12x96	.190	13...	13	.052	6.72	y	13	692.2...	844.3...	168.4...	366.7...	1.5...	H1-1b
13	M13	W12x96	.197	5.0...	13	.327	22...	y	13	475.0...	844.3...	168.4...	366.7...	1.1...	H1-1b
14	M14	W12x22	.007	2.1...	3	.007	4.25	y	3	146.0...	194.0...	9.132	73.104	1.1...	H1-1b
15	M15	W12x22	.092	5.35	13	.021	0	y	13	42.487	194.0...	9.132	39.885	1.1...	H1-1b
16	M16	W8x18	.392	14...	13	.006	0	y	13	42.254	157.4...	11.627	41.905	1.6...	H1-1a
17	M17	W8x18	.639	14...	13	.007	0	y	13	42.254	157.4...	11.627	41.905	1.6...	H1-1a
18	M18	W12x22	.010	2.4...	13	.010	0	y	13	135.8...	194.0...	9.132	73.104	1.1...	H1-1b
19	M19	W12x22	.067	4.8...	13	.020	9.75	y	13	51.168	194.0...	9.132	45.805	1.1...	H1-1b
20	M20	W12x22	.362	8.0...	13	.032	0	y	13	18.874	194.0...	9.132	22.887	1.1...	H1-1b
21	M21	W12x22	- KL/r > 200 for compression member -												
22	M22	W12x22	.894	10...	13	.043	0	y	13	10.772	194.0...	9.132	16.221	1.1...	H1-1b
23	M23	W12x22	- KL/r > 200 for compression member -												
24	M24	W12x22	.090	5.3...	13	.021	10...	y	13	43.087	194.0...	9.132	40.297	1.1...	H1-1b
25	M25	W12x22	.040	4.1...	13	.192	0	z	13	71.45	194.0...	9.132	55.224	1.1...	H1-1b
26	M26	W12x22	.008	2.07	13	.179	0	z	13	147.7...	194.0...	9.132	73.104	1.1...	H1-1b

Envelope Member Section Forces

	Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Momen...	LC	z-z Momen...	LC
1	M1	1	max	.77	20	.503	15	.025	26	0	1	0	1	0	1
2			min	-.824	19	-6.524	13	-.113	19	0	1	0	1	0	1
3		2	max	.77	20	.476	15	.025	26	0	1	.029	26	7.482	13
4			min	-.824	19	-6.776	13	-.113	19	0	1	-.127	19	-.551	15
5		3	max	.77	20	.449	15	.025	26	0	1	.057	26	15.247	13
6			min	-.824	19	-7.028	13	-.113	19	0	1	-.255	19	-1.071	15
7		4	max	.77	20	.422	15	.025	26	0	1	.086	26	23.296	13
8			min	-.824	19	-7.28	13	-.113	19	0	1	-.382	19	-1.561	15
9		5	max	.77	20	.395	15	.025	26	0	1	.114	26	31.628	13
10			min	-.824	19	-7.532	13	-.113	19	0	1	-.51	19	-2.02	15
11	M2	1	max	.612	19	1.695	13	.118	13	-.008	15	.2	20	28.466	13
12			min	-.863	20	-.165	15	-.011	26	-.54	13	-.412	19	-1.846	15
13		2	max	.612	19	1.107	13	.118	13	-.008	15	.188	20	24.787	13

Envelope Member Section Forces (Continued)

	Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Momen...	LC	z-z Momen...	LC
14			min	-.863	20	-.228	15	-.011	26	-.54	13	-.125	19	-1.33	15
15		3	max	.612	19	-.291	15	.034	25	-.008	15	.377	13	22.652	13
16			min	-.863	20	-2.21	13	-.171	20	-.54	13	-.008	15	-.649	15
17		4	max	.612	19	-.144	15	.034	25	-.008	15	.245	19	29.226	13
18			min	-.863	20	-2.798	13	-.171	20	-.54	13	-.275	20	-.354	15
19		5	max	.612	19	-.207	15	.034	25	-.008	15	.328	19	37.343	13
20			min	-.863	20	-3.386	13	-.171	20	-.54	13	-.724	20	.106	15
21	M3	1	max	2.271	19	10.985	13	.231	20	-.036	15	.216	19	41.199	13
22			min	-2.274	20	.054	15	-.059	19	-2.518	13	-.846	20	.07	15
23		2	max	2.271	19	10.78	13	.231	20	-.036	15	.162	19	31.214	13
24			min	-2.274	20	.032	15	-.059	19	-2.518	13	-.635	20	.031	15
25		3	max	2.271	19	10.574	13	.231	20	-.036	15	.108	19	21.418	13
26			min	-2.274	20	.01	15	-.059	19	-2.518	13	-.423	20	.012	15
27		4	max	2.271	19	10.369	13	.231	20	-.036	15	.054	19	11.811	13
28			min	-2.274	20	-.012	15	-.059	19	-2.518	13	-.212	20	.013	15
29		5	max	2.271	19	10.163	13	.231	20	-.036	15	0	1	2.392	13
30			min	-2.274	20	-.034	15	-.059	19	-2.518	13	0	1	.034	15
31	M4	1	max	1.618	20	3.795	13	.009	20	0	1	0	1	0	1
32			min	-1.73	19	-.293	15	-.009	19	0	1	0	1	0	1
33		2	max	1.618	20	2.239	13	.009	20	0	1	.055	20	1.41	15
34			min	-1.73	19	-.173	15	-.009	19	0	1	-.056	19	-18.286	13
35		3	max	1.618	20	-.053	15	.009	19	0	1	.11	20	2.092	15
36			min	-1.73	19	-.682	13	-.009	20	0	1	-.111	19	-27.137	13
37		4	max	1.585	20	.173	15	.009	19	0	1	.055	20	1.41	15
38			min	-1.697	19	-2.239	13	-.009	20	0	1	-.056	19	-18.286	13
39		5	max	1.585	20	.293	15	.009	19	0	1	0	1	0	1
40			min	-1.697	19	-3.795	13	-.009	20	0	1	0	1	0	1
41	M5	1	max	0	19	.804	13	0	20	0	1	.001	19	2.854	13
42			min	0	20	-.092	15	0	19	0	1	-.001	20	-.368	15
43		2	max	0	19	.759	13	0	20	0	1	.001	19	2.073	13
44			min	0	20	-.092	15	0	19	0	1	-.001	20	-.276	15
45		3	max	0	19	.714	13	0	20	0	1	0	19	1.337	13
46			min	0	20	-.092	15	0	19	0	1	0	20	-.184	15
47		4	max	0	19	.669	13	0	20	0	1	0	19	.646	13
48			min	0	20	-.092	15	0	19	0	1	0	20	-.092	15
49		5	max	0	19	.624	13	0	20	0	1	0	1	0	1
50			min	0	20	-.092	15	0	19	0	1	0	1	0	1
51	M6	1	max	.002	20	6.349	13	0	20	0	1	.001	19	24.557	13
52			min	-.002	19	.146	15	0	19	0	1	-.001	20	.296	15
53		2	max	.002	20	6.244	13	0	20	0	1	.001	19	18.26	13
54			min	-.002	19	.11	15	0	19	0	1	-.001	20	.168	15
55		3	max	.002	20	6.139	13	0	20	0	1	0	19	12.069	13
56			min	-.002	19	.074	15	0	19	0	1	0	20	.076	15
57		4	max	.002	20	6.034	13	0	20	0	1	0	19	5.982	13
58			min	-.002	19	.038	15	0	19	0	1	0	20	.02	15
59		5	max	.002	20	5.929	13	0	20	0	1	0	1	0	1
60			min	-.002	19	.002	15	0	19	0	1	0	1	0	1
61	M7	1	max	.041	15	2.729	13	0	1	0	1	0	1	0	1
62			min	-.622	13	-.21	15	0	1	0	1	0	1	0	1
63		2	max	.041	15	1.365	13	0	1	0	1	0	1	.838	15
64			min	-.622	13	-.105	15	0	1	0	1	0	1	-10.873	13
65		3	max	.041	15	0	1	0	1	0	1	0	1	1.118	15
66			min	-.622	13	0	1	0	1	0	1	0	1	-14.498	13
67		4	max	.041	15	.105	15	0	1	0	1	0	1	.838	15
68			min	-.622	13	-1.365	13	0	1	0	1	0	1	-10.873	13
69		5	max	.041	15	.21	15	0	1	0	1	0	1	0	1
70			min	-.622	13	-2.729	13	0	1	0	1	0	1	0	1
71	M8	1	max	-.007	15	2.729	13	0	1	0	1	0	1	0	1
72			min	-.946	13	-.21	15	0	1	0	1	0	1	0	1
73		2	max	-.007	15	1.365	13	0	1	0	1	0	1	.838	15
74			min	-.946	13	-.105	15	0	1	0	1	0	1	-10.873	13

