MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS

STRUCTURAL CALCULATIONS
CHESTNUT SQUARE FAMILY BUILDING
(VOL.2 SUPERSTRUCTURE)

At

1665 CHESTNUT STREET

LIVERMORE, CA

11/07/18

MBC PROJECT NO. 216-008

[redacted]

85 SECOND STREET • SUITE 501 • SAN FRANCISCO, CA 94105 • TEL: 415.546.0431 • FAX: 415.882.7257

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS

STRUCTURAL CALCULATIONS

SUBJECT: CHESTNUT SQUARE SENIOR HOUSING

JOB NO: 216-009

BY: JYL/ANSON CHENG

DATE: NOVEMBER 6, 2018

The following calculations are for Permit submittal of building at the above subject project address. The building consists of 4 story wood structure over concrete podium/basement structure.

Contents

VOL: II

The structural calculations have been carried out in accordance with the California Building Code, 2016 Edition.

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SUPERSTRUCTURE VERTICAL

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MURPHY BURR CURRY, INC
Consulting Structural Engineers

Project:Chestnut SQ F.H.
Project #: 216-008
By: JYL

Chestnut Square Family Housing

Building Weight:

Main Roof Dead Load

Solar Panel	5	
Roofing	6	
5/8" T&G. Plywood	1.8	
Roof Truss/Framing	3	
Insulation	0.5	
5/8" GYP	6	
Mis	2.7	
Roof Dead Load =	**25**	psf
Added Wall Wt + Partition Wt for Seismic =	*10*	psf
Total Floor Dead Load for Seismic Design =	**35**	psf
Roof Live Load =	**20**	psf

TYP Floor Dead Load

Finish	3	
1.5" Gypcrete	13.8	
1/4" Acousti Mat	0.5	
3/4" T&G Plywood	2.5	
TJI Joist	3	
Insulation	0.5	
Furring Channel	0.5	
2-Layers of 1/2" gyp bd	4.4	
Misc.	1.8	
Floor Dead Load =	*30*	psf
Partition =	*10*	psf
Total Floor Dead Load =	**40**	psf
Added Load for Wt of Wall for Seismic =	*7.5*	psf
Total Floor Dead Load for Seismic Design =	**47.5**	psf
Floor Live Load =	**40**	psf

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Consulting Structural Engineers

Project:F
Project #:
By: JYL

TJI Floor Joist Calculation:

210 TJI-11 7/8" @16" O.C.

W_dead=	53.2	plf
W_live=	53.2	plf
Tot=	**106.4**	plf
R=	851.2	
Simpson ITS		
Vcap=	1070	lbs

Span Max: **16 ft**

Wallow=	**106**	plf	[L/240]
Wallow=	74	pld (L.L)	[L/480]

TJI Floor Joist Calculation:

210 TJI-11 7/8" @16" O.C.

W_dead=	39.9	plf
W_live=	66.5	plf
Tot=	**106.4**	plf
R=	744.8	
Simpson ITS		
Vcap=	1070	lbs

Span Max: **14 ft**

Wallow=	**121**	plf	[L/240]

A3

MURPHY BURR CURRY, INC
Consulting Structural Engineers

Project:I
Project #:
By: JYL

Corridor Joist Calculation:

2X10@16" OC (DF#2)

W_dead=	53.2	plf
W_live=	133	plf
Tot=	**186.2**	plf
R=	558.6	lbs
Simp. LU28		
Vcap=	792	lbs

Span Max:	**6 ft**	Typ
Span	**11.5**	Corridor turn
Span	**9**	Stair 2

MURPHY BURR CURRY, INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:
Project ID:

A4

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Wood Beam

Lic. #: KW-06002966 — Licensee : MURPHY BURR CURRY, INC

Description : Corridor Joist-11.5ft span

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

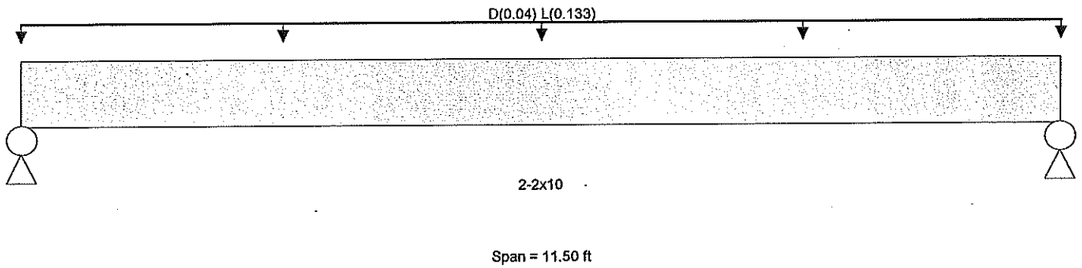
Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	900.0 psi
Fb - Compr	900.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	575.0 psi

E : Modulus of Elasticity

Ebend- xx	1,600.0 ksi
Eminbend - xx	580.0 ksi
Density	32.210 pcf

Repetitive Member Stress Increase



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Uniform Load : D = 0.040, L = 0.1330 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.705 : 1	Maximum Shear Stress Ratio	=	0.259 : 1
Section used for this span		2-2x10	Section used for this span		2-2x10
fb : Actual	=	802.19 psi	fv : Actual	=	46.71 psi
FB : Allowable	=	1,138.50 psi	Fv : Allowable	=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	5.750 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.166 in	Ratio =	829
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.216 in	Ratio =	637
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+L+H					1.10	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.50 ft	1	0.705	0.259	1.00	1.10	1.00	1.15	1.00	1.00	1.00	2.86	802.19	1138.50	0.86	46.71	180.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.2163	5.792		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.995	0.995
D Only	0.230	0.230
L Only	0.765	0.765

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Project Title:
Engineer:
Project Descr:

Project ID: A5

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Wood Beam

Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Description : Corridor Joist-11.5ft span

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
D+L	0.995	0.995

Wood Beam

Description : Corridor Joist-9ft span

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

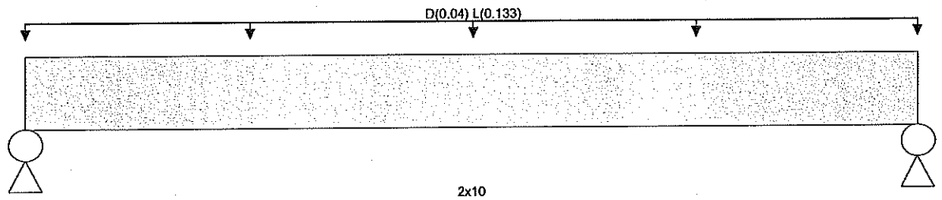
Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	900.0 psi
Fb - Compr	900.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	575.0 psi

E : Modulus of Elasticity

Ebend- xx	1,600.0 ksi
Eminbend - xx	580.0 ksi
Density	32.210 pcf

Repetitive Member Stress Increase



Span = 9.0 ft

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Uniform Load : D = 0.040, L = 0.1330 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.863 : 1	Maximum Shear Stress Ratio	=	0.389 : 1
Section used for this span		2x10	Section used for this span		2x10
fb : Actual	=	982.65 psi	fv : Actual	=	70.03 psi
FB : Allowable	=	1,138.50 psi	Fv : Allowable	=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	4.500 ft	Location of maximum on span	=	8.245 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.125 in	Ratio =	865
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.162 in	Ratio =	665
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+L+H					1.10	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.863	0.389	1.00	1.10	1.00	1.15	1.00	1.00	1.00	1.75	982.65	1138.50	0.65	70.03	180.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.1623	4.533		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.779	0.779
D Only	0.180	0.180
L Only	0.599	0.599

Wood Beam

Lic. # : KW-06002966

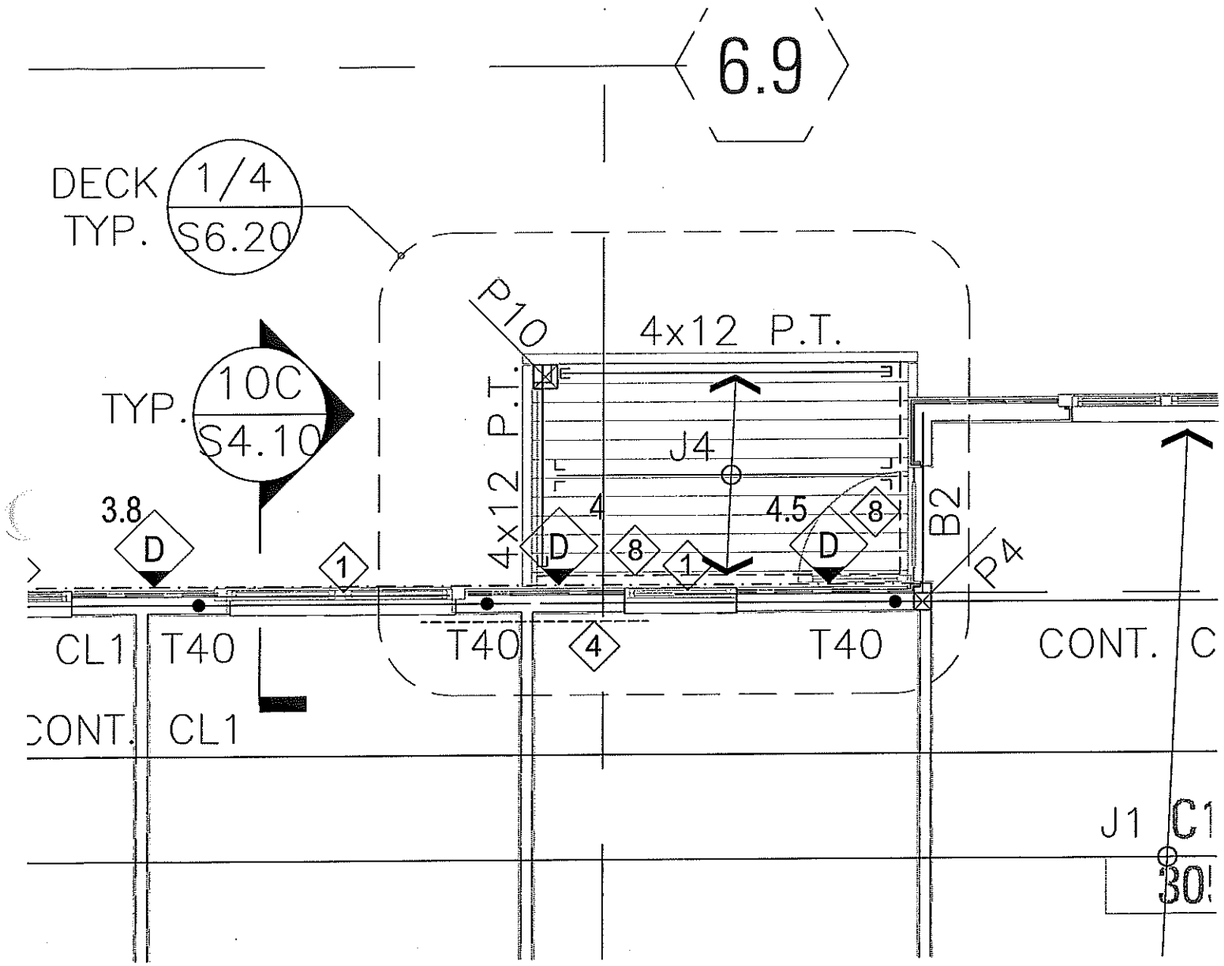
Description : Corridor Joist-9ft span

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
D+L	0.779	0.779

A8


6.9
DECK
TYP.
1/4
S6.20
TYP.
10C
S4.10
3.8
D
1
P10
4x12 P.T.
2 P.T.
4x12 P.T.
J4
4
D
8
1
4.5
D
8
B2
P4
CL1
T40
T40
4
T40
CONT.
C
CONT.
CL1
J1
C1
30

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Project Title:
Engineer:
Project Descr:

Project ID: A9

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY BURR CURRY, INC.

Wood Beam

Lic. # : KW-06002966

Description : Deck Joist-2X10 @ 16" o.c. (10.5ft span)

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-10

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

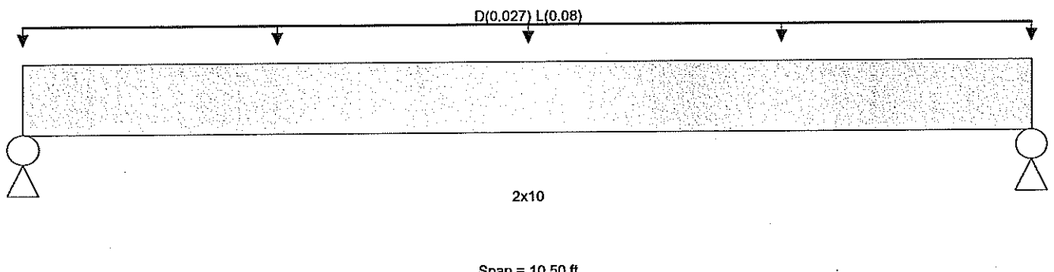
Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	900.0 psi
Fb - Compr	900.0 psi
Fc - Prll	1,350.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	575.0 psi

E : Modulus of Elasticity

Ebend- xx	1,600.0 ksi
Eminbend - xx	580.0 ksi
Density	32.210 pcf

Repetitive Member Stress Increase



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Uniform Load : D = 0.0270, L = 0.080 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.908 : 1	Maximum Shear Stress Ratio	=	0.360 : 1
Section used for this span		2x10	Section used for this span		2x10
fb : Actual	=	827.24 psi	fv : Actual	=	51.86 psi
FB : Allowable	=	910.80 psi	Fv : Allowable	=	144.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	5.250 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.139 in	Ratio =	906
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.186 in	Ratio =	677
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+L+H					1.10	1.00	1.15	0.80	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.908	0.360	1.00	1.10	1.00	1.15	0.80	1.00	1.00	1.47	827.24	910.80	0.48	51.86	144.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.1859	5.288		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.562	0.562
D Only	0.142	0.142
L Only	0.420	0.420

MURPHY BURR CURRY, INC.
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Project Title:
Engineer:
Project Descr:

Project ID: A10

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Wood Beam

Lic. #: KW-06002966 Licensee : MURPHY BURR CURRY, INC.

Description : Deck Joist-2X10 @ 16" o.c. (10.5ft span)

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
D+L	0.562	0.562

Wood Beam

Lic. # : KW-06002966

Description : 4X12 Deck Beam

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-10

Wood Species : Douglas Fir - Larch
Wood Grade : No.1

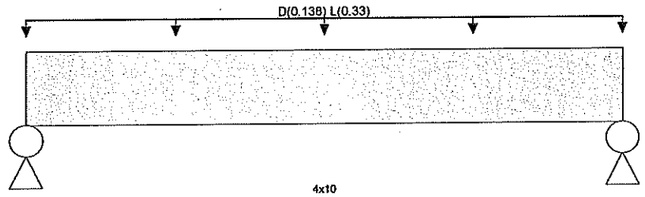
Fb - Tension	1,000.0 psi
Fb - Compr	1,000.0 psi
Fc - Prll	1,500.0 psi
Fc - Perp	625.0 psi
Fv	180.0 psi
Ft	675.0 psi

E : Modulus of Elasticity

Ebend- xx	1,700.0 ksi
Eminbend - xx	620.0 ksi
Density	32.210 pcf

Repetitive Member Stress Increase

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling



Span = 6.0 ft

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Uniform Load : D = 0.1380, L = 0.330 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.459 : 1	Maximum Shear Stress Ratio	=	0.336 : 1
Section used for this span		4x10	Section used for this span		4x10
fb : Actual	=	506.34 psi	fv : Actual	=	48.43 psi
FB : Allowable	=	1,104.00 psi	Fv : Allowable	=	144.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	3.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.025 in	Ratio =	2919
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.035 in	Ratio =	2058
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+L+H					1.20	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.459	0.336	0.80	1.20	1.00	1.15	1.00	1.00	1.00	2.11	506.34	1104.00	1.05	48.43	144.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.0350	3.022		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.404	1.404
D Only	0.414	0.414
L Only	0.990	0.990

MURPHY BURR CURRY INC.

SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A12

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY, BURR, CURRY, INC.

Wood Beam

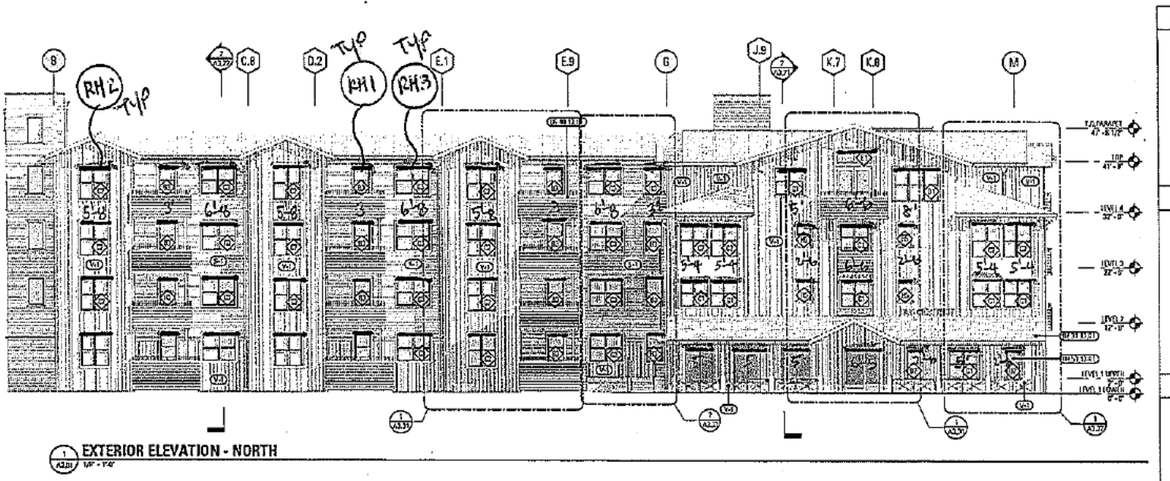
Lic. # : KW-06002966

Description : 4X12 Deck Beam

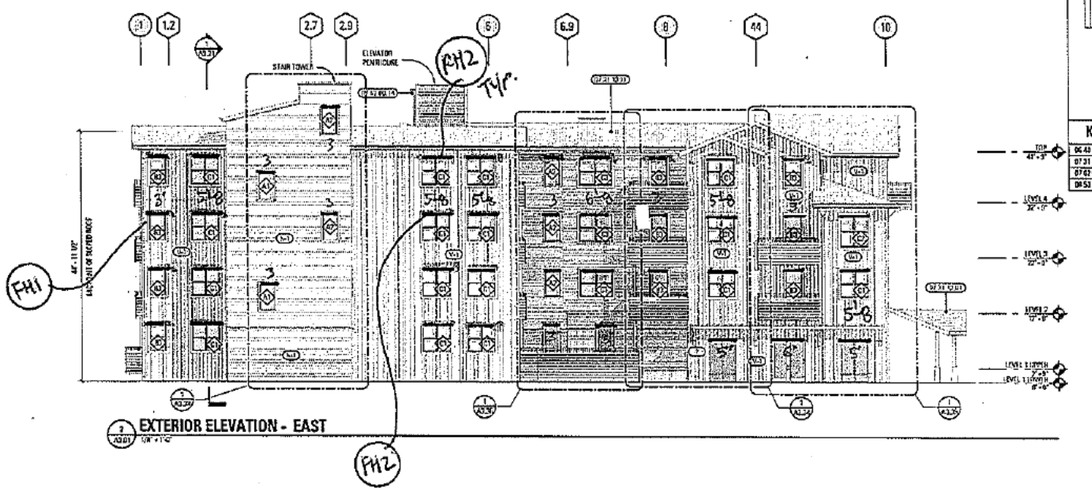
Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
D+L	1.404	1.404



1 EXTERIOR ELEVATION - NORTH



3 EXTERIOR ELEVATION - EAST

SHEET NOTES

LEGEND

FINISH SCHEDULE

WINDOW TAG - SEE SHEET A2.2

FIBER CEMENT LAP SIDING - WALL TYPE B

FIBER CEMENT BOARD AND BATTEN SIDING - WALL TYPE V

KEYNOTES

06 63 13.DE	EXTERIOR THERMALLY TREATED WOOD BASE
07 31 13.DS	ASP UNIT PAVERS
07 62 00.14	DOWNSPOUT, SEE PLUMBING DRAWINGS
08 53 13.DT	VINYL WALL FIN WINDOW

BAR architects
501 Battery Street, Suite 200
San Francisco, CA 94111
415 293 5700
www.bararch.com

CHESTNUT SQUARE
1665 CHESTNUT STREET
LIVERMORE, CA

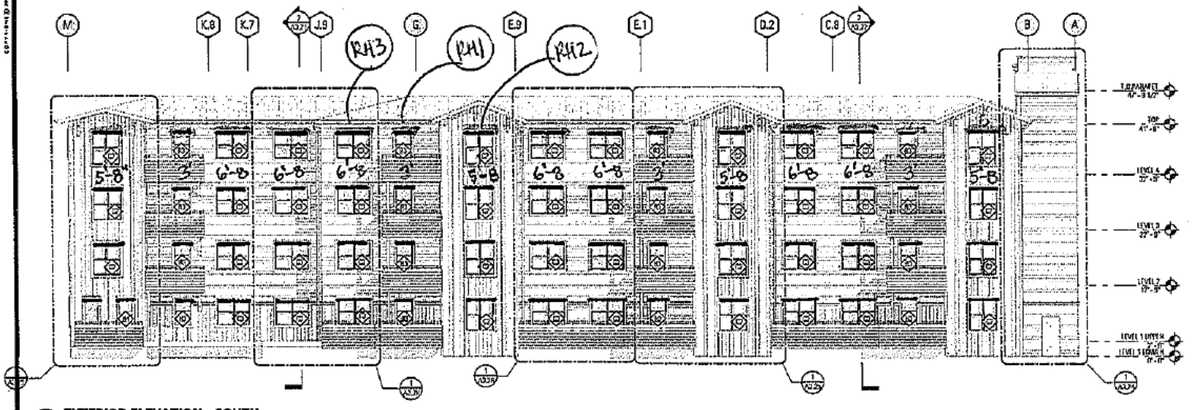
[redacted]

EXTERIOR ELEVATIONS

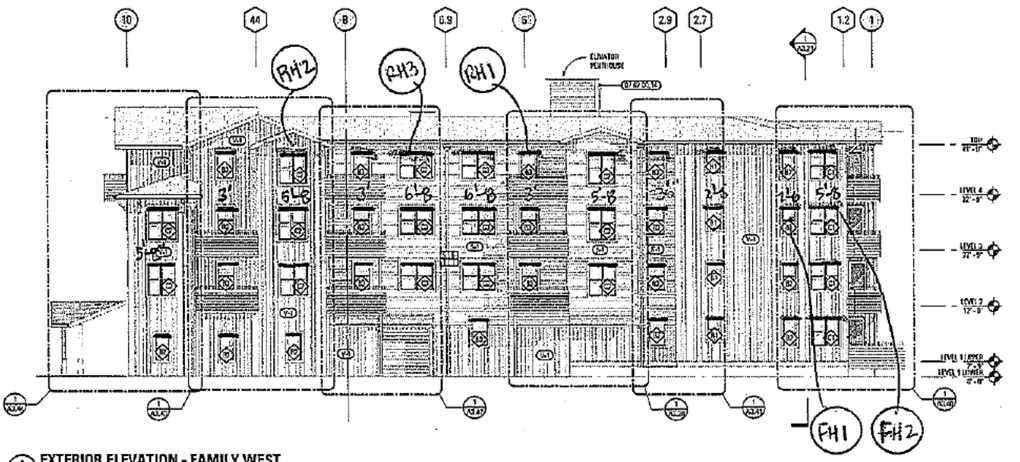
SCALE: As indicated

A3.01

A13



EXTERIOR ELEVATION - SOUTH
1/8" = 1'-0"



EXTERIOR ELEVATION - FAMILY WEST
1/8" = 1'-0"

SHEET NOTES

LEGEND

FINISH SCHEDULE

WINDOW TAG - SEE SHEET A3.99

FIBER CEMENT LAP SIDING - WALL TYPE S

FIBER CEMENT BOARD AND BATTEN SIDING - WALL TYPE V

KEYNOTES

07 62 00.14 DOWNSPOUT, SEE PLUMBING DRAWINGS

BAR architects
301 Battery Street, Suite 300
San Francisco, CA 94111
415 293 5700
www.bararch.com

CHESTNUT SQUARE
1665 CHESTNUT STREET
LIVERMORE, CA

EXTERIOR ELEVATIONS

SCALE: As indicated

A3.02

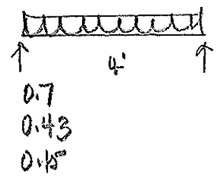
414

MURPHY BURR CURRY, INC.
CONSULTING STRUCTURAL ENGINEERS
85 SECOND STREET, SUITE 501 • SAN FRANCISCO, CA 94105
PHONE: (415) 546-0431 • FAX: (415) 882-7257

6x6 Df#1 (RH1)

$W_{DL} = 25\ psf \times \left(\frac{25.5}{2} + 1.5\right) = 357^{\#/l} + 10 \times 10 = 457^{\#/l}$

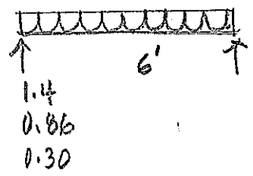
$W_{L} = 20\ psf \times \left(\frac{25.5}{2} + 1.5\right) = 285^{\#/l}$



6x8 Df#1 (RH2)

$W_{DL} = 25\ psf \times \left(\frac{25.5}{2} + 1.5\right) = 357^{\#/l} + 10 \times 10 = 457^{\#/l}$

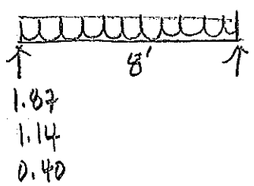
$W_{L} = 20\ psf \times \left(\frac{25.5}{2} + 1.5\right) = 285^{\#/l}$



6x10 Df#1 (RH3)

$W_{DL} = 457^{\#/l}$

$W_{L} = 285^{\#/l}$



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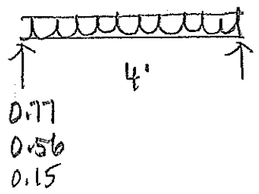
6x6 DF #1 — FH1

$W_{DL} = 40 \times 14/2 + 10 \times 10' = 380^{\#/1}$

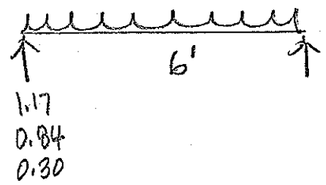
$40 \times 14/2 = 280^{\#/1}$



6x6 DF #1 — FH2

$W_{DL} = 380^{\#/1}$

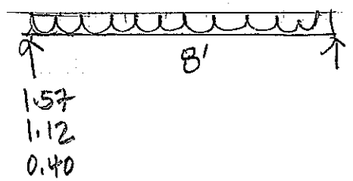
$W = 280^{\#/1}$



6x10 DF #1 — FH3

$W_{DL} = 380^{\#/1}$

$W = 280^{\#/1}$



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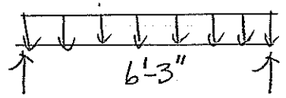
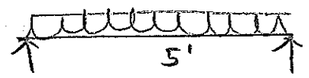
6x10 DF #1 — FH4



$W_{DL} = 40 \times \frac{12}{2} \times 3 + 10 \times 30 = 1020^{\#/'}$

$W_{LL} = 40 \times \frac{12}{2} \times 3 = 720^{\#/'}$

6x8 DF #1 — FH5



$W_{DL} = 1020^{\#/'}$

$W_{LL} = 720^{\#/'}$

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Project Title:
Engineer:
Project Descr:

Project ID: A18

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC

Multiple Simple Beam

Lic. # : KW-06002966

Description : Header (D+L)

Wood Beam Design : RH1 (D+L)

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch (North)				Wood Grade : No.1				
Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

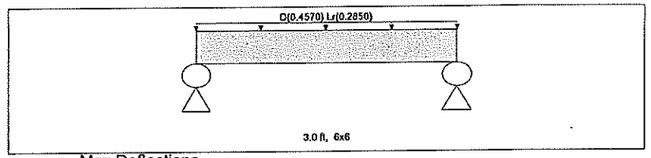
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.4570, Lr = 0.2850 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.224** : 1
fb : Actual : 364.47 psi at 1.500 ft in Span # 1
Fb : Allowable : 1,625.00 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.183** : 1
fv : Actual : 38.98 psi at 2.550 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.70		0.43				
Right Support	0.70		0.43				

Max Deflections

Downward L+Lr+S	0.004 in	Downward Total	0.011 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	8411 >360	Total Defl Ratio	3202 >180

Wood Beam Design : RH2 (D+L)

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch (North)				Wood Grade : No.1				
Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

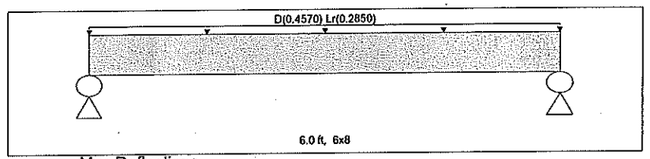
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.4570, Lr = 0.2850 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.485** : 1
fb : Actual : 786.55 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,621.48 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.306** : 1
fv : Actual : 65.00 psi at 5.380 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.40		0.86				
Right Support	1.40		0.86				

Max Deflections

Downward L+Lr+S	0.027 in	Downward Total	0.071 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2666 >360	Total Defl Ratio	1011 >180

Wood Beam Design : RH3 (D+L)

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch (North)				Wood Grade : No.1				
Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

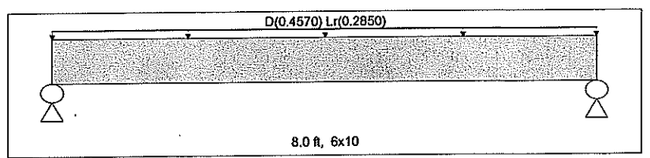
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.4570, Lr = 0.2850 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.540** : 1
fb : Actual : 874.32 psi at 4.000 ft in Span # 1
Fb : Allowable : 1,618.90 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.328** : 1
fv : Actual : 69.79 psi at 0.000 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.87		1.14				
Right Support	1.87		1.14				

Max Deflections

Downward L+Lr+S	0.042 in	Downward Total	0.111 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2285 >360	Total Defl Ratio	864 >180

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A19

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Lic. # : KW-06002966

Wood Beam Design : FH1 (D+L)

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir - Larch (North) Wood Grade : No.1

Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

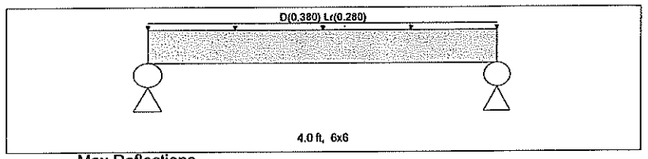
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.380, Lr = 0.280 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.355 : 1**
fb : Actual : 576.98 psi at 2.000 ft in Span # 1
Fb : Allowable : 1,625.00 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.241 : 1**
fv : Actual : 51.13 psi at 0.000 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.77		0.56				
Right Support	0.77		0.56				

Max Deflections

Downward L+Lr+S	0.013 in	Downward Total	0.032 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	3611 >360	Total Defl Ratio	1517 >180

Wood Beam Design : FH2 (D+L)

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir - Larch (North) Wood Grade : No.1

Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

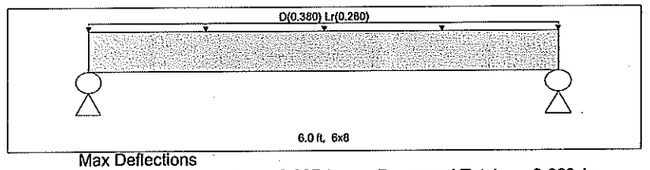
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.380, Lr = 0.280 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.432 : 1**
fb : Actual : 700.67 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,621.48 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.272 : 1**
fv : Actual : 57.90 psi at 5.380 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.17		0.84				
Right Support	1.17		0.84				

Max Deflections

Downward L+Lr+S	0.027 in	Downward Total	0.063 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2713 >360	Total Defl Ratio	1135 >180

Wood Beam Design : FH3 (D+L)

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir - Larch (North) Wood Grade : No.1

Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

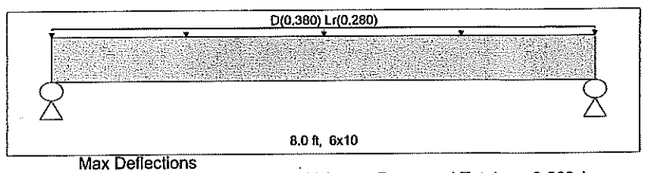
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.380, Lr = 0.280 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.481 : 1**
fb : Actual : 779.17 psi at 4.000 ft in Span # 1
Fb : Allowable : 1,618.90 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.293 : 1**
fv : Actual : 62.20 psi at 0.000 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.57		1.12				
Right Support	1.57		1.12				

Max Deflections

Downward L+Lr+S	0.041 in	Downward Total	0.099 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2326 >360	Total Defl Ratio	970 >180

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A20

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Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Lic. # : KW-06002966

Wood Beam Design : FH4 (D+L)

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch (North)				Wood Grade :	No.1			
Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

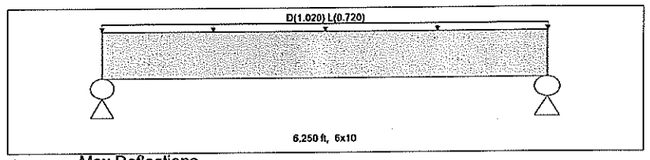
Beam self weight calculated and added to loads
Unif Load: D = 1.020, L = 0.720 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.956 : 1
fb : Actual : 1,240.49 psi at 3.125 ft in Span # 1
Fb : Allowable : 1,297.02 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.696 : 1
fv : Actual : 118.37 psi at 5.479 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	3.22	2.25					
Right Support	3.22	2.25					



Max Deflections

Downward L+Lr+S	0.040 in	Downward Total	0.096 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1897 >360	Total Defl Ratio	780 >180

Wood Beam Design : FH5 (D+L)

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch (North)				Wood Grade :	No.1			
Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

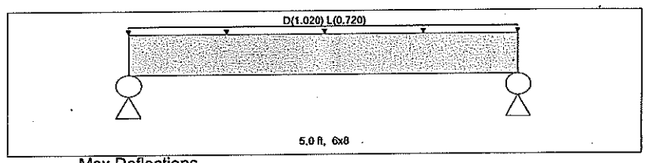
Beam self weight calculated and added to loads
Unif Load: D = 1.020, L = 0.720 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.980 : 1
fb : Actual : 1,272.03 psi at 2.500 ft in Span # 1
Fb : Allowable : 1,298.15 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.705 : 1
fv : Actual : 119.78 psi at 4.383 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.57	1.80					
Right Support	2.57	1.80					



Max Deflections

Downward L+Lr+S	0.033 in	Downward Total	0.080 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1823 >360	Total Defl Ratio	750 >180

Multiple Simple Beam

Description : Header (Window)

Wood Beam Design : H1

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch (North)				Wood Grade : No.1			Density	31.570 pcf
Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi		
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

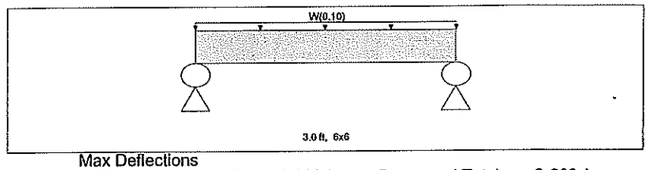
Unif Load: W = 0.10 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.037** : 1
fb : Actual : 48.69 psi at 1.500 ft in Span # 1
Fb : Allowable : 1,300.00 psi
Load Comb : +D+W

Max fv/FvRatio = **0.031** : 1
fv : Actual : 5.21 psi at 2.550 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+W

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.15		
Right Support					0.15		



Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.002 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	23972 >180

Wood Beam Design : H2

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x8 flat, Sawn - General, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch (North)				Wood Grade : No.1			Density	31.570 pcf
Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi		
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

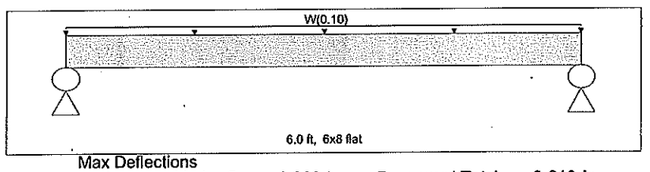
Unif Load: W = 0.10 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.114** : 1
fb : Actual : 147.73 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,300.00 psi
Load Comb : +D+W

Max fv/FvRatio = **0.057** : 1
fv : Actual : 9.63 psi at 5.560 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+W

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.30		
Right Support					0.30		



Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.018 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	3950 >180

Wood Beam Design : H3

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x10 flat, Sawn - General, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch (North)				Wood Grade : No.1			Density	31.570 pcf
Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi		
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

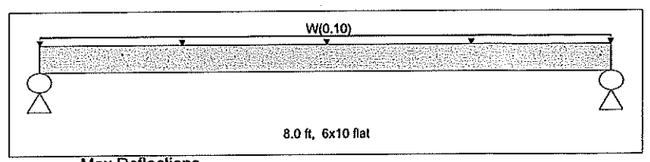
Unif Load: W = 0.10 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.158** : 1
fb : Actual : 205.85 psi at 4.000 ft in Span # 1
Fb : Allowable : 1,300.00 psi
Load Comb : +D+W

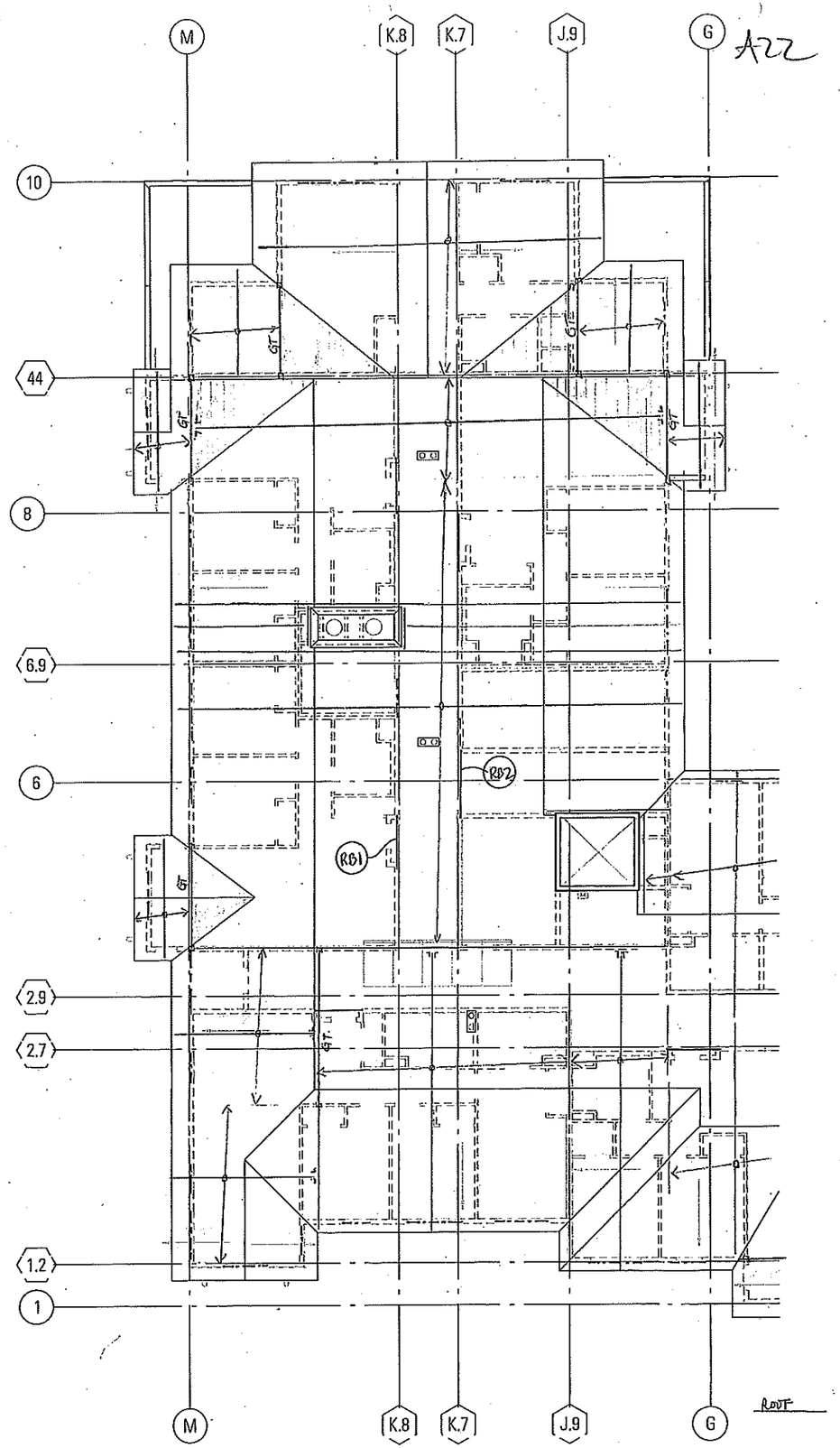
Max fv/FvRatio = **0.062** : 1
fv : Actual : 10.46 psi at 0.000 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+W

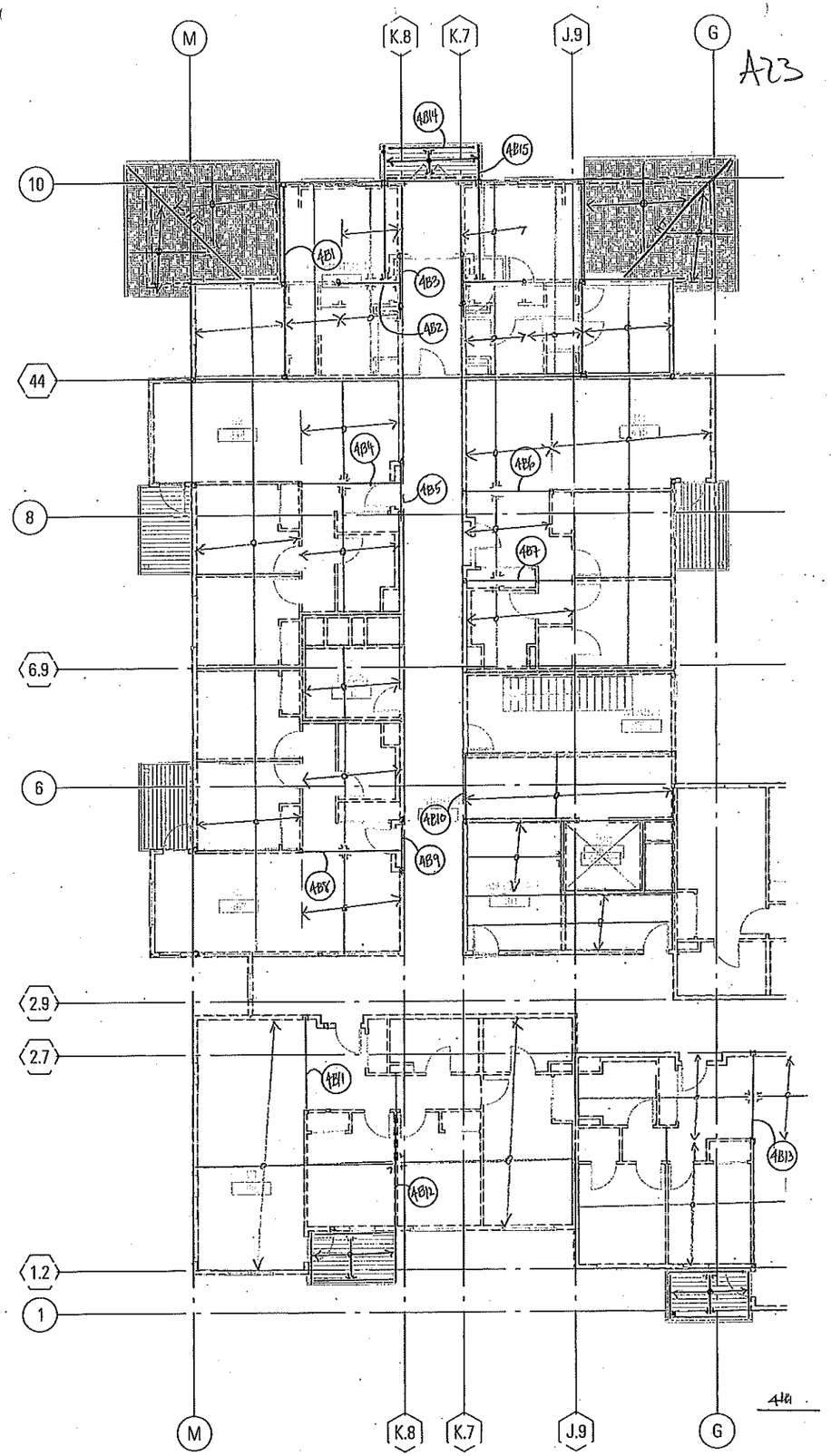
Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.40		
Right Support					0.40		

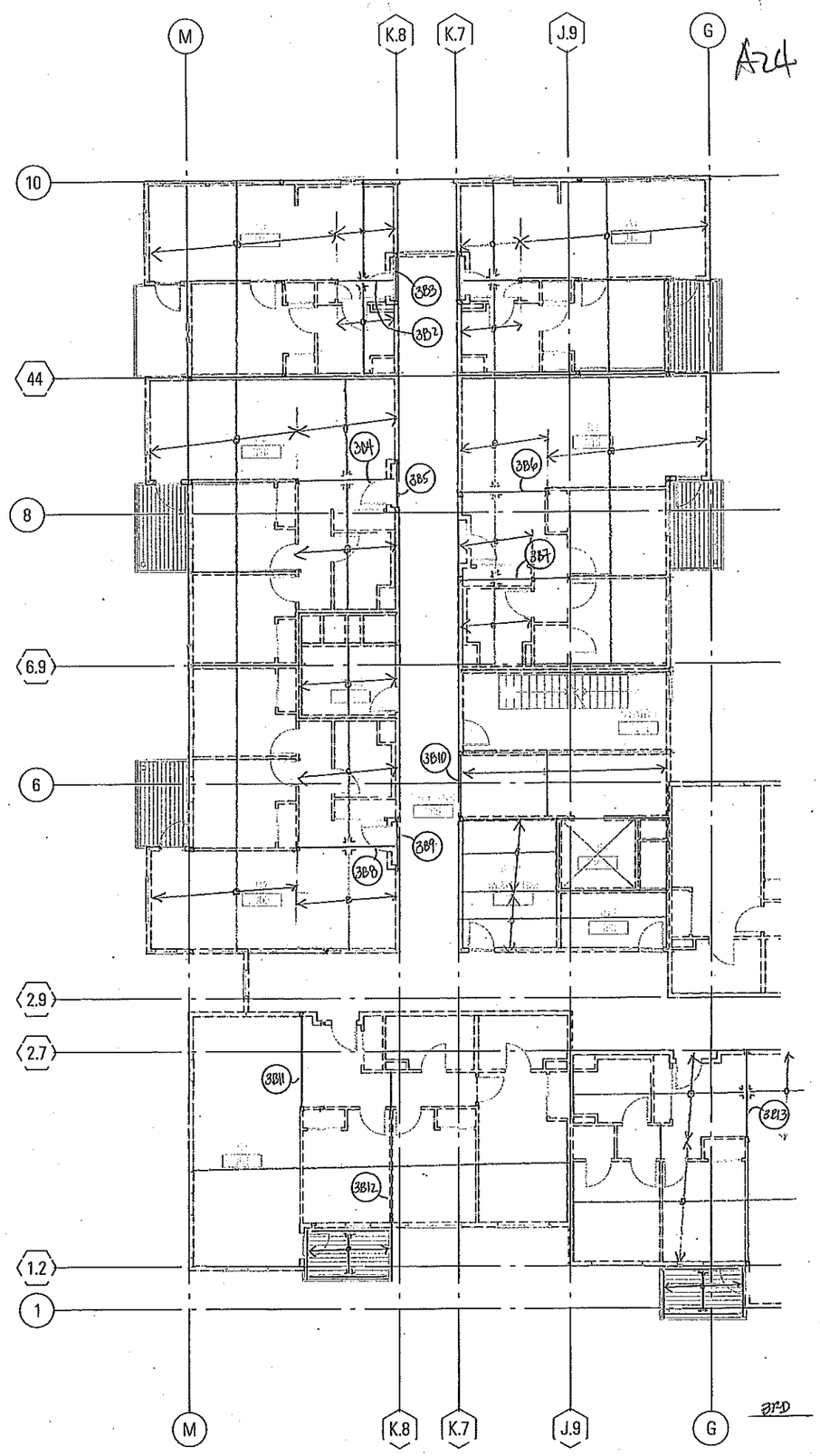


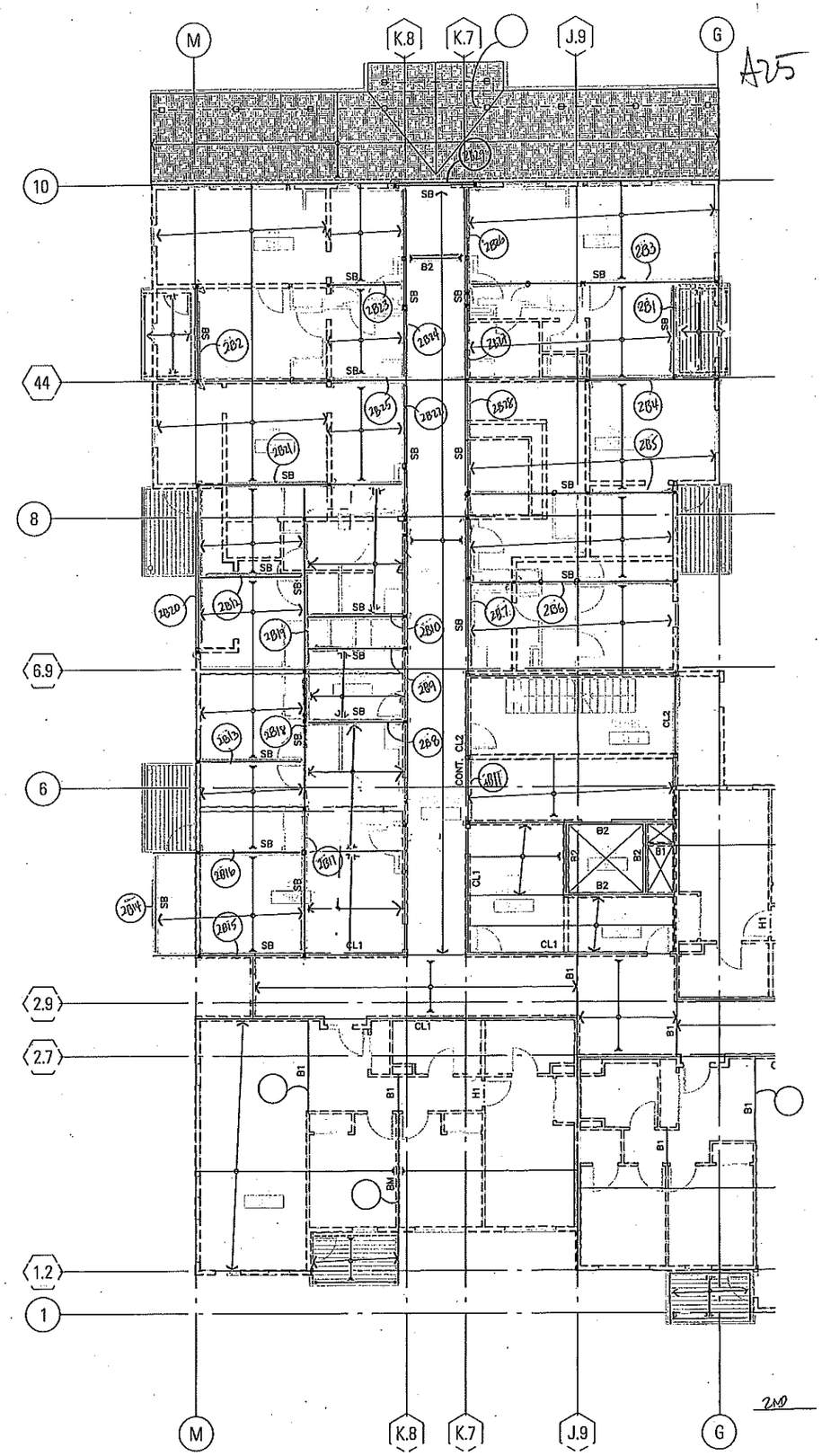
Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.045 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	2126 >180


M
K.8
K.7
J.9
G
A22
10
44
8
6.9
6
2.9
2.7
1.2
1
GT
V
RB2
RB1
ROOF

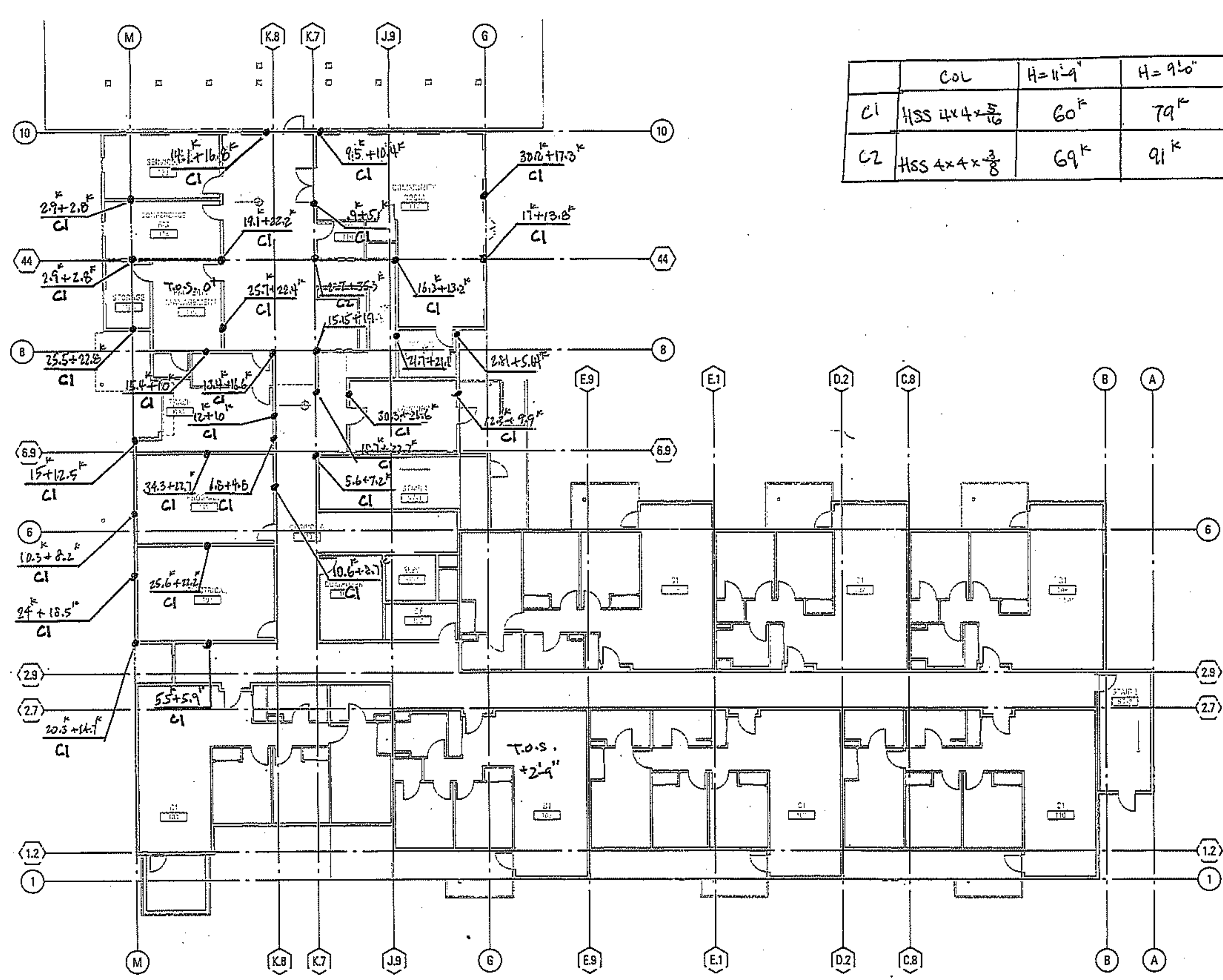

M
K.8
K.7
J.9
G
A23
10
44
8
6.9
6
2.9
2.7
1.2
1
AB1
AB2
AB3
AB4
AB5
AB6
AB7
AB8
AB9
AB10
AB11
AB12
AB13
AB14
AB15
M
K.8
K.7
J.9
G


M
K.8
K.7
J.9
G
10
44
8
6.9
6
2.9
2.7
1.2
1
3B3
3B2
3B4
3B5
3B6
3B7
3B10
3B9
3B8
3B11
3B12
3B13
A24
3F-D


M
K.8
K.7
J.9
G
10
44
8
6.9
6
2.9
2.7
1.2
1
A25
2ND
SB
B2
B1
CL1
CL2
CONT. CL2
H1
BM

A26

	COL	H = 11'-9"	H = 9'-0"
C1	HSS 4x4x$\frac{5}{16}$	60^k	79^k
C2	HSS 4x4x$\frac{3}{8}$	69^k	91^k



1 / S2.01 LEVEL 1 FRAMING PLAN

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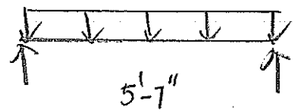
6x6 DF #1

RB1

$W_{NL} = 25 \times 32/2 = 400^{\#/1}$

$W = 20 \times 32/2 = 320^{\#/1}$



DF #1

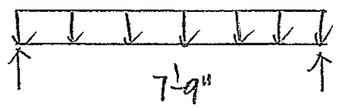
6x8

RB2

GR 6x6 PSL

$W_{DL} = 25 \times 32/2 = 400^{\#/1}$

$W = 20 \times 32/2 = 320^{\#/1}$



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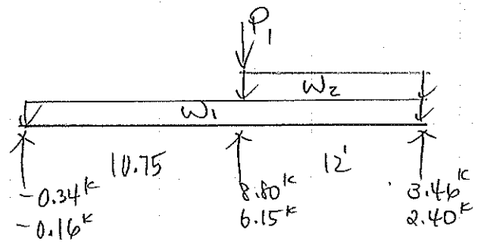
LEVEL 4 FRAMING

5¼ x 11⅞ PSL (4B1)

W_1 DL = 40 psf x 1.33 = 53.3
LL = 40 psf x 1.33 = 53.3

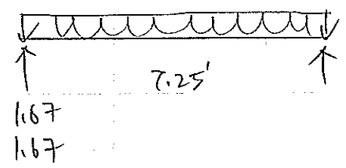


W_2 DL = $25 \times \left(\frac{35.25}{2} + 3\right) + 10 \times 10' = 616^{\#/'}$
LL = $20 \times \left(\frac{35.25}{2} + 3\right) = 413^{\#/'}$

P_1 DL = $616^{\#/'} \times 10.75/2 = 3311^{\#}$
LL = $413^{\#/'} \times 10.75/2 = 2217^{\#}$

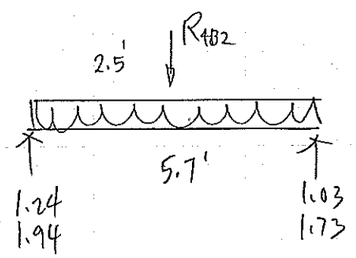
3½ x 11 7/8 PSL (4B2)

$W_{DL} = 40\ psf \times 23/2 = 460^{\#/'}$
$W_{LL} = 40\ psf \times 23/2 = 460^{\#/'}$



3½ x 11 7/8 PSL (4B3)

$W_{DL} = 30 \times 7/2 = 105^{\#/'}$
$W_{LL} = 100 \times 7/2 = 350^{\#/'}$



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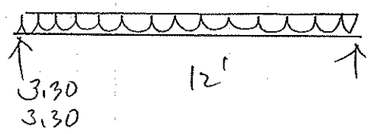
5¼ x 11 7/8 PSL (4B4)

$W_{DL} = 40 psf \times 27.5/2 = 550$

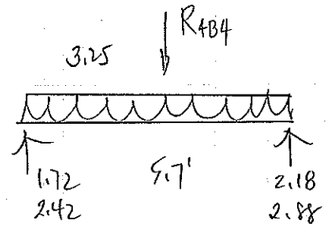
$W_{LL} = 40 psf \times 27.5/2 = 550$



3½ x 11 7/8 PSL (4B5)

$W_{DL} = 30 \times 7/2 = 105^{\#/'}$

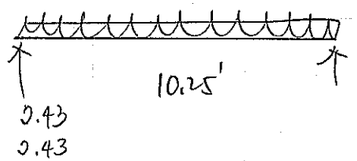
$W_{LL} = 100 \times 7/2 = 350^{\#/'}$



3½ x 11 7/8 PSL (4B6)

$W_{DL} = 40 psf \times 23.75/2 = 475^{\#/'}$

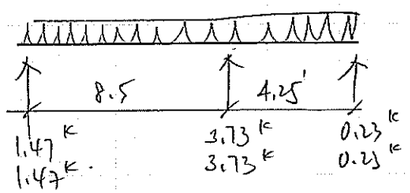
$W_{LL} = 40 psf \times 23.75/2 = 475^{\#/'}$



3½ x 11 7/8 PSL (4B7)

$W_{DL} = 40 psf \times 21.25/2 = 425^{\#/'}$

$W_{LL} = 40 psf \times 21.25/2 = 425^{\#/'}$



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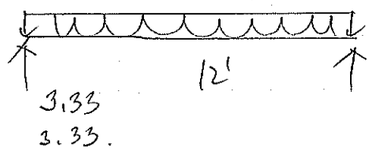
5¼ x 11⅞ PSL (4B8)

$W_{DL} = 40\ psf \times 27.75/2 = 555^{\#/'}$

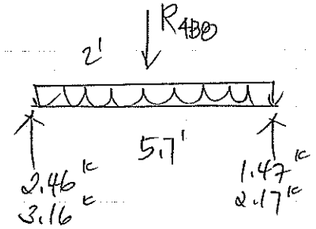
$L = 40\ psf \times 27.75/2 = 555^{\#/'}$



3½ x 11⅞ PSL (4B9)

$W_{DL} = 30\ psf \times 7/2 = 105^{\#/'}$

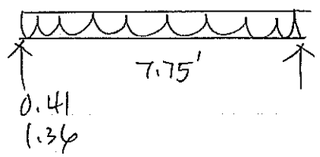
$L = 100\ psf \times 7/2 = 350^{\#/'}$



5¼ x 11⅞ PSL (4B10)

$W_{DL} = 105^{\#/'}$

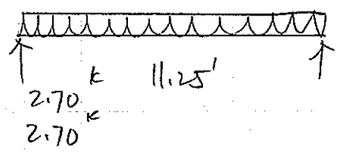
$L = 350^{\#/'}$



3½ x 11⅞ PSL (4B11)

$W_{DL} = 40\ psf \times 24/2 = 480^{\#/'}$

$L = 40\ psf \times 24/2 = 480^{\#/'}$



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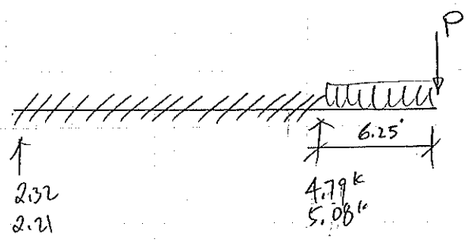
HSS 7x4x5/16 (4B12)



$P_{DL} = ((30 \times 6.25/2) + 50^{\#/f})\ 11/2 = 791^{\#}$

$u = (60 \times 11/2)\ 6.25/2 = 1031^{\#}$

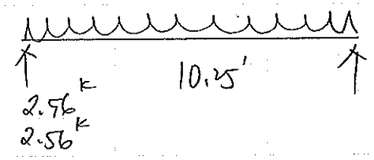
$W_{DL} = 30\ psf \times 1.33 + 50^{\#/f} = 90^{\#/f}$

$u = 60\ psf \times 1.33 = 80^{\#/f}$

3½x11⅞ PSL (4B13)

$W_{DL} = 40\ psf \times 25/2 = 500^{\#/f}$

$u = 40\ psf \times 25/2 = 500^{\#/f}$



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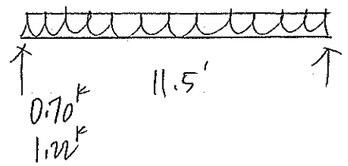
4x12 GLB (4B14)

$W_{DL} = 30 \times 4.25/2 + 50^{\#/'} = 114^{\#/'}$

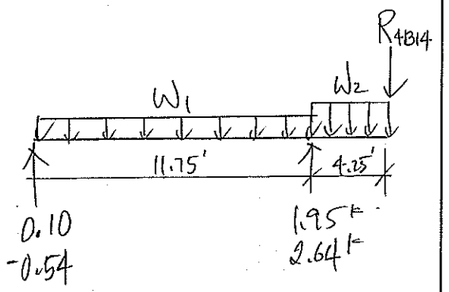
$LL = 100 \times 4.25/2 = 213^{\#/'}$



HSS 7x4x5/16 (4B15)

$W_{1\,DL} = 40 \times 1.33 = 53.3^{\#/'}$

$40 \times 1.33 = 53.3^{\#/'}$



$W_{2DL} = 30 \times 1.33' + 50^{\#/'} = 90^{\#/'}$

$LL = 100 \times 1.33' = 133^{\#/'}$

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LEVEL 3 FRAMING : (3B2) to (3B13) ARE SAME AS (4B2) to (4B13)

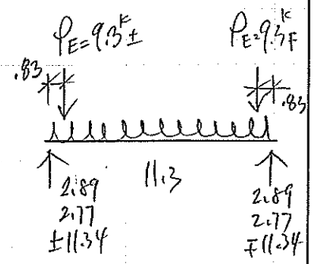
LEVEL 2 FRAMING :

W8X24 (2B1)(2B2)

$W_{DL} = 40\ psf \times 1.33 \times 3 + 30\ psf \times 1.33 + 10^{psf} \times 29' = 490^{\#/'}$

$W = 40\ psf \times 1.33 \times 3 + 60\ psf \times 1.33 = 240^{\#/'}$

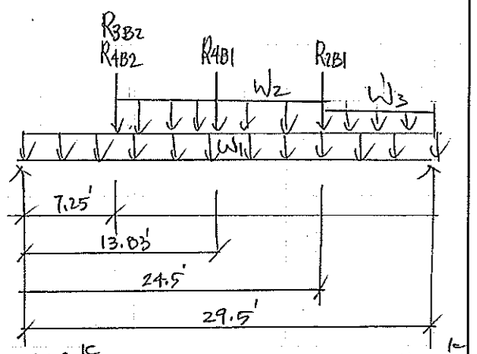
$P_E = 9.3^{k} \pm$ OK $13.29^{k} = Q_E$ (LRFD)



W18X65 (2B3)

$W_{1\ DL} = 40\ psf \times 23.25/2 = 465^{\#/'}$

$W = 40\ psf \times 23.25/2 = 465^{\#/'} \times .6 = 279^{\#/'}$



$R_{3B2} = R_{4B2} = 1.67^{k} + 1.67^{k} = 1.67^{k} + .6(1.67^{k})$

$R_{4B1} = 8.8^{k} + 6.15^{k} =$

$R_{2B1} = 2.89^{k} + 2.77^{k} = 2.89^{k} + 1.7^{k} \pm 11.34$

CAMBER : $0.75(\Delta_{dead})$
$= 0.75(0.795'')$
$\approx 0.5''$

$W_{2\ DL} = 40\ psf \times 23.25/2 \times 2 = 930^{\#/'}$

$W = 40\ psf \times 23.25/2 \times 2 = 930^{\#/'} \times .6 = 558^{\#/'}$

$W_{3\ DL} = 40\ psf \times (23.25/2 + 11/2) + [25 \times (23.25/2 + 3)] + 10 \times 29' = 1.34^{K/'}$

$W = 40\ psf \times (23.25/2 + 11/2) + [20 \times (23.25/2 + 3)] = 0.98^{K/'} \times .6 = 0.71^{K/'}$

LL. REDUCTION : AREA $= 29.5' \times \frac{23.25}{2} \times 3 = 1029\ SF$

$L = L_o(.25 + \frac{15}{\sqrt{2(1029)}})$

$L = L_o(0.58)$

USE 60% LL.

