Siddiq Khan & Associates, Inc.
Design Wind Pressures per ASCE 7-98

Project : Gainor Residence Wind Loads Kd = 0.85
Project No. : 05-618.00

Wall Coefficients and Calculated Pressures for Low Rise Bldgs qz=qh*:

Trib. Area or $L^2/3$	Zone 4 & 5 Positive Press	Zone 4 Negative Press	Zone 5 Negative Press	Positive Pressures	Negative Zone 4	Negative Zone 5
10	1.000	-1.100	-1.400	50.84	-55.14	-68.07
15	0.969	-1.069	-1.338	49.50	-53.80	-65.39
20	0.947	-1.047	-1.294	48.55	-52.85	-63.49
25	0.930	-1.030	-1.259	47.81	-52.12	-62.01
30	0.916	-1.016	-1.231	47.21	-51.51	-60.81
35	0.904	-1.004	-1.208	46.70	-51.00	-59.79
40	0.894	-0.994	-1.187	46.26	-50.56	-58.91
50	0.877	-0.977	-1.153	45.52	-49.83	-57.43
60	0.863	-0.963	-1.125	44.92	-49.22	-56.23
70	0.851	-0.951	-1.101	44.41	-48.71	-55.21
80	0.840	-0.940	-1.081	43.96	-48.27	-54.33
90	0.831	-0.931	-1.063	43.58	-47.88	-53.55
100	0.823	-0.923	-1.047	43.23	-47.54	-52.85
110	0.816	-0.916	-1.032	42.91	-47.22	-52.22
120	0.809	-0.909	-1.019	42.63	-46.93	-51.65
130	0.803	-0.903	-1.007	42.36	-46.67	-51.12
140	0.798	-0.898	-0.995	42.12	-46.42	-50.63
150	0.792	-0.892	-0.985	41.89	-46.20	-50.17
200	0.770	-0.870	-0.940	40.94	-45.25	-48.27
300	0.739	-0.839	-0.878	39.60	-43.91	-45.59
400	0.717	-0.817	-0.834	38.65	-42.95	-43.69
500	0.700	-0.800	-0.800	37.91	-42.22	-42.22

* Internal pressure 0.18qh included in above calculations to produce maximum loading

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Gainor Residence
5800 North Bay Road
City of Miami Beach, Florida

Wind Load Calculations Kd = 1.00

(Components and Cladding Only)

Siddiq Khan & Associates, Inc.
Design Wind Pressures per ASCE 7-98

Project : **Gainor Residence Wind Loads Kd = 1.0 (Doors, Windows, Roofing Pressures)**
Project No. : **05-618.00**

Mean Roof Height	**23.00 ft.**	15	13.8	... z-bar =	**15.00 ft.**	
B width	**35.00 ft.**					
Exposure	**C**					
Hurricanes?	**Y**					
Importance	**II**	**I = 1.00**				
Velocity	**146**					
"a" =	**3.50**					

Importance Factor Table 6-1

Category	Non-Hurricane	Hurricane
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

Table 6-4 Terrain Exposure Constants

Exposure	Alpha	Zg	a^	b^	Alpha-Bar	b-bar	c	l	epsilon-bar	z min.
A	5	1500	1/5.0	0.64	1/3.0	0.3	0.45	180	0.500	60
B	7	1200	1/7.0	0.84	1/4.0	0.45	0.3	320	0.333	30
C	9.5	900	1/9.5	1	1/6.5	0.65	0.2	500	0.200	15
D	11.5	700	1/11.5	1.07	1/9.0	0.8	0.15	650	0.125	7

Kzt =	1	Escarpment or Axisymetrical Hill
Kd =	1	
Calculate kh:	0.929	
Calculate qh:	**50.68**	

Calculate Gh: **Internal Pressure =** **9.12**

Iz-bar =	0.2281
Lz-bar =	427.0566
Q =	0.9209
G =	**0.8500**

2-9

Siddiq Khan & Associates, Inc.
Design Wind Pressures per ASCE 7-98

Project : Gainor Residence Wind Loads Kd = 1.0 (Doors, Windows, Roofing Pressures)
Project No. : 05-618.00

Calculate MWFRS loads per Eq. 6-15, applicable for all heights...Loads Application per Figure 6-3

Main Wind Force Resisting System for Vertical Surfaces:

	Height z	Kz	qz	windward qz*G*0.8	leeward qh*G*0.5	Sides qh*G*0.7	W+L pressure
TOF	-1.33						
GF	0.00	0.849	46.32	**31.50**	**21.54**	**30.16**	**53.04**
	15.00	0.849	46.32	**31.50**	**21.54**	**30.16**	**53.04**
	20.00	0.902	49.21	**33.47**	**21.54**	**30.16**	**55.01**
	25.00	0.945	51.58	**35.08**	**21.54**	**30.16**	**56.62**
	30.00	0.982	53.60	**36.45**	**21.54**	**30.16**	**57.99**
	35.00	1.015	55.37	**37.65**	**21.54**	**30.16**	**59.19**

Note: Internal pressure not added to above calculation

Main Wind Force Resisting System for Roofs theta = 14.04 degrees (3:12 Slope):

Wind Normal to Ridge					Leeward Side Beyond Ridge			
h/L	Cp	Pressure	w/ - internal	w/ + internal	Cp	Pressure	w/ - internal	w/ + internal
≤ 0.25	**-0.5386**	**-23.2**	**-14.1**	**-32.3**	**-0.4614**	**-19.88**	**-10.75**	**-29.00**
.5	**-0.7386**	**-31.8**	**-22.7**	**-40.9**	**-0.5000**	**-21.54**	**-12.42**	**-30.66**
≥ 1.0	**-1.0578**	**-45.6**	**-36.4**	**-54.7**	**-0.6193**	**-26.68**	**-17.56**	**-35.80**

Wind Parallel to Ridge for h/L ≤ 0.5

0 to h/2, Cp =	**-0.90**	**-38.8**	**-29.7**	**-47.9**
h/2 to h, Cp =	**-0.90**	**-38.8**	**-29.7**	**-47.9**
h to 2h, Cp =	**-0.50**	**-21.5**	**-12.4**	**-30.7**
> 2h, Cp =	**-0.30**	**-12.9**	**-3.8**	**-22.0**

Wind Parallel to Ridge for h/L ≥ 1.0

0 to h/2, Cp =	-1.30	-56.0	-46.9	-65.1
> h/2, Cp =	-0.70	-30.2	-21.0	-39.3

Siddiq Khan & Associates, Inc.
Design Wind Pressures per ASCE 7-98

Project : Gainor Residence Wind Loads Kd = 1.0 (Doors, Windows, Roofing Pressures)
Project No. : 05-618.00

Roof Pressure Coefficients and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

	Negative Press. Coefficents		Pos. Press.	Overhang Coefficents		Negative Pressures		Overhang Pressures**		Positive
Trib. Area or $L^2/3$	Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3	Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones
10	-0.900	-2.100	0.50	-2.20	-3.70	-54.7	-115.56	-111.51	-187.53	34.5
15	-0.882	-1.977	0.46	-2.20	-3.49	-53.9	-109.31	-111.51	-176.82	32.7
20	-0.870	-1.889	0.44	-2.20	-3.34	-53.2	-104.88	-111.51	-169.22	31.4
25	-0.860	-1.821	0.42	-2.20	-3.22	-52.7	-101.44	-111.51	-163.32	30.4
30	-0.852	-1.766	0.40	-2.20	-3.13	-52.3	-98.63	-111.51	-158.51	29.6
35	-0.846	-1.719	0.39	-2.20	-3.05	-52.0	-96.26	-111.51	-154.43	28.9
40	-0.840	-1.679	0.38	-2.20	-2.98	-51.7	-94.20	-111.51	-150.91	28.4
45	-0.835	-1.643	0.37	-2.20	-2.92	-51.4	-92.39	-111.51	-147.79	27.8
50	-0.830	-1.611	0.36	-2.20	-2.86	-51.2	-90.76	-111.51	-145.01	27.4
55	-0.826	-1.582	0.35	-2.20	-2.81	-51.0	-89.29	-111.51	-142.49	27.0
60	-0.822	-1.555	0.34	-2.20	-2.77	-50.8	-87.95	-111.51	-140.19	26.6
65	-0.819	-1.531	0.34	-2.20	-2.72	-50.6	-86.72	-111.51	-138.08	26.2
70	-0.816	-1.508	0.33	-2.20	-2.69	-50.5	-85.58	-111.51	-136.12	25.9
75	-0.813	-1.487	0.32	-2.20	-2.65	-50.3	-84.52	-111.51	-134.30	25.6
80	-0.810	-1.468	0.32	-2.20	-2.62	-50.2	-83.52	-111.51	-132.60	25.3
85	-0.807	-1.449	0.31	-2.20	-2.58	-50.0	-82.59	-111.51	-130.99	25.0
90	-0.805	-1.432	0.31	-2.20	-2.55	-49.9	-81.71	-111.51	-129.48	24.8
95	-0.802	-1.416	0.30	-2.20	-2.53	-49.8	-80.87	-111.51	-128.06	24.5
100	-0.800	-1.400	0.30	-2.20	-2.50	-49.7	-80.08	-111.51	-126.70	24.3

* Internal pressure 0.18qh included in above calculations to produce maximum loading
** Overhangs do not include internal pressure coefficients

Siddiq Khan & Associates, Inc.
Design Wind Pressures per ASCE 7-98

Project : Gainor Residence Wind Loads Kd = 1.0 (Doors, Windows, Roofing Pressures)
Project No. : 05-618.00

Wall Coefficients and Calculated Pressures for Low Rise Bldgs qz=qh*:

Trib. Area or $L^2/3$	Zone 4 & 5 Positive Press	Zone 4 Negative Press	Zone 5 Negative Press	Positive Pressures	Negative Zone 4	Negative Zone 5
10	1.000	-1.100	-1.400	59.81	-64.88	-80.08
15	0.969	-1.069	-1.338	58.23	-63.30	-76.93
20	0.947	-1.047	-1.294	57.11	-62.18	-74.69
25	0.930	-1.030	-1.259	56.25	-61.31	-72.96
30	0.916	-1.016	-1.231	55.54	-60.60	-71.54
35	0.904	-1.004	-1.208	54.94	-60.01	-70.34
40	0.894	-0.994	-1.187	54.42	-59.49	-69.30
50	0.877	-0.977	-1.153	53.55	-58.62	-67.57
60	0.863	-0.963	-1.125	52.84	-57.91	-66.15
70	0.851	-0.951	-1.101	52.24	-57.31	-64.95
80	0.840	-0.940	-1.081	51.72	-56.79	-63.91
90	0.831	-0.931	-1.063	51.27	-56.33	-63.00
100	0.823	-0.923	-1.047	50.86	-55.92	-62.18
110	0.816	-0.916	-1.032	50.49	-55.55	-61.44
120	0.809	-0.909	-1.019	50.15	-55.22	-60.76
130	0.803	-0.903	-1.007	49.84	-54.90	-60.14
140	0.798	-0.898	-0.995	49.55	-54.62	-59.56
150	0.792	-0.892	-0.985	49.28	-54.35	-59.03
200	0.770	-0.870	-0.940	48.16	-53.23	-56.79
300	0.739	-0.839	-0.878	46.59	-51.65	-53.64
400	0.717	-0.817	-0.834	45.47	-50.54	-51.40
500	0.700	-0.800	-0.800	44.60	-49.67	-49.66

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Siddiq Khan & Associates, Inc.
Design Wind Pressures per ASCE 7-98

Project : Gainor Residece
Project No. : 05-618.00 **Kd=1.0 for Flat Roof**

Roof Pressure Coefficents and Calculated Roof Pressures for Theta Less than 10 degrees, qz=qh*:

	Negative Press. Coefficents			Pos. Press.	Overhang Coefficents	
Trib. Area or $L^2/3$	Zone 1	Zone 2	Zone 3	All Zones	Zone 1&2	Zone 3
10	-1.000	-1.800	-2.800	0.30	-1.70	-2.80
15	-0.982	-1.677	-2.501	0.28	-1.68	-2.45
20	-0.970	-1.589	-2.288	0.27	-1.67	-2.20
25	-0.960	-1.521	-2.124	0.26	-1.66	-2.00
30	-0.952	-1.466	-1.989	0.25	-1.65	-1.85
35	-0.946	-1.419	-1.875	0.25	-1.65	-1.71
40	-0.940	-1.379	-1.777	0.24	-1.64	-1.60
50	-0.930	-1.311	-1.612	0.23	-1.63	-1.40
60	-0.922	-1.255	-1.477	0.22	-1.62	-1.24
70	-0.916	-1.208	-1.363	0.22	-1.62	-1.11
80	-0.910	-1.168	-1.265	0.21	-1.61	-0.99
90	-0.905	-1.132	-1.178	0.20	-1.60	-0.89
100	-0.900	-1.100	-1.100	0.20	-1.60	-0.80

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Negative Pressures			Overhang Pressures		Positive
Zone 1	Zone 2	Zone 3	Zone 1&2	Zone 3	All Zones
-59.8	-100.36	-151.04	-95.29	-151.04	24.3
-58.9	-94.11	-135.87	-94.40	-133.19	23.4
-58.3	-89.68	-125.10	-93.77	-120.52	22.8
-57.8	-86.24	-116.75	-93.27	-110.70	22.3
-57.4	-83.43	-109.93	-92.87	-102.67	21.9
-57.1	-81.05	-104.16	-92.53	-95.89	21.6
-56.8	-79.00	-99.16	-92.24	-90.01	21.3
-56.3	-75.56	-90.81	-91.75	-80.18	20.8
-55.9	-72.75	-83.99	-91.35	-72.16	20.4
-55.5	-70.37	-78.22	-91.01	-65.37	20.1
-55.2	-68.32	-73.23	-90.72	-59.49	19.8
-55.0	-66.50	-68.82	-90.46	-54.31	19.5
-54.7	-64.88	-64.88	-90.23	-49.67	19.3

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Truss Reaction and Anchor Designs

SIDDIQ KHAN & ASSOCIATES, INC.
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(305) 662-2301

JOB Gainor Res
SHEET NO. _____ OF _____
CALCULATED BY LHO DATE 9/27/05
CHECKED BY _____ DATE _____
SCALE _____

Given: DL = 25 psf ; LL = 30 psf

Truss Name	Left O.H.	Net Span	Right O.H.	Trib
T-1	1'	22'-8"	1'	2'-0"
T-2	1'	17'-3"	1'	2'-0"
T-3	1'	24'-2"	1'	2'-0"
T-3a	0	25'-2"	0	2'-0"
T-4	1'	25'-8"	1'	2'-0"
T-5	1'	31'-8"	1'	2'-0"
T-6	0	15'-6"	3'-6" < 2'-0" ok	2'-0"
T-7	0	24'-8"	1'	2'-0"
T-8	0	12'-8"	0	2'-0"
GT-1	1'	22'-8"	1'	4'-6"
GT-2	1	24'-2"	1'	4'-6"
GT-3	1'	25'-8"	0	12'-6"
GT-4	1'	31'-8"	1'	5'-6"
T-5	2'	31'-8"	1'	4'-3"
H-1	1.41'	9'-5"	0	4'-0"
H-2	1.41'	12'-3"	0	4'-0"

SIDDIQ KHAN & ASSOCIATES, INC.
Consulting Engineers And Planners
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MIAMI, FLORIDA 33155
(305) 662-2301

JOB Gainor Res
SHEET NO. 2 OF
CALCULATED BY Leo DATE 9/27/05
CHECKED BY DATE
SCALE

Given (Cont'd)

Truss Name	Left O.H.	Main Span	Right O.H.	Trib.
J1	1'-0"	6'-8"	0	2'-0"
J2	1'-0"	8'-8"	0	2'-0"

Siddiq Khan & Associates, Inc.

Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

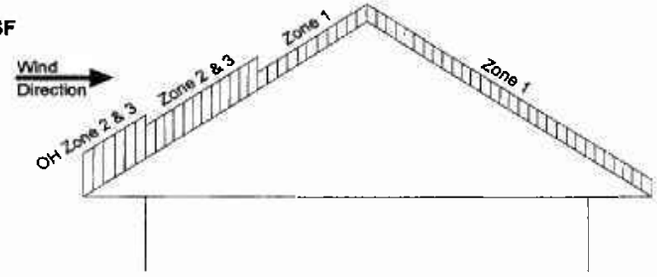
Project : Gainor Residence
Project No. : 05-618.00

Truss T-1

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	22.67 ft.	1.00 ft.	2.00 ft.	L^2/3 =	171 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	616.8 lbs		Right DL Reaction	616.8 lbs		
Left LL Reaction	740.1 lbs		Right LL Reaction	740.1 lbs		

1357



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-1316	-1263	-369
Right Reaction =	-1053	-1011	-295

D + Lr =	1357	
0.6D + W =	-893	10 psf - W = -1017
D + Lr + W =	93	
D + 0.75(Lr + W) =	224	(no increase in stress allowed for this combo)

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-1342	-1289	-376
Right Reaction =	-1053	-1011	-295

D + Lr =	1357	
0.6D + W =	-919	10 psf - W = -1042
D + Lr + W =	68	
D + 0.75(Lr + W) =	205	(no increase in stress allowed for this combo)

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	243	234	68
Right Reaction =	-393	-377	-110

D + Lr =	1357	
0.6D + W =	604	10 psf - W = 480
D + Lr + W =	1591	
D + 0.75(Lr + W) =	1347	(no increase in stress allowed for this combo)

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

4-4

Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

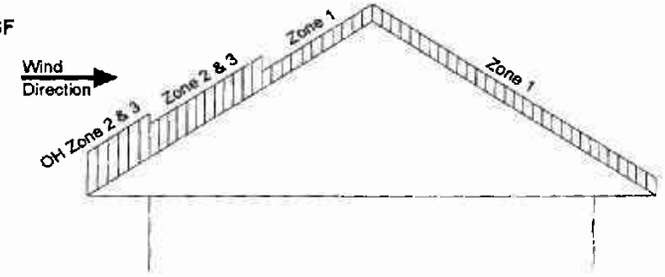
Project : Gainor Residence
Project No. : 05-618.00

* Truss T-2

	H	B	V	kd	kzt	G	qh	0.18*qh	a
	23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	17.25 ft.	1.00 ft.	2.00 ft.	L^2/3 =	99 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	481.3 lbs		Right DL Reaction	481.3 lbs		
Left LL Reaction	577.5 lbs		Right LL Reaction	577.5 lbs		

≈ 1060



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.403	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.18	-94.78	-107.88	20.70	-26.73

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-1084	-1041	-304
Right Reaction =	-829	-795	-232

D + Lr =	1059		
0.6D + W =	-752	10 psf - W =	-849
D + Lr + W =	18		
D + 0.75(Lr + W) =	134	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-1111	-1067	-311
Right Reaction =	-828	-795	-232

D + Lr =	1059		
0.6D + W =	-778	10 psf - W =	-874
D + Lr + W =	-8		
D + 0.75(Lr + W) =	114	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	197	189	55
Right Reaction =	-313	-300	-88

D + Lr =	1059		
0.6D + W =	478	10 psf - W =	381
D + Lr + W =	1248		
D + 0.75(Lr + W) =	1056	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

S-4

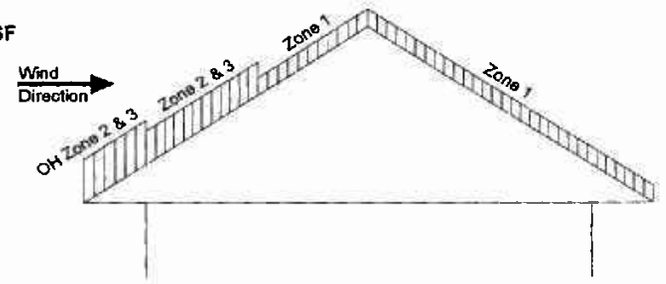
Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

Project : Gainor Residence
Project No. : 05-618.00

- **Truss T-3**

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	24.17 ft.	1.00 ft.	2.00 ft.	L^2/3 =	195 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	654.3 lbs		Right DL Reaction	654.3 lbs		
Left LL Reaction	785.1 lbs		Right LL Reaction	785.1 lbs		



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-1380	-1325	-386
Right Reaction =	-1116	-1071	-312

D + Lr =	1439		
0.6D + W =	-932	10 psf - W =	-1063
D + Lr + W =	114		
D + 0.75(Lr + W) =	249	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-1407	-1350	-394
Right Reaction =	-1115	-1071	-312

D + Lr =	1439		
0.6D + W =	-958	10 psf - W =	-1089
D + Lr + W =	89		
D + 0.75(Lr + W) =	230	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	257	246	72
Right Reaction =	-415	-398	-116

D + Lr =	1439		
0.6D + W =	639	10 psf - W =	508
D + Lr + W =	1686		
D + 0.75(Lr + W) =	1428	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

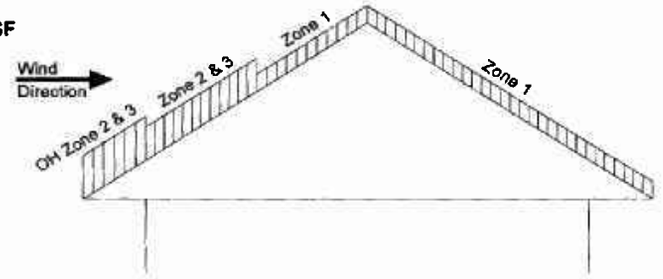
Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

Project : Gainor Residence
Project No. : 05-618.00

Truss T-3a

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	0.00 ft.	25.17 ft.	0.00 ft.	2.00 ft.	L^2/3 =	211 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	629.3 lbs		Right DL Reaction	629.3 lbs		
Left LL Reaction	755.1 lbs		Right LL Reaction	755.1 lbs		



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-1231	-1182	-345
Right Reaction =	-1075	-1032	-301

D + Lr =	1384		
0.6D + W =	-804	10 psf - W =	-930
D + Lr + W =	202		
D + 0.75(Lr + W) =	309	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-1231	-1182	-345
Right Reaction =	-1075	-1032	-301

D + Lr =	1384		
0.6D + W =	-804	10 psf - W =	-930
D + Lr + W =	202		
D + 0.75(Lr + W) =	309	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	222	213	62
Right Reaction =	-374	-359	-105

D + Lr =	1384		
0.6D + W =	591	10 psf - W =	465
D + Lr + W =	1598		
D + 0.75(Lr + W) =	1355	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

Siddiq Khan & Associates, Inc.

Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

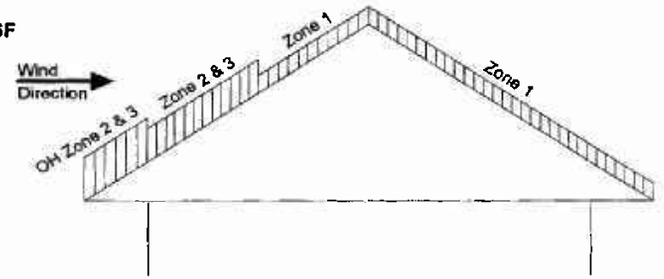
Project : Gainor Residence
Project No. : 05-618.00

Truss T-4

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	0.00 ft.	20.00 ft.	1.00 ft.	2.00 ft.	L^2/3 =	133 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	498.8 lbs		Right DL Reaction	551.3 lbs		
Left LL Reaction	598.5 lbs		Right LL Reaction	661.5 lbs		

1398



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-1007	-967	-282
Right Reaction =	-947	-909	-265

D + Lr =	1097		
0.6D + W =	-668	10 psf - W =	-768
D + Lr + W =	130		
D + 0.75(Lr + W) =	222	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-1007	-967	-282
Right Reaction =	-947	-909	-265

D + Lr =	1097		
0.6D + W =	-668	10 psf - W =	-768
D + Lr + W =	130		
D + 0.75(Lr + W) =	222	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	178	171	50
Right Reaction =	-352	-338	-99

D + Lr =	1097		
0.6D + W =	470	10 psf - W =	370
D + Lr + W =	1268		
D + 0.75(Lr + W) =	1076	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

7-38

Siddiq Khan & Associates, Inc.

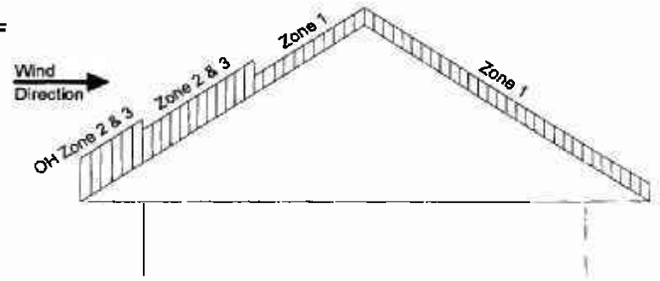
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

Project : Gainor Residence
Project No. : 05-618.00

- Truss T-4A

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	0.00 ft.	5.67 ft.	1.00 ft.	2.00 ft.	L^2/3 =	11 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	137.3 lbs		Right DL Reaction	196.2 lbs		
Left LL Reaction	164.8 lbs		Right LL Reaction	235.4 lbs		



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.895	-2.062	0.49	-2.20	-3.63

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-46.3	-96.58	-94.78	-156.57	28.82	-30.79

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta				
				D + Lr =	302		
				0.6D + W =	-395	10 psf - W =	-423
Left Reaction =	-498	-478	-139	D + Lr + W =	-176		
Right Reaction =	-472	-453	-132	D + 0.75(Lr + W) =	-97	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta				
				D + Lr =	302		
				0.6D + W =	-395	10 psf - W =	-423
Left Reaction =	-498	-478	-139	D + Lr + W =	-176		
Right Reaction =	-472	-453	-132	D + 0.75(Lr + W) =	-97	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta				
				D + Lr =	302		
				0.6D + W =	163	10 psf - W =	136
Left Reaction =	84	81	24	D + Lr + W =	383		
Right Reaction =	-157	-151	-44	D + 0.75(Lr + W) =	322	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

Siddiq Khan & Associates, Inc.

Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

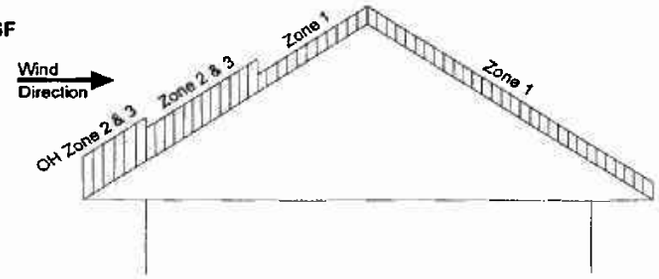
Project : Gainor Residence
Project No. : 05-618.00

* Truss T-5

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	31.67 ft.	1.00 ft.	2.00 ft.	L^2/3 =	334 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	841.8 lbs		Right DL Reaction	841.8 lbs		
Left LL Reaction	1010.1 lbs		Right LL Reaction	1010.1 lbs		

1852



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents		Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3	Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-0.800	-1.400	0.30	-2.20	-2.50	-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta				
Left Reaction =	-1699	-1631	-476	D + Lr =	1852		
Right Reaction =	-1430	-1373	-400	0.6D + W =	-1126	10 psf - W =	-1295
				D + Lr + W =	220		
				D + 0.75(Lr + W) =	376	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta				
Left Reaction =	-1726	-1657	-483	D + Lr =	1852		
Right Reaction =	-1430	-1372	-400	0.6D + W =	-1152	10 psf - W =	-1320
				D + Lr + W =	195		
				D + 0.75(Lr + W) =	357	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta				
Left Reaction =	322	309	90	D + Lr =	1852		
Right Reaction =	-526	-505	-147	0.6D + W =	814	10 psf - W =	646
				D + Lr + W =	2161		
				D + 0.75(Lr + W) =	1831	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

T-10

Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

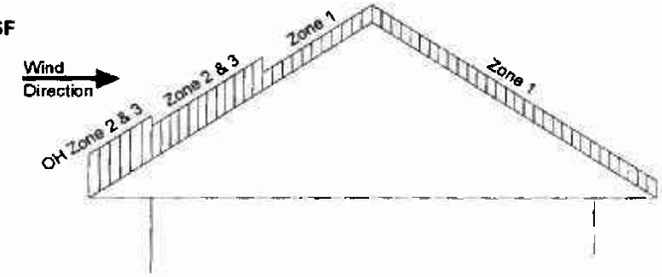
Project : Gainor Residence
Project No. : 05-618.00

• Truss T-6

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	3.50 ft.	15.50 ft.	0.00 ft.	2.00 ft.	L^2/3 =	80 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	582.3 lbs		Right DL Reaction	367.7 lbs		
Left LL Reaction	698.7 lbs		Right LL Reaction	441.3 lbs		

≈ 1282



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.810	-1.468	0.32	-2.20	-2.62

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.6	-70.98	-94.78	-112.68	21.50	-27.13

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-1575	-1512	-441
Right Reaction =	-608	-584	-170

D + Lr =	1281	
0.6D + W =	-1163	10 psf - W = -1279
D + Lr + W =	-231	
D + 0.75(Lr + W) =	-28	(no increase in stress allowed for this combo)

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-1715	-1646	-480
Right Reaction =	-594	-571	-166

D + Lr =	1281	
0.6D + W =	-1297	10 psf - W = -1413
D + Lr + W =	-365	
D + 0.75(Lr + W) =	-128	(no increase in stress allowed for this combo)

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	312	300	87
Right Reaction =	-249	-239	-70

D + Lr =	1281	
0.6D + W =	649	10 psf - W = 533
D + Lr + W =	1581	
D + 0.75(Lr + W) =	1331	(no increase in stress allowed for this combo)

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

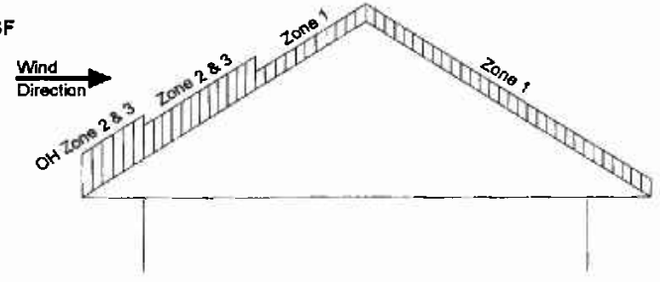
Project : Gainor Residence
Project No. : 05-618.00

Truss T-7

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	0.00 ft.	24.67 ft.	1.00 ft.	2.00 ft.	L^2/3 =	203 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	615.7 lbs		Right DL Reaction	667.8 lbs		
Left LL Reaction	738.9 lbs		Right LL Reaction	801.3 lbs		

≤ 1470



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents		Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3	Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-0.800	-1.400	0.30	-2.20	-2.50	-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-1208	-1160	-338
Right Reaction =	-1141	-1095	-319

D + Lr =	1355		
0.6D + W =	-790	10 psf - W =	-913
D + Lr + W =	195		
D + 0.75(Lr + W) =	300	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-1208	-1160	-338
Right Reaction =	-1141	-1095	-319

D + Lr =	1355		
0.6D + W =	-790	10 psf - W =	-913
D + Lr + W =	195		
D + 0.75(Lr + W) =	300	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	219	210	61
Right Reaction =	-421	-404	-118

D + Lr =	1355		
0.6D + W =	579	10 psf - W =	456
D + Lr + W =	1565		
D + 0.75(Lr + W) =	1327	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

7-12

Siddiq Khan & Associates, Inc.

Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

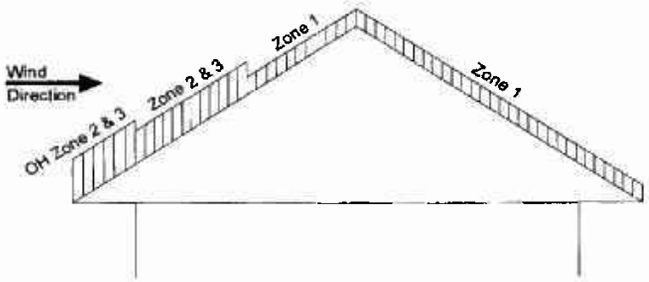
Project : Gainor Residence
Project No. : 05-618.00

Truss T-8

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	0.00 ft.	12.67 ft.	0.00 ft.	2.00 ft.	L^2/3 =	54 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	316.8 lbs		Right DL Reaction	316.8 lbs		
Left LL Reaction	380.1 lbs		Right LL Reaction	380.1 lbs		

≈ 697#



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents		Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3	Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-0.827	-1.590	0.35	-2.20	-2.83	-43.4	-76.26	-94.78	-121.74	23.01	-27.89

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta			
Left Reaction =	-748	-718	-209	D + Lr =	697	
Right Reaction =	-582	-558	-163	0.6D + W =	-528	10 psf - W = -591
				D + Lr + W =	-21	
				D + 0.75(Lr + W) =	63	(no increase in stress allowed for this combo)

Uplift Reactions For Zone 3:

		Cos theta	Sin theta			
Left Reaction =	-748	-718	-209	D + Lr =	697	
Right Reaction =	-582	-558	-163	0.6D + W =	-528	10 psf - W = -591
				D + Lr + W =	-21	
				D + 0.75(Lr + W) =	63	(no increase in stress allowed for this combo)

Downdraft Reactions All Zones:

		Cos theta	Sin theta			
Left Reaction =	130	125	36	D + Lr =	697	
Right Reaction =	-192	-184	-54	0.6D + W =	315	10 psf - W = 252
				D + Lr + W =	822	
				D + 0.75(Lr + W) =	696	(no increase in stress allowed for this combo)

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

3-1-7

Siddiq Khan & Associates, Inc.

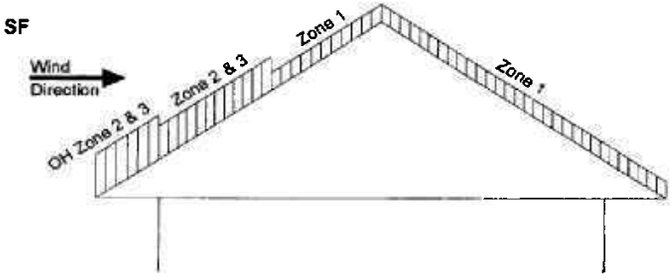
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

Project : Gainor Residence
Project No. : 05-618.00

Girder Truss GT-1

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	22.67 ft.	1.00 ft.	4.50 ft.	L^2/3 =	171 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	1387.7 lbs		Right DL Reaction	1387.7 lbs		
Left LL Reaction	1665.2 lbs		Right LL Reaction	1665.2 lbs		



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-2961	-2843	-829
Right Reaction =	-2370	-2275	-664

D + Lr =	3053
0.6D + W =	-2010
D + Lr + W =	210
D + 0.75(Lr + W) =	505

10 psf - W = -2288

(no increase in stress allowed for this combo)

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-3021	-2900	-846
Right Reaction =	-2369	-2274	-663

D + Lr =	3053
0.6D + W =	-2067
D + Lr + W =	153
D + 0.75(Lr + W) =	462

10 psf - W = -2345

(no increase in stress allowed for this combo)

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	548	526	153
Right Reaction =	-883	-848	-247

D + Lr =	3053
0.6D + W =	1359
D + Lr + W =	3579
D + 0.75(Lr + W) =	3031

10 psf - W = 1081

(no increase in stress allowed for this combo)

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

Siddiq Khan & Associates, Inc.

Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

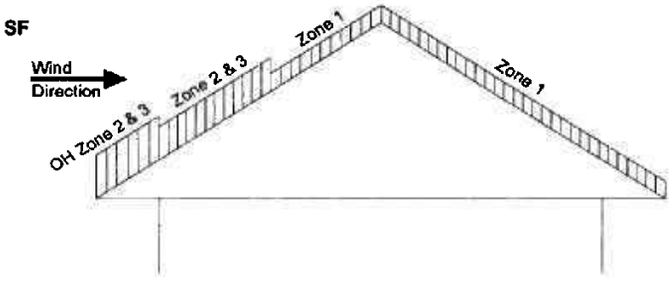
Project : Gainor Residence
Project No. : 05-618.00

• Girder Truss GT-2

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	24.17 ft.	1.00 ft.	4.50 ft.	L^2/3 =	195 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	1472.1 lbs		Right DL Reaction	1472.1 lbs		
Left LL Reaction	1766.5 lbs		Right LL Reaction	1766.5 lbs		

≈3239



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-3105	-2981	-869
Right Reaction =	-2511	-2411	-703

D + Lr =	3239		
0.6D + W =	-2098	10 psf - W =	-2392
D + Lr + W =	257		
D + 0.75(Lr + W) =	561	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-3165	-3038	-886
Right Reaction =	-2510	-2409	-703

D + Lr =	3239		
0.6D + W =	-2155	10 psf - W =	-2449
D + Lr + W =	200		
D + 0.75(Lr + W) =	518	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	577	554	162
Right Reaction =	-933	-896	-261

D + Lr =	3239		
0.6D + W =	1437	10 psf - W =	1143
D + Lr + W =	3793		
D + 0.75(Lr + W) =	3213	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

S1-4

Siddiq Khan & Associates, Inc.

Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

Project : Gainor Residence
Project No. : 05-618.00

Girder GT-A

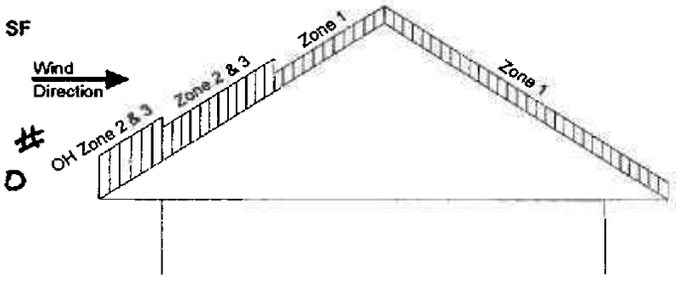
Main Girder Supporting Projector Girder Trusses

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	19.67 ft.	0.00 ft.	7.75 ft.	L^2/3 =	152 SF

Uniform DL 25.0 psf
Uniform LL 30.0 psf

Roof Slope 3.5 : 12 or 16.26 degrees



Concentrated DL 0.0 lbs 0.00 ft. Dist From Left Support
Concentrated LL 0.0 lbs
Con. Wind Up 0.0 lbs
Con. Wind Dn 0.0 lbs

≈ 4630# + 500# = 5130# for Projector

Left DL Reaction	2103.9 lbs	Right DL Reaction	1900.3 lbs
Left LL Reaction	2524.7 lbs	Right LL Reaction	2280.4 lbs

Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-4610	-4426	-1291
Right Reaction =	-3262	-3131	-913

D + Lr =	4629		
0.6D + W =	-3163	10 psf - W =	-3584
D + Lr + W =	203		
D + 0.75(Lr + W) =	678	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-4713	-4524	-1320
Right Reaction =	-3259	-3129	-913

D + Lr =	4629		
0.6D + W =	-3262	10 psf - W =	-3682
D + Lr + W =	105		
D + 0.75(Lr + W) =	604	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	837	803	234
Right Reaction =	-1137	-1092	-318

D + Lr =	4629		
0.6D + W =	2066	10 psf - W =	1645
D + Lr + W =	5432		
D + 0.75(Lr + W) =	4600	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

7-16

Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

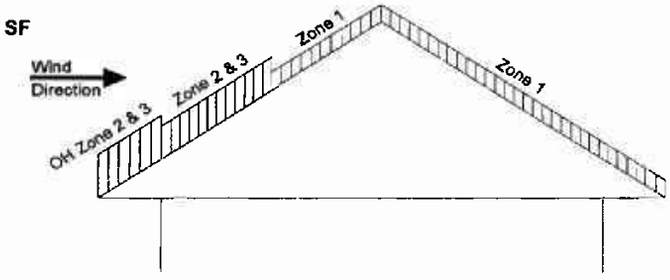
Project : Gainor Residence
Project No. : 05-618.00

Girder Truss GT-3

	H	B	V	kd	kzt	G	qh	0.18*qh	a
	23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	25.67 ft.	0.00 ft.	13.50 ft.	L^2/3 =	347 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	4675.9 lbs		Right DL Reaction	4325.2 lbs		
Left LL Reaction	5611.1 lbs		Right LL Reaction	5190.3 lbs		

≈ 10,300#



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-9759	-9369	-2733
Right Reaction =	-7375	-7080	-2065

D + Lr =	10287	
0.6D + W =	-6563	10 psf - W = -7498
D + Lr + W =	918	
D + 0.75(Lr + W) =	1858	(no increase in stress allowed for this combo)

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-9937	-9539	-2782
Right Reaction =	-7372	-7077	-2064

D + Lr =	10287	
0.6D + W =	-6734	10 psf - W = -7669
D + Lr + W =	748	
D + 0.75(Lr + W) =	1730	(no increase in stress allowed for this combo)

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	1813	1741	508
Right Reaction =	-2582	-2479	-723

D + Lr =	10287	
0.6D + W =	4546	10 psf - W = 3611
D + Lr + W =	12028	
D + 0.75(Lr + W) =	10190	(no increase in stress allowed for this combo)

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

17-1

Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

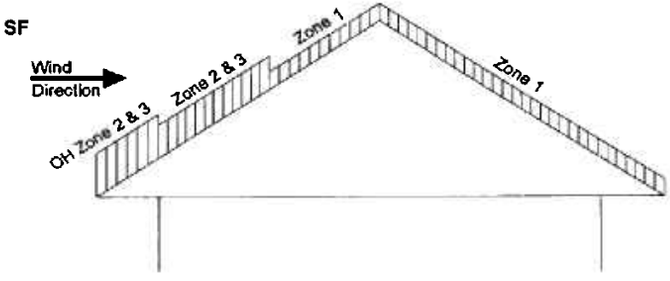
Project : Galnor Residence
Project No. : 05-618.00

Girder Truss GT-4

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	31.67 ft.	1.00 ft.	5.50 ft.	L^2/3 =	334 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	2314.8 lbs		Right DL Reaction	2314.8 lbs		
Left LL Reaction	2777.8 lbs		Right LL Reaction	2777.8 lbs		

≈5093



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-4673	-4487	-1309
Right Reaction =	-3933	-3775	-1101

D + Lr =	5093		
0.6D + W =	-3098	10 psf - W =	-3561
D + Lr + W =	606		
D + 0.75(Lr + W) =	1033	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-4746	-4556	-1329
Right Reaction =	-3932	-3774	-1101

D + Lr =	5093		
0.6D + W =	-3167	10 psf - W =	-3630
D + Lr + W =	537		
D + 0.75(Lr + W) =	981	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	886	851	248
Right Reaction =	-1446	-1388	-405

D + Lr =	5093		
0.6D + W =	2240	10 psf - W =	1777
D + Lr + W =	5943		
D + 0.75(Lr + W) =	5036	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

7-18

Siddiq Khan & Associates, Inc.

Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

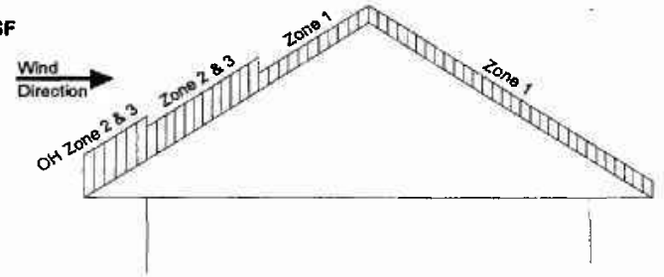
Project : Gainor Residence
Project No. : 05-618.00

Girder Truss GT-5

	H	B	V	kd	kzt	G	qh	0.18*qh	a
	23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	2.00 ft.	33.67 ft.	1.00 ft.	4.25 ft.	L^2/3 =	378 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	2006.0 lbs		Right DL Reaction	1890.2 lbs		
Left LL Reaction	2407.1 lbs		Right LL Reaction	2268.3 lbs		

≈ 4414



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.800	-1.400	0.30	-2.20	-2.50

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-42.2	-68.07	-94.78	-107.70	20.67	-26.72

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta				
Left Reaction =	-4213	-4044	-1180	D + Lr =	4413		
Right Reaction =	-3199	-3071	-896	0.6D + W =	-2840	10 psf - W =	-3242
				D + Lr + W =	369		
				D + 0.75(Lr + W) =	778	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta				
Left Reaction =	-4326	-4153	-1211	D + Lr =	4413		
Right Reaction =	-3196	-3068	-895	0.6D + W =	-2949	10 psf - W =	-3350
				D + Lr + W =	261		
				D + 0.75(Lr + W) =	697	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta				
Left Reaction =	814	781	228	D + Lr =	4413		
Right Reaction =	-1184	-1137	-332	0.6D + W =	1985	10 psf - W =	1584
				D + Lr + W =	5195		
				D + 0.75(Lr + W) =	4397	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

7-19

Siddiq Khan & Associates, Inc.

Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

Project : Gainor Residence
Project No. : 05-618.00

• Hip Beam H-1

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

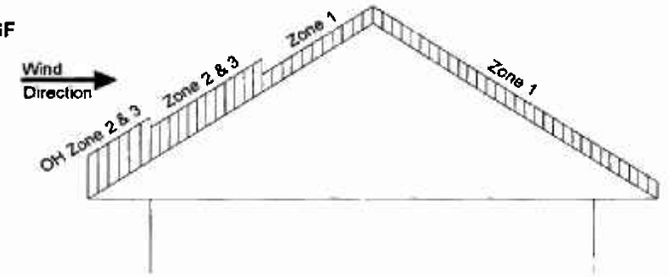
Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.42 ft.	9.43 ft.	0.00 ft.	4.00 ft.	L^2/3 =	38 SF

Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees
Uniform LL	30.0 psf				

Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support
Concentrated LL	0.0 lbs		
Con. Wind Up	0.0 lbs		
Con. Wind Dn	0.0 lbs		

≈1374#

Left DL Reaction	624.2 lbs	Right DL Reaction	460.8 lbs
Left LL Reaction	749.0 lbs	Right LL Reaction	553.0 lbs



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.842	-1.696	0.38	-2.20	-3.01

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-44.0	-80.84	-94.78	-129.59	24.32	-28.54

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-1829	-1756	-512
Right Reaction =	-886	-850	-248

D + Lr =	1373		
0.6D + W =	-1381	10 psf - W =	-1506
D + Lr + W =	-383		
D + 0.75(Lr + W) =	-131	(no increase in stress allowed for this combo)	

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-2042	-1960	-572
Right Reaction =	-871	-836	-244

D + Lr =	1373		
0.6D + W =	-1586	10 psf - W =	-1710
D + Lr + W =	-587		
D + 0.75(Lr + W) =	-284	(no increase in stress allowed for this combo)	

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	358	344	100
Right Reaction =	-299	-287	-84

D + Lr =	1373		
0.6D + W =	718	10 psf - W =	593
D + Lr + W =	1717		
D + 0.75(Lr + W) =	1444	(no increase in stress allowed for this combo)	

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

7-22

Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

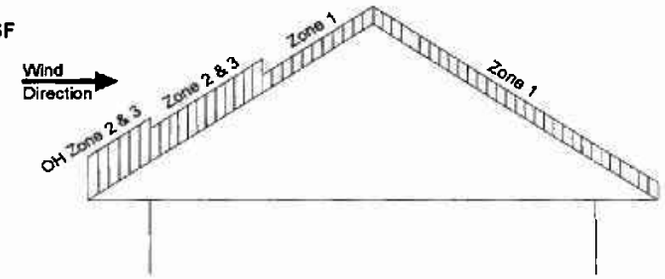
Project : Gainor Residence
Project No. : 05-618.00

• Hip Beam H-2

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.42 ft.	12.30 ft.	0.00 ft.	4.00 ft.	L^2/3 =	50 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	765.2 lbs		Right DL Reaction	606.8 lbs		
Left LL Reaction	918.2 lbs		Right LL Reaction	728.2 lbs		

≈ 1684



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents		Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3	Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-0.830	-1.608	0.36	-2.20	-2.86	-43.5	-77.04	-94.78	-123.07	23.23	-28.00

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta			
Left Reaction =	-2042	-1961	-572	D + Lr =	1683	
Right Reaction =	-1106	-1062	-310	0.6D + W =	-1502	10 psf - W = -1655
				D + Lr + W =	-277	
				D + 0.75(Lr + W) =	-17	(no increase in stress allowed for this combo)

Uplift Reactions For Zone 3:

		Cos theta	Sin theta			
Left Reaction =	-2212	-2124	-619	D + Lr =	1683	
Right Reaction =	-1097	-1053	-307	0.6D + W =	-1665	10 psf - W = -1818
				D + Lr + W =	-440	
				D + 0.75(Lr + W) =	-139	(no increase in stress allowed for this combo)

Downdraft Reactions All Zones:

		Cos theta	Sin theta			
Left Reaction =	396	380	111	D + Lr =	1683	
Right Reaction =	-381	-366	-107	0.6D + W =	839	10 psf - W = 686
				D + Lr + W =	2064	
				D + 0.75(Lr + W) =	1739	(no increase in stress allowed for this combo)

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

12-4

Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

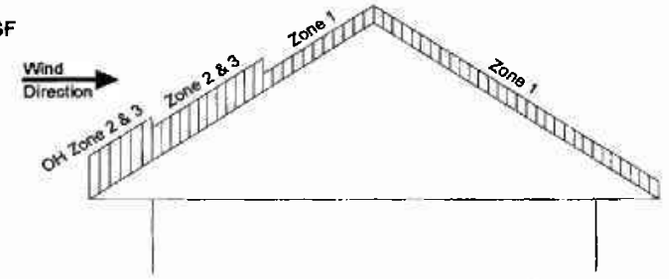
Project : Gainor Residence
Project No. : 05-618.00

›mmon Jacks J-1

	H	B	V	kd	kzt	G	qh	0.18*qh	a
	23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	6.67 ft.	0.00 ft.	2.00 ft.	L^2/3 =	15 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	220.5 lbs		Right DL Reaction	163.0 lbs		
Left LL Reaction	264.6 lbs		Right LL Reaction	195.6 lbs		

485.1 ≈ 486 #



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.883	-1.980	0.47	-2.20	-3.49

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-45.8	-93.07	-94.78	-150.55	27.82	-30.29

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-753	-723	-211
Right Reaction =	-378	-363	-106

D + Lr =	485	
0.6D + W =	-591	10 psf - W = -635
D + Lr + W =	-238	
D + 0.75(Lr + W) =	-123	(no increase in stress allowed for this combo)

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-873	-838	-244
Right Reaction =	-370	-355	-104

D + Lr =	485	
0.6D + W =	-706	10 psf - W = -750
D + Lr + W =	-353	
D + 0.75(Lr + W) =	-210	(no increase in stress allowed for this combo)

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	148	143	42
Right Reaction =	-109	-105	-31

D + Lr =	485	
0.6D + W =	275	10 psf - W = 231
D + Lr + W =	628	
D + 0.75(Lr + W) =	526	(no increase in stress allowed for this combo)

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

7-22-4

Siddiq Khan & Associates, Inc.
Roof Member Component and Cladding Reactions per ASCE 7-98
For Roof Slopes Ranging from 10 to 30 degrees

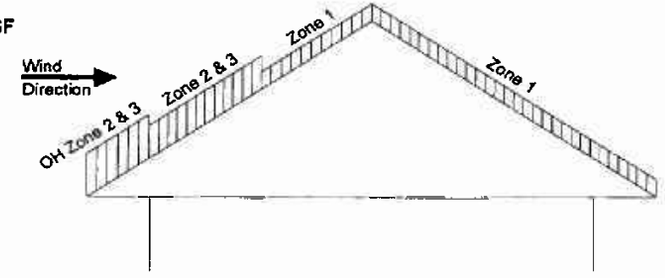
Project : Gainor Residence
Project No. : 05-618.00

›mmon Jacks J-2

H	B	V	kd	kzt	G	qh	0.18*qh	a
23	35	146	0.85	1.00	0.850	43.1	7.75	3.50 ft.

Truss Data:	Left OH	Main Span	Right OH	Truss Spac.		
Span	1.00 ft.	8.67 ft.	0.00 ft.	2.00 ft.	L^2/3 =	25 SF
Uniform DL	25.0 psf		Roof Slope	3.5 : 12	or 16.26 degrees	
Uniform LL	30.0 psf					
Concentrated DL	0.0 lbs	0.00 ft.	Dist From Left Support			
Concentrated LL	0.0 lbs					
Con. Wind Up	0.0 lbs					
Con. Wind Dn	0.0 lbs					
Left DL Reaction	269.6 lbs		Right DL Reaction	213.9 lbs		
Left LL Reaction	323.6 lbs		Right LL Reaction	256.6 lbs		

594 #



Roof Pressure Coefficents and Calculated Roof Pressures From 10 to 30 degrees, qz=qh*:

Negative Press. Coefficents		Pos. Press.	Overhang Coefficents	
Zone 1	Zone 2 & 3	All Zones	Zone 2	Zone 3
-0.860	-1.821	0.42	-2.20	-3.22

Negative Pressures		Overhang Pressures (no int)		Positive	Zone 1
Zone 1	Zone 2 & 3	Zone 2	Zone 3	All Zones	(-0.18)
-44.8	-86.20	-94.78	-138.77	25.85	-29.30

* Internal pressure 0.18qh included in above calculations to produce maximum loading

Uplift Reactions For Zone 2:

		Cos theta	Sin theta
Left Reaction =	-820	-787	-230
Right Reaction =	-436	-419	-122

D + Lr =	593
0.6D + W =	-626
D + Lr + W =	-194
D + 0.75(Lr + W) =	-78

10 psf - W = -680
(no increase in stress allowed for this combo)

Uplift Reactions For Zone 3:

		Cos theta	Sin theta
Left Reaction =	-913	-877	-256
Right Reaction =	-431	-414	-121

D + Lr =	593
0.6D + W =	-715
D + Lr + W =	-284
D + 0.75(Lr + W) =	-145

10 psf - W = -769
(no increase in stress allowed for this combo)

Downdraft Reactions All Zones:

		Cos theta	Sin theta
Left Reaction =	159	153	45
Right Reaction =	-137	-132	-38

D + Lr =	593
0.6D + W =	315
D + Lr + W =	746
D + 0.75(Lr + W) =	627

10 psf - W = 261
(no increase in stress allowed for this combo)

NOTE: Zone 2, 3, & OH pressures are applied on the left side.
Load Combos are for left reactions only.

T1-23

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JOB Gainor Res
SHEET NO. 1 OF 3
CALCULATED BY leo DATE 9/29/05
CHECKED BY DATE
SCALE

Anchor @ Tie Beam

Design @ Typical "T-H" Trusses Zone: 3/2 o.H.

Graph:

Given: $q_h = 51$ psf $w/K_z = 1.0$

$GC_{pi} = \pm 0.18$

Case 1: (Components & Cladding)

Wall Data

B' conc

$H_t = 9'-0"$

$l^2/3 = 27 ft^2 \approx 30 ft^2$

$P = q_h (GC_p - GC_{pi})$

Typical Wall/Roof

GC_p

Zone 4 & 5 (+) $+0.916$

Zone 4(-) $= -1.016$

Anchor Info

T.C. From Wall $= 4'-6" \times 2'-0" = 9'$

$\frac{W+H+F}{2}$ spacing

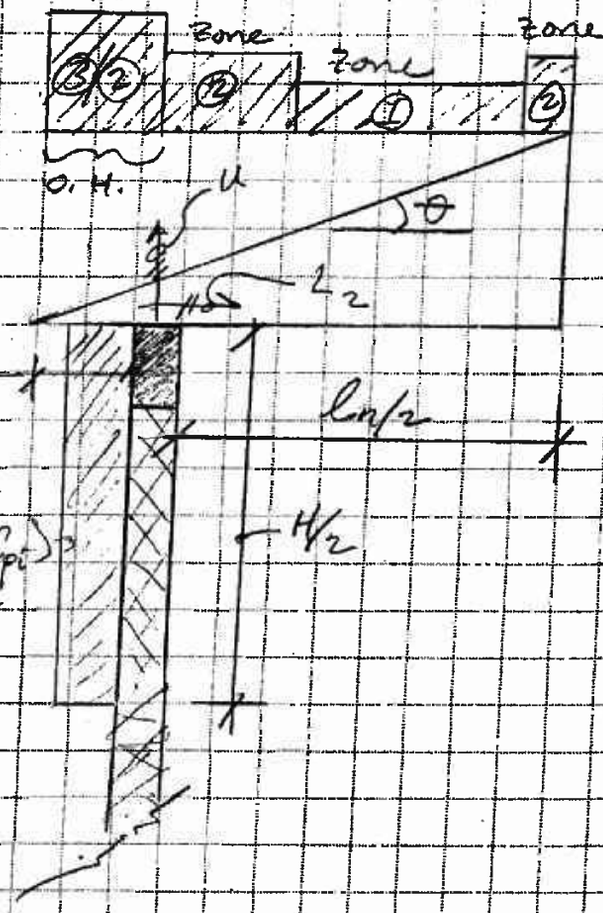
Truss Info

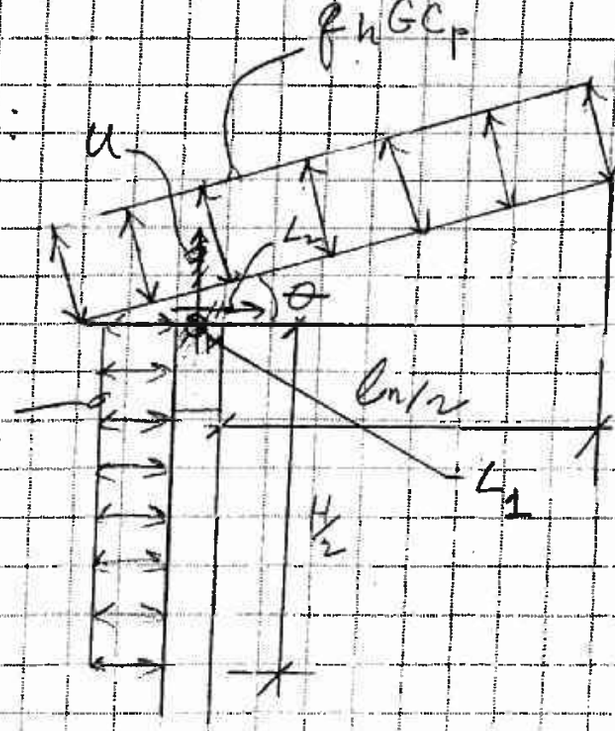
Truss Component & Cladding Calcs

Max Uplift $= 1.32^K$ @ T5

Case 2: (MWFRS)

P Windward or Leeward





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SHEET NO. 2 OF 3
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Anchor @ Tie Beam

Design

Calcs

For Case 1: C&C ; $GC_{pi} = \pm 0.18$

Zone 4 Controls $\Rightarrow GC_p = -1.02$

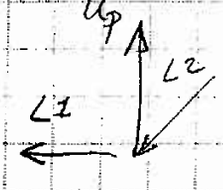
$\therefore P_{leeward} = q_h (GC_p - GC_{pi})(0.85)$

$= (51 psf)(-1.02 - 0.18)(0.85)$

$= 52.02 psf$ (max worst case) $\therefore$

Lateral Force $L_2 = 52.02 psf \times 9 ft^2 = 468.2^{\#}$ say $470^{\#}$ ✓

Uplift @ TS $U_{max} = 1413^{\#}$ ✓



Try Anchor "16" = NVHTA-16H

All. up $= 2338^{\#}$; All. $L_1 = 1181^{\#}$; All. $L_2 = 1631^{\#}$

$\therefore \frac{U}{A.U.} + \frac{L_2}{A.L_2} = \frac{1413^{\#}}{2338^{\#}} + \frac{470^{\#}}{1631^{\#}} = 0.893 < 1.0 \therefore$ okay

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SHEET NO. 3 OF 3
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Anchor @ Tie Beam Design

Calcs (Cont'd)

Case 2: MWFRS $\Rightarrow q_h = 44.0$ psf w/ $K_z = 1.85$ @ M.R.Ht = 23'

Wall Pressure. {Note: Max. Pressures on Wall are considered individually}

- $P_{L.W.} = q G_h C_p - q_h (GC_{pi})$; $G = 0.85$; $C_p = -0.5$; $GC_p = +0.18$

$P_{L.W.} = (44)(0.85)(-0.5) - (44)(0.18) = -26.67$ psf

- $P_{S.S.} = q G_h C_p - q_h (GC_{pi})$; $G = 0.85$; $C_p = 0.7$; $GC_p = -0.18$

$P_{S.S.} = (44)(0.85)(0.7) - 44(-0.18) = 34.1$ psf ← Side suction Controls

say 35 psf

$\therefore h_z = (35 \text{ psf})(9 \text{ ft}^2) = 315\#$

$L_1 = 153 \#/\text{ft}\ (2 \text{ ft}) = 306\#$

wall (D) ← see Diaphragm Calcs

- Uplift $= q_h G_h C_p - q_h (GC_{pi})$; $\frac{h}{L} = 0.67 \Rightarrow C_p = -1.04$ ← via Interpolation between 0 to h/2

$= [(44)(0.85)(-1.04) - (44)(0.18)]$ psf

$= -46.82$ psf say 47 psf

$U_{max(MWFRS)} = (47 \text{ psf} - 10 \text{ psf})(2 \text{ ft})\left(\frac{35'}{2}\right) = 1295\#$ < Case 1

(5 psf For Max. DL; $\frac{35'}{2}$ ← largest width ÷ 2)

Check Unity Equation for NVHTA-16H "Nu-Vue Anchor"

$\frac{U_{max}}{All.\,U} + \frac{L_{1\,max}}{All.\,L_1} + \frac{L_{2\,max}}{All.\,L_2} \le 1.0 \Rightarrow \frac{1295\#}{2338\#} + \frac{306\#}{1181\#} + \frac{315\#}{1631\#} = 1.0$ ∴ ok

Conclusion: use NVHTA-16H Nu-Vue Anchor "16"

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JOB Gainor Res
SHEET NO. 1 OF 2
CALCULATED BY LEO DATE 9/29/05
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SCALE @ Hips

Anchor design for Typical "GT-#" Girder Trusses
(Note: Excludes "GT-3" & "GT-5")

Given:

q_h = 44 psf w/ K_z = 0.85
GC_{pi} = ±0.18

Wall Data:

8" CMU + 7'-0" Ht.
$L/3$ = 27' ± 30 ft²

∴ GC_p

Zone 4 = +1.02 (controls)

Anchor Info

Spc. = 2'-0"

Truss Info

See Truss Component & Cladding Calcs (MWFRS)

Wind Pressure Info

$P_{L.W.(C\&C)}$ = 52.02 psf

Case 1

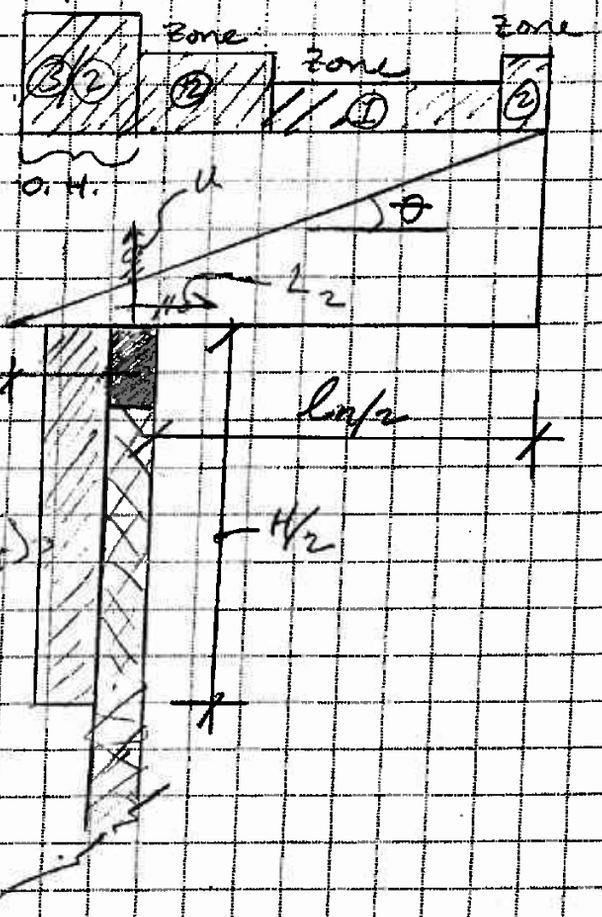
$P_{L.W.(MWFS)}$ = 35 psf (max)
$P_{W.W.(MWFS)}$ = 38 psf (max)
$P_{UP\ (MWFS)}$ = 47 psf max

Graph:

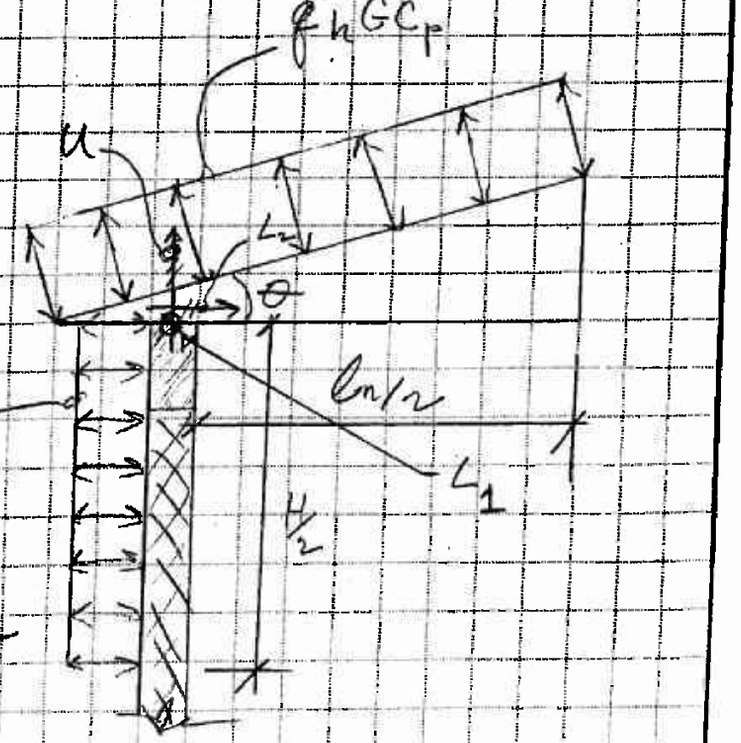
Case 1: (Components & Cladding)

$P = q_h(GC_p - GC_{pi})$

Typical Wall/Roof



Case 2:



$P_{Windward}$ or $P_{Leeward}$ } Case 2

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JOB Gainor Res
SHEET NO. 2 OF 2
CALCULATED BY Leo DATE 9/29/05
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SCALE

Anchor Design for Typical "GT-#" Girder Trusses

Calcs:

Case 1: Components & Cladding

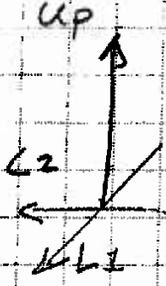
$U_{max} = 3630^{\#}$ ← from "GT-4"

$L_2 = (52.02\,psf)(4.5\,ft)(2') = 469^{\#}$

Try Anchor HD-1 & 2B

All. Up = 5858# (in 3000 psi Conc.)

All. $L_1 = 887^{\#}$; All. $L_2 = 1335^{\#}$



$\therefore \frac{3630^{\#}}{5858^{\#}} + \frac{469^{\#}}{1335^{\#}} = 0.97 < 1 \therefore$ ok

Case 2

$L_2 = P_{ss}(Tr.6) = 35\,psf(9') = 315^{\#}$ from wall ②

$L_1 = \text{Diaphragm Shear}(Tr.4) = 153^{\#}/ft\,(2) = 306^{\#}$

$U_{max} = (57\,psf)(4.5\,ft)\left(\frac{35'}{2}\right) = 2914^{\#}$ ← 47 psf − 10 psf

$\therefore \frac{2914^{\#}}{5858^{\#}} + \frac{306^{\#}}{887^{\#}} + \frac{315^{\#}}{1335^{\#}} = 1.08 \approx 1 \therefore$ ok

8% overstress

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Anchor design for GT-5

Given:

q_h = 44 psf w/ K_z = 0.85

GC_{pi} = ±0.18

Wall Data

8" cmu × 9'-0" Ht.

$L_{/B}$ = 27'-0" × 30 ft²

∴ GC_p

Zone 4 = +1.02 (controls)

Anchor Info

Spc. = 2'-0"

Truss Info

See Truss Component & Cladding Calcs (MWFRS)

Wind Pressure Info

$P_{L.W.(C\&C)}$ = 52.02 psf — Case 1

$P_{wind\ pressure}$ or $P_{leeward}$

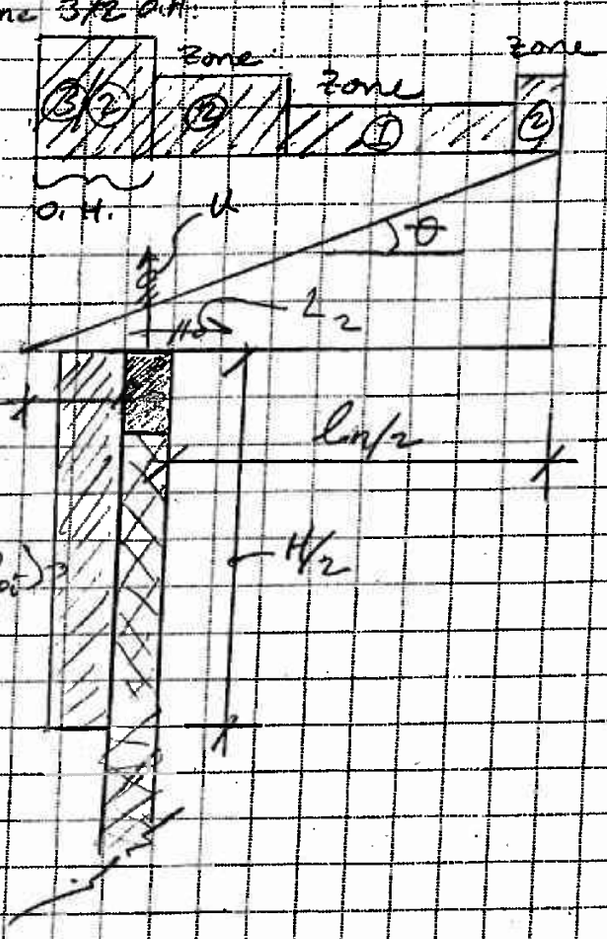
$P_{s.(MWFS)}$ = 35 psf (max)

$P_{w.w(MWFS)}$ = 38 psf (max) } Case 2

$P_{up\ (MWFS)}$ = 47 psf max }

Graph:

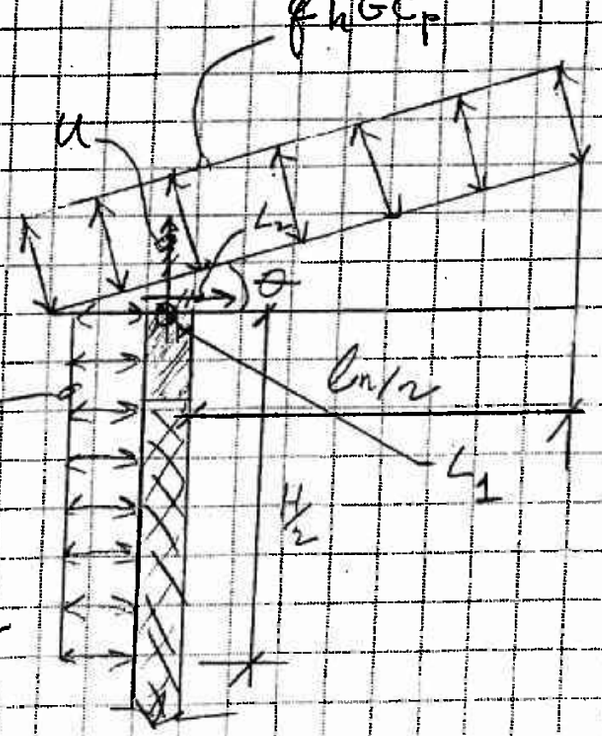
Case 1: (Components & Cladding)



$P = q_h (GC_p - GC_{pi})$

Typical Wall/Roof

Case 2:



7-29

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SCALE

Anchor Design for GT-5

Calcs

Case 1: Comp & Cladd.

$L_2 = (52.02\ psf)(9\ ft^2) = 469^{\#}$

$U_p = -2455^{\#}$ ← @ GT-5 C&C < $U_{max\ @\ GT-4} = 3630^{\#}$

∴ use HD-1 & 2B {Refer to Typical "GT-#" Design Calcs}

Case 2: MWFRS

$L_1 = 153^{\#}/ft\ (2\ ft) = 306^{\#}$

$L_2 = (35\ psf)(9\ ft^2) = 315^{\#}$

$U_p = (47\ psf - 10\ psf)(3.75\ ft)\left(\frac{31.67\ ft}{2}\right) \cong 2200^{\#} < U_{max\ (MWF\ @\ GT-4)} = 2914^{\#}$

∴ use HD-1 & 2B, okay by inspection

Conclusion: Use same as Typical "GT-#" for consistency

7-21

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JOB Gainor Res
SHEET NO. 1 OF 2
CALCULATED BY leo DATE 9/29/05
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SCALE

Anchor design for Typical Jacks "J-1 & J-2"

Graph:

Given:

q_h = 44 psf w/ K_d = 0.85

GC_{pi} = ±0.18

Wall Data

8" cmu × 9'-0" Ht.

L/B = 27'/30' ≈ 30 ft²

∴ GC_p

Zone 4 = -1.02 (controls)

Anchor Info

Spc. = 2'-0"

Truss Info

See Truss Component & Cladding Calcs (MWFRS)

Wind Pressure Info

$P_{C\&C}$ = 52.02 psf (Case 1)

$P_{M.W.F.R.S.}$ = 35 psf (max)

$P_{W.W(MWFRS)}$ = 38 psf (max)

$P_{UP(MWFRS)}$ = 47 psf max

Case 1: (Components & Cladding)

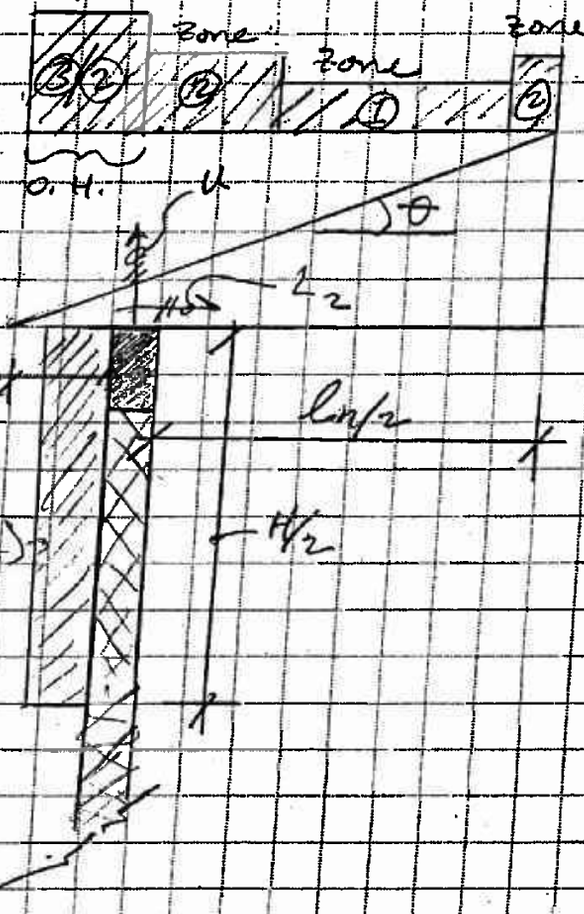
$P = q_h(GC_p - GC_{pi})$

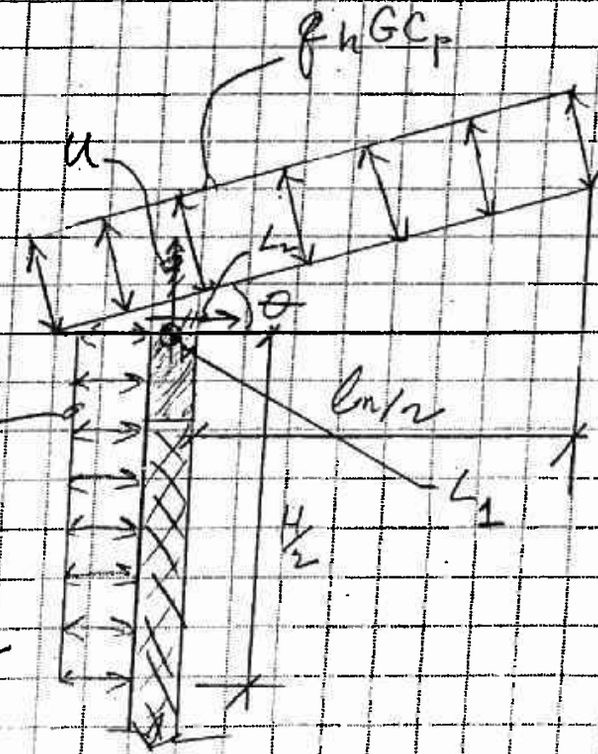
Typical Wall/Roof

Case 2:

$P_{Windward}$ or $P_{leeward}$

Case 2





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JOB Gainor Res
SHEET NO. 2 OF 2
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SCALE

Anchor Design for Typical Jacks J-1 & J-2"

Calcs

Case 1: Comp. & Cladd.

$L_2 = 469^{\#}$ ← See Typical Truss "T-#" Calcs.

$U_{p(max)} = 762^{\#}$ @ J-2 < Typical Truss "T-#" ∴ does not control

Case 2: MWFRS

From Typical Truss "T-#" Calcs

$L_1 = (134^{\#}/ft)(2') = 268^{\#} < 306^{\#}$

$L_2 = 315^{\#}$

$U_p = (37\,psf)(24)\left(\frac{8}{2}'\right) = 296^{\#}$

} less than Typical "T-#" ∴ is ∴ does not control

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Anchor design for Typical Hip Jacks "H-1 & H-2"

Given:

$q_h = 44$ psf w/ $K_d = 0.85$

$GC_{pi} = \pm 0.18$

Wall Data

8" cmu x 9'-0" Ht.

$L/3 = 27'' \approx 30$ f.a.

GCp

1) Zone 4 = +1.02 (controls)

Anchor Info

Spc. = 2'-0"

Truss Info

Skew Truss Component & Cladding Calcs (MWFRS)

Wind Pressure Info

P_{LW} (c&c) = 52.02 psf

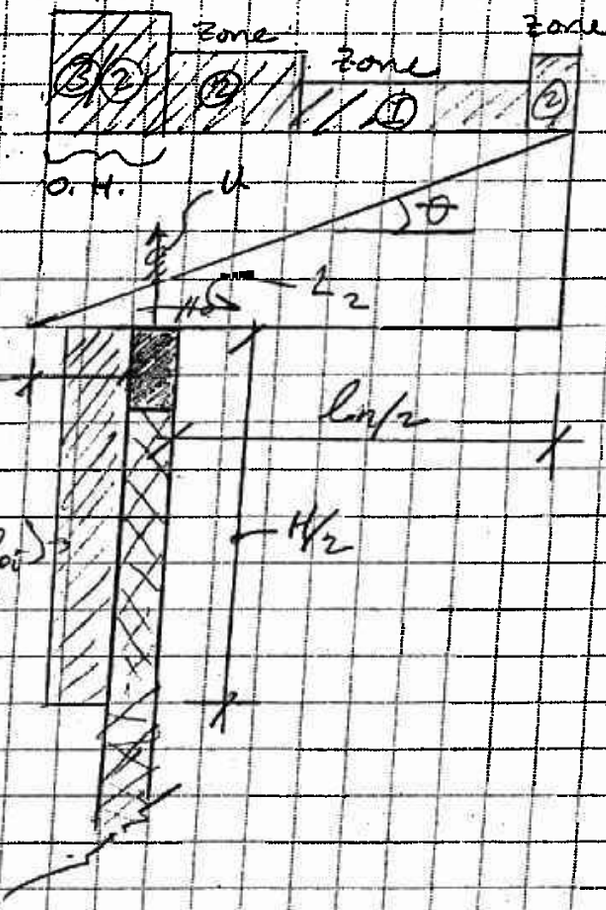
Case 1

$P_{L.W.(MWFS)} \approx 35$ psf (max)

$P_{W.W(MWFS)} = 38$ psf (max)

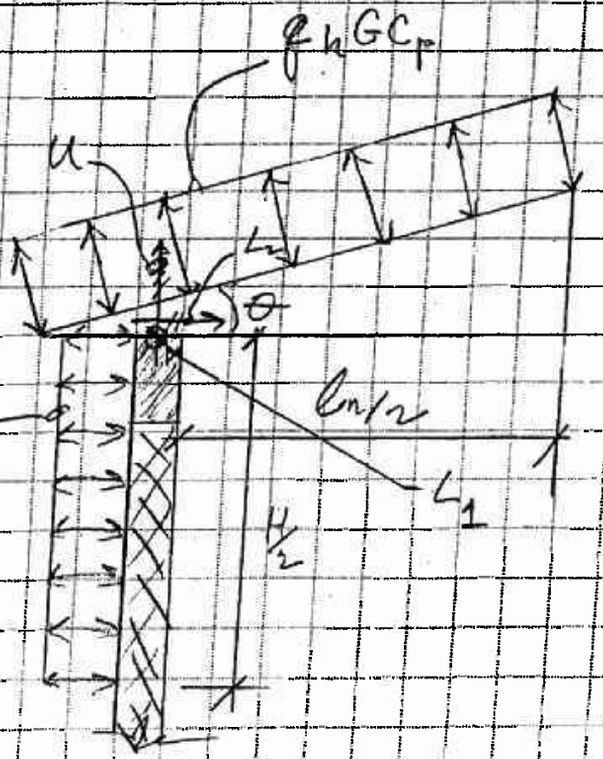
$P_{UP(MWFS)} = 47$ psf max

Graph:



Case 1: (Components & Cladding)

Case 2:



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Anchor Design for Typical Hip Jacks "H-1" & "H-2"

Calcs:

Truss Span = 8'-0"

Spacing/Trib = 4'-0"

Case 1: Comp & Cladd.