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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	y-y Momen...	LC	z-z Momen...	LC
75		3	max	-.007	15	0	1	0	1	0	1	0	1	1.118	15
76			min	-.946	13	0	1	0	1	0	1	0	1	-14.498	13
77		4	max	-.007	15	.105	15	0	1	0	1	0	1	.838	15
78			min	-.946	13	-1.365	13	0	1	0	1	0	1	-10.873	13
79		5	max	-.007	15	.21	15	0	1	0	1	0	1	0	1
80			min	-.946	13	-2.729	13	0	1	0	1	0	1	0	1
81	M9	1	max	0	20	.624	13	0	20	0	1	0	1	0	1
82			min	0	19	-.092	15	0	19	0	1	0	1	0	1
83		2	max	0	20	-.054	16	0	20	0	1	.003	20	.43	15
84			min	0	19	-.186	3	0	19	0	1	-.003	19	-1.329	13
85		3	max	0	20	-.023	15	0	20	0	1	.007	20	2.137	3
86			min	0	19	-.946	13	0	19	0	1	-.007	19	.621	16
87		4	max	0	20	.012	15	0	20	0	1	.01	20	9.551	13
88			min	0	19	-1.731	13	0	19	0	1	-.01	19	.694	15
89		5	max	0	20	.046	15	0	20	0	1	.013	20	21.761	13
90			min	0	19	-2.516	13	0	19	0	1	-.013	19	.529	15
91	M10	1	max	0	20	3.413	13	.002	19	0	1	.013	20	21.761	13
92			min	0	19	.048	15	-.002	20	0	1	-.013	19	.529	15
93		2	max	0	20	3.157	13	.002	19	0	1	.01	20	15.601	13
94			min	0	19	.059	15	-.002	20	0	1	-.01	19	.428	15
95		3	max	0	20	2.902	13	.002	19	0	1	.007	20	9.921	13
96			min	0	19	.071	15	-.002	20	0	1	-.007	19	.307	15
97		4	max	0	20	2.646	13	.002	19	0	1	.003	20	4.72	13
98			min	0	19	.082	15	-.002	20	0	1	-.003	19	.164	15
99		5	max	0	20	2.39	13	.002	19	0	1	0	1	0	1
100			min	0	19	.093	15	-.002	20	0	1	0	1	0	1
101	M11	1	max	.002	19	2.39	13	0	19	0	1	0	1	0	1
102			min	-.002	20	.093	15	0	20	0	1	0	1	0	1
103		2	max	.002	19	1.637	13	0	19	0	1	.003	19	-.177	15
104			min	-.002	20	.012	15	0	20	0	1	-.003	20	-6.765	13
105		3	max	.002	19	.013	15	.002	20	.042	13	.001	20	-.272	15
106			min	-.002	20	-.175	13	-.002	19	0	15	-.001	19	-8.773	13
107		4	max	.001	19	-.026	15	.002	19	0	1	.005	20	-.224	15
108			min	-.001	20	-1.46	13	-.002	20	0	1	-.005	19	-6.168	13
109		5	max	.001	19	-.107	15	.002	19	0	1	0	1	0	1
110			min	-.001	20	-2.212	13	-.002	20	0	1	0	1	0	1
111	M12	1	max	3.302	19	5.746	13	.005	20	0	1	0	1	0	1
112			min	-3.305	20	.029	15	-.005	19	0	1	0	1	0	1
113		2	max	3.302	19	5.746	13	.005	20	0	1	.018	20	-.097	15
114			min	-3.305	20	.029	15	-.005	19	0	1	-.018	19	-19.306	13
115		3	max	3.302	19	5.746	13	.005	20	.141	13	.037	20	-.196	15
116			min	-3.305	20	.118	15	-.005	19	.002	15	-.037	19	-38.731	13
117		4	max	3.281	19	4.588	13	.014	19	.141	13	.01	19	-.593	15
118			min	-3.285	20	.118	15	-.014	20	.002	15	-.01	20	-54.147	13
119		5	max	3.281	19	4.588	13	.014	19	.141	13	.058	19	-.99	15
120			min	-3.285	20	.118	15	-.014	20	.002	15	-.058	20	-69.563	13
121	M13	1	max	3.135	19	1.124	13	.005	20	-.049	15	.058	19	-.988	15
122			min	-3.139	20	.108	15	-.004	19	-3.473	13	-.058	20	-69.476	13
123		2	max	3.135	19	-.029	15	.005	20	-.049	15	.035	19	-1.212	15
124			min	-3.139	20	-.162	3	-.004	19	-3.473	13	-.032	20	-72.242	13
125		3	max	3.185	19	-.061	15	.049	20	-.049	15	.167	20	-1.064	15
126			min	-3.183	20	-2.811	13	-.054	19	-3.473	13	-.181	19	-62.345	13
127		4	max	3.082	19	-.039	15	.052	19	-.049	15	.27	20	-.615	15
128			min	-3.09	20	-6.153	13	-.047	20	-3.473	13	-.295	19	-38.84	13
129		5	max	3.082	19	-.176	15	.052	19	-.049	15	0	1	0	1
130			min	-3.09	20	-7.434	13	-.047	20	-3.473	13	0	1	0	1
131	M14	1	max	0	1	.467	3	0	20	0	1	0	1	0	1
132			min	0	1	.14	16	0	19	0	1	0	1	0	1
133		2	max	0	1	.234	3	0	20	0	1	0	20	-.112	16
134			min	0	1	.07	16	0	19	0	1	0	19	-.373	3
135		3	max	0	1	0	1	0	20	0	1	.001	20	-.149	16