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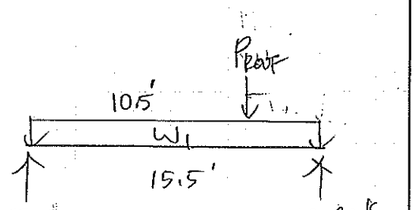
W18x50 (2B4)



$W_{1DL} = 40\ psf \times \frac{24.5}{2} \times 3 + 20 \times 29' + 25 \times (\frac{11}{2} + 3) = 1.97^{k/'}$

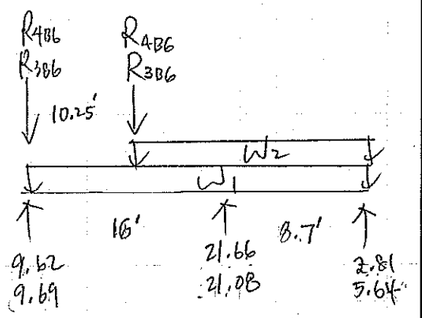
$40\ psf \times \frac{24.5}{2} \times 3 + 20 \times (\frac{11}{2} + 3) = 1.64^{k/'}$

$P_{DL} = 25 \times 25/2 \times 12.5/2 = 1.95^{k}$

$w = 20 \times 25/2 \times 12.5/2 = 1.56^{k}$

$R_{2B1} = 2.89^{k} + 2.77^{k} \pm 11.34$

W18x35 (2B5)



$W_{1\ DL} = 40\ psf \times 23.7/2 = 474^{\#/'}$

$LL = 40\ psf \times 23.7/2 = 474^{\#/'}$

$R_{3B6} = 2.43^{k} + 2.43^{k} = R_{4B6}$

$W_{2\ DL} = 40\ psf \times 23.7/2 \times 2 = 948^{\#/'}$

$LL = 40\ psf \times 23.7/2 \times 2 = 948^{\#/'}$

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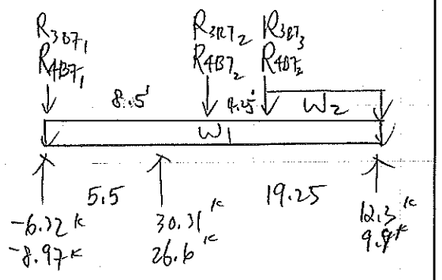
W18X50 (286)

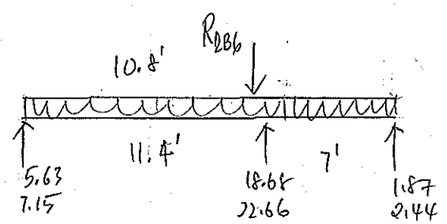


$W_{1\,DL} = 40 \times 21.25/2 = 425^{\#/'}$
$W = 40 \times 21.25/2 = 425^{\#/'}$

$W_{2\,DL} = 40 \times 21.25/2 \times 2 + 10 \times 29 = 1140$
$W = 40 \times 21.25/2 \times 2 = 850$

$R_{3B71} + R_{4B71} = 1.47^{k} \times 2 = 2.94^{k} + 2.94^{k}$
$R_{3B72} + R_{4B72} = 3.73^{k} \times 2 = 7.46^{k} + 7.46^{k}$
$R_{3B73} + R_{4B73} = 0.23^{k} \times 2 = 0.46^{k} + 0.46^{k}$

W18X35 (287)



$W_{DL} = 25 \times 32/2 + 30 \times 7/2 \times 3 + 40 \times 1.33 \times 3$
$= +10 \times 29 = 1165^{\#/'}$
$W = 20 \times 32/2 + 100 \times 7/2 \times 3 + 40 \times 1.33 \times 3$
$= 1530^{\#/'}$

$R_{2B6} = -6.32 + -8.97^{k}$

$R_{2B6} = 2.94 + .425 \times 5.5/2 = 4.10$
$2.94 + .425 \times 5.5/2 = 4.10$

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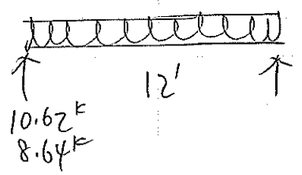
W8x40 (288)

$W_{DL} = 40\ psf \times 24/2 \times 3 + 10 \times 29 = 1730^{\#/l}$

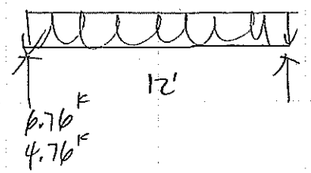
$LL = 40\ psf \times 24/2 \times 3 = 1440^{\#/l}$



W8x35 (289)

$W_{DL} = 25 \times 4.5/2 + 40 \times 12.5/2 \times 3 + 10 \times 29 = 1091^{\#/l}$

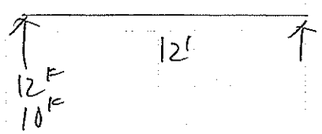
$LL = 20 \times 4.5/2 + 40 \times 12.5/2 \times 3 = 793^{\#/l}$



W8x40 (2810)

$W_{DL} = 40 \times \frac{25}{2} \times 3 + 10 \times 29 = 1970^{\#/l}$

$LL = 40 \times \frac{25}{2} \times 3 = 1680^{\#/l}$



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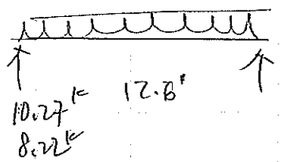
5⅛ x 11 7/8 PSL (2B11) SAME AS (4B10)

W8 x 35 (2B12) (2B13)

$W_{DL} = 40 \times 21.75/2 \times 3 + 10 \times 29 = 1595^{\#/'}$

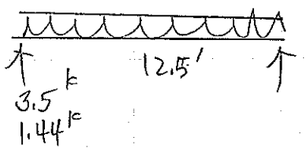
$LL = 40 \times 21.75/2 \times 3 = 1305^{\#/'}$



W8 x 21 (2B14)

$W_{DL} = 25 \times 3.5 + 40 \times 1.33 \times 3 + 10^{psf} \times 29' = 537^{\#/'}$

$20 \times 3.5 + 40 \times 1.33 \times 3 = 230^{\#/'}$



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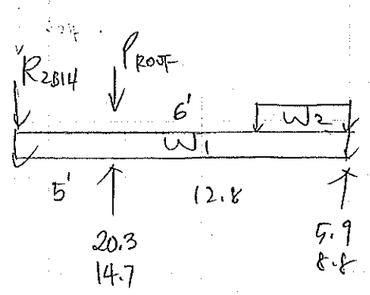
W12x35 (2B15)



$W_{DL_1} = 40\ psf \times 12.5/2 \times 3 + 10 \times 29 = 1040^{\#/'}$

$LL_1 = 40\ psf \times 12.5/2 \times 3 = 750^{\#/'}$

$W_{DL_2} = 30 \times 7/2 \times 3 = 315^{\#/'}$

$LL_2 = 100 \times 7/2 \times 3 = 1050^{\#/'}$

$R_{2B14} = 3.5^{k} + 1.44^{k}$

$P_{ROOF} = 25 \times 25/2 \times 12.5/2 = 1.95^{k}$

$20 \times 25/2 \times 12.5/2 = 1.56^{k}$

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W12X35

2B16

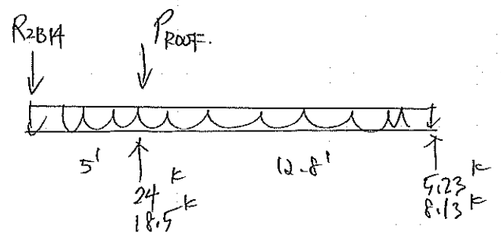
$W_{DL} = 40 \times 23/2 \times 3 + 10' \times 29 = 1670^{\#/l}$

$40 \times 23/2 \times 3 = 1380^{\#/l}$

$R_{2B14} = 3.5^{k} + 1.44^{k}$

$P_{ROOF} = 25 \times 25/2 \times 12.5/2 = 1.95^{k}$

$20 \times 20/2 \times 12.5/2 = 1.56^{k}$



W12X58

2B17

$W_{DL} = 40 \times 1.33 = 53.3^{\#/l}$

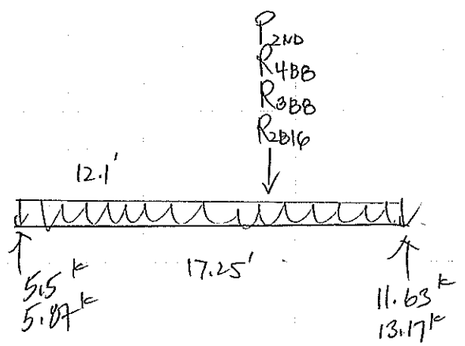
$40 \times 1.33 = 53.3^{\#/l}$

$R_{2B16} = 5.23 + 8.13^{k}$

$R_{4BB} = 3.33^{k} + 3.33^{k}$

$R_{3BB} = 3.33^{k} + 3.33^{k}$

$R_{2BB} = 3.33^{k} + 3.33^{k}$



W12X58

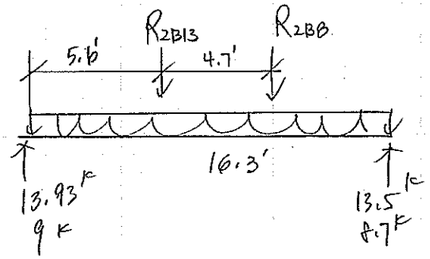
2B18

$W_{DL} = 40 \times 1.33 + 10 \times 29 = 344^{\#/l}$

$W = 40 \times 1.33 = 53.3^{\#/l}$

$R_{2B13} = 10.27^{k} + 8.22^{k}$

$R_{2BB} = 10.62^{k} + 8.64^{k}$



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W12x58

2B19

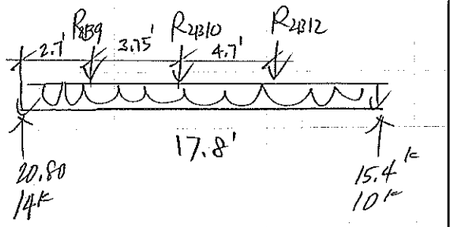
$W_{DL} = 40 \times 1.33 + 10 \times 29 = 344^{\#/'}$

$LL = 40 \times 1.33 = 53.3^{\#/'}$

$R_{2B9} = 6.76 + 4.76^{K}$

$R_{2B10} = 12^{K} + 10^{K}$

$R_{2B12} = 10.27^{K} + 8.22^{K}$



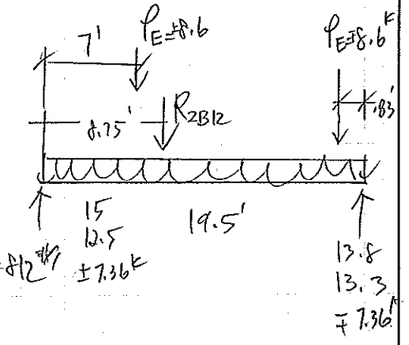
W18x71

2B20

$R_{2B12} = 10.27^{K} + 8.22^{K}$

$W_{DL} = 25 \times (25/2 + 2) + 40 \times 1.33 \times 3 + 10 \times 29 = 812^{\#/'}$
$20 \times (25/2 + 2) + 40 \times 1.33 \times 3 = 660^{\#/'}$

$P_D = 30 \times 10/2 \times 6/2 \times 3 = 1350^{\#}$
$L = 60 \times 10/2 \times 6/2 \times 3 = 2700^{\#}$



$P_E = 8.6^{K}$ OR $Q_E = 12.29^{K}$ (LRFD)

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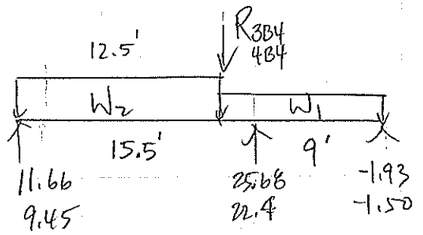
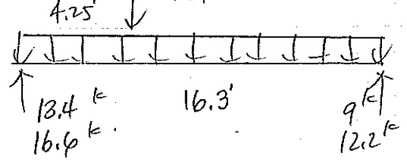
W12x50 (2B21)



$W1_{DL} = 40\ psf \times 27.7/2 = 554^{\#/l}$
$W = 40\ psf \times 27.7/2 = 554^{\#/l}$

$W2_{DL} = 40 \times 23/2 \times 3 + 10 \times 29 = 1670^{\#/l}$
$40 \times 23/2 \times 3 = 1380^{\#/l}$

$R_{3B4} = 3.30^{k} + 3.30^{k}$
$R_{4B4} = 3.30^{k} + 3.30^{k}$



W18x50 (2B22)

$R_{4B4} = 3.3^{k} + 3.3^{k}$
$R_{3B4} = 3.3^{k} + 3.3^{k}$

$W_{DL} = 30 \times 7/2 \times 3 + 40 \times 1.33 \times 3 + 10^{lb/ft} \times 29 = 765^{\#/l}$
$W = 100 \times 7/2 \times 3 + 40 \times 1.33 \times 3 = 1210^{\#/l}$

$R_{2B21} = 554 \times 4.5 = 2.5^{k}$
$554 \times 4.5 = 2.5^{k}$

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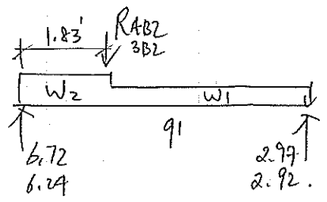
5 1/4 x 11 7/8 PSL (2B23)

W_1 DL = 40 x 23/2 = 460 #/'
LL = 40 x 23/2 = 460 #/'

W_2 DL = 40 x 23/2 x 3 + 10 x 29' = 1670 #/'
LL = 40 x 23/2 x 3 = 1380 #/'

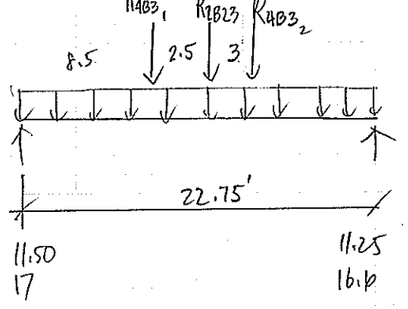
$R_{4B2} = 1.67^k + 1.67^k$
$R_{3B2} = 1.67^k + 1.67^k$



W18x65 (2B24)

$R_{3B3_1} + R_{4B3_1} = 2.48^k + 3.88^k$
$R_{3B3_2} + R_{4B2} = 2.06^k + 3.46^k$
$R_{2B23} = 2.97^k + 2.92^k$

$W_{DL} = 30 \times 7/2 \times 3 + 10 \times 29 = 605$ #/'
$W_{LL} = 100 \times 7/2 * 3 = 1050$ #/'



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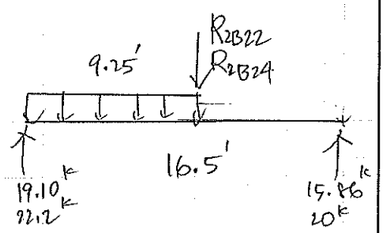
W18X86 — 2B25

$R_{2B22} = 9^K + 12.2^K$

$R_{2B24} = 11.90^K + 17^K$

$W_{NL} = 40 \times 23.5/2 \times 3 = 1410^{\#/'}$

$W = 40 \times 23.5/2 \times 3 = 1410^{\#/'}$



W18X35 — 2B26 2B27

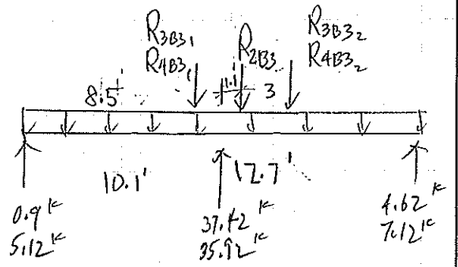
$W_{NL} = 30 \times 7/2 * 3 + 10 \times 29 = 605^{\#/'}$

$W = 100 \times 7/2 \times 3 = 1050^{\#/'}$

$R_{2B3} = 23.8^K + 13.9^K$

$R_{3B3_2} + R_{4B3_2} = 2.06^K + 8.46^K$

$R_{3B3_1} + R_{4B3_1} = 2.48^K + 3.88^K$

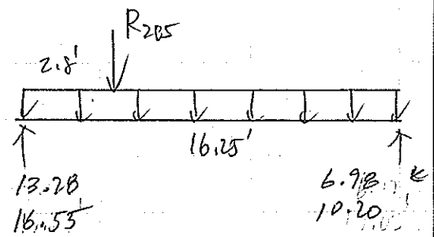


W18X50 — 2B28

$W_{NL} = 30 \times 7/2 \times 3 + 10 \times 29 = 605^{\#/'}$

$W = 100 \times 7/2 \times 3 = 1050^{\#/'}$

$R_{2B5} = 9.62 + 9.69$



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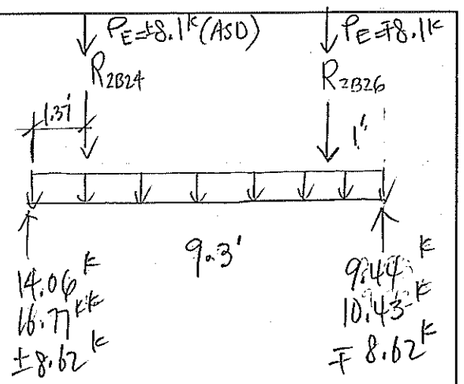
W18 x 35 — 2B29



$W_{DL} = 25 \times (9/2 + 4) + 10 \times 29 = 503^{\#/'}$

$20 \times (9/2 + 4) = 170^{\#/'}$

$R_{2B24} = 11.25^{K} + 16.6^{K}$

$R_{2B26} = 4.62^{K} + 7.12^{K}$

$W_{DL} = 40 \times 12/2 \times 3 + 10' \times 29 + 125 \times 4' = 1110^{\#/'}$

$LL = 40 \times 12/2 \times 3 + 20 \times 4' = 800^{\#/'}$

$P_E = 8.1^{K}$ (ASD) OR 11.57^{K} (LRFD) $= Q_E$

$S_{DS} = 1.036$, $\Omega = 2\frac{1}{2}$

COMB $= (1.0 + 0.14 S_{DS})D + 0.7\Omega Q_E$

$1.15D + 1.75Q_E$

COMB: $(1.0 + 0.105 S_{DS})D + 0.525\Omega Q_E + 0.75L$

$1.11D + 1.313Q_E + 0.75L$

COMB. $(0.6 - 0.14 S_{DS})D + 0.7\Omega Q_E$

$0.455D + 1.75Q_E$

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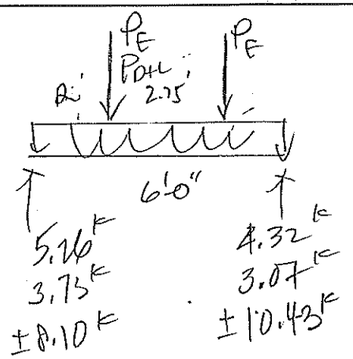
5/4 x 16 PSL ok

2000

USE W18x35 FOR COLN.

$W_{DL} = 40 \times 12/2 \times 3 + 10 \times 29 + 25 \times 4' = 1110^{\#/'}$

$40 \times 12/2 \times 3 + 20 \times 4 = 800^{\#/'}$



$P_{D+L} = 1110 \times 5/2 = 2.8^K$

$800 \times 5/2 = 2.0^K$

$P_E = 6.5^K$ OR $9.28^K = Q_E$

COMB = $1.15\,D + 1.75\,Q_E$

COMB = $1.11\,D + 1.313\,Q_E + 0.75\,L$

COMB = $0.455\,D + 1.75\,Q_E$

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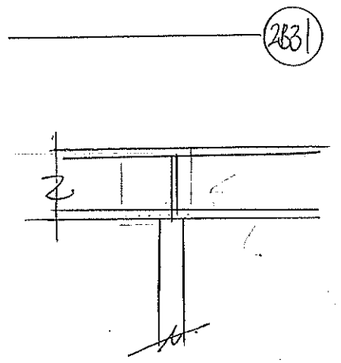

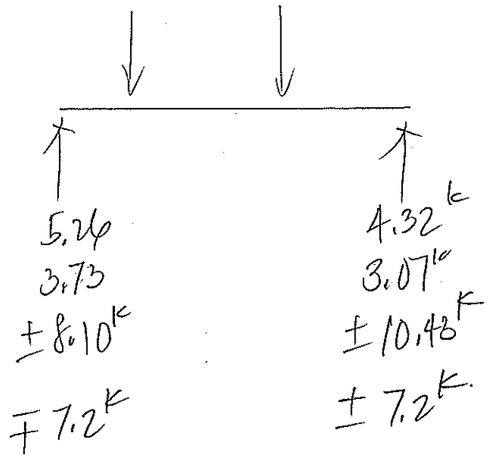

C/ T_{MAX} = 17.63K (LRFD)

C_{MAX} = 1.15(4.32K) + 1.75(17.63K) = 35.8K

T_{MAX} = 0.455(4.32K) − 1.75(17.63K) = −29K

USE HSS 4x4x5/16 POST.

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STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A47

Printed: 11 JAN 2018, 12:05PM

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Description : Roof Framing

Wood Beam Design : RB1

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir - Larch (North) Wood Grade : No.1

Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

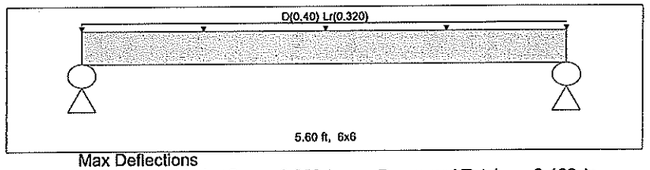
Beam self weight calculated and added to loads
Unif Load: D = 0.40, Lr = 0.320 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.759 : 1
fb : Actual : 1,232.66 psi at 2.800 ft in Span # 1
Fb : Allowable : 1,625.00 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = 0.399 : 1
fv : Actual : 84.75 psi at 0.000 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.14		0.90				
Right Support	1.14		0.90				



Max Deflections

Downward L+Lr+S	0.058 in	Downward Total	0.132 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1151 >360	Total Defl Ratio	507 >180

Wood Beam Design : RB2

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **6x8, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir - Larch (North) Wood Grade : No.1

Fb - Tension	1,300.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.570 pcf
Fb - Compr	1,300.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

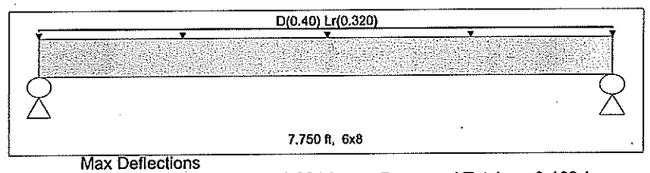
Beam self weight calculated and added to loads
Unif Load: D = 0.40, Lr = 0.320 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.786 : 1
fb : Actual : 1,273.84 psi at 3.875 ft in Span # 1
Fb : Allowable : 1,620.40 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = 0.406 : 1
fv : Actual : 86.29 psi at 7.130 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.59		1.24				
Right Support	1.59		1.24				



Max Deflections

Downward L+Lr+S	0.084 in	Downward Total	0.192 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1101 >360	Total Defl Ratio	483 >180

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Project Title:
Engineer:
Project Descr:

Project ID: A4B

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
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Wood Beam

Lic. # : KW-06002966

Description : 4B1

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

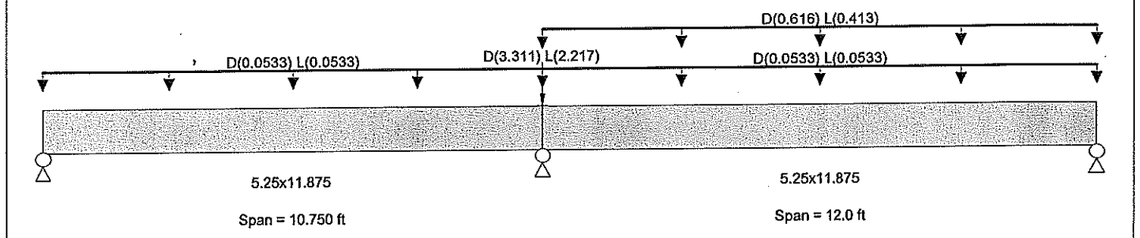
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling

Fb - Tension	2,900.0 psi
Fb - Compr	2,900.0 psi
Fc - Prll	2,900.0 psi
Fc - Perp	750.0 psi
Fv	290.0 psi
Ft	2,025.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0 ksi
Eminbend - xx	1,016.54 ksi
Density	32.210 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : D = 0.05330, L = 0.05330 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.05330, L = 0.05330 , Tributary Width = 1.0 ft
Uniform Load : D = 0.6160, L = 0.4130 , Tributary Width = 1.0 ft
Point Load : D = 3.311, L = 2.217 k @ 0.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.506 : 1	Maximum Shear Stress Ratio	=	0.556 : 1
Section used for this span		5.25x11.875	Section used for this span		5.25x11.875
fb : Actual	=	1,467.66 psi	fv : Actual	=	161.37 psi
FB : Allowable	=	2,900.00 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	6.838 ft	Location of maximum on span	=	10.750 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.099 in	Ratio =	1458
Max Upward L+Lr+S Deflection	-0.032 in	Ratio =	4083
Max Downward Total Deflection	0.242 in	Ratio =	594
Max Upward Total Deflection	-0.080 in	Ratio =	1610

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+L+H					1.00	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.750 ft	1	0.386	0.556	1.00	1.00	1.00	1.00	1.00	1.00	1.00	11.51	1,119.35	2900.00	6.71	161.37	290.00
Length = 12.0 ft	2	0.506	0.556	1.00	1.00	1.00	1.00	1.00	1.00	1.00	15.09	1,467.66	2900.00	6.71	161.37	290.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	6.436	D+L	-0.0801	6.486

MURPHY BURR CURRY, INC.
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Project Title:
Engineer:
Project Descr:

Project ID: A49

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
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Wood Beam

Lic. # : KW-06002966

Description : 4B1

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	2	0.2422	6.436		0.0000	6.486

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-0.498	14.944	5.854
D Only	-0.338	8.798	3.456
L Only	-0.159	6.146	2.399
D+L	-0.498	14.944	5.854

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A5D

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Description : 4th Level Framing

Wood Beam Design : 4B2

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

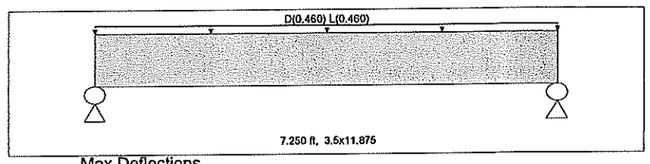
Applied Loads

Unif Load: D = 0.460, L = 0.460 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.308 : 1**
fb : Actual : 881.80 psi at 3.625 ft in Span # 1
Fb : Allowable : 2,864.63 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.304 : 1**
fv : Actual : 88.26 psi at 6.283 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.67	1.67					
Right Support	1.67	1.67					

Max Deflections

Downward L+Lr+S	0.029 in	Downward Total	0.059 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2956 >360	Total Defl Ratio	1478 >180

Wood Beam Design : 4B3

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

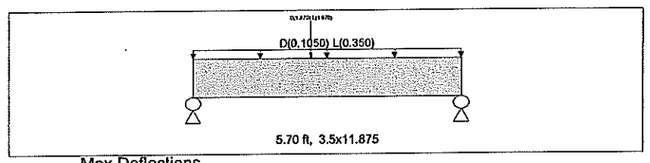
Applied Loads

Unif Load: D = 0.1050, L = 0.350 k/ft, Trib= 1.0 ft
Point: D = 1.670, L = 1.670 k @ 2.50 ft

Design Summary

Max fb/Fb Ratio = **0.330 : 1**
fb : Actual : 947.82 psi at 2.508 ft in Span # 1
Fb : Allowable : 2,873.41 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.339 : 1**
fv : Actual : 98.25 psi at 0.000 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.24	1.94					
Right Support	1.03	1.73					

Max Deflections

Downward L+Lr+S	0.020 in	Downward Total	0.034 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	3458 >360	Total Defl Ratio	2037 >180

Wood Beam Design : 4B4

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **5.25x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

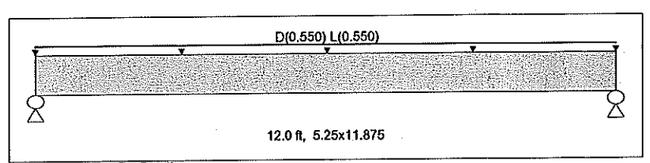
Applied Loads

Unif Load: D = 0.550, L = 0.550 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.670 : 1**
fb : Actual : 1,925.62 psi at 6.000 ft in Span # 1
Fb : Allowable : 2,875.37 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.460 : 1**
fv : Actual : 133.39 psi at 11.040 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	3.30	3.30					
Right Support	3.30	3.30					

Max Deflections

Downward L+Lr+S	0.176 in	Downward Total	0.352 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	817 >360	Total Defl Ratio	408 >180

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: AB1

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
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Multiple Simple Beam

Lic. # : KW-06002966

Wood Beam Design : 4B5

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist — Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density 32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi	

Applied Loads

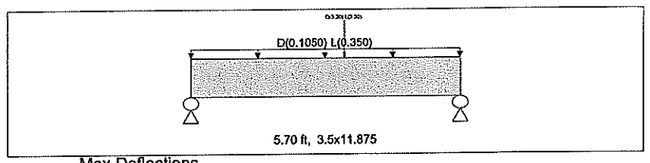
Unif Load: D = 0.1050, L = 0.350 k/ft, Trib= 1.0 ft
Point: D = 3.30, L = 3.30 k @ 3.250 ft

Design Summary

Max fb/Fb Ratio = **0.560 : 1**
fb : Actual : 1,608.85 psi at 3.249 ft in Span # 1
Fb : Allowable : 2,873.41 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.574 : 1**
fv : Actual : 166.39 psi at 4.712 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.72	2.42					
Right Support	2.18	2.88					



Max Deflections

Downward L+Lr+S	0.031 in	Downward Total	0.055 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2236 >360	Total Defl Ratio	1239 >180

Wood Beam Design : 4B6

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist — Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density 32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi	

Applied Loads

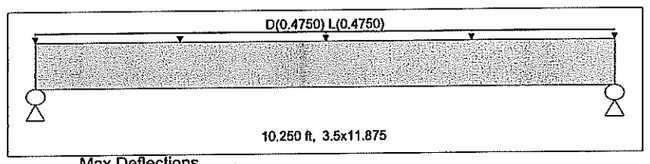
Unif Load: D = 0.4750, L = 0.4750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.640 : 1**
fb : Actual : 1,820.03 psi at 5.125 ft in Span # 1
Fb : Allowable : 2,845.20 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.493 : 1**
fv : Actual : 142.91 psi at 0.000 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.43	2.43					
Right Support	2.43	2.43					



Max Deflections

Downward L+Lr+S	0.121 in	Downward Total	0.243 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1013 >360	Total Defl Ratio	506 >180

Wood Beam Design : 4B8

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **5.25x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist — Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density 32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi	

Applied Loads

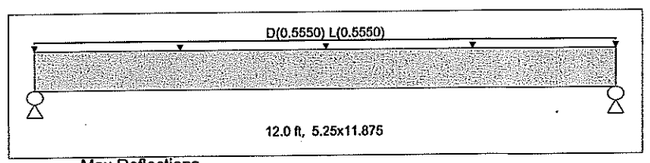
Unif Load: D = 0.5550, L = 0.5550 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.676 : 1**
fb : Actual : 1,943.13 psi at 6.000 ft in Span # 1
Fb : Allowable : 2,875.37 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.464 : 1**
fv : Actual : 134.60 psi at 11.040 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	3.33	3.33					
Right Support	3.33	3.33					



Max Deflections

Downward L+Lr+S	0.178 in	Downward Total	0.355 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	810 >360	Total Defl Ratio	405 >180

MURPHY BURR CURRY, INC.
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Project Title:
Engineer:
Project Descr:

Project ID: A52

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
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Nood Beam

Lic. # : KW-06002966

Description : 4B7

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-10

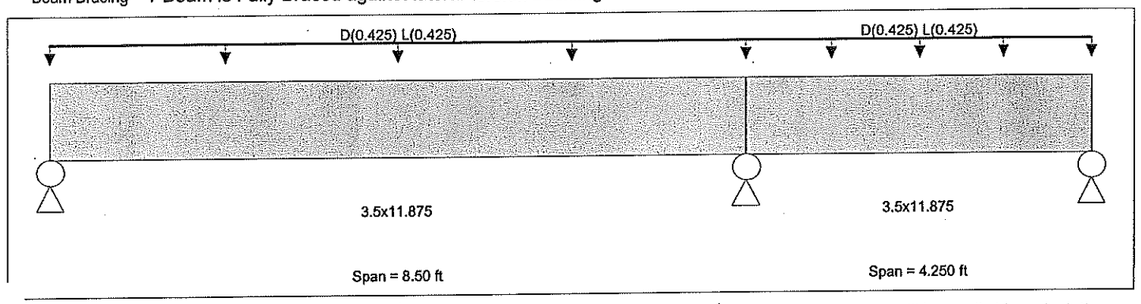
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi
Fb - Compr	2,900.0 psi
Fc - Prll	2,900.0 psi
Fc - Perp	750.0 psi
Fv	290.0 psi
Ft	2,025.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0 ksi
Eminbend - xx	1,016.54 ksi
Density	32.210 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : D = 0.4250, L = 0.4250 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.4250, L = 0.4250 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.290 : 1	Maximum Shear Stress Ratio	=	0.433 : 1
Section used for this span		3.5x11.875	Section used for this span		3.5x11.875
fb : Actual	=	839.90 psi	fv : Actual	=	125.69 psi
FB : Allowable	=	2,900.00 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	8.500 ft	Location of maximum on span	=	7.550 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.029 in	Ratio =	3545
Max Upward L+Lr+S Deflection	-0.003 in	Ratio =	17230
Max Downward Total Deflection	0.058 in	Ratio =	1772
Max Upward Total Deflection	-0.006 in	Ratio =	8615

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
+D+L+H					1.00	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.50 ft	1	0.290	0.433	1.00	1.00	1.00	1.00	1.00	1.00	1.00	5.76	839.90	2900.00	3.48	125.69	290.00
Length = 4.250 ft	2	0.290	0.433	1.00	1.00	1.00	1.00	1.00	1.00	1.00	5.76	839.90	2900.00	2.33	125.69	290.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.0575	3.846		0.0000	1.448
	2	0.0000	3.846	D+L	-0.0059	1.448

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A53

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Wood Beam

Lic. # : KW-06002966

Description : 4B7

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	2.935	7.451	0.452
D Only	1.468	3.725	0.226
L Only	1.468	3.725	0.226
D+L	2.935	7.451	0.452

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A54

Printed: 4 JAN 2018, 8:57AM

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Lic. # : KW-06002966

Wood Beam Design : 4B9

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist				Wood Grade : Parallam PSL 2.0E				
Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

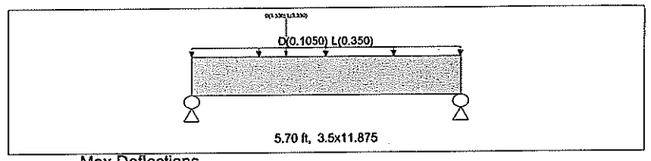
Unif Load: D = 0.1050, L = 0.350 k/ft, Trib= 1.0 ft
Point: D = 3.330, L = 3.330 k @ 2.0 ft

Design Summary

Max fb/Fb Ratio = **0.523** : 1
fb : Actual : 1,503.48 psi at 1.995 ft in Span # 1
Fb : Allowable : 2,873.41 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.643** : 1
fv : Actual : 186.60 psi at 0.000 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.46	3.16					
Right Support	1.47	2.17					



Max Deflections

Downward L+Lr+S	0.029 in	Downward Total	0.052 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2376 >360	Total Defl Ratio	1325 >180

Wood Beam Design : 4B10

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **5.25x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist				Wood Grade : Parallam PSL 2.0E				
Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

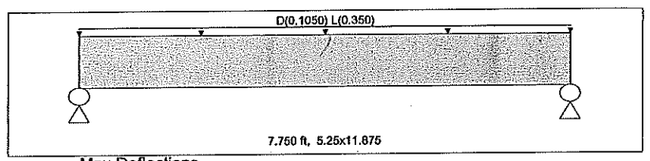
Unif Load: D = 0.1050, L = 0.350 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.115** : 1
fb : Actual : 332.22 psi at 3.875 ft in Span # 1
Fb : Allowable : 2,884.90 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.109** : 1
fv : Actual : 31.67 psi at 6.768 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.41	1.36					
Right Support	0.41	1.36					



Max Deflections

Downward L+Lr+S	0.019 in	Downward Total	0.025 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	4771 >360	Total Defl Ratio	3670 >180

Wood Beam Design : 4B11

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist				Wood Grade : Parallam PSL 2.0E				
Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

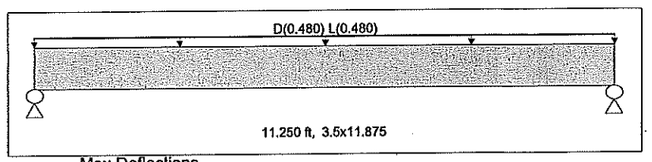
Unif Load: D = 0.480, L = 0.480 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.781** : 1
fb : Actual : 2,215.56 psi at 5.625 ft in Span # 1
Fb : Allowable : 2,837.89 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.556** : 1
fv : Actual : 161.11 psi at 10.275 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.70	2.70					
Right Support	2.70	2.70					



Max Deflections

Downward L+Lr+S	0.178 in	Downward Total	0.356 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	758 >360	Total Defl Ratio	379 >180

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A55

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Steel Beam

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Lic. # : KW-0602966

Description : 4B12

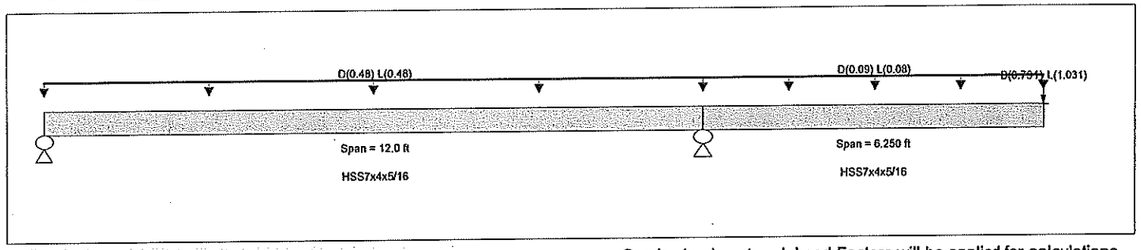
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 46.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : D = 0.480, L = 0.480 k/ft, Tributary Width = 1.0 ft
Load(s) for Span Number 2
Point Load : D = 0.7910, L = 1.031 k @ 6.250 ft
Uniform Load : D = 0.090, L = 0.080 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.489 : 1	Maximum Shear Stress Ratio =	0.119 : 1
Section used for this span	HSS7x4x5/16	Section used for this span	HSS7x4x5/16
Ma : Applied	14.708 k-ft	Va : Applied	6.986 k
Mn / Omega : Allowable	30.070 k-ft	Vn/Omega : Allowable	58.934 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	12.000ft	Location of maximum on span	12.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.135 in	Ratio =	1,110
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.214 in	Ratio =	672
Max Upward Total Deflection	-0.008 in	Ratio =	18044

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 12.00 ft	1	0.489	0.119	10.71	-14.71	14.71	50.22	30.07	1.68	1.00	6.99	98.42	58.93
Dsgn. L = 6.25 ft	2	0.489	0.049		-14.71	14.71	50.22	30.07	1.00	1.00	2.88	98.42	58.93

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.2144	5.354		0.0000	1.346
D+L	2	0.1883	6.250	D Only	-0.0083	1.346

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	4.534	9.870	
D Only	2.322	4.792	
L Only	2.213	5.078	
D+L	4.534	9.870	

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A56

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Lic. # : KW-06002966

Wood Beam Design : 4B13

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

Unif Load: D = 0.50, L = 0.50 k/ft, Trib= 1.0 ft

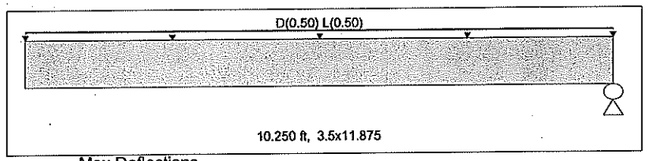
Design Summary

Max fb/Fb Ratio = 0.000 : 1
fb : Actual : 0.00 psi at 0.000 ft in Span # 1
Fb : Allowable : 0.00 psi
Load Comb :

Max fv/FvRatio = 0.000 : 1
fv : Actual : 0.00 psi at 0.000 ft in Span # 1
Fv : Allowable : 0.00 psi
Load Comb :

Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support							
Right Support							



Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.000 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	0 <180

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A57

Printed: 8 JAN 2018, 12:06PM

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Lic. # : KW-06002966

Description : 4th Level Framing

Wood Beam Design : 4th Level Cont.

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **4x12, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species : AC/AC
Wood Grade : 20F - V12

Fb - Tension	2,000.0 psi	Fc - Prll	1,500.0 psi	Fv	265.0 psi	Ebend- xx	1,500.0 ksi	Density	29.640 pcf
Fb - Compr	1,400.0 psi	Fc - Perp	560.0 psi	Ft	900.0 psi	Eminbend - xx	780.0 ksi		

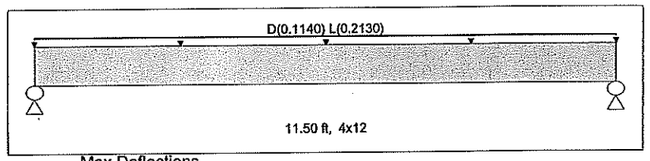
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.1140, L = 0.2130 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.455 : 1**
fb : Actual : 900.42 psi at 5.750 ft in Span # 1
Fb : Allowable : 1,977.20 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.233 : 1**
fv : Actual : 61.66 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.70	1.22					
Right Support	0.70	1.22					

Max Deflections

Downward L+Lr+S	0.135 in	Downward Total	0.213 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1020 >360	Total Defl Ratio	648 >180

Steel Beam Design : 4B15

Calculations per AISC 360-10, IBC 20012, CBC 2013, ASCE 7-10

STEEL Section : **HSS7x4x5/16, Fully Unbraced**
Using Allowable Strength Design with ASCE 7-10 Load Combinations, Major Axis Bending

Fy = 46.0 ksi E = 29,000.0 ksi

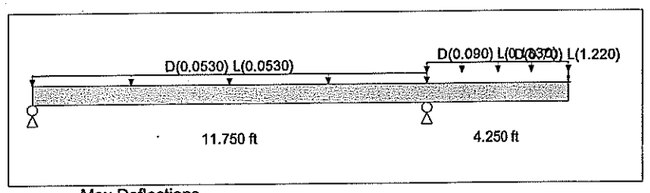
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.0530, L = 0.0530 k/ft, 0.0 ft to 11.750 ft, Trib= 1.0 ft
Unif Load: D = 0.090, L = 0.1330 k/ft, 11.750 to 16.0 ft, Trib= 1.0 ft
Point: D = 0.70, L = 1.220 k @ 16.0 ft

Design Summary

Max fb/Fb Ratio = **0.345 : 1**
Mu : Applied 10.365 k-ft at 11.750 ft in Span # 1
Mn / Omega : Allow 30.070 k-ft
Load Comb : +D+L+H, LL Comb Run (*L)

Max fv/FvRatio = **0.050 : 1**
Vu : Applied 2.958 k at 11.750 ft in Span # 1
Vn / Omega : Allow 58.934 k
Load Comb : +D+L+H, LL Comb Run (*L)



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.10	-0.54					
Right Support	1.95	2.64					

Max Deflections

Downward L+Lr+S	0.233 in	Downward Total	0.343 in
Upward L+Lr+S	-0.093 in	Upward Total	-0.122 in
Live Load Defl Ratio	436	Total Defl Ratio	296

Steel Beam

Lic. # : KW-06002966

Description : 2B1 and 2B2

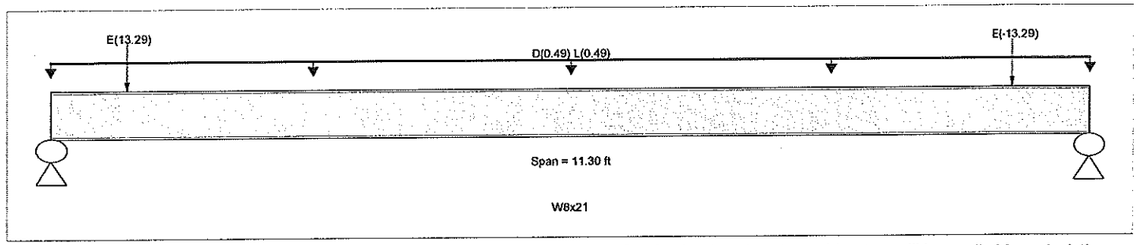
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.490, L = 0.490 k/ft, Tributary Width = 1.0 ft
Point Load : E = 13.290 k @ 0.830 ft
Point Load : E = -13.290 k @ 10.470 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.374 : 1	Maximum Shear Stress Ratio =	0.559 : 1
Section used for this span	W8x21	Section used for this span	W8x21
Ma : Applied	19.011 k-ft	Va : Applied	23.161 k
Mn / Omega : Allowable	50.898 k-ft	Vn/Omega : Allowable	41.40 k
Load Combination	+1.150D+1.750E	Load Combination	+1.150D+1.750E
Location of maximum on span	0.848ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.083 in	Ratio =	1,634
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.172 in	Ratio =	787
Max Upward Total Deflection	-0.018 in	Ratio =	7673

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 11.30 ft	1	0.368	0.137	15.98		15.98	72.45	43.38	1.14	1.00	5.66	62.10	41.40
+1.150D+1.750E													
Dsgn. L = 11.30 ft	1	0.374	0.559	19.01	-13.81	19.01	85.00	50.90	1.66	1.00	23.16	62.10	41.40
+1.150D-1.750E													
Dsgn. L = 11.30 ft	1	0.374	0.559	19.01	-13.81	19.01	85.00	50.90	1.66	1.00	23.16	62.10	41.40
+1.110D+0.750L+1.313E													
Dsgn. L = 11.30 ft	1	0.362	0.487	18.43	-8.17	18.43	85.00	50.90	1.33	1.00	20.17	62.10	41.40
+1.110D+0.750L-1.313E													
Dsgn. L = 11.30 ft	1	0.362	0.487	18.43	-8.17	18.43	85.00	50.90	1.33	1.00	20.17	62.10	41.40
+0.4550D+1.750E													
Dsgn. L = 11.30 ft	1	0.343	0.511	17.44	-15.38	17.44	85.00	50.90	1.87	1.00	21.15	62.10	41.40
+0.4550D-1.750E													
Dsgn. L = 11.30 ft	1	0.343	0.511	17.44	-15.38	17.44	85.00	50.90	1.87	1.00	21.15	62.10	41.40

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+E	1	0.1724	5.085		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	16.993	16.993

MURPHY BURR CURRY INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A59

Printed: 20 SEP 2018, 9:41AM

Steel Beam

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B1 and 2B2

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
D Only	2.887	2.887
L Only	2.769	2.769
E Only	11.338	-11.338
-E Only	-11.338	11.338
D+L	5.655	5.655
D+E	14.225	-8.451
D-E	-8.451	14.225
D+L+E	16.993	-5.682
D+L-E	-5.682	16.993

MURPHY BURR CURRY INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A60

Printed: 20 SEP 2016, 9:45AM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B3

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

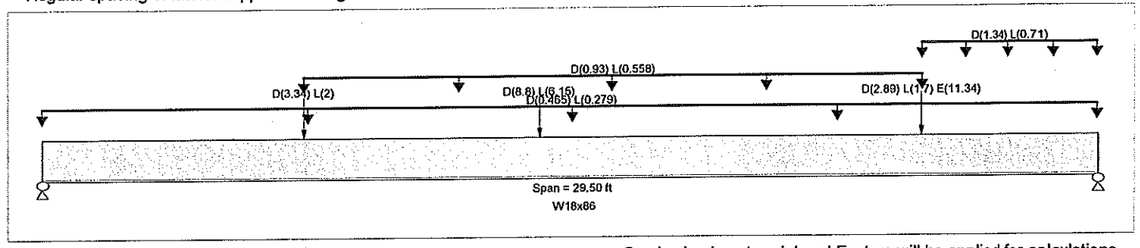
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 14.750 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 14.750 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.4650, L = 0.2790 k/ft, Tributary Width = 1.0 ft
Point Load : D = 3.340, L = 2.0 k @ 7.250 ft
Point Load : D = 8.80, L = 6.150 k @ 13.830 ft, (R4B1)
Point Load : D = 2.890, L = 1.70, E = 11.340 k @ 24.50 ft, (R2B1)
Uniform Load : D = 0.930, L = 0.5580 k/ft, Extent = 7.250 –>> 24.50 ft, Tributary Width = 1.0 ft
Uniform Load : D = 1.340, L = 0.710 k/ft, Extent = 24.50 –>> 29.50 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.863 : 1	Maximum Shear Stress Ratio =	0.334 : 1
Section used for this span	W18x86	Section used for this span	W18x86
Ma : Applied	400.316 k-ft	Va : Applied	58.931 k
Mn / Omega : Allowable	464.072 k-ft	Vn/Omega : Allowable	176.640 k
Load Combination	+1.110D+0.750L+1.313E	Load Combination	+1.110D+0.750L+1.313E
Location of maximum on span	13.865ft	Location of maximum on span	29.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.472 in	Ratio =	749
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	1.383 in	Ratio =	256
Max Upward Total Deflection	-0.119 in	Ratio =	2980

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 14.75 ft	1	0.807	0.214	374.72		374.72	775.00	464.07	1.39	1.00	37.71	264.96	176.64
Dsgn. L = 14.75 ft	1	0.798	0.269	370.34		370.34	775.00	464.07	1.34	1.00	47.57	264.96	176.64
+1.150D+1.750E													
Dsgn. L = 14.75 ft	1	0.681	0.174	316.11		316.11	775.00	464.07	1.42	1.00	30.72	264.96	176.64
Dsgn. L = 14.75 ft	1	0.681	0.290	316.00		316.00	775.00	464.07	1.20	1.00	51.26	264.96	176.64
1.150D-1.750E													
Dsgn. L = 14.75 ft	1	0.480	0.136	222.60		222.60	775.00	464.07	1.35	1.00	23.99	264.96	176.64
Dsgn. L = 14.75 ft	1	0.467	0.135	216.78		216.78	775.00	464.07	1.59	1.00	23.92	264.96	176.64
+1.110D+0.750L+1.313E													
Dsgn. L = 14.75 ft	1	0.863	0.223	400.32		400.32	775.00	464.07	1.41	1.00	39.37	264.96	176.64
Dsgn. L = 14.75 ft	1	0.858	0.334	398.37		398.37	775.00	464.07	1.26	1.00	58.93	264.96	176.64
+1.110D+0.750L-1.313E													
Dsgn. L = 14.75 ft	1	0.712	0.194	330.34		330.34	775.00	464.07	1.37	1.00	34.33	264.96	176.64
Dsgn. L = 14.75 ft	1	0.698	0.194	323.92		323.92	775.00	464.07	1.46	1.00	34.20	264.96	176.64
+0.4550D+1.750E													

MURPHY BURR CURRY INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A61

Printed: 20 SEP 2016, 9:45AM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Beam

Lic. # : KW-06002966 Licensee : MURPHY, BURR, CURRY, INC.