$U_{p(max)}$ = 1818# ← from H2 (Comp. & Cladd. Calcs)

Try Anchor "16" = MVHTA-16H

$\frac{1818\#}{2358\#} \leq 1.0$ ∴ okay

Case 2: MWFRS

$U_{p(max)} = (47-10)\,psf(32\,ft^2) = 1184\# <$ Case 1 ∴ does not control

Conclusion: Use same as Typical "T-#" Truss Anchor

MVHTA-16H Mi-Vue Anchor

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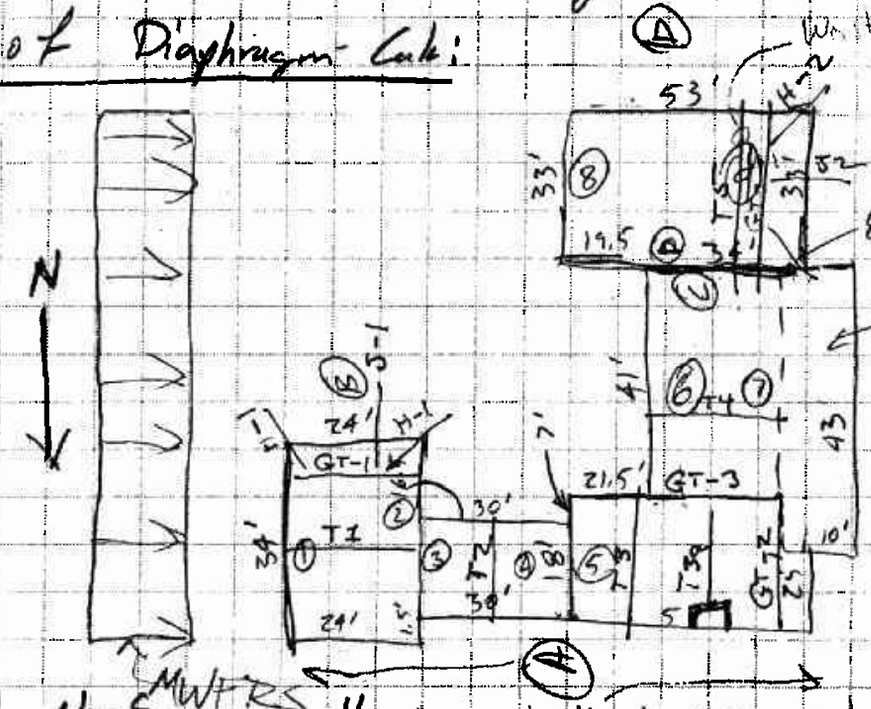
Roof Diaphragm Calc:



Wind N-S MWFRS ll to wall being check

① 12(194.4)/34 = 68.6 plf O.K.

② 12(194.4)/34 = 68.6 plf O.K.

③ 15(194.4)/18 = 162 plf O.K.

④ 15(194.4)/18 = 162 plf O.K.

⑤ 11(194.4)/25 = 85.5 plf O.K.

⑥ 56(194.4)/41 = 265 plf Blocking Req'd

⑦ 29(194.4)/43 = 131 plf O.K.

⑧ 19.5(194.4)/33 = 115 plf O.K.

⑨ 8(194.4)/33 = 47.1 plf O.K.

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Roof Diaphragm Calc Cont.

Wind E-W:

(A) $48(194.4)/111 = 84.1$ plf OK.

(B) $16.5(194.4)/24 = 133.7$ O.K.

(C) $\left(\frac{43+15}{2}\right)(194.4)/42 = 157.4$ plf. O.K.

(D) $33(194.4)/42 = 152.7$ plf. O.K.

House Construction From Late 20's
Observed Roof Sheathing 1x6 T+G Planks.
Assume 2-10d Nailing per Rafter (2" Nail)
Rafter Framing observed @ 16" O.C. Dade C. Pine
G = 0.55
From NDS 01' w/ $t_s = 3/4$" Allow in 10d = 101

$C_d = 1.33$ $C_t = 0.8$ $Z' = 101 * 1.33 \times .8 = 107.5\ \# \times 2 = 215\ \#$

Max Shear Capacity In Nails/ft = 430 #/ft/16"
$430 \times 14/16 = 323$ #/ft.

Max Shear Based on 90 psi Allow = $90(1.33)(.8) = 96$ psi

$96 = 3/2 \frac{V}{12(.75)}$ $V = 576$ #/ft ∴ 323 #/ft Controls

All Diaphragm Shear is Below 323 #/ft ∴ No Add'l Strengthening is req'd.

Withdrawl Load = 40 #/in $\frac{W'}{SF} = 40 \times .8 \times 1.33 \times 4 \times 12/16 = 128$ #/SF.
Penetration = $1\frac{1}{2}$" (128) = 192

$\frac{x}{323} + \frac{42.6}{192} = 1.0$ max diaphragm Shear w/o Blocking = 251 #/ft

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Gainor Residence
5800 North Bay Road
City of Miami Beach, Florida

2nd Floor and Roof Diaphragm Shear Calculations

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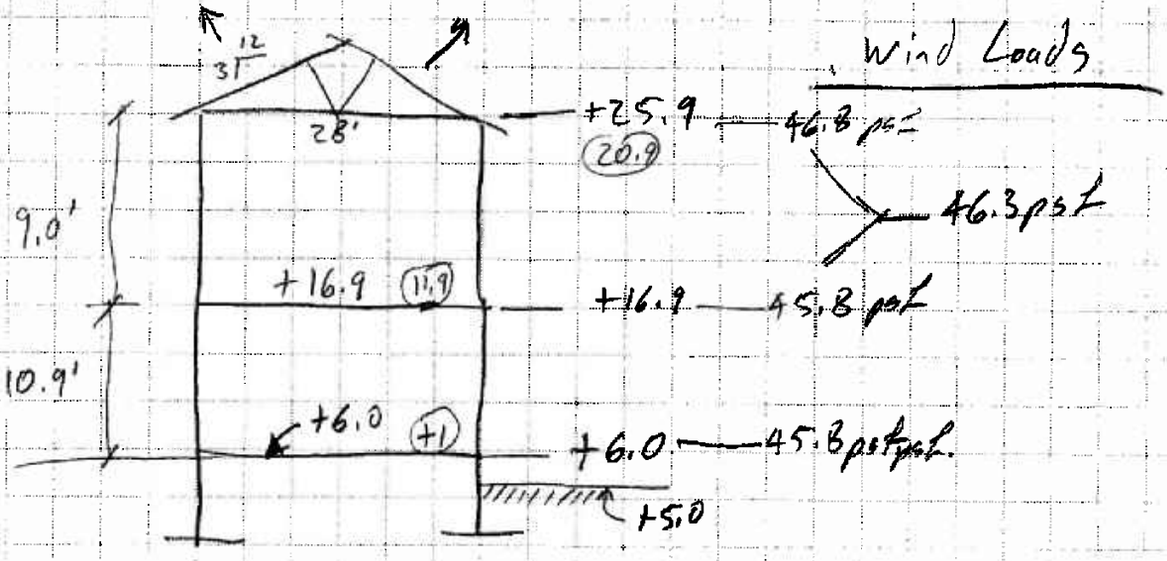
Diaphragm Shear Loads:



Wind Loads

$h/\ell = \frac{19.9}{28} = 0.71$ ∴ Roof uplift Pressures $\theta = 14.04$

	Windward	Leeward
Interpolate Between 0.5 & 1.0 →	−31.9	−27.9

$$\text{Roof Hypotenuse} = \frac{14}{\cos\theta} = 14.43$$

From Roof: $14.43(\sin\theta)(-31.9 - -27.9) = -14.00$

From Wall: $9.0/2(46.3) = 208.4$

Roof Diaphragm Shear Load = 208.4 − 14.0 = 194.4 plf

Second Floor Diaphragm: $(4.5)(45.8) + (5.45)(45.8) = 456$ plf.

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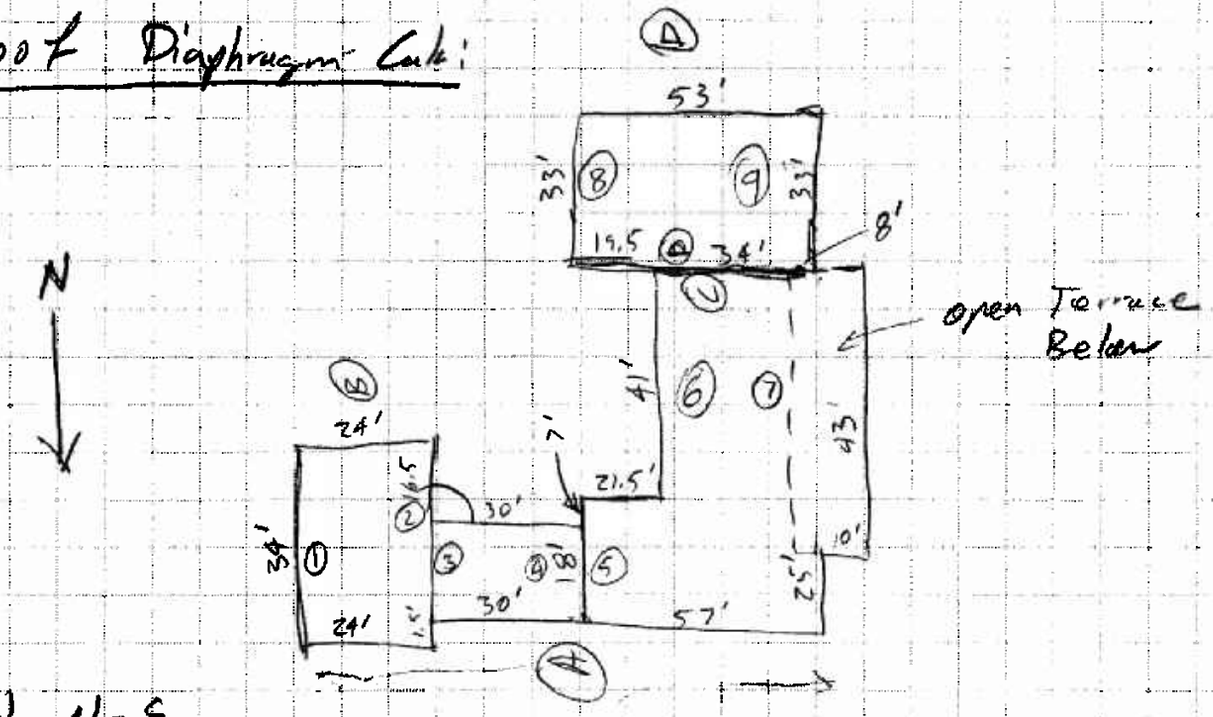
Roof Diaphragm Calc:



Wind N-S

① 12(194.4)/34 = 68.6 plf O.K.

② 12(194.4)/34 = 68.6 plf O.K.

③ 15(194.4)/18 = 162 plf O.K.

④ 15(194.4)/18 = 162 plf O.K.

⑤ 11(194.4)/25 = 85.5 plf O.K.

⑥ 56(194.4)/41 = 265 plf Blocking Req'd

⑦ 29(194.4)/43 = 131 plf O.K.

⑧ 19.5(194.4)/33 = 115 plf O.K.

⑨ 8(194.4)/33 = 47.1 plf O.K.

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Roof Diaphragm Calc Cont.

Wind E-W:

(A) $48(194.4)/111 = 84.1$ plf OK.

(B) $16.5(194.4)/24 = 133.7$ O.K.

(C) $\left(\frac{43+85}{2}\right)(194.4)/42 = 157.4$ plf. O.K.

(D) $33(194.4)/42 = 152.7$ plf. O.K.

House Construction From Late 20's (Roof Being Remove & Replaced ∴ Not Applicable)
Observed Roof Sheathing 1x6 T+G Planks.
Assume 2-10d Nailing per Rafter (2" Nail)
Rafter Framing observed @ 16" O.C. Dade C. Pine G = 0.55
From NDS 01' w/ t_s = 3/4" Allow in 10d = 101

$C_d = 1.33$ $C_t = 0.8$ $Z' = 101 * 1.33 \times .8 = 107.5\ \# \times 2 = 215\ \#$

Max Shear Capacity In Nails/ft = 430 #/ft/16"
$430 \times 12/16 = 323$ #/ft.

Max Shear Based on 90 psi Allow = $90(1.33)(.8) = 96$ psi

$96 = 3/2 \frac{V}{12(.75)}$ $V = 76$ #/ft ∴ 323 #/ft (controls)

All Diaphragm Shear is Below 323 #/ft. ∴ No Add'l Strengthening is req'd.

Withdrawl Load = 40 #/in $\frac{W'}{SF} = 40 * .8 \times 1.33 \times 4 \times 12/16 = 128$ #/SF
Penetration = $1\frac{1}{2}$" (128) = 192

$\frac{x}{323} + \frac{42.6}{192} = 1.0$ max diaphragm Shear w/o Blocking = 251 #/ft

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Roof:

Diaphragm Blocking Req'd:

Wall (6): $\frac{(265\,plf - 251)(41)}{194.4} = 2.95'$ Blocking Req'd.

Use 4' Blocking

Note To Plan Reviewer: Existg T.+k. Diaphragm shear Capacity was Calculated Prior to Decesion of Roof Replacement. However, it is kept Since it becomes the Bases For 2nd Floor Diaphragm Check.

Allow Second Floor Diaphragm Load:

No uplift ; Allow V = 323 in 1x6 T&G Deck.

Table 11N COMMON WIRE, BOX, or SINKER NAILS: Design Values (Z) for Single Shear (two member) Connections[1,2,3,4]

for Sawn Lumber or SCL with both members of identical specific gravity

Side Member Thickness	Nail Diameter	Common Wire Nail	Box Nail	Sinker Nail	G=0.67 Red Oak	G=0.55 Mixed Maple Southern Pine	G=0.5 Douglas Fir-Larch	G=0.49 Douglas Fir-Larch (N)	G=0.46 Douglas Fir(S) Hem-Fir(N)	G=0.43 Hem-Fir	G=0.42 Spruce-Pine-Fir	G=0.37 Redwood (open grain)	G=0.36 Eastern Softwoods Spruce-Pine-Fir (S) Western Cedars Western Woods	G=0.35 Northern Species
t_s	D	Pennyweight												
In.	in.				lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
3/4	0.099		6d	7d	73	61	55	54	51	48	47	39	38	36
	0.113	6d	8d	8d	94	79	72	71	65	58	57	47	46	44
	0.120			10d	107	89	80	77	71	64	62	52	50	48
	0.128		10d		121	101	91	87	80	72	70	57	56	54
	0.131	8d			127	106	95	90	82	73	71	59	58	56
	0.135		16d	12d	135	108	94	91	84	76	74	63	61	58
	0.148	10d	20d	16d	154	121	105	102	94	85	83	70	69	66
	0.162	16d	40d		183	138	121	117	108	99	96	82	80	77
	0.177			20d	200	153	134	130	120	111	108	92	90	87
	0.192	20d		30d	206	157	138	134	123	114	111	96	93	90
	0.207	30d		40d	216	166	147	143	132	122	119	103	101	97
	0.225	40d			229	178	158	154	144	132	129	112	110	106
	0.244	50d		60d	234	182	162	158	147	136	132	115	113	109
1	0.099		6d	7d	73	61	55	54	51	48	47	42	41	40
	0.113	6d	8d	8d	94	79	72	71	67	63	61	55	54	52
	0.120			10d	107	89	81	80	76	71	69	62	60	58
	0.128		10d		121	101	93	91	86	80	79	70	69	67
	0.131	8d			127	106	97	95	90	84	82	68	86	83
	0.135		16d	12d	135	113	103	101	96	89	88	71	69	66
	0.148	10d	20d	16d	154	128	118	115	109	99	96	80	77	74
	0.162	16d	40d		184	154	141	137	129	119	115	96	93	89
	0.177			20d	213	178	163	159	150	138	135	112	108	103
	0.192	20d		30d	222	183	169	166	156	142	139	116	113	108
	0.207	30d		40d	243	192	167	162	149	135	131	111	109	104
	0.225	40d			268	202	177	171	159	144	140	120	117	112
	0.244	50d		60d	274	207	181	175	162	148	143	123	120	115
1 1/4	0.099		6d	7d	73	61	55	54	51	48	47	42	41	40
	0.113	6d	8d	8d	94	79	72	71	67	63	61	55	54	52
	0.120			10d	107	89	81	80	76	71	69	62	60	58
	0.128		10d		121	101	93	91	86	80	79	70	69	67
	0.131	8d			127	106	97	95	90	84	82	73	72	70
	0.135		16d	12d	135	113	103	101	96	89	88	78	76	74
	0.148	10d	20d	16d	154	128	118	115	109	102	100	89	87	84
	0.162	16d	40d		184	154	141	138	131	122	120	106	104	101
	0.177			20d	213	178	163	159	151	141	138	123	121	117
	0.192	20d		30d	222	185	170	166	157	145	140	116	113	108
	0.207	30d		40d	243	203	186	182	169	152	147	123	119	114
	0.225	40d			268	224	200	193	177	160	155	130	127	121
	0.244	50d		60d	276	230	204	197	181	163	158	133	129	123
1 1/2	0.099			7d	73	61	55	54	51	48	47	42	41	40
	0.113		8d	8d	94	79	72	71	67	63	61	55	54	52
	0.120			10d	107	89	81	80	76	71	69	62	60	59
	0.128		10d		121	101	93	91	86	80	79	70	69	67
	0.131	8d			127	106	97	95	90	84	82	73	72	70
	0.135		16d	12d	135	113	103	101	96	89	88	78	76	74
	0.148	10d	20d	16d	154	128	118	115	109	102	100	89	87	84
	0.162	16d	40d		184	154	141	138	131	122	120	106	104	101
	0.177			20d	213	178	163	159	151	141	138	123	121	117
	0.192	20d		30d	222	185	170	166	157	147	144	128	126	120
	0.207	30d		40d	243	203	186	182	172	161	158	135	131	125
	0.225	40d			268	224	205	201	190	176	172	144	138	132
	0.244	50d		60d	276	230	211	206	196	181	179	146	141	135
1 3/4	0.113		8d		94	79	72	71	67	63	61	55	54	52
	0.120			10d	107	89	81	80	76	71	69	62	60	59
	0.128		10d		121	101	93	91	86	80	79	70	69	67
	0.135		16d	12d	135	113	103	101	96	89	88	78	76	74
	0.148	10d	20d	16d	154	128	118	115	109	102	100	89	87	84
	0.162	16d	40d		184	154	141	138	131	122	120	106	104	101
	0.177			20d	213	178	163	159	151	141	138	123	121	117
	0.192	20d		30d	222	185	170	166	157	147	144	128	126	122
	0.207	30d		40d	243	203	186	182	172	161	158	140	137	133
	0.225	40d			268	224	205	201	190	176	174	155	151	144
	0.244	50d		60d	276	230	211	206	196	183	179	159	154	147

1. Tabulated lateral design values (Z) shall be multiplied by all applicable adjustment factors (see Table 10.3.1).
2. Tabulated lateral design values (Z) are for common wire, box and sinker nails (see Appendix L) inserted in side grain with nail axis perpendicular to wood fibers; minimum nail penetration, p, into the main member equal to 10D; and nail bending yield strengths (F_{yb}):

   F_{yb} = 100,000 psi for 0.099"≤D≤0.142" F_{yb} = 90,000 psi for 0.142"<D≤0.177" F_{yb} = 80,000 psi for 0.177"<D≤0.236" F_{yb} = 70,000 psi for 0.236"<D≤0.273"
3. When 6D≤p<10D, tabulated lateral design values (Z) shall be multiplied by p/10D.
4. Nail lenth is insufficient to provide 10D penetration. Tabulated lateral design values (Z) shall be adjusted per footnote 3.

Table 11.2C Nail and Spike Withdrawal Design Values (W)[1]

Tabulated withdrawal design values (W) are in pounds per inch of penetration into side grain of main member (see Appendix L).

Specific Gravity	Common Wire Nails, Box Nails, and Common Wire Spikes Diameter, D															Threaded Nails Wire Diameter, D				
G	0.099"	0.113"	0.128"	0.131"	0.135"	0.148"	0.162"	0.192"	0.207"	0.225"	0.244"	0.263"	0.283"	0.312"	0.375"	0.120"	0.135"	0.148"	0.177"	0.207"
0.73	62	71	80	82	85	93	102	121	130	141	153	165	178	196	236	82	93	102	121	141
0.71	58	66	75	77	79	87	95	113	121	132	143	154	166	183	220	77	87	95	113	132
0.68	52	59	67	69	71	78	85	101	109	118	128	138	149	164	197	69	78	85	101	118
0.67	50	57	65	66	68	75	82	97	105	114	124	133	144	158	190	66	75	82	97	114
0.58	35	40	45	46	48	52	57	68	73	80	86	93	100	110	133	46	52	57	68	80
0.55	31	35	40	41	42	46	50	59	64	70	76	81	88	97	116	41	46	50	59	70
0.51	25	29	33	34	35	38	42	49	53	58	63	67	73	80	96	34	38	42	49	58
0.50	24	28	31	32	33	36	40	47	50	55	60	64	69	76	91	32	36	40	47	55
0.49	23	26	30	30	31	34	38	45	48	52	57	61	66	72	87	30	34	38	45	52
0.47	21	24	27	27	28	31	34	40	43	47	51	55	59	65	78	27	31	34	40	47
0.46	20	22	25	26	27	29	32	38	41	45	48	52	56	62	74	26	29	32	38	45
0.44	18	20	23	23	24	26	29	34	37	40	43	47	50	55	66	23	26	29	34	40
0.43	17	19	21	22	23	25	27	32	35	38	41	44	47	52	63	22	25	27	32	38
0.42	16	18	20	21	21	23	26	30	33	35	38	41	45	49	59	21	23	26	30	35
0.41	15	17	19	19	20	22	24	29	31	33	36	39	42	46	56	19	22	24	29	33
0.40	14	16	18	18	19	21	23	27	29	31	34	37	40	44	52	18	21	23	27	31
0.39	13	15	17	17	18	19	21	25	27	29	32	34	37	41	49	17	19	21	25	29
0.38	12	14	16	16	17	18	20	24	25	28	30	32	35	38	46	16	18	20	24	28
0.37	11	13	15	15	16	17	19	22	24	26	28	30	33	36	43	15	17	19	22	26
0.36	11	12	14	14	14	16	17	21	22	24	26	28	30	33	40	14	16	17	21	24
0.35	10	11	13	13	14	15	16	19	21	23	24	26	28	31	38	13	15	16	19	23
0.31	7	8	9	10	10	11	12	14	15	17	18	19	21	23	28	10	11	12	14	17

1. Tabulated withdrawal design values (W) for nail or spike connections shall be multiplied by all applicable adjustment factors (see Table 10.3.1).

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2nd Floor Wood Diaphragm Calcs:

Diaphragm Wall Load = 45.08 x 11' = 496 plf.

Wind N-S.

① = $\frac{12(496)}{134}$ = 175 plf O.K.

② $\frac{12(496)}{134}$ = 175 plf O.K.

③ $\frac{30(496)}{(2)18}$ = 413 plf > Blocks = $\frac{(413-323)(18)}{496}$ = 3.3'

use 4'

④ $\frac{30(496)}{(2)18}$ = 413 plf

⑤ $\frac{22'}{2}(496)/25$ = 218 plf O.K.

⑥ 24(496)/42 = 283 plf O.K.

⑦ 27 (496)/42 = 319 plf O.K.

⑧ 18 (496)/32.5 = 275 plf O.K.

⑨ 8 (496)/32.5 = 122 O.K.

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Second Floor Diaphragm Cont.

Wind E-W:

(A) $48(496)/111 = 215$ plf ; OK

(B) $16.5(496)/24 = 341$ plf $\quad \frac{(341-323)(16.5)}{496} = 0.6'$ use 2'

(C) $29(496)/27' = 533$ plf $\quad \frac{(533-323)(29)}{496} = 12.3' \rightarrow 14'$

(D) $21'(496)/26' = 400$ $\quad \frac{(400-323)(21)}{496} = 3.3'$ 4'- Blocking.

(E) $\frac{32.5}{2}(496)/53 = 152$; OK.

SIDDIQ KHAN & ASSOCIATES, INC.
Consulting Engineers And Planners
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(305) 662-2301

JOB Gainor Residence.
SHEET NO. 8-10 OF
CALCULATED BY TAK DATE 9/26/05
CHECKED BY DATE
SCALE

Wood Blocking Fix Diaphragm Shear Capacity:

Use Ledger Calc Program To Find Capacity since not in NDS Tables

Max Capacity = 194# x 1.33 = 258#/Anchor

If Provided Between Each Joist Capacity:

$2 \times 258 \times 12/16 = 387\ \#/ft$

If Provided Between Every other Joist Capacity:

$2 \times 258 \times 12/32 = 194\ \#/ft$

∴ Provide Between Every Joist.

Req'd Nailing From Top to active Diaphragm shear

Use 10d Nails; Capacity NDS 11N

$Z = 89\#$ $Z' = 89 \times 1.33 = 118\#$

$387/118 = 3.28$ ∴ Use 4-10d Nails

min. Penetration = 10d = .120" x 10 = 1.2"

Use $1\frac{1}{2}" + \frac{3}{4} = 2\frac{1}{4}"$

Req'd Nail Length = $2\frac{1}{4}"$ min.

Siddiq Khan & Associates, Inc.

Wood Ledger Design Per FBC Load Combinations and NDS equations:

Project Name : Gainor Residence

Project Number: 05-618.00

Ledger designation: 1/4" Dia Tapcon Capacity in 2x4

Dead Load =	**0**	**plf**	**Cd Dead load = 0.9**
Live Load =	**0**	**plf**	**Cd Live Load = 1.0**
Wind Downdraft =	**0**	**plf**	**Cd Wind = 1.33**
Wind Uplift =	**0**	**plf**	
Diaphragm Shear =	**258**	**plf**	
Water Ponding =	**0**	**plf**	

Note: negative sign denotes upwards load direction

		Vert.	Horiz.	Resultant	Angle to Grain
Case 1	D	0.0	n/a	0.0	90.0
Case 2	D + L	0.0	n/a	0.0	90.0
Case 3	$0.6D + W_{up}$	0.0	n/a	0.0	90.0
Case 4	$0.6D + W_{up} + W_v$	0.0	258.0	258.0	0.0
Case 5	$D + W_d + W_v$	0.0	258.0	258.0	0.0
Case 6	$D + W_d$ + Water	0.0	258.0	258.0	0.0
Case 7	$D + 0.75 (L+W_{up})$	0.0	258.0	258.0	0.0

Wood Information : Southern Pine

G = 0.55

Bolt Diameter = 0.25

Fe parallel = 6160

Fe Perpendicular = 5127

Conc. Information:

F'c = 3000

Bolt Embd. = 1.5

Fyb = 45000

Tm = 8

Fem = 7000

Nominal Ledger Size :

Ts = Width = 1.5

De = 1.75 **(effective depth from loaded edge to center of bolt)**

	Parallel	Perendicular
Re =	1.14	1.37
Rt =	5.33	5.33
K1 =	2.02	2.37
K2 =	1.07	1.18
K3 =	1.03	0.97

Parallel to Grain:		Perpendicular to Grain:	
K theta =	1.00	K theta =	1.25
NDS Eq 8.2-2	578	NDS Eq 8.2-2	385
NDS Eq 8.2-3	1298	NDS Eq 8.2-3	1014
NDS Eq 8.2-4	1431	NDS Eq 8.2-4	1107
NDS Eq 8.2-5	270	NDS Eq 8.2-5	188
NDS Eq 8.2-6	194	NDS Eq 8.2-6	147
Parallel Z =	**194**	**Perpendicular Z =**	**147**
Try Bolt Spacing =	**12**		

Capacity x 1.33 = 258#

	Resultant Load	Angle	Z'_θ	Z / Z'_θ	Ledger Shear
Case 1:	**0.0**	**90.0**	**133**	**0.000**	**0.0 psi**
Case 2:	**0.0**	**90.0**	**147**	**0.000**	**0.0 psi**
Case 3:	**0.0**	**90.0**	**196**	**0.000**	**0.0 psi**
Case 4:	**258.0**	**0.0**	**258**	**1.002**	**0.0 psi**
Case 5:	**258.0**	**0.0**	**258**	**1.002**	**0.0 psi**
Case 6:	**258.0**	**0.0**	**258**	**1.002**	**0.0 psi**
Case 7:	**258.0**	**0.0**	**194**	**1.332**	**0.0 psi**

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SHEET NO. 8-12 OF
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Ex. Floor Suction Capacity (Part of MWFRS)

Y8" Strap Found every 4TH Joist w/ 3-16d Nails, Strap is Embd Into Conc. Beam.

Assume Strap Capacity Into Conc = 750#

16d Nail Strap Capacity

$138^{\#} \times 1.33 \times 3 = 551^{\#}$

one To Two Holes Available:

$\therefore 551 + 138(1.33) = 735^{\#}$

Frictional Capacity of Wood Bearing & Embd. In Conc. T.B.

Typ. Span = 16' Dead Floor Load 10 psf Self
5 psf MEP
5 psf Finishes
20 psf.

$\mu = .45$

$20 psf \times 1.33 \times 8 = 213^{\#}$

$213 \times .45 = 96^{\#}/Joist$

$\therefore$ Total Suction Force Capacity =

$$\frac{735^{\#}}{4'} + \frac{96}{1.33} = 256^{\#}/ft$$

Side Suction = $25.63 \times 9.5' = 243^{\#}/ft < 256$ $\therefore$ OK

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Gainor Residence
5800 North Bay Road
City of Miami Beach, Florida

Masonry Wall Reinforcing Calculations

SIDDIQ KHAN & ASSOCIATES, INC.
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JOB 05-618.00
SHEET NO. 9-2 OF
CALCULATED BY TAK DATE 9/16/05
CHECKED BY DATE
SCALE

Masonry Wall Calculations:

No Horiz Joint Reinf.

Assume Type 'N' mortar (Parallel to Bed Joints) Allow = 19 psi

Find Reinf Spacing where Stress does not Exceed 19 psi

$S_x = \frac{1}{6}\,12(7.625)^2 = 116.3 - \frac{1}{6}(12)(5.125)^2 = 63.8\ in^3/ft$

$m = 19(63.8) = 1212.2\ \frac{in\text{-}\#}{ft}$ or $101\ ^{1\text{-}\#}/_{ft}$

Take 3 span Condition: Max $m = \frac{1}{10}wl^2$

$101 = \frac{1}{10}(51.5)l^2$ $Z_4 \rightarrow l^2/3 = 27 \times 30\ (l = 9')$
w/ $p = 51.5$ psf

$\sqrt{l^2} = 4.43'$ — use 48" or less.

← Floor Joists

Assume $f'm = 1500$ Ground Floor height = 10.9 − 1' = 9.9' ≈ 10'
Second Floor height = 9' + 1' Bm = 10'

					Z_4	Z_5
Typ Second Floor	D = 875	L = 600	V = 724		51.0	60.0
Typ. 1st Floor	D = 1850	L = 1400	V = 724		51.0	60.0

From Masonry Wall Calcs: use RM648 Z_4 & RM632 Z_5 For Both Flrs.
(See Following Pgs for Calcs)

2nd Lvl Typ. Masonry Openings: 0 → 2'-6" use 1#6
2'-6" → 4'-6" use 1#7
4' → 6'-6" use 2#6

1st Lvl Typ. M.O.'s: 0 → 3'-0" use 1#6
3'-0 → 4'-0" use 1#7
4'-0 → 5'-6" use 2#6

Siddiq Khan & Associates, Inc.

8" Masonry Wall Design v1.10
Code: ACI 530-02

Project Name : Gainor Residence
Project Number: 05-618.00
Design Element: Typ. Reinf Zone 5 1st Level

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	8.625	ft
Use d =	3.81	in
Select RM	6	32

Net Block Area, An =	46.0	in^2/ft
Ix, Moment of Inertia =	343.7	in^4
r, Radius of gyration =	2.59	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	32	in
d =	3.81	in
Bar Area =	0.44	in^2
p = As / b d =	0.0036089	
k=((np)^2+2np)^.5-np =	0.32	
j = 1-k/3 =	0.89	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	333.93	psi
Allow Fb = 0.33*F'm =	500.00	psi

Dead 1 =	1850	plf	e =	0.00 in	0 ft-lb/ft
Dead 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Dead 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ DL =	1850			Σ M_{DL} =	0 ft-lb/ft
Live 1 =	1400	plf	e =	0.00 in	0 ft-lb/ft
Live 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Live 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ LL =	1400			Σ M_{LL} =	0 ft-lb/ft
Wind uplift =	-724	plf	e=	0.00 in	0 ft-lb/ft
Uniform Lateral Load =	60	psf	(distances are from bottom)		750 ft-lb/ft
Partial Uniform Lateral Load =	0.0	psf	0.00 ft.	0.00 ft.	0 ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft.		0 ft-lb/ft
				Σ M_{Lat} =	750 ft-lb/ft

Masonry Moduli

F'm	Em	Bar Size	Bar Area
1000	0.9 e06 psi	4	0.20
1500	1.35 e06 psi	5	0.31
2000	1.8 e06 psi	6	0.44
2500	2.25 e06 psi	7	0.60
3000	2.5 e06 psi	8	0.79
4000	2.9 e06 psi	9	1.00
5000	3.2 e06 psi	10	1.27

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	3250.0 lb	388 lb	2367 lb	lbs/ft
	0.0	750 ft-lb	563 ft-lb	ft-lbs/ft
fa, Axial Stress	70.7	8.4	51.2	psi
fb, bending stress	0.0	357.7	268.3	psi
fs, Steel Stress	0	16049	12036	psi
Add'l steel stress to axial Tension	0	0	0	psi
fa / Fa =	0.212	0.025	0.153	
fb / Fb =	0.000	0.715	0.537	
fa/fa+fb/fb =	[redacted]			
fs / Fs =				

Units	Grouted Cells	Motar Bedding	Section Properties A in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.6	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

Siddiq Khan & Associates, Inc.

8" Masonry Wall Design v1.10
Code: ACI 530-02

Project Name : Galnor Residence
Project Number: 05-618.00
Design Element: Typ. Reinf Zone 4 1st Level

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	8.625	ft
Use d =	3.81	in
Select RM	6	48

Net Block Area, An =	40.7	in^2/ft
Ix, Moment of Inertia =	332.0	in^4
r, Radius of gyration =	2.66	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	48	in
d =	3.81	in
Bar Area =	0.44	in^2
p = As / b d =	0.0024059	
k=((np)^2+2np)^.5-np =	0.27	
j = 1-k/3 =	0.91	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	338.06	psi
Allow Fb = 0.33*F'm =	500.00	psi

Dead 1 =	1850	plf	e =	0.00 in	0 ft-lb/ft
Dead 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Dead 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ DL =	1850			Σ M_{DL} =	0 ft-lb/ft
Live 1 =	1400	plf	e =	0.00 in	0 ft-lb/ft
Live 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Live 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ LL =	1400			Σ M_{LL} =	0 ft-lb/ft
Wind uplift =	-724	plf	e=	0.00 in	0 ft-lb/ft
Uniform Lateral Load =	51	psf	(distances are from bottom)		638 ft-lb/ft
Partial Uniform Lateral Load =	0.0	psf	0.00 ft.	0.00 ft.	0 ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft.		0 ft-lb/ft
				Σ M_{Lat} =	638 ft-lb/ft

Masonry Moduli

F'm	Em
1000	0.9 e06 psi
1500	1.35 e06 psi
2000	1.8 e06 psi
2500	2.25 e06 psi
3000	2.5 e06 psi
4000	2.9 e06 psi
5000	3.2 e06 psi

Bar Size	Bar Area
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	3250.0	366 lb	2367 lb	lbs/ft
	0.0	638 ft-lb	478 ft-lb	ft-lbs/ft
fa, Axial Stress	79.9	9.5	57.9	psi
fb, bending stress	0.0	352.8	264.6	psi
fs, Steel Stress	0	20088	15066	psi
Add'l steel stress to axial Tension	0	0	0	psi
fa / Fa =	0.238	0.026	0.172	
fb / Fb =	0.000	0.706	0.529	
fa/fa+fb/fb =				
fs / Fs =				

Units	Grouted Cells	Mortar Bedding	Section Properties A in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.6	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

Siddiq Khan & Associates, Inc.

8" Masonry Wall Design v1.10
Code: ACI 530-02

Project Name : Gainor Residence
Project Number: 05-618.00
Design Element: Typ. Reinf Zone 4 2nd Flr

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	8.625	ft
Use d =	3.81	in
Select RM	6	48

Net Block Area, An =	40.7	in^2/ft
Ix, Moment of Inertia =	332.0	in^4
r, Radius of gyration =	2.86	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	48	in
d =	3.81	in
Bar Area =	0.44	in^2
p = As / b d =	0.0024059	
k=((np)^2+2np)^.5-np =	0.27	
j = 1-k/3 =	0.91	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	336.06	psi
Allow Fb = 0.33*F'm =	500.00	psi

Dead 1 =	875	plf	e =	0.00 in	0	ft-lb/ft
Dead 2 =	0	plf	e =	0.00 in	0	ft-lb/ft
Dead 3 =	0	plf	e =	0.00 in	0	ft-lb/ft
Σ DL =	875			Σ M_{DL} =	0	ft-lb/ft
Live 1 =	600	plf	e =	0.00 in	0	ft-lb/ft
Live 2 =	0	plf	e =	0.00 in	0	ft-lb/ft
Live 3 =	0	plf	e =	0.00 in	0	ft-lb/ft
Σ LL =	600			Σ M_{LL} =	0	ft-lb/ft
Wind uplift =	-724	plf	e=	0.00 in	0	ft-lb/ft
Uniform Lateral Load =	51	psf	(distances are from bottom)		638	ft-lb/ft
Partial Uniform Lateral Load =	0.0	psf	0.00 ft.	0.00 ft.	0	ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft.		0	ft-lb/ft
				Σ M_{Lat} =	638	ft-lb/ft

Masonry Moduli

F'm	Em
1000	0.9 e06 psi
1500	1.35 e06 psi
2000	1.8 e06 psi
2500	2.25 e06 psi
3000	2.5 e06 psi
4000	2.9 e06 psi
5000	3.2 e06 psi

Bar Size	Bar Area
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	1475.0	-199 lb	782 lb	lbs/ft
	0.0	638 ft-lb	478 ft-lb	ft-lbs/ft
fa, Axial Stress	36.2	-4.9	19.2	psi
fb, bending stress	0.0	352.8	264.6	psi
fs, Steel Stress	0	20088	15066	psi
Add'l steel stress to axial Tension	0	1809	0	psi
fa / Fa =	0.108	-0.015	0.057	
fb / Fb =	0.000	0.706	0.529	
fa/fa+fb/fb =				
fs / Fa =				

Units	Grouted Cells	Mortar Bedding	A in^2/ft	Section Properties I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.6	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

Siddiq Khan & Associates, Inc.