Envelope Member Section Forces (Continued)

	Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Momen...	LC	z-z Momen...	LC
136			min	0	1	0	1	0	19	0	1	-.001	19	-.497	3
137		4	max	0	1	-.07	15	0	20	0	1	.002	20	-.112	16
138			min	0	1	-.234	3	0	19	0	1	-.002	19	-.373	3
139		5	max	0	1	-.14	15	0	20	0	1	.002	20	0	1
140			min	0	1	-.468	3	0	19	0	1	-.002	19	0	1
141	M15	1	max	.07	19	1.374	13	0	19	0	1	.002	20	0	1
142			min	-.063	20	-.106	15	0	20	0	1	-.002	19	0	1
143		2	max	.07	19	.687	13	0	19	0	1	.002	20	.213	15
144			min	-.063	20	-.053	15	0	20	0	1	-.002	19	-2.757	13
145		3	max	.07	19	0	1	0	19	0	1	.001	20	.283	15
146			min	-.063	20	0	1	0	20	0	1	-.001	19	-3.676	13
147		4	max	.07	19	.053	15	0	19	0	1	0	20	.213	15
148			min	-.063	20	-.687	13	0	20	0	1	0	19	-2.757	13
149		5	max	.07	19	.106	15	0	19	0	1	0	1	0	1
150			min	-.063	20	-1.374	13	0	20	0	1	0	1	0	1
151	M16	1	max	11.986	13	.226	13	.039	13	0	1	0	1	0	1
152			min	-.772	15	-.012	15	0	15	0	1	0	1	0	1
153		2	max	11.986	13	.226	13	.039	13	0	1	.135	13	.044	15
154			min	-.772	15	-.012	15	0	15	0	1	.002	15	-.792	13
155		3	max	11.986	13	.226	13	.039	13	0	1	.27	13	.087	15
156			min	-.772	15	-.012	15	0	15	0	1	.004	15	-1.585	13
157		4	max	11.986	13	.226	13	.039	13	0	1	.405	13	.131	15
158			min	-.772	15	-.012	15	0	15	0	1	.006	15	-2.377	13
159		5	max	11.986	13	.226	13	.039	13	0	1	.54	13	.174	15
160			min	-.772	15	-.012	15	0	15	0	1	.008	15	-3.17	13
161	M17	1	max	17.148	13	.003	15	.141	13	0	1	0	1	0	1
162			min	.051	15	-.275	13	.002	15	0	1	0	1	0	1
163		2	max	17.148	13	.003	15	.141	13	0	1	.494	13	.966	13
164			min	.051	15	-.275	13	.002	15	0	1	.007	15	-.009	15
165		3	max	17.148	13	.003	15	.141	13	0	1	.989	13	1.932	13
166			min	.051	15	-.275	13	.002	15	0	1	.014	15	-.018	15
167		4	max	17.148	13	.003	15	.141	13	0	1	1.483	13	2.898	13
168			min	.051	15	-.275	13	.002	15	0	1	.021	15	-.027	15
169		5	max	17.148	13	.003	15	.141	13	0	1	1.978	13	3.864	13
170			min	.051	15	-.275	13	.002	15	0	1	.028	15	-.036	15
171	M18	1	max	.032	20	.626	13	0	1	0	1	0	1	0	1
172			min	-.032	19	-.048	15	0	1	0	1	0	1	0	1
173		2	max	.032	20	.313	13	0	1	0	1	0	1	.044	15
174			min	-.032	19	-.024	15	0	1	0	1	0	1	-.572	13
175		3	max	.032	20	0	1	0	1	0	1	0	1	.059	15
176			min	-.032	19	0	1	0	1	0	1	0	1	-.763	13
177		4	max	.032	20	.024	15	0	1	0	1	0	1	.044	15
178			min	-.032	19	-.313	13	0	1	0	1	0	1	-.572	13
179		5	max	.032	20	.048	15	0	1	0	1	0	1	0	1
180			min	-.032	19	-.626	13	0	1	0	1	0	1	0	1
181	M19	1	max	.207	20	1.252	13	0	1	0	1	0	1	0	1
182			min	-.208	19	-.097	15	0	1	0	1	0	1	0	1
183		2	max	.207	20	.626	13	0	1	0	1	0	1	.176	15
184			min	-.208	19	-.048	15	0	1	0	1	0	1	-2.289	13
185		3	max	.207	20	0	1	0	1	0	1	0	1	.235	15
186			min	-.208	19	0	1	0	1	0	1	0	1	-3.052	13
187		4	max	.207	20	.048	15	0	1	0	1	0	1	.176	15
188			min	-.208	19	-.626	13	0	1	0	1	0	1	-2.289	13
189		5	max	.207	20	.097	15	0	1	0	1	0	1	0	1
190			min	-.208	19	-1.252	13	0	1	0	1	0	1	0	1
191	M20	1	max	.135	20	2.062	13	0	1	0	1	0	1	0	1
192			min	-.146	19	-.159	15	0	1	0	1	0	1	0	1
193		2	max	.135	20	1.031	13	0	1	0	1	0	1	.478	15
194			min	-.146	19	-.079	15	0	1	0	1	0	1	-6.206	13
195		3	max	.135	20	0	1	0	1	0	1	0	1	.638	15
196			min	-.146	19	0	1	0	1	0	1	0	1	-8.274	13