Description : 2B3

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Dsgn. L = 14.75 ft	1	0.334	0.080	155.01		155.01	775.00	464.07	1.46	1.00	14.19	264.96	176.64
Dsgn. L = 14.75 ft	1	0.343	0.171	157.39		157.39	765.26	458.24	1.11	1.00	30.24	264.96	176.64
+0.4550D-1.750E													
Dsgn. L = 14.75 ft	1	0.130	0.042	60.11		60.11	775.00	464.07	1.26	1.00	7.46	264.96	176.64
Dsgn. L = 14.75 ft	1	0.120	0.065	55.79	-24.19	55.79	775.00	464.07	2.40	1.00	11.50	264.96	176.64

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+E	1	1.3835	15.045		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	39.637	56.990
D Only	23.790	30.241
L Only	13.925	17.331
E Only	1.922	9.418
-E Only	-1.922	-9.418
D+L	37.715	47.572
D+E	25.712	39.659
D-E	21.868	20.823
D+L+E	39.637	56.990
D+L-E	35.793	38.154

MURPHY BURR CURRY INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A62

Printed: 20 SEP 2016, 9:48AM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B4

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

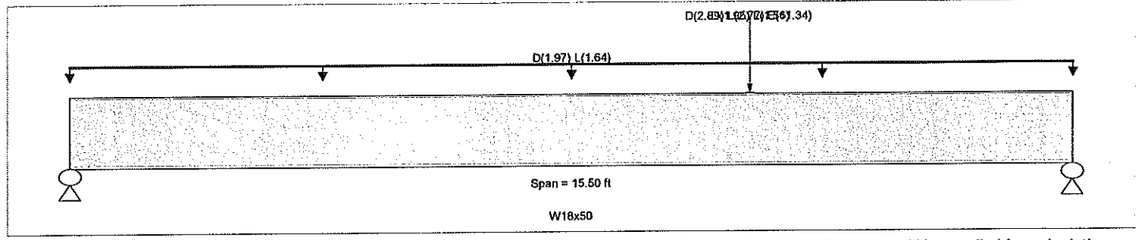
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 10.50 ft from Left-Most support
Regular spacing of lateral supports on length of beam = ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 1.970, L = 1.640 k/ft, Tributary Width = 1.0 ft
Point Load : D = 1.950, L = 1.560 k @ 10.50 ft
Point Load : D = 2.890, L = 2.770, E = 11.340 k @ 10.50 ft, (R2B1)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.868 : 1	Maximum Shear Stress Ratio =	0.335 : 1
Section used for this span	W18x50	Section used for this span	W18x50
Ma : Applied	171.330 k-ft	Va : Applied	42.836 k
Mn / Omega : Allowable	197.317 k-ft	Vn/Omega : Allowable	127.80 k
Load Combination	+1.110D+0.750L+1.313E	Load Combination	+1.110D+0.750L+1.313E
Location of maximum on span	9.920ft	Location of maximum on span	15.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.114 in	Ratio =	1,637
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.306 in	Ratio =	607
Max Upward Total Deflection	-0.056 in	Ratio =	3347

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 15.50 ft	1	0.699	0.271	134.03		134.03	320.27	191.78	1.14	1.00	34.58	191.70	127.80
+1.150D+1.750E													
Dsgn. L = 15.50 ft	1	0.719	0.276	146.98		146.98	341.30	204.37	1.22	1.00	35.22	191.70	127.80
+1.150D-1.750E													
Dsgn. L = 15.50 ft	1	0.181	0.105	38.63		38.63	355.60	212.93	1.27	1.00	13.40	191.70	127.80
+1.110D+0.750L+1.313E													
Dsgn. L = 15.50 ft	1	0.868	0.335	171.33		171.33	329.52	197.32	1.18	1.00	42.84	191.70	127.80
+1.110D+0.750L-1.313E													
Dsgn. L = 15.50 ft	1	0.464	0.195	89.19		89.19	321.11	192.28	1.15	1.00	24.89	191.70	127.80
+0.4550D+1.750E													
Dsgn. L = 15.50 ft	1	0.456	0.173	98.63		98.63	360.93	216.13	1.29	1.00	22.06	191.70	127.80
+0.4550D-1.750E													
Dsgn. L = 15.50 ft	1	0.140	0.073	1.12	-35.32	35.32	420.83	252.00	1.88	1.00	9.39	191.70	127.80

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+E	1	0.3063	8.060		0.0000	0.000

MURPHY BURR CURRY INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A63

Printed: 20 SEP 2018, 9:48AM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Beam

Description : 2B4

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	34.981	42.259
D Only	17.217	18.934
L Only	14.107	15.643
E Only	3.658	7.682
-E Only	-3.658	-7.682
D+L	31.323	34.577
D+E	20.875	26.616
D-E	13.559	11.252
D+L+E	34.981	42.259
D+L-E	27.665	26.895

Project Title:
Engineer:
Project Descr:

Project ID: A64

Printed: 25 OCT 2016, 8:50AM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee: MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. #: KW-06002966

Description : 2B5

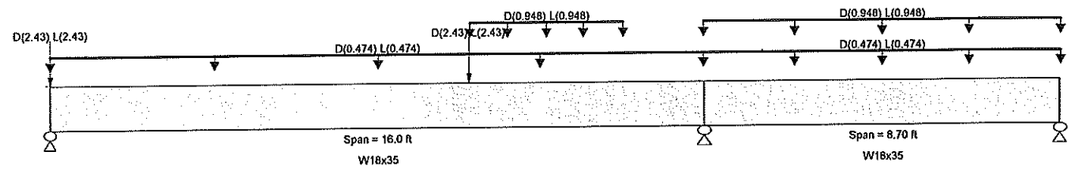
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.4740, L = 0.4740 k/ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.9480, L = 0.9480 k/ft, Extent = 10.250 –>> 14.0 ft, Tributary Width = 1.0 ft
Point Load : D = 2.430, L = 2.430 k @ 10.250 ft
Point Load : D = 2.430, L = 2.430 k @ 10.250 ft
Point Load : D = 2.430, L = 2.430 k @ 0.0 ft
Point Load : D = 2.430, L = 2.430 k @ 0.0 ft
Load for Span Number 2
Uniform Load : D = 0.4740, L = 0.4740 k/ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.9480, L = 0.9480 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.589 : 1	Maximum Shear Stress Ratio =	0.219 : 1
Section used for this span	W18x35	Section used for this span	W18x35
Ma : Applied	55.762 k-ft	Va : Applied	23.261 k
Mn / Omega : Allowable	94.751 k-ft	Vn/Omega : Allowable	106.20 k
Load Combination	+D+L+H, LL Comb Run (L*)	Load Combination	+D+L+H, LL Comb Run (LL)
Location of maximum on span	16.000ft	Location of maximum on span	16.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.068 in	Ratio =	2,836
Max Upward L+Lr+S Deflection	-0.015 in	Ratio =	7,190
Max Downward Total Deflection	0.128 in	Ratio =	1495
Max Upward Total Deflection	-0.020 in	Ratio =	5213

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H, LL Comb Run (*L)													
Dsgn. L = 16.00 ft	1	0.277	0.156	19.60	-35.41	35.41	213.57	127.88	1.95	1.00	16.59	159.30	106.20
Dsgn. L = 8.70 ft	2	0.213	0.156	12.41	-35.41	35.41	277.08	165.92	2.45	1.00	16.59	159.30	106.20
+D+L+H, LL Comb Run (L*)													
Dsgn. L = 16.00 ft	1	0.589	0.216	46.82	-55.76	55.76	158.23	94.75	1.45	1.00	22.96	159.30	106.20
Dsgn. L = 8.70 ft	2	0.336	0.120		-55.76	55.76	277.08	165.92	2.31	1.00	12.75	159.30	106.20
+D+L+H, LL Comb Run (LL)													
Dsgn. L = 16.00 ft	1	0.574	0.219	43.97	-60.50	60.50	176.06	105.42	1.61	1.00	23.26	159.30	106.20
Dsgn. L = 8.70 ft	2	0.365	0.183	5.39	-60.50	60.50	277.08	165.92	2.96	1.00	19.48	159.30	106.20

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.1284	7.877		0.0000	3.279
	2	0.0000	7.877	D+L	-0.0200	3.279

MURPHY BURR CURRY INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A65

Printed: 26 OCT 2016, 8:50AM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Beam

Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B5

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	19.314	42.739	8.454
D Only	9.623	21.661	2.813
L Only, LL Comb Run (*L)	-0.296	7.027	5.641
L Only, LL Comb Run (L*)	9.691	14.051	-2.884
L Only, LL Comb Run (LL)	9.395	21.078	2.757
D+L, LL Comb Run (*L)	9.327	28.687	8.454
D+L, LL Comb Run (L*)	19.314	35.712	-0.071
D+L, LL Comb Run (LL)	19.018	42.739	5.570

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A66

Printed: 5 JAN 2018, 11:20AM

Steel Beam

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC

Description : 2B6

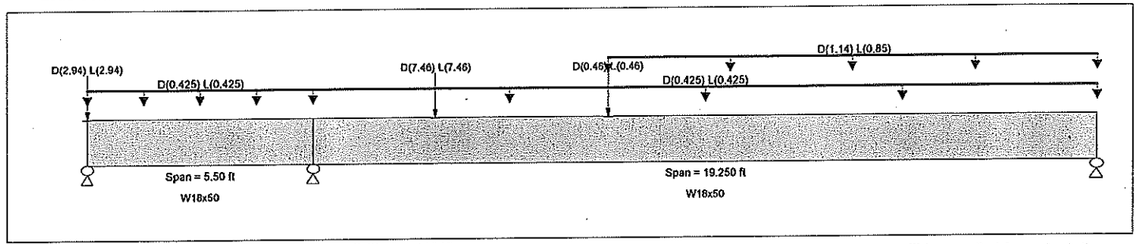
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load(s) for Span Number 1
Point Load : D = 2.940, L = 2.940 k @ 0.0 ft
Uniform Load : D = 0.4250, L = 0.4250 k/ft, Tributary Width = 1.0 ft
Load(s) for Span Number 2
Point Load : D = 7.460, L = 7.460 k @ 3.0 ft
Point Load : D = 0.460, L = 0.460 k @ 7.250 ft
Uniform Load : D = 0.4250, L = 0.4250 k/ft, Tributary Width = 1.0 ft
Uniform Load : D = 1.140, L = 0.850 k/ft, Extent = 7.250 -->> 19.250 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.428 : 1	Maximum Shear Stress Ratio =	0.273 : 1
Section used for this span	W18x50	Section used for this span	W18x50
Ma : Applied	107.798 k-ft	Va : Applied	34.874 k
Mn / Omega : Allowable	251.996 k-ft	Vn/Omega : Allowable	127.80 k
Load Combination	+D+L+H, LL Comb Run (LL)	Load Combination	+D+L+H, LL Comb Run (LL)
Location of maximum on span	5.500ft	Location of maximum on span	5.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.094 in	Ratio =	2,469
Max Upward L+Lr+S Deflection	-0.007 in	Ratio =	9,161
Max Downward Total Deflection	0.206 in	Ratio =	1121
Max Upward Total Deflection	-0.015 in	Ratio =	4322

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H, LL Comb Run (*L)													
Dsgn. L = 5.50 ft	1	0.426	0.273		-107.44	107.44	420.83	252.00	1.00	1.00	34.86	191.70	127.80
Dsgn. L = 19.25 ft	2	0.426	0.273	85.19	-107.44	107.44	420.83	252.00	1.00	1.00	34.86	191.70	127.80
+D+L+H, LL Comb Run (L*)													
Dsgn. L = 5.50 ft	1	0.232	0.144		-58.45	58.45	420.83	252.00	1.00	1.00	18.46	191.70	127.80
Dsgn. L = 19.25 ft	2	0.232	0.144	46.74	-58.45	58.45	420.83	252.00	1.00	1.00	18.46	191.70	127.80
+D+L+H, LL Comb Run (LL)													
Dsgn. L = 5.50 ft	1	0.428	0.273		-107.80	107.80	420.83	252.00	1.00	1.00	34.87	191.70	127.80
Dsgn. L = 19.25 ft	2	0.428	0.273	85.05	-107.80	107.80	420.83	252.00	1.00	1.00	34.87	191.70	127.80

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	10.662	D+L	-0.0153	3.215
D+L	2	0.2060	10.662		0.0000	3.215

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A67

Printed: 5 JAN 2018, 11:20AM

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B6

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	-15.288	56.949	22.190
D Only	-6.316	30.307	12.306
L Only, LL Comb Run (*L)	-8.972	25.390	9.884
L Only, LL Comb Run (L*)	4.044	1.252	-0.019
L Only, LL Comb Run (LL)	-4.929	26.642	9.865
D+L, LL Comb Run (*L)	-15.288	55.697	22.190
D+L, LL Comb Run (L*)	-2.272	31.559	12.288
D+L, LL Comb Run (LL)	-11.245	56.949	22.172

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A6B

Printed: 5 JAN 2018, 11:19AM

Steel Beam

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B5

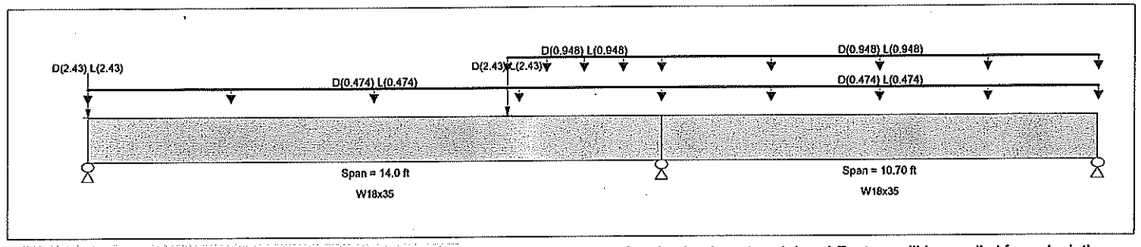
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.4740, L = 0.4740 k/ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.9480, L = 0.9480 k/ft, Extent = 10.250 –>> 14.0 ft, Tributary Width = 1.0 ft
Point Load : D = 2.430, L = 2.430 k @ 10.250 ft
Point Load : D = 2.430, L = 2.430 k @ 10.250 ft
Point Load : D = 2.430, L = 2.430 k @ 0.0 ft
Point Load : D = 2.430, L = 2.430 k @ 0.0 ft
Load for Span Number 2
Uniform Load : D = 0.4740, L = 0.4740 k/ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.9480, L = 0.9480 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.333 : 1	Maximum Shear Stress Ratio =	0.224 : 1
Section used for this span	W18x35	Section used for this span	W18x35
Ma : Applied	41.453 k-ft	Va : Applied	23.746 k
Mn / Omega : Allowable	124.334 k-ft	Vn/Omega : Allowable	106.20 k
Load Combination	+D+L+H, LL Comb Run (L*)	Load Combination	+D+L+H, LL Comb Run (LL)
Location of maximum on span	14.000ft	Location of maximum on span	14.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.038 in	Ratio =	4,423
Max Upward L+Lr+S Deflection	-0.014 in	Ratio =	9,187
Max Downward Total Deflection	0.064 in	Ratio =	2614
Max Upward Total Deflection	-0.010 in	Ratio =	12868

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H, LL Comb Run (*L)													
Dsgn. L = 14.00 ft	1	0.207	0.175	8.21	-34.30	34.30	277.08	165.92	3.00	1.00	18.61	159.30	106.20
Dsgn. L = 10.70 ft	2	0.207	0.175	25.83	-34.30	34.30	277.08	165.92	1.63	1.00	18.61	159.30	106.20
+D+L+H, LL Comb Run (L*)													
Dsgn. L = 14.00 ft	1	0.333	0.218	28.43	-41.45	41.45	207.64	124.33	1.53	1.00	23.12	159.30	106.20
Dsgn. L = 10.70 ft	2	0.250	0.110	5.28	-41.45	41.45	277.08	165.92	3.00	1.00	11.67	159.30	106.20
)+L+H, LL Comb Run (LL)													
Dsgn. L = 14.00 ft	1	0.312	0.224	23.84	-50.27	50.27	269.32	161.27	1.98	1.00	23.75	159.30	106.20
Dsgn. L = 10.70 ft	2	0.303	0.189	19.90	-50.27	50.27	277.08	165.92	2.35	1.00	20.10	159.30	106.20

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.0643	7.000		0.0000	1.646
D+L	2	0.0294	6.008	D+L	-0.0088	1.646

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A69

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B5

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	17.196	43.847	12.197
D Only	8.381	22.198	5.413
L Only, LL Comb Run (*L)	-0.630	9.061	6.784
L Only, LL Comb Run (L*)	8.815	12.588	-1.492
L Only, LL Comb Run (LL)	8.186	21.649	5.292
D+L, LL Comb Run (*L)	7.751	31.259	12.197
D+L, LL Comb Run (L*)	17.196	34.786	3.921
D+L, LL Comb Run (LL)	16.567	43.847	10.705

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A10

Printed: 5 JAN 2018, 11:18AM

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Beam

Lic. # : KW-06002966 — Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B7+

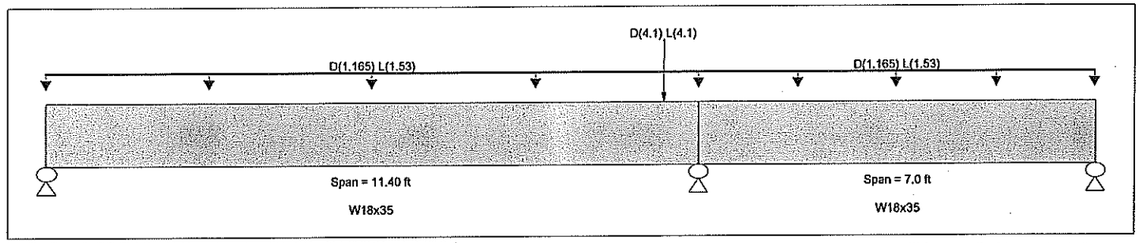
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 1.165, L = 1.530 k/ft, Tributary Width = 1.0 ft
Point Load : D = 4.10, L = 4.10 k @ 10.80 ft
Load for Span Number 2
Uniform Load : D = 1.165, L = 1.530 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.227 : 1	Maximum Shear Stress Ratio =	0.250 : 1
Section used for this span	W18x35	Section used for this span	W18x35
Ma : Applied	36.651 k-ft	Va : Applied	26.545 k
Mn / Omega : Allowable	161.496 k-ft	Vn/Omega : Allowable	106.20 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	11.400ft	Location of maximum on span	11.400 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.023 in	Ratio =	5,979
Max Upward L+Lr+S Deflection	-0.002 in	Ratio =	33,740
Max Downward Total Deflection	0.041 in	Ratio =	3334
Max Upward Total Deflection	-0.005 in	Ratio =	18487

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 11.40 ft	1	0.227	0.250	29.90	-36.65	36.65	269.70	161.50	1.49	1.00	26.54	159.30	106.20
Dsgn. L = 7.00 ft	2	0.221	0.139	3.42	-36.65	36.65	277.08	165.92	2.98	1.00	14.79	159.30	106.20

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.0410	5.262		0.0000	1.938
	2	0.0000	5.262	D+L	-0.0045	1.938

Vertical Reactions - Unfactored

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	12.778	41.336	4.319
D Only	5.628	18.679	1.874
L Only	7.150	22.657	2.445
D+L	12.778	41.336	4.319

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A11

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B7-

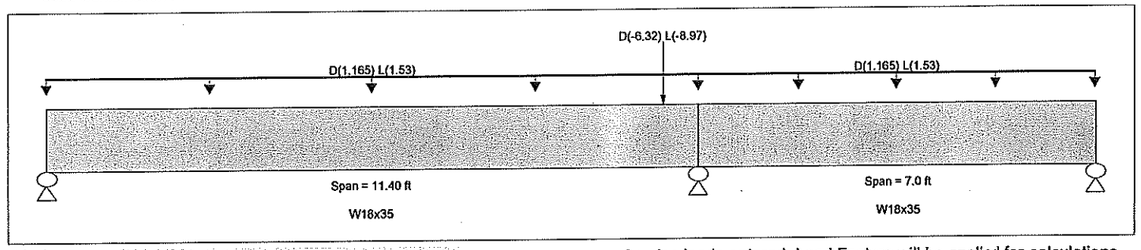
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 1.165, L = 1.530 k/ft, Tributary Width = 1.0 ft
Point Load : D = -6.320, L = -8.970 k @ 10.80 ft
Load for Span Number 2
Uniform Load : D = 1.165, L = 1.530 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.190 : 1	Maximum Shear Stress Ratio =	0.162 : 1
Section used for this span	W18x35	Section used for this span	W18x35
Ma : Applied	28.596 k-ft	Va : Applied	17.199 k
Mn / Omega : Allowable	150.643 k-ft	Vn/Omega : Allowable	106.20 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	11.400ft	Location of maximum on span	10.786 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.020 in	Ratio =	6,898
Max Upward L+Lr+S Deflection	-0.001 in	Ratio =	76,652
Max Downward Total Deflection	0.036 in	Ratio =	3848
Max Upward Total Deflection	-0.002 in	Ratio =	41506

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 11.40 ft	1	0.190	0.162	27.47	-28.60	28.60	251.57	150.64	1.39	1.00	17.20	159.30	106.20
Dsgn. L = 7.00 ft	2	0.172	0.128	5.48	-28.60	28.60	277.08	165.92	2.86	1.00	13.64	159.30	106.20

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.0356	5.086		0.0000	1.400
D+L	2	0.0006	5.331	D+L	-0.0020	1.400

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	12.248	17.225	5.470
D Only	5.393	7.983	2.385
L Only	6.855	9.242	3.085
D+L	12.248	17.225	5.470

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A12

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B8

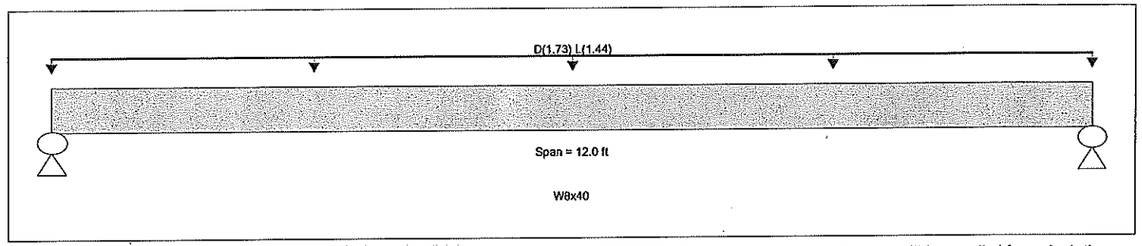
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 1.730, L = 1.440 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.582 : 1	Maximum Shear Stress Ratio =	0.324 : 1
Section used for this span	W8x40	Section used for this span	W8x40
Ma : Applied	57.777 k-ft	Va : Applied	19.259 k
Mn / Omega : Allowable	99.301 k-ft	Vn/Omega : Allowable	59.40 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	6.000ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.160 in	Ratio =	900
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.357 in	Ratio =	404
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 12.00 ft	1	0.582	0.324	57.78		57.78	165.83	99.30	1.14	1.00	19.26	89.10	59.40

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.3565	6.060		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	19.259	19.259
D Only	10.619	10.619
L Only	8.640	8.640
D+L	19.259	19.259

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A73

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC

Steel Beam

Lic. # : KW-06002966

Description : 2B9

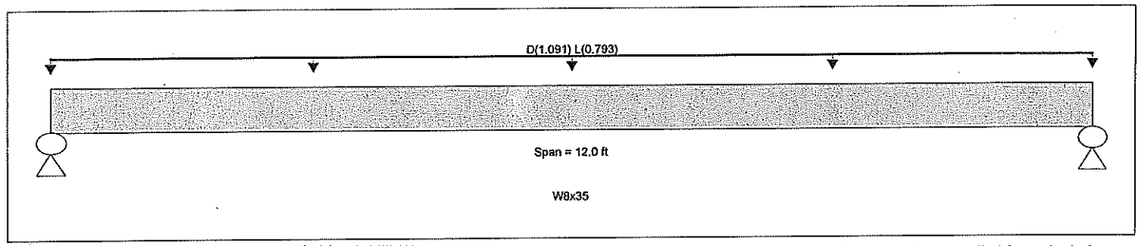
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 1.091, L = 0.7930 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.399 : 1	Maximum Shear Stress Ratio =	0.229 : 1
Section used for this span	W8x35	Section used for this span	W8x35
Ma : Applied	34.543 k-ft	Va : Applied	11.514 k
Mn / Omega : Allowable	86.577 k-ft	Vn/Omega : Allowable	50.344 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	6.000ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.101 in	Ratio =	1,422
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.245 in	Ratio =	588
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 12.00 ft	1	0.399	0.229	34.54		34.54	144.58	86.58	1.14	1.00	11.51	75.52	50.34

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.2451	6.060		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	11.514	11.514
D Only	6.756	6.756
L Only	4.758	4.758
D+L	11.514	11.514

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A74

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Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B10

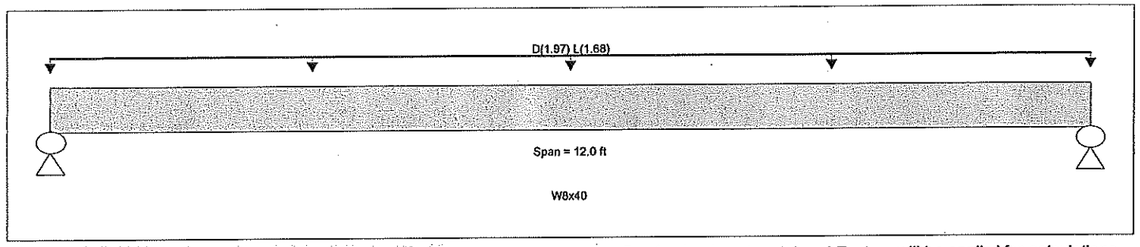
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 1.970, L = 1.680 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.669 : 1	Maximum Shear Stress Ratio =	0.373 : 1
Section used for this span	W8x40	Section used for this span	W8x40
Ma : Applied	66.417 k-ft	Va : Applied	22.139 k
Mn / Omega : Allowable	99.301 k-ft	Vn/Omega : Allowable	59.40 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	6.000ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.187 in	Ratio =	771
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.410 in	Ratio =	351
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 12.00 ft	1	0.669	0.373	66.42		66.42	165.83	99.30	1.14	1.00	22.14	89.10	59.40

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.4098	6.060		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	22.139	22.139
D Only	12.059	12.059
L Only	10.080	10.080
D+L	22.139	22.139

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A75

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
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Steel Beam

Lic. # : KW-06002966

Description : 2B12 and 2B13

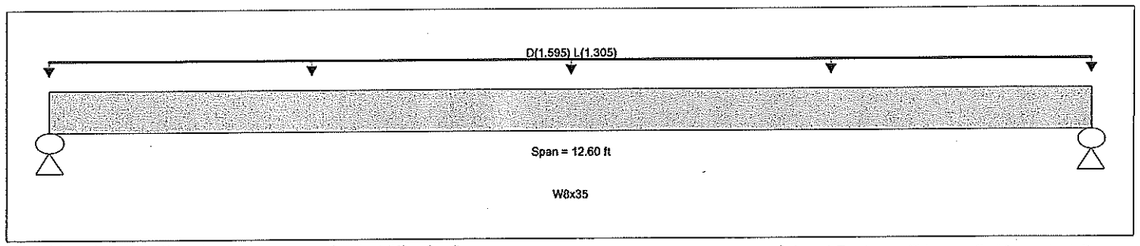
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 1.595, L = 1.305 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.673 : 1	Maximum Shear Stress Ratio =	0.367 : 1
Section used for this span	W8x35	Section used for this span	W8x35
Ma : Applied	58.246 k-ft	Va : Applied	18.491 k
Mn / Omega : Allowable	86.577 k-ft	Vn/Omega : Allowable	50.344 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	6.300 ft	Location of maximum on span	12.600 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.203 in	Ratio =	746
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.456 in	Ratio =	332
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 12.60 ft	1	0.673	0.367	58.25		58.25	144.58	86.58	1.14	1.00	18.49	75.52	50.34

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.4556	6.363		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	18.491	18.491
D Only	10.269	10.269
L Only	8.222	8.222
D+L	18.491	18.491

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A76

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B14

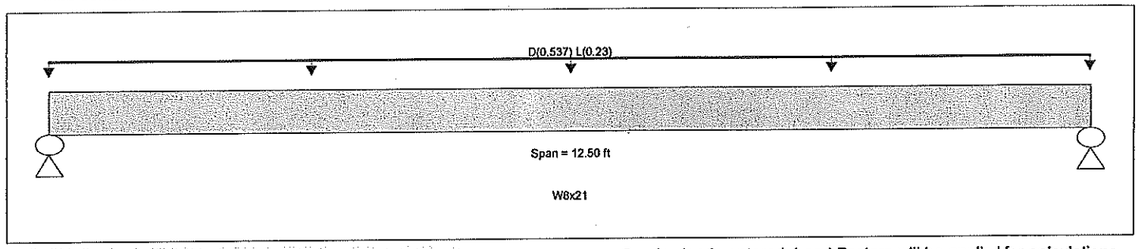
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.5370, L = 0.230 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.377 : 1	Maximum Shear Stress Ratio =	0.119 : 1
Section used for this span	W8x21	Section used for this span	W8x21
Ma : Applied	15.390 k-ft	Va : Applied	4.925 k
Mn / Omega : Allowable	40.856 k-ft	Vn/Omega : Allowable	41.40 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	6.250ft	Location of maximum on span	12.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.058 in	Ratio =	2,572
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.200 in	Ratio =	751
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 12.50 ft	1	0.377	0.119	15.39		15.39	68.23	40.86	1.14	1.00	4.92	62.10	41.40

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.1998	6.313		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.925	4.925
D Only	3.487	3.487
L Only	1.438	1.438
D+L	4.925	4.925

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A77

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B15

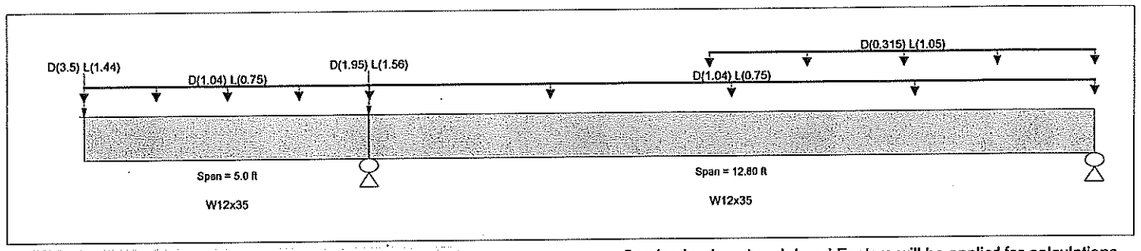
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : D = 1.040, L = 0.750 k/ft, Tributary Width = 1.0 ft
Point Load : D = 3.50, L = 1.440 k @ 0.0 ft
Point Load : D = 1.950, L = 1.560 k @ 5.0 ft
Load for Span Number 2
Uniform Load : D = 1.040, L = 0.750 k/ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.3150, L = 1.050 k/ft, Extent = 6.0 –>> 12.80 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.369 : 1	Maximum Shear Stress Ratio =	0.235 : 1
Section used for this span	W12x35	Section used for this span	W12x35
Ma : Applied	47.075 k-ft	Va : Applied	17.599 k
Mn / Omega : Allowable	127.745 k-ft	Vn/Omega : Allowable	75.0 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	5.000ft	Location of maximum on span	5.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.063 in	Ratio =	2,436
Max Upward L+Lr+S Deflection	-0.016 in	Ratio =	7,430
Max Downward Total Deflection	0.075 in	Ratio =	1604
Max Upward Total Deflection	-0.004 in	Ratio =	30640

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 5.00 ft	1	0.369	0.235		-47.08	47.08	213.33	127.74	1.00	1.00	17.60	112.50	75.00
Dsgn. L = 12.80 ft	2	0.369	0.235	33.76	-47.08	47.08	213.33	127.74	1.80	1.00	17.60	112.50	75.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.0748	0.000	L Only	-0.0039	4.462
D+L	2	0.0886	7.483		0.0000	4.462

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum		34.999	14.595
D Only		20.258	5.846
L Only		14.741	8.749
D+L		34.999	14.595

MURPHY BURR CURRY, INC
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A16

Printed: 5 JAN 2018, 11:22AM

Steel Beam

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC

Description : 2B16

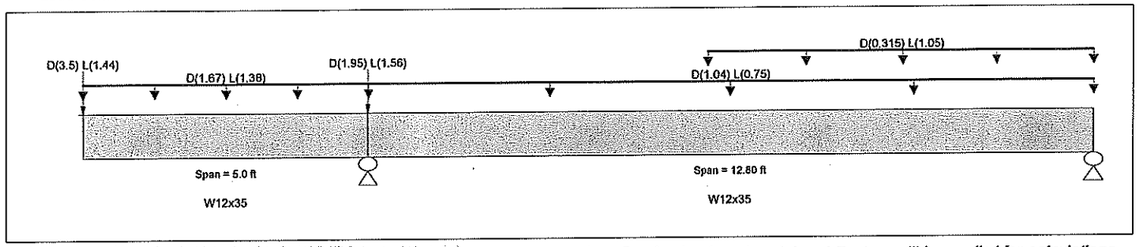
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : D = 1.670, L = 1.380 k/ft, Tributary Width = 1.0 ft
Point Load : D = 3.50, L = 1.440 k @ 0.0 ft
Point Load : D = 1.950, L = 1.560 k @ 5.0 ft
Load for Span Number 2
Uniform Load : D = 1.040, L = 0.750 k/ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.3150, L = 1.050 k/ft, Extent = 6.0 -->> 12.80 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.492 : 1	Maximum Shear Stress Ratio =	0.269 : 1
Section used for this span	W12x35	Section used for this span	W12x35
Ma : Applied	62.825 k-ft	Va : Applied	20.190 k
Mn / Omega : Allowable	127.745 k-ft	Vn/Omega : Allowable	75.0 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	5.000ft	Location of maximum on span	5.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.047 in	Ratio =	3,270
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.150 in	Ratio =	802
Max Upward Total Deflection	-0.012 in	Ratio =	12611

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 5.00 ft	1	0.492	0.269		-62.83	62.83	213.33	127.74	1.00	1.00	20.19	112.50	75.00
Dsgn. L = 12.80 ft	2	0.492	0.251	28.30	-62.83	62.83	213.33	127.74	2.19	1.00	18.83	112.50	75.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.1496	0.000		0.0000	2.166
D+L	2	0.0579	8.074	D Only	-0.0122	2.166

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum		42.530	13.364
D Only		24.023	5.231
L Only		18.507	8.133
D+L		42.530	13.364

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A79

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B17

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

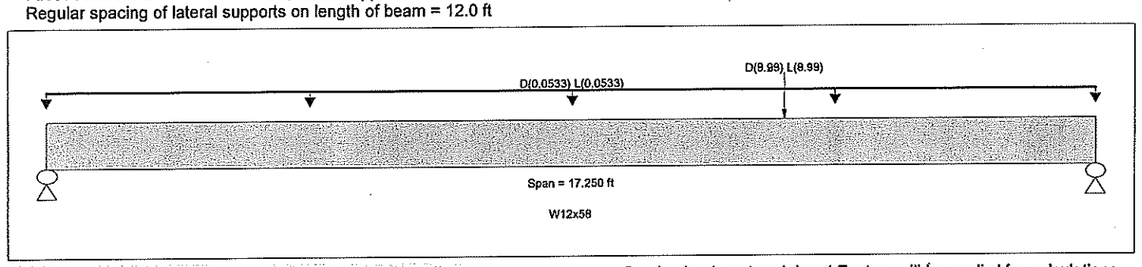
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 12.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 12.0 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.05330, L = 0.05330 k/ft, Tributary Width = 1.0 ft
Point Load : D = 5.230, L = 8.130 k @ 12.10 ft
Point Load : D = 9.990, L = 9.990 k @ 12.10 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.581 : 1	Maximum Shear Stress Ratio =	0.282 : 1
Section used for this span	W12x58	Section used for this span	W12x58
Ma : Applied	125.329 k-ft	Va : Applied	24.805 k
Mn / Omega : Allowable	215.569 k-ft	Vn/Omega : Allowable	87.840 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	12.075ft	Location of maximum on span	17.250 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.203 in	Ratio =	1,018
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.384 in	Ratio =	540
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 11.99 ft	1	0.578	0.129	124.52		124.52	360.00	215.57	1.63	1.00	11.37	131.76	87.84
Dsgn. L = 5.26 ft	1	0.581	0.282	125.33		125.33	360.00	215.57	1.62	1.00	24.80	131.76	87.84

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.3836	9.488		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	11.372	24.805
D Only	5.503	11.635
L Only	5.869	13.170
D+L	11.372	24.805

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A80

Printed: 5 JAN 2018, 11:22AM

Steel Beam

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B18

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

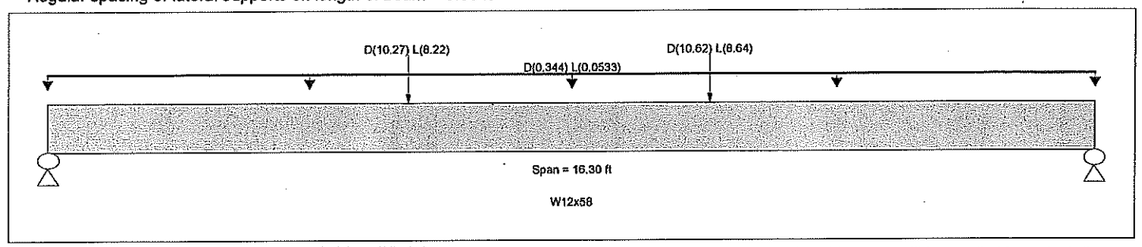
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 5.60 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 5.60 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.3440, L = 0.05330 k/ft, Tributary Width = 1.0 ft
Point Load : D = 10.270, L = 8.220 k @ 5.60 ft
Point Load : D = 10.620, L = 8.640 k @ 10.30 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.581 : 1	Maximum Shear Stress Ratio =	0.261 : 1
Section used for this span	W12x58	Section used for this span	W12x58
Ma : Applied	125.266 k-ft	Va : Applied	22.937 k
Mn / Omega : Allowable	215.569 k-ft	Vn/Omega : Allowable	87.840 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	9.780ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.177 in	Ratio =	1,106
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.435 in	Ratio =	450
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 5.54 ft	1	0.557	0.261	120.13		120.13	360.00	215.57	1.64	1.00	22.94	131.76	87.84
Dsgn. L = 5.62 ft	1	0.581	0.232	125.27	108.15	125.27	360.00	215.57	1.01	1.00	20.41	131.76	87.84
Dsgn. L = 5.13 ft	1	0.502	0.253	108.15		108.15	360.00	215.57	1.62	1.00	22.23	131.76	87.84

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.4348	8.150		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	22.937	22.232
D Only	13.926	13.514
L Only	9.011	8.718
D+L	22.937	22.232

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A81

Printed: 5 JAN 2018, 11:22AM

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B19

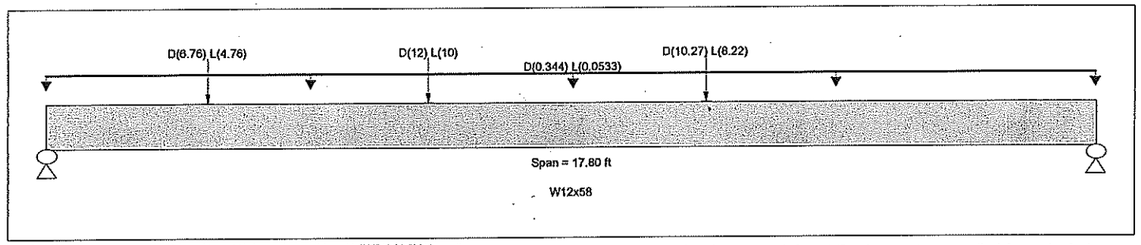
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.3440, L = 0.05330 k/ft, Tributary Width = 1.0 ft
Point Load : D = 6.760, L = 4.760 k @ 2.70 ft
Point Load : D = 12.0, L = 10.0 k @ 6.450 ft
Point Load : D = 10.270, L = 8.220 k @ 11.150 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.795 : 1	Maximum Shear Stress Ratio =	0.396 : 1
Section used for this span	W12x58	Section used for this span	W12x58
Ma : Applied	171.450 k-ft	Va : Applied	34.759 k
Mn / Omega : Allowable	215.569 k-ft	Vn/Omega : Allowable	87.840 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	6.497 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.285 in	Ratio =	749
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.695 in	Ratio =	307
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 17.80 ft	1	0.795	0.396	171.45		171.45	360.00	215.57	1.19	1.00	34.76	131.76	87.84

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.6949	8.722		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	34.759	25.353
D Only	20.800	15.384
L Only	13.960	9.969
D+L	34.759	25.353

MURPHY · BURR · CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A82

Printed: 20 SEP 2016, 11:54AM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B20

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

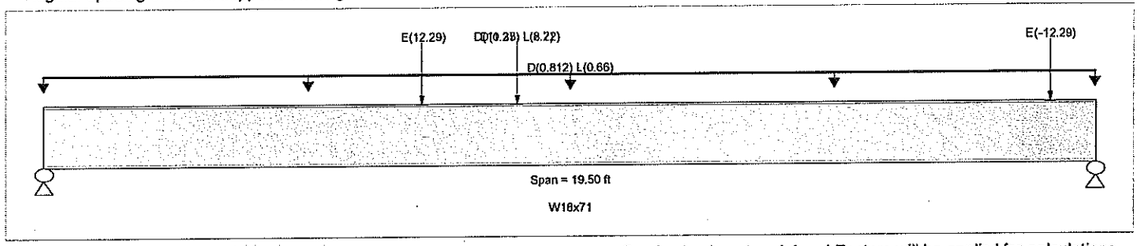
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending
Load Combination ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 10.0 ft from Left-Most support
Regular spacing of lateral supports on length of beam = ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.8120, L = 0.660 k/ft, Tributary Width = 1.0 ft
Point Load : D = 1.350, L = 2.70 k @ 8.750 ft
Point Load : D = 10.270, L = 8.220 k @ 8.750 ft
Point Load : E = 12.290 k @ 7.0 ft
Point Load : E = -12.290 k @ 18.670 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.764 : 1	Maximum Shear Stress Ratio =	0.186 : 1
Section used for this span	W18x71	Section used for this span	W18x71
Ma : Applied	216.269 k-ft	Va : Applied	34.012 k
Mn / Omega : Allowable	283.015 k-ft	Vn/Omega : Allowable	183.150 k
Load Combination	+D+0.750L+1.313E	Load Combination	+D+0.750L+1.313E
Location of maximum on span	8.775ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.149 in	Ratio =	1,569
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.400 in	Ratio =	585
Max Upward Total Deflection	-0.076 in	Ratio =	3091

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 19.50 ft	1	0.646	0.150	181.07		181.07	467.80	280.12	1.26	1.00	27.47	274.73	183.15
+1.150D+1.750E													
Dsgn. L = 19.50 ft	1	0.658	0.165	187.22		187.22	474.87	284.35	1.28	1.00	30.14	274.73	183.15
+1.150D-1.750E													
Dsgn. L = 19.50 ft	1	0.152	0.157	43.80		43.80	482.31	288.81	1.30	1.00	28.77	274.73	183.15
+D+0.750L+1.313E													
Dsgn. L = 19.50 ft	1	0.764	0.186	216.27		216.27	472.63	283.01	1.27	1.00	34.01	274.73	183.15
D+0.750L-1.313E													
Dsgn. L = 19.50 ft	1	0.380	0.175	104.07		104.07	457.38	273.88	1.23	1.00	31.98	274.73	183.15
+0.4550D+1.750E													
Dsgn. L = 19.50 ft	1	0.423	0.108	127.98	-5.26	127.98	505.38	302.63	1.36	1.00	19.70	274.73	183.15
+0.4550D-1.750E													
Dsgn. L = 19.50 ft	1	0.142	0.105	15.64	-51.87	51.87	608.33	364.27	1.94	1.00	19.16	274.73	183.15

Steel Beam

Lic. # : KW-06002966

Description : 2B20

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+E	1	0.4003	9.458		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	34.823	32.512
D Only	15.013	13.821
L Only	12.455	11.335
E Only	7.355	-7.355
-E Only	-7.355	7.355
D+L	27.468	25.156
D+E	22.368	6.466
D-E	7.658	21.177
D+L+E	34.823	17.801
D+L-E	20.113	32.512

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A84

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Beam

Lic. # : KW-06002966 — Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B21

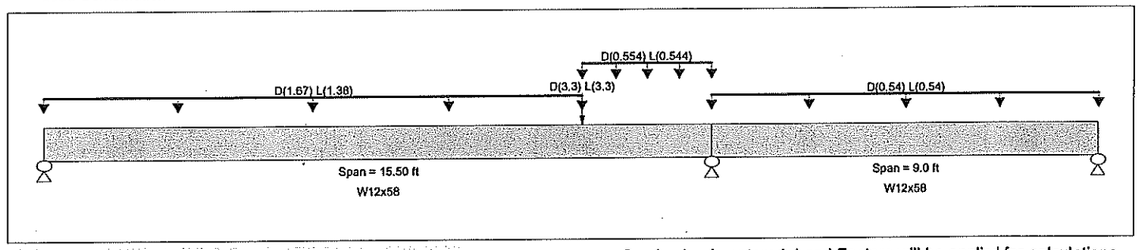
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 1.670, L = 1.380 k/ft, Extent = 0.0 –>> 12.50 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.5540, L = 0.5440 k/ft, Extent = 12.50 –>> 15.50 ft, Tributary Width = 1.0 ft
Point Load : D = 3.30, L = 3.30 k @ 12.50 ft
Point Load : D = 3.30, L = 3.30 k @ 12.50 ft
Load for Span Number 2
Uniform Load : D = 0.540, L = 0.540 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.357 : 1	Maximum Shear Stress Ratio =	0.392 : 1
Section used for this span	W12x58	Section used for this span	W12x58
Ma : Applied	76.995 k-ft	Va : Applied	34.409 k
Mn / Omega : Allowable	215.569 k-ft	Vn/Omega : Allowable	87.840 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	15.500ft	Location of maximum on span	15.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.094 in	Ratio =	1,988
Max Upward L+Lr+S Deflection	-0.018 in	Ratio =	6,030
Max Downward Total Deflection	0.206 in	Ratio =	901
Max Upward Total Deflection	-0.039 in	Ratio =	2747

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 15.50 ft	1	0.357	0.392	71.67	-76.99	76.99	360.00	215.57	1.28	1.00	34.41	131.76	87.84
Dsgn. L = 9.00 ft	2	0.357	0.156		-76.99	76.99	360.00	215.57	2.00	1.00	13.68	131.76	87.84

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.2064	7.392		0.0000	3.600
	2	0.0000	7.392	D+L	-0.0393	3.600

Vertical Reactions - Unfactored

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	21.107	48.085	-3.435
D Only	11.662	25.683	-1.930
L Only	9.445	22.401	-1.504
D+L	21.107	48.085	-3.435

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A85

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
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Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B22

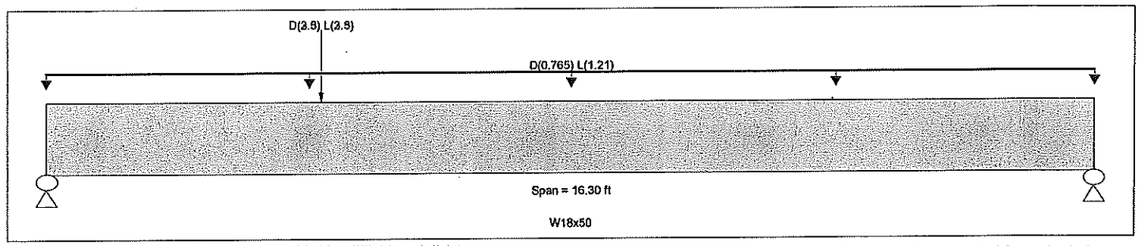
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.7650, L = 1.210 k/ft, Tributary Width = 1.0 ft
Point Load : D = 3.30, L = 3.30 k @ 4.250 ft
Point Load : D = 3.30, L = 3.30 k @ 4.250 ft
Point Load : D = 2.50, L = 2.50 k @ 4.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.610 : 1	Maximum Shear Stress Ratio =	0.234 : 1
Section used for this span	W18x50	Section used for this span	W18x50
Ma : Applied	111.489 k-ft	Va : Applied	29.959 k
Mn / Omega : Allowable	182.708 k-ft	Vn/Omega : Allowable	127.80 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	5.787 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.128 in	Ratio =	1,532
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.228 in	Ratio =	858
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 16.30 ft	1	0.610	0.234	111.49		111.49	305.12	182.71	1.14	1.00	29.96	191.70	127.80

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.2281	7.824		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	29.959	21.249
D Only	13.370	9.015
L Only	16.589	12.234
D+L	29.959	21.249

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A86

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Wood Beam

File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B23

CODE REFERENCES

Calculations per NDS 2012, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-10

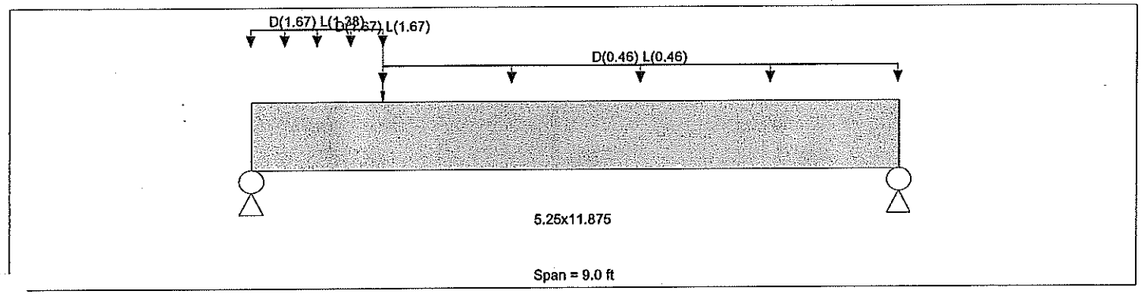
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi
Fb - Compr	2,900.0 psi
Fc - Prll	2,900.0 psi
Fc - Perp	750.0 psi
Fv	290.0 psi
Ft	2,025.0 psi

E : Modulus of Elasticity

Ebend- xx	2,000.0 ksi
Eminbend - xx	1,016.54 ksi
Density	32.210 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsion buckling



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Uniform Load : D = 0.460, L = 0.460 k/ft, Extent = 1.830 -->> 9.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 1.670, L = 1.380 k/ft, Extent = 0.0 -->> 1.830 ft, Tributary Width = 1.0 ft
Point Load : D = 1.670, L = 1.670 k @ 1.830 ft
Point Load : D = 1.670, L = 1.670 k @ 1.830 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.633 : 1	Maximum Shear Stress Ratio	=	0.826 : 1
Section used for this span		5.25x11.875	Section used for this span		5.25x11.875
fb : Actual	=	1,836.50 psi	fv : Actual	=	239.59 psi
FB : Allowable	=	2,900.00 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	2.595 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.091 in	Ratio =	1189
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.184 in	Ratio =	585
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+L+H					1.00	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.0 ft	1	0.633	0.826	1.00	1.00	1.00	1.00	1.00	1.00	1.00	18.88	1,836.50	2900.00	9.96	239.59	290.00

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.1844	4.237		0.0000	0.000

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: 187

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Nood Beam

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Lic. # : KW-06002966

Description : 2B23

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	12.963	5.895
D Only	6.720	2.974
L Only	6.243	2.920
D+L	12.963	5.895

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A88

Printed: 5 JAN 2018, 11:23AM

Steel Beam

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B24

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

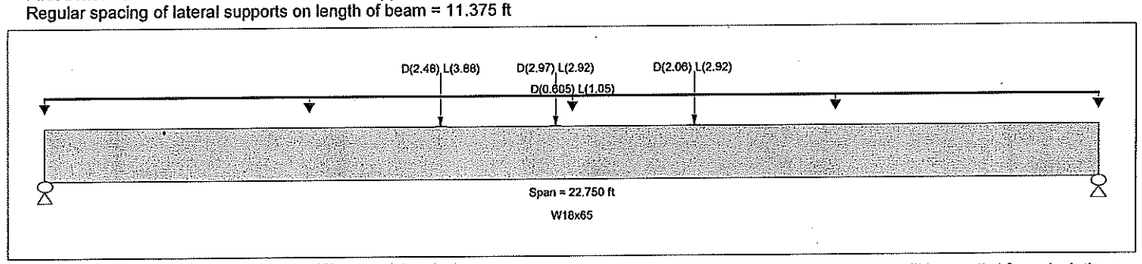
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 11.375 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 11.375 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.6050, L = 1.050 k/ft, Tributary Width = 1.0 ft
Point Load : D = 2.480, L = 3.880 k @ 8.50 ft
Point Load : D = 2.970, L = 2.920 k @ 11.0 ft
Point Load : D = 2.060, L = 2.920 k @ 14.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.583 : 1	Maximum Shear Stress Ratio =	0.172 : 1
Section used for this span	W18x65	Section used for this span	W18x65
Ma : Applied	193.518 k-ft	Va : Applied	28.506 k
Mn / Omega : Allowable	331.836 k-ft	Vn/Omega : Allowable	165.60 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	11.034 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.332 in	Ratio =	823
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.561 in	Ratio =	487
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 11.38 ft	1	0.583	0.172	193.52		193.52	554.17	331.84	1.36	1.00	28.51	248.40	165.60
Dsgn. L = 11.38 ft	1	0.580	0.168	192.49		192.49	554.17	331.84	1.38	1.00	27.85	248.40	165.60

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.5611	11.375		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	28.506	27.854
D Only	11.501	11.252
L Only	17.005	16.602
D+L	28.506	27.854

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A89

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File = h:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Steel Beam

Lic. # : KW-06002966

Description : 2B25

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

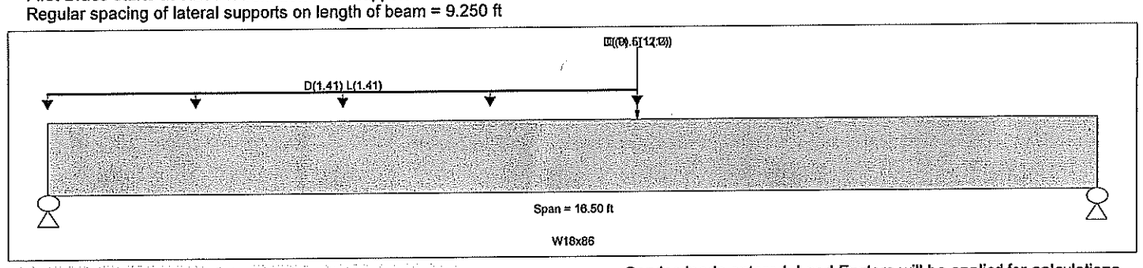
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending
Load Combination ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 9.250 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 9.250 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 1.410, L = 1.410 k/ft, Extent = 0.0 –>> 9.250 ft, Tributary Width = 1.0 ft
Point Load : D = 9.0, L = 12.20 k @ 9.250 ft
Point Load : D = 11.50, L = 17.0 k @ 9.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.555 : 1	Maximum Shear Stress Ratio =	0.234 : 1
Section used for this span	W18x86	Section used for this span	W18x86
Ma : Applied	257.753 k-ft	Va : Applied	41.322 k
Mn / Omega : Allowable	464.072 k-ft	Vn/Omega : Allowable	176.640 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	9.240ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.137 in	Ratio =	1,446
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.246 in	Ratio =	806
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 9.24 ft	1	0.555	0.234	257.75		257.75	775.00	464.07	1.47	1.00	41.32	264.96	176.64
Dsgn. L = 7.26 ft	1	0.555	0.203	257.75		257.75	775.00	464.07	1.66	1.00	35.88	264.96	176.64

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.2457	8.415		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	41.322	35.884
D Only	19.105	15.859
L Only	22.217	20.026
D+L	41.322	35.884

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A90

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Steel Beam

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC

Description : 2B26 and 2B27

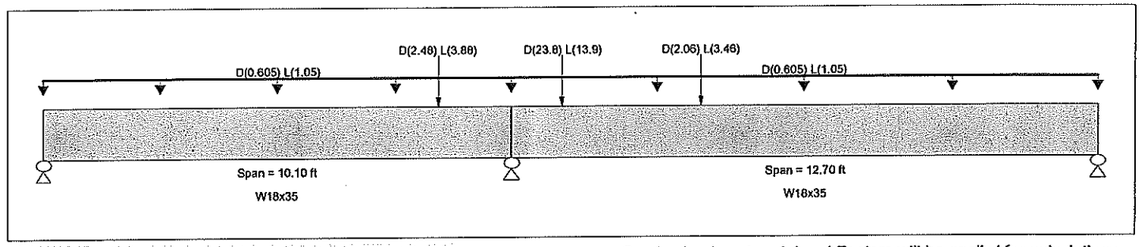
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.6050, L = 1.050 k/ft, Tributary Width = 1.0 ft
Point Load : D = 2.480, L = 3.880 k @ 8.50 ft
Load for Span Number 2
Uniform Load : D = 0.6050, L = 1.050 k/ft, Tributary Width = 1.0 ft
Point Load : D = 23.80, L = 13.90 k @ 1.10 ft
Point Load : D = 2.060, L = 3.460 k @ 4.10 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.388 : 1	Maximum Shear Stress Ratio =	0.505 : 1
Section used for this span	W18x35	Section used for this span	W18x35
Ma : Applied	51.301 k-ft	Va : Applied	53.579 k
Mn / Omega : Allowable	132.309 k-ft	Vn/Omega : Allowable	106.20 k
Load Combination	+D+L+H, LL Comb Run (*L)	Load Combination	+D+L+H, LL Comb Run (LL)
Location of maximum on span	0.000ft	Location of maximum on span	10.100 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.046 in	Ratio =	3,292
Max Upward L+Lr+S Deflection	-0.018 in	Ratio =	6,616
Max Downward Total Deflection	0.079 in	Ratio =	1940
Max Upward Total Deflection	-0.027 in	Ratio =	4519

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H, LL Comb Run (*L)													
Dsgn. L = 10.10 ft	1	0.309	0.499		-51.30	51.30	277.08	165.92	2.17	1.00	52.94	159.30	106.20
Dsgn. L = 12.70 ft	2	0.388	0.499	40.77	-51.30	51.30	220.96	132.31	1.38	1.00	52.94	159.30	106.20
+D+L+H, LL Comb Run (L*)													
Dsgn. L = 10.10 ft	1	0.215	0.283	10.70	-35.64	35.64	277.08	165.92	2.73	1.00	30.00	159.30	106.20
Dsgn. L = 12.70 ft	2	0.215	0.283	12.40	-35.64	35.64	277.08	165.92	2.37	1.00	30.00	159.30	106.20
+D+L+H, LL Comb Run (LL)													
Dsgn. L = 10.10 ft	1	0.358	0.505	3.97	-59.36	59.36	277.08	165.92	3.00	1.00	53.58	159.30	106.20
Dsgn. L = 12.70 ft	2	0.371	0.505	36.48	-59.36	59.36	267.36	160.10	1.67	1.00	53.58	159.30	106.20

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	6.350	D+L	-0.0268	6.215
D+L	2	0.0786	6.350		0.0000	6.215

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	6.013	73.344	11.740

MURPHY BURR CURRY, INC.
STRUCTURAL ENGINEERS
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A91

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Steel Beam

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 | Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B26 and 2B27

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
D Only	0.894	37.420	4.619
L Only, LL Comb Run (*L)	-2.349	25.923	7.121
L Only, LL Comb Run (L*)	5.119	10.001	-0.635
L Only, LL Comb Run (LL)	2.770	35.924	6.486
D+L, LL Comb Run (*L)	-1.454	63.343	11.740
D+L, LL Comb Run (L*)	6.013	47.421	3.984
D+L, LL Comb Run (LL)	3.665	73.344	11.105

MURPHY BURR CURRY INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A92

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Steel Beam

Lic. # : KW-06002966 — Licensee : MURPHY - BURR - CURRY, INC

Description : 2B28

CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

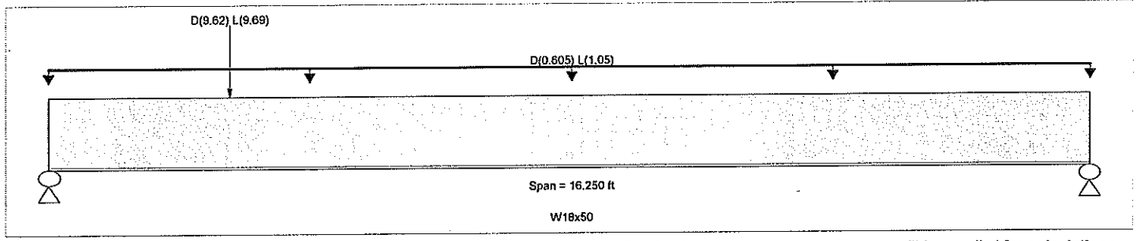
Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam bracing is defined as a set spacing over all spans
Bending Axis : Major Axis Bending
Load Combination : ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi

Unbraced Lengths

First Brace starts at 13.450 ft from Left-Most support
Regular spacing of lateral supports on length of beam = 13.450 ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.6050, L = 1.050 k/ft, Tributary Width = 1.0 ft
Point Load : D = 9.620, L = 9.690 k @ 2.80 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.440 : 1	Maximum Shear Stress Ratio =	0.233 : 1
Section used for this span	W18x50	Section used for this span	W18x50
Ma : Applied	86.560 k-ft	Va : Applied	29.836 k
Mn / Omega : Allowable	196.738 k-ft	Vn/Omega : Allowable	127.80 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	6.175 ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.104 in	Ratio =	1,872
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.181 in	Ratio =	1076
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 13.41 ft	1	0.440	0.233	86.56		86.56	328.55	196.74	1.06	1.00	29.84	191.70	127.80
Dsgn. L = 2.84 ft	1	0.167	0.134	41.96		41.96	420.83	252.00	1.57	1.00	17.18	191.70	127.80

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L	1	0.1812	7.800		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	29.836	17.181
D Only	13.285	6.980
L Only	16.552	10.201
D+L	29.836	17.181

MURPHY BURR CURRY, INC.
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Project Title:
Engineer:
Project Descr:

Project ID: A93

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Beam

Lic. # : KW-06002966 — Licensee : MURPHY, BURR, CURRY, INC.

Description : 2B29

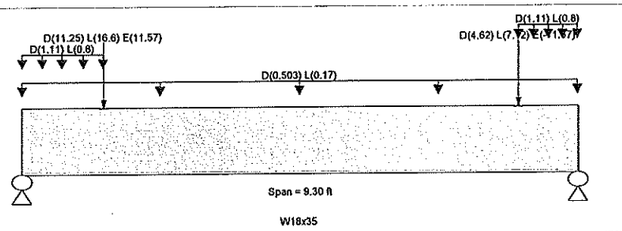
CODE REFERENCES

Calculations per AISC 360-10, IBC 2012, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending
Load Combination ASCE 7-10

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.5030, L = 0.170 k/ft, Tributary Width = 1.0 ft
Point Load : D = 11.250, L = 16.60, E = 11.570 k @ 1.370 ft
Point Load : D = 4.620, L = 7.120, E = -11.570 k @ 8.30 ft
Uniform Load : D = 1.110, L = 0.80 k/ft, Extent = 0.0 –>> 1.370 ft, Tributary Width = 1.0 ft
Uniform Load : D = 1.110, L = 0.80 k/ft, Extent = 8.30 –>> 9.30 ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.311 : 1	Maximum Shear Stress Ratio =	0.372 : 1
Section used for this span	W18x35	Section used for this span	W18x35
Ma : Applied	51.617 k-ft	Va : Applied	39.503 k
Mn / Omega : Allowable	165.918 k-ft	Vn/Omega : Allowable	106.20 k
Load Combination	+1.110D+0.750L+1.313E	Load Combination	+1.110D+0.750L+1.313E
Location of maximum on span	1.395ft	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward L+Lr+S Deflection	0.021 in	Ratio =	5,230
Max Upward L+Lr+S Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.044 in	Ratio =	2534
Max Upward Total Deflection	-0.004 in	Ratio =	28358

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+L+H													
Dsgn. L = 9.30 ft	1	0.240	0.290	39.76		39.76	277.08	165.92	1.00	1.00	30.83	159.30	106.20
+1.150D+1.750E													
Dsgn. L = 9.30 ft	1	0.247	0.294	40.93	-5.03	40.93	277.08	165.92	1.00	1.00	31.26	159.30	106.20
+1.150D-1.750E													
Dsgn. L = 9.30 ft	1	0.150	0.244	24.97	-0.27	24.97	277.08	165.92	1.00	1.00	25.95	159.30	106.20
+1.110D+0.750L+1.313E													
Dsgn. L = 9.30 ft	1	0.311	0.372	51.62		51.62	277.08	165.92	1.00	1.00	39.50	159.30	106.20
+1.110D+0.750L-1.313E													
Dsgn. L = 9.30 ft	1	0.179	0.279	29.71		29.71	277.08	165.92	1.00	1.00	29.62	159.30	106.20

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+E	1	0.0440	4.139		0.0000	0.000

Vertical Reactions - Unfactored

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	39.450	28.494
D Only	14.060	9.445
L Only	16.769	10.428

MURPHY BURR CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: A94

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Beam

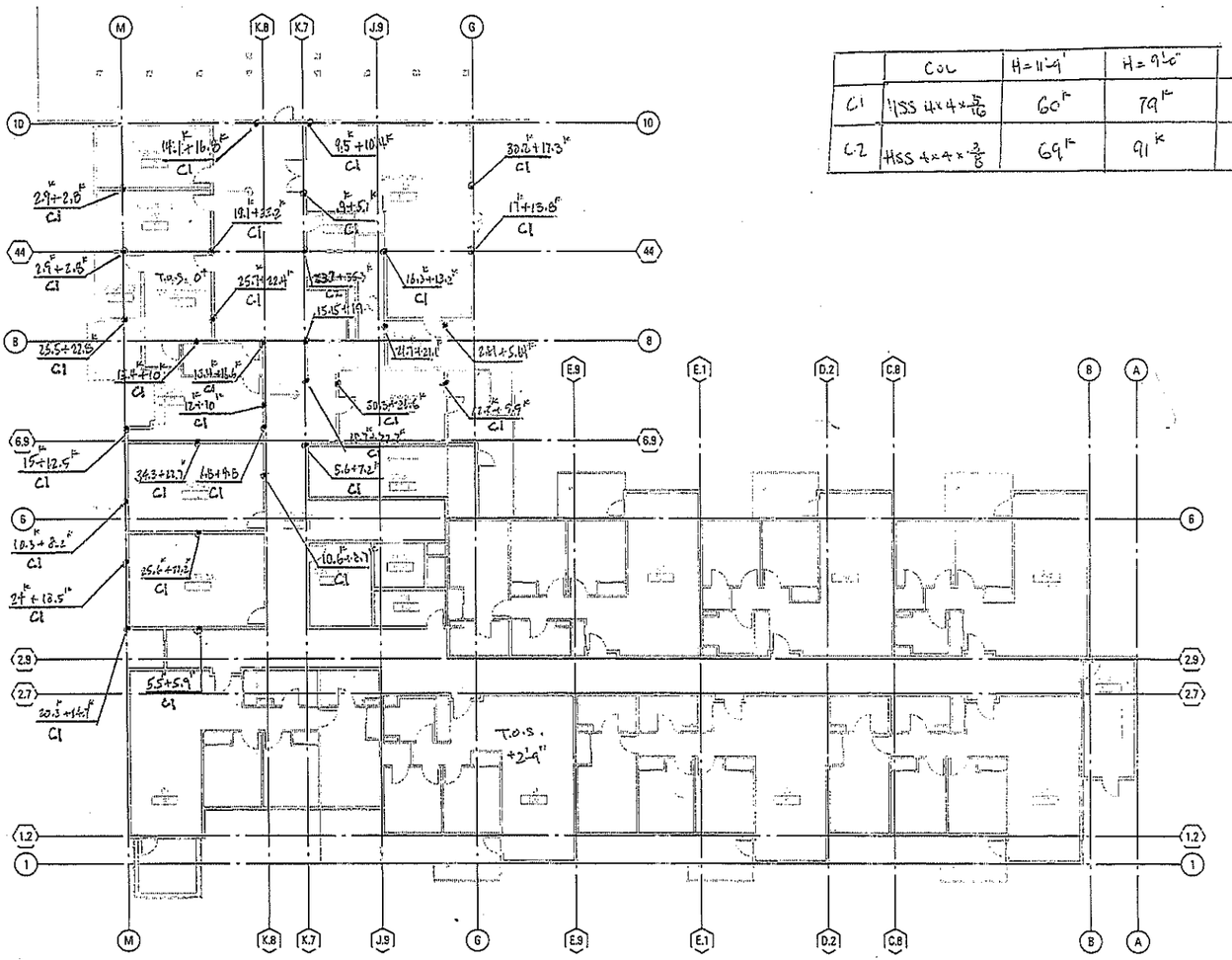
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Description : 2B29

Vertical Reactions - Unfactored

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
E Only	8.622	-8.622
-E Only	-8.622	8.622
D+L	30.829	19.873
D+E	22.681	0.823
D-E	5.438	18.066
D+L+E	39.450	11.251
D+L-E	22.207	28.494



1 S2.01 LEVEL 1 FRAMING PLAN

	Col	H=11'-4"	H=9'-0"	
C1	HSS 4x4x5/16	60^K	79^K	
C2	HSS 4x4x3/8	69^K	91^K	

B1

B2

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 46 ksi

HSS4

Shape		HSS4×4×									
		3/8		5/16		1/4		3/16		1/8	
t_{design} in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		17.3		14.8		12.2		9.42		6.46	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y	0	132	198	113	170	92.8	140	71.1	107	48.8	73.3
	1	131	197	112	169	92.4	139	70.8	106	48.6	73.0
	2	129	194	111	167	91.3	137	69.9	105	48.0	72.1
	3	126	190	109	163	89.4	134	68.5	103	47.1	70.8
	4	123	184	105	158	86.8	130	66.6	100	45.8	68.9
	5	118	177	101	152	83.6	126	64.2	96.6	44.2	66.5
	6	112	168	96.5	145	79.8	120	61.5	92.4	42.4	63.7
	7	106	159	91.2	137	75.6	114	58.3	87.7	40.3	60.6
	8	98.8	149	85.4	128	71.0	107	54.9	82.5	38.0	57.2
	9	91.6	138	79.3	119	66.1	99.3	51.3	77.1	35.6	53.5
	10	84.1	126	73.0	110	61.0	91.7	47.5	71.4	33.1	49.7
	11	76.5	115	66.6	100	55.9	84.0	43.6	65.6	30.5	45.8
	12	69.0	104	60.3	90.6	50.8	76.3	39.8	59.8	27.9	41.9
	13	61.7	92.8	54.0	81.2	45.7	68.7	36.0	54.0	25.3	38.0
	14	54.7	82.2	48.0	72.2	40.8	61.3	32.2	48.5	22.8	34.3
	15	47.9	72.0	42.2	63.5	36.1	54.3	28.7	43.1	20.4	30.6
	16	42.1	63.3	37.1	55.8	31.7	47.7	25.3	38.0	18.0	27.1
	17	37.3	56.1	32.9	49.4	28.1	42.3	22.4	33.6	16.0	24.0
	18	33.3	50.0	29.3	44.1	25.1	37.7	20.0	30.0	14.2	21.4
	19	29.9	44.9	26.3	39.6	22.5	33.8	17.9	26.9	12.8	19.2
	20	27.0	40.5	23.8	35.7	20.3	30.5	16.2	24.3	11.5	17.3
	21	24.4	36.7	21.5	32.4	18.4	27.7	14.7	22.1	10.5	15.7
	22	22.3	33.5	19.6	29.5	16.8	25.2	13.4	20.1	9.53	14.3
	23	20.4	30.6	18.0	27.0	15.4	23.1	12.2	18.4	8.72	13.1
	24	18.7	28.1	16.5	24.8	14.1	21.2	11.2	16.9	8.01	12.0
	25					13.0	19.5	10.4	15.6	7.38	11.1
	26									6.82	10.3
Properties											
A_g in.^{2}		4.78		4.10		3.37		2.58		1.77	
I_z in.^{4}		10.3		9.14		7.80		6.21		4.40	
r_y in.		1.47		1.49		1.52		1.55		1.58	
ASD		LRFD		Note: Heavy line indicates KL/r_y equal to or greater than 200.							
Ω_c = 1.67		ϕ_c = 0.90									

MURPHY BURR CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:
Project ID:

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY BURR CURRY, INC.