8" Masonry Wall Design v1.10
Code: ACI 530-02

Project Name : Gainor Residence
Project Number: 05-618.00
Design Element: Typ. Reinf Zone 5 2nd FLR

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	0.635	ft
Use d =	3.81	in
Select RM	6	32

Net Block Area, An =	46.0	in^2/ft
Ix, Moment of Inertia =	343.7	in^4
r, Radius of gyration =	2.59	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	32	in
d =	3.81	in
Bar Area =	0.44	in^2
p = As / b d =	0.0036089	
k=((np)^2+2np)^.5-np =	0.32	
j = 1-k/3 =	0.89	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	333.93	psi
Allow Fb = 0.33*F'm =	500.00	psi

Dead 1 =	875	plf	e =	0.00 in	0	ft-lb/ft
Dead 2 =	0	plf	e =	0.00 in	0	ft-lb/ft
Dead 3 =	0	plf	e =	0.00 in	0	ft-lb/ft
Σ DL =	875			Σ M_{DL} =	0	ft-lb/ft
Live 1 =	600	plf	e =	0.00 in	0	ft-lb/ft
Live 2 =	0	plf	e =	0.00 in	0	ft-lb/ft
Live 3 =	0	plf	e =	0.00 in	0	ft-lb/ft
Σ LL =	600			Σ M_{LL} =	0	ft-lb/ft
Wind uplift =	-724	plf	e=	0.00 in	0	ft-lb/ft
Uniform Lateral Load =	60	psf	(distances are from bottom)		750	ft-lb/ft
Partial Uniform Lateral Load =	0.0	psf	0.00 ft.	0.00 ft.	0	ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft.		0	ft-lb/ft
				Σ M_{Lat} =	750	ft-lb/ft

Masonry Moduli

F'm	Em
1000	0.9 e06 psi
1500	1.35 e06 psi
2000	1.8 e06 psi
2500	2.25 e06 psi
3000	2.5 e06 psi
4000	2.9 e06 psi
5000	3.2 e06 psi

Bar Size	Bar Area
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	1475.0	-199 lb	782 lb	lbs/ft
	0.0	750 ft-lb	563 ft-lb	ft-lbs/ft
fa, Axial Stress	32.1	-4.3	17.0	psi
fb, bending stress	0.0	357.7	268.3	psi
fs, Steel Stress	0	16049	12036	psi
Add'l steel stress to axial Tension	0	1206	0	psi
fa / Fa =	0.096	-0.013	0.051	
fb / Fb =	0.000	0.715	0.537	
fa/fa+fb/fb =				
fs / Fs =				

Units	Grouted Cells	Mortar Bedding	A in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	82.0	378.8	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	328.2	85.8	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

Siddiq Khan & Associates, Inc.

8" Masonry Wall Design w/ Opening v1.10
Code: ACI 530-02

Project Name : **Gainor Residence**
Project Number: **05-618.00**
Design Element: **Typical Zone 4 M.O. upto 3'-0"** 1st Level

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	0.635	ft
Use d =	3.81	in
Masonry Opening =	3.00	ft
Wall Reinforcing, RM	8	48
No. of bars & size at M.O.	1	#6

Net Block Area, An =	40.7	in^2/ft
Ix, Moment of Inertia =	332.0	in^4
r, Radius of gyration =	2.66	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	28	in
d =	3.81	in
Bar Area =	0.44	in^2
p = As / b d =	0.00412448	
k=((np)^2+2np)^.5-np =	0.34	
j = 1-k/3 =	0.89	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	336.06	psi
Allow Fb = 0.33*F'm =	500	psi

Dead + 0.75 (Live + Lateral) =	3250 lb/ft	388 lb/ft	2357 lb/ft
0.6 Dead + lateral + uplift =	0 ft-lb/ft	638 ft-lb/ft	478 ft-lb/ft
Dead + live =	0 ft-lb	2444 ft-lb	1833 ft-lb
fa, Axial Stress	131.2	15.6	95.1 psi
fb, bending stress	0.0	476.7	357.5 psi
fs, Steel Stress	0	19740	14805 psi
Add'l steel stress to axial Tension	0	0	0 psi
fa / Fa =	0.390	0.046	0.283
fb / Fb =	0.000	0.953	0.715
fa/fa+fb/fb =			
fa / Fs =			

Dead 1 =	1850	plf	e =	0.00 in	0 ft-lb/ft
Dead 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Dead 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ DL =	1850			Σ M_{DL} =	0 ft-lb/ft
Live 1 =	1400	plf	e =	0.00 in	0 ft-lb/ft
Live 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Live 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ LL =	1400			Σ M_{LL} =	0 ft-lb/ft
Wind uplift =	-724	plf	e=	0.00 in	0 ft-lb/ft
Uniform Lateral Load =	51	psf	(distances are from bottom)		638 ft-lb/ft
Partial Uniform Lateral Load =	0	psf	0.00 ft.	0.00 ft.	0 ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft.		0 ft-lb/ft
				Σ M_{Lat} =	638 ft-lb/ft

Masonry Moduli

F'm	Em	Bar Size	Bar Area
1000	0.9 e06 psi	4	0.20
1500	1.35 e06 psi	5	0.31
2000	1.8 e06 psi	6	0.44
2500	2.25 e06 psi	7	0.60
3000	2.5 e06 psi	8	0.79
4000	2.9 e06 psi	9	1.00
5000	3.2 e06 psi	10	1.27

Units	Grouted Cells	Motar Bedding	Section Properties A in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.6	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

Siddiq Khan & Associates, Inc.

8" Masonry Wall Design w/ Opening v1.10
Code: ACI 530-02

Project Name : Gainor Residence
Project Number: 05-618.00
Design Element: Typical Zone 4 M.O. upto 4'-0" 1st Level

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	0.635	ft
Use d =	3.81	in
Masonry Opening =	4.00	ft
Wall Reinforcing, RM	6	48
No. of bars & size at M.O.	1	#7

Net Block Area, An =	40.7	in^2/ft
Ix, Moment of Inertia =	332.0	in^4
r, Radius of gyration =	2.66	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	28	in
d =	3.81	in
Bar Area =	0.60	in^2
p = As / b d =	0.0056243	
k=((np)^2+2np)^.5-np =	0.39	
j = 1-k/3 =	0.87	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	336.06	psi
Allow Fb = 0.33*F'm =	500	psi

Dead 1 =	1850	plf	e =	0.00 in	0 ft-lb/ft
Dead 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Dead 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ DL =	1850			Σ M_{DL} =	0 ft-lb/ft
Live 1 =	1400	plf	e =	0.00 in	0 ft-lb/ft
Live 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Live 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ LL =	1400			Σ M_{LL} =	0 ft-lb/ft
Wind uplift =	-724	plf	e=	0.00 in	0 ft-lb/ft
Uniform Lateral Load =	51	psf	(distances are from bottom)		638 ft-lb/ft
Partial Uniform Lateral Load =	0	psf	0.00 ft.	0.00 ft.	0 ft-lb/ft
Concentrated lateral load =	0	plf	0.00 ft.		0 ft-lb/ft
				Σ M_{Lat} =	638 ft-lb/ft

Masonry Moduli

F'm	Em
1000	0.9 e06 psi
1500	1.35 e06 psi
2000	1.8 e06 psi
2500	2.25 e06 psi
3000	2.5 e06 psi
4000	2.9 e06 psi
5000	3.2 e06 psi

Bar Size	Bar Area
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	3250 lb/ft 0 ft-lb/ft 0 ft-lb	386 lb/ft 638 ft-lb/ft 2763 ft-lb	2357 lb/ft 478 ft-lb/ft 2072 ft-lb	
fa, Axial Stress	148.3	17.6	107.6	psi
fb, bending stress	0.0	485.7	364.2	psi
fs, Steel Stress	0	16639	12479	psi
Add'l steel stress to axial Tension	0	0	0	psi
fa / Fa =	0.441	0.052	0.320	
fb / Fb =	0.000	0.971	0.728	
fa/fa+fb/fb =	[redacted]			
fs / Fs =				

Units	Grouted Cells	Motar Bedding	A in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.6	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

Siddiq Khan & Associates, Inc.

8" Masonry Wall Design w/ Opening v1.10

Code: ACI 530-02

Project Name : Gainor Residence

Project Number: 05-618.00

Design Element: Typical Zone 4 M.O. upto 5'-6" | 5t Level

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	0.635	ft
Use d =	3.81	in
Masonry Opening =	5.50	ft
Wall Reinforcing, RM	6	48
No. of bars & size at M.O.	2	#6

Net Block Area, An =	40.7	in^2/ft
Ix, Moment of Inertia =	332.0	in^4
r, Radius of gyration =	2.66	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	36	in
d =	3.81	in
Bar Area =	0.88	in^2
p = As / b d =	0.00641586	
k=((np)^2+2np)^.5-np =	0.40	
j = 1-k/3 =	0.87	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	336.06	psi
Allow Fb = 0.33*F'm =	500	psi

Dead 1 =	1850	plf	e =	0.00 in	0 ft-lb/ft
Dead 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Dead 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ DL =	1850		Σ M_{DL} =		0 ft-lb/ft
Live 1 =	1400	plf	e =	0.00 in	0 ft-lb/ft
Live 2 =	0	plf	e =	0.00 in	0 ft-lb/ft
Live 3 =	0	plf	e =	0.00 in	0 ft-lb/ft
Σ LL =	1400		Σ M_{LL} =		0 ft-lb/ft
Wind uplift =	-724	plf	e=	0.00 in	0 ft-lb/ft
Uniform Lateral Load =	51	psf	(distances are from bottom)		638 ft-lb/ft
Partial Uniform Lateral Load =	0	psf	0.00 ft.	0.00 ft.	0 ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft.		0 ft-lb/ft
			Σ M_{Lat} =		638 ft-lb/ft

Masonry Moduli

F'm	Em
1000	0.9 e06 psi
1500	1.35 e06 psi
2000	1.8 e06 psi
2500	2.25 e06 psi
3000	2.5 e06 psi
4000	2.9 e06 psi
5000	3.2 e06 psi

Bar Size	Bar Area
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	3250 lb/ft 0 ft-lb/ft 0 ft-lb	386 lb/ft 638 ft-lb/ft 3666 ft-lb	2367 lb/ft 478 ft-lb/ft 2749 ft-lb	
fa, Axial Stress	153.1	18.2	111.0	psi
fb, bending stress	0.0	480.6	360.4	psi
fs, Steel Stress	0	15167	11375	psi
Add'l steel stress to axial Tension	0	0	0	psi
fa / Fa =	0.455	0.054	0.330	
fb / Fb =	0.000	0.961	0.721	
fa/fa+fb/fb =				
fs / Fs =				

Units	Grouted Cells	Mortar Bedding	A in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.6	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

Siddiq Khan & Associates, Inc.

8" Masonry Wall Design w/ Opening v1.10
Code: ACI 530-02

Project Name : Gainor Residence
Project Number: 05-618.00
Design Element: Typical Zone 4 M.O. upto 2'-6" 2nd Flr

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	0.635	ft
Use d =	3.81	in
Masonry Opening =	2.50	ft
Wall Reinforcing, RM	6	48
No. of bars & size at M.O.	1	#6

Net Block Area, An =	40.7	in^2/ft
Ix, Moment of Inertia =	332.0	in^4
r, Radius of gyration =	2.66	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	28	in
d =	3.81	in
Bar Area =	0.44	in^2
p = As / b d =	0.00412448	
k=((np)^2+2np)^.5-np =	0.34	
j = 1-k/3 =	0.89	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	336.06	psi
Allow Fb = 0.33*F'm =	500	psi

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	1475 lb/ft	-199 lb/ft	782 lb/ft	
	0 ft-lb/ft	638 ft-lb/ft	478 ft-lb/ft	
	0 ft-lb	2284 ft-lb	1713 ft-lb	
fa, Axial Stress	55.7	-7.5	29.5	psi
fb, bending stress	0.0	445.6	334.2	psi
fs, Steel Stress	0	18453	13840	psi
Add'l steel stress to axial Tension	0	6483	0	psi
fa / Fa =	0.166	-0.022	0.088	
fb / Fb =	0.000	0.891	0.668	
fa/Fa+fb/Fb =				
fa / Fs =				

Dead 1 =	875	plf	e = 0.00 in	0 ft-lb/ft
Dead 2 =	0	plf	e = 0.00 in	0 ft-lb/ft
Dead 3 =	0	plf	e = 0.00 in	0 ft-lb/ft
Σ DL =	875		ΣM_{DL} =	0 ft-lb/ft
Live 1 =	600	plf	e = 0.00 in	0 ft-lb/ft
Live 2 =	0	plf	e = 0.00 in	0 ft-lb/ft
Live 3 =	0	plf	e = 0.00 in	0 ft-lb/ft
Σ LL =	600		ΣM_{LL} =	0 ft-lb/ft
Wind uplift =	-724	plf	e= 0.00 in	0 ft-lb/ft
Uniform Lateral Load =	51	psf	(distances are from bottom)	638 ft-lb/ft
Partial Uniform Lateral Load =	0	psf	0.00 ft. 0.00 ft.	0 ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft.	0 ft-lb/ft
			ΣM_{Lat} =	638 ft-lb/ft

Masonry Moduli

F'm	Em	Bar Size	Bar Area
1000	0.9 e06 psi	4	0.20
1500	1.35 e06 psi	5	0.31
2000	1.8 e06 psi	6	0.44
2500	2.25 e06 psi	7	0.60
3000	2.5 e06 psi	8	0.79
4000	2.9 e06 psi	9	1.00
5000	3.2 e06 psi	10	1.27

Units	Grouted Cells	Mortar Bedding	Section Properties A in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.6	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

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Siddiq Khan & Associates, Inc.

8" Masonry Wall Design w/ Opening v1.10

Code: ACI 530-02

Project Name : Gainor Residence

Project Number: 05-618.00

Design Element: Typical Zone 4 M.O. upto 4'-6" 2nd Flr

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	0.635	ft
Use d =	3.81	in
Masonry Opening =	4.50	ft
Wall Reinforcing, RM	6	48
No. of bars & size at M.O.	1	#7

Net Block Area, An =	40.7	in^2/ft
Ix, Moment of Inertia =	332.0	in^4
r, Radius of gyration =	2.66	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	28	in
d =	3.81	in
Bar Area =	0.60	in^2
ρ = As / b d =	0.0056243	
k=((np)^2+2np)^.5-np =	0.39	
j = 1-k/3 =	0.87	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	336.06	psi
Allow Fb = 0.33*F'm =	500	psi

Dead 1 =	875	plf	e = 0.00 in → 0 ft-lb/ft
Dead 2 =	0	plf	e = 0.00 in → 0 ft-lb/ft
Dead 3 =	0	plf	e = 0.00 in → 0 ft-lb/ft
Σ DL =	875		Σ M_{DL} = 0 ft-lb/ft
Live 1 =	600	plf	e = 0.00 in → 0 ft-lb/ft
Live 2 =	0	plf	e = 0.00 in → 0 ft-lb/ft
Live 3 =	0	plf	e = 0.00 in → 0 ft-lb/ft
Σ LL =	600		Σ M_{LL} = 0 ft-lb/ft
Wind uplift =	-724	plf	e= 0.00 in → 0 ft-lb/ft
Uniform Lateral Load =	51	psf	(distances are from bottom) → 638 ft-lb/ft
Partial Uniform Lateral Load =	0	psf	0.00 ft. – 0.00 ft. → 0 ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft. → 0 ft-lb/ft
			Σ M_{Lat} = 638 ft-lb/ft

Masonry Moduli

F'm	Em
1000	0.9 e06 psi
1500	1.35 e06 psi
2000	1.8 e06 psi
2500	2.25 e06 psi
3000	2.5 e06 psi
4000	2.9 e06 psi
5000	3.2 e06 psi

Bar Size	Bar Area
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	1475 lb/ft 0 ft-lb/ft 0 ft-lb	-199 lb/ft 638 ft-lb/ft 2922 ft-lb	782 lb/ft 478 ft-lb/ft 2191 ft-lb	
fa, Axial Stress	71.2	-9.6	37.7	psi
fb, bending stress	0.0	513.7	385.3	psi
fs, Steel Stress	0	17599	13199	psi
Add'l steel stress to axial Tension	0	6081	0	psi
fa / Fa =	0.212	-0.029	0.112	
fb / Fb =	0.000	1.027	0.771	
fa/fa+fb/fb =				
fs / Fs =				

Units	Grouted Cells	Motar Bedding	A in^2/ft	Section Properties I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.6	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

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Siddiq Khan & Associates, Inc.

8" Masonry Wall Design w/ Opening v1.10
Code: ACI 530-02

Project Name : **Gainor Residence**
Project Number: **05-618.00**
Design Element: **Typical Zone 4 M.O. upto 6'-6"** 2nd Flr

Data Input :

Height Of Masonry Wall =	10.00	ft
F'm (with inspection) =	1500	psi
Fs =	24000	psi
Wall Thickness =	0.635	ft
Use d =	3.81	in
Masonry Opening =	6.50	ft
Wall Reinforcing, RM	6	48
No. of bars & size at M.O.	2	#6

Net Block Area, An =	40.7	in^2/ft
Ix, Moment of Inertia =	332.0	in^4
r, Radius of gyration =	2.66	
E_m =	1350000	psi
Es = 29 x 10^6 =	29000000	psi
Modular Ratio n =	21	
Use b =	36	in
d =	3.81	in
Bar Area =	0.88	in^2
p = As / b d =	0.00841586	
k=((np)^2+2np)^.5-np =	0.40	
j = 1-k/3 =	0.87	
Fa = 0.25 F'm (1-(h / 140 r)^2) =	336.06	psi
Allow Fb = 0.33*F'm =	500	psi

Dead 1 =	875	plf	e = 0.00 in	0 ft-lb/ft
Dead 2 =	0	plf	e = 0.00 in	0 ft-lb/ft
Dead 3 =	0	plf	e = 0.00 in	0 ft-lb/ft
Σ DL =	875		Σ M_{DL} =	0 ft-lb/ft
Live 1 =	600	plf	e = 0.00 in	0 ft-lb/ft
Live 2 =	0	plf	e = 0.00 in	0 ft-lb/ft
Live 3 =	0	plf	e = 0.00 in	0 ft-lb/ft
Σ LL =	600		Σ M_{LL} =	0 ft-lb/ft
Wind uplift =	-724	plf	e= 0.00 in	0 ft-lb/ft
Uniform Lateral Load =	51	psf	(distances are from bottom)	638 ft-lb/ft
Partial Uniform Lateral Load =	0	psf	0.00 ft. 0.00 ft.	0 ft-lb/ft
Concentrated lateral Load =	0	plf	0.00 ft.	0 ft-lb/ft
			Σ M_{Lat} =	638 ft-lb/ft

Masonry Moduli

F'm	Em
1000	0.9 e06 psi
1500	1.35 e06 psi
2000	1.8 e06 psi
2500	2.25 e06 psi
3000	2.5 e06 psi
4000	2.9 e06 psi
5000	3.2 e06 psi

Bar Size	Bar Area
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27

	Dead + 0.75 (Live + Lateral) =	0.6 Dead + lateral + uplift =	Dead + live =	
	1475 lb/ft	-199 lb/ft	782 lb/ft	
	0 ft-lb/ft	638 ft-lb/ft	478 ft-lb/ft	
	0 ft-lb	3984 ft-lb	2988 ft-lb	
fa, Axial Stress	75.5	-10.2	40.0	psi
fb, bending stress	0.0	522.4	391.8	psi
fs, Steel Stress	0	16486	12364	psi
Add'l steel stress to axial Tension	0	5653	0	psi
fa / Fa =	0.226	-0.030	0.119	
fb / Fb =	0.000	1.045	0.784	
fa/Fa+fb/Fb =	[redacted]			
fs / Fs =				

Units	Grouted Cells	Motar Bedding	Section Properties A in^2/ft	I_x in^4/ft	S_x in^3/ft	r in
Hollow	none - 1	Face Shell	30.0	308.7	81.0	2.84
Hollow	none - 2	Full	41.5	334.0	87.6	2.84
100% Solid	none - 3	Full	91.5	443.3	116.3	2.19
Hollow	8 in o.c.	Full	91.5	443.3	116.3	2.19
Hollow	16 in o.c.	Face Shell	62.0	378.8	99.3	2.43
Hollow	24 in o.c.	Face Shell	51.3	355.3	93.2	2.53
Hollow	32 in o.c.	Face Shell	46.0	343.7	90.1	2.59
Hollow	40 in o.c.	Face Shell	42.8	336.7	88.3	2.63
Hollow	48 in o.c.	Face Shell	40.7	332.0	87.1	2.66
Hollow	56 in o.c.	Face Shell	39.1	328.7	86.2	2.68
Hollow	64 in o.c.	Face Shell	38.0	326.2	85.6	2.70
Hollow	72 in o.c.	Face Shell	37.1	324.3	85.0	2.71

21-9

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Concrete Beam / Column Designs

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JOB
SHEET NO. 10-2 OF
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Attachment of New TB To Ex. TB

Max Typ. Truss Reaction = 1320#

Max Girder Truss Uplift = 5740#

Hilti Hse 2421 Epoxy For #5 Bar 5-5/8" Embd Develops Ultimate Strength of Bar. (Provided 6")

Spac ^ For Full Load ⇒ 8 7/16" ⇒ 1.0 Factor

Edge Dist = 4" ⇒ $(0.76)^2$ ∴

Allow Tension = 60 KSI $(.76)^2$ = 34.7 KSI x 0.31 = 10.7K

Allow Tension = 10.7K

Development Into Bm w/ std. Hook.

$$\frac{.02(60000)}{\sqrt{5000}}(5/8) = 16.97" \times 0.7 = 11.8"$$

10"/11.8" = 0.85 0.85 (60) = 51 KSI ∴ Epoxy Tension Controls.

If Provided @ 4' O.C.:

Tension = 1320 x 2 = 2640 V_{max} = 265 #/ft (4') = 1060

$N_u = 1.6(2.64) = 4.22^K$ $V_u = 1.6(1.06) = 1.69$

$\phi V_n = 0.31(60)(.75)(.6) = 8.37^K > 1.69$ ∴ OK

$\phi N_n = 0.31(60)(.9) = 16.74^K > 4.22^K$ ∴ OK

@ Girder Truss Provide 2 #5's @ 2' Centered w/ G.T.

HSE 2421 Epoxy Adhesive Anchor

HSE 2421 Ultimate Bond Strength and Steel Strength for Rebar in Normal Weight Concrete[3]

Nominal Rebar Size	Embedment Depth	Tension						
		Based on Bond or Concrete $f_c \geq 2000$ psi	Embedment to Develop Rebar Yield Strength in. (mm)		Embedment to Develop Rebar Ultimate Strength in. (mm)		ASTM A615 Gr 60 Rebar[1]	
							Yield Strength	Ultimate Tensile Strength
	in. (mm)	lb (kN)	f_c = 2000 psi	f_c = 4000 psi	f_c = 2000 psi	f_c = 4000 psi	lb (kN)	lb (kN)
#3	3⅜ (86)	9765 (43.4)	2 3/8 (60)	1 7/8 (48)	3 1/2 (89)	2 3/4 (70)	6600 (29.4)	9900 (44.0)
#4	4½ (114)	17365 (77.2)	3 1/8 (79)	2 3/4 (70)	4 3/4 (121)	4 (102)	12000 (53.4)	18000 (80.0)
#5	5⅝ (143)	24865 (110.6)	4 1/4 (108)	3 3/4 (95)	6 3/8 (162)	5 5/8 (143)	18600 (82.7)	27900 (124.1) 90 KsI
#6	6¾ (171)	36900 (164.1)	5 (127)	4 3/4 (121)	7 1/4 (184)	7 (178)	26400 (117.4)	39600 (176.2)
#7	7⅞ (200)	37190[2] (165.4)	7 5/8 (194)	6 1/8 (156)	11 1/2 (292)	9 1/8 (232)	36000 (160.1)	54000 (240.2)
#8	9 (229)	55985 (249.0)	7 3/4 (197)	6 1/2 (165)	11 1/2 (292)	9 3/4 (248)	47400 (210.8)	71100 (316.3)
#9	10⅛ (257)	61475[2] (273.5)	9 7/8 (251)	8 (203)	14 7/8 (378)	11 1/2 (292)	60000 (266.9)	90000 (400.3)
#10	11¼ (286)	75895[2] (337.6)	11 3/8 (289)	9 1/4 (235)	17 (432)	14 (356)	76200 (339.0)	114300 (508.4)
#11	12⅜ (314)	82935 (368.9)	14 (356)	10 1/2 (267)	21 (533)	15 3/4 (400)	93600 (416.3)	140400 (624.5)

1. Based on minimum steel strength and cross-sectional area of rebar.
2. Values based on bond stress calculations.
3. Minimum concrete thickness must be equal to 1.5 times the anchor embedment.

HSE 2421 Allowable and Ultimate Bond Strength and Steel Strength for Metric Rebar in Concrete (Canada Only)[3,4,5]

Bar Number	Embedment Depth		Allowable Tensile Bond Strength for f_c > 14MPa (2000 psi)		Ultimate Tensile Bond Strength for f_c > 14MPa (2000 psi)		Strength Properties of Metric Rebar[1] f_y = 400 MPa	
							Yield Strength	Tensile Strength
	mm	(in)	kN	(lb)	kN	(lb)	kN	kN
10M	115	(4½)	19.3	(4340)	77.2	(17365)	40	60
	230	(9)	38.6	(8680)	154.4	(34730)		
15M	145	(5⅝)	27.6	(6215)	110.6	(24865)	80	120
	290	(10¼)	55.3	(12430)	221.2	(49730)		
20M	200	(7⅞)	41.0	(19225)	164.1	(36900)	120	180
	340	(13½)	82.0	(18450)	328.2	(73800)		
25M	230	(9)	62.2	(13995)	249	(55985)	200	300
	460	(18)	124.5	(27990)	498	(111970)		
30M	260	(10⅛)	68.3[2]	(15365)	273.5[2]	(61475)	280	420
	520	(20¼)	136.7[2]	(30735)	547[2]	(122950)		
35M	315	(12⅜)	92.2	(20730)	368.9	(82935)	400	600
	630	(24¾)	184.4	(41465)	737.8	(165870)		

1. Based on minimum steel strength and nominal cross-sectional area of rebar.
2. Values based on bond stress calculations.
3. Minimum concrete thickness must be equal to 1.5 times the anchor embedment.
4. Testing done with imperial rebar in same size holes.
5. Allowable tension for adhesive bond based on a safety factor of 4.0.

10-3

Load Adjustment Factors for 1/2" Diameter Anchor

Anchor Diameter		1/2" diameter							
Adjustment Factor		Spacing Tension/Shear, f_A		Edge Distance Tension, f_{RN}		Edge Distance Shear (⊥ toward edge), $f_{RV\perp}$		Edge Distance Shear (‖ to or away from edge), $f_{RV\parallel}$	
Embedment Depth, in.		4 1/2	6	4 1/2	6	4 1/2	6	4 1/2	6
Spacing (s)/Edge Distance (c), in.	2 1/4	0.70		0.70		0.30		0.60	
	3	0.75	0.70	0.75	0.70	0.42	0.30	0.67	0.60
	3 1/2	0.78	0.73	0.78	0.73	0.49	0.36	0.71	0.63
	4	0.82	0.75	0.82	0.75	0.57	0.42	0.76	0.67
	4 1/2	0.85	0.78	0.85	0.78	0.65	0.48	0.80	0.70
	5	0.88	0.80	0.88	0.80	0.73	0.53	0.84	0.73
	5 1/2	0.92	0.83	0.92	0.83	0.81	0.59	0.89	0.77
	6	0.95	0.85	0.95	0.85	0.88	0.65	0.93	0.80
	6 3/4	1.00	0.89	1.00	0.89	1.00	0.74	1.00	0.85
	7		0.90		0.90		0.77		0.87
	7 1/2		0.93		0.93		0.83		0.90
	8		0.95		0.95		0.88		0.93
	8 1/2		0.98		0.98		0.94		0.97
	9		1.00		1.00		1.00		1.00

NOTE: Tables apply for listed embedment depths. Reduction factors for other embedment depths must be calculated using equations below

Spacing Tension/Shear

$s_{min} = 0.5\ h_{ef}$, $s_{cr} = 1.5\ h_{ef}$

$f_A = 0.30(s/h_{ef}) + 0.55$

for $s_{cr} > s > s_{min}$

Edge Distance Tension

$c_{min} = 0.5\ h_{ef}$, $c_{cr} = 1.5\ h_{ef}$

$f_{RN} = 0.30(c/h_{ef}) + 0.55$

for $c_{cr} > c > c_{min}$

Edge Distance Shear (⊥ toward edge)

$c_{min} = 0.5\ h_{ef}$, $c_{cr} = 1.5\ h_{ef}$

$f_{RV\perp} = 0.70(c/h_{ef}) - 0.05$

for $c_{cr} > c > c_{min}$

Edge Distance Shear (‖ to or away from edge)

$c_{min} = 0.5\ h_{ef}$, $c_{cr} = 1.5\ h_{ef}$

$f_{RV\parallel} = 0.40(c/h_{ef}) + 0.40$

for $c_{cr} > c > c_{min}$

Load Adjustment Factors for 5/8" and 3/4" Diameter Anchors

Anchor Diameter		5/8" diameter								3/4" diameter							
Adjustment Factor		Spacing Tension/Shear f_A		Edge Distance Tension f_{RN}		Edge Distance Shear (⊥ toward edge) $f_{RV\perp}$		Edge Distance Shear (‖ to or away from edge) $f_{RV\parallel}$		Spacing Tension/Shear f_A		Edge Distance Tension f_{RN}		Edge Distance Shear (⊥ toward edge) $f_{RV\perp}$		Edge Distance Shear (‖ to or away from edge) $f_{RV\parallel}$	
Embedment Depth, in.		5 5/8	7 1/2	5 5/8	7 1/2	5 5/8	7 1/2	5 5/8	7 1/2	6 3/4	9	6 3/4	9	6 3/4	9	6 3/4	9
Spacing (s)/Edge Distance (c), in.	2 13/16	0.70		0.70		0.30		0.60									
	3	0.71		0.71		0.32		0.61									
	3 3/8	0.73		0.73		0.37		0.64		0.70		0.70		0.30		0.60	
	3 3/4	0.75	0.70	0.75	0.70	0.42	0.30	0.67	0.60	0.72		0.72		0.34		0.62	
	4	0.76	0.71	0.76	0.71	0.45	0.32	0.68	0.61	0.73		0.73		0.36		0.64	
	4 1/2	0.79	0.73	0.79	0.73	0.51	0.37	0.72	0.64	0.75	0.70	0.75	0.70	0.42	0.30	0.67	0.60
	5	0.82	0.75	0.82	0.75	0.57	0.42	0.76	0.67	0.77	0.72	0.77	0.72	0.47	0.34	0.70	0.62
	5 5/8	0.85	0.78	0.85	0.78	0.65	0.48	0.80	0.70	0.80	0.74	0.80	0.74	0.53	0.39	0.73	0.65
	6	0.87	0.79	0.87	0.79	0.70	0.51	0.83	0.72	0.82	0.75	0.82	0.75	0.57	0.42	0.76	0.67
	6 1/2	0.90	0.81	0.90	0.81	0.76	0.56	0.86	0.75	0.84	0.77	0.84	0.77	0.62	0.46	0.79	0.69
	6 3/4	0.91	0.82	0.91	0.82	0.79	0.58	0.88	0.76	0.85	0.78	0.85	0.78	0.65	0.48	0.80	0.70
	7	0.92	0.83	0.92	0.83	0.82	0.60	0.90	0.77	0.86	0.78	0.86	0.78	0.68	0.49	0.81	0.71
	7 1/2	0.95	0.85	0.95	0.85	0.88	0.65	0.93	0.80	0.88	0.80	0.88	0.80	0.73	0.53	0.84	0.73
	8	0.98	0.87	0.98	0.87	0.95	0.70	0.97	0.83	0.91	0.82	0.91	0.82	0.78	0.57	0.87	0.76
	8 7/16	1.00	0.89	1.00	0.89	1.00	0.74	1.00	0.85	0.93	0.83	0.93	0.83	0.83	0.61	0.90	0.78
	9		0.91		0.91		0.79		0.88	0.95	0.85	0.95	0.85	0.88	0.65	0.93	0.80
	9 1/2		0.93		0.93		0.84		0.91	0.97	0.87	0.97	0.87	0.94	0.69	0.96	0.82
	10		0.95		0.95		0.88		0.93	0.99	0.88	0.99	0.88	0.99	0.73	0.99	0.84
	10 1/8		0.96		0.96		0.90		0.94	1.00	0.89	1.00	0.89	1.00	0.74	1.00	0.85
	11 1/4		1.00		1.00		1.00		1.00		0.93		0.93		0.83		0.90
	12										0.95		0.95		0.88		0.93
	13										0.98		0.98		0.96		0.98
	13 1/2										1.00		1.00		1.00		1.00

10-4

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JOB Gainor Residence
SHEET NO. 10-5 OF
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Given:

Beam Info:

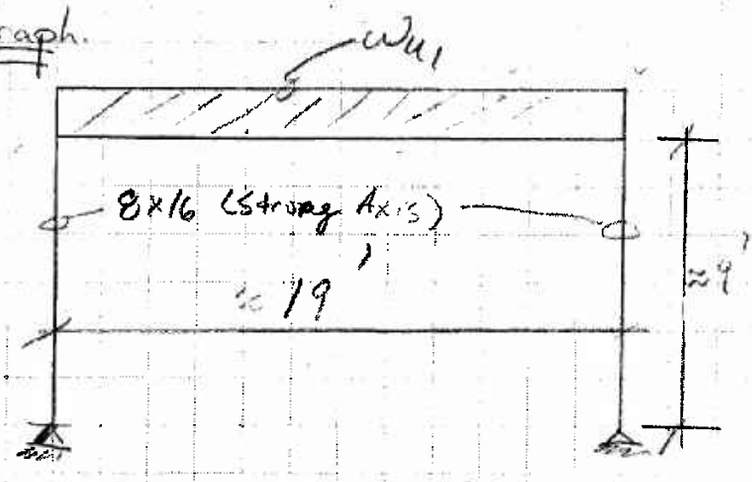
8x12 (min.) $\Rightarrow$ S.W. = 100 #/ft

Trib Left = 1'

Trib Right = 7'/2 = 3.5'

f'_c = 4000 psi, f_y = 60 ksi

Graph.



Case 1: (Gravity) {Only}

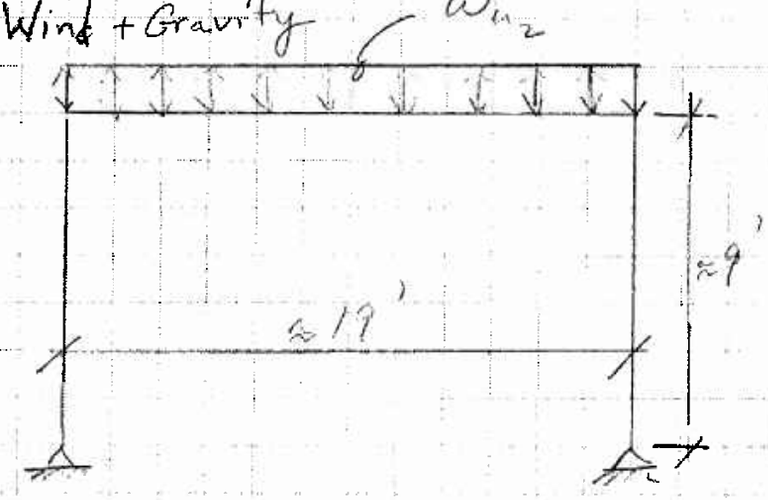
Loading

L_R = 30 psf

$DL_{\#1}$ = 25 psf

$DL_{\#2}$ = 10 psf (max w/Wind Combo per Miami-Dade Co.)

5055 # from Tim (9/26/05)



Uplift = 234 #/ft

Find: Frame Design

Formula: FBC 2004

Calcs:

Case 1:

$w_{DL} = (4.5')(25\ psf) + 100\ \#/ft = 212.5\ \#/ft \approx 213\ \#/ft$

$w_L = (4.5')(30\ psf) = 135\ \#/ft$

$w_u = 1.2(213\ \#/ft) + 1.6(135\ \#/ft) = 472\ \#/ft$

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SHEET NO. 10-6 OF
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Calcs (Cont'd)

Case 1: let $d = 12" - 3" = 9"$; $A_{s\,min} = 0.0033(8"X9") = 0.24^{\#"}$

Top

$$M_u^{(-)} = \frac{w_u l^2}{12} = \frac{(0.472^{k/ft} X 19')^2}{12} = 14.2^{k\cdot ft}$$

$$A_s \simeq \frac{M_u}{4d} = \frac{14.2^{k\cdot ft}}{(4)(9")} \simeq 0.4^{\#"} > A_{s\,min}$$

↑ controls (use 2#6 Top-Cont)

Bott

$$M_u^{+} \simeq \frac{w_u l^2}{24} = 7.1^{k\cdot ft}$$

$A_{s\,req'd}^{+} \simeq 0.2^{\#"} < A_{s\,min}$ 0.24" ← controls

Shear; $\phi V_c = 0.75(2)\sqrt{4000_{psi}}(8"X9") \div 1000^{\#/k} = 6.83^{k}$; $\frac{\phi V_c}{2} = 3.42^{k}$

$$V_u = \frac{w_u l}{2} = \frac{(0.472^{k/ft} X 19')}{2} = 4.5^{k} > \frac{\phi V_c}{2}, \text{ but} < \phi V_c$$

E.E. ∴ use min stirrups

$\frac{d}{2} = 4.5" \Rightarrow$ use #3 closed ties @ 4" max ∧ ← max spc.

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JOB Gainor Res.
SHEET NO. 10-7 OF 5
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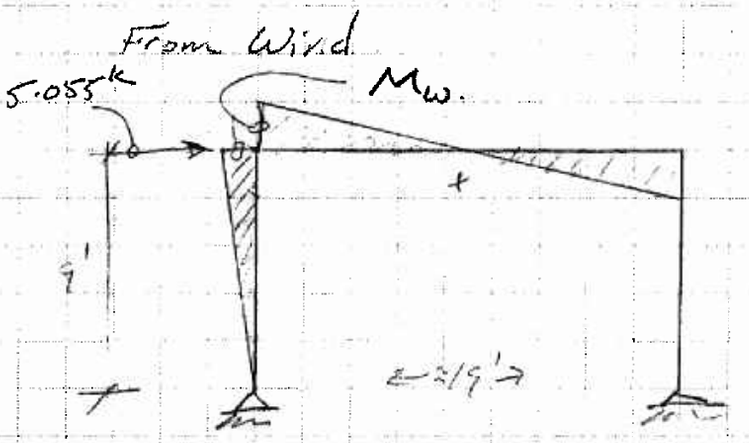
Case 2

(From Wind Alone)

$M_w^{(-)} = \frac{Ph}{2} = -22.75^{k\cdot ft}$



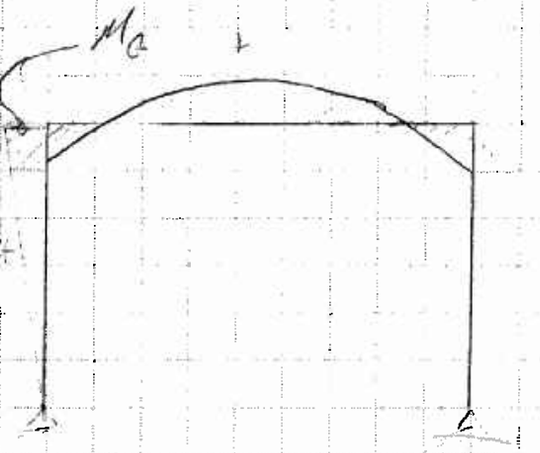
(From Gravity)

D = 10 psf (roof)

L_R = 30 psf

W = -234 #/ft

From Gravity



$w_D = 4.5(10_{psf}) = 45^{\#/ft} + S.W. = 145^{\#/ft}$

$w_{L_R} = 4.5(30_{psf}) = 135^{\#/ft}$

Check ACI 2005 Load Combo's

FBC 2004 Eq #

1.2D + 1.6L_R + 0.8W (16-3)

(1.2)(145) + 1.6(135) − 0.8(234 #/ft) = 203 #/ft ↓ ← Controls @ Top Ends

1.2D + 1.6W + 0.5L_R (16-4)

(1.2)(145) − (1.6)(234 #/ft) + (0.5)(135 #/ft) = −133 #/ft ↑

Controls @ Bott. End.