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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	y-y Momen...	LC	z-z Momen...	LC
197		4	max	.135	20	.079	15	0	1	0	1	0	1	.478	15
198			min	-.146	19	-1.031	13	0	1	0	1	0	1	-6.206	13
199		5	max	.135	20	.159	15	0	1	0	1	0	1	0	1
200			min	-.146	19	-2.062	13	0	1	0	1	0	1	0	1
201	M21	1	max	1.862	20	2.729	13	0	1	0	1	0	1	0	1
202			min	-2.027	19	-.21	15	0	1	0	1	0	1	0	1
203		2	max	1.862	20	1.365	13	0	1	0	1	0	1	.838	15
204			min	-2.027	19	-.105	15	0	1	0	1	0	1	-10.873	13
205		3	max	1.862	20	0	1	0	1	0	1	0	1	1.118	15
206			min	-2.027	19	0	1	0	1	0	1	0	1	-14.498	13
207		4	max	1.862	20	.105	15	0	1	0	1	0	1	.838	15
208			min	-2.027	19	-1.365	13	0	1	0	1	0	1	-10.873	13
209		5	max	1.862	20	.21	15	0	1	0	1	0	1	0	1
210			min	-2.027	19	-2.729	13	0	1	0	1	0	1	0	1
211	M22	1	max	.005	15	2.729	13	0	1	0	1	0	1	0	1
212			min	-.273	13	-.21	15	0	1	0	1	0	1	0	1
213		2	max	.005	15	1.365	13	0	1	0	1	0	1	.838	15
214			min	-.273	13	-.105	15	0	1	0	1	0	1	-10.873	13
215		3	max	.005	15	0	1	0	1	0	1	0	1	1.118	15
216			min	-.273	13	0	1	0	1	0	1	0	1	-14.498	13
217		4	max	.005	15	.105	15	0	1	0	1	0	1	.838	15
218			min	-.273	13	-1.365	13	0	1	0	1	0	1	-10.873	13
219		5	max	.005	15	.21	15	0	1	0	1	0	1	0	1
220			min	-.273	13	-2.729	13	0	1	0	1	0	1	0	1
221	M23	1	max	1.369	19	2.729	13	0	1	0	1	0	1	0	1
222			min	-1.359	20	-.21	15	0	1	0	1	0	1	0	1
223		2	max	1.369	19	1.365	13	0	1	0	1	0	1	.838	15
224			min	-1.359	20	-.105	15	0	1	0	1	0	1	-10.873	13
225		3	max	1.369	19	0	1	0	1	0	1	0	1	1.118	15
226			min	-1.359	20	0	1	0	1	0	1	0	1	-14.498	13
227		4	max	1.369	19	.105	15	0	1	0	1	0	1	.838	15
228			min	-1.359	20	-1.365	13	0	1	0	1	0	1	-10.873	13
229		5	max	1.369	19	.21	15	0	1	0	1	0	1	0	1
230			min	-1.359	20	-2.729	13	0	1	0	1	0	1	0	1
231	M24	1	max	.038	19	1.365	13	0	1	0	1	0	1	0	1
232			min	-.038	20	-.105	15	0	1	0	1	0	1	0	1
233		2	max	.038	19	.682	13	0	1	0	1	0	1	.21	15
234			min	-.038	20	-.053	15	0	1	0	1	0	1	-2.718	13
235		3	max	.038	19	0	1	0	1	0	1	0	1	.279	15
236			min	-.038	20	0	1	0	1	0	1	0	1	-3.624	13
237		4	max	.038	19	.053	15	0	1	0	1	0	1	.21	15
238			min	-.038	20	-.682	13	0	1	0	1	0	1	-2.718	13
239		5	max	.038	19	.105	15	0	1	0	1	0	1	0	1
240			min	-.038	20	-1.365	13	0	1	0	1	0	1	0	1
241	M25	1	max	.003	19	1.06	13	0	1	.198	13	0	1	0	1
242			min	-.003	20	-.082	15	0	1	.003	15	0	1	0	1
243		2	max	.003	19	.53	13	0	1	.198	13	0	1	.126	15
244			min	-.003	20	-.041	15	0	1	.003	15	0	1	-1.639	13
245		3	max	.003	19	0	1	0	1	.198	13	0	1	.168	15
246			min	-.003	20	0	1	0	1	.003	15	0	1	-2.186	13
247		4	max	.003	19	.041	15	0	1	.198	13	0	1	.126	15
248			min	-.003	20	-.53	13	0	1	.003	15	0	1	-1.639	13
249		5	max	.003	19	.082	15	0	1	.198	13	0	1	0	1
250			min	-.003	20	-1.06	13	0	1	.003	15	0	1	0	1
251	M26	1	max	.004	20	.532	13	0	1	-.003	15	0	1	0	1
252			min	-.004	19	-.041	15	0	1	-.184	13	0	1	0	1
253		2	max	.004	20	.266	13	0	1	-.003	15	0	1	.032	15
254			min	-.004	19	-.02	15	0	1	-.184	13	0	1	-.413	13
255		3	max	.004	20	0	1	0	1	-.003	15	0	1	.042	15
256			min	-.004	19	0	1	0	1	-.184	13	0	1	-.55	13
257		4	max	.004	20	.02	15	0	1	-.003	15	0	1	.032	15

Envelope Member Section Forces (Continued)

	Member	Sec		Axial[k]	LC	y Shear[k]	LC	z Shear[k]	LC	Torque[k-ft]	LC	y-y Momen...	LC	z-z Momen...	LC
258			min	-.004	19	-.266	13	0	1	-.184	13	0	1	-.413	13
259		5	max	.004	20	.041	15	0	1	-.003	15	0	1	0	1
260			min	-.004	19	-.532	13	0	1	-.184	13	0	1	0	1

Envelope Member Section Deflections

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [r...	LC	(n) L/y Ra...	LC	(n) L/z Ra...	LC
1	M1	1	max	.007	19	.008	15	.002	20	1.982e-03	13	7754.288	15	NC	1
2			min	-.006	20	-.135	13	-.002	19	2.737e-05	15	455.218	13	NC	1
3		2	max	.007	19	.006	15	.002	20	1.982e-03	13	NC	15	NC	1
4			min	-.006	20	-.102	13	-.002	19	2.737e-05	15	630.905	13	NC	1
5		3	max	.007	19	.004	15	.001	20	1.982e-03	13	NC	15	NC	1
6			min	-.006	20	-.071	13	0	19	2.737e-05	15	1002.124	13	NC	1
7		4	max	.007	19	.002	15	.001	20	1.982e-03	13	NC	7	NC	1
8			min	-.006	20	-.042	13	0	25	2.737e-05	15	2170.364	13	NC	1
9		5	max	.007	19	.001	15	.001	13	1.982e-03	13	NC	1	NC	1
10			min	-.006	20	-.017	13	0	15	2.737e-05	15	NC	1	NC	1
11	M2	1	max	.007	19	.001	15	.001	13	1.982e-03	13	NC	1	NC	1
12			min	-.006	20	-.017	13	0	15	2.737e-05	15	NC	1	NC	1
13		2	max	.007	19	.017	13	0	13	3.293e-03	13	NC	7	NC	1
14			min	-.006	20	0	15	0	15	4.616e-05	15	3573.205	13	NC	1
15		3	max	.007	19	.026	13	0	13	4.604e-03	13	NC	7	NC	1
16			min	-.006	20	0	15	0	15	6.495e-05	15	2707.742	13	NC	1
17		4	max	.007	19	.015	13	.001	13	5.915e-03	13	NC	7	NC	1
18			min	-.006	20	0	15	0	15	8.373e-05	15	3369.024	13	NC	1
19		5	max	.006	19	0	15	.002	13	7.226e-03	13	NC	1	NC	1
20			min	-.006	20	-.024	13	0	15	1.025e-04	15	NC	1	NC	1
21	M3	1	max	.006	19	0	15	.002	13	7.226e-03	13	NC	1	NC	1
22			min	-.006	20	-.024	13	0	15	1.025e-04	15	NC	1	NC	1
23		2	max	.006	19	0	15	.002	19	7.538e-03	13	NC	7	NC	1
24			min	-.006	20	-.049	13	0	26	1.07e-04	15	1765.737	13	NC	1
25		3	max	.006	19	0	15	.002	19	7.85e-03	13	NC	15	NC	1
26			min	-.006	20	-.077	13	-.001	20	1.114e-04	15	822.307	13	NC	1
27		4	max	.006	19	0	15	.003	19	8.162e-03	13	NC	15	NC	1
28			min	-.006	20	-.109	13	-.002	20	1.158e-04	15	519.994	13	NC	1
29		5	max	.006	19	0	15	.003	19	8.474e-03	13	NC	15	NC	1
30			min	-.006	20	-.141	13	-.003	20	1.203e-04	15	375.7	13	NC	1
31	M4	1	max	0	1	0	1	0	1	2.979e-03	13	NC	1	NC	1
32			min	0	1	0	1	0	1	-1.077e-04	15	NC	1	NC	1
33		2	max	0	19	.014	15	0	20	2.979e-03	13	NC	7	NC	1
34			min	0	20	-.192	13	0	19	-1.077e-04	15	1843.759	13	NC	1
35		3	max	0	19	.021	15	.001	20	2.979e-03	13	NC	7	NC	1
36			min	0	20	-.293	13	-.001	19	-1.077e-04	15	1288.737	13	NC	1
37		4	max	0	19	.018	15	.003	20	2.979e-03	13	NC	7	NC	1
38			min	0	20	-.259	13	-.004	19	-1.077e-04	15	1843.759	13	NC	1
39		5	max	.001	19	.008	15	.007	20	2.979e-03	13	NC	1	NC	1
40			min	-.001	20	-.135	13	-.007	19	-1.077e-04	15	NC	1	NC	1
41	M5	1	max	0	1	0	1	0	1	0	1	NC	1	NC	1
42			min	0	1	0	1	0	1	0	1	NC	1	NC	1
43		2	max	0	1	0	15	0	19	0	1	NC	1	NC	1
44			min	0	1	0	13	0	20	0	1	NC	1	NC	1
45		3	max	0	20	0	15	0	19	0	1	NC	1	NC	1
46			min	0	19	-.001	13	0	20	0	1	NC	1	NC	1
47		4	max	0	20	0	15	0	19	0	1	NC	1	NC	1
48			min	0	19	-.002	13	0	20	0	1	NC	1	NC	1
49		5	max	0	20	0	15	0	19	0	1	NC	1	NC	1
50			min	0	19	-.004	13	0	20	0	1	NC	1	NC	1
51	M6	1	max	0	1	0	1	0	1	0	1	NC	1	NC	1
52			min	0	1	0	1	0	1	0	1	NC	1	NC	1
53		2	max	0	19	0	15	0	19	0	1	NC	1	NC	1