Steel Base Plate

Lic. # : KW-06002966

Description : 1"X12" SQ Baseplate @ HSS4X4

Code References

Calculations per AISC Design Guide # 1, IBC 2012, CBC 2013, ASCE 7-10
Load Combination Set : ASCE 7-10

General Information

Material Properties

AISC Design Method	Load Resistance Factor Design		Φ c : LRFD Resistance Factor	0.60
Steel Plate Fy	=	36.0 ksi		
Concrete Support f'c	=	4.0 ksi		
Assumed Bearing Area :	Full Bearing		Allowable Bearing Fp per J8	3.40 ksi

Column & Plate

Column Properties

Steel Section : HSS4x4x3/8

Depth	4 in	Area	4.78 in^2
Width	4 in	Ixx	10.3 in^4
Flange Thickness	0.349 in	Iyy	10.3 in^4
Web Thickness	0 in		

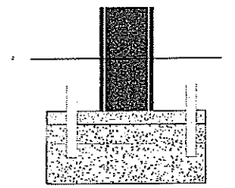
Plate Dimensions

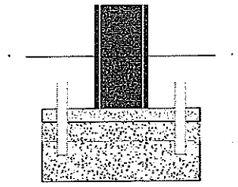
N : Length	12.0 in
B : Width	12.0 in
Thickness	1.0 in

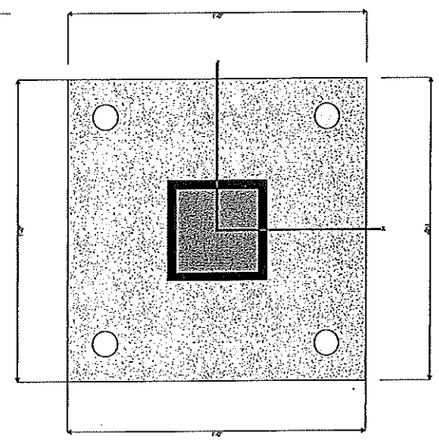
Support Dimensions

Width along 'X'	12.0 in
Length along 'Z'	12.0 in

Column assumed welded to base plate.





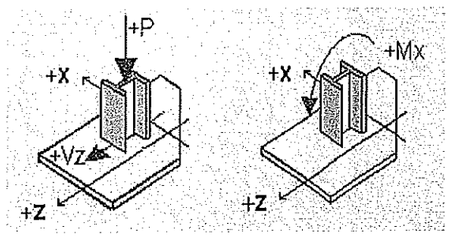


Applied Loads

	P-Y	V-Z	M-X
D : Dead Load	25.0 k	0.0 k	0.0 k-ft
L : Live	36.0 k	0.0 k	0.0 k-ft
Lr : Roof Live	0.0 k	0.0 k	0.0 k-ft
S : Snow	0.0 k	0.0 k	0.0 k-ft
W : Wind	0.0 k	0.0 k	0.0 k-ft
E : Earthquake	15.0 k	0.0 k	0.0 k-ft
H : Lateral Earth	0.0 k	0.0 k	0.0 k-ft

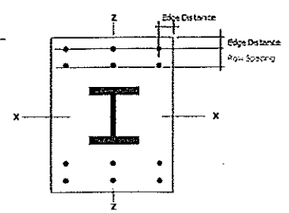
"P" = Gravity load, "+" sign is downward.

"+" Moments create higher soil pressure at +Z edge.
"+" Shears push plate towards +Z edge.



Anchor Bolts

Anchor Bolt or Rod Description	.75
Max of Tension or Pullout Capacity	15.0 k
Shear Capacity	8.0 k
Edge distance : bolt to plate	1.50 in
Number of Bolts in each Row	2.0
Number of Bolt Rows	1.0



MURPHY BURR CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: B4

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Base Plate

Lic. # : KW-06002966 Licensee : MURPHY BURR CURRY, INC

Description : 1"X12" SQ Baseplate @ HSS4X4

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	Load Resistance Factor Design
Governing Load Combination	+1.20D+0.50L+0.20S+2.750E+1.60H
Governing Load Case Type	Axial Load Only
Design Plate Size	1'-0" x 1'-0" x 1"
Pu : Axial	89.250 k
Mu : Moment	0.000 k-ft
Mu : Max. Moment	5.209 k-in
fb : Max. Bending Stress	20.837 ksi
Fb : Allowable :	32.400 ksi
Fy * Phi	
Bending Stress Ratio	0.643
	Bending Stress OK
fu : Max. Plate Bearing Stress	0.620 ksi
Fp : Allowable :	2.040 ksi
min(0.85*f'c*sqrt(A2/A1), 1.7* f'c)*Phi	
Bearing Stress Ratio	0.304
	Bearing Stress OK

Load Comb. : +1.20D+1.60L+0.50S+1.60H

Axial Load Only, No Moment

Loading

Pu : Axial	87.600	k
Design Plate Height	12.000	in
Design Plate Width	12.000	in
Will be different from entry if partial bearing used.		
A1 : Plate Area	144.000	in^2
A2: Support Area	144.000	in^2
sqrt(A2/A1)	1.000	

Distance for Moment Calculation

" m "	4.100	in
" n "	4.100	in
X	0.000	in^2
Lambda	0.000	
n'	1.010	in
n' * Lambda	0.000	in
L = max(m, n, n')	4.100	in

Bearing Stresses

Fp : Allowable	2.040 ksi
fu : Max. Bearing Pressure	0.608 ksi
Stress Ratio	**0.298**

Plate Bending Stresses

Mmax = Fu * L^2 / 2	5.113 k-in
fb : Actual	20.452 ksi
Fb : Allowable	32.400 ksi
Stress Ratio	**0.631**

Load Comb. : +1.20D+0.50L+0.20S+2.750E+1.60H

Axial Load Only, No Moment

Loading

Pu : Axial	89.250	k
Design Plate Height	12.000	in
Design Plate Width	12.000	in
Will be different from entry if partial bearing used.		
A1 : Plate Area	144.000	in^2
A2: Support Area	144.000	in^2
sqrt(A2/A1)	1.000	

Distance for Moment Calculation

" m "	4.100	in
" n "	4.100	in
X	0.000	in^2
Lambda	0.000	
n'	1.010	in
n' * Lambda	0.000	in
L = max(m, n, n')	4.100	in

Bearing Stresses

Fp : Allowable	2.040 ksi
fu : Max. Bearing Pressure	0.620 ksi
Stress Ratio	**0.304**

Plate Bending Stresses

Mmax = Fu * L^2 / 2	5.209 k-in
fb : Actual	20.837 ksi
Fb : Allowable	32.400 ksi
Stress Ratio	**0.643**

MURPHY BURR CURRY INC
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: B5

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Steel Base Plate

Lic. #: KW-06002966 Licensee: MURPHY BURR CURRY INC

Description : 1"X12" SQ Baseplate @ HSS4X4

Load Comb. : +1.20D+0.50L+0.20S-2.750E+1.60H

Axial Load Only, No Moment

Loading

Pu : Axial	6.750	k
Design Plate Height	12.000	in
Design Plate Width	12.000	in
Will be different from entry if partial bearing used.		
A1 : Plate Area	144.000	in^2
A2: Support Area	144.000	in^2
sqrt(A2/A1)	1.000	

Distance for Moment Calculation

" m "	4.100	in
" n "	4.100	in
X	0.000	in^2
Lambda	0.000	
n'	1.010	in
n' * Lambda	0.000	in
L = max(m, n, n')	4.100	in

Bearing Stresses

Fp : Allowable	2.040	ksi
fu : Max. Bearing Pressure	0.047	ksi
Stress Ratio	**0.023**	

Plate Bending Stresses

Mmax = Fu * L^2 / 2	0.394	k-in
fb : Actual	1.576	ksi
Fb : Allowable	32.400	ksi
Stress Ratio	**0.049**	

III. MISCELLANEOUS CALCULATION

- STUD WALL	MA.1 – MA.13
- SHEARWALL TRANSFER DETAIL	MB.1 – MB.7
- STAIR FRAMING	MC.1 – MC.12
- Non-Load Bearing/shearwall anchorage	MD.1
- Hilti Nails (PDF)	

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SUPERSTRUCTURE LATERAL

85 Second Street • Suite 501 • San Francisco, CA 94105 • Tel: 415.546.0431 • Fax: 415.882.7257

MURPHY BURR CURRY, INC
Consulting Structural Engineers

Chestnut Square Family Housing

Building Weight for Seismic

Area Loads

Area Loads - Roof

Roof		25 psf
totals		25 psf
Partitions+wall		10 psf
Dead Load	**Roof**	**35 psf**

AREA SEGMENTS	l	w	SF
			15307
		Total Area	15307 sf

Weight **535.7 k**

Chestnut Square Family Housing

Building Weight Summary

LEVEL	AREA [SF]	AREA WEIGHT [K]	
Roof	15307	536	
4th	14661	716	
3rd	14509	711	
2nd	15267	747	
SUB-TOT		2709	[k]

MURPHY BURR CURRY, INC
Consulting Structural Engineers

Area Loads

Area Loads -4th

Floor		30 psf
totals		30 psf
Partitions+wall		17.5 psf
Dead Load	**Floor**	**47.5 psf**

AREA SEGMENTS	l	w	SF
			14661
		Total Area	14661 sf

Weight **696.4 k**

Area Loads

Area Loads -4th Floor Deck

Floor		30 psf
totals		30 psf
Partitions+wall		0 psf
Dead Load	**Floor**	**30 psf**

AREA SEGMENTS	l	w	SF
			650
		Total Area	650 sf

Weight **19.5 k**

MURPHY BURR CURRY, INC
Consulting Structural Engineers

Area Loads

Area Loads - 3rd

Floor		30 psf
totals		30 psf
Partitions+wall		17.5 psf
Dead Load	**Floor**	**47.5 psf**

AREA SEGMENTS	l	w	SF
			14509
		Total Area	14509 sf

Weight **689.2 k**

Area Loads

Area Loads - 3rd Floor Deck

Floor		30 psf
totals		30 psf
Partitions+wall		0 psf
Dead Load	**Floor**	**30 psf**

AREA SEGMENTS	l	w	SF
			715
		Total Area	715 sf

Weight **21.5 k**

MURPHY BURR CURRY, INC
Consulting Structural Engineers

Area Loads

Area Loads - 2nd

Floor		30 psf
totals		30 psf
Partitions+wall		17.5 psf
Dead Load	**Floor**	**47.5 psf**

AREA SEGMENTS	l	w	SF
			15267
		Total Area	15267 sf

Weight **725.2 k**

Area Loads

Area Loads - 2nd Floor Deck

Floor		30 psf
totals		30 psf
Partitions+wall		0 psf
Dead Load	**Floor**	**30 psf**

AREA SEGMENTS	l	w	SF
			715
		Total Area	715 sf

Weight **21.5 k**

C5

MURPHY BURR CURRY, INC
Consulting Structural Engineers

Project:Chestnut SQ F. H.
Project No: 216-008
1/8/2018

Chestnut Square Family Housing

BASE SHEAR CALCULATION
CALIFORNIA BUILDING CODE 2016 EDITION (ASCE7-10)

Site Calss: D

Project: Chestnut SQ, Livermore, CA

R	=	6.5 wood	(ASCE7- Table 12.2-1)
Ω	=	3	(ASCE7- Table 12.2-1)
hn	=	42.75 ft	
I	=	1	
Sms	=	1.551 g	(See Attached Calculation Print out/soil report)
Sm1	=	0.9 g	(See Attached Calculation Print out/soil report)
SDS	=	1.036 g	
SD1	=	0.600 g	
Category	=	D	CBC 1613A.5.6
Ct	=	0.020	(ASCE7- Table 12.8-2)
x	=	0.750	(ASCE7- Table 12.8-2)
Ta	=	0.334374 sec	"= Ct(hn)^x"
TL	=	12.000 sec	(From map ASCE7-05, 11.4.5)
p	=	1.000	(ASCE7- 12.3.4.2, 1.0 meets req'd "b")

SEISMIC RESPONSE COEFFICIENT:
Cs = SDS/(R/I)

Cs = 0.159 (DETERMINED ACCORDING TO ASCE7- 12.8-2)

Cs = SD1/[T*(R/I)], FOR T<=TL

Cs = 0.276 (NEED NOT TO EXCEED THIS VALUE, ASCE7- 12.8-3)

Cs = SD1*TL/[T^2*(R/I)], FOR T>TL

Cs = 0.00 (NEED NOT TO EXCEED THIS VALUE, ASCE7- 12.8-4)

Cs = 0.05 W (SHALL NOT BE LESS THAN THIS VALUE, ASCE7- 12.8-5)

Cs= 0.5*S1/(R/I), Where S1 >= 0.6g
S1= 0.6
Ss= 1.551
Cs = 0.12 (SHALL NOT BE LESS THAN THIS VALUE, ASCE7- 12.8-6)

SEISMIC RESPONSE COEFFICIENT:	**Cs**	**=**	**0.159**	
SEISMIC DESIGN BASE SHEAR: V=CsW:	**p**	**=**	**1.000**	
	V	**=**	**0.159 W**	**[LRFD]**
	V	**=**	**0.112 W**	[ASD]
	W	**=**	**2708.9 [k]**	
	V	**=**	302.2 **[k]**	[ASD]

MURPHY BURR CURRY, INC
Consulting Structural Engineers

Chestnut Square Family Housing

VERTICAL DISTRIBUTION OF LATERAL LOAD (ASCE7- 12.8.3)

Fx = Cvx*V, $Cvx = \frac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$

T = 0.334 sec
k = 1 (k=1 for T<=0.5sec, k=2 for T>=2.5 sec, ASCE7-05 12.8.3)

V = 302 [K] [ASD]

FLOOR LEVEL	h(x) [ft]	W(x) [k]	W(x) * h(x)^k [k-ft]	W * h sum (W*h^k)	Fx (ASD) [k]	SF Area [sf]	Fx (ASD) [psf]	Vx (ASD) [k]
Roof	42.75	535.7	22903	0.32	96.1	15307	6.3	96.1
4th	32.75	715.9	23446	0.33	98.4	14661	6.7	194.5
3rd	22.75	710.6	16167	0.22	67.8	14509	4.7	262.3
2nd	12.75	746.6	9520	0.13	39.9	15267	2.6	302.2
		2708.9	72035	1.00	302.2		20.3	

C7

MURPHY BURR CURRY, INC
Consulting Structural Engineers

Project: Chestnut F.H.
Project No: 216-008
1/8/2018
Page:_____

Chestnut Square Family Housing

DESIGN WIND PRESSURE CALCULATION
CALIFORNIA BUILDING CODE 2013 EDITION

SPEED: 110 MPH
EXPOSURE: C

Puse = 20 [psf]

East/West Directions:

level	Tri. Height (ft)	P Used (psf)	Length (ft)	shear (plf)	shear (kips)	story shear (kips)
Roof	10	20	180	200	36.0	36.0
4th	10	20	180	200	36.0	72.0
3rd	10	20	180	200	36.0	108.0
2nd	11.375	20	180	228	41.0	149.0

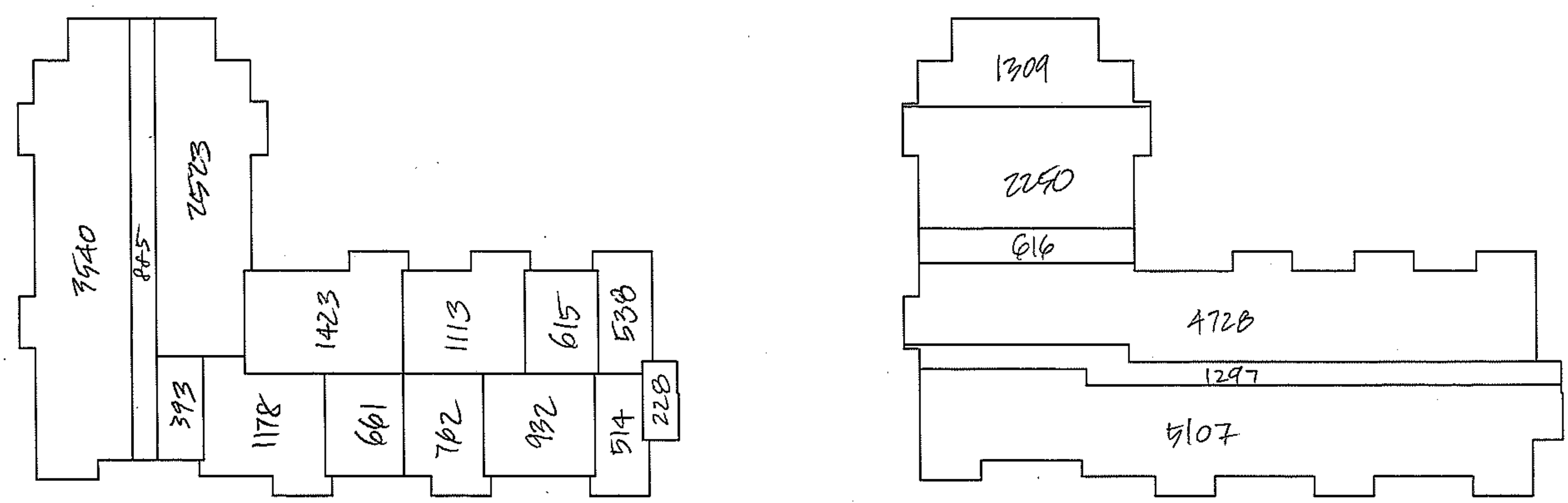
North/South Directions:

level	Tri. Height (ft)	P Used (psf)	Length (ft)	shear (plf)	shear (kips)	story shear (kips)
Roof	10	20	128	200	25.6	25.6
4th	10	20	128	200	25.6	51.2
3rd	10	20	128	200	25.6	76.8
2nd	11.375	20	128	228	29.1	105.9

Seismic/Wind Comparison

	Level	Wind [k]	Seismic [k]	Control
E/W Dir	Roof	36.0	96	
	4th	36.0	98	
	3rd	36.0	68	
	2nd	41.0	40	
		149.0	**302**	Seismic

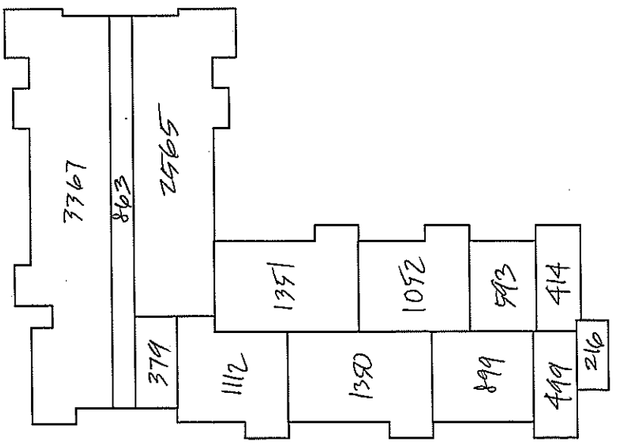
	Level	Wind [k]	Seismic [k]	Control
N/W Dir	Roof	25.6	96	
	4th	25.6	98	
	3rd	25.6	68	
	2nd	29.1	40	
		105.9	**302**	Seismic

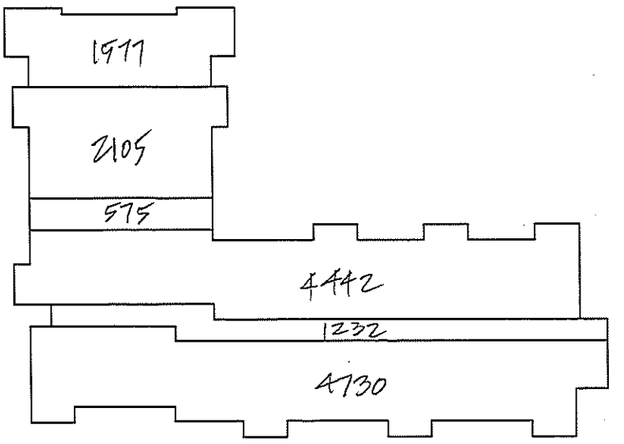


AREA = 15307 SF

ROOF

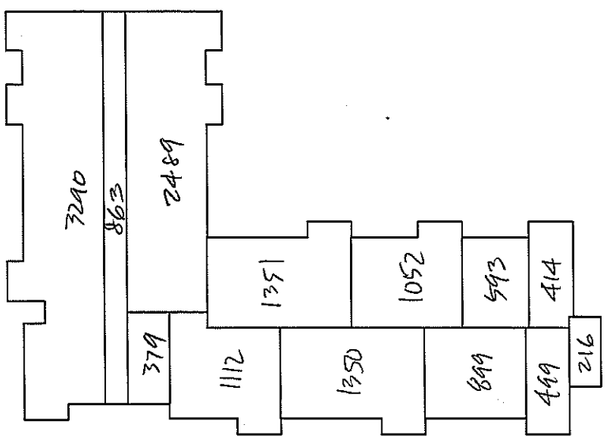
D1

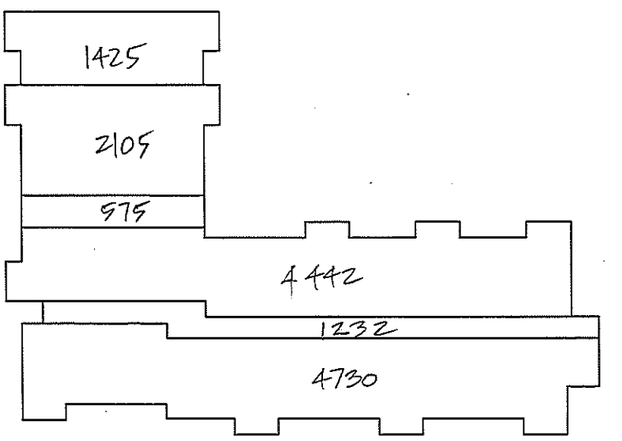




AREA = 14661 SF

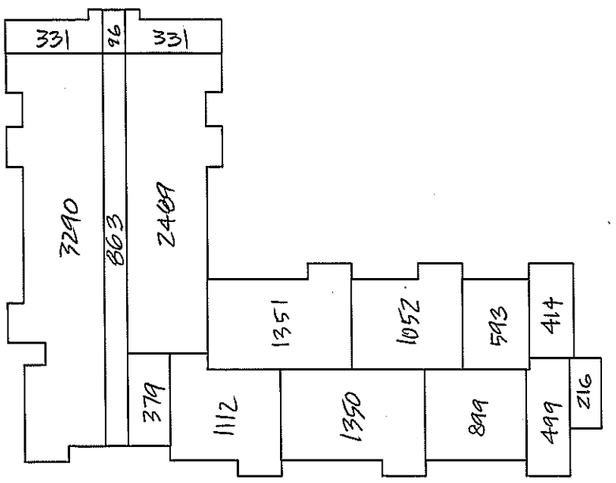
4411

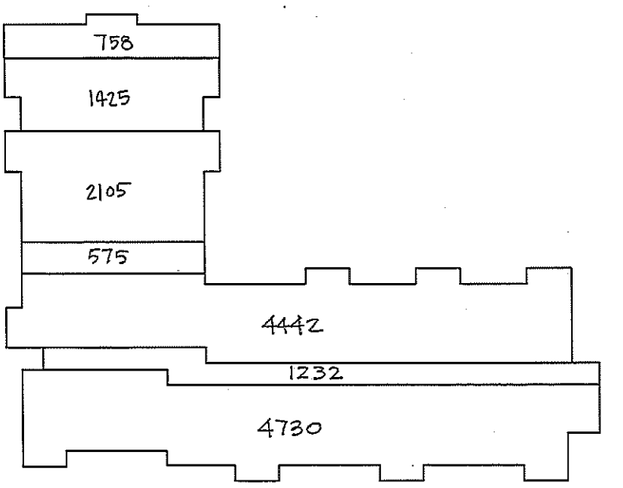




AREA = 14509 SF

3XD

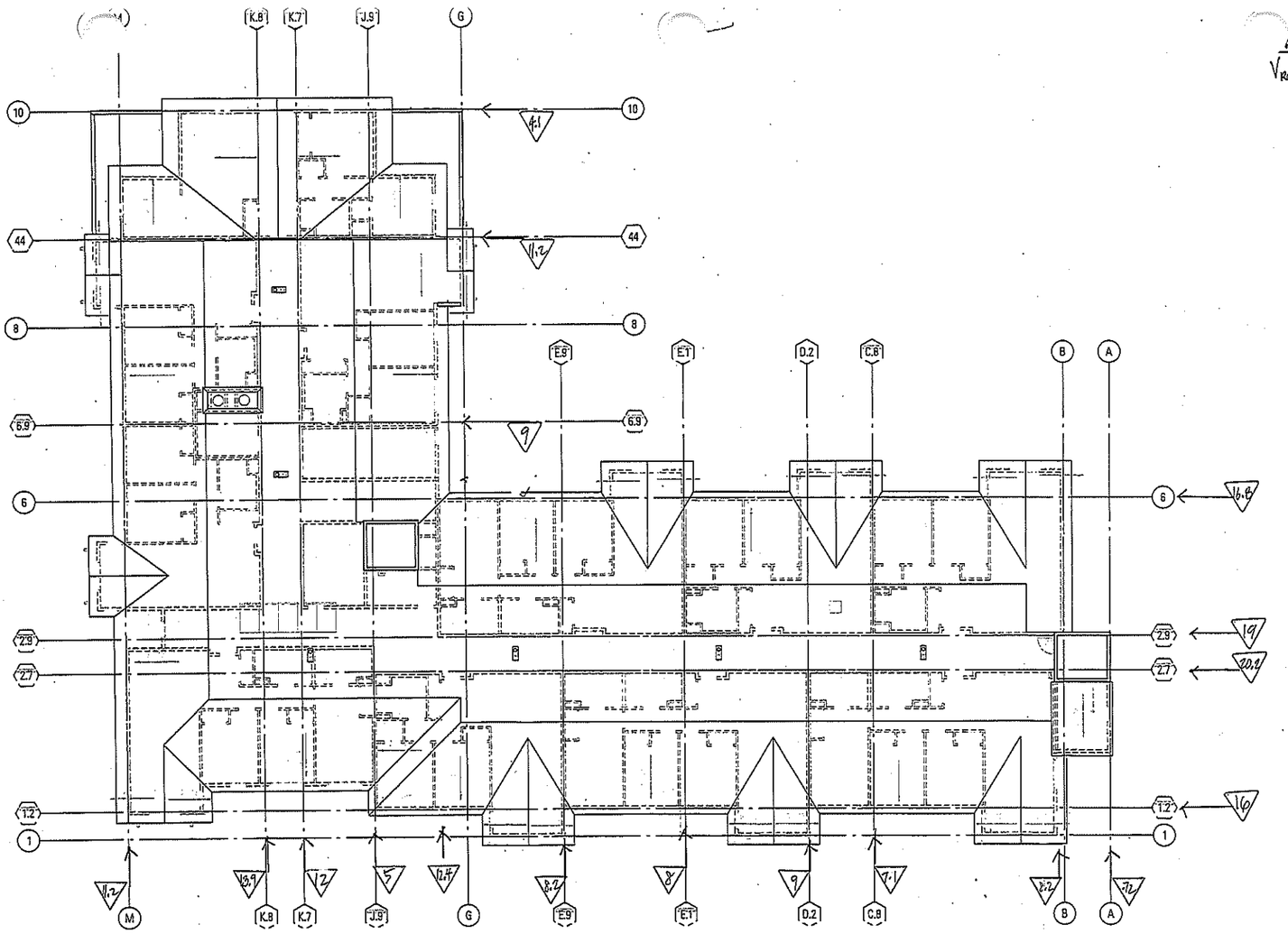


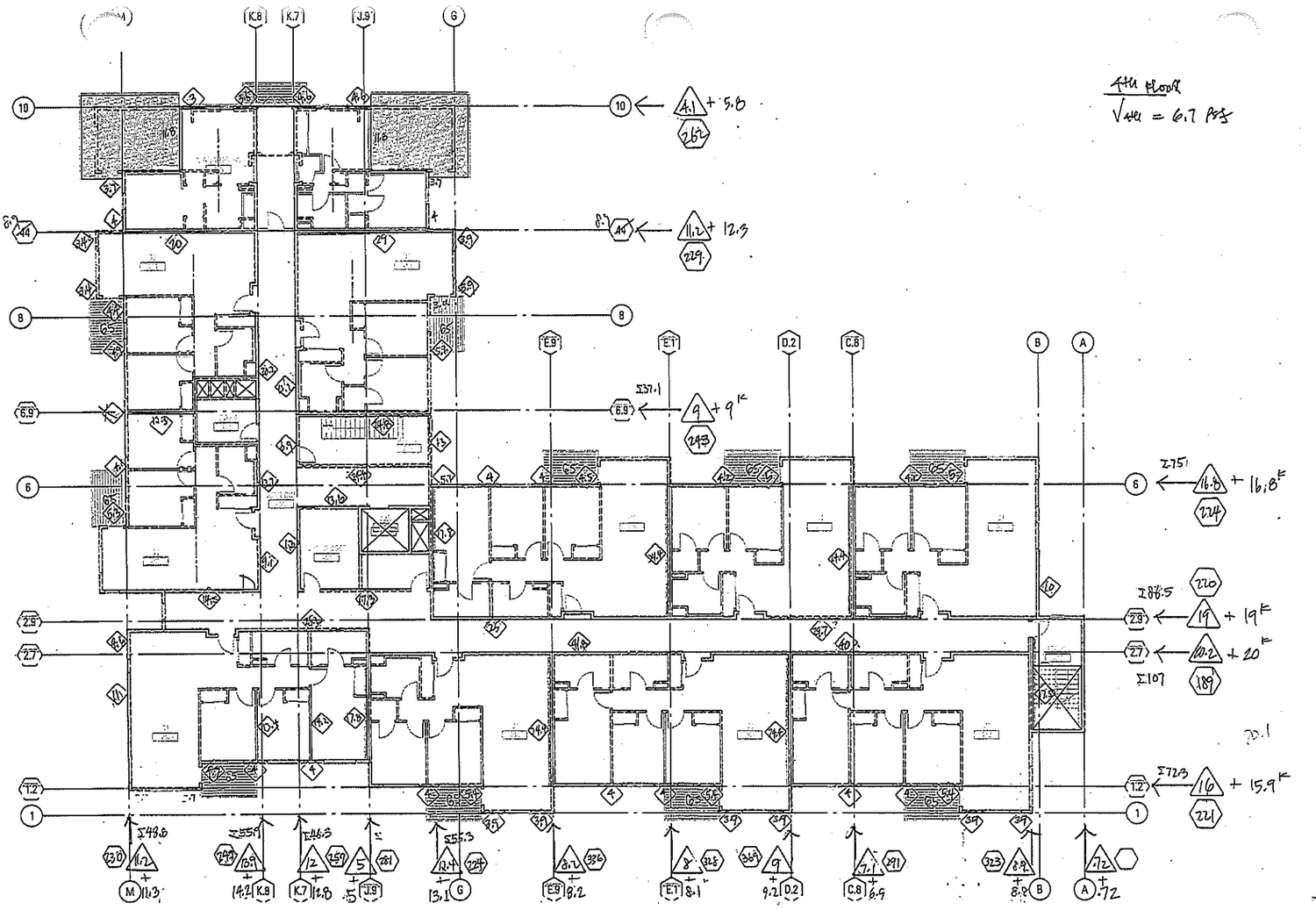


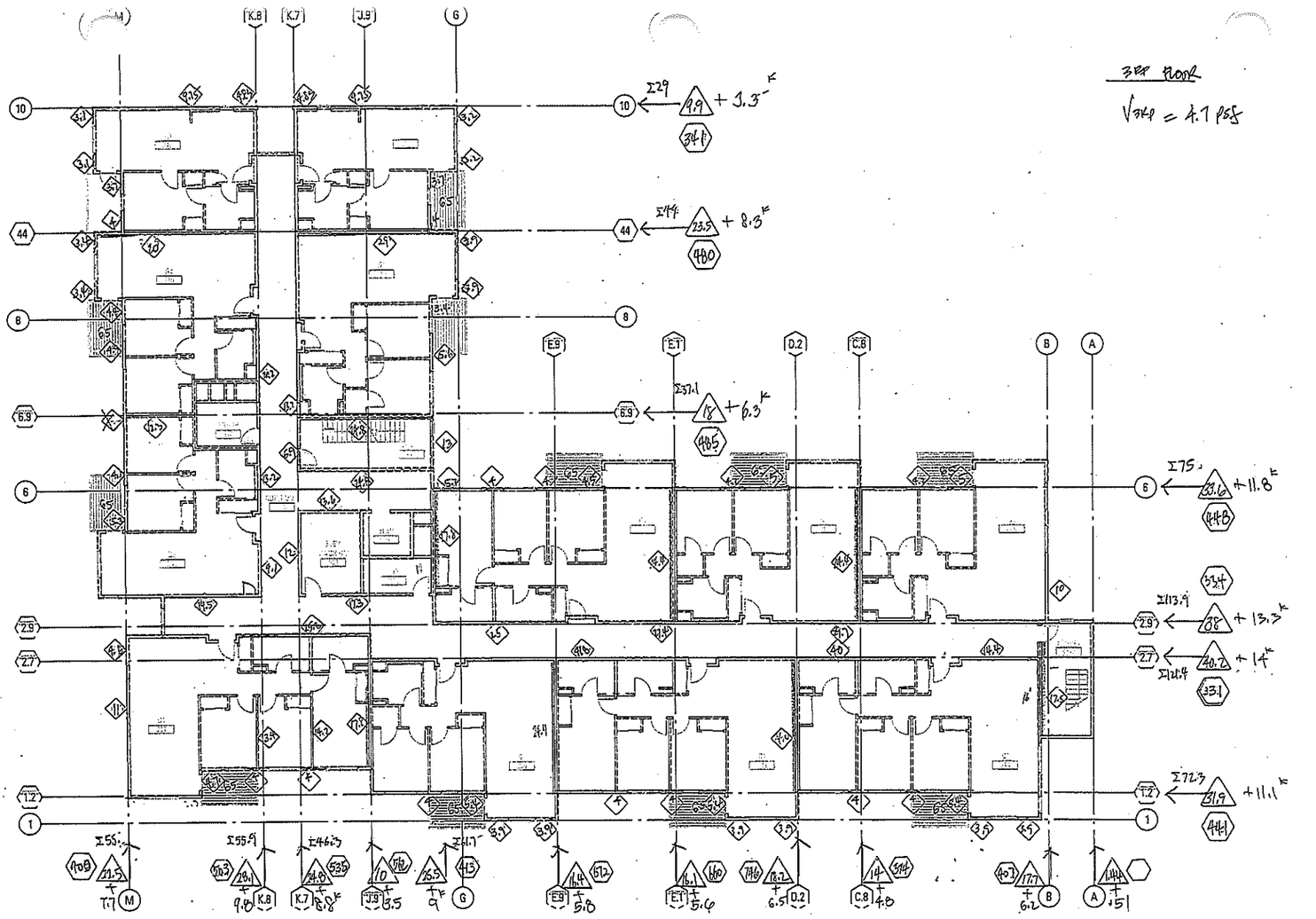
AREA = 14267 SF

2ND

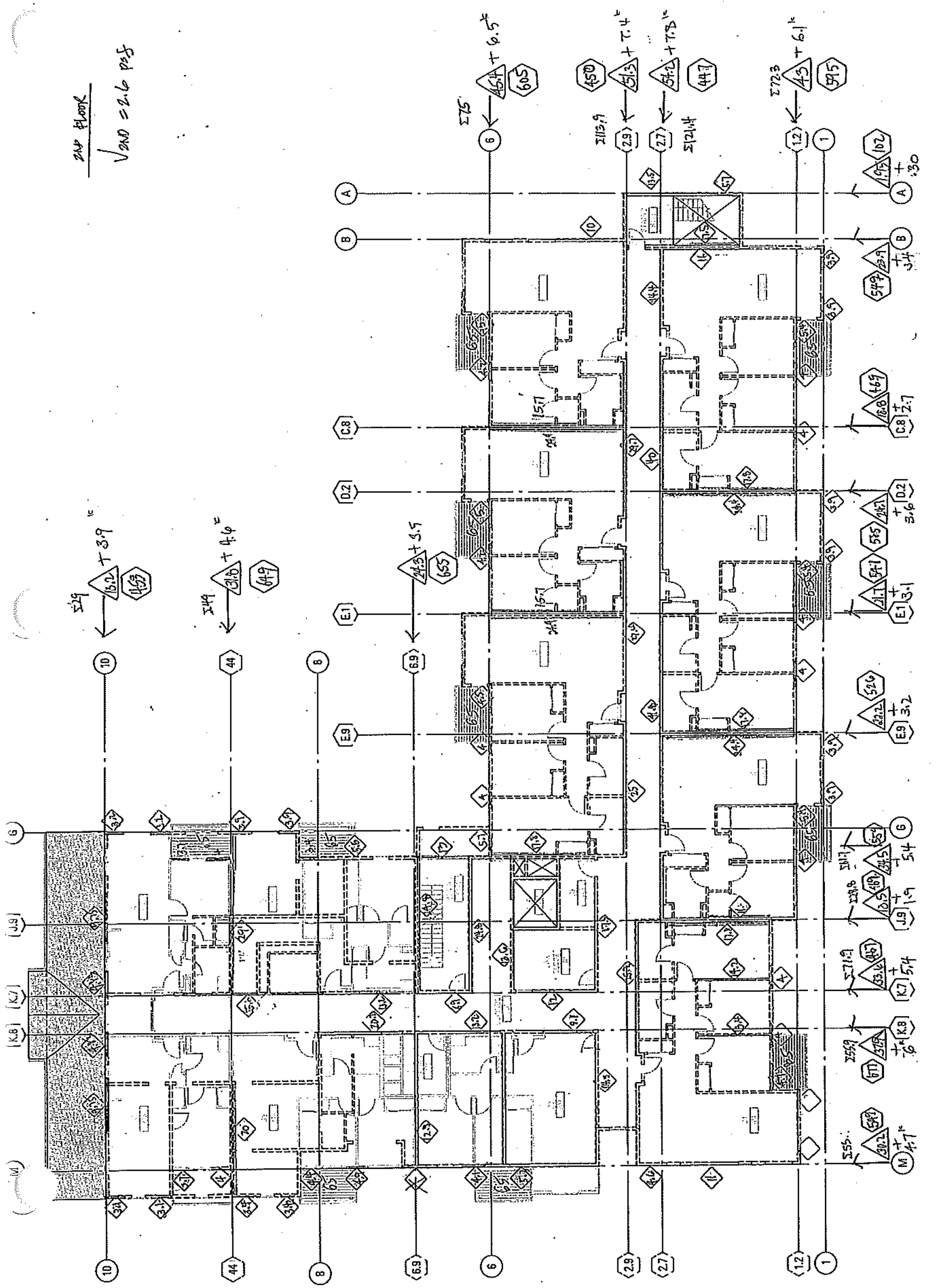
$V_{ROOF} = 6.3$ psf

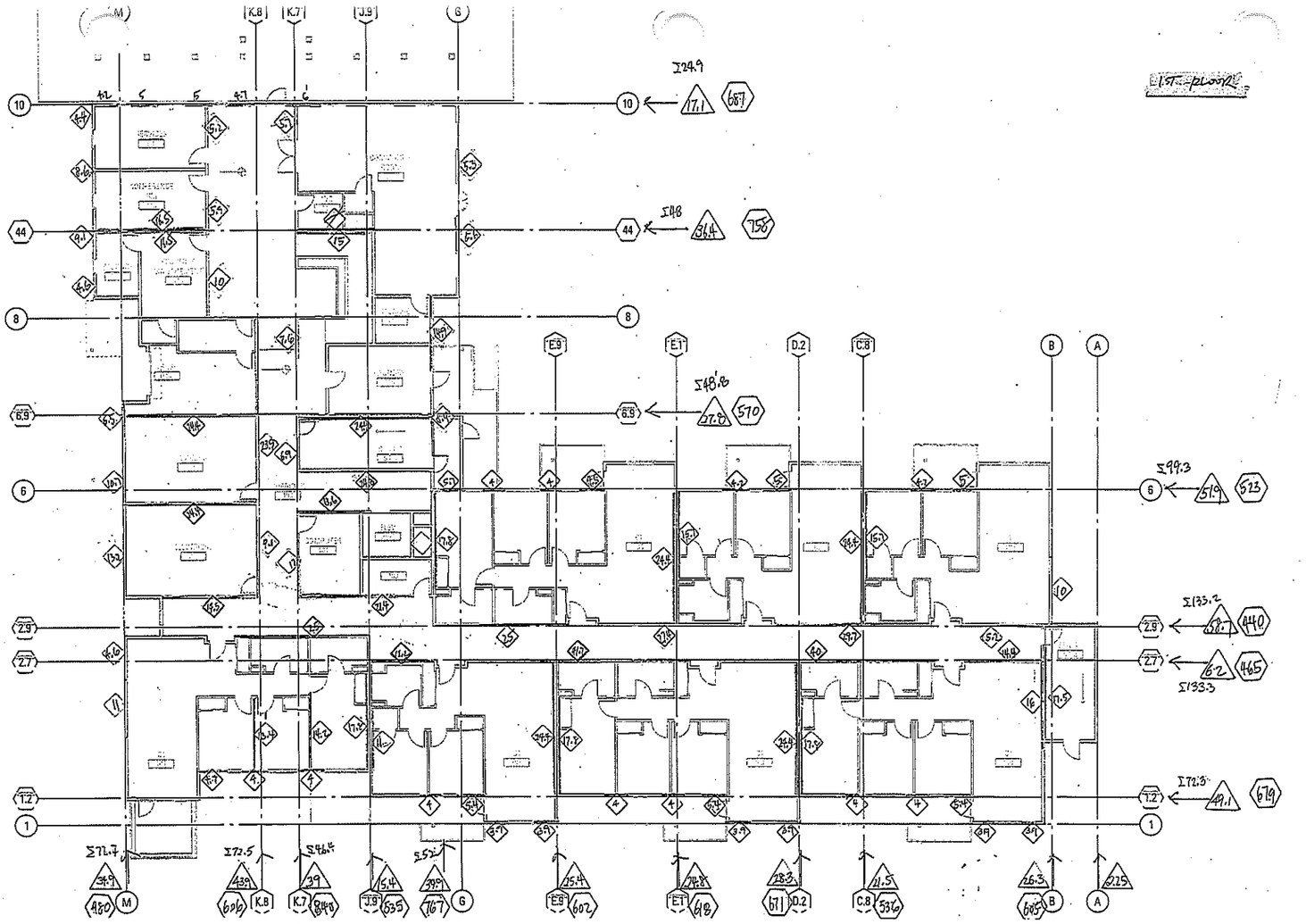



AHU flow
$\sqrt{AHU} = 6.7$ Pa


3RD floor
√340 = 4.7 psf

D8




1ST. FLOOR
Σ24.9
17.1
687
Σ148
34.4
756
Σ148.8
27.9
570
Σ99.3
519
523
Σ133.2
52.7
440
Σ133.3
6.2
465
Σ72.3
49.1
679
Σ72.7
Σ72.5
Σ46.4
Σ52
M
K.8
K.7
J.9
G
E.9
E.1
D.2
C.8
B
A

TIE DOWN SCHEDULE

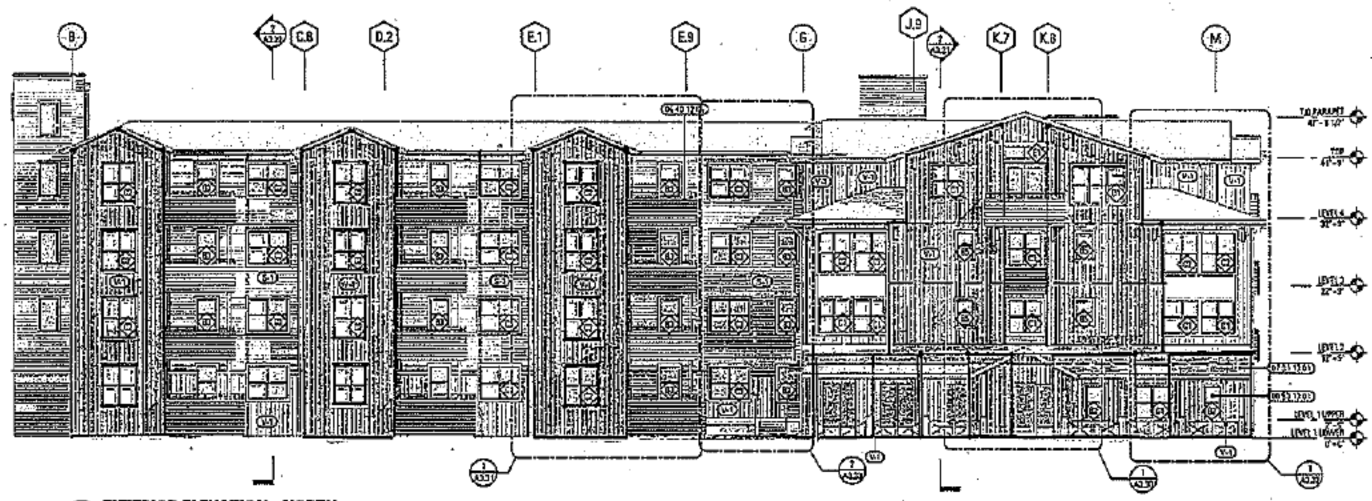
MARK	T11	T21	T22	T31	T32	T33	T40	T41	T42
SIZE									
ROOF	COMP. STUDS (3)		COMP. STUDS (3)	COMP. STUDS (3)	COMP. STUDS (3)		COMP. STUDS (3)	COMP. STUDS (3)	COMP. STUDS (3)
LEVEL 4						(3k) 6" WALL 2-2x6 / 4" WALL 2-2x4	(3k) 6" WALL 2-2x6 / 4" WALL 2-2x4	(3k) 6" WALL 2-2x6 / 4" WALL 2-2x4	(6k) 6" WALL 2-2x6 / 4" WALL 2-2x4
LEVEL 3			(6k) 6" WALL 2-2x6 / 4" WALL 2-2x4	(6k) 6" WALL 2-2x6 / 4" WALL 2-2x4	(12k) 6" WALL 2-2x6 / 4" WALL 3-2x4	(8k) 6" WALL 2-2x6 / 4" WALL 3-2x4	(6k) 6" WALL 2-2x6 / 4" WALL 2-2x4	(8k) 6" WALL 2-2x6 / 4" WALL 3-2x4	(12k) 6" WALL 2-2x6 / 4" WALL 3-2x4
LEVEL 2	*	(6k) 6" WALL 2-2x6 / 4" WALL 2-2x4	(12k) 6" WALL 2-2x6 / 4" WALL 3-2x4	(12k) 6" WALL 2-2x6 / 4" WALL 3-2x4	(20k) 6" WALL 2-2x6 / 4" WALL 1-4x6	(13k) 6" WALL 2-2x6 / 4" WALL 1-4x6	(10k) 6" WALL 2-2x6 / 4" WALL 1-4x4	(14k) 6" WALL 2-2x6 / 4" WALL 1-4x6	(20k) 6" WALL 2-2x6 / 4" WALL 1-4x6
LEVEL 1	(12k) 6" WALL 2-3x4 / 4" WALL 2-2x6	(12k) 6" WALL 2-3x4 / 4" WALL 2-2x6	BEAM	(18k) 6" WALL 2-2x6 / 4" WALL 2-3x4	(24k) 6" WALL 1-6x6 / 4" WALL 1-4x8	BEAM	(15k) 6" WALL 2-2x6 / 4" WALL 1-4x4	(20k) 6" WALL 2-2x6 / 4" WALL 1-4x6	(24k) 6" WALL 1-6x6 / 4" WALL 1-4x8
ANCHOR BOLTS (SIMPSON)	(1) (5/8"ø)	(1) (5/8"ø)	(2) ATS-SBC5H (5/8"ø H.S.)	(1) (5/8"ø)	(2) (5/8"ø HS)	(2) ATS-SBC5H (5/8"ø H.S.)	(1) (1"ø)	(1) (1 1/8"ø)	(2) (5/8"ø HS)

NOTES :

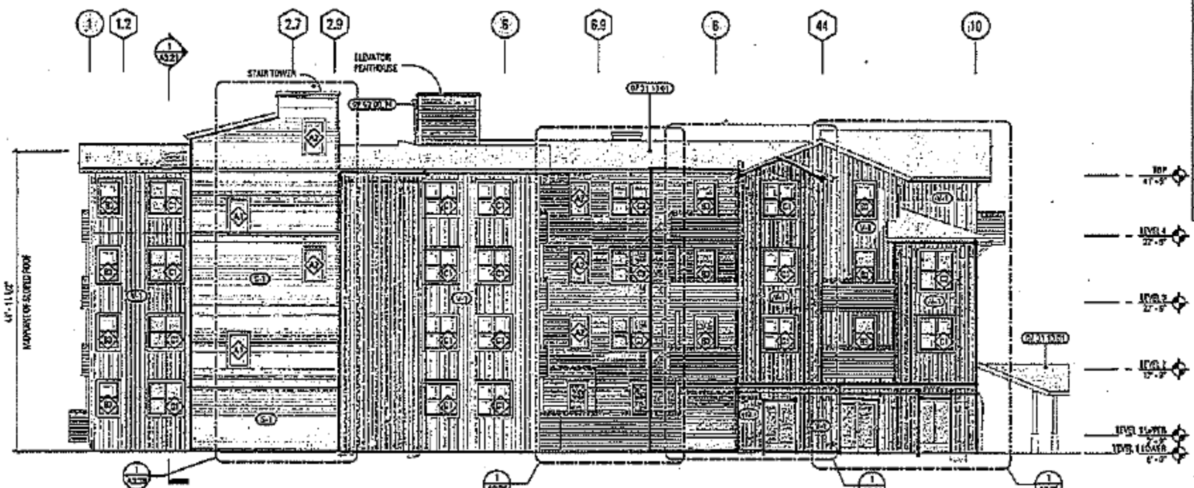
1) STANDARD ROD SHALL BE SIMP. ATS THREADED ROD WITH Fu=60 KSI AND Fy=43 KSI.
2) HS ROD SHALL BE SIMP. ATS THREADED ROD WITH Fu=120 KSI AND Fy=92 KSI.
3) STUDS NOTED IN SCHEDULE SHALL BE STUDS REQUIRED EA. SIDE OF TIE DOWN SYSTEM. TYP. U.N.O. ON PLAN
4) ALL THREADED ROD ANCHOR SHALL HAVE 1½"x5" SQ. PLATE WASHER.
5) * INDICATES EXTEND TIE ROD TO TOP OF FLOOR BEAM. SEE DETAILS 1/S4.20 OR 9/S4.20
6) MANUFACTURE TO PROVIDE ROD DESIGN WITH ACCUMULATION OF TOTAL VERTICAL MOVEMENT INCLUDE ROD ELONGATION, BEARING PLATE GRAIN DEFORMATION, LOOSENESS DUE TO TAKE UP DEVICES AND OTHER COMPONENTS OF THE TIEDOWN SYSTEM RESISTING NOT TO EXCEED 0.179 INCH PER FLOOR
7) ALL TIEDOWN RODS SHALL BE TIED IN PLACE PRIOR TO FOUNDATION INSPECTION.

6 / S4.20 TIE DOWN SCHEDULE

D10



1 EXTERIOR ELEVATION - NORTH
1/8" = 1'-0"



2 EXTERIOR ELEVATION - EAST
1/8" = 1'-0"

SHEET NOTES

LEGEND

FINISH SCHEDULE

◇	WINDOW TAG - SEE SHEET A8.21
	FIBER CEMENT LAP SIDING - WALL TYPE S
	FIBER CEMENT BOARD AND BATTEN SIDING - WALL TYPE V

KEYNOTES

06 40 13.06	EXTERIOR THERMALLY TREATED WOOD RAIL
07 31 13.01	ASPHALT SHINGLES
07 62 00.14	DOWNSPOUT, SEE PLUMBING DRAWINGS
08 53 13.06	VINYL MAS-FIN WINDOW

architects
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San Francisco, CA 94111
415 293 5700
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CHESTNUT SQUARE
1665 CHESTNUT STREET
LIVERMORE, CA

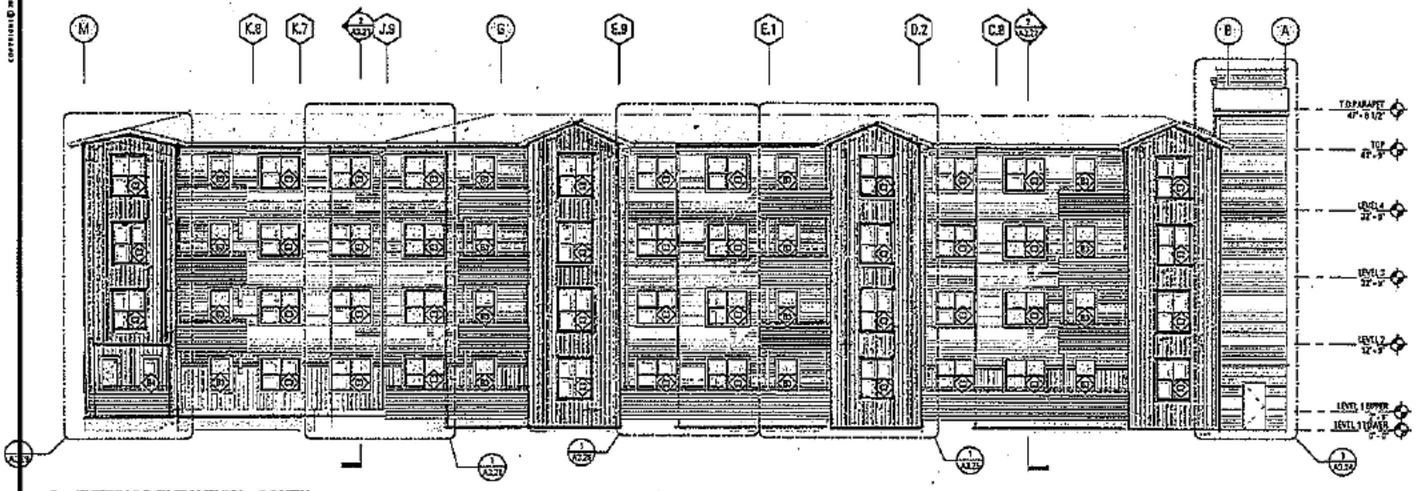
[redacted]

EXTERIOR ELEVATIONS

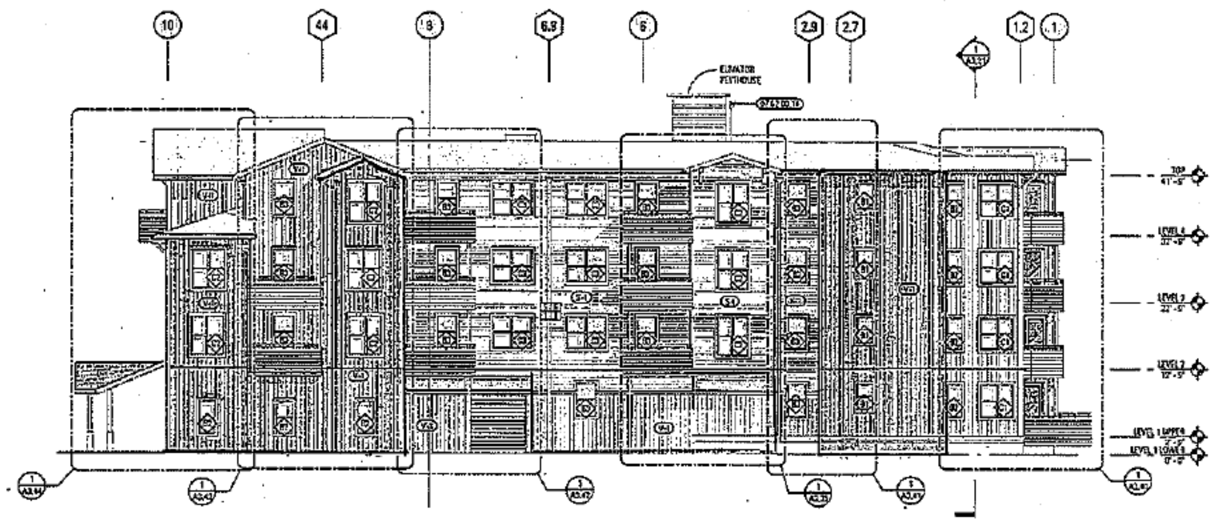
SCALE: As indicated

A3.01

D12



EXTERIOR ELEVATION - SOUTH
1/8" = 1'-0"



EXTERIOR ELEVATION - FAMILY WEST
1/8" = 1'-0"

SHEET NOTES

LEGEND

FINISH SCHEDULE

- WINDOW TAG - SEE SHEET A3.22
- FIBERCEMENT LAP SIDING - WALL TYPE S
- FIBERCEMENT BOARD AND BATTEN SIDING - WALL TYPE V

KEYNOTES

07 62 00.14 DOWNSPOUT, SEE PLUMBING DRAWINGS

architects

CHESTNUT SQUARE

1665 CHESTNUT STREET
LIVERMORE, CA

[redacted]

EXTERIOR ELEVATIONS

SCALE As indicated

A3.02

Shearwall & Tiedown Calculation

E/W Direction

4TH FLOOR (E/W DIRECTION)

Wall		h [FT]	LENGTH [FT]	WIDTH [FT]	H/W ratio	SHEAR [PLF]	M [K-FT]	ARM [FT]	T/C INCREMENTAL [K]	T/C ABOVE [K]	CUMMULATIVE T/C [K]	TIE DOWN	
4.6+4.6	10	9	9.2	14.67		262	21.7	13.67	1.6	0.0	1.6	T33	3-STORY/BM
-													
-													
-													
20	8.9	9	20	20	0.45	220	41.2	19.00	2.2	0.0	2.2	T40	4-STORY
29	8.9	9	29	29	0.31	229	59.8	28.00	2.1	0.0	2.1	T33	3-STORY/HSS
12.3	6.9	9	12.3	12.3	0.73	243	26.9	11.30	2.4	0.0	2.4	T41	4-STORY
-													
24.8	6.9	9	24.8	24.8	0.36	243	54.2	23.80	2.3	0.0	2.3	T41	4-STORY
24.8	6	9	24.8	24.8	0.36	224	50.0	23.80	2.1	0.0	2.1	T41	4-STORY
-													
13.6	6	9	13.6	13.6	0.66	224	27.4	12.60	2.2	0.0	2.2	T41	4-STORY
14.5	2.9	9	14.5	14.5	0.62	220	28.7	13.50	2.1	0.0	2.1	T40	4-STORY
17.3	2.9	9	17.3	17.3	0.52	220	34.3	16.30	2.1	0.0	2.1	T40	4-STORY
25	2.9	9	25	25	0.36	220	49.5	24.00	2.1	0.0	2.1	T40	4-STORY
-													
29.7	2.9	9	29.7	29.7	0.30	220	58.8	28.70	2.0	0.0	2.0	T40	4-STORY
-													
25	2.7	9	25	25	0.36	189	42.5	24.00	1.8	0.0	1.8	T40	4-STORY
-													
41.8	2.7	9	41.8	41.8	0.22	189	71.1	40.80	1.7	0.0	1.7	T40	4-STORY
40	2.7	9	40	40	0.23	189	68.0	39.00	1.7	0.0	1.7	T40	4-STORY
-													
4.7+4	1.2	9	8.7	12		221	17.3	11.00	1.6	0.0	1.6	T40	4-STORY
4+4	1.2	9	8	14.67		221	15.9	13.67	1.2	0.0	1.2	T40	4-STORY
4+5.4	1.2	9	9.4	12.5		221	18.7	11.50	1.6	0.0	1.6	T40	4-STORY
3.9+3.9	1.2	9	7.8	13.5		221	15.5	12.50	1.2	0.0	1.2	T40	4-STORY

Shearwall & Tiedown Calculation

3RD FLOOR (E/W DIRECTION)

Wall		h [FT]	LENGTH [FT]	WIDTH [FT]	H/W ratio	SHEAR [PLF]	M [K-FT]	ARM [FT]	T/C INCREMENTAL [K]	T/C ABOVE [K]	CUMMULATIVE T/C [K]	TIE DOWN	SHEAR TYPE
4.6+4.6	10	9	9.2	14.67		341	28.2	13.67	2.1	1.6	3.7	T33	3-STORY/BM
9.75+4.83_L	10	9	14.58	17.08		341	44.7	16.08	2.8	0.0	2.8	T22	2-STORY/BM
9.75+4.83_R	10	9	14.58	17.08		341	44.7	16.08	2.8	1.6	4.4	T33	3-STORY/BM
-													
20	8.9	9	20	20	0.45	480	86.4	19.00	4.5	2.2	6.7	T40	4-STORY
29	8.9	9	29	29	0.31	480	125.3	28.00	4.5	2.1	6.6	T33	3-STORY/HSS
12.3	6.9	9	12.3	12.3	0.73	485	53.7	11.30	4.8	2.4	7.1	T41	4-STORY
-													
24.8	6.9	9	24.8	24.8	0.36	485	108.3	23.80	4.5	2.3	6.8	T41	4-STORY
24.8	6	9	24.8	24.8	0.36	448	100.0	23.80	4.2	2.1	6.3	T41	4-STORY
-													
13.6	6	9	13.6	13.6	0.66	448	54.8	12.60	4.4	2.2	6.5	T41	4-STORY
14.5	2.9	9	14.5	14.5	0.62	334	43.6	13.50	3.2	2.1	5.4	T40	4-STORY
17.3	2.9	9	17.3	17.3	0.52	334	52.0	16.30	3.2	2.1	5.3	T40	4-STORY
25	2.9	9	25	25	0.36	334	75.2	24.00	3.1	2.1	5.2	T40	4-STORY
27.4	2.9	9	27.4	27.4	0.33	334	82.4	26.40	3.1	0.0	3.1	T31	3-STORY
29.7	2.9	9	29.7	29.7	0.30	334	89.3	28.70	3.1	2.0	5.2	T40	4-STORY
-													
25	2.7	9	25	25	0.36	331	74.5	24.00	3.1	1.8	4.9	T40	4-STORY
-													
41.8	2.7	9	41.8	41.8	0.22	331	124.5	40.80	3.1	1.7	4.8	T40	4-STORY
40	2.7	9	40	40	0.23	331	119.2	39.00	3.1	1.7	4.8	T40	4-STORY
14.4	2.7	9	14.4	14.4	0.63	331	42.9	13.40	3.2	0.0	3.2	T31	3-STORY
4.7+4	1.2	9	8.7	12		441	34.5	11.00	3.1	1.6	4.7	T40	4-STORY
4+4	1.2	9	8	14.67		441	31.8	13.67	2.3	1.2	3.5	T40	4-STORY
4+5.4	1.2	9	9.4	12.5		441	37.3	11.50	3.2	1.6	4.9	T40	4-STORY
3.9+3.9	1.2	9	7.8	13.5		441	31.0	12.50	2.5	1.2	3.7	T40	4-STORY