0.9D + 1.6W = 0.9(145 #/ft) − 1.6(234 #/ft) = −244 #/ft

$M_{u_1}^{(-)} = \frac{w_u l^2}{12} = \frac{(-0.244^{k/ft})(19')^2}{12} = -7.25^{k\cdot ft}$

Total Moment (Wind + Dead + L. - Roof)

$M_{u_B}^{(-)} = M_w^{(-)}(1.6) + M_{u_1}^{(-)} = 22.75^{(k\cdot ft)}(1.6) + 7.25^{k\cdot ft} = 30$ k.ft

$A_{s_{req'd}} \approx \frac{M_u}{\phi j} = \frac{30^{k\cdot ft}}{(.9)(9")} = 0.83^{in^2}$ use 2#6 Bott. - Cont. ← Controls

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JOB Gainor Residence
SHEET NO. 10-8 OF 5
CALCULATED BY LEO DATE 9/27/05
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SCALE _____

Calcs (Cont'd)

Case 2 (cont'd)

@ Top

$M_{u_T}^{(+)} = \frac{w_u l^2}{24} = +6.11^{k\cdot ft} < 14.2^{k\cdot ft} \leftarrow M_{u_{@Top}}$ from Case I

∴ Case I - Top Controls

Shear

$V_D = \frac{M_D}{(\frac{l}{2})} = \frac{+22.75^{k\cdot ft}}{(9.5\,ft)} = 2.4^{k}$

$V_{u_D} = 1.6(2.4^{k}) = 3.84^{k}$

$V_{u_1} = \frac{w_u l}{2} = \frac{(0.24^{k/ft} \times 19')}{2} = 2.32^{k}$

$V_{u_T} = 3.84^{k} + 2.32^{k} = 6.16 < \phi V_c = 6.83^{k}$

use #3 @ 12" Ties @ ½ max

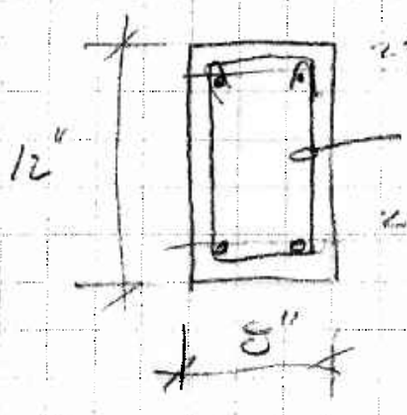
$\phi V_s = \frac{0.75(0.22^{in^2} \times 60^{ksi} \times 9")}{4"} = 22.3^{k}$

$\phi V_n = \phi V_c + \phi V_s = 6.83^{k} + 22.3^{k} = 29.13^{k} \geq V_{u_T}$ ∴ ok

SIDDIQ KHAN & ASSOCIATES, INC.
Consulting Engineers And Planners
7400 S.W. 50th Terrace, Suite 105
MIAMI, FLORIDA 33155
(305) 662-2301

JOB Gainor Res
SHEET NO. 10-9 OF 5
CALCULATED BY LEO DATE 9/27/05
CHECKED BY DATE
SCALE

Summary



x #6" cont w/ STD. 90° HK EE
#3 ties @ 4" max
x #6 cont w/ STD. 90° HK. EE.

Deflection check

$$I_{col} = 0.7 I_g = 0.7\left(\frac{(8"\times 16")^3}{12}\right) = 1911.5 \text{ in}^4$$

$$E = 57{,}000\sqrt{5000} = 4030 \text{ ksi}$$

$$\Delta_{max} = \frac{(P)(\ell^3)}{3EI} \div 2 \text{ for \# of col.} = \frac{(5{,}055^{\#})(9'\times 12")^3}{(3)(4030\times 10^3)(1911.5 \text{ in}^4)} \times \frac{1}{2}$$

$$= 0.14"$$

Development of #6 bar w/ STD. 90° HK

$$l_{dh} = \left(\frac{0.02\beta\lambda f_y}{\sqrt{f'_c}}\right) d_b = \frac{1200(3/4)}{\sqrt{5000}} \simeq 13" < 16 - 2" = 14" \therefore \text{ok}$$

TABLE-INP.1 GENERAL INFORMATION

Project : Gainor Residence		Date : 09/27/05	Time : 11:07:24

File Name : C:\PROGRA~1\PCABEAM\DATA\GAINOR1.BMS

Engineer : Leo		Beam ID : 2nd flr-Moment Frame	
Design Code : ACI 318-89		Run Option	: Design
Unit System : U.S. in-lb		Moment Redistribution	: Considered
Infinite rigid beam-column joints in analysis			: Considered

TABLE-INP.2 MATERIAL PROPERTIES

Beam Concrete Type :	Normal Weight		
Unit Weight	, Wc =	150.0	(pcf)
Compressive Strength	, f'c =	5.0	(ksi)
Tensile Strength	, fct =	473.8	(psi)
Modulus of Rupture	, fr =	530.3	(psi)
Modulus of Elasticity	, Ec =	4286.8	(ksi)
Column Concrete Type :	Normal Weight		
Unit Weight	, Wc =	150.0	(pcf)
Compressive Strength	, f'c =	5.0	(ksi)
Tensile Strength	, fct =	473.8	(psi)
Modulus of Rupture	, fr =	530.3	(psi)
Modulus of Elasticity	, Ec =	4286.8	(ksi)
Reinforcement :			
Yield Strength (Long.)	, fy =	60.0	(ksi)
Yield Strength (Tran.)	, fyv =	60.0	(ksi)
Modulus of Elasticity	, Es =	29000.0	(ksi)

TABLE-INP.3 BEAM GEOMETRY

NUMBER OF SPANS : 3 TYPE : Spandrel EXPOSURE : Exterior

SPAN NUM	TOTAL LENGTH (ft)	MINIMUM DEPTH (in)	TOTAL INERTIA * (in^4)	SECTION TYPE	TOTAL DEPTH (in)	WIDTH LOC.	TOTAL (in)	EFFECTIVE (in)
1	.67	.25	.115200E+04	Rbm Beam	12.00		8.00	8.00
2	19.00	10.86	.115200E+04	Rbm Beam	12.00		8.00	8.00
3	.67	.25	.115200E+04	Rbm Beam	12.00		8.00	8.00

{*} Moment of inertia selected for structural analysis

TABLE-INP.4 COLUMN GEOMETRY

COLUMN NUMBER	STIFFNESS FACTOR (%)		TOTAL HEIGHT (ft)	MOMENT INERTIA (in^4)	SECTION TYPE	COLUMN DEPTH (in)	COLUMN WIDTH (in)
1	100	Top	.0	.000000E+00	Pin	.0	.0
	100	Bot	9.0	.115200E+04	Rect	12.0	8.0
2	100	Top	.0	.000000E+00	Pin	.0	.0
	100	Bot	9.0	.115200E+04	Rect	12.0	8.0

TABLE-INP.5 VERTICAL SUPERIMPOSED AND PARTIAL LOADS

SPAN NUM.	DEAD LOADS TYPE		LOAD	DISTANCE (ft)	LIVE LOADS TYPE		LOAD	DISTANCE (ft)
1	Sup		.0 (psf)		Sup		.0 (psf)	
	Slf		100.0 (plf)					
2	Sup		.0 (psf)		Sup		.0 (psf)	
	Slf		100.0 (plf)					
1	Unf	i	112.5 (plf)	.5	1	Unf	i 135.0 (plf)	.5
		j	112.5	18.5			j 135.0	18.5
3	Sup		.0 (psf)		Sup		.0 (psf)	
	Slf		100.0 (plf)					

Total Unfactored :
Dead Load = 3.859 kips Live Load = 2.430 kips

TABLE-INP.7 MOMENT REDISTRIBUTION PERCENTAGES

JOINT NUM.	MOMENT PERCENTAGE LEFT (%)	RIGHT (%)
1	0	10
2	10	0

10-12

TABLE-INP.8 LOADING COMBINATIONS AND PATTERNS

PATTERN	LOADING COMBINATION	LIVE LOAD DESCRIPTION
1	U1: 1.20(D) + 1.60(75% L)	Two Adjacent Spans Loaded
2	1.20(D) + 1.60(75% L)	Alternate Odd Spans Loaded
3	1.20(D) + 1.60(75% L)	Alternate Even Spans Loaded
4	1.20(D) + 1.60(100% L)	All Spans Loaded

TABLE-INP.9 GENERAL REINFORCEMENT DESIGN PARAMETERS

EXPOSURE : Exterior
TYPE OF STIRRUPS : Closed
LATERAL SUPPORTED : No

REINFORCEMENT		BAR SIZE MIN	BAR SIZE MAX	CLEAR COVER VERTICAL (in)	CLEAR COVER SIDE (in)	BAR SPACING MIN (in)	BAR SPACING MAX (in)
Longitudinal	Top	# 4	# 14	2.00	1.50	1.00	18.00
	Bot	# 4	# 14	2.00	1.50	1.00	18.00
Transverse		# 3	# 5			2.00	24.00

TABLE-ANA.2 LOADING PATTERN MOMENTS AND SHEARS AT SUPPORTS

JOINT NUM.		LOAD PTRN	Mu (ft-k)	Mu/Mo (%)	Vu (kips)	Vu/Vo (%)	LOAD PTRN	Mu (ft-k)	Mu/Mo (%)	Vu (kips)	Vu/Vo (%)
1	Left	1	.0	(100)	.0	(100)	2	.0	(100)	.0	(100)
	Rght		11.0	(100)	3.8	(100)		6.7	(100)	2.3	(100)
	Top		.0	(100)				.0	(100)		
	Bot		-10.9	(100)				-6.7	(100)		
	Left	3	.0	(100)	.0	(100)	4	.0	(100)	.0	(100)
	Rght		11.0	(100)	3.8	(100)		12.4	(100)	4.2	(100)
	Top		.0	(100)				.0	(100)		
	Bot		-10.9	(100)				-12.4	(100)		
2	Left	1	-11.0	(100)	-3.8	(100)	2	-6.7	(100)	-2.3	(100)
	Rght		.0	(100)	.0	(100)		.0	(100)	.0	(100)
	Top		.0	(100)				.0	(100)		
	Bot		10.9	(100)				6.7	(100)		
	Left	3	-11.0	(100)	-3.8	(100)	4	-12.4	(100)	-4.2	(100)
	Rght		.0	(100)	.0	(100)		.0	(100)	.0	(100)
	Top		.0	(100)				.0	(100)		
	Bot		10.9	(100)				12.4	(100)		

Joint Sign Convention : moments positive when clockwise

TABLE-DES.1 REQUIRED LONGITUDINAL REINFORCEMENT AT CRITICAL SECTIONS

SPAN NUM.	CRITICAL SECTION (ft)	EFFECTIVE WIDTH (in)	EFFECTIVE DEPTH (in)		REQUIRED Mu (ft-k)	LOAD PTRN		REQUIRED STEEL AREA (sq.in.)	LAYER DIST. (in)	BAR No.	BAR SIZE
1	.17	8.00	9.15	Top	.0	1		.24 Min	2.8	2	# 4
		8.00	9.15	Bot	.0	0		.24 Min	2.8	2	# 4
2	.50	8.00	9.15	Top	10.3	4		.26	2.8	2	# 4
		8.00	9.15	Bot	2.2	0	Int	.24 Min	2.8	2	# 4
	9.50			Top	2.2	0	Int	.24 Min	2.8	2	# 4
				Bot	8.8	4		.24 Min	2.8	2	# 4
	18.50			Top	10.3	4		.26	2.8	2	# 4
				Bot	2.2	0	Int	.24 Min	2.8	2	# 4
3	.50	8.00	9.15	Top	.0	4		.24 Min	2.8	2	# 4
		8.00	9.15	Bot	.0	0		.24 Min	2.8	2	# 4

TABLE 2: NV358 – 14G Double NVTH Straps with 2 ply 18G seat

Assembly Product Code	14G Strap Product Code	Dimension H (inches)	Total No. of Fasteners in 2 Straps 10d x 5"	Total No. of Fasteners in Seat 10d x 3"	Allowable Loads (lbs)		
					Uplift	L1	L2
NV358-12	NVTH18	12	8	8	2245	1981	1839
NV358-14	NVTH18	14	10	8	2525	2208	2068
NV358-16	NVTH20	16	12	8	2806	2452	2296
NV358-20	NVTH24	20	14	8	3088	2697	2525
NV358-22	NVTH26	22	16	8	3367	2942	2758
NV358-24	NVTH28	24					
NV358-26	NVTH30	26					

TABLE 3: NV458 – 14G Double NVTH Straps with 3 ply 18G seat

Assembly Product Code	14G Strap Product Code	Dimension H (inches)	Total No. of Fasteners in 2 Straps 10d x 5"	Total No. of Fasteners in Seat 10d x 3"	Allowable Loads (lbs)		
					Uplift	L1	L2
NV458-12	NVTH18	12	8	8	2245	2783	2078
NV458-14	NVTH18	14	10	8	2525	3131	2338
NV458-16	NVTH20	16	12	8	2806	3479	2597
NV458-20	NVTH24	20	14	8	3088	3827	2857
NV458-22	NVTH26	22	16	8	3367	4175	3117
NV458-24	NVTH28	24					
NV458-26	NVTH30	26					

TABLE-DES.3 PROPOSED LONGITUDINAL REINFORCEMENT SCHEDULE

SPAN NUM.	LAYER DIST. (in)	CONTINUOUS BARS No.	SIZE	LENGTH (ft)	DISCONTINUOUS BARS No.	SIZE	LENGTH (ft)
1 Top	2.85	2	# 4	.67			
Bot	2.85	2	# 4	.67			
2 Top	2.85	2	# 4	19.00			
Bot	2.85	2	# 4	19.00			
3 Top	2.85	2	# 4	.67			
Bot	2.85	2	# 4	.67			

Cantilivers do not consider reinforcement compatibility at supports

10-16

TABLE-DES.4 PROVIDED FLEXURAL CAPACITIES

SPAN NUM.	SECTION LOCATION (ft)		DESIGN f(Mn) (ft-k)	Mu / f(Mn) (%)	NEUT. AXIS (in)	REINFORCEMENT RATIO	R/Rmax (%)	FACTOR Z (k/in)	Z / Zmax (%)	LAYER DIST. (in)	BARS - No	SIZE
1	.17	Top	19.1	(0)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
		Bot	19.1	(0)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
2	.50	Top	19.1	(53)	1.71	.00546	(29)	.00	(0)	2.85	2	# 4
		Bot	19.1	(11)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
	9.50	Top	19.1	(11)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
		Bot	19.1	(45)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
	18.50	Top	19.1	(53)	1.71	.00546	(29)	.00	(0)	2.85	2	# 4
		Bot	19.1	(11)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
3	.50	Top	19.1	(0)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
		Bot	19.1	(0)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4

Z factor is set equal to zero if service moment is less than cracking moment

10-17

TABLE-DES.5 MOMENT REDISTRIBUTION VERIFICATION

SPAN NUM.	CROSS SECTION	-- REINFORCEMENT RATIOS -- NEGATIVE	POSITIVE	BALANCED	(R-R') / Rb	REDISTRIBUTION PROV'D (%)	ALLOWED (%)
1	Rght Top	.005462	.005462	.037296	.00 Ok	.0	20.0 Ok
2	Left Top	.005462	.005462	.037296	.00 Ok	.0	20.0 Ok

10-18

TABLE-DES.6 REQUIRED TRANSVERSE REINFORCEMENT AT BEAM SEGMENTS

SPAN NUM.	BEAM SEGMENT START (ft)	END (ft)	REQUIRED Vu (kips)	Vu / f(Vc) (%)	LOAD PTRN	REQUIRED Av/s (in)	--- STIRRUPS --- LEGS	SIZE	SPACING (in)	
1	.00	.17	.02 @c	(0)	4	.00000	0	#	0 @ .00	{e}
2	.50	1.26	3.86 @d	(43)	4 R	.00000	0	#	0 @ .00	{e}
	1.26	4.56	3.80	(43)	4 R	.00000	0	#	0 @ .00	{e}
	4.56	7.85	2.01	(22)	4 R	.00000	0	#	0 @ .00	{e}
	7.85	11.15	.67	(7)	4 R	.00000	0	#	0 @ .00	{e}
	11.15	14.44	2.01	(22)	4 R	.00000	0	#	0 @ .00	{e}
	14.44	17.74	3.80	(43)	4 R	.00000	0	#	0 @ .00	{e}
	17.74	18.50	3.86 @d	(43)	4 R	.00000	0	#	0 @ .00	{e}
3	.50	.67	.02 @c	(0)	1	.00000	0	#	0 @ .00	{e}

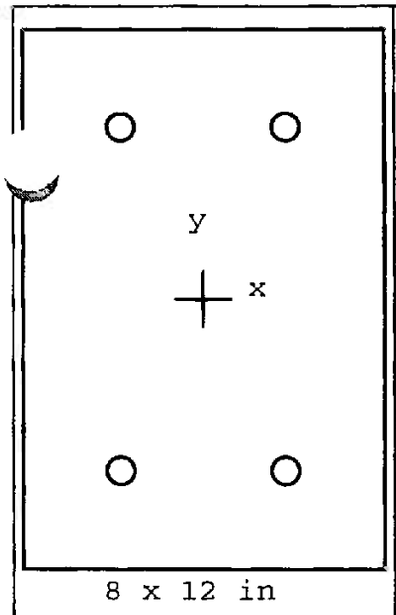
{e} Vu < Vc/2 , transverse reinforcement is not required by the code

TABLE-DES.7 PROVIDED SHEAR REINFORCEMENT AND CAPACITY

SPAN NUM.	BEAM START (ft)	SEGMENT END (ft)	DESIGN f(Vn) (kips)	Vu/ f(Vn) (%)	LOAD PTRN	STIRRUPS LEGS	No.	SIZE	SPACING (in)
1	.000	.170	8.80	(0)	4				
2	.500	18.500	34.48	(11)	4	2 L	54	# 3 @	4.0
3	.500	.670	8.80	(0)	1				

TABLE-DEF.2 MAXIMUM UNIFORM SERVICE LOAD DEFLECTIONS

SPAN NUM.	SECTION	DEFLECTION (in)	Ieff LOC.	Ma (ft-k)	Ma/Mcr (%)	Icr (in^4)	Icr/Igt (%)	Ieff (in^4)
1 TL	.00	-.003828	Midl	.01	(0)	.160156E+03	(13)	.119786E+04
			Rght	.00	(0)	.160156E+03	(13)	.119786E+04
DL	.00	-.002340	Midl	.01	(0)	.160156E+03	(13)	.119786E+04
			Rght	.00	(0)	.160156E+03	(13)	.119786E+04
2 TL	9.97	.058270	Left	7.65	(86)	.160156E+03	(13)	.119786E+04
			Midl	6.47	(73)	.160156E+03	(13)	.119786E+04
			Rght	7.65	(86)	.160156E+03	(13)	.119786E+04
DL	9.97	.035623	Left	4.68	(53)	.160156E+03	(13)	.119786E+04
			Midl	3.95	(44)	.160156E+03	(13)	.119786E+04
			Rght	4.68	(53)	.160156E+03	(13)	.119786E+04
3 TL	.67	-.003828	Left	.00	(0)	.160156E+03	(13)	.119786E+04
			Midl	.00	(0)	.160156E+03	(13)	.119786E+04
DL	.67	-.002340	Left	.00	(0)	.160156E+03	(13)	.119786E+04
			Midl	.00	(0)	.160156E+03	(13)	.119786E+04



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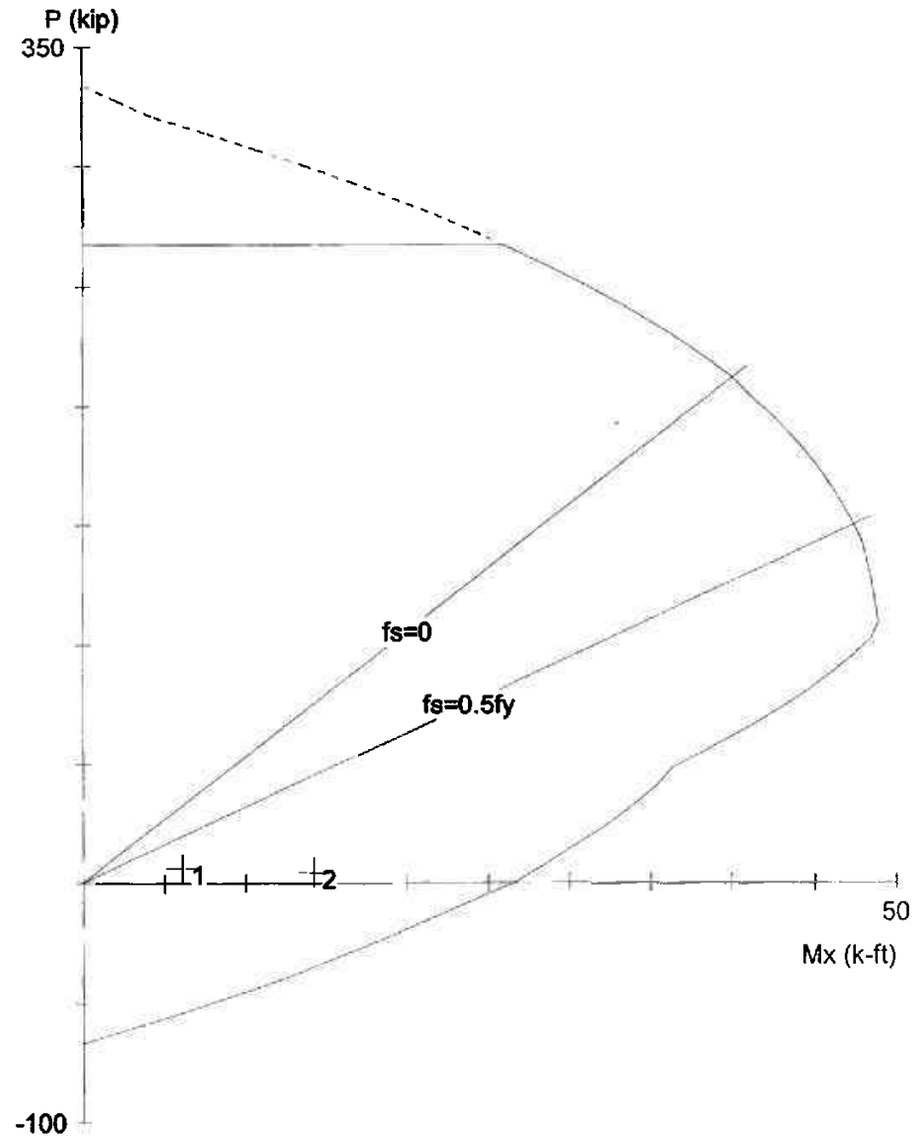

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Design

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 09/27/05

Time: 12:14:47

PCACOL V3.00 (PCA 1999) - Licensed to: SKA, Inc., Miami, FL

File: C:\PROGRA~1\PCACOL\DATA\MC-1.COL

Project: Gainor Residence

Column: Moment (Typ)

Engineer: Leo

f'c = 5 ksi	fy = 60 ksi	Ag = 96 in^2	4 #5 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 1.24 in^2	Rho = 1.29%
fc = 4.25 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 1152 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 512 in^4
Beta1 = 0.8		Clear spacing = 3.00 in	Clear cover = 1.88 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7		

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General Information:

File Name: C:\PROGRA~1\PCACOL\DATA\MC-1.COL
Project: Gainor Residence
Column: Moment (Typ) Engineer: Leo
Code: ACI 318-95 Units: English

Run Option: Design Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:

f'c = 5 ksi fy = 60 ksi
Ec = 4030.51 ksi Es = 29000 ksi
fc = 4.25 ksi Rupture strain = Infinity
Ultimate strain = 0.003 in/in
Beta1 = 0.8

Section:

Rectangular: Width = 8 in Depth = 12 in

Gross section area, Ag = 96 in^2
Ix = 1152 in^4 Iy = 512 in^4
Xo = 0 in Yo = 0 in

Reinforcement:

Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area, As = 1.24 in^2 at 1.29%
4 #5 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	6.2	6.1	28.2	4.611
2	4.5	14.2	27.8	1.956

*** Program completed as requested! ***

10-23

TABLE-INP.1 GENERAL INFORMATION

Project : Gainor Residence		Date : 09/27/05	Time : 11:07:24

File Name : C:\PROGRA~1\PCABEAM\DATA\GAINOR1.BMS

Engineer : Leo Beam ID : 2nd flr-Moment Frame

Design Code : ACI 318-89		Run Option	: Design
Unit System : U.S. in-lb		Moment Redistribution	: Considered

Infinite rigid beam-column joints in analysis : Considered

TABLE-INP.2 MATERIAL PROPERTIES

Beam Concrete Type :	Normal Weight			
Unit Weight	, Wc =	150.0	(	pcf)
Compressive Strength	, f'c =	5.0	(	ksi)
Tensile Strength	, fct =	473.8	(	psi)
Modulus of Rupture	, fr =	530.3	(	psi)
Modulus of Elasticity	, Ec =	4286.8	(	ksi)
Column Concrete Type :	Normal Weight			
Unit Weight	, Wc =	150.0	(	pcf)
Compressive Strength	, f'c =	5.0	(	ksi)
Tensile Strength	, fct =	473.8	(	psi)
Modulus of Rupture	, fr =	530.3	(	psi)
Modulus of Elasticity	, Ec =	4286.8	(	ksi)
Reinforcement :				
Yield Strength (Long.)	, fy =	60.0	(	ksi)
Yield Strength (Tran.)	, fyv =	60.0	(	ksi)
Modulus of Elasticity	, Es =	29000.0	(	ksi)

TABLE-INP.3 BEAM GEOMETRY

NUMBER OF SPANS : 3 TYPE : Spandrel EXPOSURE : Exterior

SPAN NUM	TOTAL LENGTH (ft)	MINIMUM DEPTH (in)	TOTAL INERTIA * (in^4)	SECTION TYPE	TOTAL DEPTH (in)	WIDTH LOC.	TOTAL (in)	EFFECTIVE (in)
1	.67	.25	.115200E+04	Rbm Beam	12.00		8.00	8.00
2	19.00	10.86	.115200E+04	Rbm Beam	12.00		8.00	8.00
3	.67	.25	.115200E+04	Rbm Beam	12.00		8.00	8.00

{*} Moment of inertia selected for structural analysis

TABLE-INP.4 COLUMN GEOMETRY

COLUMN NUMBER	STIFFNESS FACTOR (%)		TOTAL HEIGHT (ft)	MOMENT INERTIA (in^4)	SECTION TYPE	COLUMN DEPTH (in)	COLUMN WIDTH (in)
1	100	Top	.0	.000000E+00	Pin	.0	.0
	100	Bot	9.0	.115200E+04	Rect	12.0	8.0
2	100	Top	.0	.000000E+00	Pin	.0	.0
	100	Bot	9.0	.115200E+04	Rect	12.0	8.0

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TABLE-INP.5 VERTICAL SUPERIMPOSED AND PARTIAL LOADS

SPAN NUM.	DEAD LOADS TYPE		LOAD	DISTANCE (ft)	LIVE LOADS TYPE		LOAD	DISTANCE (ft)
1	Sup		.0 (psf)		Sup		.0 (psf)	
	Slf		100.0 (plf)					
2	Sup		.0 (psf)		Sup		.0 (psf)	
	Slf		100.0 (plf)					
1	Unf	i	412.5 (plf)	.5	1 Unf	i	135.0 (plf)	.5
		j	412.5	18.5		j	135.0	18.5
3	Sup		.0 (psf)		Sup		.0 (psf)	
	Slf		100.0 (plf)					

Total Unfactored :
Dead Load = 9.259 kips Live Load = 2.430 kips

TABLE-INP.7 MOMENT REDISTRIBUTION PERCENTAGES

JOINT NUM.	MOMENT PERCENTAGE LEFT (%)	RIGHT (%)
1	0	10
2	10	0

10-26

TABLE-INP.8 LOADING COMBINATIONS AND PATTERNS

PATTERN	LOADING COMBINATION	LIVE LOAD DESCRIPTION
1	U1: 1.20(D) + 1.60(75% L)	Two Adjacent Spans Loaded
2	1.20(D) + 1.60(75% L)	Alternate Odd Spans Loaded
3	1.20(D) + 1.60(75% L)	Alternate Even Spans Loaded
4	1.20(D) + 1.60(100% L)	All Spans Loaded

TABLE-INP.9 GENERAL REINFORCEMENT DESIGN PARAMETERS

EXPOSURE : Exterior
TYPE OF STIRRUPS : Closed
LATERAL SUPPORTED : No

REINFORCEMENT		BAR SIZE MIN	BAR SIZE MAX	CLEAR COVER VERTICAL (in)	CLEAR COVER SIDE (in)	BAR SPACING MIN (in)	BAR SPACING MAX (in)
Longitudinal	Top	# 4	# 14	2.00	1.50	1.00	18.00
	Bot	# 4	# 14	2.00	1.50	1.00	18.00
Transverse		# 3	# 5			2.00	24.00

10-27

TABLE-ANA.2 LOADING PATTERN MOMENTS AND SHEARS AT SUPPORTS

JOINT NUM.		LOAD PTRN	Mu (ft-k)	Mu/Mo (%)	Vu (kips)	Vu/Vo (%)	LOAD PTRN	Mu (ft-k)	Mu/Mo (%)	Vu (kips)	Vu/Vo (%)
1	Left	1	.0	(100)	.0	(100)	2	.0	(100)	.0	(100)
	Rght		20.4	(100)	7.0	(100)		16.2	(100)	5.5	(100)
	Top		.0	(100)				.0	(100)		
	Bot		-20.4	(100)				-16.1	(100)		
	Left	3	.0	(100)	.0	(100)	4	.0	(100)	.0	(100)
	Rght		20.4	(100)	7.0	(100)		21.8	(100)	7.5	(100)
	Top		.0	(100)				.0	(100)		
	Bot		-20.4	(100)				-21.8	(100)		
2	Left	1	-20.4	(100)	-7.0	(100)	2	-16.2	(100)	-5.5	(100)
	Rght		.0	(100)	.0	(100)		.0	(100)	.0	(100)
	Top		.0	(100)				.0	(100)		
	Bot		20.4	(100)				16.1	(100)		
	Left	3	-20.4	(100)	-7.0	(100)	4	-21.8	(100)	-7.5	(100)
	Rght		.0	(100)	.0	(100)		.0	(100)	.0	(100)
	Top		.0	(100)				.0	(100)		
	Bot		20.4	(100)				21.8	(100)		

Joint Sign Convention : moments positive when clockwise

10-28

TABLE-DES.1 REQUIRED LONGITUDINAL REINFORCEMENT AT CRITICAL SECTIONS

SPAN NUM.	CRITICAL SECTION (ft)	EFFECTIVE WIDTH (in)	EFFECTIVE DEPTH (in)		REQUIRED Mu (ft-k)	LOAD PTRN		REQUIRED STEEL AREA (sq.in.)		LAYER DIST. (in)	BAR - No.	SIZE	
1	.17	8.00	9.15	Top	.0	1		.24	Min	2.8	2	# 4	
		8.00	9.15	Bot	.0	0		.24	Min	2.8	2	# 4	
2	.50	8.00	9.15	Top	18.1	4		.46		2.8	3	# 4	{i}
		8.00	9.15	Bot	3.9	0	Int	.24	Min	2.8	2	# 4	
	9.50			Top	3.9	0	Int	.24	Min	2.8	2	# 4	
				Bot	15.5	4		.39		2.8	2	# 4	{i}
	18.50			Top	18.1	4		.46		2.8	3	# 4	{i}
				Bot	3.9	0	Int	.24	Min	2.8	2	# 4	
3	.50	8.00	9.15	Top	.0	4		.24	Min	2.8	2	# 4	
		8.00	9.15	Bot	.0	0		.24	Min	2.8	2	# 4	

{i} Crack width (Z factor) controls maximum c.c. spacing when Ms > Mcr

10-29

TABLE-DES.3 PROPOSED LONGITUDINAL REINFORCEMENT SCHEDULE

SPAN NUM.		LAYER DIST. (in)	CONTINUOUS BARS No.	SIZE	LENGTH (ft)	DISCONTINUOUS BARS No.	SIZE	LENGTH (ft)
1	Top	2.85	2	# 4	.67			
	Bot	2.85	2	# 4	.67			
2	Top	2.85	3	# 4	19.00			
	Bot	2.85	2	# 4	19.00			
3	Top	2.85	2	# 4	.67			
	Bot	2.85	2	# 4	.67			

Cantilivers do not consider reinforcement compatibility at supports

10-30

TABLE-DES.4 PROVIDED FLEXURAL CAPACITIES

SPAN NUM.	SECTION LOCATION (ft)		DESIGN f(Mn) (ft-k)	Mu / f(Mn) (%)	NEUT. AXIS (in)	REINFORCEMENT RATIO	R/Rmax (%)	FACTOR Z (k/in)	Z / Zmax (%)	LAYER DIST. (in)	BARS - No	SIZE
1	.17	Top	19.1	(0)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
		Bot	19.1	(0)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
2	.50	Top	25.3	(71)	1.92	.00819	(43)	126.33	(87)	2.85	3	# 4
		Bot	19.5	(19)	1.87	.00546	(18)	.00	(0)	2.85	2	# 4
	9.50	Top	25.3	(15)	1.92	.00819	(29)	.00	(0)	2.85	3	# 4
		Bot	19.5	(79)	1.87	.00546	(18)	144.61	(99)	2.85	2	# 4
	18.50	Top	25.3	(71)	1.92	.00819	(43)	126.33	(87)	2.85	3	# 4
		Bot	19.5	(19)	1.87	.00546	(18)	.00	(0)	2.85	2	# 4
3	.50	Top	19.1	(0)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4
		Bot	19.1	(0)	1.71	.00546	(19)	.00	(0)	2.85	2	# 4

Z factor is set equal to zero if service moment is less than cracking moment

TABLE-DES.5 MOMENT REDISTRIBUTION VERIFICATION

NUM.	CROSS SECTION	-- REINFORCEMENT RATIOS -- NEGATIVE	POSITIVE	BALANCED	(R-R') / Rb	REDISTRIBUTION PROV'D (%)	ALLOWED (%)
1	Rght Top	.008194	.005462	.037296	.07 Ok	.0	18.5 Ok
2	Left Top	.008194	.005462	.037296	.07 Ok	.0	18.5 Ok

TABLE-DES.6 REQUIRED TRANSVERSE REINFORCEMENT AT BEAM SEGMENTS

SPAN NUM.	BEAM SEGMENT START (ft)	BEAM SEGMENT END (ft)	REQUIRED Vu (kips)	Vu / f(Vc) (%)	LOAD PTRN	REQUIRED Av/s (in)	--- STIRRUPS --- LEGS	SIZE	SPACING (in)	
1	.00	.17	.02 @c	(0)	4	.00000	0	# 0 @	.00	{e}
2	.50	1.26	6.81 @d	(77)	4 R	.00667 Min	2	# 3 @	4.00	
	1.26	4.56	6.71	(76)	4 R	.00667 Min	2	# 3 @	4.00	
	4.56	7.85	3.55	(40)	4 R	.00000	0	# 0 @	.00	{e}
	7.85	11.15	1.18	(13)	4 R	.00000	0	# 0 @	.00	{e}
	11.15	14.44	3.55	(40)	4 R	.00000	0	# 0 @	.00	{e}
	14.44	17.74	6.71	(76)	4 R	.00667 Min	2	# 3 @	4.00	
	17.74	18.50	6.81 @d	(77)	4 R	.00667 Min	2	# 3 @	4.00	
3	.50	.67	.02 @c	(0)	1	.00000	0	# 0 @	.00	{e}

{e} Vu < Vc/2 , transverse reinforcement is not required by the code

TABLE-DES.7 PROVIDED SHEAR REINFORCEMENT AND CAPACITY

SPAN NUM.	BEAM START (ft)	SEGMENT END (ft)	DESIGN f(Vn) (kips)	Vu/ f(Vn) (%)	LOAD PTRN	STIRRUPS LEGS	STIRRUPS No. SIZE	STIRRUPS SPACING (in)
1	.000	.170	8.80	(0)	4			
2	.500	18.500	34.48	(21)	4	2 L	54 # 3 @	4.0
3	.500	.670	8.80	(0)	1			

10-34

TABLE-DEF.2 MAXIMUM UNIFORM SERVICE LOAD DEFLECTIONS

SPAN NUM.	SECTION	DEFLECTION (in)	Ieff LOC.	Ma (ft-k)	Ma/Mcr (%)	Icr (in^4)	Icr/Igt (%)	Ieff (in^4)
1 TL	.00	-.007137	Midl	.01	(0)	.160156E+03	(13)	.119786E+04
			Rght	.00	(0)	.160156E+03	(13)	.119786E+04
DL	.00	-.005648	Midl	.01	(0)	.160156E+03	(13)	.119786E+04
			Rght	.00	(0)	.160156E+03	(13)	.119786E+04
2 TL	9.97	.239313	Left	14.25	(158)	.221334E+03	(18)	.467092E+03
			Midl	12.05	(136)	.160557E+03	(13)	.576347E+03
			Rght	14.25	(158)	.221334E+03	(18)	.467093E+03
DL	9.02	.112583	Left	11.28	(125)	.221334E+03	(18)	.716905E+03
			Midl	9.54	(107)	.160557E+03	(13)	.999165E+03
			Rght	11.28	(125)	.221334E+03	(18)	.716905E+03
3 TL	.67	-.007137	Left	.00	(0)	.221334E+03	(18)	.119786E+04
			Midl	.00	(0)	.160156E+03	(13)	.119786E+04
DL	.67	-.005648	Left	.00	(0)	.221334E+03	(18)	.119786E+04
			Midl	.00	(0)	.160156E+03	(13)	.119786E+04

TABLE.SUM.1 SUMMARY OF PRINTOUT TABLES

* Program completed as requested *

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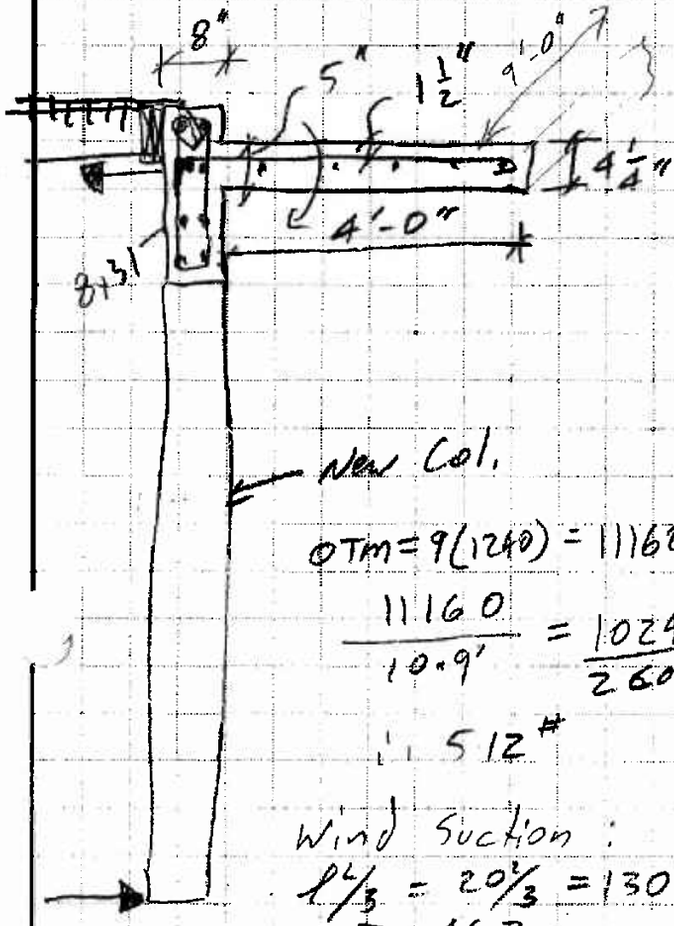
JOB 05-618.00
SHEET NO. 10-37 OF
CALCULATED BY TAK DATE
CHECKED BY DATE
SCALE

Design Bm 2B-1 Second Flood w/ Added Balcony:



Cant. Slab DL = 75 SDL = 20
LL = 60

Serv. $m = 155 \times 4^2/2 = 1240$ #/ft

$M_u = (114 + 96)4^2/2 = 1680$ in-#/ft

A_s Req'd $= \frac{1.68}{4(3.5)} = 0.12$ in²/ft

#5 Development in 8" Bm:

$\frac{1200}{\sqrt{4000}}(.5)(.8) = 6.8"$

Actual $= 8 - 1.5 - .375 = 6.13"$

Req'd $A_s = \frac{6.8}{6.3}(.12) = 0.13$ in²/ft

Provide #4 @ 10" T. E.W.