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Envelope Member Section Deflections (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [r...	LC	(n) L/y Ra...	LC	(n) L/z Ra...	LC
54			min	0	20	-.004	13	0	20	0	1	NC	1	NC	1
55		3	max	0	19	0	15	0	19	0	1	NC	7	NC	1
56			min	0	20	-.011	13	0	20	0	1	4372.997	13	NC	1
57		4	max	0	19	0	15	0	19	0	1	NC	7	NC	1
58			min	0	20	-.021	13	0	20	0	1	2340.849	13	NC	1
59		5	max	0	19	0	15	0	19	0	1	NC	15	NC	1
60			min	0	20	-.031	13	0	20	0	1	1539.672	13	NC	1
61	M7	1	max	0	1	0	1	0	1	0	1	NC	1	NC	1
62			min	0	1	0	1	0	1	0	1	NC	1	NC	1
63		2	max	0	13	.018	15	.002	20	0	1	NC	7	NC	1
64			min	0	15	-.236	13	-.002	19	0	1	1099.183	13	NC	1
65		3	max	0	13	.026	15	.003	20	0	1	NC	15	NC	1
66			min	0	15	-.334	13	-.003	19	0	1	783.168	13	NC	1
67		4	max	0	13	.019	15	.005	20	0	1	NC	7	NC	1
68			min	0	15	-.244	13	-.005	19	0	1	1099.183	13	NC	1
69		5	max	.001	13	.001	15	.006	20	0	1	NC	1	NC	1
70			min	0	15	-.017	13	-.007	19	0	1	NC	1	NC	1
71	M8	1	max	0	1	0	1	0	1	0	1	NC	1	NC	1
72			min	0	1	0	1	0	1	0	1	NC	1	NC	1
73		2	max	0	13	.018	15	.002	20	0	1	NC	7	NC	1
74			min	0	15	-.238	13	-.002	19	0	1	1099.183	13	NC	1
75		3	max	0	13	.025	15	.003	20	0	1	NC	15	NC	1
76			min	0	15	-.338	13	-.003	19	0	1	783.168	13	NC	1
77		4	max	.001	13	.018	15	.005	20	0	1	NC	7	NC	1
78			min	0	15	-.25	13	-.005	19	0	1	1099.183	13	NC	1
79		5	max	.002	13	0	15	.006	20	0	1	NC	1	NC	1
80			min	0	15	-.024	13	-.006	19	0	1	NC	1	NC	1
81	M9	1	max	0	19	0	15	0	19	0	1	NC	1	NC	1
82			min	0	20	-.004	13	0	20	0	1	NC	1	NC	1
83		2	max	0	19	.011	11	0	19	0	1	NC	1	NC	1
84			min	0	20	.001	16	0	20	0	1	NC	1	NC	1
85		3	max	0	19	.028	3	0	19	0	1	NC	1	NC	1
86			min	0	20	.008	15	0	20	0	1	6334.043	3	NC	1
87		4	max	0	19	.029	3	0	19	0	1	NC	1	NC	1
88			min	0	20	.006	15	0	20	0	1	5265.409	13	NC	1
89		5	max	0	19	0	15	0	20	0	1	NC	1	NC	1
90			min	0	20	-.031	13	0	19	0	1	NC	1	NC	1
91	M10	1	max	0	19	0	15	0	20	0	1	NC	1	NC	1
92			min	0	20	-.031	13	0	19	0	1	NC	1	NC	1
93		2	max	0	19	-.003	15	0	20	0	1	NC	15	NC	1
94			min	0	20	-.077	13	0	19	0	1	1984.12	13	NC	1
95		3	max	0	19	-.006	15	0	20	0	1	NC	15	NC	1
96			min	0	20	-.132	13	0	19	0	1	889.37	13	NC	1
97		4	max	0	19	-.01	15	0	20	0	1	9743.781	15	NC	1
98			min	0	20	-.195	13	0	19	0	1	549.783	13	NC	1
99		5	max	0	19	-.013	15	.001	20	0	1	7111.588	15	NC	1
100			min	0	20	-.261	13	-.001	19	0	1	392.336	13	NC	1
101	M11	1	max	.001	20	-.013	15	0	20	-5.082e-03	15	NC	1	NC	1
102			min	-.001	19	-.261	13	0	19	-3.822e-01	13	NC	1	NC	1
103		2	max	.001	20	-.016	15	0	20	-5.082e-03	15	NC	1	NC	1
104			min	-.001	19	-.645	13	0	19	-3.822e-01	13	6612.354	13	NC	1
105		3	max	.001	20	-.019	15	0	20	-5.092e-03	15	NC	1	NC	1
106			min	-.001	19	-1.015	13	0	19	-3.829e-01	13	4779.662	13	NC	1
107		4	max	.001	20	-.021	15	0	20	-5.102e-03	15	NC	1	NC	1
108			min	-.001	19	-1.365	13	0	19	-3.836e-01	13	6798.119	13	NC	1
109		5	max	.001	20	-.023	15	0	20	-5.102e-03	15	NC	1	NC	1
110			min	-.001	19	-1.702	13	0	19	-3.836e-01	13	NC	1	NC	1
111	M12	1	max	0	1	0	1	0	1	-2.179e-03	15	NC	1	NC	1
112			min	0	1	0	1	0	1	-1.557e-01	13	NC	1	NC	1
113		2	max	0	20	-.006	15	0	20	-2.179e-03	15	NC	7	NC	1
114			min	0	19	-.473	13	0	19	-1.557e-01	13	3372.221	13	NC	1