2ND FLOOR (E/W DIRECTION)

Wall		h (FT)	LENGTH (FT)	WIDTH (FT)	H/W ratio	SHEAR (PLF)	M (K-FT)	ARM (FT)	T/C INCREMENTAL (K)	T/C ABOVE (K)	CUMMULATIVE T/C (K)	TIE DOWN	SHEAR TYPE
4.6+4.6	10	9	9.2	14.67		453	37.5	13.67	2.7	3.7	6.4	T33	3-STORY/SM
9.75+4.83_L	10	9	14.58	17.08		453	59.4	16.08	3.7	2.8	6.5	T22	2-STORY/SM
9.75+4.83_R	10	9	14.58	17.08		453	59.4	16.08	3.7	4.4	8.1	T33	3-STORY/SM
-													
20	8.9	9	20	20	0.45	649	116.8	19.00	6.1	6.7	12.9	T40	4-STORY
29	8.9	9	29	29	0.31	649	169.4	28.00	6.0	6.6	12.7	T33	3-STORY/HSS
12.3	6.9	9	12.3	12.3	0.73	655	72.5	11.30	6.4	7.1	13.5	T41	4-STORY
-													
24.8	6.9	9	24.8	24.8	0.36	655	146.2	23.80	6.1	6.8	13.0	T41	4-STORY
24.8	6	9	24.8	24.8	0.36	605	135.0	23.80	5.7	6.3	12.0	T41	4-STORY
-													
13.6	6	9	13.6	13.6	0.66	605	74.1	12.60	5.9	6.5	12.4	T41	4-STORY
14.5	2.9	9	14.5	14.5	0.62	450	58.7	13.50	4.4	5.4	9.7	T40	4-STORY
17.3	2.9	9	17.3	17.3	0.52	450	70.1	16.30	4.3	5.3	9.6	T40	4-STORY
25	2.9	9	25	25	0.36	450	101.3	24.00	4.2	5.2	9.4	T40	4-STORY
27.4	2.9	9	27.4	27.4	0.33	450	111.0	26.40	4.2	3.1	7.3	T31	3-STORY
29.7	2.9	9	29.7	29.7	0.30	450	120.3	28.70	4.2	5.2	9.4	T40	4-STORY
-													
25	2.7	9	25	25	0.36	447	100.6	24.00	4.2	4.9	9.1	T40	4-STORY
-													
41.8	2.7	9	41.8	41.8	0.22	447	168.2	40.80	4.1	4.8	8.9	T40	4-STORY
40	2.7	9	40	40	0.23	447	160.9	39.00	4.1	4.8	8.9	T40	4-STORY
14.4	2.7	9	14.4	14.4	0.63	447	57.9	13.40	4.3	3.2	7.5	T31	3-STORY
4.7+4	1.2	9	8.7	12		595	46.6	11.00	4.2	4.7	8.9	T40	4-STORY
4+4	1.2	9	8	14.67		595	42.8	13.67	3.1	3.5	6.6	T40	4-STORY
4+5.4	1.2	9	9.4	12.5		595	50.3	11.50	4.4	4.9	9.2	T40	4-STORY
3.9+3.9	1.2	9	7.8	13.5		595	41.8	12.50	3.3	3.7	7.1	T40	4-STORY

Shearwall & Tiedown Calculation

1st FLOOR (E/W DIRECTION)

Wall		h (FT)	LENGTH (FT)	WIDTH (FT)	H/W ratio	SHEAR (PLF)	M (K-FT)	ARM (FT)	T/C INCREMENTAL (K)	T/C ABOVE (K)	CUMMULATIVE T/C (K)	TIE DOWN	SHEAR TYPE
4.6+4.6	10	9	9.2	14.67		0	0.0	13.67	0.0	6.4	6.4	SEE ELEV	
9.75+4.83_L	10	9	14.58	17.08		0	0.0	16.08	0.0	6.5	6.5	SEE ELEV	
9.75+4.83_R	10	9	14.58	17.08		0	0.0	16.08	0.0	8.1	8.1	SEE ELEV	
6	10	11.75	6	6	1.96	687	48.4	5.00	9.7	0.0	9.7	T11	1-STORY
20/16.5	8.9	9	16.5	16.5	0.55	758	112.6	15.50	7.3	12.9	20.1	T41	4-STORY
29	8.9	9	29	29	0.31	0	0.0	28.00	0.0	12.7	12.7	HSS	3-STORY/HSS
12.3/24.4	6.9	9	24.4	24.4	0.37	570	125.2	23.40	5.3	13.5	18.9	T41	4-STORY
24.4	6.9	9	24.4	24.4	0.37	570	125.2	23.40	5.3	0.0	5.3	T11	1-STORY
24.8	6.9	9	24.8	24.8	0.36	570	127.2	23.80	5.3	13.0	18.3	T41	4-STORY
24.8	6	9	24.8	24.8	0.36	523	116.7	23.80	4.9	12.0	16.9	T41	4-STORY
24.4	6	9	24.4	24.4	0.37	523	114.8	23.40	4.9	0.0	4.9	T11	1-STORY
13.6	6	9	13.6	13.6	0.66	523	64.0	12.60	5.1	12.4	17.5	T41	4-STORY
14.5	2.9	9	14.5	14.5	0.62	440	57.4	13.50	4.3	9.7	14.0	T40	4-STORY
17.3	2.9	9	17.3	17.3	0.52	440	68.5	16.30	4.2	9.6	13.8	T40	4-STORY
25	2.9	9	25	25	0.36	440	99.0	24.00	4.1	9.4	13.5	T40	4-STORY
27.4	2.9	9	27.4	27.4	0.33	440	108.5	26.40	4.1	7.3	11.4	T31	3-STORY
29.7	2.9	9	29.7	29.7	0.30	440	117.6	28.70	4.1	9.4	13.4	T40	4-STORY
15.2	2.9	9	15.2	15.2	0.59	440	69.2	14.20	4.2	0.0	4.2	T11	1-STORY
25	2.7	9	25	25	0.36	465	104.6	24.00	4.4	9.1	13.4	T40	4-STORY
12.2	2.7	9	12.2	12.2	0.74	465	51.1	11.20	4.6	0.0	4.6	T11	1-STORY
41.8	2.7	9	41.8	41.8	0.22	465	174.9	40.80	4.3	8.9	13.2	T40	4-STORY
40	2.7	9	40	40	0.23	465	167.4	39.00	4.3	8.9	13.2	T40	4-STORY
14.4	2.7	9	14.4	14.4	0.63	465	60.3	13.40	4.5	7.5	12.0	T31	3-STORY
4.7+4	1.2	9	8.7	12		679	53.2	11.00	4.8	8.9	13.8	T40	4-STORY
4+4	1.2	9	8	14.67		679	48.9	13.67	3.6	6.6	10.2	T40	4-STORY
4+5.4	1.2	9	9.4	12.5		679	57.4	11.50	5.0	9.2	14.2	T40	4-STORY
3.9+3.9	1.2	9	7.8	13.5		679	47.7	12.50	3.8	7.1	10.9	T40	4-STORY
-													
16.5	8.9	11.75	16.5	16.5	0.71	758	147.0	15.50	9.5	0.0	9.5	T11	1-STORY
15	8.9	11.75	15	15	0.78	758	133.6	14.00	9.5	0.0	9.5	T11	1-STORY
4.7+5	10	11.75	9.7	11.95	1.21	687	78.3	10.95	7.2	7.1	14.2	HSS	1-STORY
4.2+5	10	11.75	9.2	12.2	1.28	687	74.3	11.20	6.8	0.0	6.8	T11/HSS	1-STORY

Shearwall & Tiedown Calculation

N/S Direction

4TH FLOOR (N/S DIRECTION)

Wall		h [FT]	LENGTH [FT]	WIDTH [FT]	H/W ratio	SHEAR [PLF]	M [K-FT]	ARM [FT]	T/C INCREMENTAL [K]	T/C ABOVE [K]	CUMMULATIVE T/C [K]	TIE DOWN	
-													
3.7+4	M	9	7.7	10.9		230	15.9	9.90	1.6	0.0	1.6	T33/BM	See elev
3.4+3.4	M.1	9	6.8	12.5		230	14.1	11.50	1.2	0.0	1.2	T31	See elev
4.4+4.9	M	9	9.3	12.5		230	19.3	11.50	1.7	0.0	1.7	T33/BM	See elev
4.1+5.3	M	9	9.8	12.67		230	20.3	11.67	1.7	0.0	1.7	T40	4-STORY
4.6+11	M	9	15.6	18.83		230	32.3	17.83	1.8	0.0	1.8	T40	4-STORY
20.2	k.8	9	20.2	20.2	0.45	249	45.3	19.20	2.4	0.0	2.4	T41	4-STORY
13.2	k.8	9	13.2	13.2	0.68	249	29.6	12.20	2.4	0.0	2.4	T41	4-STORY
9.1	k.8	9	9.1	9.1	0.99	249	20.4	8.10	2.5	0.0	2.5	T41	4-STORY
13.4	k.8	9	13.4	13.4	0.67	249	30.0	12.40	2.4	0.0	2.4	T41	4-STORY
13.2	k.7	9	13.2	13.2	0.68	259	30.8	12.20	2.5	0.0	2.5	T33	4-STORY
6.9	k.7	9	6.9	6.9	1.30	259	16.1	5.90	2.7	0.0	2.7	T42	4-STORY
12	k.7	9	12	12	0.75	259	28.0	11.00	2.5	0.0	2.5	T42	4-STORY
14.2	k.7	9	14.2	14.2	0.63	259	33.1	13.20	2.5	0.0	2.5	T42	4-STORY
17.8	J.9	9	17.8	17.8	0.51	261	45.0	16.80	2.7	0.0	2.7	T41	4-STORY
-													
-													
3.7+4	G.2	9	7.7	10.92	1.17	224	15.5	9.92	1.6	0.0	1.6	T33	3-STORY/HSS
3.9+3.9	G	9	7.8	13.5	1.15	224	15.7	12.50	1.3	0.0	1.3	T40	4-STORY
3.4+5.6	G.2	9	9	12	1.00	224	18.1	11.00	1.6	0.0	1.6	T41	4-STORY
13	G.2	9	13	13	0.69	224	26.2	12.00	2.2	0.0	2.2	T41	4-STORY
17.8	G.2	9	17.8	17.8	0.51	224	35.9	16.80	2.1	0.0	2.1	T41	4-STORY
24.4	E.9	9	24.4	24.4	0.37	336	73.8	23.40	3.2	0.0	3.2	T41	4-STORY
-													
24.4	E.1	9	24.4	24.4	0.37	328	72.0	23.40	3.1	0.0	3.1	T41	4-STORY
-												T21	
24.4	D.2	9	24.4	24.4	0.37	369	81.0	23.40	3.5	0.0	3.5	T42	4-STORY
-													
24.4	C.8	9	24.4	24.4	0.37	291	63.9	23.40	2.7	0.0	2.7	T41	4-STORY
-													
10	8	9	10	10	0.90	323	29.1	9.00	3.2	0.0	3.2	T41	4-STORY
17.5	8	9	17.5	17.5	0.51	323	50.8	16.50	3.1	0.0	3.1	T41	4-STORY
-													

Shearwall & Tiedown Calculation

3RD FLOOR (N/S DIRECTION)

Wall		h [FT]	LENGTH [FT]	WIDTH [FT]	H/W ratio	SHEAR [PLF]	M [K-FT]	ARM [FT]	T/C INCREMENTAL [K]	T/C ABOVE [K]	CUMMULATIVE T/C [K]	TIE DOWN	
3.1+3.1	M.1	9	6.2	12.1		408	22.8	11.10	2.1	0.0	2.1	T31	See elev
4+4	M.1	9	7.7	10.9		408	28.3	9.90	2.9	1.6	4.5	T33/BM	See elev
3.4+3.4	M.1	9	6.8	12.5		408	25.0	11.50	2.2	1.2	3.4	T31	See elev
4.4+4.9	M.1	9	9.3	12.5		408	34.1	11.50	3.0	1.7	4.6	T33/BM	See elev
4.1+5.3	M	9	9.8	12.67		408	36.0	11.67	3.1	1.7	4.8	T40	4-STORY
4.8+11	M	9	15.6	18.83		408	57.3	17.83	3.2	1.8	5.0	T40	4-STORY
20.2	k.8	9	20.2	20.2	0.45	503	91.4	19.20	4.8	2.4	7.1	T41	4-STORY
13.2	k.8	9	13.2	13.2	0.68	503	59.8	12.20	4.9	2.4	7.3	T41	4-STORY
9.1	k.8	9	9.1	9.1	0.99	503	41.2	8.10	5.1	2.5	7.6	T41	4-STORY
13.4	k.8	9	13.4	13.4	0.67	503	60.7	12.40	4.9	2.4	7.3	T41	4-STORY
13.2	k.7	9	13.2	13.2	0.68	535	63.6	12.20	5.2	2.5	7.7	T33	4-STORY
6.9	k.7	9	6.9	6.9	1.30	535	33.2	5.90	5.6	2.7	8.4	T42	4-STORY
12	k.7	9	12	12	0.75	535	57.8	11.00	5.3	2.5	7.8	T42	4-STORY
14.2	k.7	9	14.2	14.2	0.63	535	68.4	13.20	5.2	2.5	7.7	T42	4-STORY
17.8	J.9	9	17.8	17.8	0.51	562	90.0	16.80	5.4	2.7	8.0	T41	4-STORY
-													
3.2+3.2	G	9	6.4	12.1	1.41	413	23.8	11.10	2.1	0.0	2.1	T31	3-STORY
3.7+4	G.2	9	7.7	10.92	1.17	413	28.6	9.92	2.9	1.6	4.5	T33	3-STORY/HSS
3.9+3.9	G	9	7.8	13.5	1.15	413	29.0	12.50	2.3	1.3	3.6	T40	4-STORY
3.4+5.6	G.2	9	9	12	1.00	413	33.5	11.00	3.0	1.6	4.7	T41	4-STORY
13	G.2	9	13	13	0.69	413	48.3	12.00	4.0	2.2	6.2	T41	4-STORY
17.8	G.2	9	17.8	17.8	0.51	413	66.2	16.80	3.9	2.1	6.1	T41	4-STORY
24.4	E.9	9	24.4	24.4	0.37	672	147.6	23.40	6.3	3.2	9.5	T41	4-STORY
-													
24.4	E.1	9	24.4	24.4	0.37	660	144.9	23.40	6.2	3.1	9.3	T41	4-STORY
-												T21	
24.4	D.2	9	24.4	24.4	0.37	746	163.8	23.40	7.0	3.5	10.5	T42	4-STORY
-													
24.4	C.8	9	24.4	24.4	0.37	574	126.1	23.40	5.4	2.7	8.1	T41	4-STORY
-													
10	8	9	10	10	0.90	407	36.6	9.00	4.1	3.2	7.3	T41	4-STORY
17.5	8	9	17.5	17.5	0.51	407	64.1	16.50	3.9	3.1	7.0	T41	4-STORY
16	8	9	16	16	0.56	407	58.6	15.00	3.9	0.0	3.9		3-STORY

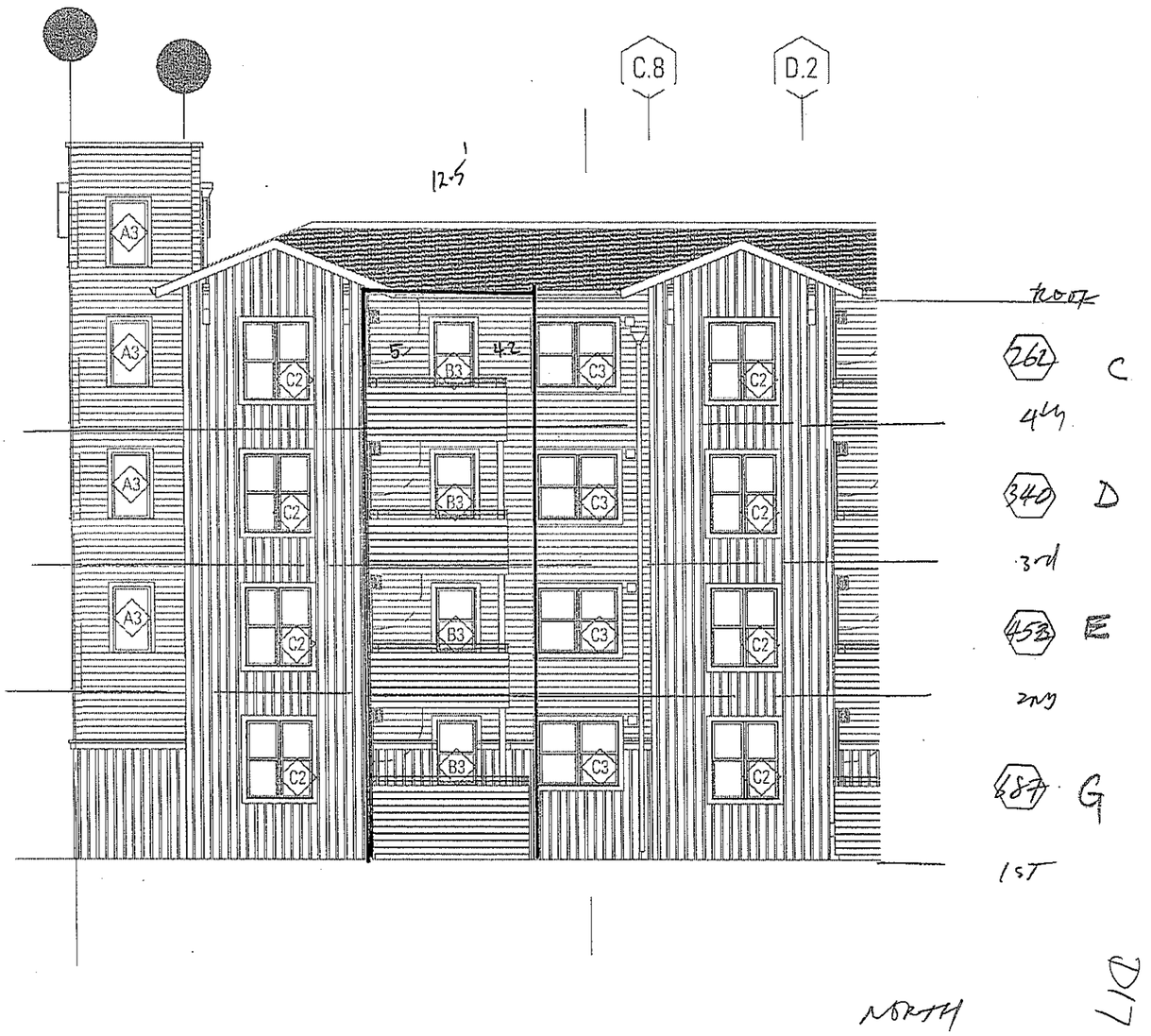
2ND FLOOR (N/S DIRECTION)

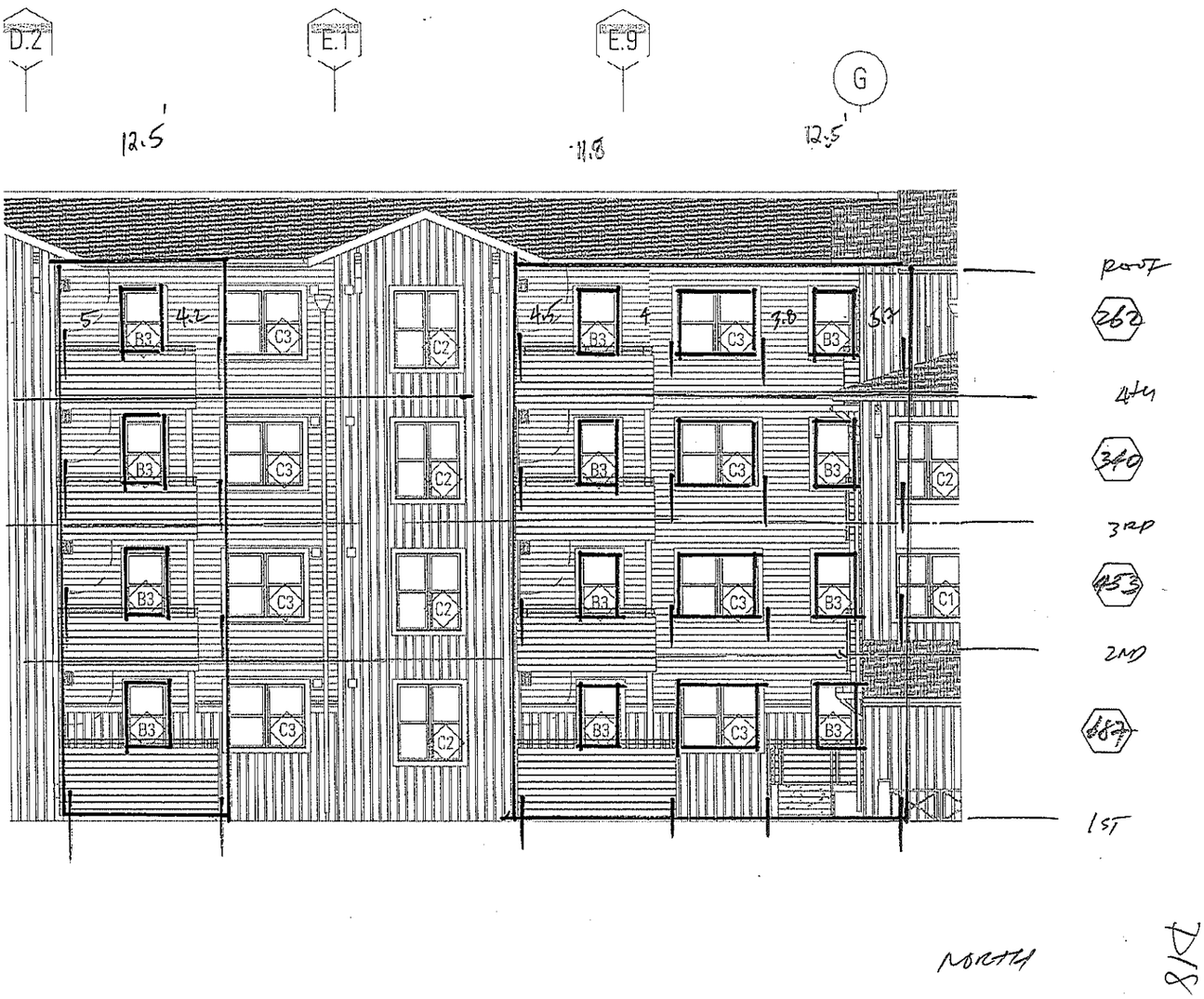
Wall		h [FT]	LENGTH [FT]	WIDTH [FT]	H/W ratio	SHEAR [PLF]	M [K-FT]	ARM [FT]	T/C INCREMENTAL [K]	T/C ABOVE [K]	CUMMULATIVE T/C [K]	TIE DOWN	
3.1+3.1	M.1	9	6.2	12.1		549	30.6	11.10	2.8	2.1	4.8	T31	See elev
3.7+4	M.1	9	7.7	10.9		549	38.0	9.90	3.6	4.5	8.3	T33/BM	See elev
3.4+3.4	M.1	9	6.8	12.5		549	33.6	11.50	2.9	3.4	6.3	T31	See elev
4.4+4.9	M.1	9	9.3	12.5		549	46.0	11.50	4.0	4.6	8.6	T33/BM	See elev
4.1+5.3	M	9	9.8	12.67		549	48.4	11.67	4.1	4.8	9.0	T40	4-STORY
4.6+11	M	9	15.6	18.83		549	77.1	17.83	4.3	5.0	9.3	T40	4-STORY
20.2	k.8	9	20.2	20.2	0.45	677	123.1	19.20	6.4	7.1	13.5	T41	4-STORY
13.2	k.8	9	13.2	13.2	0.68	677	80.4	12.20	6.6	7.3	13.9	T41	4-STORY
9.1	k.8	9	9.1	9.1	0.99	677	55.4	8.10	6.8	7.6	14.4	T41	4-STORY
13.4	k.8	9	13.4	13.4	0.67	677	81.6	12.40	6.6	7.3	13.9	T41	4-STORY
13.2	k.7	9	13.2	13.2	0.68	467	56.5	12.20	4.5	7.7	12.3	T33	4-STORY
6.9	k.7	9	6.9	6.9	1.30	467	29.0	5.90	4.9	8.4	13.3	T42	4-STORY
12	k.7	9	12	12	0.75	467	50.4	11.00	4.6	7.8	12.4	T42	4-STORY
14.2	k.7	9	14.2	14.2	0.63	467	59.7	13.20	4.5	7.7	12.2	T42	4-STORY
17.8	J.9	9	17.8	17.8	0.51	469	75.1	16.80	4.5	8.0	12.5	T41	4-STORY
11	J.9	9	11	11	0.82	469	46.4	10.00	4.8	0.0	4.8	T21	2-STORY
3.2+3.2	G	9	6.4	12.1	1.41	559	32.2	11.10	2.9	2.1	5.0	T31	3-STORY
3.7+4	G.2	9	7.7	10.92	1.17	559	38.7	9.92	3.9	4.5	8.4	T33	3-STORY/HSS
3.9+3.9	G	9	7.8	13.5	1.15	559	39.2	12.50	3.1	3.6	6.7	T40	4-STORY
3.4+5.6	G.2	9	9	12	1.00	559	45.3	11.00	4.1	4.7	8.8	T41	4-STORY
13	G.2	9	13	13	0.69	559	65.4	12.00	5.5	6.2	11.7	T41	4-STORY
17.8	G.2	9	17.8	17.8	0.51	559	89.6	16.80	5.3	6.1	11.4	T41	4-STORY
24.4	E.9	9	24.4	24.4	0.37	526	115.5	23.40	4.9	9.5	14.4	T41	4-STORY
17.8	E.9	9	17.8	17.8	0.51	526	84.3	16.80	5.0	0.0	5.0	T21	2-STORY
24.4	E.1	9	24.4	24.4	0.37	541	118.8	23.40	5.1	9.3	14.3	T41	4-STORY
15.7	E.1	9	15.7	15.7	0.57	541	76.4	14.70	5.2	0.0	5.2	T21	2-STORY
24.4	D.2	9	24.4	24.4	0.37	585	128.5	23.40	5.5	10.5	16.0	T42	4-STORY
17.8	D.2	9	17.8	17.8	0.51	585	93.7	16.80	5.6	0.0	5.6	T21	2-STORY
24.4	C.8	9	24.4	24.4	0.37	465	102.1	23.40	4.4	8.1	12.5	T41	4-STORY
16	C.8	9	16	16	0.56	465	67.0	15.00	4.5	0.0	4.5	T21	2-STORY
10	B	9	10	10	0.90	549	49.4	9.00	5.5	7.3	12.8	T41	4-STORY
17.5	B	9	17.5	17.5	0.51	549	86.5	16.50	5.2	7.0	12.2	T41	4-STORY
16	B	9	16	16	0.56	549	79.1	15.00	5.3	3.9	9.2	T31	3-STORY

Shearwall & Tiedown Calculation

1st FLOOR (N/S DIRECTION)

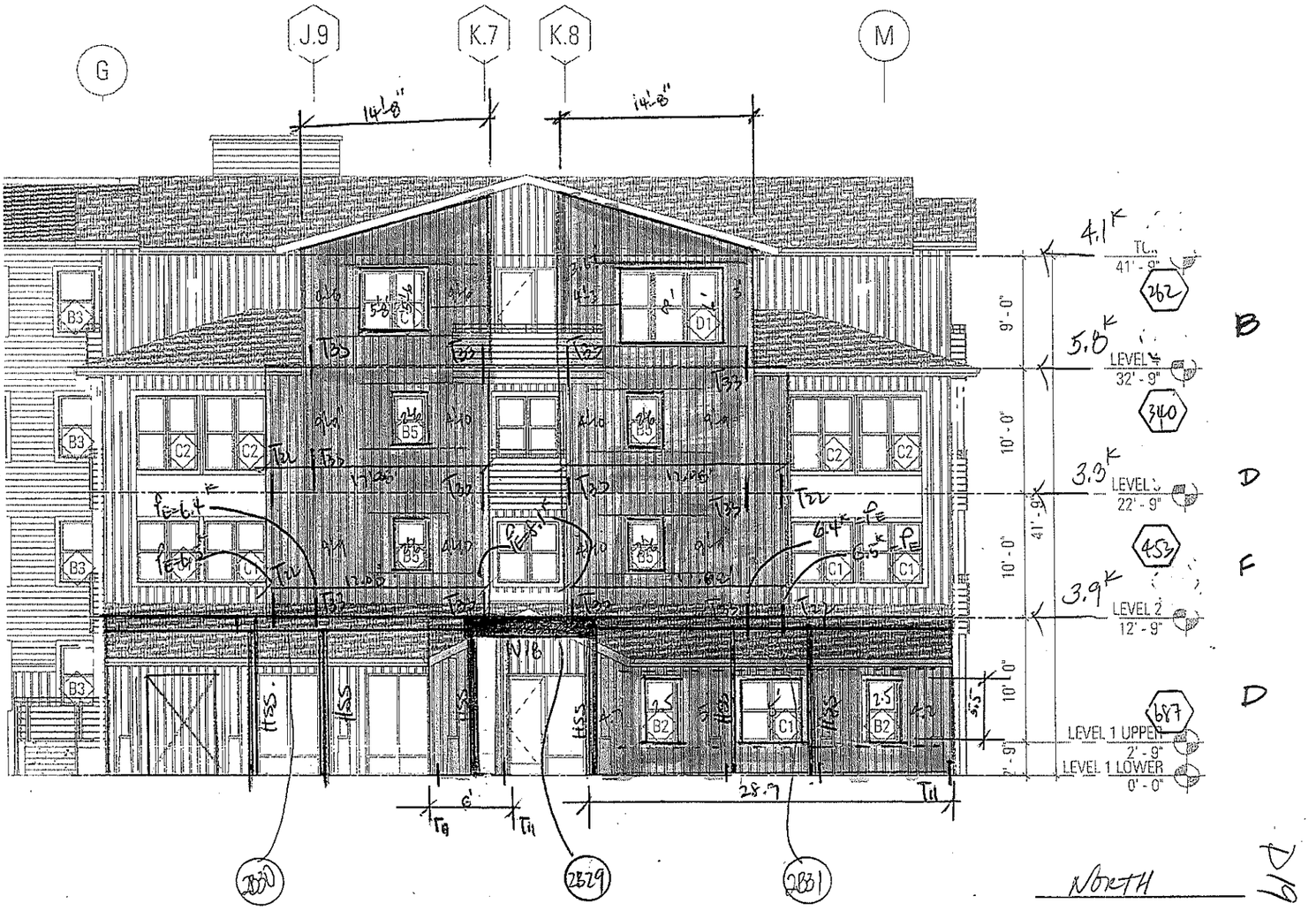
Wall		h [FT]	LENGTH [FT]	WIDTH [FT]	H/W ratio	SHEAR [PLF]	M [K-FT]	ARM [FT]	T/C INCREMENTAL [K]	T/C ABOVE [K]	CUMMULATIVE T/C [K]	TIE DOWN	
3.1+3.1	M.1	9	6.2	12.1		0	0.0	11.10	0.0	4.8	4.8	T31	See elev
3.7+4	M.1	9	7.7	10.9		0	0.0	9.90	0.0	8.3	8.3	T33/BM	See elev
3.4+3.4	M.1	9	6.8	12.5		0	0.0	11.50	0.0	6.3	6.3	T31	See elev
4.4+4.9	M.1	9	9.3	12.5		0	0.0	11.50	0.0	8.6	8.6	T33/BM	See elev
4.1+5.3/10.7	M	9	10.7	9		480	46.2	8.00	5.8	9.0	14.7	T40	4-STORY
4.6+11	M	9	15.6	18.83		480	67.4	17.83	3.8	9.3	13.1	T40	4-STORY
20.2/28.9	k.8	9	28.9	28.9	0.31	606	157.6	27.90	5.6	13.5	19.2	T41	4-STORY
13.2	k.8	9	13.2	13.2	0.68	0	0.0	12.20	0.0	13.9	13.9	T41	4-STORY
9.1	k.8	9	9.1	9.1	0.99	606	43.6	8.10	5.1	14.4	20.6	T41	4-STORY
13.4	k.8	9	13.4	13.4	0.67	606	73.1	12.40	5.9	13.9	19.8	T41	4-STORY
13.2	k.7	9	13.2	13.2	0.68	0	0.0	12.20	0.0	12.3	12.3	T33	4-STORY
6.9	k.7	9	6.9	6.9	1.30	840	52.2	5.90	8.8	13.3	22.1	T42	4-STORY
12	k.7	9	12	12	0.75	840	90.7	11.00	8.2	12.4	20.6	T42	4-STORY
14.2	k.7	9	14.2	14.2	0.63	840	107.4	13.20	8.1	12.2	20.3	T42	4-STORY
17.8	J.9	9	17.8	17.8	0.51	535	85.7	16.80	5.1	12.5	17.6	T41	4-STORY
11	J.9	9	11	11	0.82	535	53.0	10.00	5.3	4.6	9.9	T21	2-STORY
3.2+3.2	G	9	6.4	12.1	1.41	0	0.0	11.10	0.0	5.0	5.0	T31	3-STORY
3.7+4	G.2	9	7.7	10.92	1.17	0	0.0	9.92	0.0	8.4	8.4	T33	3-STORY/HSS
3.9+3.9	G	9	7.8	13.5	1.15	0	0.0	12.50	0.0	6.7	6.7	T40	4-STORY
5.6+3.5/14.9	G.2	9	14.9	12	0.60	767	102.9	11.00	9.4	8.8	18.2	T41	4-STORY
13	G.2	9	13	13	0.69	0	0.0	12.00	0.0	11.7	11.7	T41	4-STORY
17.8	G.2	9	17.8	17.8	0.51	767	122.9	16.80	7.3	11.4	18.7	T41	4-STORY
24.4	E.9	9	24.4	24.4	0.37	602	132.2	23.40	5.6	14.4	20.0	T41	4-STORY
17.8	E.9	9	17.8	17.8	0.51	602	96.4	16.80	5.7	5.0	10.8	T21	2-STORY
24.4	E.1	9	24.4	24.4	0.37	618	135.7	23.40	5.8	14.3	20.1	T41	4-STORY
15.7	E.1	9	15.7	15.7	0.57	618	87.3	14.70	5.9	5.2	11.1	T21	2-STORY
24.4	D.2	9	24.4	24.4	0.37	671	147.4	23.40	6.3	16.0	22.3	T42	4-STORY
17.8	D.2	9	17.8	17.8	0.51	671	107.5	16.80	6.4	5.6	12.0	T21	2-STORY
24.4	C.8	9	24.4	24.4	0.37	532	116.8	23.40	5.0	12.5	17.5	T41	4-STORY
15.7	C.8	9	15.7	15.7	0.57	532	75.2	14.70	5.1	4.5	9.6	T21	2-STORY
10	B	9	10	10	0.90	605	54.5	9.00	6.1	12.8	18.8	T41	4-STORY
17.5	B	9	17.5	17.5	0.51	605	95.3	16.50	5.8	12.2	18.0	T41	4-STORY
16	B	9	16	16	0.56	605	87.1	15.00	5.8	9.2	15.0	T31	3-STORY
5.2	K.9	11.75	5.2	5.2	2.26	606	37.0	4.20	8.8	0.0	8.8	T11	1-STORY
5.9	K.9	11.75	5.9	5.9	1.99	606	42.0	4.90	8.6	0.0	8.6	T11	1-STORY
10	K.9	11.75	10	10	1.18	606	71.2	9.00	7.9	0.0	7.9	T11	1-STORY
5.7	K.7	11.75	5.7	5.7	2.06	840	56.3	4.70	12.0	0.0	12.0	T11	1-STORY
7.6	K.7	9	7.6	7.6	1.18	840	57.5	6.60	8.7	0.0	8.7	HSS	1-STORY
8.4	G.2	9	8.4	8.4	1.07	747	56.5	7.40	7.6	0.0	7.6	T11	1-STORY
5.3	G	11.75	5.3	5.3	2.22	747	46.5	4.30	10.8	0.0	10.8	T11	1-STORY
6.6	G	11.75	6.6	6.6	1.78	747	57.9	5.60	10.3	0.0	10.3	T11	1-STORY
4.5+9.4/9.5+4.8	M.1	11.75	27.8	35.5		469	153.2	34.8	4.4	6.3	10.8	T31/T40	See elev
6.5	M	9	6.5	6.5	1.38	469	27.4	5.5	5.0	0.0	5.0	T11	See elev
13.2	M	9	13.2	13.2	0.68	469	55.7	12.2	4.6	0.0	4.6	T11	See elev

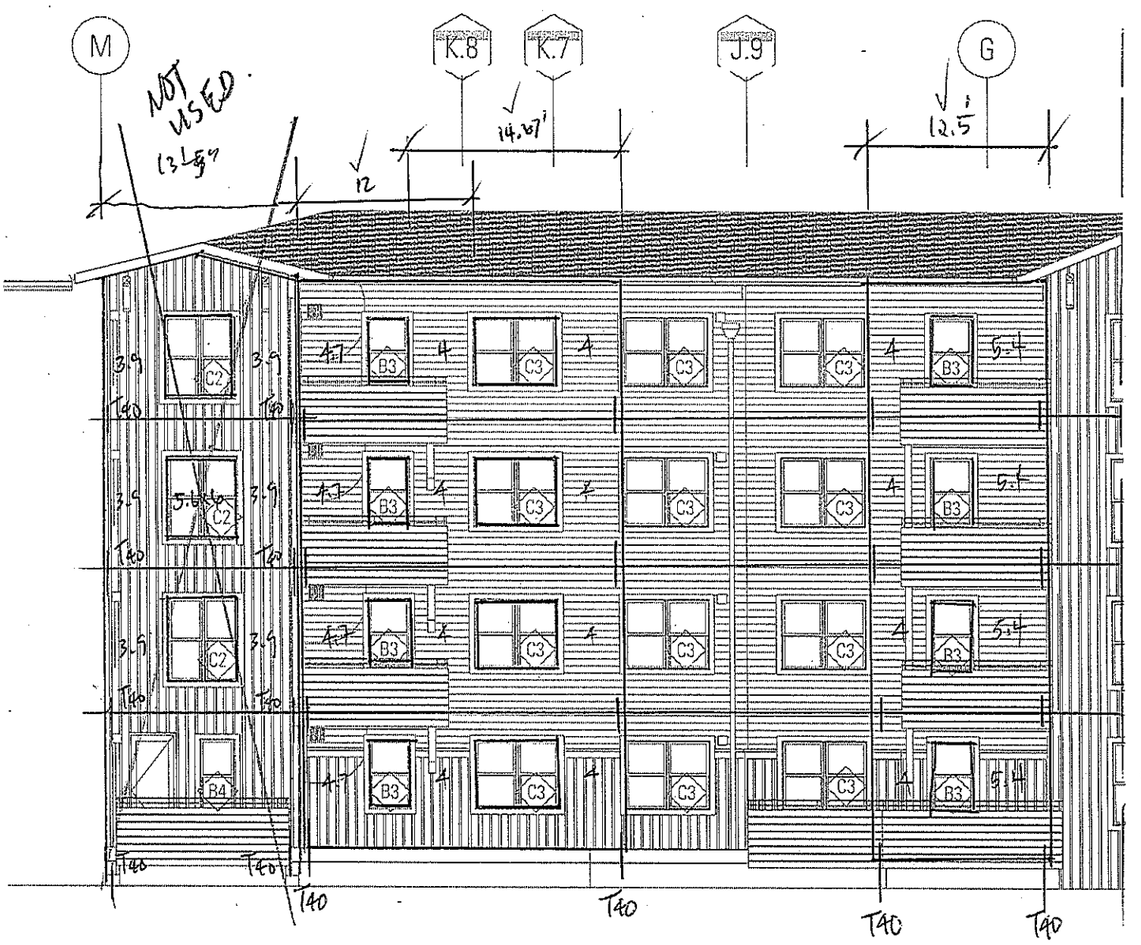

C.8
D.2
12.5'
A3
A3
A3
A3
C2
C2
C2
C2
B3
B3
B3
B3
C3
C3
C3
C3
C2
C2
C2
C2
5'
42'
ROOF
262
C
44'
340
D
32'
453
E
20'
484
G
1ST
NORTH
D17



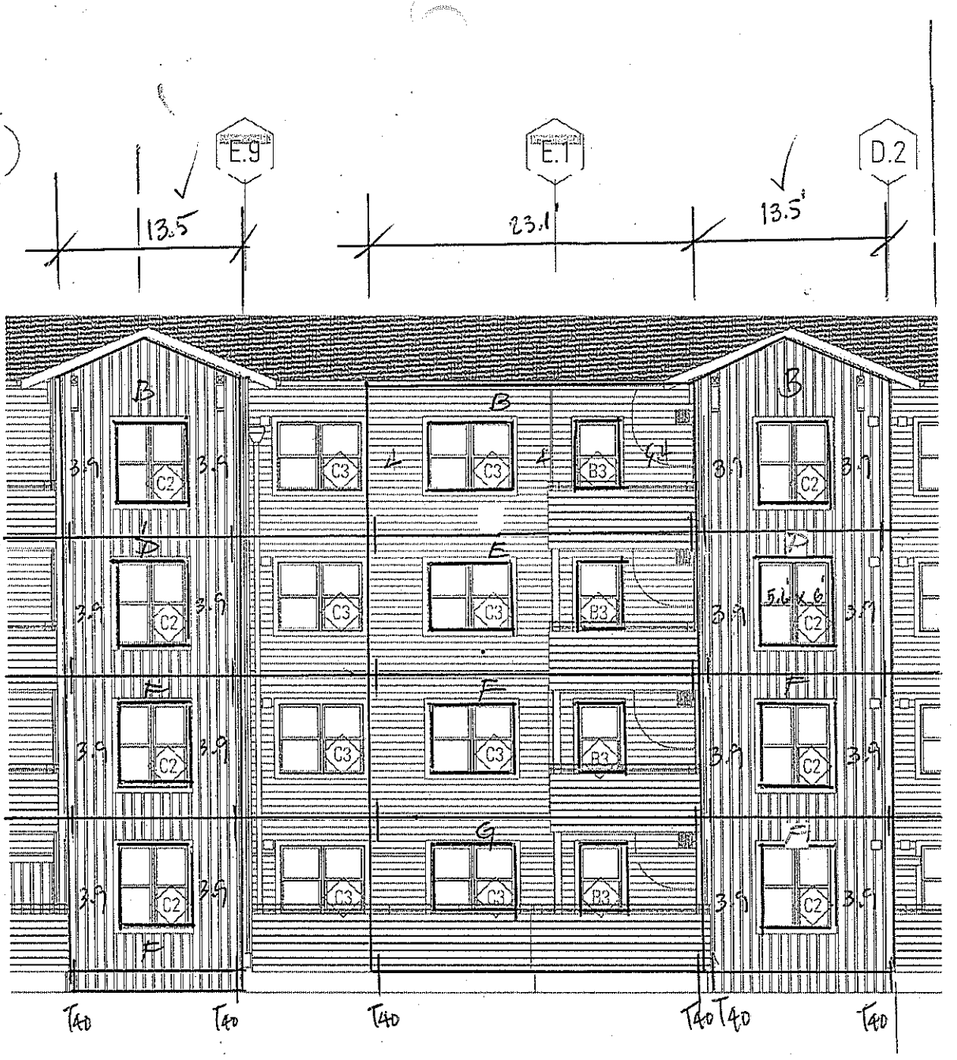
NORTH

A18


G
J.9
K.7
K.8
M
14'-8"
14'-8"
4.1K
TC.
41' - 9"
262
B
5.8K
LEVEL 3
32' - 9"
340
D
3.3K
LEVEL 2
22' - 9"
452
F
3.9K
LEVEL 2
12' - 9"
D
687
LEVEL 1 UPPER
2' - 9"
LEVEL 1 LOWER
0' - 0"
9' - 0"
10' - 0"
41' - 9"
10' - 0"
10' 0"
3'-5"
7' - 9"
B3
C2
B5
C1
D1
B2
HSS
N16
28.9
2830
2829
2831
NORTH


M
NOT USED
13'-5"
12
K.8
K.7
14.67'
J.9
G
12.5'
C2
B3
C3
B4
T40
3.9
4.7
5.4
5.1
6.14
221 B
441 E
595 F
659 G
SOUTH

D20



221

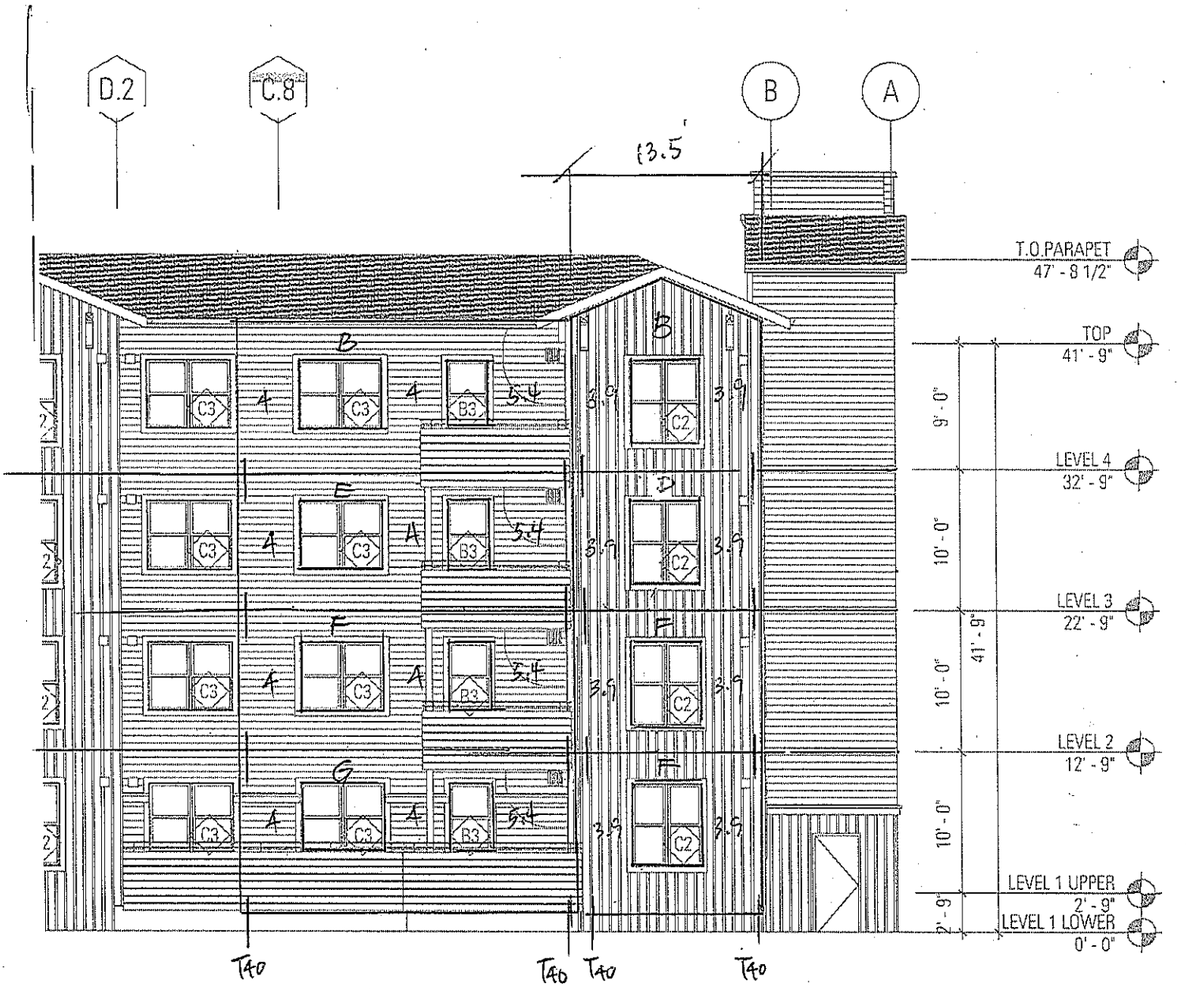
441

595

679

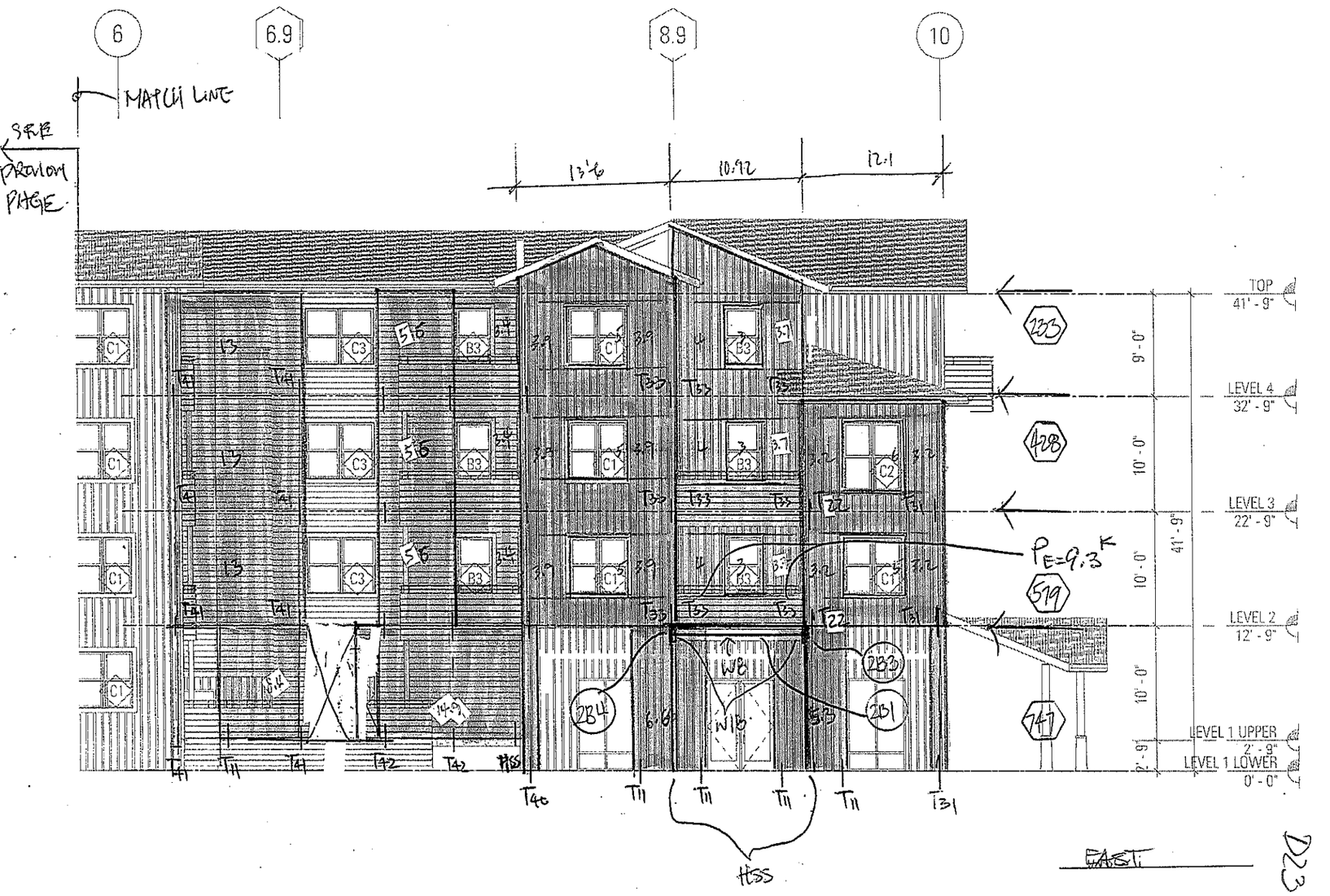
SOUTH

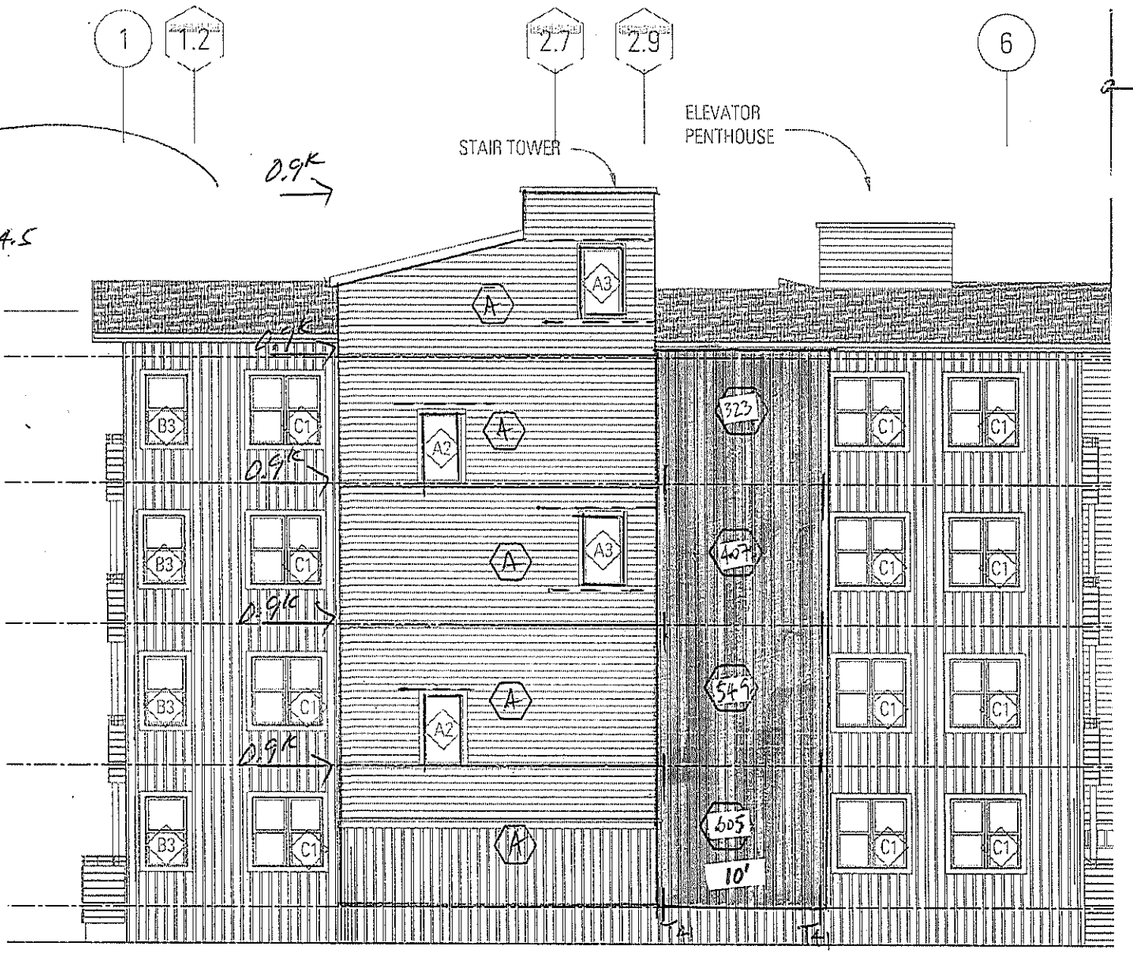
121


D.2
C.8
B
A
13.5'
T.O.PARAPET
47' - 8 1/2"
TOP
41' - 9"
LEVEL 4
32' - 9"
LEVEL 3
22' - 9"
LEVEL 2
12' - 9"
LEVEL 1 UPPER
2' - 9"
LEVEL 1 LOWER
0' - 0"
9' - 0"
10' - 0"
10' - 0"
10' - 0"
2' - 9"
41' - 9"
T40
T40
T40
T40
C3
C2
B3

SOUTH.