$\phi T_m = 9(1240) = 11160$

$\frac{11160}{10.9'} = \frac{1024\#}{2601s}$

∴ 512 #

Wind Suction:

$\ell^2/3 = 20^2/3 = 130$ sf

P = 46.7

$F = 46.7\frac{(9)(20)}{2} = 4203$ #/2 sides

$= 2102$ #

512 + 2102 = 2614 #

Use Anchor (5d) (48" Strap Emb Into Conc.) Allow 1598 #

2614/1598 = 1.63 → Need 2 Straps Per Side

Provide 4 Straps Anchor (5d)

Bm. Span = 5' $W_u = 1.2$ k/ft $M_u = 1.2(5)^2/8 = 3.75$ ft-k

V_u @ End $= 1.2(5)/2 = 3$ k

∴ Provide 8x

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Bm 2B-2 : $DL = 1.6^K/ft$ $LL = 1.4\ K/ft$ $l_n = 12'$ $b = 8$ $d = 24"$

$U = 1.2(1.6) + 1.6(1.4) = 4.16\ ^K/ft.$

$V_u = 4.16(6) = 24.96^K$

$M_u = 4.16(12)^2/8 = 74.88^K$ $A_s = \frac{74.9}{4(21)} = 0.9$

$A_{s\,min} = .0035(24)(8) = 0.67$ ∴ Provide 2 #7 T&B

$\phi V_c = 106(8)(21) = 17.81^K$

$s = \frac{.75(.22)60(21)}{8} = 25.98"$ ∴ use $d/2 \cong 10"$ T.O.

-) Shear Fric't Attachment along Edge:

$\mu = 1.0$ $f_y = 60$ $\phi = 0.75$

$24.96 = 0.75(A_{vn})(60)(1.0)$ ∴ $A_{vf} = 0.55 in^2$

Assume Some Fixity @ End: $M_u = 4.16(12)^2/12 = 50^{'-k}$

$A_{s\,req'd} = 0.59$

$A_{s_T} + A_{vf} = .55 + .59 = 1.14$

Provided $= 1.2\ in^2$ ∴ OK.

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RB-2: Max Span 9' Max Trib width 17'

	D	L	Up
Roof	425	510	-800
Bm	100		
	525	510	-800

$U_1 = 1.2(525) + 1.6(510) = 1446$ plf

$U_2 = 0.9(529) - 1.6(800) = -808$ plf.

$+M_u = 1446(8')/8 = 14.64'^{-k}$ $V_u = 5.8^k$

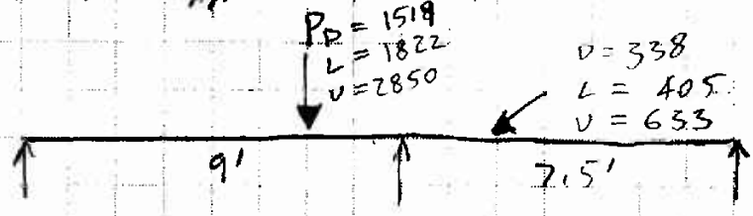
$-M_u = -.808(8')/8 = -8.18'^{-k}$ $\phi V_c = 8.2^k$

Tips #3 5@4½" EE Bal@9"

$A_s = \frac{14.64}{4(9.7)} = 0.38\ in^2$; 2 #5 T.+B. Works.

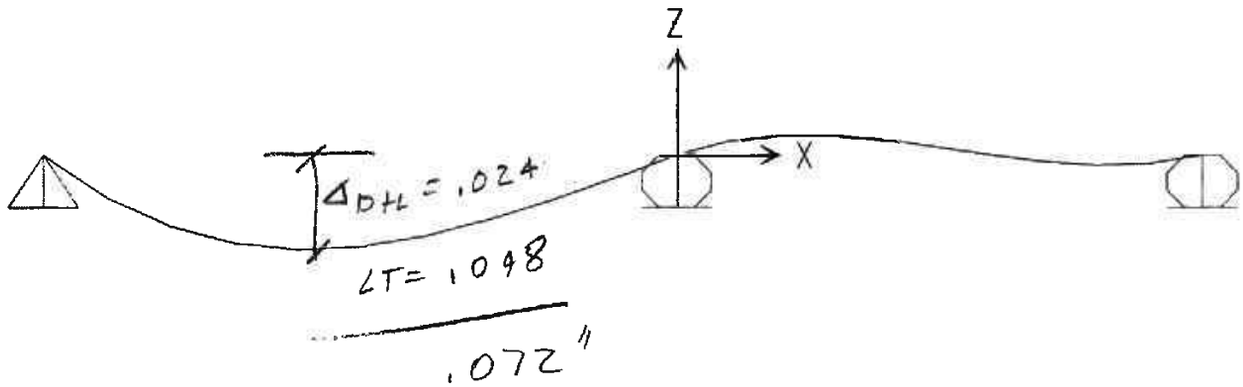
Provide 2 #5 T+B 4S#3 Tips @ 4½" EE Bal@9"

RB-3: 2 span cont. Approx 8' Trib = 13.5'

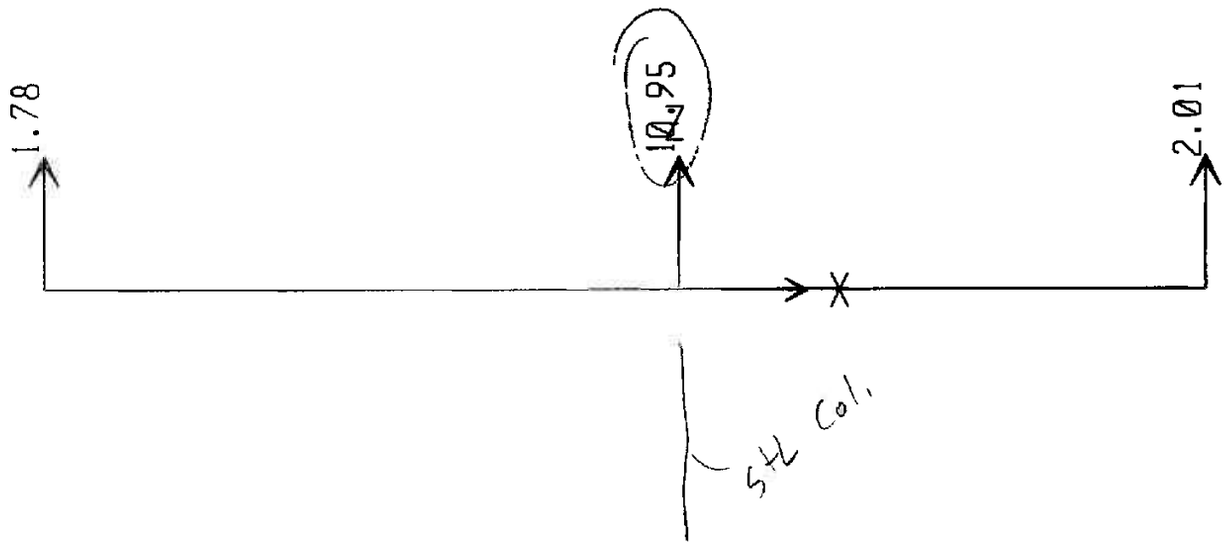


See SAP2K Analysis output.

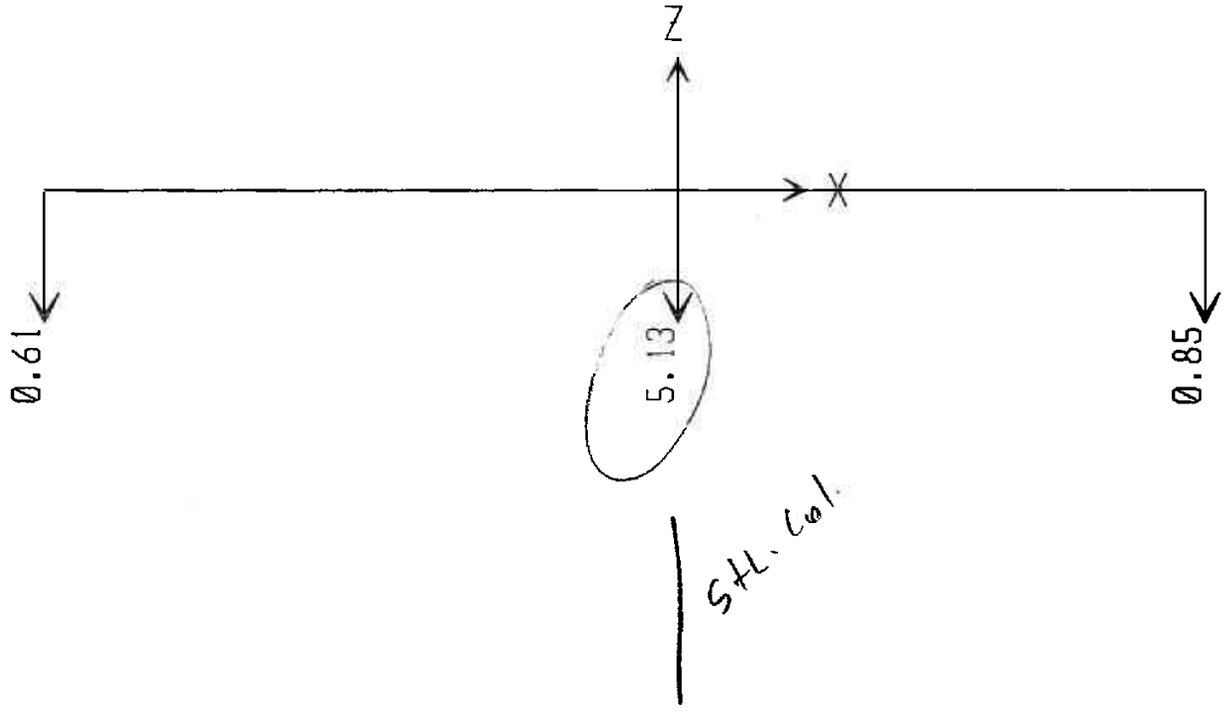
RB-3



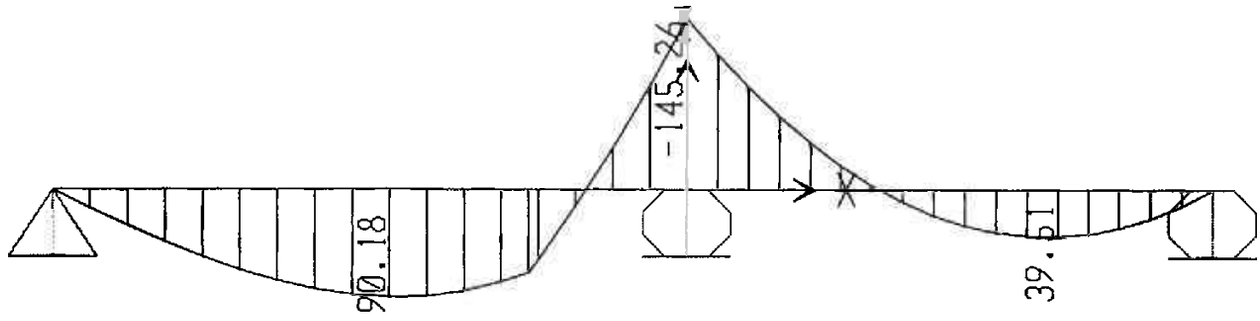
RB-3 Service Reactions (D+L)



Service (0.6D + W)
RB-3

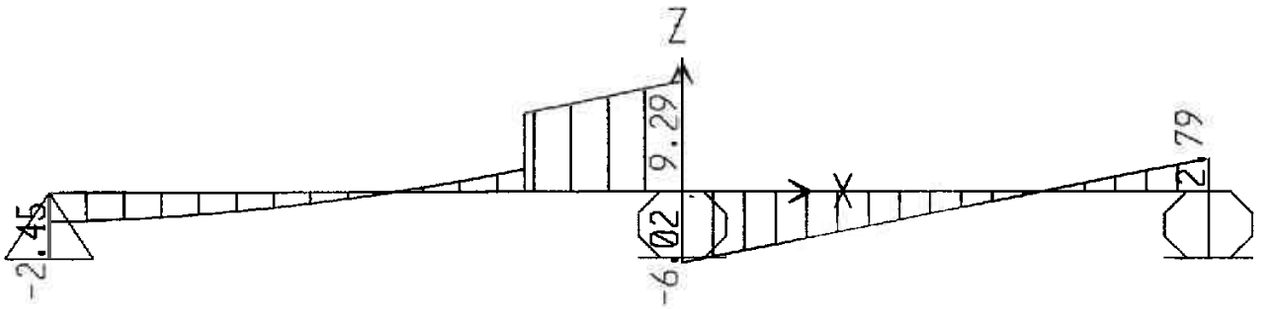


RB-3 Ultimate moment

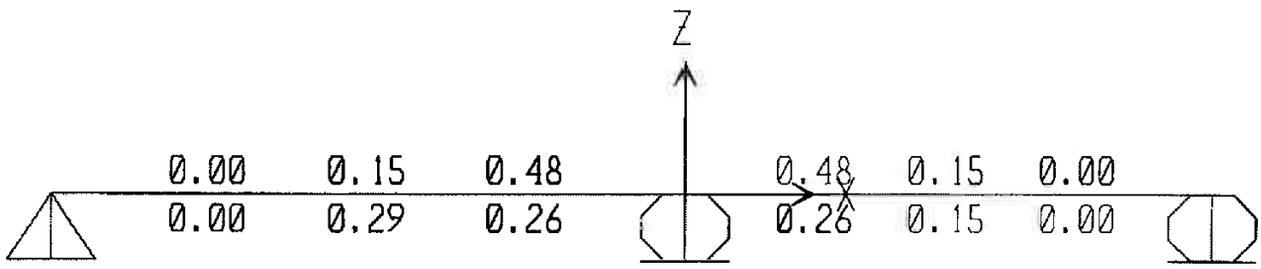


SAP2000 v9.1.6 - File:beam - Moment 3-3 Diagram (DCON2) - Kip, in, F Units

RB-3 Ultimate Shear

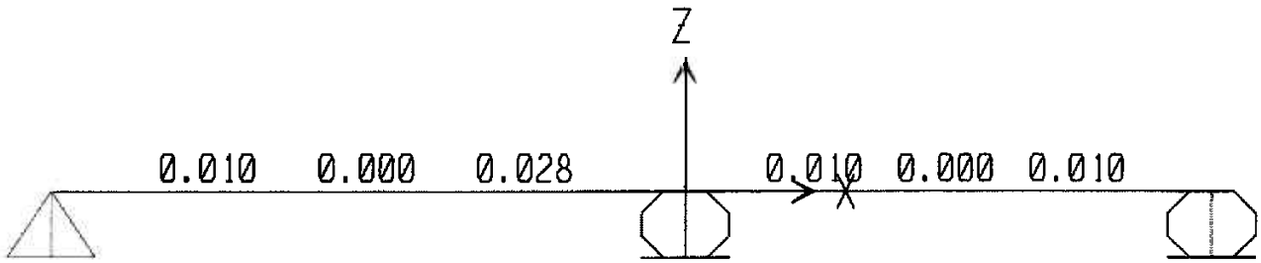


RB-3 Reinf.



2 #5 T&B

RB-3 Shear Reinf.



.028 x 12 = .336

.22/.336 = .65 x 12 = 7.86"

24"/4½" = 5.3 Provide 6 Ties @ 4½" E.E. Bal @ 9"

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Gainor Residence
5800 North Bay Road
City of Miami Beach, Florida

Steel Column Designs

Siddiq Khan & Associates, Inc.
Column Design Per AISC 9th Edition
Gainor Residence 5800 N. Bay Rd.
Job # 05-618.00

Typ. Exterior Steel Column SC-1
HSS 6x2x5/16"
Fy = 46 ksi

Unbraced Length =	9.00	ft
	108.00	in
Ixx =	2.620	in^4 — Iy
Area =	4.360	in^2
k =	1.00	
r =	0.775	in — Ry
Fy =	46	ksi
E =	29000	ksi
kl/r =	139	
Cc =	111.55	
Calculated Fa =	7.690	ksi = F'ex

Allow Axial Load =	33.5 kips

$P = 22.5^K$ $f_a = 5.16$

wind on col $\Rightarrow$ $f^2/3 = 27$

$Z_4 = 52.1$

$$m = \frac{52.1(7.5')(9)^2}{8(1000)} = 4^{1-K}$$

$$f_b = \frac{4(12)}{16} = 3.0\ ksi$$

$F_b = 0.6 F_y = 27.6$

Combined Stresses:

$$\frac{22.5}{33.5} + \frac{3.0}{\left(1 - \frac{5.16}{7.69}\right)27.6} = 1.002 \therefore O.K.$$

Siddiq Khan & Associates, Inc.

Column Design Per AISC 9th Edition
Gainor Residence 5800 N. Bay Rd.
Job # 05-618.00

Typ. Interior Steel Column SC-2

HSS 3.5"x3.5"x1/4"
Fy = 46 ksi

Unbraced Length =	**9.00**	**ft**
	108.00	in
Ixx =	**5.29**	**in^4**
Area =	**3.09**	**in^2**
k =	**1.00**	
r =	**1.31**	**in**
Fy =	**46**	**ksi**
E =	**29000**	**ksi**

kl/r =	**83**	
Cc =	**111.55**	
Calculated Fa =	**17.643**	**ksi**

Allow Axial Load =	**54.5**	**kips**

Actual = 11K ; OK

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Job # 05-618.00

Typ. Exterior Steel Column SC-3

HSS 4x4x1/4"
Fy = 46 ksi

Unbraced Length =	5.50	ft
	66.00	in
Ixx =	8.220	in^4
Area =	3.590	in^2
k =	1.00	
r =	1.510	in
Fy =	46	ksi
E =	29000	ksi

kl/r =	**44**	
Cc =	**111.55**	
Calculated Fa =	**23.515**	**ksi**

Allow Axial Load =	**84.4**	**kips**

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JOB Gainor Residence
SHEET NO. 12-2 OF
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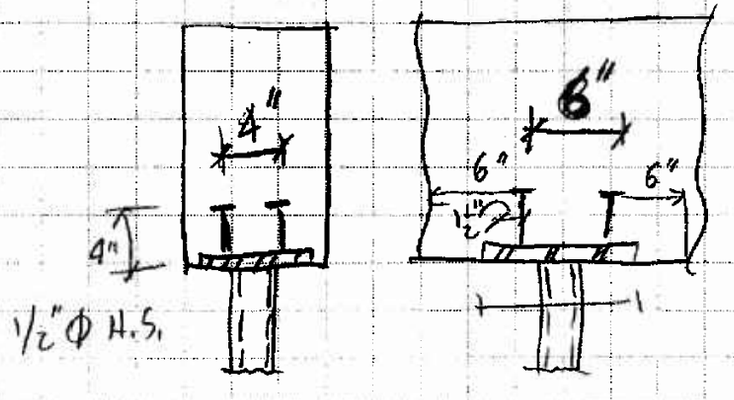
Design of R @ Top of HSS $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$" SC-2

$T = 5.13^K \times 1.6 = 8.21^K$

$C = 10.95^K$



$A_{no} = 9(4)^2 = 144$

$A_n = 18(8) = 144$

$N_b = 24\sqrt{5000}(4)^{1.5} = 13,576^K$

$\psi_1 = 1 \quad \psi_2 = 0.8 \quad \psi_3 = 1.0$

$\phi = 0.75$

Breakout $N_{cb} = \frac{144}{144}(.75)(1.8)(13.576) = 8.14^K$

$\frac{8.21}{8.14} = 1.008$ ∴ Acceptable

Pullout

$\phi N_p = .2(8)(5000)(.75) = 6000 > \frac{8.21}{4} = 2.05$ ∴ OK

Blowout: $N_{sb} = 160(2)\sqrt{0.20}\sqrt{5000} = 10.119^K$

$\phi N_{sbg}\left(1 + \frac{6}{6(2)}\right) 10.119(.75) = 11.4^K > 8.21^K$
∴ OK

Plate Size = 7" × 9" w/ 4 - ½" ϕ H.S.

R Bend, O.H. = 2.75" $P = 10.95/2 = 5.5^K$

$M = 5.5 \times 1.375 = 7.6^{K-"}$

Try 5/8" R $S_x = 0.45$ $f_b = \frac{7.6}{.45} = 16.9$ KSI ∴ OK

use 5/8" THICK R

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(305) 662-2301

JOB Gainor Residence
SHEET NO. 12-3 OF
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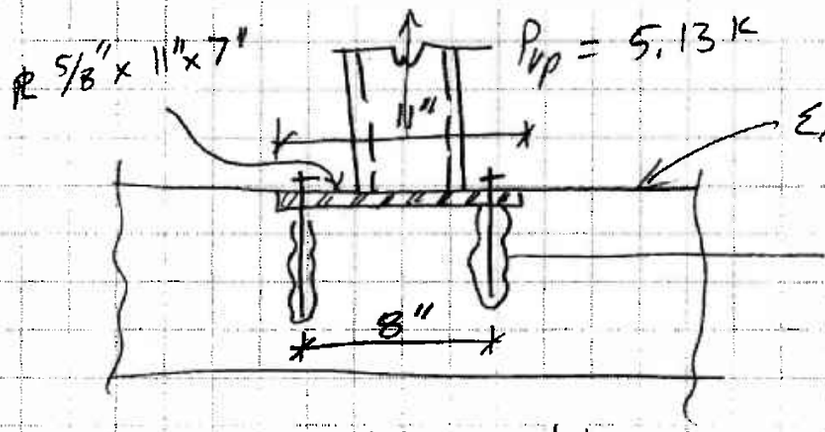
(SC-2)



2 - 5/8" ϕ A193 B7 Threaded Rods w/ 6" Embd Into Conc.

Try 2 - 5/8" ϕ Threaded Rods in Epoxy.

Allow Uplift 6" Embd = 6250 Edge Spac.

Edge Dist L_A = 0.76 Spac F_A = 0.98

Allow = $0.76^2 (.98)(6250)(2) = 7075.6^{\#}$ ∴ OK

TAKE 10% Horiz = 513#

Allow V = $5795 (.98)(.76)^2 (2) = 6560^{\#}$

$\frac{x}{6560} + \frac{5130}{7076} = 1$ ∴ $x = 1804^{\#}$

1804 > 513 ∴ OK

Install 2#5 Vert 2' Apart Centered over Col. In Wall Below.

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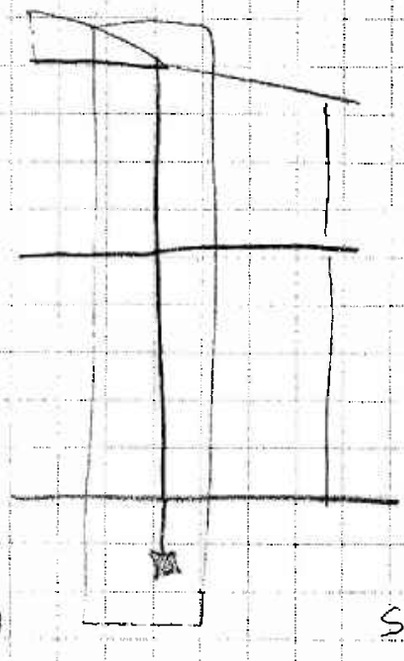
Ext. Wall Receiving Grav. Load For SH. Channels.

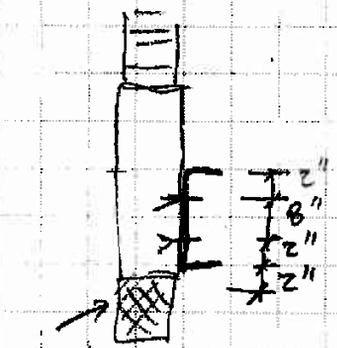


	D	L	U
Roof	500	600	924 #
TB	125		
CMU	500		
2nd	225	800	
Bm	250		
CMU	500		
	2100	1400	

Steel Channel Spans ~ 8' to 10'

$M = (1600 + 1400)(10)^2/8 = 37500$ '-#

Use C12x25 $f_b = \frac{37500(12)}{24.1} = 18672$ psi
< 24 ksi ∴ OK



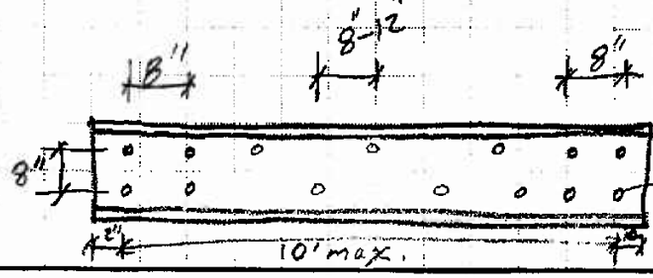
Portion To be Removed.

Design is non-composite

∴ Attach as Follows:

max 30^k using 5/8" Ø Hilti Kwik Bolts.
Both min Edge Dist = 4" Emb'd = 4"
min edge Dist for max = 6"
Spac = 2d = 8" ∴ Reduction = 0.67
Max Allow = 3826(.67) = 2563

30^k/2.56 = 11.7 → min of 12 Kwik Bolts



4 @ End.
In SQ Pattern.

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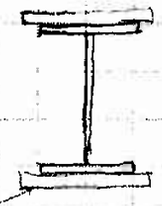
Check Ex. Floor over Garage For
Exercise Eq. Loads: DL = 25 LL = 100

Ex. STL Beam Found W12 $b_f = 5''$ $t_f = 1/4''$ Assume $t_w = \frac{1}{4}$

No Such Bm Found in AISC.



New Coven Rs To Be Installed See Calcs

$A = .25(5) \times 2 + .25(11.5) = 5.38 in^2$

$I_x = \frac{1}{12}(.25)(11.5)^3 + .25(5)(5.875)^2(2) = 118 in^3$

$S_x = \frac{118}{6} = 19.67$

$M_{max} = 19.67(24) = \frac{472.1}{12} = 39.34'-\#$

$\ell = 21'$

$\Delta_{max} = \frac{39.34(21)^2}{161(118)} = 0.91'' < \ell = 276$

Max Trib width $\cong \frac{11.5}{2} + \frac{10.0}{2} \cong 11'$

Load on STL Bm $= 11(125) = 1375^{\#}$

$M = \frac{1375 \times 21^2}{8} = 75.8'-K$

Working Backwards $w = 714^{\#}/ft/11' = 65 psf$
$25 psf$
$40 psf$ max LL.

$S_{x\,req'd} = \frac{75.9}{2} = 37.95 in^3$ Add 1/4" Coven R's 6½" wide

$118 in + 6.5(.125)(6.125)^2(2) = 240 in^4$ or $38.4 in^3$

$38.4 > 37.95$ ∴ OK

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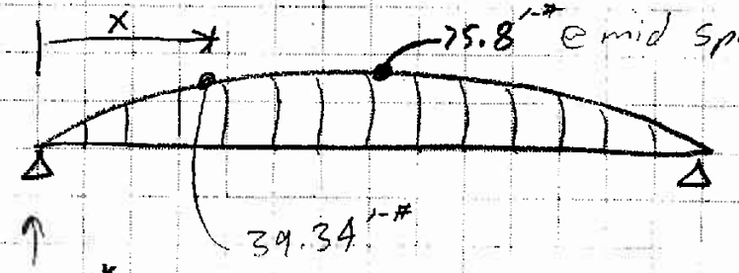
Ex. STL. Bm over Garage Cont.

Check defl.: $\frac{75.8(21)^2}{161(240)} = .865''$ or $\ell/291$.

FBC Allows $\ell/240$ In Floor w/wood/steel Framing. ∴ OK



$R = V = 14.44^K$

$\frac{1.375(x)}{2}(21-x) = 39.34$

$-1.375x^2 + 28.88x - 78.68 = 0$

$\frac{-28.88 \pm \sqrt{28.88^2 - (4)(-1.375)(-78.68)}}{2(-1.375)} =$

$\frac{-28.88 \pm 20.03}{-2.75} = 3.22'$

Allow Shear Capacity
$= .25(12)(36)(.4) = 43^K$
$14.44 < 43^K$ ∴ OK

$21' - 6' = \underline{15'}$ of Cover Pl on Top & Both.

Q of Pls $= 6.5 \times .25 \times 6.125 = 9.95$

$\frac{VQ}{I} = \frac{14.44(9.95)}{240} = .6^K/in$ or $7.2^K/ft$

Weld Capacity $= 1/16(.707)(.3)(70) = 0.93$ $.93(3) = 2.78^K/in$ / 3/16 weld

Check Pl thickness Ratio $= \frac{b}{t} = \frac{6.5}{.25} = 26$ $\frac{190}{\sqrt{36}} = 31.66$ ∴ Compact

Try 3"@12" welds → $16.68^K > 7.2^K$
Try 2"@12" welds → $11.12 > 7.2^K$ ← use 2"@12" 3/16 welds.
on Plans used 2"@8"

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Check Ex Joists @ Exercise Rm. Typ. Span = 10' spa = 16"

Load on Joists = 125 psf x 1.33' = 166 plf. SX = 15.23

Ex Joist Found to Be of Dade Co. Pine

$M = 166(10)^2/8 = 2075$ ft-#

∴ Assume $F_b = 1200$ psi $f_b = \frac{2075(12)}{15.23} = 1621$ psi < 1200 ∴ N.G.

Try adding Joists @ 32" O.C. ∴ 3 Joists in 32"

$M = \frac{125(2.67)(10)^2}{8} = 5540$ ft-#

$f_b = \frac{4172(12)}{3(15.23)} = 1096$ psi < 1200 ∴ OK.

∴ Add 2x8 @ 32" O.C. Between Ex Joist

End Reactions: 556 # $V_H = \frac{3}{2}\frac{556}{12} = 69.5$ psi

< 90 psi

∴ OK

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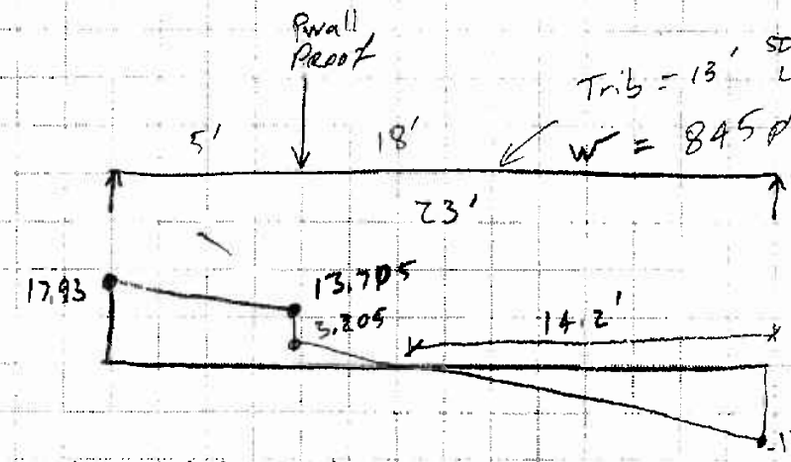
Check STL Bm @ Master Bd Rm (over Living Rm):

Bm Property Field measured: $d = 15''$ $b_f = 5\frac{3}{8}$ $t_f = \frac{1}{2}''$
t_w assumed to be $1/4''$

Closest Bm Match is W16x26



Trib = 13' SDL = 25 LL = 40
w = 845 plf

$P_{wall} = 10 \times 9' \times 13' = 1.2^K$
Roof $P_D = 13^2 \times 25 = 4.23^K$
$P_L = 5.07^K$
$P_U = 7.93^K$
$E_D = 5.43$
$E_L = \frac{5.07^K}{10.5^K}$

Allow $V = .25(14)(.4)(36) = 50^K$
$17.93 < 50^K$ ∴ OK

$M = \frac{14.2(12)}{2} = 85.2^{'-K}$

$f_b = \frac{85.2(12)}{38.4} = 26.63$ KSI

∴ Need to Add Both PL For Add'l Stresses:

Try Adding $3/8'' \times 4\frac{3}{4}''$ PL

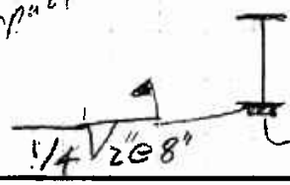
$A = 7.68 + .375(4.75) = 9.46$
N.A. From Existg = $1.78(7.69)/9.46 = 1.45''$ down.
$I_x = 301 + 7.68(1.45)^2 = 317 in^4$
$+ 0 + 1.78(6.24)^2 = 69.3$
$386 in^4$ Critical $S_x = \frac{386}{8.95} = 43.13$

PL $\frac{b}{t} = \frac{4.75}{0.375} = 12.66$

$f_b = 26.63 \times \frac{38.4}{43.1} = 23.7$ KSI $<$ 24 KSI ∴ OK

$\frac{190}{\sqrt{36}} = 31.67$
∴ Compact

$Q = 11.1$ $VQ/I = \frac{17.93(11.1)}{386} = 0.52^K/in$
$= 3.12^K/ft/side$



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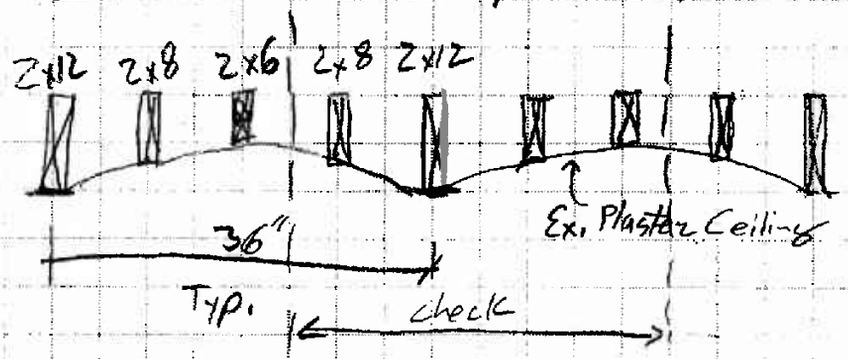
Check Ex. Floor Joist System of Master Bd Rm:



$S_{x12} = 33$
$S_{x8} = 14(2)$
$S_{x6} = 7.6$
68.6

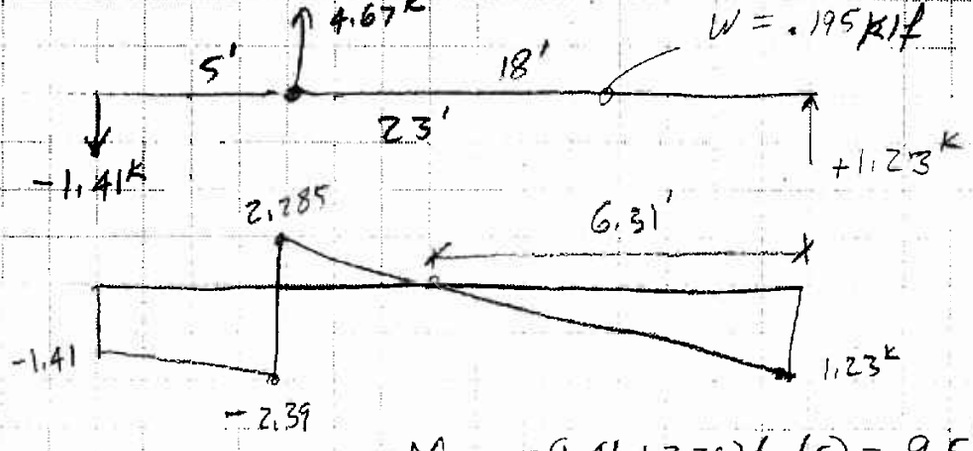
D + L = 25 + 40 = 65 psf
w = 65 x 3' = 195 #/ft

$$M = \frac{195(12')^2}{8} = 3510$$

$$\text{Act } f_b = \frac{3510(12)}{68.6} = 614 \text{ psi} \quad \therefore \text{OK}$$

Check The uplift moment of STL Bm @ Living w/ Added Cover Rm:

0.6D = (1.2 + 4.23) = 3.26 − 7.93 = −4.67 K



$-M_{max} = (1.41 + 2.39)/2\,(5) = 9.5$ ft-K
$+M_{max} = 6.31(1.23)/2 = 3.9$ ft-K

$$f_b = \frac{9.5(12)}{43} = 2.65 \text{ KSI} \quad \therefore \text{OK.}$$

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Design Beam Supporting Stud wall & Roof:

Roof: $55 \times 13' = 715$ #/ft
Wall: $15 \times 9' = 135$ #/ft
850 #/ft
+ 50 # Bm
900 #/ft

Note Floor Joists ⊥ Parallel To Bm.

Span = 13'

$M = \frac{13^2}{8}(.900) = 19'^K$ $S_{x\,req'd} = \frac{19}{2} = 9.5\ in^3$

Need 6" Deep Bm To Fit. Try W6 x15 $S_x = 9.72$

$\Delta = \frac{19(13)^2}{161(29.1)} = 0.69"$

Δ_{max} Wood Floor & Roof $= \frac{l}{240} = 0.65$

Try W6 x 20 $I_x = 41.4$

$\Delta = \frac{29.1}{41.4}(.69) = 0.49 \approx 1/322$; OK

USE W6x20 Bm. Top Flange Bracing

$L_c = \frac{76(6)}{\sqrt{36}} = 76" = 6.33'$

check uplift

What is Allow if unbraced: (uplift cond) $\left(\frac{2}{3} - \frac{36\left(\frac{156}{1.64}\right)^2}{1530000}\right)36 = 16.33$ KSI

$-(46.9 - 10)(13') + .6(15)(9') = -400$ #/ft $m = \frac{.4(13^2)}{8} = 8.5'^K$

$f_b = \frac{8.5(12)}{13.4} = 7.61 < 16.33$ ∴ O.K.

USE W6x20

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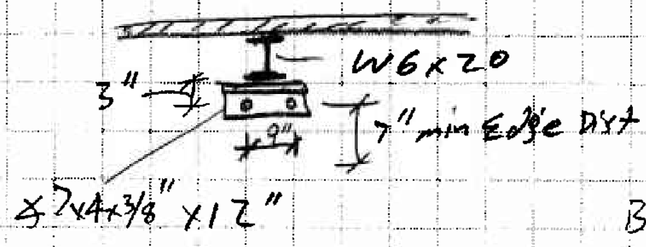
W 6 Bm as Support on Ex. Conc Arch Bm



assume J = 0.9
Jd = 4(.9) = 3.6"

Bm Reaction = .9(13)/2 = 5.85
− .4(13)/2 = −2.60

$M_{\measuredangle} = 5.85(2) = 11.7^{\prime\prime - K}$

$T = 11.7/3.6 = 3.25/2 = 1.625$

$V = 5.85/2 = 2.925$

Allow Capacity Hilti: — V = 3826 T = 2925 Embd = 4"

$\frac{1.625}{2925} + \frac{2925}{3826} = 1.32$ ∴ N.G.

Try w/ 3/4" φ Allow V = 5782 + 3687 Embd = 4 3/4"

$\frac{1625}{3687} + \frac{2925}{5782} = 0.95$ ∴ OK

Spac = 9½"(.95) ≈ 9.0"

Edge Dist = 7"

Uplift: $M_{\measuredangle} = 2.6(2) = 5.2$ Jd = 2.7" T = 1.93^K

$\frac{1930}{3687} + \frac{1300}{5782} = 0.75$ ∴ OK

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New 2-2x10 For Stairs Down to Lower Sec. Flr. Private Client.
Span = 16.5' Trib = 2'
SDL = 25 LL = 40 TDL = 65.