RISA
Company : HCP ENGINEERING
Designer :
Job Number :
Model Name :
Aug 15, 2017
11:22 AM
Checked By:_____

Envelope Member Section Deflections (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [r...	LC	(n) L/y Ra...	LC	(n) L/z Ra...	LC
115		3	max	0	20	-.012	15	0	20	-2.179e-03	15	NC	7	NC	1
116			min	0	19	-.927	13	0	19	-1.557e-01	13	2117.54	13	NC	1
117		4	max	0	20	-.018	15	0	20	-2.192e-03	15	NC	7	NC	1
118			min	0	19	-1.342	13	0	19	-1.566e-01	13	2465.65	13	NC	1
119		5	max	0	20	-.023	15	.001	20	-2.204e-03	15	NC	1	NC	1
120			min	0	19	-1.702	13	-.001	19	-1.575e-01	13	NC	1	NC	1
121	M13	1	max	0	20	-.023	15	.001	20	-2.207e-03	15	NC	1	NC	1
122			min	0	19	-1.702	13	-.001	19	-1.577e-01	13	NC	1	NC	1
123		2	max	.001	20	-.022	15	.001	20	-1.675e-03	15	NC	15	NC	1
124			min	-.001	19	-1.589	13	-.001	19	-1.203e-01	13	990.084	13	NC	1
125		3	max	.002	20	-.017	15	.001	20	-1.142e-03	15	NC	15	NC	1
126			min	-.002	19	-1.266	13	-.001	19	-8.288e-02	13	797.709	13	NC	1
127		4	max	.002	20	-.009	15	.003	20	-6.091e-04	15	NC	15	NC	1
128			min	-.002	19	-.76	13	-.003	19	-4.546e-02	13	1201.605	13	NC	1
129		5	max	.002	20	0	15	.006	20	-7.639e-05	15	NC	1	NC	1
130			min	-.002	19	-.141	13	-.007	19	-8.04e-03	13	NC	1	NC	1
131	M14	1	max	0	1	0	1	0	1	5.946e-01	13	NC	1	NC	1
132			min	0	1	0	1	0	1	6.208e-03	15	NC	1	NC	1
133		2	max	0	1	0	16	0	19	5.946e-01	13	NC	1	NC	1
134			min	0	1	0	3	0	20	6.208e-03	15	NC	1	NC	1
135		3	max	0	1	0	16	0	19	5.946e-01	13	NC	1	NC	1
136			min	0	1	0	3	0	20	6.208e-03	15	NC	1	NC	1
137		4	max	0	1	0	16	0	19	5.946e-01	13	NC	1	NC	1
138			min	0	1	0	3	0	20	6.208e-03	15	NC	1	NC	1
139		5	max	0	1	0	1	0	1	5.946e-01	13	NC	1	NC	1
140			min	0	1	0	1	0	1	6.208e-03	15	NC	1	NC	1
141	M15	1	max	0	1	0	1	0	1	5.945e-01	13	NC	1	NC	1
142			min	0	1	0	1	0	1	6.207e-03	15	NC	1	NC	1
143		2	max	0	20	-.004	15	0	20	5.945e-01	13	NC	1	NC	1
144			min	0	19	-.392	13	0	19	6.207e-03	15	8610.354	13	NC	1
145		3	max	0	20	-.009	15	0	20	5.945e-01	13	NC	1	NC	1
146			min	0	19	-.775	13	0	19	6.207e-03	15	6134.877	13	NC	1
147		4	max	0	20	-.014	15	.001	20	5.945e-01	13	NC	1	NC	1
148			min	0	19	-1.146	13	-.001	19	6.207e-03	15	8610.354	13	NC	1
149		5	max	0	20	-.021	15	.002	20	5.945e-01	13	NC	1	NC	1
150			min	0	19	-1.508	13	-.002	19	6.207e-03	15	6209.169	15	NC	1
151	M16	1	max	0	1	0	1	0	1	1.058e-04	13	NC	1	NC	1
152			min	0	1	0	1	0	1	-5.886e-06	15	NC	1	NC	1
153		2	max	0	15	.002	15	0	15	1.058e-04	13	NC	1	NC	7
154			min	-.004	13	-.029	13	-.038	13	-5.886e-06	15	5740.581	13	4335.448	13
155		3	max	0	15	.003	15	0	15	1.058e-04	13	NC	7	NC	7
156			min	-.008	13	-.047	13	-.061	13	-5.886e-06	15	3587.863	13	2709.655	13
157		4	max	0	15	.002	15	0	15	1.058e-04	13	NC	7	NC	7
158			min	-.012	13	-.041	13	-.053	13	-5.886e-06	15	4100.415	13	3096.748	13
159		5	max	.001	15	.006	20	.002	13	1.058e-04	13	NC	1	NC	1
160			min	-.017	13	-.007	19	0	15	-5.886e-06	15	NC	1	NC	1
161	M17	1	max	0	1	0	1	0	1	1.131e-06	15	NC	1	NC	1
162			min	0	1	0	1	0	1	-1.245e-04	13	NC	1	NC	1
163		2	max	0	15	.036	13	-.002	15	1.131e-06	15	NC	1	NC	15
164			min	-.006	13	0	15	-.141	13	-1.245e-04	13	4708.491	13	1184.581	13
165		3	max	0	15	.057	13	-.003	15	1.131e-06	15	NC	7	NC	15
166			min	-.012	13	0	15	-.226	13	-1.245e-04	13	2942.807	13	740.363	13
167		4	max	0	15	.05	13	-.003	15	1.131e-06	15	NC	7	NC	15
168			min	-.018	13	0	15	-.197	13	-1.245e-04	13	3363.208	13	846.13	13
169		5	max	0	15	.006	20	.003	13	1.131e-06	15	NC	1	NC	1
170			min	-.024	13	-.006	19	0	15	-1.245e-04	13	NC	1	NC	1
171	M18	1	max	0	1	0	1	0	1	-1.481e-03	15	NC	1	NC	1
172			min	0	1	0	1	0	1	-1.057e-01	13	NC	1	NC	1
173		2	max	0	19	-.003	15	0	20	-1.481e-03	15	NC	1	NC	1
174			min	0	20	-.232	13	0	19	-1.057e-01	13	NC	1	NC	1
175		3	max	0	19	-.006	15	0	20	-1.481e-03	15	NC	1	NC	1