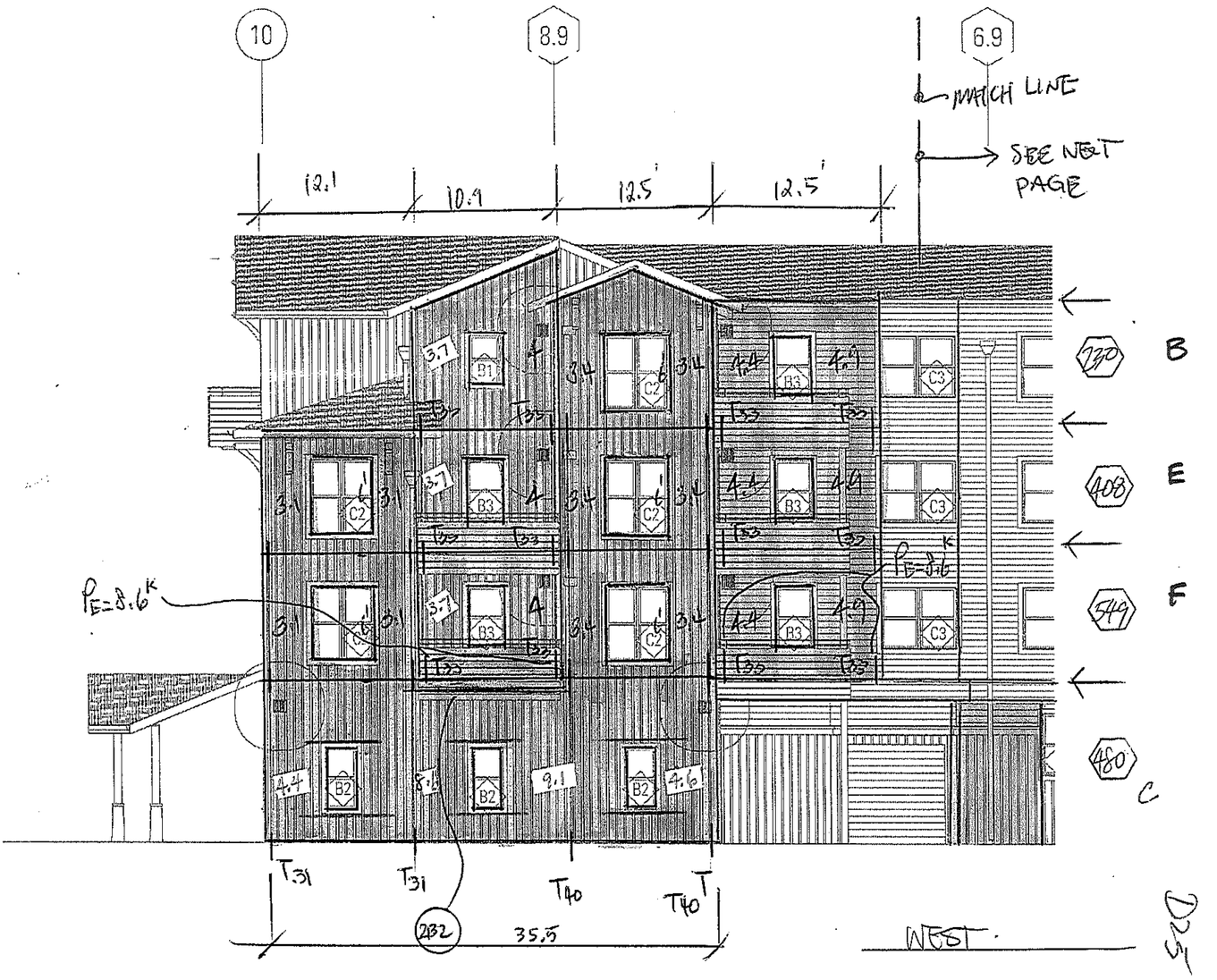
022


6
6.9
8.9
10
MATCH LINE
SEE PREVIOUS PAGE
13'6
10.92
12.1
TOP
41' - 9"
LEVEL 4
32' - 9"
LEVEL 3
22' - 9"
LEVEL 2
12' - 9"
LEVEL 1 UPPER
2' - 9"
LEVEL 1 LOWER
0' - 0"
9' - 0"
10' - 0"
10' - 0"
10' - 0"
2' - 9"
41' - 9"
P_E=9.3 K
W8
W18
HSS
EAST
D23


1
1.2
2.7
2.9
6
STAIR TOWER
ELEVATOR PENTHOUSE
MATCH LINE
SEE NEXT PAGE
@ STAIR WINDOW CONTROL
$P_{WIND} = 200^{\#} \times 4.5 = 900^{\#}$
TYPE (A) SHGR BY INSPECTION.
0.9K
0.9K
0.9K
0.9K
A
A2
A3
B3
C1
323
407
549
605
10'

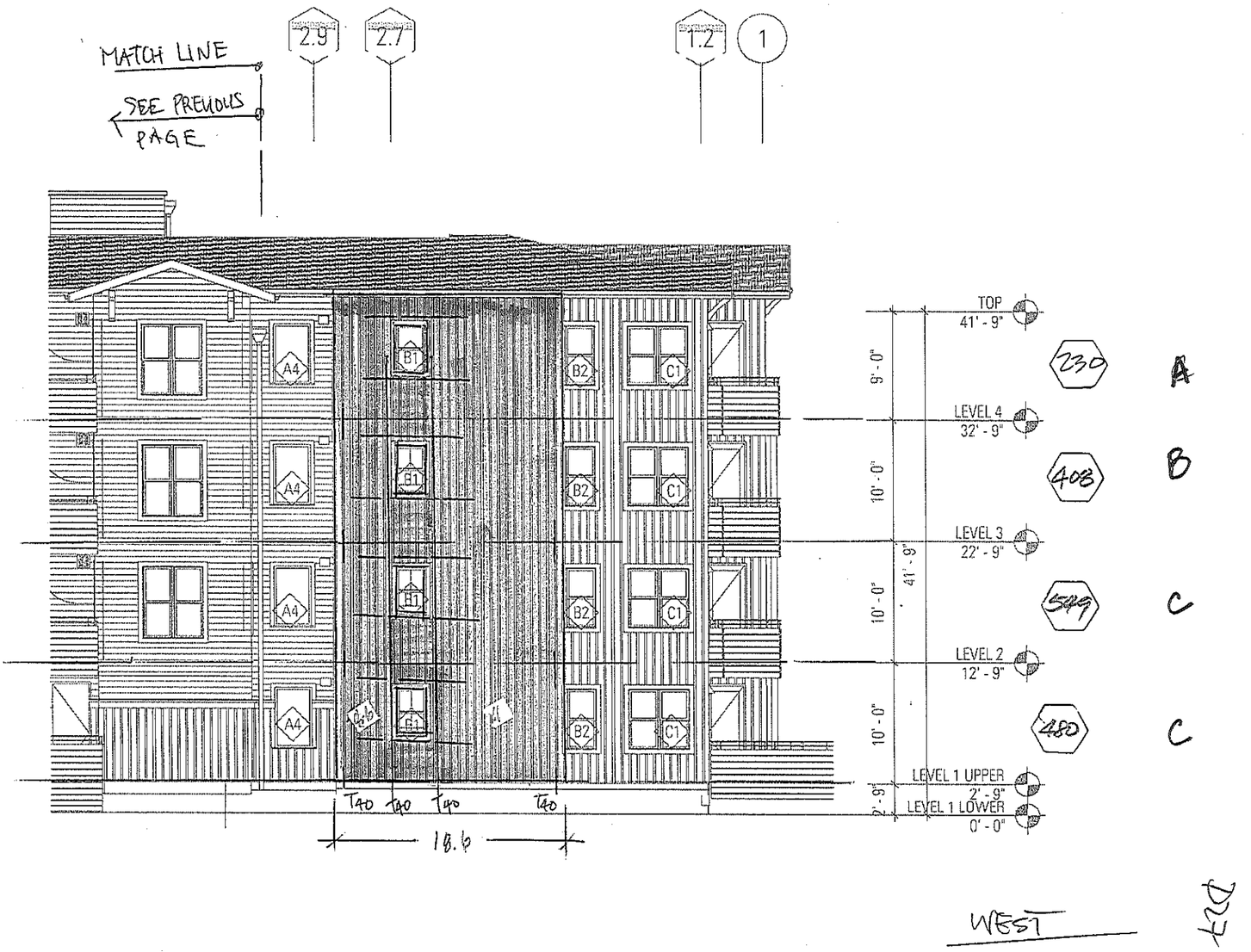
EAST

D14


10
8.9
6.9
MATCH LINE
SEE NEXT PAGE
12.1
10.9
12.5'
12.5'
B
E
F
C
$P_E = 8.6^K$
$P_E = 8.6^K$
T31
T31
T40
T40
232
35.5
WEST
D25


8
6.9
6
2.9
SEE PREVIOUS PAGE
MATCH LINE
12.67'
MATCH LINE
SEE NEXT PAGE
ROOF
4TH
3RD
2ND
1ST
B3
C3
A4
5.3
6.5
13.2'
T55
T40
T11

WEST


MATCH LINE
SEE PREVIOUS PAGE
2.9
2.7
1.2
1
A4
B1
B2
C1
TOP
41' - 9"
LEVEL 4
32' - 9"
LEVEL 3
22' - 9"
LEVEL 2
12' - 9"
LEVEL 1 UPPER
2' - 9"
LEVEL 1 LOWER
0' - 0"
9' - 0"
10' - 0"
10' - 0"
10' - 0"
2' - 9"
41' - 9"
T40
18.6
230
408
549
480
A
B
C
C
WEST

Perforated Strapped Wood Shear Wall

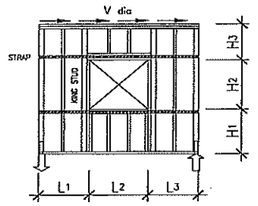
line 10:4th Level - Ltotal= 14'-8" Wall

INPUT DATA

LATERAL FORCE ON DIAPHRAGM: V_{WIND} = pif,for wind — direct shear
(SERVICE LOADS) $V_{SEISMIC}$ = 162 pif,for seismic — V= 262 plf

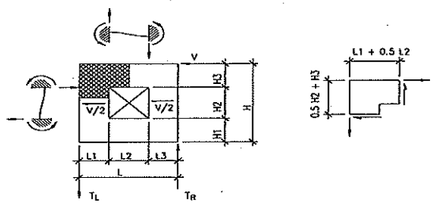
DIMENSIONS:	L_1 =	4.5	ft,	L_2 =	5.6	ft,	L_3 =	4.5	ft
	H_1 =	2.75	ft,	H_2 =	5	ft,	H_3 =	1.25	ft
KING STUD SECTION		2	pcs, b =		3	in, h =		6	in
EDGE STUD SECTION		2	pcs, b =		4	in, h =		6	in

PANEL GRADE (0 or 1) = 0 <= Structural I
MINIMUM NOMINAL PANEL THICKNESS = 15/32 in — w_{DL} = 300 plf,wall dead load
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d — w_{LL} = 80 plf,wall live load
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

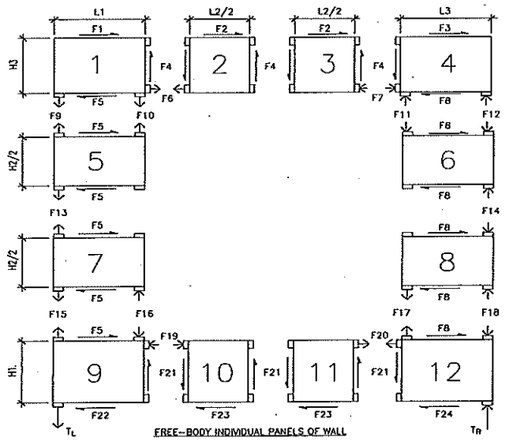


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE--BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.1 < 2 [Satisfactory]
THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.50	1.25	-39	F1	-178	F13	606
2	2.80	1.25	485	F2	1357	F14	606
3	2.80	1.25	485	F3	-178	F15	1261
4	4.50	1.25	-39	F4	606	F16	655
5	4.50	2.50	262	F5	1179	F17	655
6	4.50	2.50	262	F6	1357	F18	1261
7	4.50	2.50	262	F7	1357	F19	3014
8	4.50	2.50	262	F8	1179	F20	3014
9	4.50	2.75	-408	F9	-49	F21	-466
10	2.80	2.75	-169	F10	655	F22	-1835
11	2.80	2.75	-169	F11	655	F23	3014
12	4.50	2.75	-408	F12	-49	F24	-1835

THE UNIT SHEAR FORCE v_n = 485 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

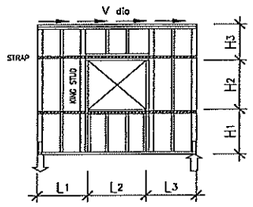
B

Perforated Strapped Wood Shear Wall

Line 10: 3rd Level - Ltotal= 17'-1" Wall

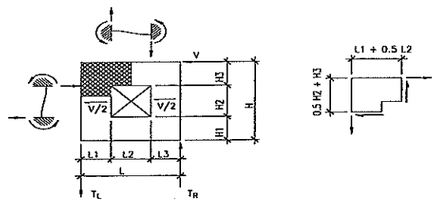
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:		V_{WIND} =		plf,for wind			direct shear		
(SERVICE LOADS)		$V_{SEISMIC}$ =	290	plf,for seismic		V=	340	plf	
DIMENSIONS:	L_1 =	9.75	ft, L_2 =	2.5	ft, L_3 =	4.83	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS		=	15/32	in	w_{DL} =	740	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	320	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

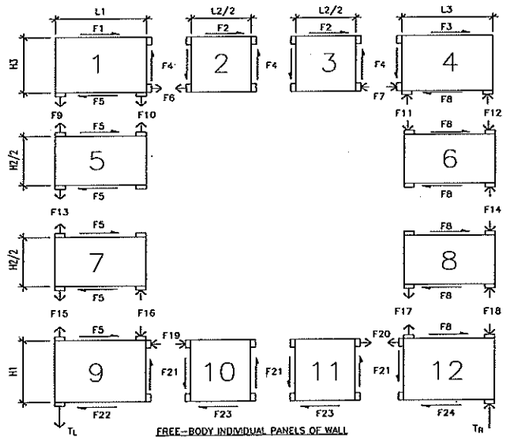


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE--BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.0 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	9.75	1.25	216	F1	2104	F13	1088
2	1.25	1.25	871	F2	1088	F14	1088
3	1.25	1.25	871	F3	676	F15	1907
4	4.83	1.25	140	F4	1088	F16	819
5	9.75	2.50	327	F5	3193	F17	913
6	4.83	2.50	365	F6	1088	F18	2002
7	9.75	2.50	327	F7	1088	F19	9954
8	4.83	2.50	365	F8	1765	F20	5280
9	9.75	2.75	-693	F9	270	F21	-1088
10	1.25	2.75	-396	F10	819	F22	-6761
11	1.25	2.75	-396	F11	813	F23	9954
12	4.83	2.75	-728	F12	175	F24	-3516

THE UNIT SHEAR FORCE v_s = 871 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	685	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

D

Perforated Strapped Wood Shear Wall

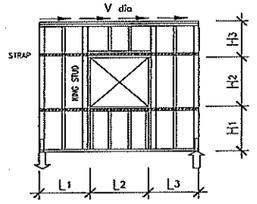
Line 10:2nd Level - Ltotal= 17'-1" Wall

INPUT DATA

LATERAL FORCE ON DIAPHRAGM: V_{WIND} = pli,for wind — direct shear
(SERVICE LOADS) $V_{SEISMIC}$ = 387 pli,for seismic V= 453 plf

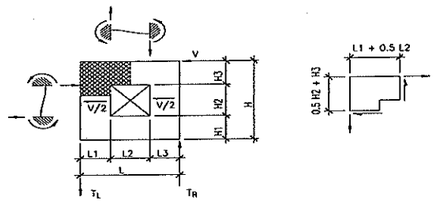
DIMENSIONS: L_1 = 9.75 ft, L_2 = 2.5 ft, L_3 = 4.83 ft
H_1 = 2.75 ft, H_2 = 5 ft, H_3 = 1.25 ft

KING STUD SECTION	2 pcs, b = 3	in, h = 6	in
EDGE STUD SECTION	2 pcs, b = 4	in, h = 6	in
PANEL GRADE (0 or 1) =	0 <= Structural I		
MINIMUM NOMINAL PANEL THICKNESS =	15/32	in	w_{DL} = 1160 plf,wall dead load
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)	2	10d	w_{LL} = 560 plf,wall live load
SPECIFIC GRAVITY OF FRAMING MEMBERS	0.5		

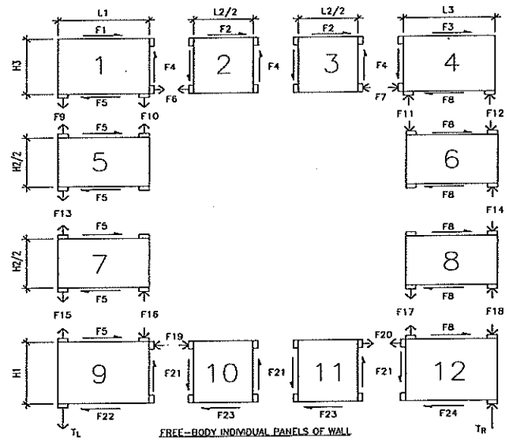


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 3 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.0 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (psf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	9.75	1.25	288	F1	2804	F13	1450
2	1.25	1.25	1160	F2	1450	F14	1450
3	1.25	1.25	1160	F3	901	F15	2541
4	4.83	1.25	187	F4	1450	F16	1091
5	9.75	2.50	436	F5	4254	F17	1217
6	4.83	2.50	487	F6	1450	F18	2667
7	9.75	2.50	436	F7	1450	F19	13262
8	4.83	2.50	487	F8	2351	F20	7035
9	9.75	2.75	-924	F9	359	F21	-1450
10	1.25	2.75	-527	F10	1091	F22	-9008
11	1.25	2.75	-527	F11	1217	F23	13262
12	4.83	2.75	-970	F12	233	F24	-4684

THE UNIT SHEAR FORCE v_s = 1160 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 3 in)

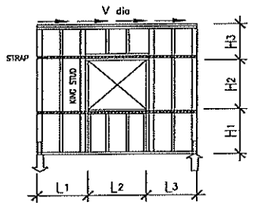
THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

Perforated Strapped Wood Shear Wall

4th Level line 6 - L.total= 12'-3" Wall

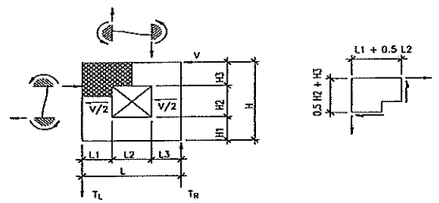


INPUT DATA

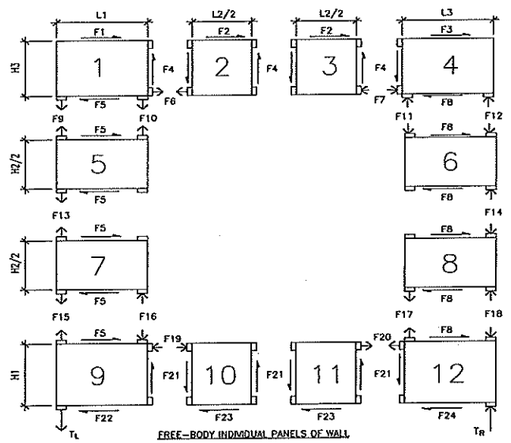
LATERAL FORCE ON DIAPHRAGM:			V_{wind} =		plf,for wind			direct shear	
(SERVICE LOADS)			$V_{seismic}$ =	195	plf,for seismic		V=	262	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	5	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS		=	15/32	in	w_{DL} =	563	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	250	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 3 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h/w = 1.3 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	44	F1	175	F13	731
2	1.55	1.25	585	F2	906	F14	731
3	1.55	1.25	585	F3	370	F15	1407
4	5.00	1.25	74	F4	731	F16	676
5	4.00	2.50	270	F5	1082	F17	638
6	5.00	2.50	255	F6	906	F18	1369
7	4.00	2.50	270	F7	906	F19	3128
8	5.00	2.50	255	F8	1276	F20	3766
9	4.00	2.75	-512	F9	55	F21	-731
10	1.55	2.75	-266	F10	676	F22	-2046
11	1.55	2.75	-266	F11	638	F23	3128
12	5.00	2.75	-498	F12	93	F24	-2489

THE UNIT SHEAR FORCE v_s = 595 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 3 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

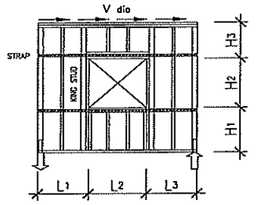
D32

Perforated Strapped Wood Shear Wall

3rd Level - Ltotal line 6 = 12'-3" Wall

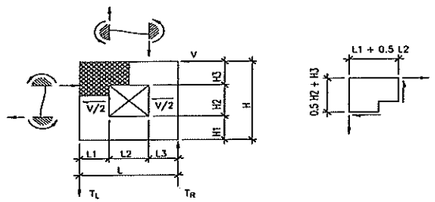
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:			V_{WIND} =		plf,for wind			direct shear	
(SERVICE LOADS)			$V_{SEISMIC}$ =	253	plf,for seismic		V=	340	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	5	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS		=	15/32	in	w_{DL} =	816	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	343	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

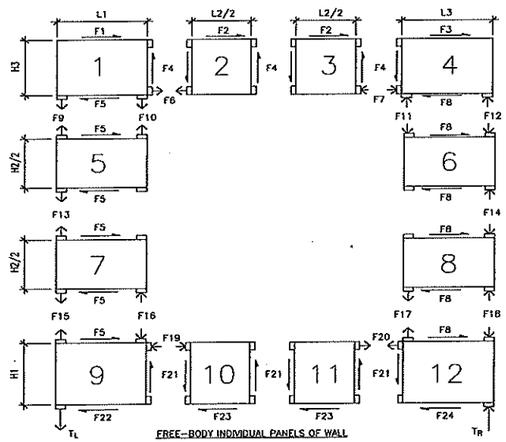


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 2 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE--BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	57	F1	228	F13	948
2	1.55	1.25	759	F2	1176	F14	948
3	1.55	1.25	759	F3	480	F15	1826
4	5.00	1.25	96	F4	948	F16	877
5	4.00	2.50	351	F5	1404	F17	828
6	5.00	2.50	331	F6	1176	F18	1777
7	4.00	2.50	351	F7	1176	F19	4059
8	5.00	2.50	331	F8	1656	F20	4887
9	4.00	2.75	-664	F9	71	F21	-948
10	1.55	2.75	-345	F10	877	F22	-2655
11	1.55	2.75	-345	F11	828	F23	4059
12	5.00	2.75	-648	F12	120	F24	-3230

THE UNIT SHEAR FORCE v_s = 759 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 2 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

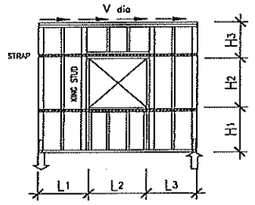
Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

D

Perforated Strapped Wood Shear Wall

2nd Level line 6- Ltotal= 12'-3" Wall

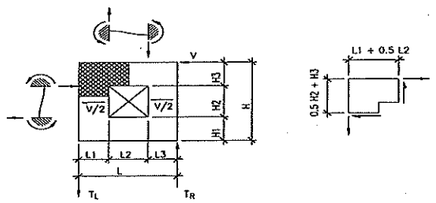


INPUT DATA

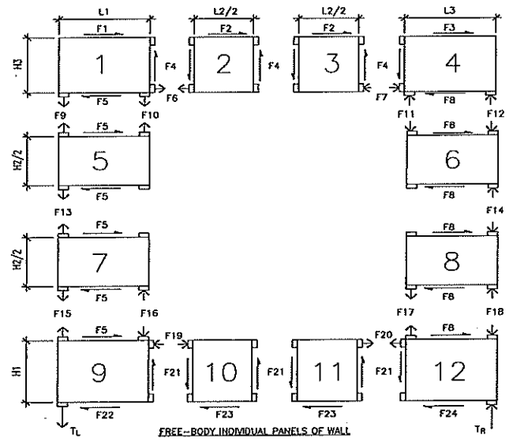
LATERAL FORCE ON DIAPHRAGM:		V_{WIND} =		plf,for wind			direct shear		
(SERVICE LOADS)		$V_{SEISMIC}$ =	337	plf,for seismic		V=	453	plf	
DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	5	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS		=	15/32	in	w_{DL} =	786	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	166	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE--BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	76	F1	303	F13	1264
2	1.55	1.25	1011	F2	1567	F14	1264
3	1.55	1.25	1011	F3	640	F15	2432
4	5.00	1.25	128	F4	1264	F16	1169
5	4.00	2.50	468	F5	1870	F17	1103
6	5.00	2.50	441	F6	1567	F18	2367
7	4.00	2.50	468	F7	1567	F19	5149
8	5.00	2.50	441	F8	2207	F20	6187
9	4.00	2.75	-620	F9	95	F21	-1086
10	1.55	2.75	-395	F10	1169	F22	-3279
11	1.55	2.75	-395	F11	1103	F23	5149
12	5.00	2.75	-796	F12	160	F24	-3980

THE UNIT SHEAR FORCE v_s = 1011 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

E

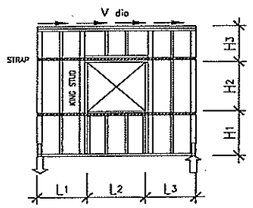
Perforated Strapped Wood Shear Wall

4th Level - Ltotal= 12'-6" Wall

INPUT DATA

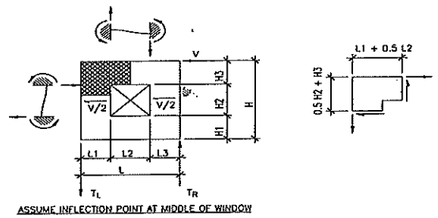
LATERAL FORCE ON DIAPHRAGM (SERVICE LOADS): V_{wind} = plf, for wind; $V_{seismic}$ = 166 plf, for seismic; direct shear V= 221 plf

DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	5.4	ft	
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft	
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in	
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in	
PANEL GRADE (0 or 1)	=	0	<= Structural I					
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in		w_{DL} =	563	plf,wall dead load	
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d		w_{LL} =	250	plf,wall live load	
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5						

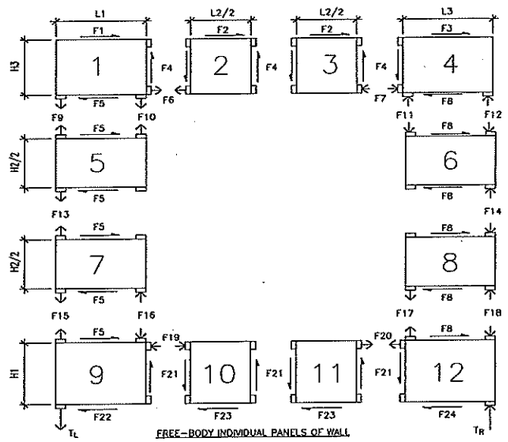


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	37	F1	150	F13	623
2	1.55	1.25	499	F2	773	F14	623
3	1.55	1.25	499	F3	382	F15	1200
4	5.40	1.25	71	F4	623	F16	576
5	4.00	2.50	231	F5	922	F17	535
6	5.40	2.50	214	F6	773	F18	1158
7	4.00	2.50	231	F7	773	F19	2667
8	5.40	2.50	214	F8	1155	F20	3429
9	4.00	2.75	-436	F9	47	F21	-623
10	1.55	2.75	-227	F10	576	F22	-1745
11	1.55	2.75	-227	F11	535	F23	2667
12	5.40	2.75	-421	F12	88	F24	-2274

THE UNIT SHEAR FORCE v_s = 499 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	685	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

B

MURPHY BURR CURRY, INC.
CONSULTING STRUCTURAL ENGINEERS

Project: Chestnut F.H.
Project #: 216-008
By: JYL
9/20/2018
Page:_____

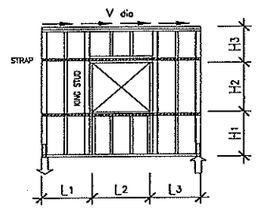
D35

Perforated Strapped Wood Shear Wall

3rd Level - Ltotal= 12'-6" Wall

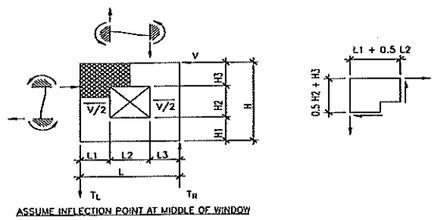
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:			$v_{s,WIND}$ =		plf,for wind			direct shear	
(SERVICE LOADS)			$v_{s,SEISMIC}$ =	332	plf,for seismic		V=	441	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	5.4	ft		
	H_3 =	2.75	ft, H_2 =	5	ft, H_1 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS		=	15/32	in	w_{DL} =	816	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	343	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

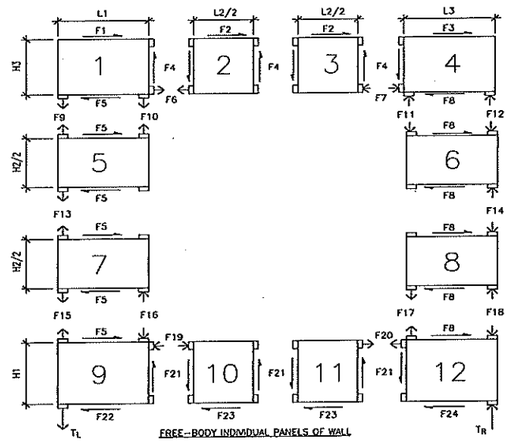


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE--BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)
1	4.00	1.25	75
2	1.55	1.25	995
3	1.55	1.25	995
4	5.40	1.25	141
5	4.00	2.50	460
6	5.40	2.50	427
7	4.00	2.50	460
8	5.40	2.50	427
9	4.00	2.75	-871
10	1.55	2.75	-452
11	1.55	2.75	-452
12	5.40	2.75	-840

NO.	FORCE (lbf)
F1	298
F2	1542
F3	763
F4	1244
F5	1841
F6	1542
F7	1542
F8	2305
F9	93
F10	1150
F11	1067
F12	177

NO.	FORCE (lbf)
F13	1244
F14	1244
F15	2394
F16	1150
F17	1067
F18	2311
F19	5323
F20	6842
F21	-1244
F22	-3482
F23	5323
F24	-4537

THE UNIT SHEAR FORCE v_s = 995 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

E

MURPHY BURR CURRY, INC.
CONSULTING STRUCTURAL ENGINEERS

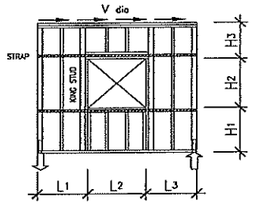
Project: Chestnut F.H.
Project #: 216-008
By: JYL
9/20/2018
Page:_____

D36

Perforated Strapped Wood Shear Wall — 2nd Level - Ltotal= 12'-6" Wall

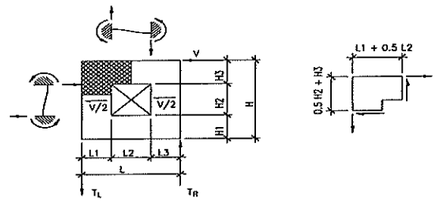
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:		V_{WIND} =		plf,for wind			direct shear		
(SERVICE LOADS)		$V_{SEISMIC}$ =	447	plf,for seismic			V=	595	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	5.4	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in		w_{DL} =	785	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d		w_{LL} =	186	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5							

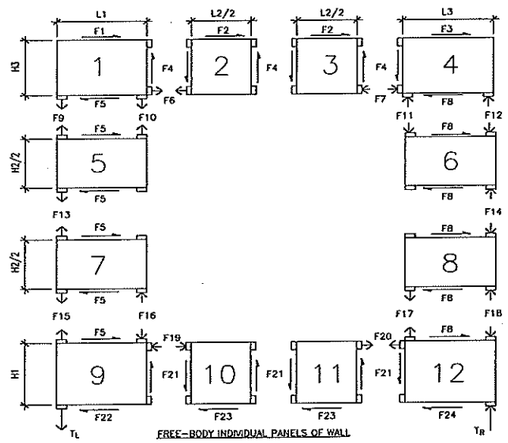


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 2 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	101	F1	403	F13	1678
2	1.55	1.25	1342	F2	2081	F14	1678
3	1.55	1.25	1342	F3	1029	F15	3230
4	5.40	1.25	191	F4	1678	F16	1552
5	4.00	2.50	621	F5	2483	F17	1440
6	5.40	2.50	576	F6	2081	F18	3118
7	4.00	2.50	621	F7	2081	F19	5613
8	5.40	2.50	576	F8	3110	F20	7115
9	4.00	2.75	-783	F9	126	F21	-600
10	1.55	2.75	-218	F10	1552	F22	-3130
11	1.55	2.75	-218	F11	1440	F23	5613
12	5.40	2.75	-742	F12	238	F24	-4005

THE UNIT SHEAR FORCE v_s = 1342 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 2 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

F

D37

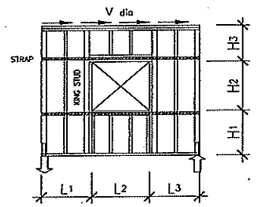
Perforated Strapped Wood Shear Wall

1st Level - Llotal= 12'-6" Wall

INPUT DATA

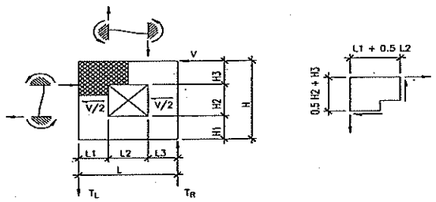
LATERAL FORCE ON DIAPHRAGM: V_{WIND} = plf,for wind — direct shear
(SERVICE LOADS) $V_{SEISMIC}$ = 511 plf,for seismic — V= 679 plf

DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	5.4	ft	
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft	
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in	
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in	
PANEL GRADE (0 or 1)	=	0	<= Structural I					
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in		w_{DL} =	1040	plf,wall dead load	
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d		w_{LL} =	240	plf,wall live load	
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5						

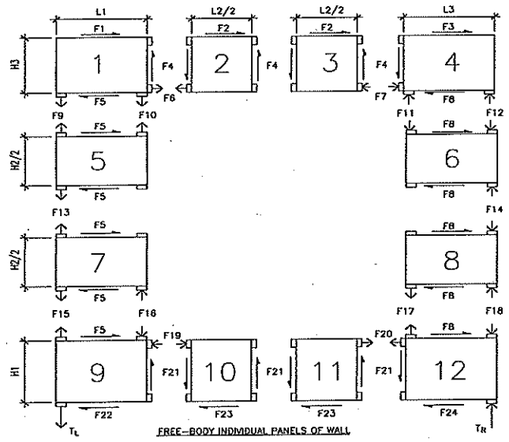


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 2 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE--BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lb/f)	NO.	FORCE (lb/f)
1	4.00	1.25	115	F1	460	F13	1915
2	1.55	1.25	1532	F2	2374	F14	1915
3	1.55	1.25	1532	F3	1174	F15	3686
4	5.40	1.25	217	F4	1915	F16	1771
5	4.00	2.50	708	F5	2834	F17	1843
6	5.40	2.50	657	F6	2374	F18	3558
7	4.00	2.50	708	F7	2374	F19	7181
8	5.40	2.50	657	F8	3549	F20	9166
9	4.00	2.75	-1087	F9	144	F21	-1218
10	1.55	2.75	-443	F10	1771	F22	-4348
11	1.55	2.75	-443	F11	1643	F23	7181
12	5.40	2.75	-1040	F12	272	F24	-5617

THE UNIT SHEAR FORCE v_u = 1532 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 2 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

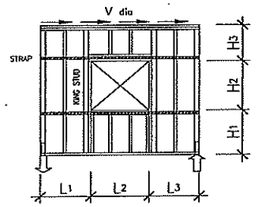
Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

9

Perforated Strapped Wood Shear Wall — 4th Level - Ltotal= 12'-0" Wall

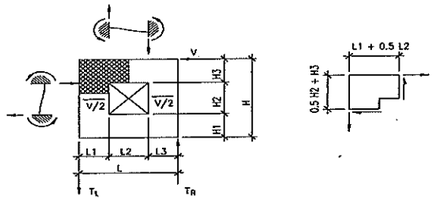
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:		V_{WIND} =		plf,for wind			direct shear		
(SERVICE LOADS)		$V_{SEISMIC}$ =	163	plf,for seismic		V=	221	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	4.7	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in		w_{DL} =	563	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d		w_{LL} =	250	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5							

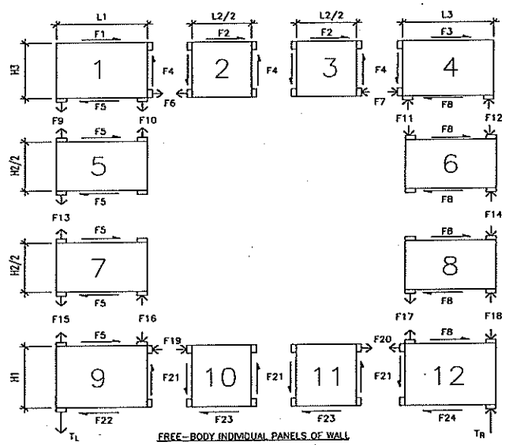


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (psf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	37	F1	147	F13	611
2	1.55	1.25	489	F2	758	F14	611
3	1.55	1.25	489	F3	261	F15	1176
4	4.70	1.25	55	F4	611	F16	565
5	4.00	2.50	226	F5	904	F17	542
6	4.70	2.50	217	F6	758	F18	1153
7	4.00	2.50	226	F7	758	F19	2615
8	4.70	2.50	217	F8	1018	F20	2888
9	4.00	2.75	-428	F9	48	F21	-611
10	1.55	2.75	-222	F10	565	F22	-1711
11	1.55	2.75	-222	F11	542	F23	2615
12	4.70	2.75	-419	F12	69	F24	-1970

THE UNIT SHEAR FORCE v_u = 489 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

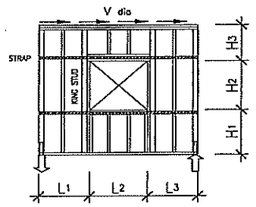
B

Perforated Strapped Wood Shear Wall

3rd Level - Ltotal= 12'-0" Wall

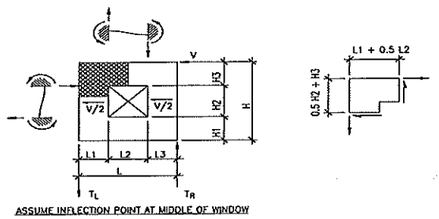
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:		V_{WIND} =		plf,for wind			direct shear		
(SERVICE LOADS)		$V_{SEISMIC}$ =	325	plf,for seismic			V=	441	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	4.7	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS		=	15/32	in	w_{DL} =	816	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	343	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

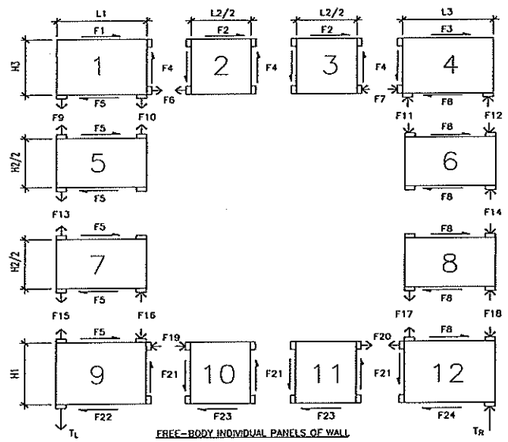


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	73	F1	293	F13	1219
2	1.55	1.25	975	F2	1512	F14	1219
3	1.55	1.25	975	F3	520	F15	2347
4	4.70	1.25	111	F4	1219	F16	1128
5	4.00	2.50	451	F5	1805	F17	1081
6	4.70	2.50	432	F6	1512	F18	2300
7	4.00	2.50	451	F7	1512	F19	5162
8	4.70	2.50	432	F8	2032	F20	5897
9	4.00	2.75	-839	F9	91	F21	-1181
10	1.55	2.75	-429	F10	1128	F22	-3358
11	1.55	2.75	-429	F11	1081	F23	5162
12	4.70	2.75	-822	F12	138	F24	-3665

THE UNIT SHEAR FORCE v_s = 975 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

E

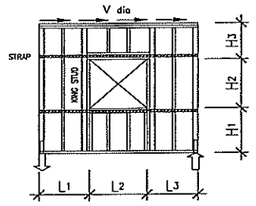
MURPHY BURR CURRY, INC.
CONSULTING STRUCTURAL ENGINEERS

Project: Chestnut F.H.
Project #: 216-008
By: JYL
9/20/2018
Page:_____

D40

Perforated Strapped Wood Shear Wall

2nd Level - Ltotal= 12'-0" Wall

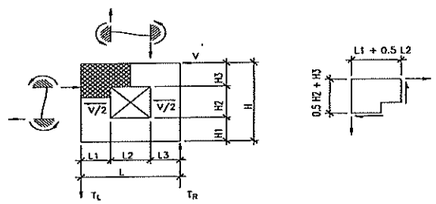


INPUT DATA

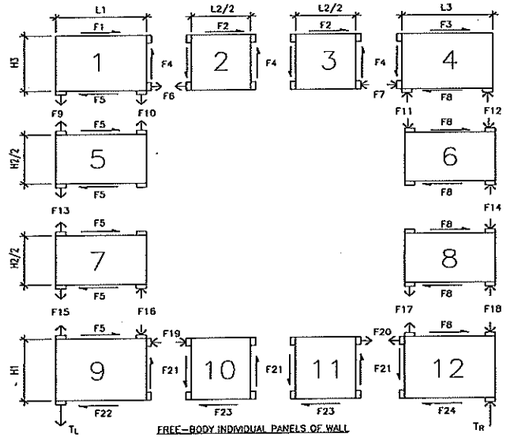
LATERAL FORCE ON DIAPHRAGM:		V_{WIND} =		plf,for wind			direct shear	
(SERVICE LOADS)		$V_{SEISMIC}$ =	439	plf,for seismic		V=	595	plf
DIMENSIONS:	L_1 = 4	ft, L_2 =	3.1	ft, L_3 =	4.7	ft		
	H_1 = 2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION	2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION	2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1) =	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS =		15/32	in	w_{DL} =	786	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d	w_{LL} =	186	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5						

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 3 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (psf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	99	F1	395	F13	1645
2	1.55	1.25	1316	F2	2040	F14	1645
3	1.55	1.25	1316	F3	702	F15	3187
4	4.70	1.25	149	F4	1645	F16	1522
5	4.00	2.50	609	F5	2435	F17	1458
6	4.70	2.50	583	F6	2040	F18	3103
7	4.00	2.50	609	F7	2040	F19	5347
8	4.70	2.50	583	F8	2742	F20	6056
9	4.00	2.75	-728	F9	123	F21	-481
10	1.55	2.75	-175	F10	1522	F22	-2913
11	1.55	2.75	-175	F11	1458	F23	5347
12	4.70	2.75	-705	F12	187	F24	-3314

THE UNIT SHEAR FORCE v_s = 1316 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 3 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

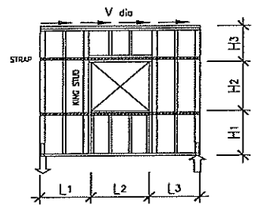
F

Perforated Strapped Wood Shear Wall

1st Level - Ltotal= 12'-0" Wall

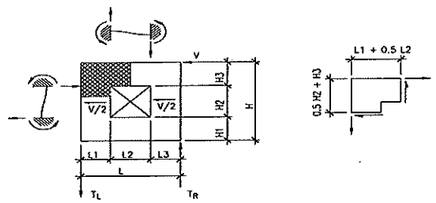
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:			V_{WIND} =			plf,for wind		direct shear	
(SERVICE LOADS)			$V_{SEISMIC}$ =	501		plf,for seismic	V=	678	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	3.1	ft, L_3 =	4.7	ft		
	H_3 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS		=	15/32	in	w_{DL} =	1040	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	240	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

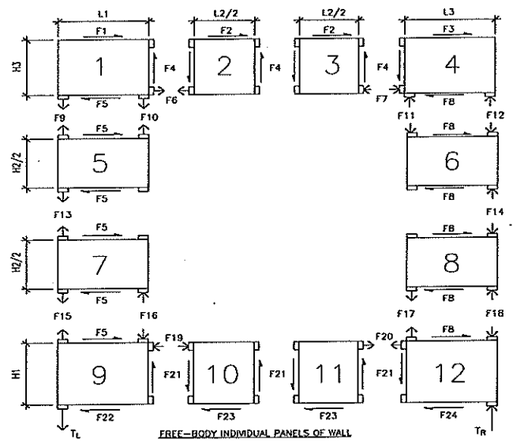


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 2 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIMDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIMDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	113	F1	451	F13	1877
2	1.55	1.25	1502	F2	2328	F14	1877
3	1.55	1.25	1502	F3	801	F15	3814
4	4.70	1.25	170	F4	1877	F16	1737
5	4.00	2.50	695	F5	2778	F17	1664
6	4.70	2.50	666	F6	2328	F18	3542
7	4.00	2.50	695	F7	2328	F19	6834
8	4.70	2.50	666	F8	3129	F20	7771
9	4.00	2.75	-1014	F9	141	F21	-1052
10	1.55	2.75	-383	F10	1737	F22	-4056
11	1.55	2.75	-383	F11	1664	F23	6834
12	4.70	2.75	-988	F12	213	F24	-4642

THE UNIT SHEAR FORCE v_u = 1502 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 2 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

6

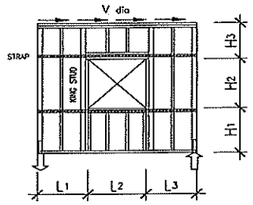
Perforated Strapped Wood Shear Wall

4th Level - Ltotal= 13'-6" Wall

INPUT DATA

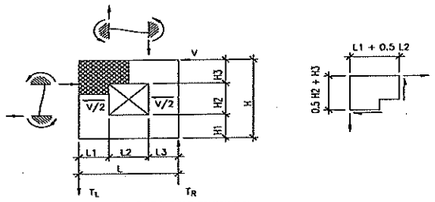
LATERAL FORCE ON DIAPHRAGM: V_{WIND} = plf,for wind — direct shear
(SERVICE LOADS) $V_{SEISMIC}$ = 128 plf,for seismic V= 221 plf

DIMENSIONS:	L_1 =	3.9	ft, L_2 =	5.7	ft, L_3 =	3.9	ft	
	H_1 =	1.75	ft, H_2 =	6	ft, H_3 =	1.25	ft	
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in	
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in	
PANEL GRADE (0 or 1)	=	0	<= Structural I					
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in		w_{DL} =	100	plf,wall dead load	
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d		w_{LL} =	80	plf,wall live load	
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5						

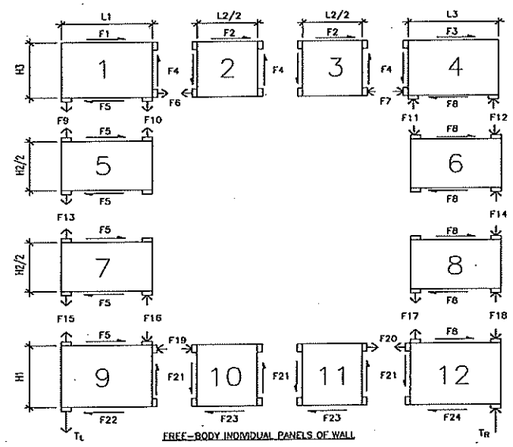


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.5 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (psf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	3.90	1.25	-96	F1	-375	F13	543
2	2.85	1.25	434	F2	1237	F14	543
3	2.85	1.25	434	F3	-375	F15	1206
4	3.90	1.25	-96	F4	543	F16	663
5	3.90	3.00	221	F5	862	F17	663
6	3.90	3.00	221	F6	1237	F18	1206
7	3.90	3.00	221	F7	1237	F19	1890
8	3.90	3.00	221	F8	862	F20	1890
9	3.90	1.75	-264	F9	-120	F21	202
10	2.85	1.75	115	F10	663	F22	-1028
11	2.85	1.75	115	F11	663	F23	1890
12	3.90	1.75	-264	F12	-120	F24	-1028

THE UNIT SHEAR FORCE v_b = 434 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

B

Perforated Strapped Wood Shear Wall

3rd Level - L.total= 13'-6" Wall

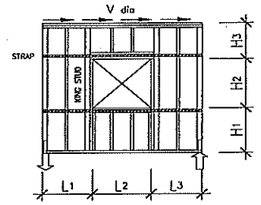
INPUT DATA

LATERAL FORCE ON DIAPHRAGM: V_{WIND} = 255 plf,for wind; $V_{SEISMIC}$ = 441 plf,for seismic — direct shear V= 441 plf

(SERVICE LOADS)

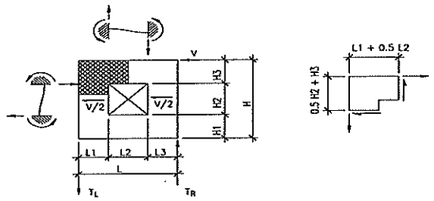
DIMENSIONS: L_1 = 3.9 ft, L_2 = 5.7 ft, L_3 = 3.9 ft; H_1 = 1.75 ft, H_2 = 6 ft, H_3 = 1.25 ft

KING STUD SECTION	2	pcs, b =	3	in, h =	6	in
EDGE STUD SECTION	2	pcs, b =	4	in, h =	6	in
PANEL GRADE (0 or 1) =	0	<= Structural I				
MINIMUM NOMINAL PANEL THICKNESS =	15/32	in	w_{DL} =	533	plf,wall dead load	
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)	2	10d	w_{LL} =	133	plf,wall live load	
SPECIFIC GRAVITY OF FRAMING MEMBERS	0.5					

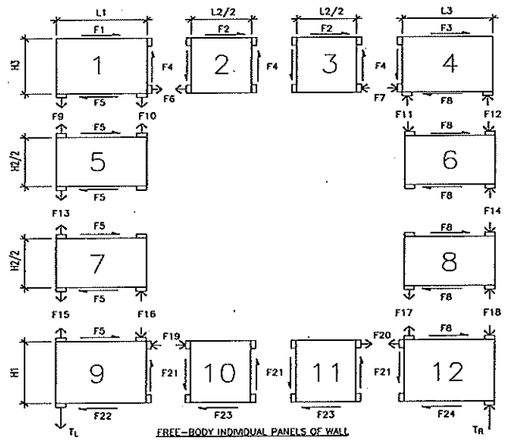


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 2 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.5 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	3.90	1.25	-192	F1	-749	F13	1083
2	2.85	1.25	866	F2	2469	F14	1083
3	2.85	1.25	866	F3	-749	F15	2406
4	3.90	1.25	-192	F4	1083	F16	1323
5	3.90	3.00	441	F5	1720	F17	1323
6	3.90	3.00	441	F6	2469	F18	2406
7	3.90	3.00	441	F7	2469	F19	6784
8	3.90	3.00	441	F8	1720	F20	6784
9	3.90	1.75	-1298	F9	-240	F21	-949
10	2.85	1.75	-542	F10	1323	F22	-5064
11	2.85	1.75	-542	F11	1323	F23	6784
12	3.90	1.75	-1298	F12	-240	F24	-5064

THE UNIT SHEAR FORCE v_s = 866 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 2 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

D

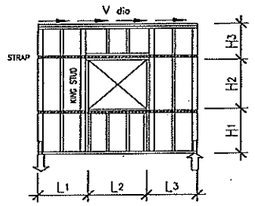
Perforated Strapped Wood Shear Wall

2nd Level - Ltotal= 13'-6" Wall

INPUT DATA

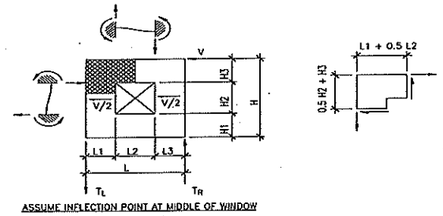
LATERAL FORCE ON DIAPHRAGM: $V_{,WIND}$ = pif,for wind — direct shear
(SERVICE LOADS) $V_{,SEISMIC}$ = 344 plf,for seismic V= 595 plf

DIMENSIONS:	L_1 =	3.9	ft, L_2 =	5.7	ft, L_3 =	3.9	ft	
	H_1 =	1.75	ft, H_2 =	6	ft, H_3 =	1.25	ft	
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in	
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in	
PANEL GRADE (0 or 1)	=	0	<= Structural1					
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in		w_{DL} =	786	plf,wall dead load	
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d		w_{LL} =	186	plf,wall live load	
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5						

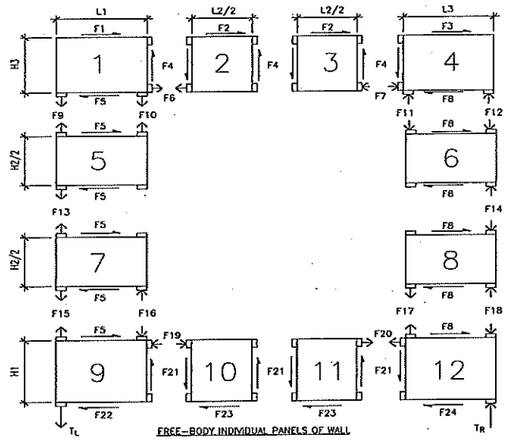


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 3 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.5 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	3.90	1.25	-259	F1	-1011	F13	1461
2	2.85	1.25	1169	F2	3331	F14	1461
3	2.85	1.25	1169	F3	-1011	F15	3246
4	3.90	1.25	-259	F4	1461	F16	1785
5	3.90	3.00	595	F5	2321	F17	1785
6	3.90	3.00	595	F6	3331	F18	3246
7	3.90	3.00	595	F7	3331	F19	9555
8	3.90	3.00	595	F8	2321	F20	9555
9	3.90	1.75	-1855	F9	-324	F21	-1461
10	2.85	1.75	-835	F10	1785	F22	-7234
11	2.85	1.75	-835	F11	1785	F23	9555
12	3.90	1.75	-1855	F12	-324	F24	-7234

THE UNIT SHEAR FORCE v_s = 1169 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 3 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

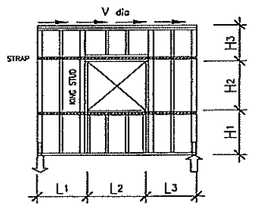
Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

F

Perforated Strapped Wood Shear Wall — 1st Level - Ltotal= 13'-6" Wall

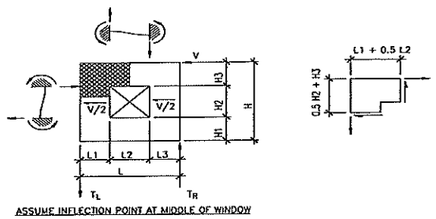
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:		V_{WIND} =		plf,for wind			direct shear		
(SERVICE LOADS)		$V_{SEISMIC}$ =	392	plf,for seismic		V=	679	plf	
DIMENSIONS:	L_1 = 3.9	ft, L_2 =	5.7	ft, L_3 =	3.9	ft			
	H_1 = 1.75	ft, H_2 =	6	ft, H_3 =	1.25	ft			
KING STUD SECTION	2	pcs, b =	3	in, h =	6	in			
EDGE STUD SECTION	2	pcs, b =	4	in, h =	6	in			
PANEL GRADE (0 or 1) =	0	<= Structural I							
MINIMUM NOMINAL PANEL THICKNESS =		15/32	in	w_{DL} =	1040	plf,wall dead load			
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d	w_{LL} =	240	plf,wall live load			
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5							

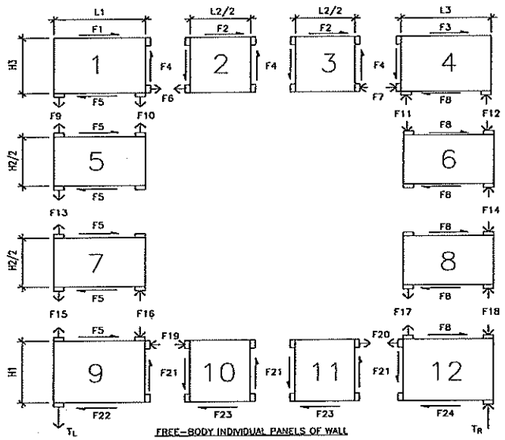


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 2 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.5 < 2 (Satisfactory)

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (psf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	3.90	1.25	-296	F1	-1153	F13	1667
2	2.85	1.25	1334	F2	3801	F14	1667
3	2.85	1.25	1334	F3	-1153	F15	3704
4	3.90	1.25	-296	F4	1667	F16	2037
5	3.90	3.00	679	F5	2648	F17	2037
6	3.90	3.00	679	F6	3801	F18	3704
7	3.90	3.00	679	F7	3801	F19	10903
8	3.90	3.00	679	F8	2648	F20	10903
9	3.90	1.75	-2117	F9	-370	F21	-1667
10	2.85	1.75	-953	F10	2037	F22	-8255
11	2.85	1.75	-953	F11	2037	F23	10903
12	3.90	1.75	-2117	F12	-370	F24	-8255

THE UNIT SHEAR FORCE v_s = 1334 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 2 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 : -

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

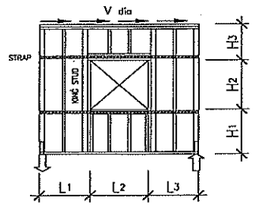
Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

F

Perforated Strapped Wood Shear Wall — 4th Level - Ltotal= 14'-8" Wall

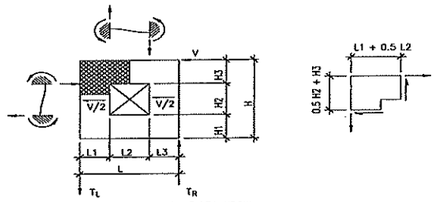
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:			V_{WIND} =		plf,for wind		direct shear		
(SERVICE LOADS)			$V_{SEISMIC}$ =	121	plf,for seismic		V=	221	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	6.6	ft, L_3 =	4	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS	=		15/32	in	w_{DL} =	563	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	250	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

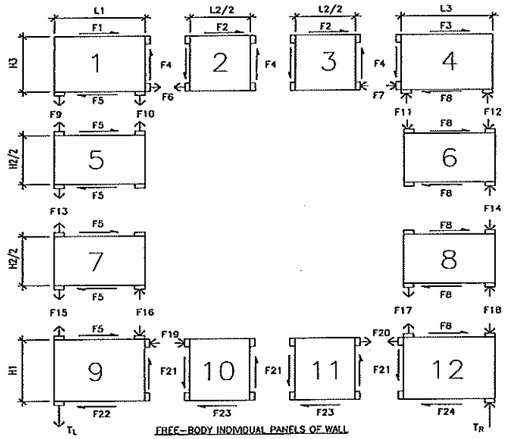


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	-79	F1	-315	F13	454
2	3.30	1.25	363	F2	1199	F14	454
3	3.30	1.25	363	F3	-315	F15	1007
4	4.00	1.25	-79	F4	454	F16	553
5	4.00	2.50	221	F5	884	F17	553
6	4.00	2.50	221	F6	1199	F18	1007
7	4.00	2.50	221	F7	1199	F19	2348
8	4.00	2.50	221	F8	884	F20	2348
9	4.00	2.75	-366	F9	-98	F21	-454
10	3.30	2.75	-165	F10	553	F22	-1464
11	3.30	2.75	-165	F11	553	F23	2348
12	4.00	2.75	-366	F12	-98	F24	-1464

THE UNIT SHEAR FORCE v_s = 363 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note; The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

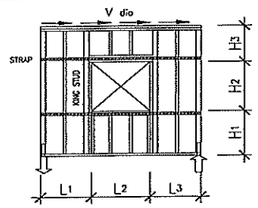
B

Perforated Strapped Wood Shear Wall

3rd Level - Ltotal= 14'-8" Wall

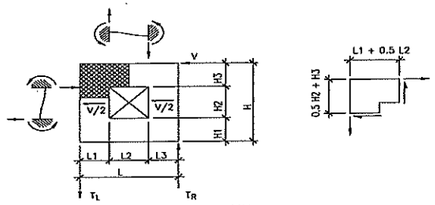
INPUT DATA

LATERAL FORCE ON DIAPHRAGM:			V_{WIND} =			plf,for wind		direct shear	
(SERVICE LOADS)			$V_{SEISMIC}$ =	242		plf,for seismic		V= 441	plf
DIMENSIONS:	L_1 =	4	ft, L_2 =	6.6	ft, L_3 =	4	ft		
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft		
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in		
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in		
PANEL GRADE (0 or 1)	=	0	<= Structural I						
MINIMUM NOMINAL PANEL THICKNESS		=	15/32	in	w_{DL} =	816	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)			2	10d	w_{LL} =	343	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS			0.5						

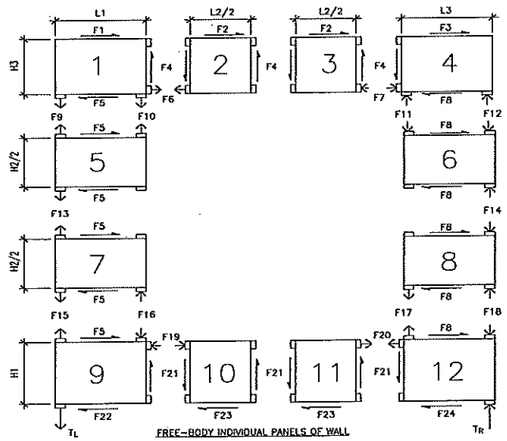


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 2 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h/w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (psf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	-157	F1	-628	F13	906
2	3.30	1.25	725	F2	2392	F14	906
3	3.30	1.25	725	F3	-628	F15	2009
4	4.00	1.25	-157	F4	906	F16	1103
5	4.00	2.50	441	F5	1764	F17	1103
6	4.00	2.50	441	F6	2392	F18	2009
7	4.00	2.50	441	F7	2392	F19	4566
8	4.00	2.50	441	F8	1764	F20	4566
9	4.00	2.75	-730	F9	-196	F21	-906
10	3.30	2.75	-330	F10	1103	F22	-2922
11	3.30	2.75	-330	F11	1103	F23	4566
12	4.00	2.75	-730	F12	-196	F24	-2922

THE UNIT SHEAR FORCE v_b = 725 plf, (1 Side Diaphragm Required, the Max. Nail Spacing = 2 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

D

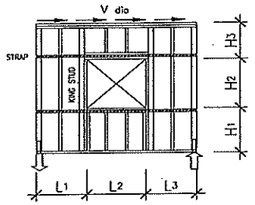
Perforated Strapped Wood Shear Wall

2nd Level - Ltotal= 14'-8" Wall

INPUT DATA

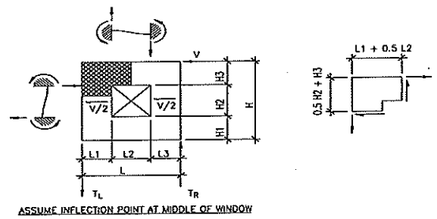
LATERAL FORCE ON DIAPHRAGM: V_{WIND} = plf,for wind; direct shear
(SERVICE LOADS) $V_{SEISMIC}$ = 326 plf,for seismic V= 595 plf

DIMENSIONS:	L_1 =	4	ft, L_2 =	6.6	ft, L_3 =	4	ft	
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft	
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in	
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in	
PANEL GRADE (0 or 1)	=	0	<= Structural I					
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in		w_{DL} =	786	plf,wall dead load	
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d		w_{LL} =	186	plf,wall live load	
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5						

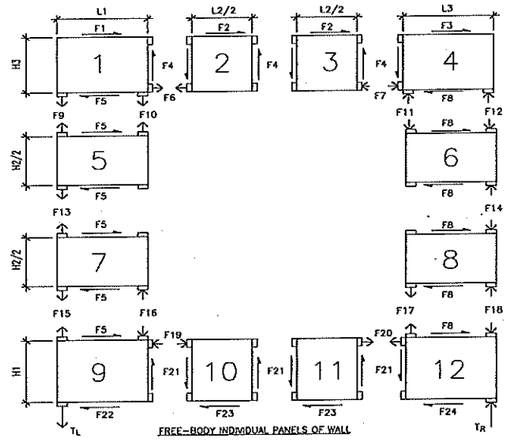


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.00	1.25	-212	F1	-848	F13	1223
2	3.30	1.25	978	F2	3228	F14	1223
3	3.30	1.25	978	F3	-848	F15	2710
4	4.00	1.25	-212	F4	1223	F16	1488
5	4.00	2.50	595	F5	2380	F17	1488
6	4.00	2.50	595	F6	3228	F18	2710
7	4.00	2.50	595	F7	3228	F19	6322
8	4.00	2.50	595	F8	2380	F20	6322
9	4.00	2.75	-985	F9	-265	F21	-1223
10	3.30	2.75	-445	F10	1488	F22	-3942
11	3.30	2.75	-445	F11	1488	F23	6322
12	4.00	2.75	-985	F12	-265	F24	-3942

THE UNIT SHEAR FORCE v_u = 978 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

F

Perforated Strapped Wood Shear Wall

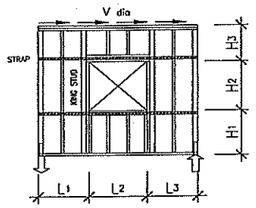
1st Level - Ltotal= 14'-8" Wall

INPUT DATA

LATERAL FORCE ON DIAPHRAGM: V_{WIND} = 670 plf,for wind; $V_{SEISMIC}$ = 372 plf,for seismic; direct shear V= 670 plf

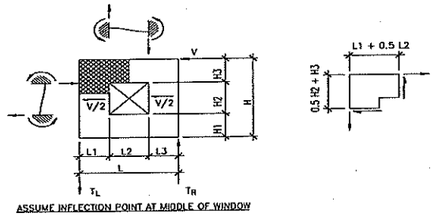
(SERVICE LOADS)

DIMENSIONS:	L_1 =	4	ft, L_2 =	6.6	ft, L_3 =	4	ft
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	1.25	ft
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in
PANEL GRADE (0 or 1)	=	0	<= Structural I				
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in	w_{DL} =	1040	plf,wall dead load	
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d	w_{LL} =	240	plf,wall live load	
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5					

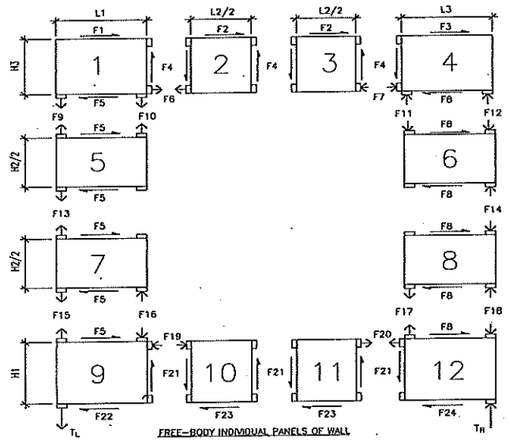


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 3 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h/w = 1.3 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)
1	4.00	1.25	-242
2	3.30	1.25	1116
3	3.30	1.25	1116
4	4.00	1.25	-242
5	4.00	2.50	679
6	4.00	2.50	679
7	4.00	2.50	679
8	4.00	2.50	679
9	4.00	2.75	-1125
10	3.30	2.75	-507
11	3.30	2.75	-507
12	4.00	2.75	-1125

NO.	FORCE (lb/)	NO.	FORCE (lb/)
F1	-967	F13	1395
F2	3683	F14	1395
F3	-967	F15	3093
F4	1395	F16	1698
F5	2716	F17	1698
F6	3683	F18	3093
F7	3683	F19	7214
F8	2716	F20	7214
F9	-302	F21	-1395
F10	1698	F22	-4498
F11	1698	F23	7214
F12	-302	F24	-4498

THE UNIT SHEAR FORCE v_s = 1116 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 3 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

F

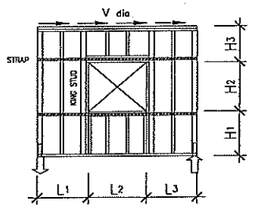
Perforated Strapped Wood Shear Wall

1st Level - Ltotal= 11'-8" Wall

INPUT DATA

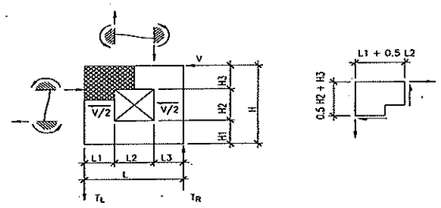
LATERAL FORCE ON DIAPHRAGM: V_{WIND} = pif,for wind — direct shear
(SERVICE LOADS) $V_{SEISMIC}$ = 540 pif,for seismic V= 687 plf

DIMENSIONS:	L_1 =	4.2	ft, L_2 =	2.5	ft, L_3 =	5	ft	
	H_1 =	2.75	ft, H_2 =	5	ft, H_3 =	4	ft	
KING STUD SECTION		2	pcs, b =	3	in, h =	6	in	
EDGE STUD SECTION		2	pcs, b =	4	in, h =	6	in	
PANEL GRADE (0 or 1)	=	0	<= Structural I					
MINIMUM NOMINAL PANEL THICKNESS	=	15/32	in	w_{DL} =	1600	plf,wall dead load		
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d	w_{LL} =	800	plf,wall live load		
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5						

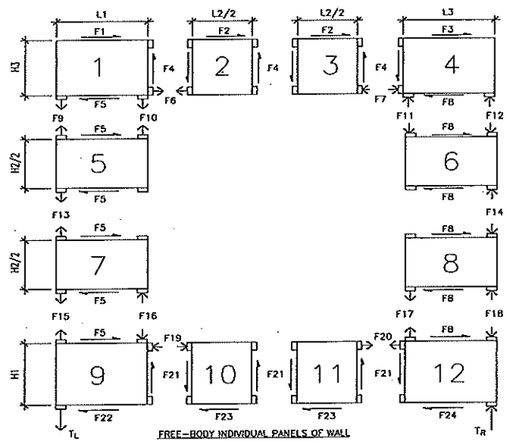


DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD.



ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE–BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.2 < 2 [Satisfactory]

THE FORCES OF FREE-BODY INDIVIDUAL PANELS OF WALL ARE GIVEN BY FIGURE ABOVE AND TABLES AS

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	4.20	4.00	440	F1	1847	F13	3511
2	1.25	4.00	878	F2	1097	F14	3511
3	1.25	4.00	878	F3	2279	F15	5264
4	5.00	4.00	456	F4	3511	F16	1752
5	4.20	2.50	701	F5	2944	F17	1688
6	5.00	2.50	675	F6	1097	F18	5199
7	4.20	2.50	701	F7	1097	F19	9856
8	5.00	2.50	675	F8	3376	F20	11500
9	4.20	2.75	-1648	F9	1759	F21	-2780
10	1.25	2.75	-1011	F10	1752	F22	-6922
11	1.25	2.75	-1011	F11	1688	F23	9856
12	5.00	2.75	-1625	F12	1823	F24	-8124

THE UNIT SHEAR FORCE v_s = 878 plf, (2 Sides Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER CBC Table 2306.4.1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Structural I	10d	1 5/8	15/32	340	510	665	870

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

"ASCE710W.xls" Program
Version 1.0

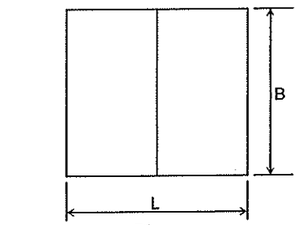
WIND LOADING ANALYSIS - Wall Components and Cladding

Per ASCE 7-10 Code for Buildings of Any Height
Using Part 1 & 3: Analytical Procedure (Section 30.4 & 30.6)

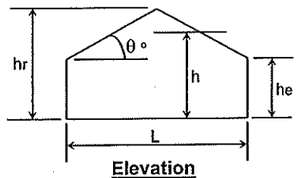
Job Name:	Chestnut Housing	Subject:			
Job Number:	216-009	Originator:		Checker:	

Input Data:

Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	45.00	ft. (hr >= he)
Eave Height, he =	40.00	ft. (he <= hr)
Building Width =	235.00	ft. (Normal to Building Ridge)
Building Length =	58.00	ft. (Parallel to Building Ridge)
Roof Type =	Monoslope	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 28.6-1 & Figure 26.11-1)
Hurricane Region?	Y	
Component Name =	Wall	(Girt, Siding, Wall, or Fastener)
Effective Area, Ae =	1392	ft.^2 (Area Tributary to C&C)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	1.22	deg.
Mean Roof Ht., h =	40.00	ft. (h = he, for roof angle <=10 deg.)