$w = 65 \times 2' = 130$ plf

$M = 130(16.5)^2/8 = 4424$
$V = 130(16.5)/2 = 1073$

$S_x = 42.78$

$f_b = \frac{4424(12)}{42.78} = 1241$

Allow $F'_b = 1200$ psi.
$C_d = 1.0$ $C_m = 1.0$ $C_t = 1.0$
$C_L = 1.0$

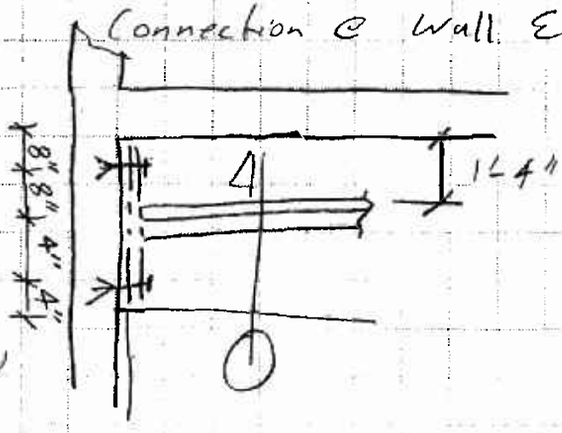
$\frac{1241}{1200} = 1.03\%$ over
∴ Acceptable.

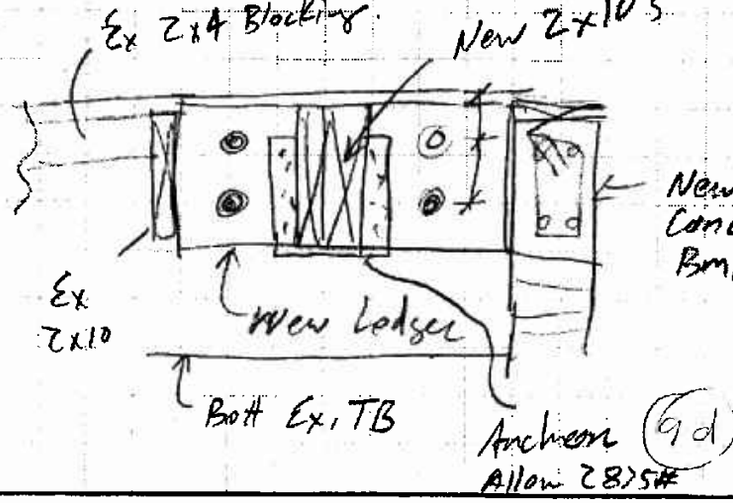
$V = \frac{1.5(1073)}{27.75} = 58$ psi

$58 < 175$ ∴ OK

$\Delta = \frac{5(10,833)(198)^4}{384(1600000)(197.9)} = 0.68'' \approx \ell/291 > \ell/240$ ∴ OK

Connection @ Wall End.





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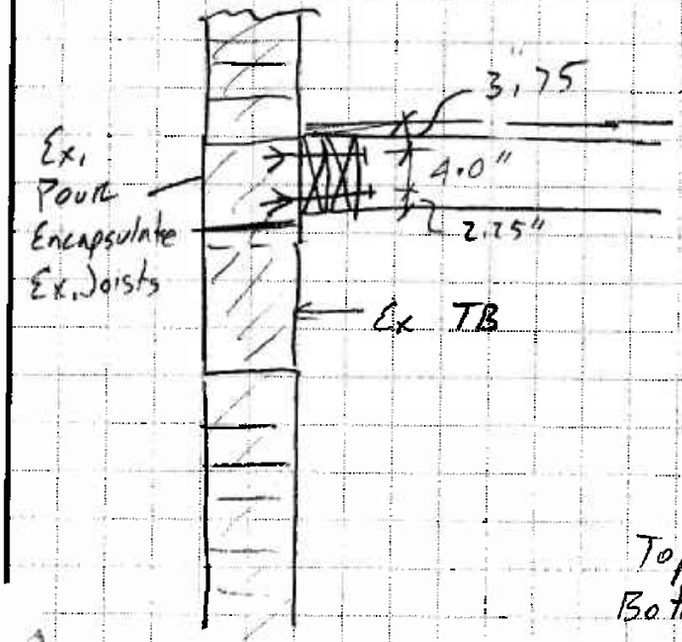
Check Bolts In Wall:



Try 4 - $\frac{1}{2}$" ϕ Kwik Bolts

Spac = 2 Embd = $3\frac{1}{2} \times 2 = 7$"

Edge = 1.5 $(3\frac{1}{2})$ = 5.25"

Top Edge Factor = 0.72

Spac = 0.74

Allow shear = 1962 In Conc.

Top = 1962 (2) * 0.72 (0.74) = 2091 #

Bott = 1962 (2) (0.74) = 2851

4942 #

4942 > 1073 ∴ OK

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Wd Bm Support Trusses:

$l = 7'$ Trib $W = 13'$ $D = 25$ $L = 30$

Area = 46 SF ∴ $W_{up} = -43.7$

$D+L = 55$ $D+W = 10-43.7 = -33.7$

Load on Bm Grav $= 13(55) = 715$ plf

up $= 13(33.7) = 438$ plf.

Bm made of 3-2x10's SYP No 2 $F_b = 1050$ $F_v = 175$

S_x 21.39 $I_x = 98.93$ $E = 1600$ ksi

$C_d = 1.0$
$C_m = 1.0$
$C_t = 1.0$
$C_r = 1.0$
See Attached Run For $C_d = 0.99$

$M = 715(7)^2/8 = 4380$ '-#

$f_b = \frac{4380(12)}{3(21.39)} = 820$ psi

$V = 715(3.5) = 2503$ #

$f_v = \frac{2503(1.5)}{3(1.5)(9.25)} = 90.2$ psi $<$ 175 ∴ OK

$l_u = 84$ $l_e = 1.54(84) = 130"$

From Attached Calc Bm Capacity = 4942 '-# > 4380
∴ OK

Provided Run is Gravity As Unbraced, Therefore uplift Need Not Be checked Since uplift is Less than Gravity.

Siddiq Khan & Associates, Inc.

Column and Beam Adjustment Factors for Per 2001 NDS

Project : **Gainor Residence**

Project No. : **05-618.00**

7' Wood Bm @ Roof
W_G = 715 plf.
ℓe = 130 (84 * 1.54)

Species: **SYP No. 2**

Section and Material:		Beam Data:		Adjustment Factors:		
		ℓe =	130	Cd =	1	
b =	3.5	Kbe =	0.439	C_F (Fb) =	1	
d =	9.25			C_F (Fc) =	1	
Fb =	1200	Column Data:		Cr =	1	
Fc =	1500	Length =	130	Ct =	1	
E =	1600000	Ke_1 =	1.00	Cm (Fb) =	1	(if FbC_F < 1150 Then Cm = 1.0)
		Ke_2 =	1.00	Cm (Fc) =	1	(if FcC_F < 750 Then Cm = 1.0)
		Kce =	0.300	Cm (E) =	1	
		c =	0.80			

Beam Stability Factor C_L:

Rb =	9.9
Fb* =	**1200.0**
E' =	1600000
Fbe =	7155.4
Fbe/Fb* =	5.963
C_L =	0.990
F'b =	**1188**
Beam Cap. =	**4942 ft-#**

Column Stability Factor C_P:

$\ell e/d_1$ =	14.05	if < 50 then OK
$\ell e/d_2$ =	37.14	if < 50 then OK
Fc* =	**1500**	
Fce =	347.9	
Fce/Fc* =	0.232	
C_P =	0.220	
F'c =	**329.4**	

Col Capacity = 10664 lbs

S1-21

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JOB 05-618.00 Garner Residence.
SHEET NO. 12-16 OF
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New G.T. To Wood Stud Wall @ Interior.

Max. Uplift = 4.67^K

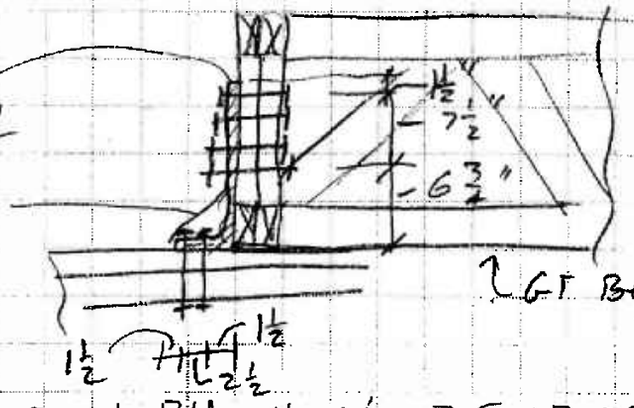
Wood G = 0.55. 5/8" ϕ A307 Thru Bolts.

200^K ∴ C_d = 1.33 C_t = 0.8

$Z' = (1.33)(.8)(1150) = 1224^{\#}$/Bolt.

4670/1224 = 3.82 ∴ use 4 5/8" ϕ Thru Bolts.

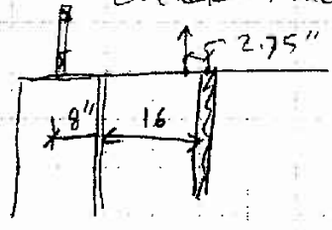
Spac. = 4D = 2½"
End Spac = 5D = 3.125



Bent Plate Height = 3.5 + 3.125 + 3(2.5) + 1.5 = 15.625
Bent Plate width = (1½)2 + 2.5 = 5½"
Add ½" For Radius. 21.625
use 21¾"

Bent PL ¼" x 3" x 21¾"

Check Adj. Stud. 960# ↑ check Reaction on PL Gusset Restrains Rotation.



Reaction to Stud = $\frac{2.75}{16}(4.67) = 803^{\#}$

Uplift = 960/2 = 480 + 803 = 1283#

Anchor (10c) good for 1605# ∴ O/C

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JOB 05-618.00 Gainor Residence
SHEET NO. 12-17 OF
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Anchorage of Wd Bm @ Conc Bm

$V = 2503^{\#}$ (Grav. Controls)

$1.5 \times 3 = 4.5"$

Dbl Shear 1/4" ℄ Table 11G $G = 0.55$

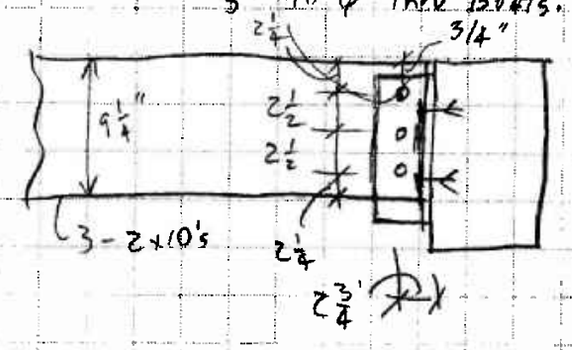
Typ 5/8" Bolts. $Z_\perp = 1420$ 3½"

$Z_\perp = 1510$ 5½"

$Z_\perp = 1465^{\#}$

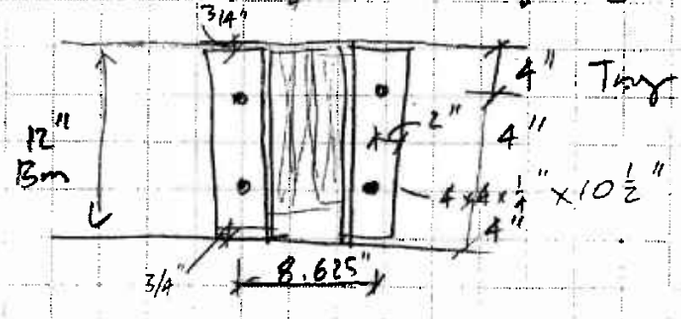
$C_M = 0.96$

$Z' = \left(\frac{2.25}{2.5}(1465)(2) + 1465\right)0.96$



$Z' = 3938^{\#} > 2503^{\#}$ ∴ OK

Use $4 \times 4 \times \frac{1}{4}$ S+L angle (1 Per Side)



Typ 4 - ½" ϕ Kwik Bolts 3½ Embd.

Allow $V = 1962$ > Per NOA

$T = 2041$

Edge Dist Factors:

$T = 0.86$

$V = 0.60$

Spac Dist Factors = 0.74

Shear Thru Bolt Body 0.93

$T_{on} = \frac{2.75(2503)}{(10.5 - 1.5)0.9} = \frac{850^{\#}}{2 \text{ bolts}} = 425$

$S_{hear} = \frac{2503}{4} = 626^{\#}$

Adjusted Allow

$T_{on} = 2041(0.86)(.74) = 1299$

$S_{hear} = 1962(0.60)(.74)(.93) = 810$

$\frac{425}{1299} + \frac{X}{810} = 1 \quad X = 545^{\#}$

$2 \times 545 + 2(810) = 2710^{\#} > 2503^{\#}$ ∴ OK

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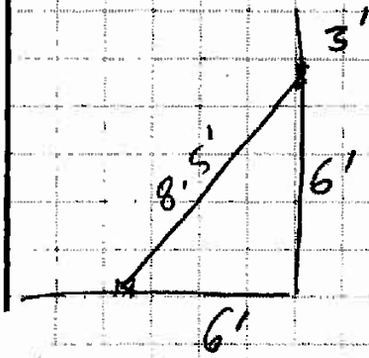
Temp. Wind Brace (For Ext. Walls)

$q = .00256(75)^2(.85)^2 = 10.4$

$P = 10.4(1.3)(.85) = 11.5 \text{ psf}$



Wall Reaction To Diagonal:

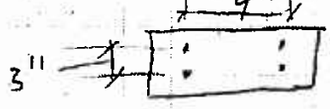
$$\frac{11.5(9)^2}{2(6)} = 77.6 \text{ #/ft.}$$

6' Spac = 465#

Diag Force = $465\sqrt{2} = 657$#

Try Usp Clip & (1) A56, Anchor 148)
Allow Lateral = 735# (From NOA # 01-0417.11)
However Load For withdrawl not Given ∴ Calculate:
For Nails Loaded in withdrawl:
6 Nails 1.5" Penetration
40#/in $40 \times 1.5 = 60$# $C_d = 1.33$
$W' = 60 \times 1.33 = 80$#
$6 \times 80 = 480\# > 465$ ∴ 6' O.K.

Attachment of Wood To wall w/ 2x6 Ledger



Try 2 - 1/4 Tapcons. T = 195 V = 264

Try 4 Tapcons $\frac{465}{195(4)} + \frac{465}{264(4)} = 1.03$ ∴ Acceptable.

Use (4) 1/4 Tapcons 1 1/4" Embed.

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Temp Wood Brace Attach To Floor:

2x6 Cont Wood R.

Attach W- (2) 10-d Nails Per Ea. Floor Joist.

$6' = \frac{72''}{16''} \div 4.5$ Joists Reduce Spac To 4

$4(16) = 64'' = \underline{5'-4''}$

4 - Joists w/ 2 - 10d Nails

Horiz = Vert = 414#/8 = 52# Shear & Tension.

$Z'_{\perp} = \frac{(101)(60)(1.33)}{[.707(60) + .707(101)]1.33} = 53^{\#} > 52$; OK.

Siddiq Khan & Associates, Inc.

Column and Beam Adjustment Factors for Per 2001 NDS

Project : **Gainor Residence**

Project No. : **05-618.00**

Species: SYP No. 3 Stud, Temp Bracing

Section and Material:		Beam Data:		Adjustment Factors:		
		ℓe =	102	Cd =	1.25	
b =	1.5	Kbe =	0.439	C_F (Fb) =	1	
d =	3.5			C_F (Fc) =	1	
Fb =	850	Column Data:		Cr =	1	
Fc =	975	Le_1 =	102	Ct =	1	
E =	1400000	Le_2 =	51	Cm (Fb) =	1	(if FbC_F < 1150 Then Cm = 1.0)
		Ke_1 =	1.00	Cm (Fc) =	0.8	(if FcC_F < 750 Then Cm = 1.0)
		Ke_2 =	1.00	Cm (E) =	0.9	
		Kce =	0.300			
		c =	0.80			

Beam Stability Factor C_L:		**Column Stability Factor C_P:**		
Rb =	12.6	$\ell e/d_1$ =	29.14	if < 50 then OK
Fb* =	**1062.5**	$\ell e/d_2$ =	34.00	if < 50 then OK
E' =	1260000	**Fc* =**	**975**	
Fbe =	3486.2	Fce =	327.0	
Fbe/Fb* =	3.281	Fce/Fc* =	0.335	
C_L =	0.979	C_P =	0.308	
F'b =	**1040**	**F'c =**	**300.3**	

Beam Cap. = **266 ft-#** **Col Capacity =** **1576 lbs** > 657 ∴ OK

02-21

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JOB Gainor Res.
SHEET NO. 12-21 OF
CALCULATED BY TAK DATE
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SCALE

Increased opening @ Living / Conservatory.

	D	L	U
H. Roof	75	90	141
Wall	650		
L. Roof	175	210	328
Bm	200		
2nd Flr	50	80	
	1150	380	469

D+L = 1530
0.6D+W = +221 (No Uplift)

Largest Span = 12' $M = 1.53(12)^2/8 = 27.54'^K$

$V = 1.53(12)/2 = 9.18^K$

Try adding Stl ℄ to Bm.
Try ½ ℄.

$n = \frac{29}{3.12} = 9.3$

$I_c = \frac{1}{12}(8)(24)^3 = 9216$

$I_{stl} = \frac{1}{12}\frac{1}{2}(12)^3 = 72$

$I_{stl_T} = 72(9.3) = 669.6$

		A	$\bar{y}$	
Centroid =	C	192	× 12	= 2304
	S	6(9.3)	× 18	= 1004
		247.8		3308

$\bar{y} = 13.5"$

$I_T = 9216 + (192)(1.5)^2 + 669.6 + 55.8(4.5)^2 = 11448$

$Q = 6(9.3)(4.5) = 251$

$f_{Top} = \frac{27.54(12)13.5}{11448} = 0.39^{KSI}$

$f_{Bot\ Conc} = \frac{27.54(12)(10.5)}{11448} = 0.303^{KSI}$

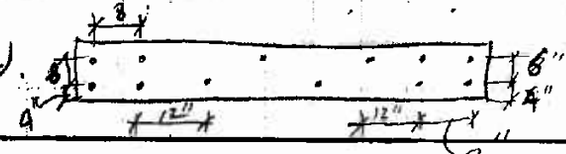
Slow flow = $\frac{9.18(251.1)}{11448} = .201^{K/in}$

$f_{Bot\ Stl} = .303(9.3) = 2.82^{KSI}$

∴ Bolts @ 12" → .201 × 12 = 2.41^K

Allow Hi Hi Knk Bolts In Conc. = 3826 × .7 × .93 = 2490 ∴ OK

∴ Provide @ 12" Staggered.



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JOB Gainor Res.
SHEET NO. 12-22 OF
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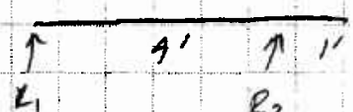
Balcony Wood Designs:

Out lookers 5' span ; DL = 25 LL = 30 Up = 95 psf
Net uplift use 90
2' spac.



Uplift
$R_2 = \frac{90(5)^2(2)}{(2)(4)} = 563^{\#}$

Gravity: $563 \times \frac{55}{90} = 344$

Span on 4x8 UD Bm = 8'

$W = 90 \times 3' = 270^{\#}/ft$ $M = 270 \times 8^2/8 = 2160$
$S_x = 30.67$ $f_b = \frac{2160(12)}{30.67} = 845$ psi
Allow = 1050 (1.33) = 1396 psi > 845 ; OK

$V = 270 \times 4' = 1080^{\#}$ $f_v = \frac{1.5(1080)}{34.4} = 41$ psi
; OK

Lag Screw into Post End Grain:

$7\frac{1}{4}" - 2"$ Recess w/ Plate $= 5\frac{1}{4}"$

Try 5/8" ϕ $W = 453^{\#}/in$ $C_{eg} = 0.75$ $C_d = 1.33$
$W' = 451^{\#}/in$

$\frac{1080}{451} = 2.39 \approx 2\frac{1}{2}"$ $5\frac{1}{4} + 2\frac{1}{2} = 7.75"$

Use 5/8" ϕ x 8" Screw w 3" Embd.

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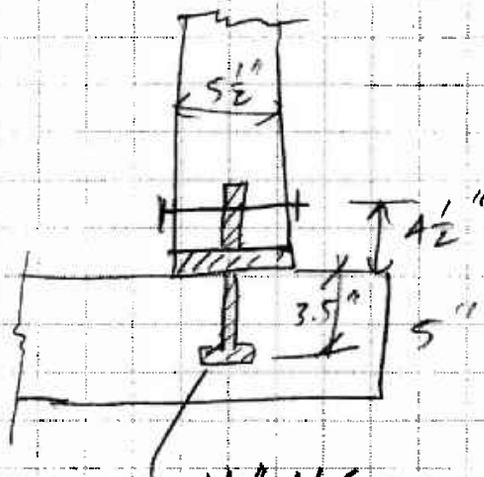
Post Attachment @ Both.



$C_d = 1.33$
$C_m = 0.7$

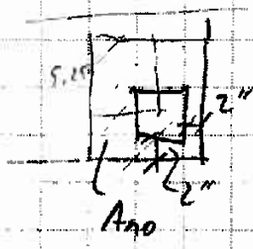
$Z' = 980^{\#}$ ($2\frac{1}{2}$" width)

$Z' = 980 \times 2 = 1960^{\#} \times .70 \times 1.33 = 1824^{\#}$

$1824 > 1080^{\#}$; OK

$\phi = 0.7$

$P_u = 1080(1.6) = 1728^{\#}$



$A_n = 9(3.5)^2 = 110$

$N_b = 24\sqrt{5000}\ 3.5^{1.5} = 11112^{\#}$

$A_{no} = 7.25^2 = 53$

$\phi N_{cb} = 11.112^{K}(.7)\left(\frac{53}{110}\right) = 3.74^{K} > 1.73^{K}$; OK

Rafter Attachment to 4x8 Beam

Use ½ Φ Lag Screw $V = 563^{\#}$
$W = 334^{\#}$ $C_d = 1.33$
$\frac{563}{334(1.33)} = 1.26$" Provide 2" min.

Provide ½" Φ x 8" Lag Screw in Recessed Hole in Rafter. w/ min of $2\frac{1}{2}$" Embd.

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JOB Gainor Residence
SHEET NO. 12-24 OF
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Exist.
2nd flr. Beams (2x10's @ 16" o/c)
Cap. Check.

Given:

Exist
Beam = 2x10's @ 16" o/c

DL = 25 psf

LL = 40 psf

Span ≤ 18' (max)

Wood Species: Use SYP #2

Find: Check Exist'g Member Capacities.

Formula: FBC 2004 & NDS 2001 (ASD)

Calcs:

$w_{DL} = 1.33'(25\ \#/ft^2) \approx 33.3\ \#/ft$ use 34 #/ft

$w_{LL} = 1.33'(40\ \#/ft^2) = 53.2\ \#/ft$ use 54 #/ft

$\Delta_{DL+LL} < \frac{l}{240}$ @ E = 1700 ksi ... ok

Title :
Dsgnr:
Description :

Job #
Date: 10:17AM, 18 OCT 05

Scope :

Timber Beam & Joist

Description Gainor Res. Existing Floor Beams @ 2nd flr

Timber Member Information

Calculations are designed to 1997 NDS and 1997 UBC Requirements

		Span 1
Timber Section		**2x10**
Beam Width	in	1.500
Beam Depth	in	9.250
Le: Unbraced Length	ft	0.00
Timber Grade		
Fb - Basic Allow	psi	12,000.0
Fv - Basic Allow	psi	175.0
Elastic Modulus	ksi	1,700.0
Load Duration Factor		1.000
Member Type		Manuf/Pine
Repetitive Status		Repetitive

Center Span Data

Span	ft	16.00
Dead Load	#/ft	34.00
Live Load	#/ft	54.00

Results

Ratio = 0.3932

Mmax @ Center	in-k	33.79
@ X =	ft	8.00
fb : Actual	psi	1,579.8
Fb : Allowable	psi	12,000.0
		Bending OK
fv : Actual	psi	68.8
Fv : Allowable	psi	175.0
		Shear OK

Reactions

@ Left End	DL	lbs	272.00
	LL	lbs	432.00
	Max. DL+LL	lbs	704.00
@ Right End	DL	lbs	272.00
	LL	lbs	432.00
	Max. DL+LL	lbs	704.00

Deflections

Ratio OK

Center DL Defl	in	-0.298
L/Defl Ratio		644.1
Center LL Defl	in	-0.473
L/Defl Ratio		405.5
Center Total Defl	In	-0.772
Location	ft	8.000
L/Defl Ratio		248.9

12-25

Title :
Dsgnr:
Description :

Job #
Date: 10:47AM, 18 OCT 05

Scope :

Timber Beam & Joist

Description Gainor Res. Existing Floor Beams @ 2nd flr

Timber Member Information

Calculations are designed to 1997 NDS and 1997 UBC Requirements

		Span 1
Timber Section		**2x10**
Beam Width	in	1.500
Beam Depth	in	9.250
Le: Unbraced Length	ft	0.00
Timber Grade		
Fb - Basic Allow	psi	12,000.0
Fv - Basic Allow	psi	175.0
Elastic Modulus	ksi	1,600.0
Load Duration Factor		1.000
Member Type		Manuf/Pine
Repetitive Status		Repetitive

Center Span Data

Span	ft	16.00
Dead Load	#/ft	34.00
Live Load	#/ft	54.00

Results

Ratio = 0.3932

Mmax @ Center	in-k	33.79
@ X =	ft	8.00
fb : Actual	psi	1,579.8
Fb : Allowable	psi	12,000.0
		Bending OK
fv : Actual	psi	68.8
Fv : Allowable	psi	175.0
		Shear OK

Reactions

@ Left End	DL	lbs	272.00
	LL	lbs	432.00
	Max. DL+LL	lbs	704.00
@ Right End	DL	lbs	272.00
	LL	lbs	432.00
	Max. DL+LL	lbs	704.00

Deflections

Ratio > 240 !

Center DL Defl	in	-0.317
L/Defl Ratio		606.2
Center LL Defl	in	-0.503
L/Defl Ratio		381.7
Center Total Defl	in	-0.820
Location	ft	8.000
L/Defl Ratio		234.2

close enough
∴ ok

12-26

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JOB Gainor Res.
SHEET NO. 12-27 OF 1
CALCULATED BY Leo DATE 10/18/05
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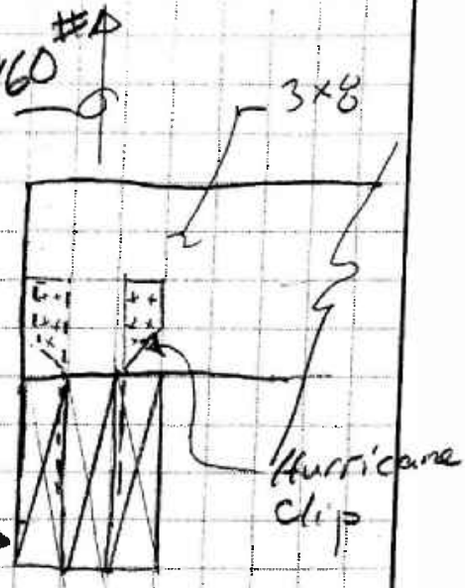
Hurricane Anchorage

@ Exist. 3x8 Roof Beams

Given:

Graph:

Uplift = 960#



See Graph →

Find: Hurricane Clip

Formula: FBC 2004

Calcs:

Req. Uplift Cap = 960# (min)

Try Nu-Vue Hurricane Clip ⟹ 525# Allow. per Clips

Use (2) Nu-Vue "H" clips = 1050# > 960# ∴ ok

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JOB Gainor Res.
SHEET NO. 12-28 OF 2
CALCULATED BY Leo DATE 10/18/05
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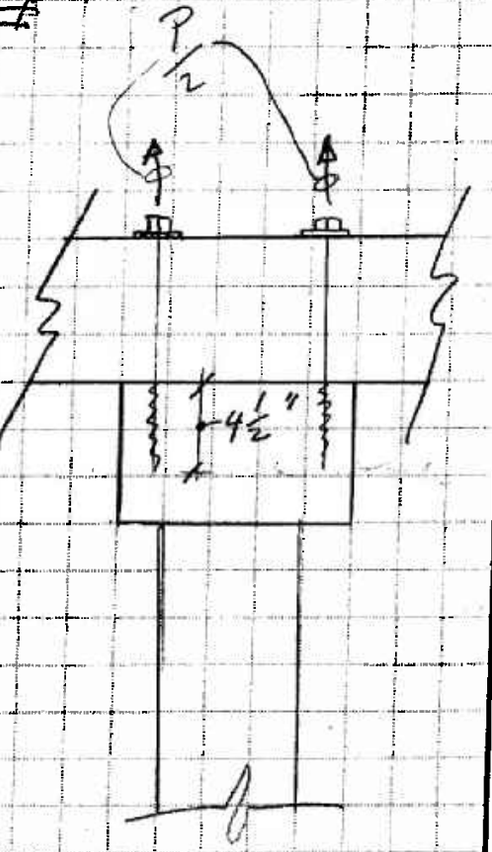
Given:

$P = 90\ psf \times (8.5' \times 7.75')$
$= 5929\#$ say $6000\#$

Reaction per bolt $= \frac{6000\#}{2} = 3000\#$
↑ # of bolts

Graph:



Find: ∅ of bolt

Ref: FBC 2004 & NDS 2001 (ASD)

Calcs:

From Table 11A of NDS 2001:

5/8" ∅ Lag bolts (@ G=0.55) $= 516\#/in$ of pen.

Stress Conc.

∴ Cap. of (1) 5/8" ∅ Lag bolt in withdrawal $= 516\#/in\ (4.5")(1.33)$
$= 3088\#/bolt > 3000\#$
↑ w/ 1.33 stress inc.

∴ ok

11.2 Withdrawal Design Values

11.2.1 Lag Screws

11.2.1.1 The withdrawal design values, in lbs./in. of penetration, for a single lag screw inserted in side grain, with the lag screw axis perpendicular to the wood fibers, shall be determined from Table 11.2A or Equation 11.2-1, within the range of specific gravities and screw diameters given in Table 11.2A. Tabulated nominal design values, W, shall be multiplied by all applicable adjustment factors (see Table 10.3.1) to obtain allowable design values, W'.

$$W = 1800 G^{3/2} D^{3/4} \qquad (11.2\text{-}1)$$

11.2.1.2 When lag screws are loaded in withdrawal from end grain, nominal withdrawal design values, W, shall be multiplied by the end grain factor, $C_{eg} = 0.75$.

11.2.1.3 When lag screws are loaded in withdrawal, the allowable tensile strength of the lag screw at the net (root) section shall not be exceeded (see 10.2.3).

Table 11.2A Lag Screw Withdrawal Design Values (W)[1]

Tabulated withdrawal design values (W) are in pounds per inch of thread penetration into side grain of main member. Length of thread penetration in main member shall not include the length of the tapered tip (see Appendix L).

Specific Gravity	Lag Screw Unthreaded Shank Diameter, D										
G	1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1-1/8"	1-1/4"
0.73	397	469	538	604	668	789	905	1016	1123	1226	1327
0.71	381	450	516	579	640	757	868	974	1077	1176	1273
0.68	357	422	484	543	600	709	813	913	1009	1103	1193
0.67	349	413	473	531	587	694	796	893	987	1078	1167
0.58	281	332	381	428	473	559	641	719	795	869	940
0.55	260	307	352	395	437	516	592	664	734	802	868
0.51	232	274	314	353	390	461	528	593	656	716	775
0.50	225	266	305	342	378	447	513	576	636	695	752
0.49	218	258	296	332	367	434	498	559	617	674	730
0.47	205	242	278	312	345	408	467	525	580	634	686
0.46	199	235	269	302	334	395	453	508	562	613	664
0.44	186	220	252	283	312	369	423	475	525	574	621
0.43	179	212	243	273	302	357	409	459	508	554	600
0.42	173	205	235	264	291	344	395	443	490	535	579
0.41	167	198	226	254	281	332	381	428	473	516	559
0.40	161	190	218	245	271	320	367	412	455	497	538
0.39	155	183	210	236	261	308	353	397	438	479	518
0.38	149	176	202	227	251	296	340	381	422	461	498
0.37	143	169	194	218	241	285	326	367	405	443	479
0.36	137	161	186	209	231	273	313	352	389	425	460
0.35	132	156	179	200	222	262	300	337	373	407	441
0.31	110	130	149	167	185	218	250	281	311	339	367

1. Tabulated withdrawal design values (W) for lag screw connections shall be multiplied by all applicable adjustment factors (see Table 10.3.1).

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Generator Pad Structure Design

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Consulting Engineers And Planners
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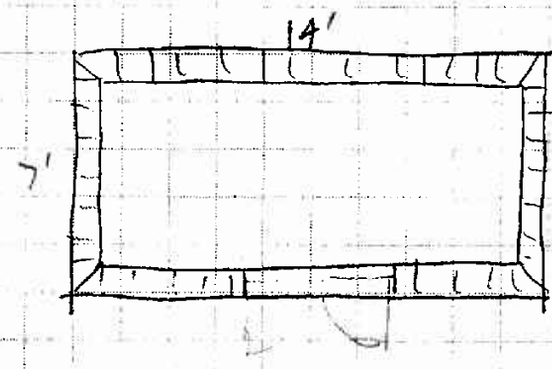
MIAMI, FLORIDA 33155
(305) 662-2301

JOB Kainor Res.
SHEET NO. 13-2 OF
CALCULATED BY TAK DATE
CHECKED BY DATE
SCALE

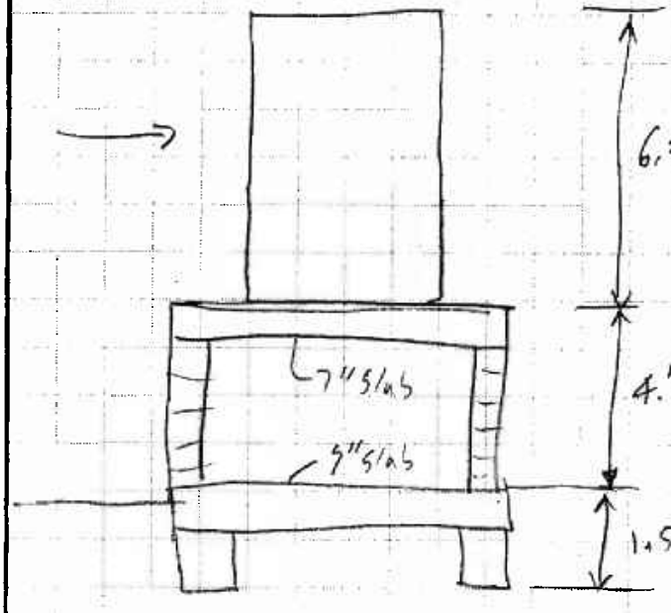
Generator Support Structure:

Pin Pile Capacity
5 Tons Comp = 10k
2 Tons Tens = 4k
1 Ton Lateral = 2k

Pad Size:



Gen. Size = 143" x 60" x 76" (height)
wt = 6053#



Wind Loads:

$q = .00256(146)^2(.85)^2 = 39.4$

$P = 39.4(.85)(1.3) = 43.5$ psf

$OTM/Ft = \frac{10.33^2(43.5)}{2} = 2321'^{-\#}/ft$
$\times 14' = 32500$

ROTM:		
Gen	6053	3.5'
Slab	8820	3.5'
Slab	8820	3.5'
Wall	8400	3.5'
GBs	9072	3.5'
	41165#	$\times 3.5 = 144077'^{-\#}$

$.6D > W$
$.6(144077) > 32500$
$86446 > 32500$ ∴ O/K
∴ No Pile Tension

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JOB Gainon Res.
SHEET NO. 13-3 OF
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Try 8 Pin Piles

Add 75 psf For Light Storage
Below. $75 \times 12.67 \times 5.67 = 5388^{\#}$

Total Load $= 41.17^{K} + 5.4^{K} = 46.6^{K}$

$46.6/2$ sides $= 23.3^{K}$/side $\quad W = \frac{23.3}{14'} = 1.67^{K}/ft$

Inner Pin Piles $4.67(1.67) = 7.8^{K}$
Corner Pin Piles $2.83(1.67) = 4.3^{K}$

Load on Piles From wind $= \frac{32500}{60'} = \frac{5416^{\#}}{14'} = 387^{\#}/ft$

Inner Piles $= 387(4.67) = 1.8^{K}$

$D + L + W = 7.8 + 1.8 = 9.6^{K} < 10^{K}$ ∴ OK

$0.6D + W = 7.8(.6) - 1.8 = +2.88^{K} < 10^{K}$ ∴ OK

$\Sigma DL_s = 35112^{\#} \times 1.2 = 42134$

$\Sigma LL = 11441^{\#} \times 1.6 = 18306$

60440

$W = 60.44^{K}/2/14 = 2.16^{K}/ft$

$Wind = 2.88(1.6) = 4.6^{K}$

$\therefore W_u = \frac{35040}{14} = 2.51^{K}/ft$

Design Grade Bm max Span $= 6'$

$b = 12''$
$d = 11''$
$h = 18''$

$M_u = 2.51 \times 6^2/8 = 11.3^{'-K}$

$V_u = 2.51 \times 6/2 = 7.5^{K}$

$A_s\ reqd = 11.3/4(11) = 0.25$

$\phi V_c = 106(11)(12) = 13.99^{K}$

$P_u = 2.51(4.67) = 11.72^{K}$

$T_u = 11.72 \times \frac{2.5''}{12} = 2.44^{'-K}$

Min $A_s = .0035(12)(18) = 0.76 in^2$ ∴ Provide 2#6 T&B
#3 Ties @ 5" See follow pg
For Torsion Calcs

Siddiq Khan & Associates, Inc.

Concrete Bm. Torsion Design Code: ACI 318-02

Project: Gainor Residnce
Job No. 05-618.00
Description: GB-1

b	12	in	
d	11	in	
h	18	in	
Vu	7.5	k	
Tu	2.44	ft-k	
F'c	5000	psi	
Fy	60000	psi	
Fs	60000	psi	
Shear Angle	45	degrees	
Calculate Vc	18668	lbs	phi Vc = 14001 lbs
Trans Bars	# 3	= 0.11 si	
Clear top	2		
Clear Sides	2		
Clear Bott	3		
Ph	40.5		
Aoh	96.3		
Acp	216.0		
Pcp	60.0		

11.6.1 Neglect Tu if less than:
1/4 Tcr = 3.4 ft-k

11.6.3.1 Check Section Strength Capability
94.3 psi Must be less Than 530.3 psi Per equation 11-18

11.6.3.6 Calculate Transverse Shear Reinforcing
At/ft = 0.05 si/ft Per equation 11-21

11.6.3.7 Calculate Additional Longitudinal Reinforcment
A_ℓ = 0.16 si Per equation 11-22

11.6.5.2 Minimum Allowable Torsion Reinforcment
Av + 2At = 0.12 si/ft s=11.00 in Per equation 11-23

11.6.5.3 Minimum Allowable Add'l Longitudinal Reinforcment
A_ℓ min.= 0.87 si * Per equation 11-24

* if negative then ignore and use 11.6.3.7

11.6.6.1 Min. Allowable Spacing of Torsional Reinforcment
Min. Sapcing: 5.1 in

Shear Spa =	0.00	in		
req'd Vs =	0.000			
At/s; Av/2s+At/s =	0.120	s =	11.00	check against Min. spacing requirments of 11.6.6.1

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Design For Gen. Slab:

Top Slab: $d = 7" - 1.5 - .25 = 5.25"$

Gen wt = 6053# in 60" width & 143" Length

Load on Slab = $\frac{6053 \text{ lbs}}{60(143)} = 102$ psf.

$102 \times 1.25 \cong 130$ psf.
↑ use 25% increase for vibration.