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Envelope Member Section Deflections (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [r...	LC	(n) L/y Ra...	LC	(n) L/z Ra...	LC
176			min	0	20	-.464	13	0	19	-1.057e-01	13	9711.67	15	NC	1
177		4	max	0	19	-.009	15	0	20	-1.481e-03	15	NC	1	NC	1
178			min	0	20	-.696	13	0	19	-1.057e-01	13	6435.451	15	NC	1
179		5	max	0	19	-.012	15	0	20	-1.481e-03	15	NC	1	NC	1
180			min	0	20	-.927	13	0	19	-1.057e-01	13	4800.43	15	NC	1
181	M19	1	max	0	1	0	1	0	1	0	1	NC	1	NC	1
182			min	0	1	0	1	0	1	0	1	NC	1	NC	1
183		2	max	0	19	-.005	15	0	20	0	1	NC	1	NC	1
184			min	0	20	-.436	13	0	19	0	1	NC	1	NC	1
185		3	max	0	19	-.011	15	0	20	0	1	NC	1	NC	1
186			min	0	20	-.865	13	0	19	0	1	8108.078	13	NC	1
187		4	max	0	19	-.017	15	.001	20	0	1	NC	1	NC	1
188			min	0	20	-1.287	13	-.001	19	0	1	7016.03	15	NC	1
189		5	max	0	19	-.023	15	.001	20	0	1	NC	1	NC	1
190			min	0	20	-1.702	13	-.001	19	0	1	5023.279	15	NC	1
191	M20	1	max	0	1	0	1	0	1	-6.3e-04	15	NC	1	NC	1
192			min	0	1	0	1	0	1	-4.56e-02	13	NC	1	NC	1
193		2	max	0	19	.003	15	0	20	-6.3e-04	15	NC	7	NC	1
194			min	0	20	-.31	13	0	19	-4.56e-02	13	2549.347	13	NC	1
195		3	max	0	19	.002	15	.001	20	-6.3e-04	15	NC	7	NC	1
196			min	0	20	-.575	13	-.001	19	-4.56e-02	13	1816.41	13	NC	1
197		4	max	0	19	-.003	15	.002	20	-6.3e-04	15	NC	7	NC	1
198			min	0	20	-.78	13	-.002	19	-4.56e-02	13	2549.347	13	NC	1
199		5	max	0	19	-.012	15	.003	20	-6.3e-04	15	NC	1	NC	1
200			min	0	20	-.939	13	-.003	19	-4.56e-02	13	NC	1	NC	1
201	M21	1	max	0	1	0	1	0	1	1.577e-05	15	NC	1	NC	1
202			min	0	1	0	1	0	1	-2.749e-03	13	NC	1	NC	1
203		2	max	0	19	.018	15	.002	20	1.577e-05	15	NC	7	NC	1
204			min	0	20	-.267	13	-.002	19	-2.749e-03	13	1099.183	13	NC	1
205		3	max	.002	19	.025	15	.003	20	1.577e-05	15	NC	15	NC	1
206			min	-.002	20	-.396	13	-.003	19	-2.749e-03	13	783.168	13	NC	1
207		4	max	.003	19	.018	15	.005	20	1.577e-05	15	NC	7	NC	1
208			min	-.002	20	-.338	13	-.005	19	-2.749e-03	13	1099.183	13	NC	1
209		5	max	.003	19	0	15	.006	20	1.577e-05	15	NC	1	NC	1
210			min	-.003	20	-.141	13	-.006	19	-2.749e-03	13	NC	1	NC	1
211	M22	1	max	0	1	0	1	0	1	-3.148e-06	15	NC	1	NC	1
212			min	0	1	0	1	0	1	-2.798e-05	13	NC	1	NC	1
213		2	max	0	13	.018	15	.002	20	-3.148e-06	15	NC	7	NC	1
214			min	0	15	-.225	13	-.002	19	-2.798e-05	13	1099.183	13	NC	1
215		3	max	0	13	.025	15	.003	20	-3.148e-06	15	NC	15	NC	1
216			min	0	15	-.312	13	-.003	19	-2.798e-05	13	783.168	13	NC	1
217		4	max	0	13	.017	15	.005	20	-3.148e-06	15	NC	7	NC	1
218			min	0	15	-.212	13	-.005	19	-2.798e-05	13	1099.183	13	NC	1
219		5	max	0	13	.026	13	.006	20	-3.148e-06	15	NC	1	NC	1
220			min	0	15	0	15	-.007	19	-2.798e-05	13	NC	1	NC	1
221	M23	1	max	0	1	0	1	0	1	2.311e-03	13	NC	1	NC	1
222			min	0	1	0	1	0	1	-1.379e-04	15	NC	1	NC	1
223		2	max	0	20	.02	15	.002	20	2.311e-03	13	NC	7	NC	1
224			min	0	19	-.266	13	-.002	19	-1.379e-04	15	1099.183	13	NC	1
225		3	max	.001	20	.029	15	.003	20	2.311e-03	13	NC	15	NC	1
226			min	-.001	19	-.393	13	-.003	19	-1.379e-04	15	783.168	13	NC	1
227		4	max	.002	20	.024	15	.005	20	2.311e-03	13	NC	7	NC	1
228			min	-.002	19	-.333	13	-.005	19	-1.379e-04	15	1099.183	13	NC	1
229		5	max	.002	20	.008	15	.006	20	2.311e-03	13	NC	1	NC	1
230			min	-.002	19	-.135	13	-.007	19	-1.379e-04	15	NC	1	NC	1
231	M24	1	max	0	1	0	1	0	1	2.386e-03	13	NC	1	NC	1
232			min	0	1	0	1	0	1	-8.097e-05	15	NC	1	NC	1
233		2	max	0	20	.006	15	0	20	2.386e-03	13	NC	1	NC	1
234			min	0	19	-.088	13	0	19	-8.097e-05	15	8793.462	13	NC	1
235		3	max	0	20	.012	15	0	20	2.386e-03	13	NC	1	NC	1
236			min	0	19	-.167	13	0	19	-8.097e-05	15	6265.342	13	NC	1

Envelope Member Section Deflections (Continued)

	Member	Sec		x [in]	LC	y [in]	LC	z [in]	LC	x Rotate [r...	LC	(n) L/y Ra...	LC	(n) L/z Ra...	LC
237		4	max	0	20	.017	15	0	20	2.386e-03	13	NC	1	NC	1
238			min	0	19	-.235	13	0	19	-8.097e-05	15	8793.462	13	NC	1
239		5	max	0	20	.021	15	.001	20	2.386e-03	13	NC	1	NC	1
240			min	0	19	-.293	13	-.001	19	-8.097e-05	15	NC	1	NC	1
241	M25	1	max	0	1	0	1	0	1	0	1	NC	1	NC	1
242			min	0	1	0	1	0	1	0	1	NC	1	NC	1
243		2	max	0	20	-.004	15	0	20	-2.51e-04	15	NC	1	NC	1
244			min	0	19	-.197	13	0	19	-1.797e-02	13	NC	1	NC	1
245		3	max	0	20	-.008	15	0	20	-5.02e-04	15	NC	1	NC	1
246			min	0	19	-.39	13	0	19	-3.594e-02	13	NC	1	NC	1
247		4	max	0	20	-.012	15	0	20	-7.53e-04	15	NC	1	NC	1
248			min	0	19	-.58	13	0	19	-5.392e-02	13	7921.33	15	NC	1
249		5	max	0	20	-.017	15	.001	20	-1.004e-03	15	NC	1	NC	1
250			min	0	19	-.766	13	-.001	19	-7.189e-02	13	5753.884	15	NC	1
251	M26	1	max	0	20	-.012	15	0	20	-1.575e-03	15	NC	1	NC	1
252			min	0	19	-.927	13	0	19	-1.124e-01	13	NC	1	NC	1
253		2	max	0	20	-.014	15	0	20	-1.457e-03	15	NC	1	NC	1
254			min	0	19	-1.01	13	0	19	-1.04e-01	13	NC	1	NC	1
255		3	max	0	20	-.016	15	0	20	-1.34e-03	15	NC	1	NC	1
256			min	0	19	-1.092	13	0	19	-9.558e-02	13	NC	1	NC	1
257		4	max	0	20	-.019	15	.001	20	-1.223e-03	15	NC	1	NC	1
258			min	0	19	-1.174	13	-.001	19	-8.717e-02	13	7732.621	15	NC	1
259		5	max	0	20	-.021	15	.001	20	-1.105e-03	15	NC	1	NC	1
260			min	0	19	-1.255	13	-.001	19	-7.877e-02	13	5776.298	15	NC	1