Wall External Pressure Coefficients, GCp:

GCp Zone 4 Pos. =	0.63	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Pos. =	0.63	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 4 Neg. =	-0.72	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)
GCp Zone 5 Neg. =	-0.72	(Fig. 30.4-1, GCp is reduced by 10% for roof angle <=10 deg.)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.11-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 30.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	1.04	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 30.3.2, Eq. 30.3-1)

qh = 27.48 psf qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

Design Net External Wind Pressures (Sect. 30.4 & 30.6):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qz for windward walls, q = qh for leeward walls and side walls
qi = qh for all walls (conservatively assumed per Sect. 30.6)

MA2

"ASCE710W.xls" Program
Version 1.0

Wind Load Tabulation for Wall Components & Cladding

Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	1.04	27.48	22.26	-24.73	22.26	-24.73
	15.00	1.04	27.48	22.26	-24.73	22.26	-24.73
	20.00	1.04	27.48	22.26	-24.73	22.26	-24.73
	25.00	1.04	27.48	22.26	-24.73	22.26	-24.73
	30.00	1.04	27.48	22.26	-24.73	22.26	-24.73
	35.00	1.04	27.48	22.26	-24.73	22.26	-24.73
	40.00	1.04	27.48	22.26	-24.73	22.26	-24.73
For z = hr:	45.00	1.04	27.48	22.26	-24.73	22.26	-24.73
For z = he:	40.00	1.04	27.48	22.26	-24.73	22.26	-24.73
For z = h:	40.00	1.04	27.48	22.26	-24.73	22.26	-24.73

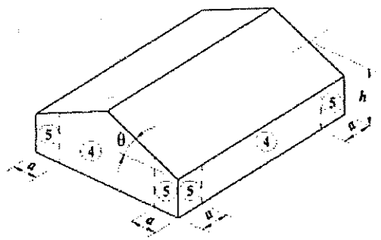
Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (end zones), 'a' = 5.80 ft.
3. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
4. References : a. ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"
by: Kishor C. Mehta and James M. Delahay (2004).

$P_w = 0.6(24.7 \text{ psf})$

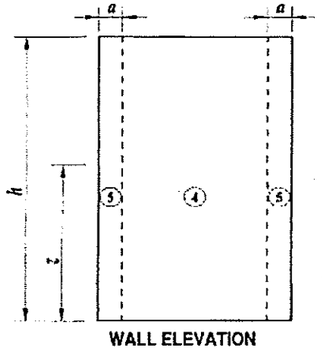
$= 14.8 \text{ psf}$

USE 20 psf

Wall Components and Cladding:



Wall Zones for Buildings with h <= 60 ft.



WALL ELEVATION

Wall Zones for Buildings with h > 60 ft.

MA5

MBC
Consulting Structural Engineers

Project:Chestnut F.H.
Project # :216-009

Wall Load Summary
4 Story Wood

INTERIOR WALL (Unit)

LEVEL	UNIT DEAD [PSF]	UNIT LIVE [PSF]	TRIB WIDTH [FT]	WALL HIGH [FT]	WALL WEIGHT [PLF]	FLOOR DEAD [PLF]	FLOOR LIVE [PLF]	TOTAL LOAD [PLF]	CUMM LOAD [PLF]	4" STUDS
4th to Roof	25	20	7.75	9	90	194	155	439	439	2X4 @ 16
3rd to 4th	40	40	11.875	9	90	475	475	1040	1479	2X4 @ 16
2nd to 3rd	40	40	11.875	9	90	475	475	1040	2519	3X4 @ 16
1st to 2nd	40	40	11.875	9	90	475	475	1040	3559	2-3X4@ 16
Total					360	1618.75	1580	3558.75		

INTERIOR WALL (Unit)

LEVEL	UNIT DEAD [PSF]	UNIT LIVE [PSF]	TRIB WIDTH [FT]	WALL HIGH [FT]	WALL WEIGHT [PLF]	FLOOR DEAD [PLF]	FLOOR LIVE [PLF]	TOTAL LOAD [PLF]	CUMM LOAD [PLF]	4" STUDS
4th to Roof	25	20	2	9	90	50	40	180	180	2X4 @ 16
3rd to 4th	40	40	10.625	9	90	425	425	940	1120	2X4 @ 16
2nd to 3rd	40	40	10.625	9	90	425	425	940	2060	3X4 @ 16
1st to 2nd	40	40	10.625	9	90	425	425	940	3000	2-2x4@16
Total					360	1325	1315	3000		

PARTY WALL

LEVEL	UNIT DEAD [PSF]	UNIT LIVE [PSF]	TRIB WIDTH [FT]	WALL HIGH [FT]	WALL WEIGHT [PLF]	FLOOR DEAD [PLF]	FLOOR LIVE [PLF]	TOTAL LOAD [PLF]	CUMM LOAD [PLF]	4" STUDS
4th to Roof	25	20	2	9	90	50	40	180	180	2X4 @ 16
3rd to 4th	40	40	6.75	9	90	270	270	630	810	2X4 @ 16
2nd to 3rd	40	40	6.75	9	90	270	270	630	1440	3X4 @ 16
1st to 2nd	40	40	6.75	9	90	270	270	630	2070	3X4 @ 16
Total					360	860	850	2070		

EXTERIOR WALL LOAD (Floor Framing Perpendicular to Wall)

LEVEL	UNIT DEAD [PSF]	UNIT LIVE [PSF]	TRIB WIDTH [FT]	WALL HIGH [FT]	WALL WEIGHT [PLF]	FLOOR DEAD [PLF]	FLOOR LIVE [PLF]	TOTAL LOAD [PLF]	CUMM LOAD [PLF]	6" STUDS
4th to Roof	25	20	8.75	9	135	219	175	529	529	2x6 @ 16
3rd to 4th	40	40	6.75	9	135	270	270	675	1204	2x6 @ 16
2nd to 3rd	40	40	6.75	9	135	270	270	675	1879	2x6 @ 16
1st to 2nd	40	40	6.75	11.75	176.25	270	270	716	2595	2x6 @ 16
Total					581.25	1028.75	985	2595		

EXTERIOR WALL LOAD (Floor Framing // to wall)

LEVEL	UNIT DEAD [PSF]	UNIT LIVE [PSF]	TRIB WIDTH [FT]	WALL HIGH [FT]	WALL WEIGHT [PLF]	FLOOR DEAD [PLF]	FLOOR LIVE [PLF]	TOTAL LOAD [PLF]	CUMM LOAD [PLF]	6" STUDS
4th to Roof	25	20	14.75	9	90	369	295	754	754	2x6 @ 16
3rd to 4th	40	40	1.33	9	90	53	53.2	196	950	2x6 @ 16
2nd to 3rd	40	40	1.33	9	90	53	53.2	196	1147	2x6 @ 16
1st to 2nd	40	40	1.33	11.75	117.5	53	53.2	224	1370	2x6 @ 16
Total					387.5	528.35	454.6	1370.45		

INTERIOR WALL (Corridor)

LEVEL	UNIT DEAD [PSF]	UNIT LIVE [PSF]	TRIB WIDTH [FT]	WALL HIGH [FT]	WALL WEIGHT [PLF]	FLOOR DEAD [PLF]	FLOOR LIVE [PLF]	TOTAL LOAD [PLF]	CUMM LOAD [PLF]	4" STUDS
4th to Roof	25	20	18.5	9	90	463	370	923	923	2x4 @ 8
3rd to 4th	30	100	5.875	9	90	176	587.5	854	1776	2x4 @ 8
2nd to 3rd	30	100	5.875	9	90	176	587.5	854	2630	2x4 @ 8
1st to 2nd	30	100	5.875	9	90	176	587.5	854	3484	2x4 @ 8
Total					360	991.25	2132.5	3483.75		

MAG

Interior Wall Design: With 5 psf Lateral Load (2X4) 3rd and 4th

Interior Pary Wall : 3rd and 4th

	2x4 @ 16" oc		H=9'-0"
Dead	=		[plf]
Live	=		[plf]
Wall	=		[plf]
Total	=	1479	[plf]
Lateral Load	=	6.65	[plf]

stress D+L = 1479 / 1584 = 0.93 < 1 OK

stress D+L+E = 0.93 + 0.211 = 1.14 < 1.6 OK

InteriorWall Design: With 5 psf Lateral Load (3X4) 2nd

Interior Party Wall: 1st and 2nd

	3x4 @ 16" oc		H=9'-0"
Dead	=		[plf]
Live	=		[plf]
Wall	=		[plf]
Total	=	2519	[plf]
Lateral Load	=	6.65	[plf]

stress D+L = 2519 / 2640 = 0.95 < 1 OK

stress D+L+E = 0.95 + 0.124 = 1.08 < 1.6 OK

Interiorr Wall Design: With 5 psf Lateral Load (2-3x4@16) 1st

	2-3X4 @ 16" oc		H=9-0"
Dead	=		[plf]
Live	=		[plf]
Wall	=		[plf]
Total	=	3559	[plf]
Lateral Load	=	6.65	[plf]

stress D+L = 3559 / 5280 = 0.67 < 1 OK

stress D+L+E = 0.67 + 0.062 = 0.74 < 1.6 OK

MA-7

MBC
Consulting Structural Engineers

Project:Chestnut F.H.
Project # :216-009

Interiorr Wall Design: With 5 psf Lateral Load (2-2X4@16) 1st

2-2X4 @ 16" oc H=9-0"

Dead	=		[plf]
Live	=		[plf]
Wall	=		[plf]
Total	=	3000	[plf]
Lateral Load	=	6.65	[plf]

stress D+L = 3000 / 3168 = 0.95 < 1 OK

stress D+L+E = 0.95 + 0.1055 = 1.05 < 1.6 OK

Exterior Wall Design: With 20 psf Wind Load (all level)

2x6 @ 16" oc H=11'-75"

Dead	=		[plf]
Live	=		[plf]
Wall	=		[plf]
Total	=	2595	[plf]
Wind	=	26.6	[plf]

stress D+L = 2595 / 3601 = 0.72 < 1 OK

stress D+L+W = 0.72 + 0.66 = 1.38 < 1.6 OK

Interior Corridor Walls

2X4 @ 8" oc H=9-0"

Dead	=		[plf]
Live	=		[plf]
Wall	=		[plf]
Total	=	3484	[plf]
Lateral Load	=	6.65	[plf]

stress D+L = 3484 / 3168 = 1.10 < 1.25 OK

stress D+L+E = 1.10 + 0.211 = 1.31 < 1.6 OK

MA 8

MBC
Consulting Structural Engineers

Project:Chestnut S.H.
Project # :216-009
Pg:_____
By:_____

Capacity of Stud Wall :

Timber = DF#2 Larch

Fc =	1300 psi	KcE =	0.3
Fc(perp) =	625 psi	c =	0.8
E =	1,600,000 psi	CD =	1
ref :	NDS -Table 4A	Fc* =	1495 psi
		CF =	1.15

2x4 Stud Wall @ 16" o.c.

Wall Heigh ft	$\left(\frac{l_e}{d}\right)_x$	E' ksi	FcE ksi	Cp	F'c ksi	A ft^2	Allow P lb	Allow W plf
8	27.4	1,600	0.638	0.380	494	5.25	2594	1946
9	30.9	1,600	0.504	0.309	402	5.25	2112	1584
10	34.3	1,600	0.408	0.256	332	5.25	1744	1308
11	37.7	1,600	0.337	0.214	278	5.25	1461	1096
12	41.1	1,600	0.284	0.182	236	5.25	1240	930
13	44.6	1,600	0.242	0.156	203	5.25	1064	798
14	**48.0**	**1,600**	**0.208**	**0.135**	**176**	**5.25**	**922**	**692**
15	51.4	1,600	0.181	0.118	154	5.25	807	605
16	54.9	1,600	0.160	0.104	136	5.25	712	534
18	61.7	1,600	0.126	0.083	108	5.25	565	424

3x4 Stud Wall @ 16" o.c.

Wall Heigh ft	$\left(\frac{l_e}{d}\right)_x$	E' ksi	FcE ksi	Cp	F'c ksi	A ft^2	Allow P lb	Allow W plf
8	27.4	1,600	0.638	0.380	494	8.75	4324	3243
9	30.9	1,600	0.504	0.309	402	8.75	3520	2640
10	34.3	1,600	0.408	0.256	332	8.75	2907	2180
11	37.7	1,600	0.337	0.214	278	8.75	2435	1826
12	41.1	1,600	0.284	0.182	236	8.75	2066	1549
13	44.6	1,600	0.242	0.156	203	8.75	1773	1330
14	48.0	1,600	0.208	0.135	176	8.75	1537	1153
15	51.4	1,600	0.181	0.118	154	8.75	1345	1009
16	54.9	1,600	0.160	0.104	136	8.75	1186	890
17	58.3	1,600	0.141	0.093	120	8.75	1054	790
18	61.7	1,600	0.126	0.083	108	8.75	942	706

4x4 Stud Wall @ 16" o.c.

Wall Heigh ft	$\left(\frac{l_e}{d}\right)_x$	E' ksi	FcE ksi	Cp	F'c ksi	A ft^2	Allow P lb	Allow W plf
8	27.4	1,600	0.638	0.380	494	12.25	6054	4540
9	30.9	1,600	0.504	0.309	402	12.25	4928	3696
10	34.3	1,600	0.408	0.256	332	12.25	4070	3053
11	37.7	1,600	0.337	0.214	278	12.25	3409	2557
12	41.1	1,600	0.284	0.182	236	12.25	2892	2169
13	44.6	1,600	0.242	0.156	203	12.25	2482	1862
14	48.0	1,600	0.208	0.135	176	12.25	2152	1614
15	51.4	1,600	0.181	0.118	154	12.25	1883	1412
16	54.9	1,600	0.160	0.104	136	12.25	1660	1245
17	58.3	1,600	0.141	0.093	120	12.25	1475	1106
18	61.7	1,600	0.126	0.083	108	12.25	1319	989

MBC
Consulting Structural Engineers

Capacity of Stud Wall :

Timber = DF#2 Larch

Fc =	1300 psi	KcE =	0.3
Fc(perp) =	625 psi	c =	0.8
E =	1,700,000 psi	CD =	1
ref : NDS -Table 4A		Fc* =	1430 psi
		CF =	1.10

2x6 Stud Wall @ 16" o.c.

Wall Heigh ft	$\left(\frac{l_e}{d}\right)_s$	E' ksi	FcE ksi	Cp	Fc ksi	A ft^2	Allow P lb	Allow W plf
8	17.5	1,700	1.674	0.742	965	8.25	7963	5972
9	19.6	1,700	1.323	0.663	862	8.25	7115	5336
10	21.8	1,700	1.071	0.585	760	8.25	6270	4703
11	24.0	1,700	0.885	0.512	665	8.25	5489	4117
12	26.2	1,700	0.744	0.448	582	8.25	4802	3601
13	28.4	1,700	0.634	0.393	510	8.25	4210	3158
14	30.5	1,700	0.547	0.346	449	8.25	3708	2781
16	34.9	1,700	0.418	0.272	354	8.25	2920	2190
17	37.1	1,700	0.371	0.244	317	8.25	2612	1959
18	39.3	1,700	0.331	0.219	285	8.25	2348	1761

3x6Stud Wall @ 16" o.c.

Wall Heigh ft	$\left(\frac{l_e}{d}\right)_s$	E' ksi	FcE ksi	Cp	Fc ksi	A ft^2	Allow P lb	Allow W plf
8	17.5	1,700	1.674	0.742	965	13.75	13272	9954
9	19.6	1,700	1.323	0.663	862	13.75	11859	8894
10	21.8	1,700	1.071	0.585	760	13.75	10450	7838
11	24.0	1,700	0.885	0.512	665	13.75	9149	6862
12	26.2	1,700	0.744	0.448	582	13.75	8003	6002
13	28.4	1,700	0.634	0.393	510	13.75	7017	5263
14	30.5	1,700	0.547	0.346	449	13.75	6180	4635
16	34.9	1,700	0.418	0.272	354	13.75	4867	3650
17	37.1	1,700	0.371	0.244	317	13.75	4354	3265
18	39.3	1,700	0.331	0.219	285	13.75	3914	2935

MURPHY BURR CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: MA10

Printed: 14 SEP 2016, 4:03PM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Lic. # : KW-06002966

Description : Interior Stud Wall for 5 psf

Wood Beam Design : 2x4: 9ft : 5psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **2x4, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

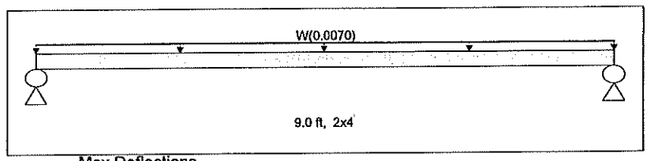
Applied Loads

Unif Load: W = 0.0070 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.211** : 1
fb : Actual : 277.71 psi at 4.500 ft in Span # 1
Fb : Allowable : 1,319.18 psi
Load Comb : +W+H

Max fv/FvRatio = **0.047** : 1
fv : Actual : 8.46 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +W+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.03		
Right Support					0.03		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.121 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	891 >180

Wood Beam Design : 3x4: 9ft : 5psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3x4, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

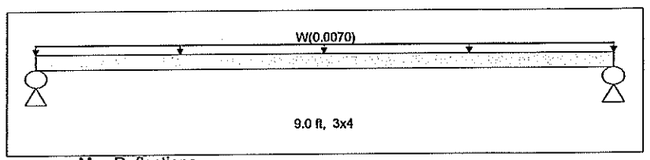
Applied Loads

Unif Load: W = 0.0070 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.124** : 1
fb : Actual : 166.63 psi at 4.500 ft in Span # 1
Fb : Allowable : 1,341.15 psi
Load Comb : +W+H

Max fv/FvRatio = **0.028** : 1
fv : Actual : 5.08 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +W+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.03		
Right Support					0.03		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.073 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	1485 >180

Wood Beam Design : 2x6: 9ft : 5psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

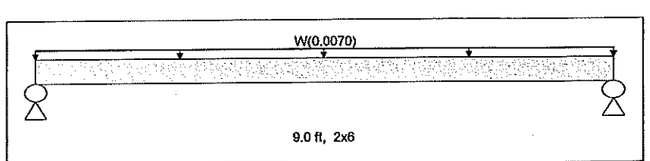
Applied Loads

Unif Load: W = 0.0070 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.100** : 1
fb : Actual : 112.46 psi at 4.500 ft in Span # 1
Fb : Allowable : 1,127.84 psi
Load Comb : +W+H

Max fv/FvRatio = **0.029** : 1
fv : Actual : 5.15 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +W+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.03		
Right Support					0.03		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.031 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	3459 >180

MURPHY BURR CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: MAII

Printed: 14 SEP 2018, 4:03PM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Wood Beam Design : 3x4: 11.75ft : 5psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3x4, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

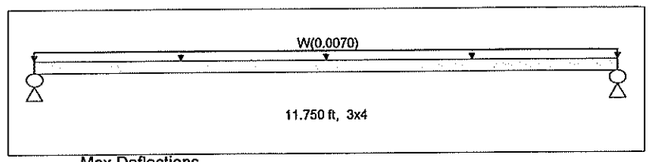
Applied Loads

Unif Load: W = 0.0070 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.212** : 1
fb : Actual : 284.01 psi at 5.875 ft in Span # 1
Fb : Allowable : 1,338.03 psi
Load Comb : +W+H

Max fv/FvRatio = **0.037** : 1
fv : Actual : 6.72 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +W+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.04		
Right Support					0.04		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.211 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	667 >180

Wood Beam Design : 2x6: 11.75ft : 5psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

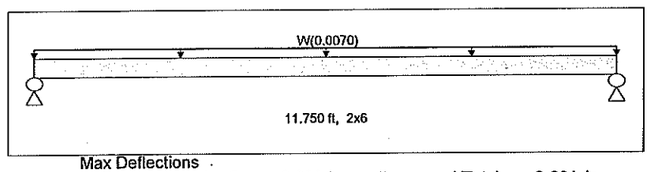
Applied Loads

Unif Load: W = 0.0070 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.174** : 1
fb : Actual : 191.69 psi at 5.875 ft in Span # 1
Fb : Allowable : 1,103.52 psi
Load Comb : +W+H

Max fv/FvRatio = **0.038** : 1
fv : Actual : 6.93 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +W+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.04		
Right Support					0.04		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.091 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	1554 >180

Wood Beam Design : 3x6: 11.75ft : 5psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

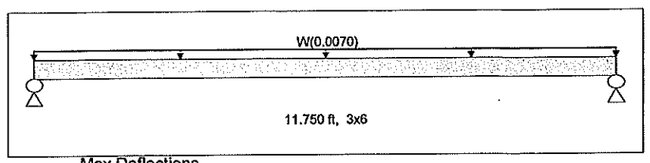
Applied Loads

Unif Load: W = 0.0070 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.100** : 1
fb : Actual : 115.01 psi at 5.875 ft in Span # 1
Fb : Allowable : 1,155.02 psi
Load Comb : +W+H

Max fv/FvRatio = **0.023** : 1
fv : Actual : 4.16 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +W+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.04		
Right Support					0.04		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.054 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	2590 >180

MURPHY BURR CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: MA12

Printed: 14 SEP 2016, 4:08PM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Lic. # : KW-06002966

Description : Ext Stud Wall for 20 psf

Wood Beam Design : 2X6: 9ft : 20 psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

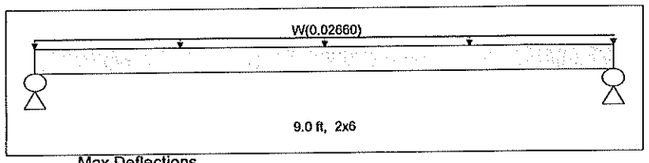
Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900 psi	Fc - Prll	1350 psi	Fv	180 psi	Ebend- xx	1600 ksi	Density	32.21 pcf
Fb - Compr	900 psi	Fc - Perp	625 psi	Ft	575 psi	Eminbend - xx	580 ksi		

Applied Loads

Unif Load: W = 0.02660 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio =	0.379 : 1		
fb : Actual :	427.36 psi	at 4.500 ft	in Span # 1
Fb : Allowable :	1,127.84 psi		
Load Comb :	+W+H		
Max fv/FvRatio =	0.109 : 1		
fv : Actual :	19.59 psi	at 0.000 ft	in Span # 1
Fv : Allowable :	180.00 psi		
Load Comb :	+W+H		



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.12		
Right Support					0.12		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.119 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	910 >180

Wood Beam Design : 3X6: 9ft : 20 psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

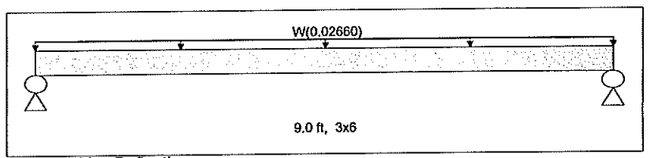
Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: W = 0.02660 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio =	0.221 : 1		
fb : Actual :	256.42 psi	at 4.500 ft	in Span # 1
Fb : Allowable :	1,159.10 psi		
Load Comb :	+W+H		
Max fv/FvRatio =	0.065 : 1		
fv : Actual :	11.75 psi	at 0.000 ft	in Span # 1
Fv : Allowable :	180.00 psi		
Load Comb :	+W+H		



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.12		
Right Support					0.12		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.071 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	1517 >180

Wood Beam Design : 2X6: 11.75ft : 20 psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

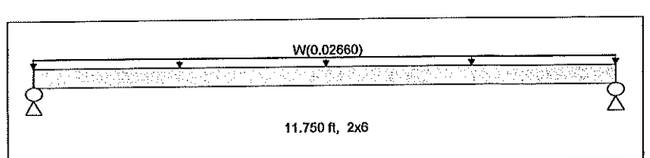
Wood Species :	Douglas Fir - Larch				Wood Grade : No.2				
Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: W = 0.02660 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio =	0.660 : 1		
fb : Actual :	728.42 psi	at 5.875 ft	in Span # 1
Fb : Allowable :	1,103.52 psi		
Load Comb :	+W+H		
Max fv/FvRatio =	0.146 : 1		
fv : Actual :	26.33 psi	at 0.000 ft	in Span # 1
Fv : Allowable :	180.00 psi		
Load Comb :	+W+H		



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.16		
Right Support					0.16		

Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.345 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	409 >180

MURPHY · BURR · CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: MA13

Printed: 14 SEP 2018, 4:08PM

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY - BURR - CURRY, INC.

Multiple Simple Beam

Lic. # : KW-06002966

Wood Beam Design : 3X6: 11.75ft : 20 psf

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir - Larch
Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	32.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

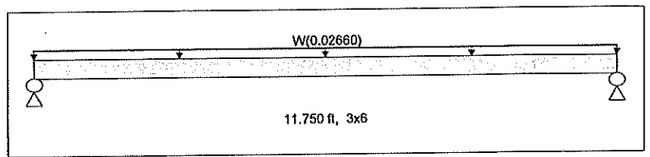
Unif Load: W = 0.02660 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.378 : 1
fb : Actual : 437.05 psi at 5.875 ft in Span # 1
Fb : Allowable : 1,155.02 psi
Load Comb : +W+H

Max fv/FvRatio = 0.088 : 1
fv : Actual : 15.80 psi at 0.000 ft in Span # 1
Fv : Allowable : 180.00 psi
Load Comb : +W+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support					0.16		
Right Support					0.16		



Max Deflections

Downward L+Lr+S	0.000 in	Downward Total	0.207 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	0 <360	Total Defl Ratio	681 >180

MB 1

MBC Consultant Structural Engineers

Project: Chestnut . F. H.
Project #: 216-008
11/6/2018

SHEARWALL DESIGN

WALL TYPE **A** **Va = 340 [PLF] (ASD)**

WALL PLY+NAILING:

PLYWOOD	1/2" STRUCTU 1		
NAILING	10d @	6	" o.c.

SILL PLATE CONNECTION: **2x SILL**

NAILING 16d @ 4 " o.c.

P =	1.25	
cd =	0.64	
Z =	141	[#/NAIL]
Z' =	145.1	[#/NAIL]
Va =	435	[PLF]

DBL. TOP PLATE: **2-2x PLATE**

A35 OR MPA1 CLIPS @ 16 " o.c.

$V_{160\%}$=	695	[#/clip]
V a =	521	[PLF]
Vreq =	425	[PLF, 25% Increase]

ANCHOR BOLTS: **3x SILL**

BOLT 5/8" DIA. A.B. @ 48 " o.c.

Z =	1180	[#/BOLT]
Z' =	1888	[#/BOLT]
Va =	472	[PLF]

MB 2

MBC Consultant Structural Engineers

Project: Chestnut . F. H.
Project #: 216-008
11/6/2018

SHEARWALL DESIGN

WALL TYPE B Va = 510 [PLF] (ASD)

WALL PLY+NAILING:

PLYWOOD	1/2" STRUCTU 1		
NAILING	10d @	4	" o.c.

SILL PLATE CONNECTION: 2x SILL

NAILING	16d @	4	" o.c.

P = 2.25
cd = 1.00
Z = 141 [#/NAIL]
Z' = 225.6 [#/NAIL]

Va = 677 [PLF]

DBL. TOP PLATE: 2-2x PLATE

A35 OR MPA1 CLIPS @	12	" o.c.

$V_{160\%}$ = 695 [#/clip]

V a = 695 [PLF]

Vreq = 637.5 [PLF, 25% Increase]

SDS	1/4" SDS x 6" @	10	" o.c.

P = 3
cd = 1.00
Z = 350 [#/NAIL]
Z' = 560.0 [#/NAIL]

Va = 672 [PLF]

ANCHOR BOLTS: 3x SILL

BOLT	5/8" DIA. A.B. @	32	" o.c.

Z = 1180 [#/BOLT]
Z' = 1888 [#/BOLT]

Va = 708 [PLF]

MB3

MBC Consultant Structural Engineers

Project: Chestnut . F. H.
Project #: 216-008
11/6/2018

SHEARWALL DESIGN

WALL TYPE C Va = 665 [PLF] (ASD)

WALL PLY+NAILING:

PLYWOOD	1/2" STRUCTU 1		
NAILING	10d @	3	" o.c.

SILL PLATE CONNECTION: 2x SILL

SDS	1/4" SDS x 6" @	10	" o.c.

P = 3.75
cd = 1.00
Z = 350 [#/NAIL]
Z' = 560.0 [#/NAIL]

Va = 672 [PLF]

DBL. TOP PLATE: 2-2x PLATE

A35 OR MPA1 CLIPS @		10	" o.c.

$V_{160\%}$ = 695 [#/clip]

V a = 834 [PLF]

V req = 831.25 [PLF, 25% Increase]

SDS	1/4" SDS x 6" @	8	" o.c.

P = 3
cd = 1.00
Z = 350 [#/NAIL]
Z' = 560.0 [#/NAIL]

Va = 840 [PLF]

ANCHOR BOLTS: 3x SILL

BOLT	5/8" DIA. A.B. @	32	" o.c.

Z = 1180 [#/BOLT]
Z' = 1888 [#/BOLT]

Va = 708 [PLF]

MB4

MBC Consultant Structural Engineers

Project: Chestnut . F. H.
Project #: 216-008
11/6/2018

SHEARWALL DESIGN

WALL TYPE | D | **Va = 870 [PLF] (ASD)**

WALL PLY+NAILING:

PLYWOOD	1/2" STRUCTU	1	
NAILING	10d @	2	" o.c.

SILL PLATE CONNECTION: **2x SILL**

SDS	**1/4" SDS x 6" @**	**7**	**" o.c.**

P =	3.75	
cd =	1.00	
Z =	350	[#/NAIL]
Z' =	560.0	[#/NAIL]
Va =	960	[PLF]

DBL. TOP PLATE: **2-2x PLATE**

SDS	**1/4" SDS x6" @**	**6**	**" o.c.**

P =	3	
cd =	1.00	
Z =	350	[#/NAIL]
Z' =	560.0	[#/NAIL]
Va =	1120	[PLF]
V req =	1087.5	[PLF, 25% Increase]

ANCHOR BOLTS: **3x SILL**

BOLT	**5/8" DIA. A.B. @**	**24**	**" o.c.**

Z =	1180	[#/BOLT]
Z' =	1888	[#/BOLT]
Va =	944	[PLF]

MB5

MBC Consultant Structural Engineers

Project: Chestnut . F. H.
Project #: 216-008
11/6/2018

SHEARWALL DESIGN

WALL TYPE | E | **Va = 1020 [PLF] (ASD)**

WALL PLY+NAILING:

PLYWOOD	1/2" STRUCTU 1 (2 SIDE)		
NAILING	10d @	4	" o.c.

SILL PLATE CONNECTION: **2x SILL**

SDS	**1/4" SDS x 6" @**	**6**	**" o.c.**
P =	3.75		
cd =	1.00		
Z =	350 [#/NAIL]		
Z' =	560.0 [#/NAIL]		
Va =	1120 [PLF]		

DBL. TOP PLATE: **2-2x PLATE**

SDS	**1/4" SDS x6" @**	**6**	**" o.c.**
P =	3		
cd =	1.00		
Z =	350 [#/NAIL]		
Z' =	560.0 [#/NAIL]		
Va =	1120 [PLF]		
V req =	637.5 [PLF, 25% increase]	*	

ANCHOR BOLTS: **3x SILL**

BOLT	**5/8" DIA. A.B. @**	**16**	**" o.c.**
Z =	1180 [#/BOLT]		
Z' =	1888 [#/BOLT]		
Va =	1416 [PLF]		

* At Exterior Wall location 1/2 of shear is transfer at exterior cont. side of plywood

MBC

MBC Consultant Structural Engineers

Project: Chestnut . F. H.
Project #: 216-008
11/6/2018

SHEARWALL DESIGN

WALL TYPE F **Va = 1330 [PLF] (ASD)**

WALL PLY+NAILING:

PLYWOOD	1/2" STRUCTU 1 (2 SIDE)			
NAILING	10d @	3	" o.c.	

SILL PLATE CONNECTION: **2x SILL**

SDS	**1/4" SDS x 6" @**	**5**	**" o.c.**
P =	3.75		
cd =	1.00		
Z =	350 [#/NAIL]		
Z' =	560.0 [#/NAIL]		
Va =	1344 [PLF]		

DBL. TOP PLATE: **2-2x PLATE**

SDS	**1/4" SDS x6" @**	**5**	**" o.c.**	
P =	3			
cd =	1.00			
Z =	350 [#/NAIL]			
Z' =	560.0 [#/NAIL]			
Va =	1344 [PLF]			
V req =	831.25 [PLF, 25% increase]			*
	f			

ANCHOR BOLTS: **3x SILL**

BOLT	**5/8" DIA. A.B. @**	**16**	**" o.c.**
Z =	1180 [#/BOLT]		
Z' =	1888 [#/BOLT]		
Va =	1416 [PLF]		

* At Exterior Wall location 1/2 of shear is transfer at exterior cont. side of plywood

MB7

MBC Consultant Structural Engineers

Project: Chestnut . F. H.
Project #: 216-008
11/6/2018

SHEARWALL DESIGN

WALL TYPE **G** **Va = 1680 [PLF] (ASD)**

WALL PLY+NAILING:

PLYWOOD	1/2" STRUCTU 1 (2 SIDE)		
NAILING	10d @	2	" o.c.

SILL PLATE CONNECTION: **2x SILL**

SDS	**1/4" SDS x 6" @**	**4**	**" o.c.**

P =	3.75
cd =	1.00
Z =	350 [#/NAIL]
Z' =	560.0 [#/NAIL]
Va =	1680 [PLF]

DBL. TOP PLATE: **2-2x PLATE**

SDS	**1/4" SDS x6" @**	**4**	**" o.c.**

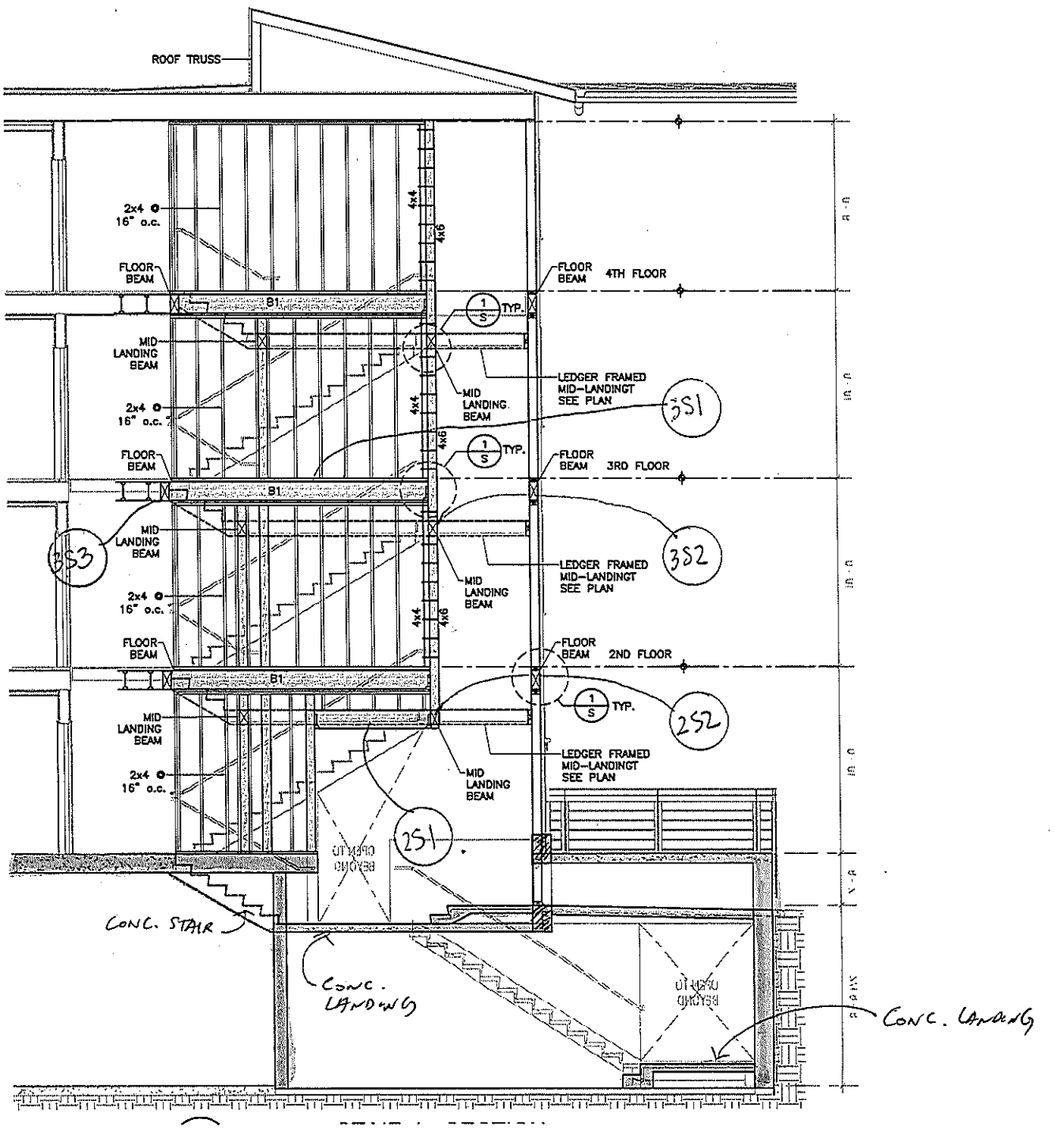
P =	3
cd =	1.00
Z =	350 [#/NAIL]
Z' =	560.0 [#/NAIL]
Va =	1680 [PLF]
V req =	1050 [PLF, 25% increase] *

ANCHOR BOLTS: **3x SILL**

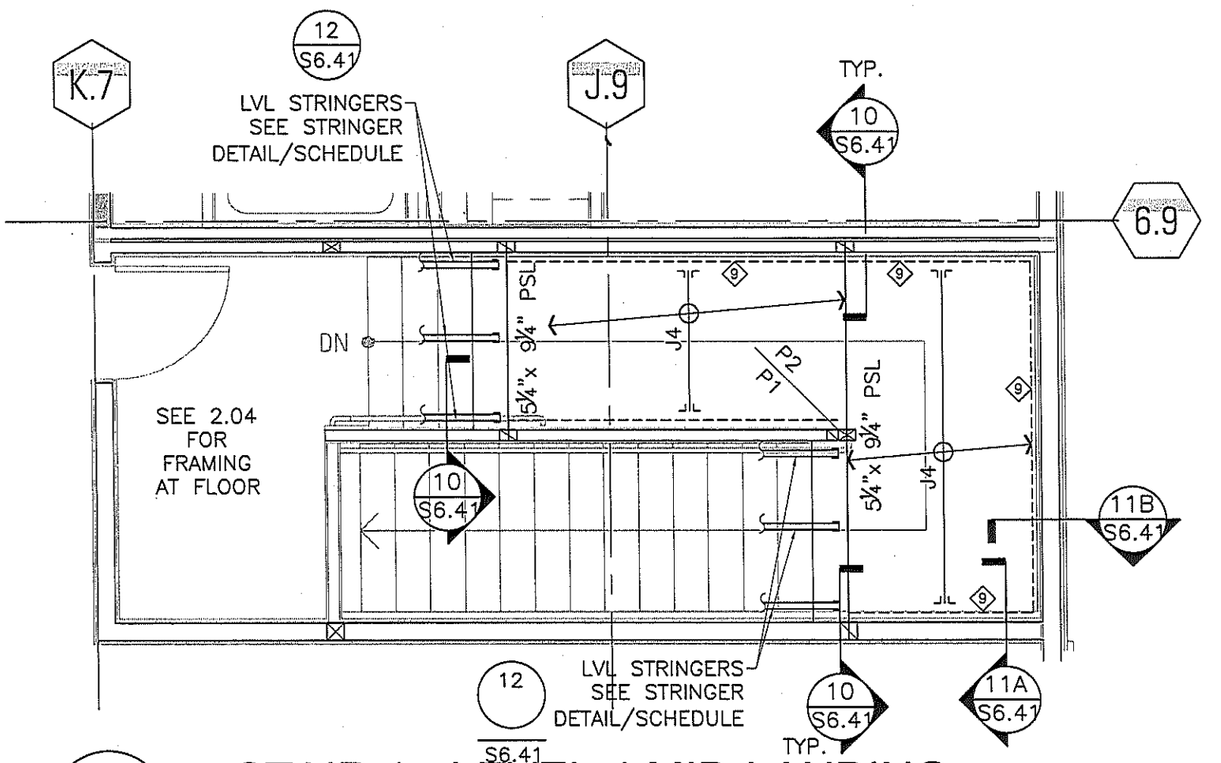
BOLT	**5/8" DIA. A.B. @**	**12**	**" o.c.**

Z =	1180 [#/BOLT]
Z' =	1888 [#/BOLT]
Va =	1888 [PLF]

* At Exterior Wall location 1/2 of shear is transfer at exterior cont. side of plywood


ROOF TRUSS
2x4 @ 16" o.c.
FLOOR BEAM
B1
MID LANDING BEAM
4x4
4x6
FLOOR BEAM 4TH FLOOR
1/S TYP.
LEDGER FRAMED MID-LANDINGT SEE PLAN
MID LANDING BEAM
351
FLOOR BEAM 3RD FLOOR
353
352
FLOOR BEAM 2ND FLOOR
252
251
CONC. STAIR
CONC. LANDING
CONC. LANDING

MC 1

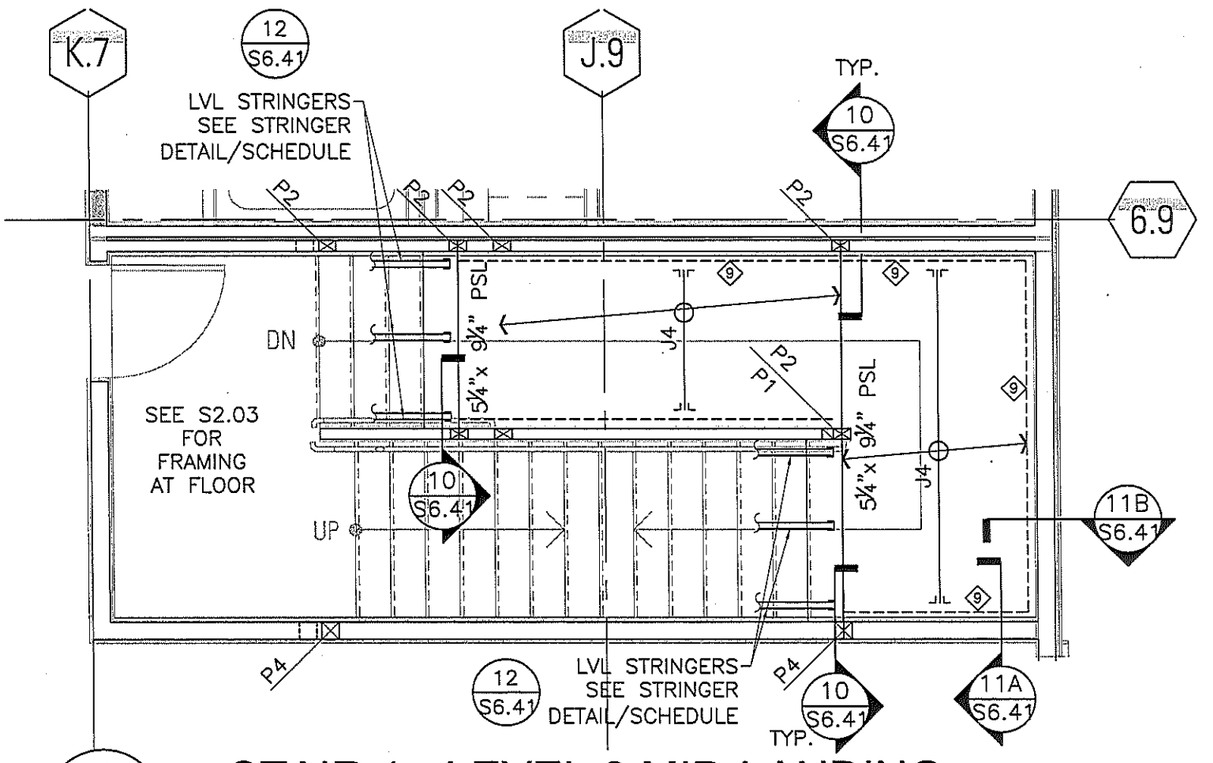


4
S6.40

STAIR 1 - LEVEL 4 MID-LANDING

1/4" = 1'-0"

MC2

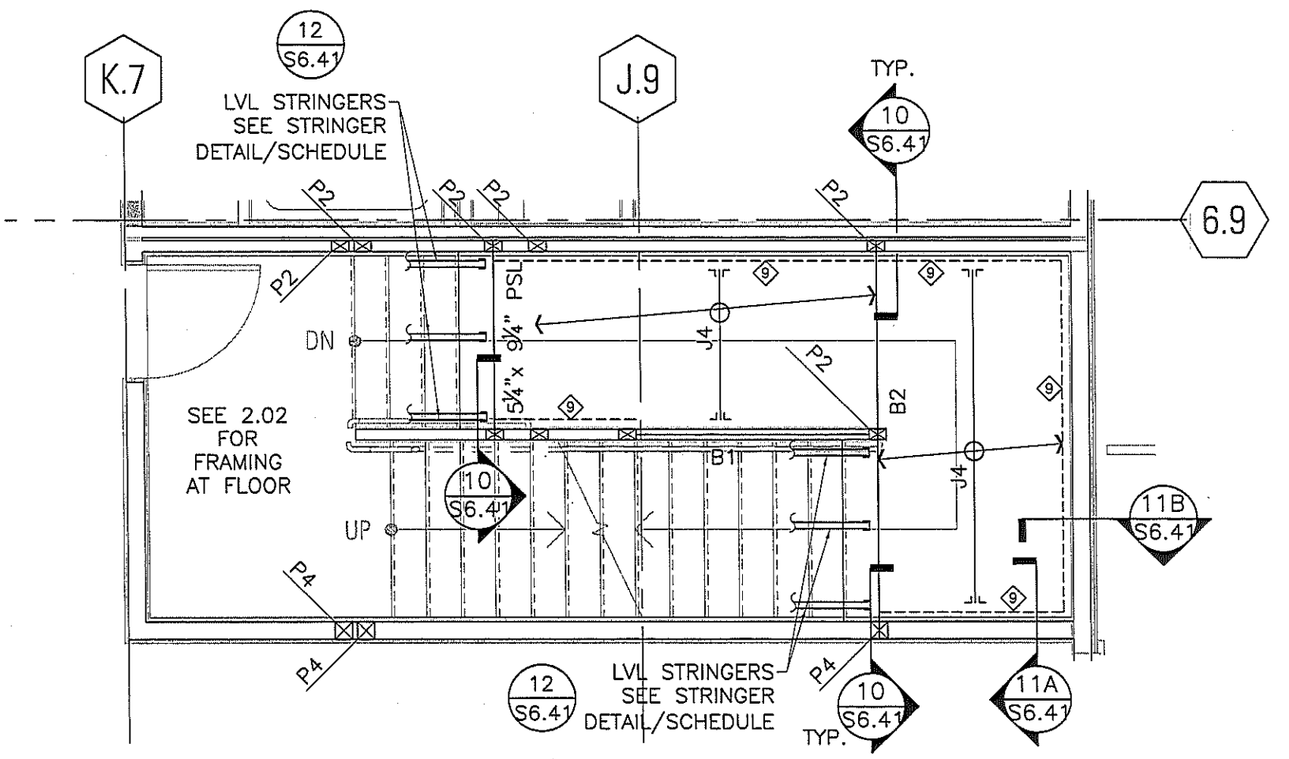


4
S6.40

STAIR 1 - LEVEL 3 MID-LANDING

1/4" = 1'-0"

MC 3

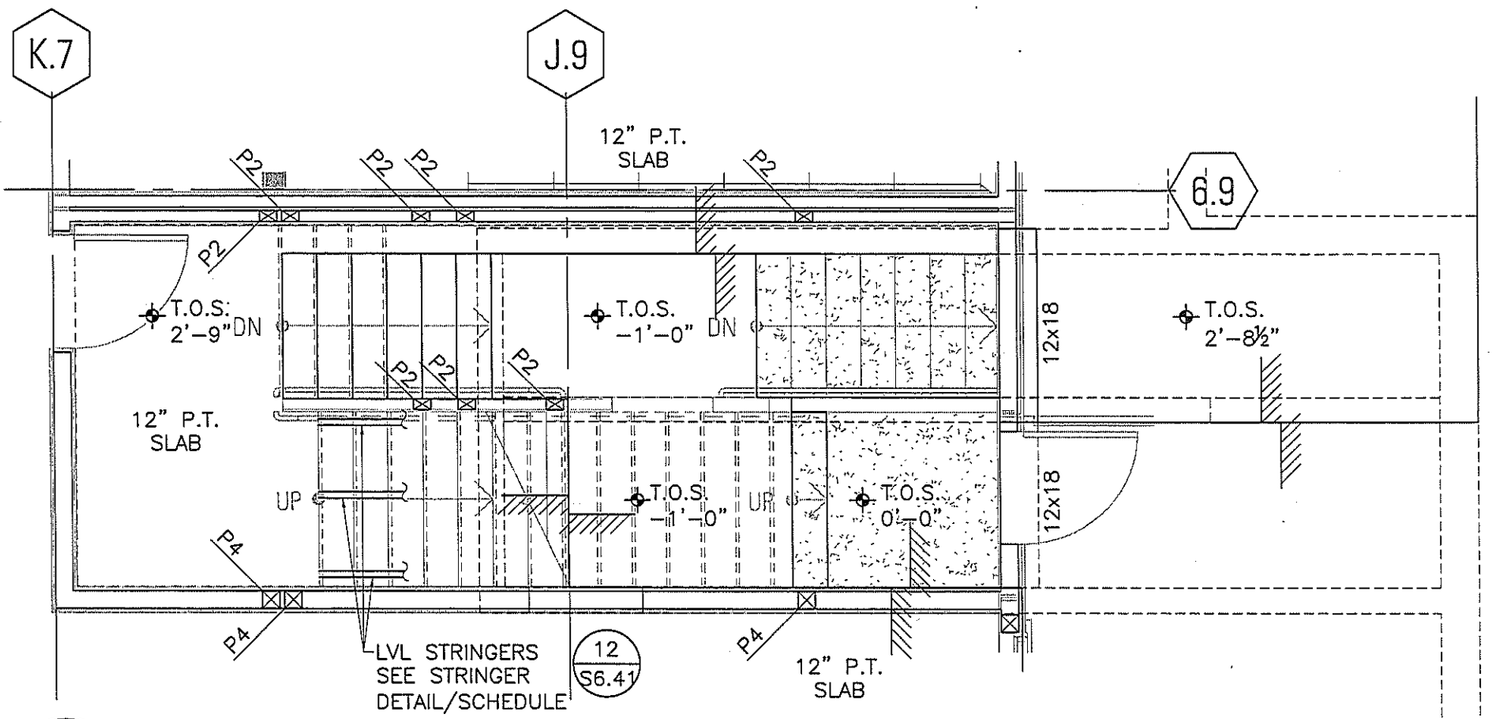


3
S6.40

STAIR 1 - LEVEL 2 MID-LANDING

1/4" = 1'-0"

MC4



2
S6.40

STAIR 1 - LEVEL 1 FRAMING PLAN

1/4" = 1'-0"

MC5

MURPHY BURR CURRY, INC.
CONSULTING STRUCTURAL ENGINEERS
85 SECOND STREET, SUITE 501 • SAN FRANCISCO, CA 94105
PHONE: (415) 546-0431 • FAX: (415) 882-7257

STAIR SUPPORT

9'-0 STRINGER

@ CENTER $W_{DL} = 30 \times 2.5 = 75^{\#/1}$

$LL = 100 \times 2.5 = 250^{\#/1}$

$R = 1.47^{K}$

@ EDGE $W_{DL} = 30 \times 1.25 = 37.5^{\#/1}$

$LL = 100 \times 1.25 = 125^{\#/1}$

$R = 0.73$ ✓

USE (2) 5 3/4" LVL.

13'-0" STRINGER

@ Center $W_{DL} = 75^{\#/1}$

$LL = 250^{\#/1}$

$R = 2.12^{K}$

@ EDGE $W_{DL} = 37.5^{\#/1}$

$LL = 125^{\#/1}$

$R = 1.06^{K}$

USE (2) 7 1/2 LVL @ EDGE

USE (3) 7 1/2 LVL @ Center

Simp. HUS410 ok $R_{cap} = 2.1^{K}$

MURPHY BURR CURRY, INC.
CONSULTING STRUCTURAL ENGINEERS
85 SECOND STREET, SUITE 501 • SAN FRANCISCO, CA 94105
PHONE: (415) 546-0431 • FAX: (415) 882-7257

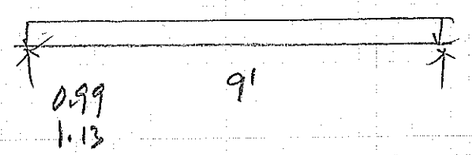
Page No.: MC7 Of:
Project:
Project No.:
Date: By:

STAIR Support

3¼ x 11 7/8 PSL (331)

$W_{DL} = 30 \times 5/2 + 15 \times 9' = 240^{\#/'}$

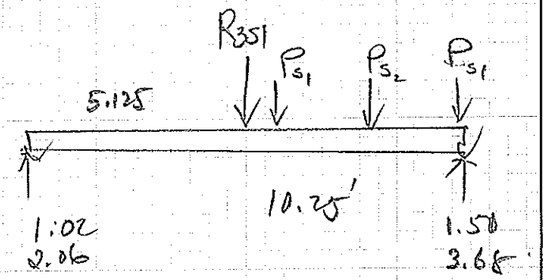
$W = 100 \times 5/2 = 250^{\#/'}$



5¼ x 11 7/8 PSL (882)

$W_{DL} = 30 \times 1.33 = 40$

$W = 100 \times 1.33 = 133$



$R_{351} = 0.99 + 1.13$

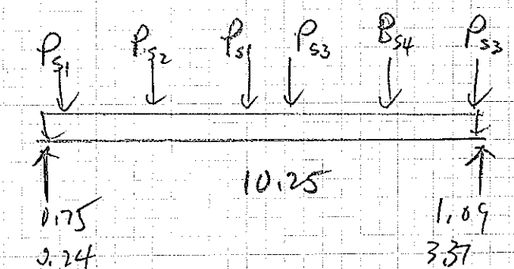
$P_{S1} = 0.25^{k} + 0.81^{k}$

$P_{S2} = 0.49^{k} + 1.63^{k}$

5¼ x 11 7/8 PSL (353)

$P_{S3} = 37.5 \times 2 = 75$
$125 \times 2 = 250$

$P_{S4} = 75 \times 2 = 150$
$250 \times 2 = 500$



MURPHY BURR CURRY, INC.
CONSULTING STRUCTURAL ENGINEERS
85 SECOND STREET, SUITE 501 • SAN FRANCISCO, CA 94105
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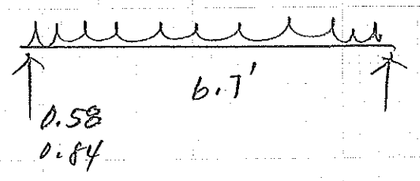
$3\frac{1}{2} \times 11\frac{7}{8}$ PSL

(2S1)

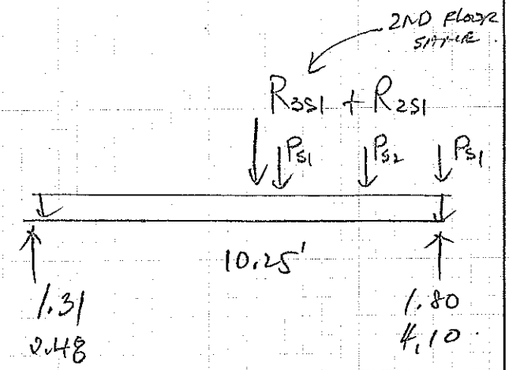
$W_{bL} = 30 \times 2.5 + 15 \times 6 = 165^{\#/'}$

$W = 100 \times 2.5 = 250^{\#/'}$



$5\frac{1}{4} \times 11\frac{7}{8}$ PSL

(2S2)



$R_{3S1} = 0.99 + 1.13^{K}$

$R_{2S1} = 0.58 + 0.84^{K}$

$P_{S1} = 0.25 + 0.81^{K}$

$P_{S2} = 0.49 + 1.63^{K}$

MURPHY BURR CURRY INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: MC9

Printed: 31 OCT 2018, 12:30PM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. #: KW-06002966 Licensee : MURPHY BURR CURRY, INC.

Multiple Simple Beam

Description :

Wood Beam Design : 9'-0" Span Stringer at center

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **2-1.75x7.25, Microllam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 1.9 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	1,900.0 ksi	Density 32.210 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	965.71 ksi	

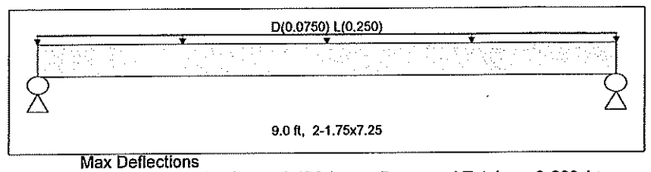
Applied Loads

Unif Load: D = 0.0750, L = 0.250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.499 : 1
fb : Actual : 1,287.85 psi at 4.500 ft in Span # 1
Fb : Allowable : 2,578.62 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.263 : 1
fv : Actual : 74.93 psi at 0.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.34	1.13					
Right Support	0.34	1.13					

Max Deflections

Downward L+Lr+S	0.176 in	Downward Total	0.228 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	614 >360	Total Defl Ratio	472 >180

Wood Beam Design : 9'-0" Span Stringer at edge

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **2-1.75x5.5, Microllam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 1.9 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	1,900.0 ksi	Density 32.210 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	965.71 ksi	

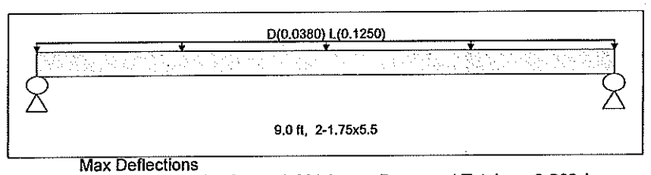
Applied Loads

Unif Load: D = 0.0380, L = 0.1250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.434 : 1
fb : Actual : 1,122.33 psi at 4.500 ft in Span # 1
Fb : Allowable : 2,584.33 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.180 : 1
fv : Actual : 51.44 psi at 0.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.17	0.56					
Right Support	0.17	0.56					

Max Deflections

Downward L+Lr+S	0.201 in	Downward Total	0.262 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	536 >360	Total Defl Ratio	411 >180

Wood Beam Design : 13'-0" Span Stringer at center

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **(3)1.75x7.5, TJ: Microllam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 1.9 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	1,900.0 ksi	Density 32.210 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	965.71 ksi	

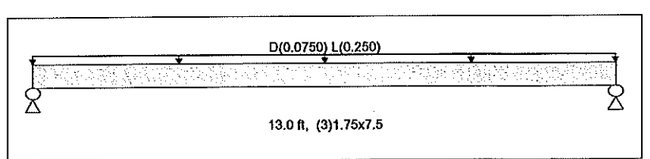
Applied Loads

Unif Load: D = 0.0750, L = 0.250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.647 : 1
fb : Actual : 1,673.90 psi at 6.500 ft in Span # 1
Fb : Allowable : 2,586.47 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.256 : 1
fv : Actual : 72.97 psi at 12.393 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.49	1.63					
Right Support	0.49	1.63					

Max Deflections

Downward L+Lr+S	0.461 in	Downward Total	0.599 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	338 <360	Total Defl Ratio	260 >180

MURPHY BURR CURRY, INC.
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Project Title:
Engineer:
Project Descr:

Project ID: MC10

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Licensee : MURPHY BURR CURRY, INC

Multiple Simple Beam

Lic. # : KW-06002966

Wood Beam Design : 13'-0" Span Stringer at edge

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **2-1.75x7.25, Microllam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending

Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 1.9 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	1,900.0 ksi	Density	32.210 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	965.71 ksi		

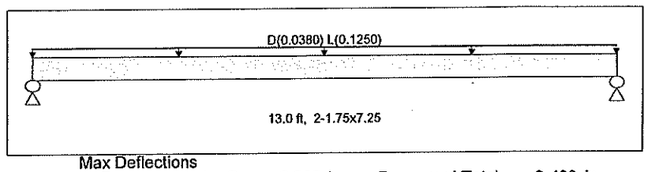
Applied Loads

Unif Load: D = 0.0380, L = 0.1250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.525 : 1
fb : Actual : 1,347.64 psi at 6.500 ft in Span # 1
Fb : Allowable : 2,567.00 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.201 : 1
fv : Actual : 57.20 psi at 12.437 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.25	0.81					
Right Support	0.25	0.81					

Max Deflections

Downward L+Lr+S	0.382 in	Downward Total	0.499 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	407 >360	Total Defl Ratio	312 >180

MURPHY BURR CURRY, INC.
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Project Title:
Engineer:
Project Descr:

Project ID: MC11

Printed: 31 OCT 2016, 1:03PM

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ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21
Lic. # : KW-06002966 Licensee : MURPHY BURR CURRY, INC.

Multiple Simple Beam

Description : Stair Beams

Wood Beam Design : 3S1

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

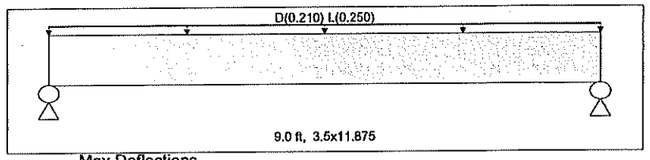
Beam self weight calculated and added to loads
Unif Load: D = 0.210, L = 0.250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.243 : 1
fb : Actual : 693.17 psi at 4.500 ft in Span # 1
Fb : Allowable : 2,853.72 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.207 : 1
fv : Actual : 59.96 psi at 0.000 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.99	1.13					
Right Support	0.99	1.13					



Max Deflections

Downward L+Lr+S	0.038 in	Downward Total	0.071 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	2843 >360	Total Defl Ratio	1514 >180

Wood Beam Design : 3S2

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **5.25x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

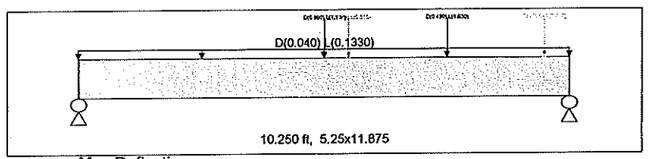
Beam self weight calculated and added to loads
Unif Load: D = 0.040, L = 0.1330 k/ft, Trib= 1.0 ft
Point: D = 0.990, L = 1.130 k @ 5.125 ft
Point: D = 0.250, L = 0.810 k @ 5.625 ft
Point: D = 0.250, L = 0.810 k @ 9.750 ft
Point: D = 0.490, L = 1.630 k @ 7.688 ft

Design Summary

Max fb/Fb Ratio = 0.450 : 1
fb : Actual : 1,295.38 psi at 5.125 ft in Span # 1
Fb : Allowable : 2,879.42 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.328 : 1
fv : Actual : 95.26 psi at 9.293 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.02	2.06					
Right Support	1.51	3.68					



Max Deflections

Downward L+Lr+S	0.107 in	Downward Total	0.159 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1148 >360	Total Defl Ratio	773 >180

Wood Beam Design : 3S3

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **5.25x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.040, L = 0.1330 k/ft, Trib= 1.0 ft
Point: D = 0.0750, L = 0.250 k @ 4.625 ft
Point: D = 0.250, L = 0.810 k @ 5.625 ft
Point: D = 0.250, L = 0.810 k @ 9.750 ft
Point: D = 0.490, L = 1.630 k @ 7.688 ft
Point: D = 0.0750, L = 0.250 k @ 0.50 ft
Point: D = 0.150, L = 0.50 k @ 2.563 ft

MURPHY BURR CURRY, INC.
SAN FRANCISCO | CALIFORNIA

Project Title:
Engineer:
Project Descr:

Project ID: MC12

Printed: 31 OCT 2018, 1:03PM

File = H:\Projects 2016\M216-008 Chestnut Square - Family Building\Calcs\216-008 framing.ec6
ENERCALC, INC. 1983-2014, Build:6.14.1.21, Ver:6.14.1.21

Multiple Simple Beam

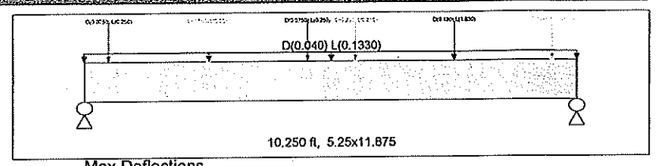
Lic. # : KW-06002966 — Licensee : MURPHY BURR CURRY, INC.