Material Takeoff

	Material	Size	Pieces	Length[ft]	Weight[K]
1	Hot Rolled Steel				
2	A992- SHapes: W's	W12x22	13	174.9	3.9
3	A992- SHapes: W's	W12x50	5	51.9	2.6
4	A992- SHapes: W's	W12x65	4	42.9	2.8
5	A992- SHapes: W's	W12x96	2	36.3	3.5
6	A992- SHapes: W's	W8x18	2	28.1	.5
7	Total HR Steel		26	334.1	13.2

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CONNECTION AT PODIUM

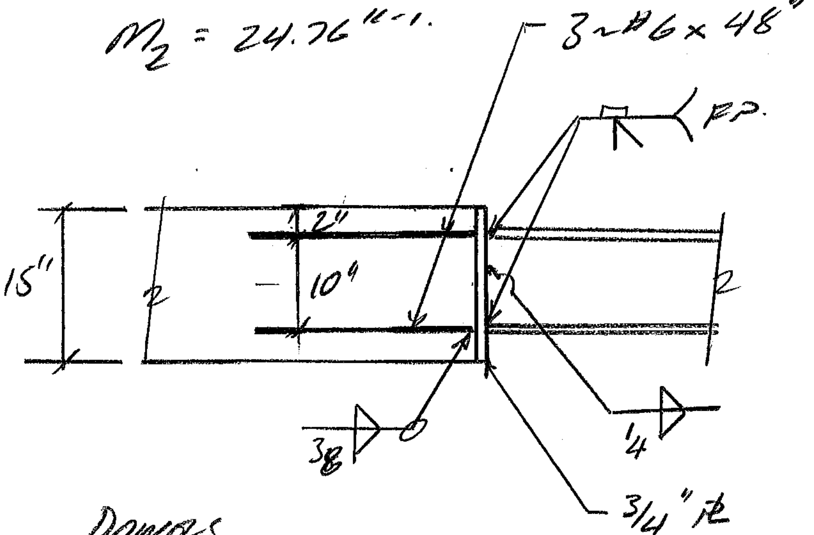
JOINT N5 BEAM'S - MOMENT CONN

$P_x = 2.405^k$

$P_y = 2.273^k$

$P_z = 13.1^k$

$M_z = 24.76^{k\text{-}'}$



DOWELS

$$T = C = \frac{24.76(12)}{10} = 29.5^k$$

3~#6 x 48"(min) DWL
T & B

$T_n = 3(0.44)(60) = 79^k > 29.5^k$

WELD

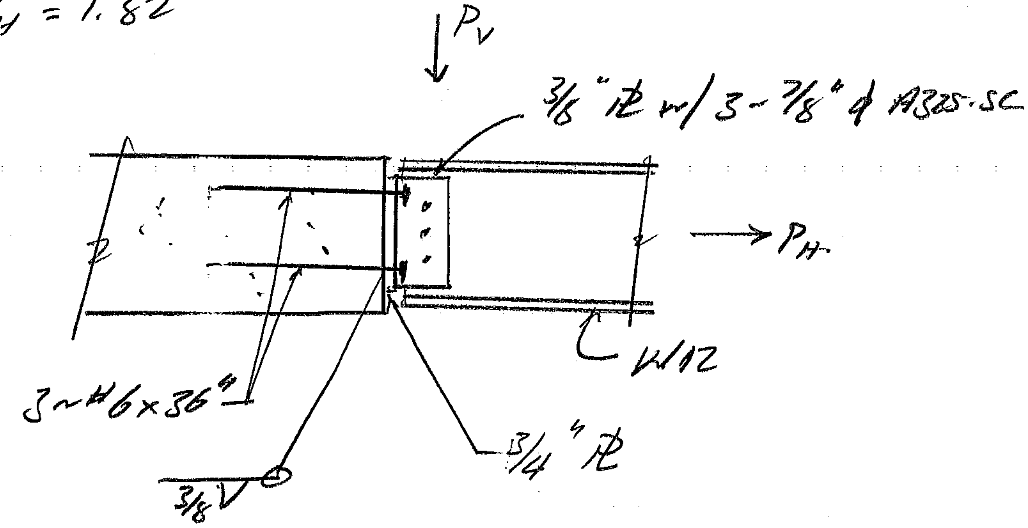
3/8

$T_n = 5.4(2.35) \times 3 = 38.1^k$
$> 29.5^k$

$V = 13.1^k < \frac{V_n}{\Omega} = 53.9^k$

SIMPLE CONN

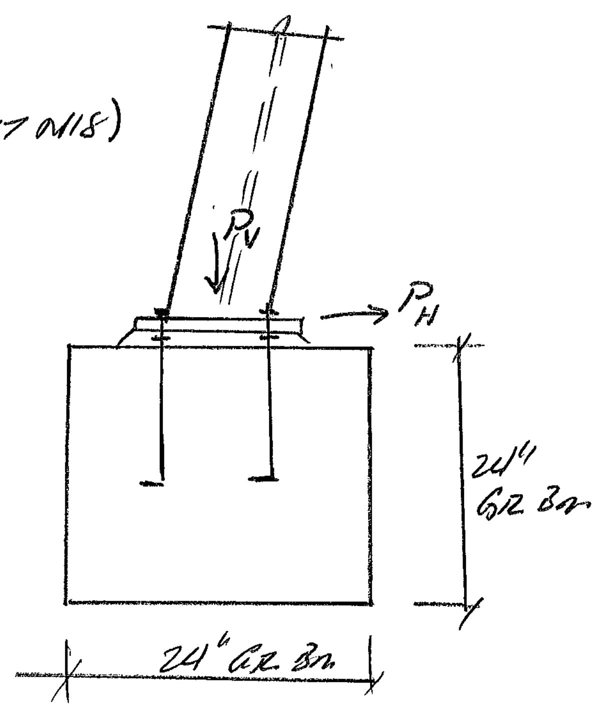
$P_V = 2.8^k$ (JOINT N25).
$P_H = 1.82^k$



$V_A = 2.2(4) = 8.8^k > 2.8^k$

FOUNDATION

$P_V = 17.1^k$ (JOINT N18)
$P_{H_X} : 1.29^k$
$P_{H_Y} : 0.275^k$



B.P 12" x 14"

$t_{PL} = 0.91\sqrt{\dfrac{17.1}{12(14)}}$

$= 0.29"$

Use 3/4" THK B.P.
w/ 4 ~ 3/4" ϕ A.B