Design Summary

Max fb/Fb Ratio = 0.334 : 1
fb : Actual : 962.00 psi at 5.638 ft in Span # 1
Fb : Allowable : 2,879.42 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.268 : 1
fv : Actual : 77.58 psi at 9.293 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.75	2.24					
Right Support	1.09	3.37					



Max Deflections

Downward L+Lr+S	0.094 in	Downward Total	0.124 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1313 >360	Total Defl Ratio	989 >180

Wood Beam Design : 2S1

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **3.5x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist — Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

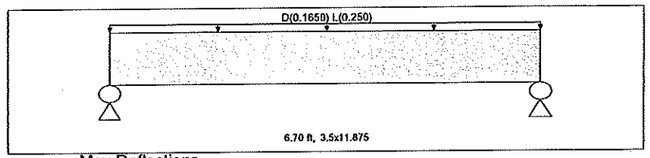
Beam self weight calculated and added to loads
Unif Load: D = 0.1650, L = 0.250 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = 0.121 : 1
fb : Actual : 347.32 psi at 3.350 ft in Span # 1
Fb : Allowable : 2,867.83 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.125 : 1
fv : Actual : 36.25 psi at 5.717 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.58	0.84					
Right Support	0.58	0.84					



Max Deflections

Downward L+Lr+S	0.012 in	Downward Total	0.020 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	6891 >360	Total Defl Ratio	4060 >180

Wood Beam Design : 2S2

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10

BEAM Size : **5.25x11.875, Parallam, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-10 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist — Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	32.210 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

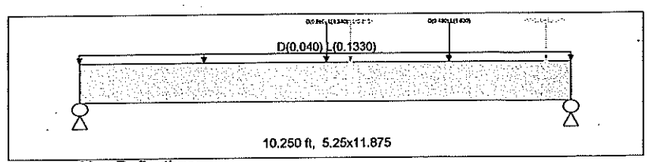
Beam self weight calculated and added to loads
Unif Load: D = 0.040, L = 0.1330 k/ft, Trib= 1.0 ft
Point: D = 0.990, L = 1.130 k @ 5.125 ft
Point: D = 0.250, L = 0.810 k @ 5.625 ft
Point: D = 0.250, L = 0.810 k @ 9.750 ft
Point: D = 0.490, L = 1.630 k @ 7.688 ft
Point: D = 0.580, L = 0.840 k @ 5.125 ft

Design Summary

Max fb/Fb Ratio = 0.573 : 1
fb : Actual : 1,649.26 psi at 5.125 ft in Span # 1
Fb : Allowable : 2,879.42 psi
Load Comb : +D+L+H

Max fv/FvRatio = 0.387 : 1
fv : Actual : 112.35 psi at 9.293 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.31	2.48					
Right Support	1.80	4.10					



Max Deflections

Downward L+Lr+S	0.129 in	Downward Total	0.197 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	950 >360	Total Defl Ratio	624 >180

MURPHY BURR CURRY, INC.
CONSULTING STRUCTURAL ENGINEERS
85 SECOND STREET, SUITE 501 • SAN FRANCISCO, CA 94105
PHONE: (415) 546-0431 • FAX: (415) 882-7257

2x SILL : 9ft

$V_{REQ} = 5\,psf \times 9/2 = 22.5^{\#/'}$

2x SILL : 11" - 9"

$V_{REQ} = 5\,psf \times 11.75/2 = 30^{\#/'}$

0.157ø -XU PIN : 3/4" GMS $V_{ALLOW} = 125^{\#}$.

NAIL @ 24" O.C. MAX W/ 3/4" MIN/MAX EMBED.

NAIL LENGTH: 1½" + ¾ = 2¼"

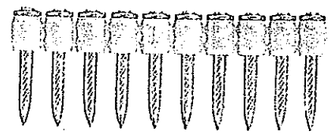
X-U 57 PB x 0.157"ø -

P.D.F. @ 24' O.C.

Premium collated nail for concrete and steel X-U MX

Applications and advantages

- Fastening to concrete and steel, e.g. formwork, wood lath, plastic sheeting, drywall track, sheet metal, etc.
- Fully-knurled point for increased performance on high strength steel and hard concrete
- MX strips of 10 collated fasteners provide high productivity and easy fastener loading



Fastener Designation	Fastener shank length	Fastener shank diameter	Fastener only (qty 100)	Fastener only (qty 1000)	Kit item number (10 x 100) fasteners and cartridges			
					.27 caliber Green (Light)	.27 caliber Yellow (Medium)	.27 caliber Red (Heavy)	.27 caliber Black (X-heavy)
Collated premium fastener X-U 15 MX	5/8 in	0.106 in	-	342000	3510122	3510123	3510124	-
Collated premium fastener X-U 16 MX	5/8 in	0.157 in	237344	-	3510119	3510120	3510121	3496885
Collated premium fastener X-U 19 MX	3/4 in	0.157 in	237345	-	3505357	3496895	3496894	3496890
Collated premium fastener X-U 22 MX	7/8 in	0.157 in	237346	-	3511733	3496897	3496896	-
Collated premium fastener X-U 27 MX	1-1/16 in	0.157 in	237347	-	3511734	3496899	3496898	-
Collated premium fastener X-U 32 MX	1-1/4 in	0.157 in	237348	-	3511735	3496901	3496900	3502612
Collated premium fastener X-U 37 MX	1-7/16 in	0.157 in	237349	-	3511736	3496904	3496903	3496902
Collated premium fastener X-U 42 MX	1-5/8 in	0.157 in	237350	-	-	3496934	3496933	-
Collated premium fastener X-U 47 MX	1-7/8 in	0.157 in	237351	-	-	3496936	3496935	3511713
Collated premium fastener X-U 52 MX	2-1/16 in	0.157 in	237352	-	-	3496938	3496937	3511715
Collated premium fastener X-U 57 MX	2-1/4 in	0.157 in	237353	-	-	3496940	3496939	3511716
Collated premium fastener X-U 62 MX	2-7/16 in	0.157 in	237354	-	-	3496942	3496941	3511718
Collated premium fastener X-U 72 MX	2-13/16 in	0.157 in	237356	-	-	3496944	3496943	3511720

Premium nail for concrete and steel X-U P8 — Hilti does not Make in galvanized

Applications and advantages

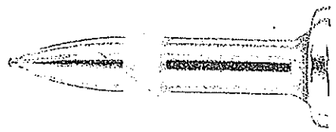
- Attach a variety of materials from thin metals, 2x4s to concrete or steel
- Universal fastener for applications on both standard and high strength concrete or steel

Fastener Designation	Fastener shank length	Fastener shank diameter	Washer diameter (plastic)	Fastener only (qty 100)	Fastener only (qty 1000)	Kit item number (10 x 100) fasteners and cartridges					
						.25 caliber Yellow (Medium)	.25 caliber Red (Heavy)	.27 caliber Green (Light)	.27 caliber Yellow (Medium)	.27 caliber Red (Heavy)	.27 caliber Black (X-heavy)
Premium fastener X-U 16 P8	5/8 in	0.157 in	8 mm	237330	-	3496592	3496591	3505458	3496866	3496865	3496863
Premium fastener X-U 19 P8	3/4 in	0.157 in	8 mm	237331	-	3496868	3496867	3505358	3496871	3496870	3496869
Premium fastener X-U 22 P8	7/8 in	0.157 in	8 mm	237332	-	-	-		3496873	3496872	-
Premium fastener X-U 27 P8	1-1/16 in	0.157 in	8 mm	237333	-	-	-	3511730	3496875	3496874	-
Premium fastener X-U 32 P8	1-1/4 in	0.157 in	8 mm	237334	-	3496877	3496876	3511731	3496879	3496878	3502611
Premium fastener X-U 37 P8	1-7/16 in	0.157 in	8 mm	237335	-	-	-	3511732	3496882	3496881	3496880
Premium fastener X-U 42 P8	1-5/8 in	0.157 in	8 mm	237336	-	-	-	-	-	-	-
Premium fastener X-U 47 P8	1-7/8 in	0.157 in	8 mm	237337	-	-	-	-	3496928	3496927	3511602
Premium fastener X-U 52 P8	2-1/16 in	0.157 in	8 mm	237338	-	-	-	-	3496930	3496929	3511714
Premium fastener X-U 57 P8	2-1/4 in	0.157 in	8 mm	237339	-	-	-	-	-	-	-
Premium fastener X-U 62 P8	2-7/16 in	0.157 in	8 mm	237340	-	-	-	-	3496932	3496931	3511717
Premium fastener X-U 72 P8	2-13/16 in	0.157 in	8 mm	237342	-	-	-	-	-	-	3511719
Premium fastener X-U 15 P8TH	5/8 in	0.145 in	8 mm	-	342215	3496906	3496905	3505356	3496908	3496907	-
Premium fastener X-U 16 P8TH	5/8 in	0.157 in	8 mm	237329	-	3496910	3496909	-	3496913	3496912	3496911
Premium fastener X-U 19 P8TH	3/4 in	0.157 in	8 mm	385781	-	-	-	3505359	3496916	3496915	3496914
Fastener X-U 27 P8TH	1-1/16 in	0.157 in	8 mm	385782	-	-	-	-	3496922	3496921	-

Universal nail X-CR P8

Applications and advantages

- Fastening in outdoor applications, directly or indirectly exposed to the weather
- Stainless steel
- Corrosion resistant



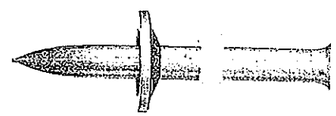
Order Designation	Fastener shank length	Fastener shank diameter	Sales pack quantity	Item nun
Stainless steel fastener X-CR 34 P8	1-5/16 in	0.145 in	100 pc	247
Stainless steel fastener X-CR 39 P8	1-9/16 in	0.157 in	100 pc	247
Stainless steel fastener X-CR 44P8	1-3/4 in	0.157 in	100 pc	247
Stainless steel fastener X-CR 29 P8	1-1/8 in	0.145 in	100 pc	247
Stainless steel fastener X-CR 54 P8	2-1/8 in	0.157 in	100 pc	247
Stainless steel fastener X-CR 16 P8	5/8 in	0.145 in	100 pc	247
Stainless steel fastener X-CR 14 P8	9/16 in	0.146 in	100 pc	306
Stainless steel fastener X-CR 18 P8	11/16 in	0.145 in	100 pc	247
Stainless steel fastener X-CR 21 P8	13/16 in	0.145 in	100 pc	247
Stainless steel fastener X-CR 24 P8	15/16 in	0.145 in	100 pc	247

✓ To use with green or yellow body

Corrosion-resistant fastener (with stainless steel washer) X-CR P8 S

Applications and advantages

- Fastening a variety of materials to concrete or steel where a high degree of corrosion resistance is required
- Fastening metal sheets or brackets to concrete
- High corrosion resistance
- High pull-over values

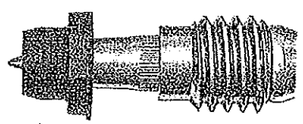


Order Designation	Fastener shank length	Fastener shank diameter	Sales pack quantity	Item nun
Stainless steel fastener X-CR 39 P8 S12	1-9/16 in	0.157 in	100 pc	247
Stainless steel fastener X-CR 44 P8 S12	1-3/4 in	0.157 in	100 pc	247
Stainless steel fastener X-CR 48 P8 S15	1-7/8 in	0.157 in	100 pc	258

Threaded stud (for steel, with 8 mm washer) X-CR M8

Applications and advantages

- Fastening grating and checker plate in corrosive environments like marine, offshore, petrochemical, power plants, etc.
- Fastening light duty electrical and mechanical elements
- Corrosion resistant fastener ideal for fastening directly to steel



Order Designation	Thread length	Fastener shank length	Sales pack quantity	Item nur
Threaded stud X-CR M8-9-12 P8	3/8 in	1/2 in	100 pc	372
Threaded stud X-CR M8-15-12 P8	9/16 in	1/2 in	100 pc	372

Threaded stud (for steel, with 10 mm washer) X-CR M8

Applications and advantages

- Fastening grating and checker plate in corrosive environments like marine, offshore, petrochemical, power plants, etc.
- Fastening light duty electrical and mechanical elements
- Corrosion resistant fastener ideal for fastening directly to steel

Order Designation	Thread length	Fastener shank length	Sales pack quantity	Item nur
Threaded stud X-CR M8-9-12 FP10	3/8 in	1/2 in	100 pc	372
Threaded stud X-CR M8-15-12 FP10	9/16 in	1/2 in	100 pc	372

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ESR-2379

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DIVISION: 03 00 00—CONCRETE
SECTION: 03 16 00—CONCRETE ANCHORS

REPORT HOLDER:

HILTI, INC.

7250 DALLAS PARKWAY, SUITE 1000
PLANO, TEXAS 75024

EVALUATION SUBJECT:

EXTERIOR OR PERIMETER SILL AND INTERIOR PLATE ANCHORAGES

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DIVISION: 03 00 00—CONCRETE
Section: 03 16 00—Concrete Anchors

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HNATechnicalServices@hilti.com

EVALUATION SUBJECT:

EXTERIOR OR PERIMETER SILL AND INTERIOR PLATE ANCHORAGES

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2015, 2012 and 2009 *International Building Code®* (IBC)
- 2015, 2012 and 2009 *International Residential Code®* (IRC)
- 2013 *Abu Dhabi International Building Code* (ADIBC)†

†The ADIBC is based on the 2009 IBC. 2009 IBC code sections referenced in this report are the same sections in the ADIBC.

Property evaluated:

Structural

2.0 USES

The Hilti exterior or perimeter sill and interior plate anchorages described in this report are used as alternatives to the cast-in-place anchors described in 2015 IBC Section 2308.3.1 (2012 and 2009 IBC Section 2308.6) and IRC Section R403.1.6 for the anchorage of wood sill plates to normalweight concrete foundations. The fasteners may be used under the IRC when an engineered design is submitted in accordance with IRC Section R301.1.3.

3.0 DESCRIPTION

3.1 Fasteners:

The Hilti exterior or perimeter sill and interior plate anchorages are powder-actuated fasteners (PAFs) with premounted steel washers. The X-CF 72 fasteners and premounted washers are manufactured from carbon steel complying with SAE J403 Grade 1060 or modified Grade 1070, with an electro-deposited zinc coating complying with ASTM B633 SC 1, Type III. The X-CP 72 fasteners and premounted washers are manufactured from carbon steel complying with SAE J403 Grade 1060 or modified Grade 1070, and are mechanically galvanized in accordance with ASTM B695 Type I, minimum Class 55. See Figures 1 and 2 for depictions of the fasteners.

3.2 Concrete:

Normalweight concrete must conform to IBC Chapter 19 or IRC Section R402.2, as applicable. The minimum concrete compressive strength at the time of fastener installation is noted in the tables of this report.

3.3 Sill Plates:

The sill plates must be nominally 2-inch-thick lumber that is naturally durable in accordance with 2015 and 2012 IBC Section 202 (2009 IBC Section 2302) and IRC Section R202; or preservative-treated in accordance with 2015 IBC Section 2303.1.9 (2012 and 2009 IBC Section 2303.1.8) or IRC Section R317.1, as applicable.

4.0 DESIGN AND INSTALLATION

4.1 Design:

The Hilti fasteners may be used to attach wood sill plates to concrete for structural walls in Seismic Design Categories A and B. Allowable shear and tension loads for the fasteners are provided in Table 1. Bearing area and thickness of the washers are also given in Table 1. For shear loads, spacing of fasteners must be determined based on the lesser of the allowable shear load from Table 1 and the allowable load based on the fastener/wood sill plate/concrete foundation interaction, determined in accordance with the ANSI/AWC National Design Specification (NDS) for Wood Construction, with a fastener bending yield strength, F_{yb} = 90,000 psi (621 MPa) and a concrete dowel bearing strength, F_e = 7,500 psi (52 MPa). For tension loads, spacing of fasteners must be determined based on the lesser of the allowable tension load from Table 1 and the pull-through capacity of the fastener with respect to the wood sill plate, determined in accordance with Section 3.10 of the NDS, using the washer bearing area from Table 1.

For fasteners subjected to both tension and shear loads, compliance with the following interaction equation must be verified:

$$(p/P_a) + (v/V_a) \leq 1.0$$

where:

p = Actual applied tension load on fastener, lbf (N).

P_a = Allowable tension load on fastener, lbf (N).

v = Actual applied shear load on fastener, lbf (N).

V_a = Allowable shear load on fastener, lbf (N).

Hilti fasteners listed in Table 2 may be used to attach wood sill plates to concrete for interior, nonstructural walls [maximum horizontal transverse load on the walls must not exceed 5 psf (0.24 kN/m^2)] in Seismic Design Categories A through F, when installed as described in Table 2.

4.2 Installation:

The fasteners must be installed in accordance with this report and the Hilti, Inc., published installation instructions. A copy of the instructions must be available on the jobsite at all times during installation.

Installation of the X-CF 72 and X-CP 72 fasteners is limited to dry, interior locations. The X-CP 72 fasteners may be installed in preservative-treated lumber complying with 2015 IBC Section 2303.1.9 (2012 and 2009 IBC Section 2303.1.8) or IRC Section R317, as applicable. The X-CF 72 fasteners may be used to attach naturally durable wood to concrete or to attach fire-retardant-treated wood to concrete in accordance with 2015 IBC Section 2304.10.5.4 (2012 and 2009 IBC Section 2304.9.5.4) and Hilti's recommendations. Fastener placement requires the use of a low-velocity, powder-actuated tool in accordance with the manufacturer's recommendations.

The concrete must attain a minimum compressive strength of 2,000 psi (13.8 MPa) prior to installation of the fasteners. The fasteners must be installed through the sill plate. Minimum concrete edge distance is $1^3/_4$ inches (44 mm). Concrete thickness must be a minimum of $4^1/_2$ inches (114 mm).

5.0 CONDITIONS OF USE

The exterior or perimeter sill and interior plate anchorages described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 The fasteners are manufactured and identified in accordance with this report.

5.2 Fastener installation complies with this report and the Hilti, Inc., instructions. In the event of a conflict between this report and the Hilti, Inc., published instructions, this report governs.

5.3 Calculations demonstrating that the applied loads are less than the allowable loads described in Section 4.1 must be submitted to the code official for approval. These calculations must be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is constructed. **Exception:** Fasteners used in nonstructural walls in accordance with Table 2.

5.4 The fasteners may be used to attach wood sill plates to concrete for structural walls in Seismic Design Categories A and B. The fasteners may be used to attach wood sill plates to concrete for interior, nonstructural walls in Seismic Design Categories A through F.

5.5 The use of fasteners is limited to installation in uncracked concrete. Cracking occurs when $f_t > f_r$ due to service loads or deformations.

5.6 The minimum concrete thickness must be $4^1/_2$ inches (114 mm).

5.7 Installation is limited to dry, interior locations, which include exterior walls which are protected by an exterior wall envelope.

5.8 Installation must comply with Section 4.2 regarding fasteners in contact with preservative-treated and fire-retardant-treated wood.

5.9 The fasteners must be installed by personnel certified by Hilti, Inc., and having a current, Hilti-issued operator's license.

5.10 The Hilti products addressed in this report are manufactured under a quality control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Power-Actuated Fasteners Driven into Concrete, Steel and Masonry Elements (AC70), dated February 2016.

7.0 IDENTIFICATION

Each package of fasteners is identified with the manufacturer's name (Hilti), the fastener type and size, and the evaluation report number (ESR-2379). Additionally, an "H" is imprinted on the fastener heads, as shown in Figures 1 and 2. Furthermore, each fastener type has a unique alphanumeric code as well as the name "HILTI" stamped on the premounted washer. Refer to Figures 3 and 4 for depictions of the premounted washers.

TABLE 1—ALLOWABLE LOADS FOR FASTENERS DRIVEN INTO MINIMUM 2,000 psi NORMALWEIGHT CONCRETE[1,2]

FASTENER TYPE	SHANK LENGTH (inches)	SHANK DIAMETER (inch)	WASHER THICKNESS (inch)	WASHER BEARING AREA (in^2)	EMBEDMENT	CONCRETE EDGE DISTANCE (inches)	TENSION (lbf)	SHEAR (lbf)
X-CF 72	$2^7/_8$	0.145	0.059	0.543	Washer bearing on sill plate	≥ 3	130	210
						$1^3/_4$	130	165
X-CP 72	$2^7/_8$	0.145	0.059	0.527	Washer bearing on sill plate	≥ 3	175	250
						$1^3/_4$	150	105

For SI: 1 inch = 25.4 mm, 1 foot = 305 mm, 1 lbf = 4.4 N.

[1]Wood members connected to the substrate must be investigated for compliance with the applicable code in accordance with referenced design criteria, for both lateral resistance and fastener pull-through.
[2]The concrete base material must have a minimum compressive strength of 2,000 psi (13.8 MPa) at the time of fastener installation. Concrete must have a minimum compressive strength at 28 days (f'_c) of 2,500 psi (17.2 MPa) [minimum of 24 MPa is required under ADIBC Appendix L, Section 5.1.1].

TABLE 2—FASTENER SPACING FOR WOOD SILL PLATE ANCHORAGE OF INTERIOR NONSTRUCTURAL WALLS[1,2,3,4,5,6,7]

FASTENER TYPE	SHANK LENGTH (inches)	SHANK DIAMETER (inch)	EMBEDMENT	CONCRETE EDGE DISTANCE (inches)	MAXIMUM FASTENER SPACING (ft.)	MAXIMUM WALL HEIGHT (ft.)
X-CF 72	$2^7/_8$	0.145	Washer bearing on sill plate	≥ 3	3	14
				$1^3/_4$	3	14
X-CP 72	$2^7/_8$	0.145		≥ 3	3	14
				$1^3/_4$	2	14

For SI: 1 inch = 25.4 mm, 1 foot = 305 mm.

[1]Spacings noted above apply to normalweight concrete having a minimum compressive strength of 2,000 psi (13.8 MPa) at the time of fastener installation. Concrete must have a minimum compressive strength at 28 days (f'_c) of 2,500 psi (17.2 MPa) [minimum of 24 MPa is required under ADIBC Appendix L, Section 5.1.1].
[2]Interior nonstructural walls are limited to locations where bearing walls, shear walls or braced walls are not required by the approved plans. Maximum horizontal transverse load on the walls must not exceed 5 psf (0.24 kN/m^2).
[3]Fasteners must be driven into the center of the sill plate with a minimum concrete edge distance as shown in the table.
[4]Walls must have fasteners placed at 6 inches from ends of sill plates with maximum spacing between, as shown in this table.
[5]Walls must be laterally supported at the top and the bottom.
[6]Sill or bottom plates must comply with IBC Section 2304.1 and be of lumber with a specific gravity of 0.50 or greater.
[7]Minimum fastener spacing must be 4 inches on center or shall comply with Section 12.1.6 of the 2015 NDS (Section 11.1.6 of NDS-12 for the 2012 IBC, Section 11.1.5 of NDS-05 for the 2009 IBC) to prevent splitting of the wood.

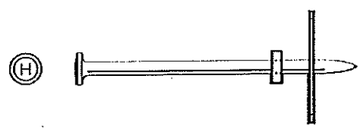

FIGURE 1—HILTI X-CF SILL PLATE FASTENER

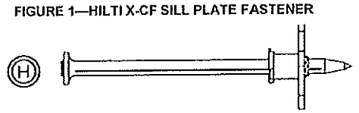

FIGURE 2—HILTI X-CP SILL PLATE FASTENER

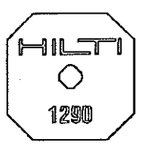

FIGURE 3—HILTI X-CF WASHER

FIGURE 4—HILTI X-CP WASHER

ICC-ES Evaluation Report

ESR-2379 FBC Supplement

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DIVISION: 03 00 00—CONCRETE
Section: 03 16 00—Concrete Anchors

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EVALUATION SUBJECT:

EXTERIOR OR PERIMETER SILL AND INTERIOR PLATE ANCHORAGES

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Hilti Exterior or Perimeter Sill and Interior Plate Anchorages, recognized in ICC-ES master report ESR-2379, have also been evaluated for compliance with the codes noted below.

Applicable code editions:

- 2014 *Florida Building Code—Building*
- 2014 *Florida Building Code—Residential*

2.0 CONCLUSIONS

The Exterior or Perimeter Sill and Interior Plate Anchorages described in Sections 2.0 through 7.0 of the master report ESR-2379, comply with the *Florida Building Code—Building* and the *Florida Building Code—Residential*, provided the design and installation are in accordance with the 2012 *International Building Code®* provisions noted in the master evaluation report, and the following additional conditions apply:

- Design wind loads must be based on Section 1609 of the *Florida Building Code—Building* or Section R301.2.1 of the *Florida Building Code—Residential*, as applicable.
- Load combinations must be in accordance with Section 1605.2 or Section 1605.3 of the *Florida Building Code—Building*, as applicable.

Use of the Hilti fasteners has also been found to be in compliance with the High-Velocity Hurricane Zone provisions of the *Florida Building Code—Building* and the *Florida Building Code—Residential* under the following conditions:

- Design wind loads must be based on Section 1620 of the *Florida Building Code—Building*, as applicable.
- The fasteners have not been evaluated for use as cast-in-place anchors for compliance with the High-velocity Hurricane Zone provisions and this use is outside the scope of this evaluation report.

For products falling under Florida Rule 9N-3, verification that the report holder's quality-assurance program is audited by a quality-assurance entity approved by the Florida Building Commission for the type of inspections being conducted is the responsibility of an approved validation entity (or the code official when the report holder does not possess an approval by the Commission).

This supplement expires concurrently with the master report, reissued August 2016.

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SECTION: 05 05 23—METAL FASTENINGS
DIVISION: 06 00 00—WOOD, PLASTICS AND COMPOSITES
SECTION: 06 05 23—WOOD, PLASTIC AND COMPOSITE FASTENINGS
DIVISION: 09 00 00—FINISHES
SECTION: 09 22 16.23—FASTENERS

REPORT HOLDER:

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EVALUATION SUBJECT:

HILTI LOW-VELOCITY POWER-ACTUATED FASTENERS

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Section: 04 05 19.16—Masonry Anchors

DIVISION: 05 00 00—METALS
Section: 05 05 23—Metal Fastenings

DIVISION: 06 00 00—WOOD, PLASTICS AND COMPOSITES
Section: 06 05 23—Wood, Plastic and Composite Fastenings

DIVISION: 09 00 00—FINISHES
Section: 09 22 16.23—Fasteners

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EVALUATION SUBJECT:

HILTI LOW-VELOCITY POWER-ACTUATED FASTENERS

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2015, 2012, 2009 and 2006 *International Building Code*® (IBC)
- 2015, 2012, 2009 and 2006 *International Residential Code*® (IRC)
- 2013 *Abu Dhabi International Building Code* (ADIBC)†

†The ADIBC is based on the 2009 IBC. 2009 IBC code sections referenced in this report are the same sections in the ADIBC.

Property evaluated:

Structural

2.0 USES

Hilti low-velocity power-actuated fasteners (PAFs) are used to attach wood, cold-formed steel, and other building elements to base materials of normalweight and sand-lightweight concrete, steel deck panels filled with sand-lightweight concrete, concrete masonry and steel base materials. The fasteners are alternatives to the cast-in-place anchors described in 2015 IBC Section 1901.3 (2012 IBC Section 1908; 2009 and 2006 IBC Section 1911) for placement in concrete; the embedded anchors described in Section 8.1.3 of TMS 402-13, referenced in Section 2107 of the 2015 IBC (Section 2.1.4 of TMS 402-11, -08 and -05, referenced in Section 2107 of the 2012, 2009 and 2006 IBC, respectively) for placement in masonry; and the welds and bolts used to attach materials to steel, described in IBC Sections 2204.1 and 2204.2, respectively. For structures regulated under the IRC, the fasteners may be used where an engineered design is submitted in accordance with IRC Section R301.1.3.

3.0 DESCRIPTION

3.1 Fasteners:

The Hilti low-velocity power-actuated fasteners are manufactured from hardened steel complying with the material specifications in the manufacturer's quality documentation. See Table 1 for fastener descriptions, including shank type and diameter, head diameter, coating and applicable allowable load tables. Maximum point length is the maximum specified length from the tip of the fastener to the location where the diameter of the shank becomes constant. Minimum effective shank length is the minimum specified length from the underside of the fastener head to the tip of the fastener, except for fasteners with premounted washers, where the minimum effective shank length is the minimum specified length from the underside of the washer, in its installed condition, to the tip of the fastener.

3.2 Substrate Materials:

3.2.1 Concrete: Normalweight and sand-lightweight concrete must comply with IBC Chapter 19 or IRC Section R402.2, as applicable. The minimum concrete compressive strength at the time of fastener installation must be as noted in Table 3.

3.2.2 Concrete Masonry: Concrete masonry units (CMUs) must be minimum 8-inch-thick (203 mm), normalweight blocks complying with ASTM C90. Mortar must comply with ASTM C270, Type N. Grout must be coarse grout complying with ASTM C476. Concrete masonry walls must have a minimum compressive strength, f'_m, of 1,500 psi (10.3 MPa).

3.2.3 Steel: Structural steel used in supports must comply with the minimum requirements of ASTM A36,

ASTM A572 Grade 50 or ASTM A992, as applicable, and must have the minimum yield and tensile strengths and thickness shown in Table 2.

3.2.4 Steel Deck Panels: Steel deck panel properties and configurations must be as described in Tables 4 and 5 and Figures 1 through 3.

4.0 DESIGN AND INSTALLATION

4.1 Design:

4.1.1 General: Selection of fasteners must take into consideration the applicable base material and the length of the fastener. The minimum fastener length must be determined as follows:

- For installation into concrete, concrete-filled steel deck panels and concrete masonry materials and into steel base materials where there is no point penetration, the minimum effective shank length shown in Table 1 must equal or exceed the sum of the thickness of the attached material and the minimum embedment depth (penetration) shown in the applicable tables in this report.
- For installation through steel base material, the minimum effective shank length shown in Table 1 must equal or exceed the sum of the following: the thickness of the attached material, the thickness of the base material and the required point penetration shown in the applicable tables in this report.

4.1.2 Allowable Loads: The applicable allowable load tables for Hilti power-actuated fasteners driven into different base materials may be determined by referencing Table 1.

The most critical applied loads, excluding seismic load effects, resulting from the load combinations in IBC Section 1605.3.1 or 1605.3.2 must not exceed the allowable loads. For fasteners which are subjected to seismic loads, see Section 4.1.5 for additional information. The stress increases and load reductions described in IBC Section 1605.3 are not allowed.

Allowable shear loads and tension (pullout) loads in this report apply to the connection of the fastener to the base material only. Other limit states applicable to the design of a connection, such as fastener pull-through (pull-over) and lateral bearing on the attached material, which are governed by the properties of the attached material, are outside the scope of this report. Design of the connection to the attached material must comply with the applicable requirements of the IBC. When designing the connection of wood members to the base material, the bending yield strength of the PAFs can be assumed to be the same as that of a nail with the same shank diameter.

4.1.3 Combined Loading: For fasteners subjected to both tension and shear loads, compliance with the following interaction equation must be verified:

$$(p/P_a) + (v/V_a) \leq 1.0$$

where:

p = Actual tension load on the fastener, lbf (N).

P_a = Allowable tension load for the fastener, lbf (N).

v = Actual shear load on the fastener, lbf (N).

V_a = Allowable shear load for the fastener, lbf (N).

4.1.4 Steel-to-steel Connections: When the Hilti fasteners listed in Table 2 are used in connections of two steel elements in accordance with Section E5 of AISI S100-12, connection capacity must be determined in accordance with Sections 4.1.4.1 and 4.1.4.2, as applicable.

4.1.4.1 Connection Strength - Tension: To determine tensile connection strength in accordance with Section E5.2 of AISI S100-12, fastener tension strength, the pull-out strength and the pull-over strength must be known. These characteristics must be determined as follows:

- **Pull-out Strength:** See Table 2 for available pull-out strength.
- **Pull-over Strength:** The available pull-over strengths must be calculated in accordance with Section E5.2.3 of AISI S100-12.
- **PAF Tensile Strength:** The allowable fastener tension strengths, determined in accordance with Section E5.2.1 of AISI S100-12, exceed the corresponding allowable pull-out strengths in Table 2.

4.1.4.2 Connection Strength - Shear: To determine shear connection strength in accordance with Section E5.3 of AISI S100-12, the fastener shear strength, bearing and tilting strength, pull-out strength in shear, net section rupture strength and shear strength limited by edge distance must be known. These characteristics must be determined as follows:

- **Bearing and Tilting Strength:** The available bearing and tilting strengths must be calculated in accordance with Section E5.3.2 of AISI S100-12.
- **Pull-out Strength in Shear:** The available pull-out strength in shear must be the applicable allowable shear strength from Table 2, or must be calculated in accordance with Section E5.3.3 of AISI S100-12.
- **Net Section Rupture Strength and Shear Strength Limited by Edge Distance:** The net section rupture strength must be determined in accordance with Section E5.3.4 of AISI S100-12 and the shear strength limited by edge distance must be determined in accordance with Section E5.3.5 of AISI S100-12.
- **PAF Shear Strength:** The allowable fastener shear strengths, determined in accordance with Section E5.3.1 of AISI S100-12, exceed the corresponding allowable shear strengths in Table 2.

4.1.5 Seismic Considerations: The Hilti fasteners are recognized for use when subjected to seismic loads as follows:

1. The Hilti fasteners may be used for attachment of nonstructural components listed in Section 13.1.4 of ASCE 7, which are exempt from the requirements of ASCE 7.
2. Concrete base materials: The Hilti fasteners installed in concrete may be used to support acoustical tile or lay-in panel suspended ceiling systems, distributed systems and distribution systems where the service load on any individual fastener does not exceed the lesser of 90 lbf (400 N) or the published allowable load in Tables 3, 4 and 5, as applicable.
3. Steel base materials: When the Hilti fasteners listed in Table 2 (except for the X-R) are installed in steel and subjected to seismic loads, the most critical load applied to each individual fastener must be determined from the applicable equations in IBC Section 1605.3.1 or Section 1605.3.2, and must not exceed the allowable load shown in Table 2. The X-R fastener may be used where the service load on any individual fastener does not exceed the lesser of 250 lbf (1112 N) or the published allowable load

shown in Table 2. Recognition of the Hilti fasteners installed in steel base material for use in the design of lateral force resisting systems, such as shear walls and diaphragms, is outside the scope of this report.

4. For interior, nonstructural walls that are not subject to sustained tension loads and are not a bracing application, the power-actuated fasteners may be used to attach steel track to concrete or steel in all Seismic Design Categories. In Seismic Design Categories D, E, and F, the allowable shear load due to transverse pressure shall be no more than 90 pounds (400 N) when attaching to concrete; or the allowable load shown in Table 2 when attaching to steel. Substantiating calculations shall be submitted addressing the fastener-to-base-material capacity and the fastener-to-attached-material capacity. Interior nonstructural walls are limited to locations where bearing walls, shear walls or braced walls are not required by the approved plans. The design load on the fastener must not exceed the allowable load established in this report for the concrete or steel base material.

4.2 Installation:

4.2.1 General: The fasteners must be installed in accordance with this report and the Hilti, Inc., published installation instructions. A copy of the instructions must be available on the jobsite at all times during installation. Additional installation requirements are set forth in the tables in this report.

Fastener placement requires a low-velocity powder-actuated tool used in accordance with Hilti, Inc. recommendations.

Installers must be certified by Hilti, Inc., and have a current, Hilti-issued, operator's license.

4.2.2 Fastening to Steel: When installation is in steel, minimum spacing between fasteners must be 1 inch (25.4 mm) on center, and minimum edge distance must be $^1/_2$ inch (12.7 mm).

4.2.3 Fastening to Concrete: Fasteners are to be driven into the concrete after the concrete attains the concrete strength specified in the tables of this report. Unless otherwise noted, minimum spacing between fasteners must be 4 inches (102 mm) on center and minimum edge distance must be 3 inches (76 mm). Unless otherwise noted, concrete thickness must be a minimum of three times the embedment depth of the fastener.

4.2.4 Fastening to Masonry: Fasteners are to be driven into the masonry after the mortar and grout materials have attained the specified strength. For CMUs, no more than one power-actuated fastener may be installed into each individual CMU cell.

4.2.5 Fastening to Sand-lightweight Concrete-filled Steel Deck Panels: Installation in sand-lightweight concrete-filled steel deck panels must comply with Tables 4 and 5 and Figures 1 through 3. Minimum distances from fastener centerline to rolled deck panel flute edges must be as depicted in Figures 1 through 3.

4.2.6 Use with Treated Lumber: The Hilti carbon steel fasteners described in Table 1 may be used in contact with fire-retardant-treated wood in dry, interior locations only, in accordance with 2015 IBC Section 2304.10.5.4 (2012 and 2009 IBC Section 2304.9.5.4) and Hilti's recommendations. Use of fasteners in contact with preservative-treated wood or fire-retardant-treated wood in exterior applications is outside the scope of this report

5.0 CONDITIONS OF USE

The Hilti Low-Velocity Power-Actuated Fasteners described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Fasteners must be manufactured and identified in accordance with this report.

5.2 Fasteners must be installed in accordance with this report and the Hilti, Inc., instructions. In the event of conflict between this report and Hilti, Inc., published instructions, the more restrictive requirements govern.

5.3 Calculations demonstrating that the actual loads are less than the allowable loads described in Section 4.1 must be submitted to the code official for approval. The calculations must be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is constructed.

5.4 For steel-to-steel connections that meet the applicability requirements of Section E5 of AISI S100-12, calculations demonstrating that the available connection strength has been determined in accordance with Section E5 of AISI S100-12 and Section 4.1.4 of this report, and equals to or exceeds the applied load, must be submitted to the code official. The calculations must be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed.

5.5 Refer to Section 4.1.5 for seismic considerations.

5.6 The use of the fasteners is limited to installations in uncracked concrete or masonry. Cracking occurs when $f_t > f_r$ due to service loads or deformations.

5.7 Hilti X-CR and X-R stainless steel fasteners may be used in exterior, damp environments. All other fasteners in this report must be limited to installation in dry, interior environments, which include exterior walls which are protected by an exterior wall envelope.

5.8 Installation must comply with Section 4.2.6 regarding fasteners in contact with preservative-treated and fire-retardant-treated wood.

5.9 Installers must be certified by Hilti, Inc., and have a current, Hilti-issued, operator's license.

5.10 The Hilti products addressed in this report are manufactured under a quality-control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Power-actuated Fasteners Driven into Concrete, Steel, and Masonry Elements (AC70), dated February 2016, including seismic load test data in accordance with Annex A of AC70.

7.0 IDENTIFICATION

All carbon steel fasteners are identified by an "H" imprinted on the fastener head. The stainless steel fasteners are identified by "HI" imprinted on the fastener head. Where applicable, the word "Hilti" is stamped on the steel washers. All fasteners are packaged in containers noting the fastener type, size, manufacturer's name, and evaluation report number (ESR-1663).

TABLE 1—FASTENER DESCRIPTION AND APPLICATIONS[1]

FASTENER	FASTENER DESCRIPTION	SHANK TYPE	SHANK DIAMETER (inch)	HEAD DIAMETER (inch)	MAXIMUM POINT LENGTH (inch)	MINIMUM EFFECTIVE SHANK LENGTH (inch)	FASTENER MATERIAL/ COATING	APPLICABLE BASE MATERIAL	APPLICABLE LOAD TABLES
##	Powder-actuated Heavy Duty Fastener	Knurled, straight	0.177	0.390	0.43	See Footnote 2	Carbon steel galvanized per ASTM B633, SC1, Type III	Steel	2
DS ##	Powder-actuated Heavy Duty Fastener	Smooth, straight	0.177	0.390	0.43	See Footnote 2		Steel	2
								Concrete Conc.-filled deck	3, 4
X-C ##	Powder-actuated Standard Fastener	Knurled, straight	0.138	0.321	0.30	See Footnote 2		Concrete Conc.-filled deck	3, 4, 5
								CMU	6
X-C22P8TH	Powder-actuated Standard Fastener	Knurled, straight	0.138	0.321	0.30	0.807		Concrete Conc.-filled deck	3, 4, 5
X-C20THP	Powder-actuated Standard Fastener	Knurled, straight	0.138	0.321	0.30	0.728		Concrete Conc.-filled deck	4
X-W6	Powder-actuated $^1/_4$-20 Threaded Stud	Smooth, straight	0.145	n/a	n/a	n/a		Concrete Conc.-filled deck	3, 4
								CMU	6
W10	Powder-actuated $^3/_8$-16 Threaded Stud	Smooth, straight	0.205	n/a	n/a	n/a		Concrete Conc.-filled deck	3, 4
X-CR ##	Powder-actuated Stainless Steel Fastener	Smooth, straight	0.145	0.321	0.43	See Footnote 2	Stainless steel	Steel	2
								Concrete Conc.-filled deck	3, 4
X-R	Powder-actuated Stainless Steel Fastener	Smooth, straight	0.145	0.321	0.35	0.531	Stainless steel	Steel	2

For SI: 1 inch = 25.4 mm.

[1]## denotes numbers used in fastener designation to represent nominal fastener length in mm.
[2]When multiple lengths of a fastener are addressed, the minimum effective shank length can be calculated in terms of the designated length as (##-1) in mm and (##-1)/25.4 in inches.

TABLE 2—ALLOWABLE LOADS FOR LOW-VELOCITY FASTENERS DRIVEN INTO STEEL[1,2,4]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	ALLOWABLE LOADS (lbf)											
Steel Thickness (inch):			$^1/_8$		$^3/_{16}$		$^1/_4$		$^3/_8$		$^1/_2$		$^3/_4$	
Load Direction:			Tension	Shear	Tension	Shear	Tension	Shear	Tension	Shear	Tension	Shear	Tension	Shear
Heavy Duty Knurled Shank	EDS	0.177	–	–	305[8]	615	625[8]	870	715[8]	870	890[8]	960	–	–
Heavy Duty Smooth Shank	DS	0.177	–	–	365[8]	725	580[9]	725	695[8]	725	735	860	–	–
Stainless Steel Smooth Shank	X-CR X-R	0.145	–	–	460[8]	460	615[8]	500	–	–	–	–	–	–
Stainless Steel Smooth Shank	X-CR[3] X-R[3,9]	0.145	300[5]	190	615[8]	495	760[8]	500	220[5]	325[6]	225[7]	335[7]	–	–

For SI: 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 ksi = 6.89 MPa.

[1]Fasteners must be driven to where the point of the fastener penetrates through the steel base material.
[2]Unless otherwise noted, allowable load capacities are based on base steel with a minimum yield strength (F_y) of 36 ksi and a minimum tensile strength (F_u) of 58 ksi.
[3]Allowable load capacity based on base steel with minimum yield strength (F_y) of 50 ksi and minimum tensile strength (F_u) of 65 ksi.
[4]Unless otherwise noted, allowable loads are applicable to static and seismic loads in accordance with Section 4.1.
[5]The fastener must penetrate through the steel, but full fastener point penetration through the steel is not necessary.
[6]Fastener point penetration through the steel is not necessary, provided a minimum embedment depth of 0.354 is achieved.
[7]Fastener point penetration through the steel is not necessary, provided a minimum embedment depth of 0.470 is achieved.
[8]For steel-to-steel connections designed in accordance with Section 4.1.4, the tabulated allowable load may be increased by a factor of 1.25, and the design strength may be taken as the tabulated allowable load multiplied by a factor of 2.0.
[9]Tabulated loads for the X-R fastener apply to static load conditions only. For seismic loading, allowable loads must be limited in accordance with Section 4.1.5, Item 3.

TABLE 3—ALLOWABLE LOADS FOR LOW-VELOCITY FASTENERS DRIVEN INTO NORMAL-WEIGHT CONCRETE[1,2]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT DEPTH (inches)	ALLOWABLE LOADS (lbf)					
Concrete Compressive Strength:				2,000 psi		4,000 psi		6,000 psi	
Load Direction:				Tension (lbf)	Shear (lbf)	Tension (lbf)	Shear (lbf)	Tension (lbf)	Shear (lbf)
Standard Nail	X-C (Black Collated Strip or Guidance Washer)	0.138	3/4	45	75	65	105	95	195
			1	85	150	160	200	105	270
			1 1/4	130	210	270	290	165	325
			1 1/2	175	260	270	360	—	—
Standard Nail	X-C (White Collated Strip or Guidance Washer)	0.138	3/4	45	75	60	105	—	—
			1	85	150	90	200	—	—
			1 1/4	130	210	130	290	—	—
			1 1/2	175	260	245	360	—	—
Drywall Track Nail	X-C22 P8 TH (Black Collated Strip or Guidance Washer)	0.138	3/4	55	130	90	170	100	200
Drywall Track Nail	X-C22 P8 TH (White Collated Strip or Guidance Washer)	0.138	3/4	55	130	90	170	—	—
Heavy Duty Nail	DS	0.177	3/4	50	120	125	135	—	—
			1	130	195	155	240	—	—
			1 1/4	220	385	270	425	—	—
			1 1/2	300	405	355	450	—	—
1/4-20 Threaded Stud	X-W6	0.145	3/4	40	55	40	55	—	—
			1	85	195	110	225	—	—
3/8-16 Threaded Stud	W10	0.205	1	85	95	100	105	—	—
			1 1/4	175	345	200	380	—	—
			1 5/8	285	380	385	395	—	—
Stainless Steel Nail	X-CR	0.145	3/4	30	40	65	40	—	—
			1	55	185	120	190	100	170
			1 1/4	110	290	125	300	120	440
			1 1/2	265	405	350	450	—	—

For SI: 1 inch = 25.4 mm, 1 psi = 6895 Pa, 1 lbf = 4.4 N.

[1]Fasteners must not be driven until the concrete has reached the designated minimum compressive strength.
[2]The fasteners listed in the table above may be used for static load conditions and for the seismic load conditions described in Section 4.1.5, as applicable. The tabulated allowable loads apply to static load conditions. For seismic load conditions, the allowable loads must be limited in accordance with Section 4.1.5, Items 2 and 4, as applicable.

TABLE 4—ALLOWABLE LOADS FOR LOW-VELOCITY FASTENERS DRIVEN INTO MINIMUM f'_c=3,000 psi SAND-LIGHTWEIGHT CONCRETE[1,4]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT DEPTH (inches)	ALLOWABLE LOADS (lbf)				
Fastener Location:				Installed into Concrete		Installed Through Steel Deck Panel into Concrete[2,3]		
						Upper Flute	Lower Flute	
Load Direction:				Tension	Shear	Tension	Tension	Shear
Standard Nail	X-C (Black Collated Strip or Guidance Washer)	0.138	3/4	120	175	120	95	265
			1	180	260	215	155	485
			1 1/4	225	400	250	200	500
			1 1/2	285	400	285	210	555
Standard Nail	X-C (White Collated Strip or Guidance Washer)	0.138	3/4	110	175	120	-	265
			1	135	180	215	145	485
			1 1/4	220	260	250	200	500
			1 1/2	285	315	285	210	555
Drywall Track Nail	X-C20 THP	0.138	5/8	55	110	-	45	285
	X-C22 P8TH (Black Collated Strip or Guidance Washer)	0.138	3/4	120	220	120	95	260
	X-C22 P8TH (White Collated Strip or Guidance Washer)	0.138	3/4	110	220	120	60	260
Heavy Duty Nail	DS[5]	0.177	3/4	100	200	-	-	200
			1	180	360	-	180	405
			1 1/4	300	520	-	-	515
			1 1/2	450	680	-	325	625
Stainless Steel Nail	X-CR	0.145	1	230	240	-	-	240
			1 1/4	320	400	-	-	400
			1 1/2	405	500	-	-	500
1/4-20 Threaded Stud	X-W6	0.145	3/4	125	185	125	115	185
			1	175	185	160	180	185
3/8-16 Threaded Stud	W10	0.205	1	265	185	-	-	185
			1 1/4	280	380	160	210	685
			1 5/8	445	540	435	325	945

For SI: 1 inch = 25.4 mm, 1 psi = 6895 Pa, 1 lbf = 4.4 N.

[1]Fasteners must not be driven until the concrete has reached the designated minimum compressive strength.
[2]The steel deck panel profile must be 3-inch-deep composite floor deck panel, with a minimum 0.0329-inch base-metal thickness, a minimum yield strength of 33 ksi and a minimum tensile strength of 45 ksi. Lower and upper flute widths must be a minimum of 3 7/8 inches. Figure 1 shows the nominal flute dimensions, fastener locations and load orientations for the deck panel profile.
[3]Sand-lightweight concrete fill depth above top of steel deck panel must be a minimum of 3 1/4 inches.
[4]The fasteners listed in the table above may be used for static load conditions and for the seismic load conditions described in Section 4.1.5, as applicable. The tabulated allowable loads apply to static load conditions. For seismic load conditions, the allowable loads must be limited in accordance with Section 4.1.5, Items 2 and 4, as applicable.
[5]DS fasteners installed at 1 1/2-inch embedment through steel deck panel into the lower flute must be installed at a minimum distance of 6 inches from the edge of the floor deck panel.

TABLE 5—ALLOWABLE LOADS FOR LOW-VELOCITY FASTENERS DRIVEN INTO MINIMUM f'_c = 3,000 psi SAND-LIGHTWEIGHT CONCRETE-FILLED 1 1/2-INCH-DEEP, B-DECK STEEL PANEL[1,5]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT DEPTH (inch)	ALLOWABLE LOADS (lbf)		
				Installed Through Steel Deck Panel Into Concrete[2,3]		
				Upper Flute	Lower Flute	
				Tension	Tension	Shear
Drywall track nail	X-C22 P8 TH[4]	0.138	3/4	90	110	295
Standard nail	X-C[4]	0.138	3/4	80	80	315
			1	205	205	445

For SI: 1 inch = 25.4 mm, 1 psi = 6895 Pa, 1 lbf = 4.4 N.

[1]Fasteners must not be driven until the concrete has reached the designated minimum compressive strength.
[2]Steel deck panel profiles are 1 1/2-inch-deep, B-type deck panel with a minimum base-metal thickness of 0.0329 inch, and a minimum yield strength of 38 ksi and a minimum tensile strength of 45 ksi. Fasteners may be installed through steel deck panels having either normal or inverted orientations with minimum lower flute widths of 1 3/4 and 3 1/2 inches, respectively. Fasteners must be placed at centerline of deck panel flutes. Figures 2 and 3 describe additional flute dimensions, fastener locations, and load orientations for both deck panel profiles.
[3]Sand-lightweight concrete fill above top of steel deck panel must be a minimum of 2 1/2 inches.
[4]Allowable load values apply to fasteners with black or white collated strip or guidance washer.
[5]The fasteners listed in the table above may be used for static load conditions and for the seismic load conditions described in Section 4.1.5, as applicable. The tabulated allowable loads apply to static load conditions. For seismic load conditions, the allowable loads must be limited in accordance with Section 4.1.5, Items 2 and 4, as applicable.

TABLE 6—ALLOWABLE LOADS FOR LOW-VELOCITY FASTENERS DRIVEN INTO CONCRETE MASONRY UNITS[1,2,3,10]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	EMBED-MENT DEPTH (inch)	ALLOWABLE LOADS (lbf)									
Masonry Type:				Hollow CMU				Grouted CMU					
Fastener Location:				Face Shell[4]		Mortar Joint[5]		Face Shell[4]		Mortar Joint[5]		Top of Grouted Cell[8]	
Load Direction:				Tension	Shear[6]	Tension	Shear[6]	Tension	Shear[6]	Tension	Shear[7]	Tension	Shear[6]
Standard Nail	X-C[9]	0.138	1	40	85	15	50	85	85	45	85	115	175
1/4-20 Threaded Stud	X-W6	0.145	1	105	175	80	110	125	175	135	150	—	—

For SI: 1 inch = 25.4 mm, 1 lbf = 4.4 N.

[1]Fasteners must be installed a minimum of 4 inches from the top, bottom and end of the wall.
[2]See Section 3.2.2 for CMU, mortar and grout requirements.
[3]No more than one low-velocity fastener may be installed in an individual CMU cell.
[4]Fastener must be located a minimum of 1 1/2 inches from the mortar joints, center web and end web of the CMU.
[5]Fasteners must not be installed in the head joints. Fasteners installed in the bed joints must be installed a minimum of 8 inches from the end of the wall. Multiple fasteners in a bed joint must be spaced a minimum of 8 inches.
[6]Shear load can be in any direction.
[7]Shear direction must be horizontal along the CMU wall plane.
[8]Fastener located in center of grouted cell must be installed vertically.
[9]Allowable load values apply to fasteners with black or white collated strip or guidance washer.
[10]The fasteners listed in the table above may be used for static load conditions and for the seismic load condition described in Item 1 of Section 4.1.5.

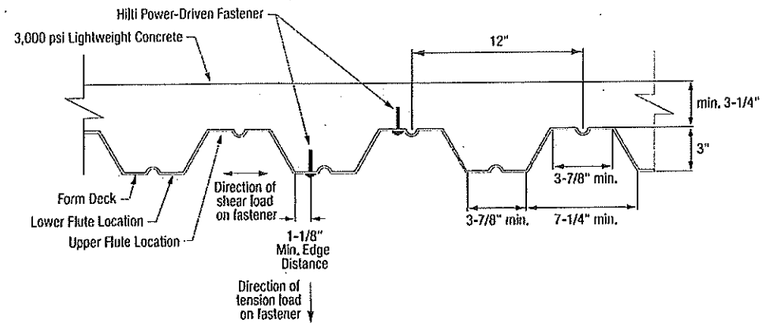


FIGURE 1—HILTI FASTENER LOCATIONS IN 3-INCH-DEEP COMPOSITE FLOOR DECK PANEL

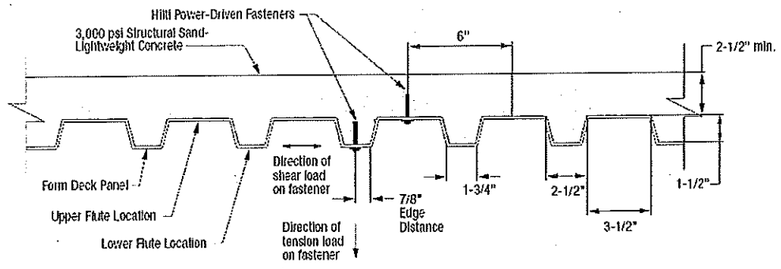
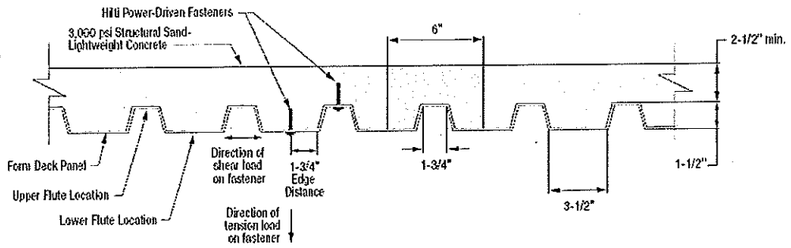


FIGURE 2—HILTI FASTENER LOCATIONS IN $1^1/_2$-INCH-DEEP COMPOSITE FLOOR DECK PANEL, NORMAL DECK PANEL PROFILE ORIENTATION



For SI: 1 inch = 25.4 mm, 1 psi = 6895 Pa.

FIGURE 3—HILTI FASTENER LOCATIONS IN $1^1/_2$-INCH-DEEP COMPOSITE FLOOR DECK PANEL, INVERTED DECK PANEL PROFILE ORIENTATION

ICC-ES Evaluation Report

ESR-1663 FBC Supplement

Reissued March 2017

This report is subject to renewal March 2019.

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EVALUATION SUBJECT:

HILTI LOW-VELOCITY POWER-ACTUATED FASTENERS

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Hilti Low-Velocity Fasteners, recognized in ICC-ES master report ESR-1663, have also been evaluated for compliance with the codes noted below.

Applicable code editions:

- 2014 *Florida Building Code—Building*
- 2014 *Florida Building Code—Residential*

2.0 CONCLUSIONS

The Hilti EDS, DS, X-C, X-C20 THP, X-C22 P8TH, X-CR, X-R, X-W6, and W10 power-actuated fasteners, described in Sections 2.0 through 7.0 and in Tables 1 through 5 of the master report ESR-1663, comply with the *Florida Building Code—Building*, and the *Florida Building Code—Residential*, provided the design and installation are in accordance with the 2012 *International Building Code®* provisions noted in the master report, and the following additional conditions apply:

- Design wind loads must be based on Section 1609 of the *Florida Building Code—Building* or Section 301.2.1.1 of the *Florida Building Code—Residential*, as applicable.
- Load combinations must be in accordance with Section 1605.2 or Section 1605.3 of the *Florida Building Code—Building*, as applicable.

Use of the Hilti fasteners has also been found to be in compliance with the High-Velocity Hurricane Zone provisions of the *Florida Building Code—Building* and the *Florida Building Code—Residential* under the following conditions:

- Design wind loads must be based on Section 1620 of the *Florida Building Code—Building.*
- The use of Hilti EDS, DS, X-C, X-C20 THP, X-C22 P8TH, X-CR, X-R, X-W6, and W10 power-actuated fasteners as a means of attachment for wood blocking, as defined in Section 2330.1.1 of the *Florida Building Code—Building*, in a roof assembly in the High-Velocity Hurricane Zone, is prohibited.
- The fasteners have not been evaluated for use as cast-in-place anchors for compliance with the High-velocity Hurricane Zone provisions and this use is outside the scope of this evaluation report.

For products falling under Florida Rule 9N-3, verification that the report holder's quality-assurance program is audited by a quality-assurance entity approved by the Florida Building Commission for the type of inspections being conducted is the responsibility of an approved validation entity (or the code official, when the report holder does not possess an approval by the Commission).

This supplement expires concurrently with the master evaluation report, reissued March 2017.

TABLE 3—ALLOWABLE LOADS FOR FASTENERS DRIVEN INTO NORMAL-WEIGHT CONCRETE[1,2,4]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT DEPTH (inches)	ALLOWABLE LOADS (lbf)							
Concrete Compressive Strength:				2000 psi		4000 psi		6000 psi		8000 psi	
Load Direction:				Tension	Shear	Tension	Shear	Tension	Shear	Tension	Shear
Universal Knurled Shank	X-U	0.157	3/4	100	125	100	125	105	205	–	–
			1	165	190	170	225	110[3]	280[3]	–	–
			1 1/4	240	310	280	310	180	425	–	–
			1 1/2	275	420	325	420	–	–	–	–
Smooth Shank	X-P	0.157	3/4[5]	100	155	100	175	105	205	135	205
			1[5]	165	220	180	225	150	300	150	215
			1 1/4[5]	240	310	280	310	180	425	–	–
			1 1/2[5]	310	420	–	–	–	–	–	–

For SI: 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 psi = 6895 Pa.

[1]Unless otherwise noted, values apply to normal weight cast-in-place concrete. Fasteners must not be driven until the concrete has reached the designated minimum compressive strength.
[2]Unless otherwise noted, concrete thickness must be a minimum of 3 times the embedment depth of the fastener.
[3]This allowable load value for the X-U fastener also applies to normal weight hollow core concrete slabs with f'_c of 6600 psi and minimum dimensions shown in Figure 7, when installed in accordance with Section 4.2.4.
[4]The fasteners listed in the table above may be used for static load conditions and for the seismic load conditions described in Section 4.1.7, as applicable. The tabulated allowable loads apply to static load conditions. For seismic load conditions, the allowable loads must be limited in accordance with Section 4.1.7, Items 2 and 3, as applicable.
[5]Applies to fastening of cold-formed steel up to 54 mil thick using the X-P 22, X-P 27, X-P 34 and X-P 40 fasteners, respectively, for the 3/4, 1, 1 1/4 and 1 1/2 inch embedment depths.

TABLE 4—ALLOWABLE LOADS FOR FASTENERS DRIVEN INTO NORMAL-WEIGHT CONCRETE USING DX-KWIK[1,2,3,4]

FASTENER DESCRIPTION	FASTENER	SHANK DIAMETER (inch)	MINIMUM EMBEDMENT (inches)	ALLOWABLE LOADS (lbf)			
Concrete Compressive Strength:				4,000 psi		6,000 psi	
Load Direction:				Tension	Shear	Tension	Shear
Universal Knurled Shank	X-U 47 P8 w/ DX-KWIK	0.157	1 1/2	395	405	360	570

For SI: 1 inch = 25.4 mm, 1 lbf = 4.4 N, 1 psi = 6895 Pa.

[1]X-U Fastener is installed using the DX-KWIK drilled pilot hole installation procedure described in Section 4.2.5.
[2]Pilot holes must not be drilled until the concrete has reached the designated minimum compressive strength.
[3]Concrete thickness must be a minimum of 3 times the embedment depth of the fastener.
[4]The fasteners listed in the table above may be used for static load conditions and for the seismic load conditions described in Section 4.1.7, as applicable. The tabulated allowable loads apply to static load conditions. For seismic load conditions, the allowable loads must be limited in accordance with Section 4.1.7, Items 2 and 3, as applicable.

MA4

SHEET NOTES:

1. SEE SHEET S1.02 FOR FRAMING SCHEDULES.
2. SEE SHEET S1.02 FOR KEY NOTES.
3. SEE SHEET S1.02 FOR FRAMING NOTES.
4. SEE SHEET S1.02 FOR ALL OTHER FRAMING PLAN NOTES.

11.75 @ 1ST

9'-0 walls

CONFERENCE RM 120
COMMUNITY ROOM 117
PROPERTY MANAGEMENT 115
UNISEX RESTROOM 119
CLOSET 117b
SERVICES 116
STORAGE 111
IT CLOSET 116b
TRASH 114a
TRASH 114
STORAGE 117a
LAUNDRY 115
STAIR 1 ST-1
PROGRAM 112
CORRIDOR 100
ELECTRICAL 102
FIRE PUMP 102a
IDF 103
SEE SHEET S2.01T AND S2.01R FOR LEVEL 1 SLAB REQUIREMENTS
C1 105
B1 109
C1 104
C1 106
C1 108
C1 110
STAIR 2 ST-2

M K.8 K.7 J.9 G E.9 E.1 D.2 C.8 C B A

10 9 8.9 6.9 6 2.9 2.7 2 1.2 1

10'-7.5 10'-2 13'-2.5 11'-4 10'-2" 10'-2 13'-5

* = 2-2x4 @16

◇ = 2-3x4 @16

⊂⊃ = 4x4 @8

1 / S2.01 LEVEL 1 FRAMING PLAN

1/8"=1'-0"