Slab Spans $7' - 0.67(2) = 5'-8"$

Slab DL = $90 \times 1.2 = 108$
LL = $130 \times 1.6 = 208$
316

$M_u = \frac{316 \times 5.67^2}{8} = 1.27$ ft-k $A_s = \frac{1.27}{4(5.25)} = 0.06$

$V_u = 316 \times 5.67/2 = 0.896$ $\phi V_c = 5.25(7)(106) = 3.9$ k OK

Req'd spac = $\frac{540}{36} - 2.5(1.5) = 11.25"$ use 10"

Provide #4 @ 10" T&B

Temp. $.0018(7)(12) = 0.151$ use #4 @ 12"

Both. Slab DL = 90 LL = 75 $U = 1.2(90) + 1.6(75) = 228$

$M_u = \frac{.228(5.67)^2}{8} = 0.92$ ft-k $A_s = \frac{0.92}{4(4.75)} = 0.05 in^2/ft$

∴ use #4 @ 10" T&B & #4 @ 12" for Temp.

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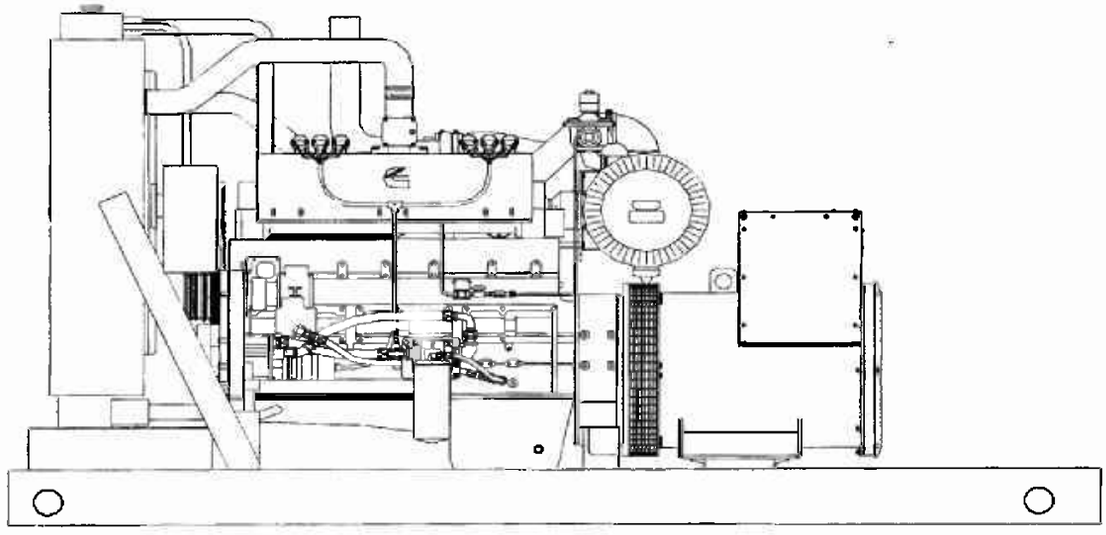
LPG/NG

180 kW @ 60 Hz. Stand-By Power	**180F*Z4**	155 kW @ 60 Hz. Prime Power



- Katolight's commitment to quality has been an industry standard since 1952
- Katolight specializes in custom designing any application to meet the most difficult specifications
- Each and every unit is factory tested. This can eliminate costly startup and installation delays
- Katolight supplies a broad range of accessories to match any requirement worldwide
- Katolight generator sets come standard with a 2 year, 1500 hour limited warranty
- Optional warranty periods are also available, contact factory for details
- This model accepts 100% of nameplate rating, per NFPA 110

Model #	Volts	Hz	Phase	Power Factor	Natural Gas Standby Ratings		Natural Gas Prime Ratings		LP Gas Standby Ratings		LP Gas Prime Ratings		Connection
					Amps	kW/kVA	Amps	kW/kVA	Amps	kW/kVA	Amps	kW/kVA	
180FRZ4	277/480	60	3	0.8	271	180/225	233	155/193.75	271	180/225	233	155/193.75	12 LEAD HI WYE
180FPZ4	120/208	60	3	0.8	625	180/225	538	155/193.75	625	180/225	538	155/193.75	12 LEAD LOW WYE
180FJZ4	120/240	60	3	0.8	541	180/225	466	155/193.75	541	180/225	466	155/193.75	12 LEAD HI DELTA
180FNZ4	347/600	60	3	0.8	217	180/225	186	155/193.75	217	180/225	186	155/193.75	4 LEAD WYE
180FGZ4	120/240	60	1	1.0	750	180/180	646	155/155	750	180/180	646	155/155	12 LEAD ZIG-ZAG
180FDZ4	120/240	60	1	1.0	750	180/180	646	155/155	750	180/180	646	155/155	4 LEAD

KATOLIGHT CORPORATION

180F*Z4 LPG/NG Gen-Set

ENGINE TECHNICAL DATA

	60 Hz	
Model:	GTA855-G1	
Type:	4-Cycle	
Aspiration:	Turbocharged/Aftercooled	
Cylinder Arrangement: (Number, Inline, V, etc.)	6-inline	
Displacement - Cu. In. (lit)	855 (14)	
Bore - in. (cm) x stroke - in. (cm)	5.5 (14.0) x 6.0 (15.2)	
Compression Ratio:	NG-10:1/LPG-8.5:1	
Rated RPM	1800	
Rating	**Standby**	**Prime**
BMEP: psi (kPa)	151 (1,041)	135 (945)
Maximum Power at Rated RPM - bhp(kW)	293 (218)	266 (198)

INSTALLATION DATA *

Exhaust System		
Gas Temp. (Stack): °F (°C)	1,350 (732)	1,283 (695)
Gas Volume at Stack Temp.: CFM (m^3/min)	2,081 (58.9)	1,889 (53.5)
Maximum Allowable Back Pressure: in. H_2O (kPa)	27.2 (6.8)	27.2 (6.8)
Emissions - HC: g/hp-hr	C/F	C/F
Emissions - CO: g/hp-hr	C/F	C/F
Emissions - NO_x: g/hp-hr	C/F	C/F
Cooling System		
Ambient Capacity of Radiator: °F (°C)	100 (38)	100 (38)
Maximum Allowable Static Pressure on Rad. Exhaust: in. H_2O (kPa)	0.5 (.12)	0.5 (.12)
Coolant Flow: Engine Circuit: gpm (lit/min)	113 (428)	113 (428)
Aftercooler Circuit: gpm (lit/min)	26 (98)	26 (98)
Heat Rejection: Engine Circuit: BTUM (kW)	10,988 (193)	9,975 (175)
Aftercooler Circuit: BTUM (kW)	1,418 (25)	1,287 (23)
Heat Radiated to Ambient: BTUM (kW)	2,266 (39.8)	2,129 (37.4)
Air Requirements		
Aspirating: CFM (m^3/min)	772 (21.9)	701 (19.8)
Air Flow Required for Rad. Cooled Unit: CFM (m^3/min)	35,000 (991)	31,775 (900)
Air Flow Required for Heat Exchanger/Remote Rad. based on 20°F Rise: CFM (m^3/min)	6,294 (178)	5,914 (167)

Fuel Consumption: (NG-1000 BTU/ft^3 / LP-2500 BTU/ft^3)	**NG**	**LPG**	**NG**	**LPG**
At 100% of Power Rating: ft^3/hr (m^3/hr)	2,350 (67)	940 (27)	2,133 (60)	853 (24)
At 75% of Power Rating: ft^3/hr (m^3/hr)	1,763 (50)	705 (20)	1,600 (45)	640 (18)
At 50% of Power Rating: ft^3/hr (m^3/hr)	1,175 (33)	470 (13)	1,066 (30)	427 (12)

Sound Level Data ■

Sound level at:	**Full Load**	**No Load**	**Full Load**	**No Load**
23 ft (7m) opu w/ critical grade muffler (dBA)	90	87	89	87
23 ft (7m) Sound Attenuated Enclosure (dBA)	82	79	81	79

Dimensions & Weight

Length: in. (cm)	143 (363)
Width: in. (cm)	60 (152)
Height: in. (cm)	75.75 (192)
Weight (dry): lb. (kg)	5,695 (2,583)

Liquid Capacity

Total oil system: gal (lit)	10.0 (37.9)
Engine jacket water capacity: gal (lit)	5.5 (21)
Aftercooler water capacity: gal (lit)	1.0 (3.8)
System coolant capacity: gal (lit)	26.5 (100.3)

5695 − 430 × 8.53 = 3358 → 3053#

Fuel Inlet

Fuel connection size:	2" NPT
Fuel supply pressure in H_2O (mm H_2O)	10-20 (254-508)

Electrical System

Electric volts DC	24
Cold cranking Amps under 0°F (-17.8°C)	900

Remote Radiator System

Connection sizes:	
Jacket water radiator inlet in. (cm)	2.5 (6.35)
Aftercooler radiator inlet in. (cm	1 (2.5)
Jacket water radiator outlet in. (cm)	2.5 (6.35)
Aftercooler radiator outlet in. (cm)	1 (2.5)
Static head allowable above engine ft H_2O (kPa)	46 (137)
Total system friction pressure max. allowable psi (kPa)	5 (34.5)

Heat Exchanger System

Connection sizes:	
Heat ex. inlet in. (cm)	2.0 (5.1)
Heat ex. outlet in. (cm)	2.0 (5.1)
Water consumption: @ 90°F (32°C) gpm (lit/min)	34 (128)

*Installation data based on 480 volt, 60 HZ. application and open power unit.

■ For sound level readings with other enclosures, please contact factory.

Sound level data acquired per Test Method SAE J1074. Installation factors and site conditions can affect sound levels.

Deration Factor: Altitude: Derate: 3% per 1,000 ft (305m) above 3,000 ft (914 m). Temperature: Derate: 2% per 20°F (11°C) above 100°F (38°C)

180F*Z4 LPG/NG Gen-Set

KATOLIGHT CORPORATION

STANDARD EQUIPMENT

CONTROL PANEL

- Model #45 control panel
- AC voltmeter, 3 ½", 2% accuracy
- AC ammeter, 3 ½", 2% accuracy
- Combination VM/AM selector switch, 4 position
- Frequency meter, 3 ½", 55-65 Hz.
- Vibration shock mounts (4)
- Engine control - KASSEC-24 VDC, with cyclic cranking timer
- 4 engine shutdowns with separate failure lights
  - * High water temperature
  - * Low oil pressure
  - * Engine overspeed
  - * Engine overcrank
- Engine gauges - 2"
  - * Battery voltmeter
  - * Water temperature
  - * Oil pressure
  - * Running time meter - 5 digits
- 3 position mode switch (auto-off-manual)

ENGINE

- Air cleaner
- Oil pump
- Full flow oil filter
- Jacket water pump
- Thermostat
- Exhaust manifold – dry
- Blower fan & fan drive
- Radiator - unit mounted
- Vibration isolators - pad type
- Electric starting motor - 24V

ENGINE (cont.)

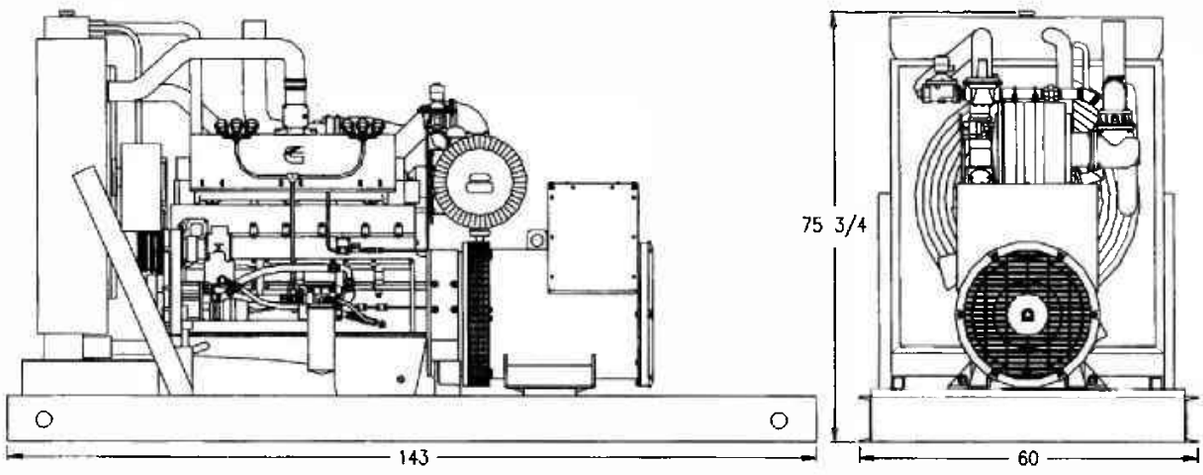
- Governor – Electric Isochronous
- Base - formed steel
- Flywheel & Enclosure
- Charging alternator - 24V
- Battery box & cables
- Flexible fuel & exhaust connectors

GENERATOR

- A.C. Generator
- Brushless design
- Single bearing
- Direct connection with flex plate
- Class H insulation
- All models manufactured to meet NEMA MG1- 22.4 and CSA standards
- Telephone influence factor is well within NEMA standards
- Wave form deviation factor is no more than 5%, well within NEMA standards
- Harmonic content is 3.0% maximum
- Permanently lubricated ball type bearings
- Generator is self-ventilated
- Drip-proof construction

VOLTAGE REGULATOR

- Voltage adjust rheostat
- EMI filter (Internal Electromagnetic Interference)
- Underspeed protection
- Overexcitation protection
- Fully encapsulated
- Regulation - 1%



Drawing above for illustration purposes only, based on standard open power 480 volt generator.
Lengths may vary with other voltages.

KATOLIGHT CORPORATION

180F*Z4 NG Gen-Set

Control Panel

** NOTE: #45 series control panel is standard on all units, see page 2 of spec sheet for standard features.

- Model #45 Series Control Panel Options
  - Emergency stop button
  - Alarm buzzer with silencing switch
  - Auxiliary relay for dry contacts (2 max.)
  - A separate low water level light is optional
  - Hooded panel lights (2) and on/off switch
  - NEMA 12 Panel Face
  - Additional LED lights (4 max.) One or two of the following conditions may be indicated:
    - unit not in auto
    - low fuel level
    - low water level
    - low water temp.
    - EPS supplying load
    - pre-alarm oil
    - pre-alarm temp.
    - charger malfunction
- Model #50 Series Control Panel
  STANDARD FEATURES: same as #45 series control panel except for these added features:
  - Hooded panel lights (2) and on/off switch
  - 4 Engine shutdowns
  - 12 light engine control package meeting NFPA-110 requirement
  - Repetitive alarm buzzer and silencing switch
  - Light and alarm press to test

  #50 SERIES OPTIONS
  - Emergency stop button
  - Additional space for one $3^1/_2$ meter
  - Auxiliary relay for dry contacts (2 max.)
  - A separate low water level light is optional
  - Additional LED lights (4 max.) One to four additional conditions may be indicated: customer to specify
  - NEMA 12 Panel Face
- Model #60 and #80 Series Custom Control Panels
  It may be necessary to use a 60 or 80 series control panel on certain units where numerous options are required.
- Microprocessor Control Panel – KDGC

GEN-SET OPTIONS

Cooling System

- Remote Radiator
- High Ambient Radiator
- Heat Exchanger Cooling
- Radiator Duct Flange

DISTRIBUTED BY:

Fuel System

- Fuel Strainer
- Dual Fuel
  - Manual Change-over
  - Auto Change-over

Exhaust System

- Residential Grade Muffler
- Critical Grade Muffler
- Hospital Grade Muffler
- Rain Cap

Engine Electrical System

- Battery
  - Lead-Acid
  - Nicad
- Battery Warmer Plate
- Battery Rack
- Battery Charger
  - Automatic
  - Trickle
  - Mounted & Wired

Generator

- Main Line Circuit Breaker
  - Shunt trip
  - Auxiliary switch
- PMG Excitation & DVR 2000 Regulator
- Space Heaters 120/240 volt
- Special Testing
- Additional Temperature Rise Generators Available (80°C, 105°C, & 130°C)

Additional Optional Equipment

- Spring vibration isolators
- Oil Drain Extension
- Enclosures
  - Sound Attenuated
  - Weather Proof
  - Aluminum
  - Interior lights AC or DC
  - Floor Plate
- Jacket Water Heater
- Crankcase Oil Heater
- Remote Annunciator
- 12 Light Annunciator
  - Flush Mounted
  - Surface Mounted
  - 4 additional lights, if needed
- Export Boxing
- Warranties
  - 2 Year
  - 5 Year
- Operating instructions under plexi-glass
- Service indicator light
- Wind rated enclosure

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Gainor Residence
5800 North Bay Road
City of Miami Beach, Florida

Product Control Notice of Acceptance

Anchor	Type	Allow. Gravity	Allow. Uplift	Allow. L1	Allow. L2	Strap / Joist / Truss	Seat / Sill / Ledger	Manuf.	Notice of Acceptance	Expir. Date	Remarks
1a	NVHTA -12H	N/A	2078	1050	1450	10-10d nails x 1 1/2"	6-10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	1 ply truss
1b	NVHTA -16H	N/A	2338	1181	1631	12-10d nails x 1 1/2"	6-10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	1 ply truss
1c	NVHTA - 20H	N/A	2598	1312	1812	14-10d nails x 1 1/2"	6-10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	1 ply truss
1d	NVHTA - 24H	N/A	2858	1444	1994	16-10d nails x 1 1/2"	6-10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	1 ply truss
2a	NVSTA - 12H	N/A	1545	823	1239	7-10d nails x 1 1/2"	6-10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	1 ply truss
2b	NVSTA - 16H	N/A	1664	887	1335	8-10d nails x 1 1/2"	6-10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	1 ply truss
2c	NVSTA - 20H	N/A	1783	950	1430	9-10d nails x 1 1/2"	6-10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	1 ply truss
2d	NVSTA - 24H	N/A	1783	950	1430	9-10d nails x 1 1/2"	6-10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	1 ply truss
3a	NV358 - 12	N/A	2525	2206	2068	10-10d nails x 3"	8-10d nails x 3"	Nu-Vue	04-0510.03	12/23/2009	2 ply Truss
3b	NV358 - 16	N/A	2806	2452	2298	12-10d nails x 3"	8-10d nails x 3"	Nu-Vue	04-0510.03	12/23/2009	2 ply Truss
3c	NV358 - 20	N/A	3086	2697	2528	14-10d nails x 3"	8-10d nails x 3"	Nu-Vue	04-0510.03	12/23/2009	2 ply Truss
3d	NV358 - 24	N/A	3367	2942	2758	16-10d nails x 3"	8-10d nails x 3"	Nu-Vue	04-0510.03	12/23/2009	2 ply Truss
4a	NV458 - 12	N/A	2525	3131	2338	10-10d nails x 3"	10-10d nails x 3"	Nu-Vue	04-0510.03	12/23/2009	3 ply Truss
4b	NV458 - 16	N/A	2806	3479	2597	12-10d nails x 3"	10-10d nails x 3"	Nu-Vue	04-0510.03	12/23/2009	3 ply Truss
4c	NV458 - 20	N/A	3086	3827	2857	14-10d nails x 3"	10-10d nails x 3"	Nu-Vue	04-0510.03	12/23/2009	3 ply Truss
4d	NV458 - 24	N/A	3367	4175	3117	16-10d nails x 3"	10-10d nails x 3"	Nu-Vue	04-0510.03	12/23/2009	3 ply Truss
5a	NVTA - 16	N/A	1117	783	735	8-10d nails x 1 1/2"	Embd. 4" Conc or Masonry BM	Nu-Vue	04-0510.03	12/23/2009	Embd. Conc. To Wood
5b	NVTA - 24	N/A	1117	783	735	8-10d nails x 1 1/2"	Embd. 4" Conc or Masonry BM	Nu-Vue	04-0510.03	12/23/2009	Embd. Conc. To Wood
5c	NVTA - 36	N/A	1396	783	735	10-10d nails x 1 1/2"	Embd. 4" Conc or Masonry BM	Nu-Vue	04-0510.03	12/23/2009	Embd. Conc. To Wood
5d	NVTA - 48	N/A	1598	783	735	13-10d nails x 1 1/2"	Embd. 4" Conc or Masonry BM	Nu-Vue	04-0510.03	12/23/2009	Embd. Conc. To Wood
6a	NVRT - 16	N/A	907	N/A	N/A	6 - 16d nails	6 - 16d nails	Nu-Vue	04-0510.03	12/23/2009	Wood to Wood; flat or twisted
6b	NVRT - 20	N/A	1059	N/A	N/A	7 - 16d nails	7 - 16d nails	Nu-Vue	04-0510.03	12/23/2009	Wood to Wood; flat or twisted
6c	NVRT - 24	N/A	1125	N/A	N/A	8 - 16d nails	8 - 16d nails	Nu-Vue	04-0510.03	12/23/2009	Wood to Wood; flat or twisted
6d	NVRT - 36	N/A	1125	N/A	N/A	9 - 16d nails	9 - 16d nails	Nu-Vue	04-0510.03	12/23/2009	Wood to Wood; flat or twisted
7a	NVRT - 16	N/A	704	N/A	N/A	6 - 16d nails	4 1/4" Tapcons w/ 1 1/4" embd.	Nu-Vue	04-0510.03	12/23/2009	Wood to Conc; flat or twisted
7b	NVRT - 20	N/A	880	N/A	N/A	7 - 16d nails	5 1/4" Tapcons w/ 1 1/4" embd.	Nu-Vue	04-0510.03	12/23/2009	Wood to Conc; flat or twisted
7c	NVRT - 24	N/A	880	N/A	N/A	7 - 16d nails	5 1/4" Tapcons w/ 1 1/4" embd.	Nu-Vue	04-0510.03	12/23/2009	Wood to Conc; flat or twisted
7d	NVRT - 36	N/A	880	N/A	N/A	7 - 16d nails	5 1/4" Tapcons w/ 1 1/4" embd.	Nu-Vue	04-0510.03	12/23/2009	Wood to Conc; flat or twisted
8	NVUH26	2233	1213	N/A	N/A	10 - 16d nails	20 - 16d nails	Nu-Vue	04-1202.01	8/21/2008	Joist Hanger w/ dbl 2x Ledger
9a	NVJH24	726	493	N/A	N/A	4 - 10d nails x 1 1/2"	6 - 10d nails x 1 1/2"	Nu-Vue	03-0730.03	1/1/2009	Single Joist Hanger w/ dbl 2x Ledger
9b	NVJH26	1210	818	N/A	N/A	6 - 10d nails x 1 1/2"	10 - 10d nails x 1 1/2"	Nu-Vue	03-0730.03	1/1/2009	Single Joist Hanger w/ dbl 2x Ledger
9c	NVJH28	1694	954	N/A	N/A	7 - 10d nails x 1 1/2"	14 - 10d nails x 1 1/2"	Nu-Vue	03-0730.03	1/1/2009	Single Joist Hanger w/ dbl 2x Ledger
9d	EHUH28-2	2875	1690	N/A	N/A	11-10d x 1 3/4"	22-16d nails x 2"	USP	03-0611.05	8/21/2008	Double Joist Hanger w/ dbl 2x Ledger
9e	EHUH28-2	4185	2155	N/A	N/A	14-10d x 1 3/4"	32-16d nails x 2"	USP	03-0611.05	8/21/2008	Double Joist Hanger w/ dbl 2x Ledger
9f	EHUH210-2	5190	1645	N/A	N/A	16-10d x 1 3/4"	40-16d nails x 2"	USP	03-0611.05	8/21/2008	Double Joist Hanger w/ dbl 2x Ledger
10a	NVTP4	N/A	818	N/A	N/A	6 - 10d nails x 1 1/2"	N/A	Nu-Vue	03-0730.03	1/1/2009	2x4 Wood Plate to Vert. Stud
10b	NVTP4	N/A	1605	N/A	N/A	12 - 10d nails x 1 1/2"	N/A	Nu-Vue	03-0730.03	1/1/2009	2x4 Wood Plate to Vert. Stud
10c	NVTP6	N/A	1605	N/A	N/A	12 - 10d nails x 1 1/2"	N/A	Nu-Vue	03-0730.03	1/1/2009	2x6 Wood Plate to Vert. Stud
10d	NVTP8	N/A	1605	N/A	N/A	12 - 10d nails x 1 1/2"	N/A	Nu-Vue	03-0730.03	1/1/2009	2x8 Wood Plate to Vert. Stud
11a	NVHC 43	N/A	687	407	308	9-10d nails x 1 1/2"	9-10d nails x 1 1/2"	Nu-Vue	03-0730.03	1/1/2009	Hurricane Clip for single 2x
11b	NVHC 43/2	N/A	917	547	432	10 -10d nails x 1 1/2"	10 -10d nails x 1 1/2"	Nu-Vue	03-0730.03	1/1/2009	Hurricane Clip for double 2x
12a	NVHCR	N/A	525	253	333	6 - 10d nails x 1 1/2"	6 - 10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	Hurr. Clip; need L & R for FBC uplift Requirements
12b	NVHCL	N/A	525	253	333	6 - 10d nails x 1 1/2"	6 - 10d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	Hurr. Clip; need L & R for FBC uplift Requirements
13	NVSNP3	578	594	594	594	6 - 8d nails x 1 1/2"	6 - 8d nails x 1 1/2"	Nu-Vue	04-0510.03	12/23/2009	Adjustable Clip Angle
14a	JA3G	265	N/A	N/A	N/A	4 - 10d nails x 1 1/2"	4 - 16d nails	USP	01-0417.11	7/9/2006	Framing Angle
14b	JA5G	550	735	N/A	N/A	6 - 10d nails x 1 1/2"	6 - 16d nails	USP	01-0417.11	7/9/2006	Framing Angle
15	LUGT2	N/A	1850	1220	460	18 - 10d nails x 1 1/2"	5 - 1/4" Tapcons 1 1/2" Embd.	USP	03-0219.02	6/20/2007	Conc/Masonry Bm to truss / rafter connection
16a	C44	N/A	820	860	860	6 - 16d nails	6 - 16d nails	USP	03-508.05	5/2/2007	4x4 wood post to wood sill or cap plate
16b	C46	N/A	745	950	950	10 - 16d nails	10 - 16d nails	USP	03-508.05	5/2/2007	4x6 wood post to wood sill or cap plate
16c	C66	N/A	870	1690	1690	10 - 16d nails	10 - 16d nails	USP	03-508.05	5/2/2007	6x6 wood post to wood sill or cap plate
16d	C88	N/A	1125	2390	3070	16 - 16d nails	16 - 16d nails	USP	03-508.05	5/2/2007	8x8 wood post to wood sill or cap plate
16e	PAU44	6885	1825	N/A	N/A	12 - 16d nails or 2 1/2" dia. Bolts	5/8" dia. Anchor Bolt	USP	03-508.05	5/2/2007	4x4 wood post to concrete
16f	PAU66	14300	2425	N/A	N/A	12 - 16d nails or 2 1/2" dia. Bolts	5/8" dia. Anchor Bolt	USP	03-508.05	5/2/2007	6x6 wood post to concrete
17a	HJH 26 L / R	1920	1890	N/A	N/A	12 - 16d nails	7-16d hip & 8-16d jack	USP	05-0204.01	12/2/2006	Skewed plate Connection
17b	HJH 28 L / R	2560	2185	N/A	N/A	16 - 16d nails	7-16d hip & 8-16d jack	USP	05-0204.01	12/2/2006	Skewed plate Connection

SKA Anchor Schedule Last updated 10/03/2005

MIAMI-DADE

BUILDING CODE COMPLIANCE OFFICE (BCCO)
PRODUCT CONTROL DIVISION

MIAMI-DADE COUNTY, FLORIDA
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140 WEST FLAGLER STREET, SUITE 1603
MIAMI, FLORIDA 33130-1563
(305) 375-2901 FAX (305) 375-2908

NOTICE OF ACCEPTANCE (NOA)

NU-VUE Industries Inc.
1055 East 29 Street.
Hialeah, Florida 33013

SCOPE:

This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed by Miami-Dade County Product Control Division and accepted by the Board of Rules and Appeals (BORA) to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Division (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. BCPRC reserves the right to revoke this acceptance, if it is determined by Miami-Dade county Product Control Division that this product or material fails to meet the requirements of the applicable building code.
This product is approved as described herein, and has been designed to comply with the Florida Building Code including High velocity Hurricane Zone.

DESCRIPTION: Wood Connectors.

APPROVAL DOCUMENT: Drawing No.Nu-5, titled "Skewed Nail Plate, NV358 & NV458 with Double NVTH Straps, NVTH/NVTHS Anchors & NVHC Hurricane Clips and NVSTA & NVHTA Heavy Duty Anchors with Seat" sheet 1, through 4 of 4, dated 04/15/04 with last revision on 09/20/04, prepared by Nu-Vue Industries Inc signed and sealed by V. N. Tolat, PE, bearing the Miami-Dade County Product Control Approval stamp with the Notice of Acceptance (NOA) number and approval date by the Miami-Dade County Product Control Division.

MISSILE IMPACT RATING: None

LABELING: Each unit shall bear a permanent label with the manufacturer's name or logo, city, state and following statement: "Miami-Dade County Product Control Approved", unless otherwise noted herein.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.
This NOA consists of this page 1, evidence page & approval document mentioned above.
The submitted documentation was reviewed by **Candido F. Font PE.**

12/23/04

PRODUCT CONTROL ACCEPTED

NOA No: 04-0510.03
Expiration Date: December 23, 2009
Approval Date: December 23, 2004

Nu-Vue Industries, Inc.

NOTICE OF ACCEPTANCE: EVIDENCE PAGE

A DRAWINGS

1. Drawings prepared by Nu-Vue Industries Inc, titled "Skewed Nail Plate, NV358 & NV458 with Double NVTH Straps, NVTH/NVTHS Anchors & NVHC Hurricane Clips and NVSTA & NVHTA Heavy Duty Anchors with Seat", Drawing No. NU-5, sheets No. 1, through 4 of 4, dated 04/15/04 with last revision on 09/20/04, signed and sealed by V. N. Tolat, PE.

B TEST

Test reports on wood connectors per ASTM D1761 by Product Testing Inc, signed and sealed by C. R. Caudel, PE.

	Report No.	Wood Connector	Direction	Date
1.	PT # 03-4482	NVSNP3	Downward	09/15/03
2	PT # 03-4625	NVHCL/R	Up& Sideways	01/21/04
3	PT # 04-4641	NVSTA-24H	Up& Sideways	03/17/04
4.	PT # 04-4698	NVTH24	Up& Sideways	04/15/04
5	PT # 03-4590	NVTH26/NV458	Sideways	12/31/03
6.	PT # 04-4642	NVHTA-24H	Up& Sideways	03/22/04
7.	PT # 03-4543	NVTH20/NV358	Up& Sideways	12/19/03

C CALCULATIONS

Report of Design Capacities prepared by V. N. Tolat on 04/27/04, sheet 1 through 12, signed and sealed by V. N. Tolat PE.

D QUALITY ASSURANCE

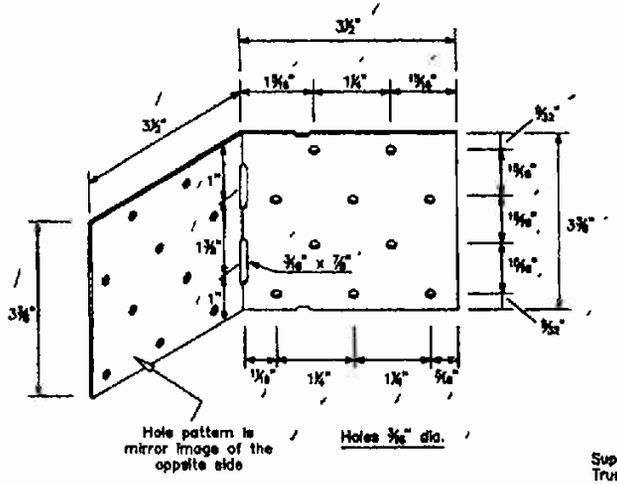
1. Product Control Division

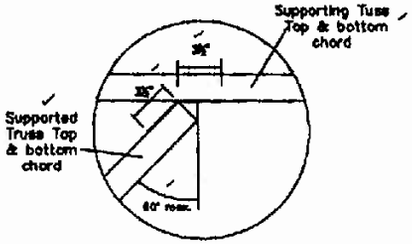
E STATEMENTS

1. Code compliance letter issue by V. N. Tolat on 04/27/04 signed and sealed by V. N. Tolat, PE.
2. No interest letter issued by V. N. Tolat on 11/03/04 signed and sealed by V. N. Tolat, PE.

12/23/04

Candido F. Font PE.
Sr. Product Control Examiner
NOA No 04-0510.03
Expiration Date: December 23, 2009
Approval Date: December 23, 2004





NVSNP3 Installation

TABLE 1
SKEWED NAIL PLATE

Product Code	Steel Gauge	Fastener Schedule	Allowable Loads (lbs)	
		Each End	Gravity	Uplift
NVSNP3	16	(6) 8d x 1½"	578	594

For Uplift, use two NVSNP3, one at top chord and one at bottom chord of the supporting and supported Trusses in compliance with section 2321.7 of the FBC.

Notes:

1. Use 6 nail holes in each flange.
2. Do not bend nail plate more than once.
3. Supporting member shall be in the acute angle side with other flange behind the end of supported member (see sketch.)

Approved as complying with the Florida Building Code
Date 12/23/04
NOA# 04-0510.03
Miami Dade County Product Control Division
By

General Notes:

1. Steel shall conform to ASTM A653, structural grade 33 (Min. yield 33 ksi) and a minimum galvanized coating of G 60 per ASTM A525.
2. Allowable loads are based on National Desing specifications (NDS) for wood construction, 1997 Edition.
3. Design loads are for Southern Pine species with a specific gravity of 0.55. Allowable loads for other species shall be adjusted accordingly.
4. Common wire nail values are based on NDS table 12.3F, G=0.55 and have been reduced for Penetration Depth factor Cd.
5. Allowable loads for wind uplift have already been increased by a duration factor of 33% for anchor nail. This increase is not allowed for steel stress if dead load and wind loads are combined. Load values shown are, without 33% steel stress increase.
6. Allowable loads for more than a single connection cannot be added together. A design load which is divided into components in the direction given must be evaluated as follows:

$$\frac{\text{Actual Uplift}}{\text{Allowable Uplift}} + \frac{\text{Actual L1}}{\text{Allowable L1}} + \frac{\text{Actual L2}}{\text{Allowable L2}} \leq 1.0$$

7. Allowable loads are based on 1½" thick wood members unless otherwise noted.
8. All tie beams and grouted concrete masonry shall comply with chapter 21 of FBC. Concrete for tie beams and grout and mortar for concrete masonry shall be a minimum of 2500 psi. Concrete masonry shall comply with ASTM C90.
9. All tests have been conducted in accordance with ASTM D-1761.

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11/3/04

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SKEWED NAIL PLATE

DWG #:	Sheet:	Date:	Revisions:
NU-5	1 of 4	APRIL 15, 2004	Sept. 20, 2004

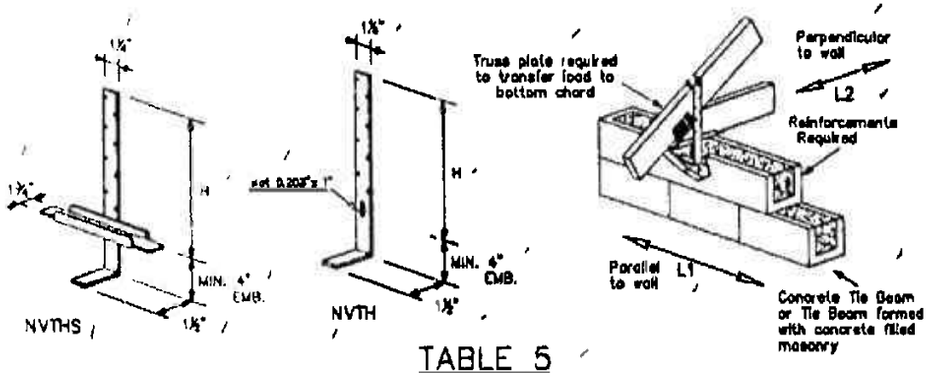


TABLE 4

Truss Anchors NVTH and Riveted Truss Anchors with Seat NVTHS

H Length (in)	Product Code		Gauge seat	Gauge strap
12	NVTH-18	NVTHS 212	18	14
14	NVTH-18	NVTHS 214	18	14
16	NVTH-20	NVTHS 216	18	14
18	NVTH-22	NVTHS 218	18	14
20	NVTH-24	NVTHS 220	18	14
22	NVTH-26	NVTHS 222	18	14
24	NVTH-28	NVTHS 224	18	14
26	NVTH-30	NVTHS 226	18	14
32	NVTH-36	NVTHS 232	18	14
44	NVTH-48	NVTHS 244	18	14

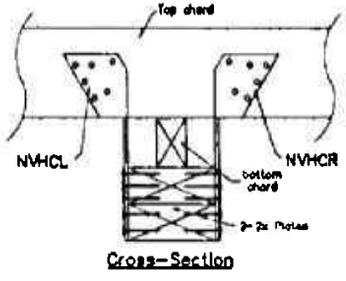
No. of Fasteners in Strap 10d x 1.5"	Maximum Allowable Load (lbs)		
	Uplift	L1	L2
5	700	560	525
6	838	671	630
7	977	783	735
8	1117	783	735
9	1256	783	735
10	1396	783	735
11	1490	783	735
12	1544	783	735
13	1598	783	735

TABLE 5

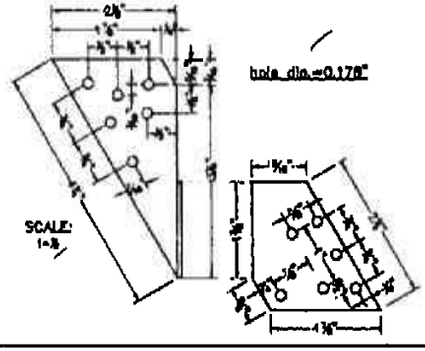
HURRICANE CLIPS

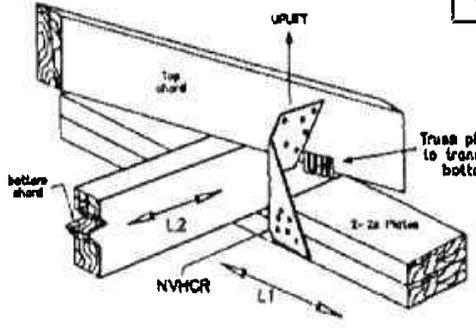
Product Code	Description	Gauge	Fasteners 10d x 1½"		Allowable Loads (lbs)		
			Header	Joist	Uplift	L1	L2
NVHCR	HURRICANE CLIP – RIGHT	18	6	6	525	253	333
NVHCL	HURRICANE CLIP – LEFT	18	6	6	525	253	333

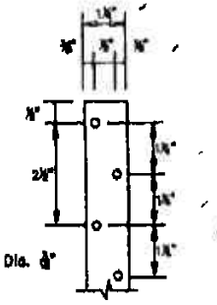
For Uplift, use two clips, one on each side to comply with section 2321.7 of the FBC



Cross-Section







NVTH, NVTHS

Approved as complying with the Florida Building Code 12/28/04 NOA 04-0816.03 Miami-Dade County Product Control Division

VIPIN N. TOLAT, PE (CIVIL)
FL. REG. # 12847
15123 LANTERN CREEK LANE
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11/3/04

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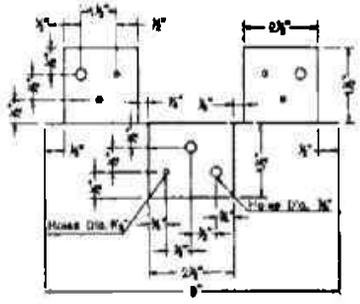
NVTH / NVTHS ANCHORS & NVHC HURRICANE CLIPS

DWG #:	Sheet:	Date:	Revisions:
NU-5	3 of 4	APRIL 15, 2004	Sept. 20, 2004

TABLE 6

NVSTA–Heavy Duty Anchors with 14G NVTH Straps and 1 ply 20G seat

Assembly Product Code	14G Strap Product Code	Dimension H (inches)	Total No. of Fasteners in Strap 10d x 1½"	Total No. of Fasteners in 20 GA. Seat 10d x 1½"	Allowable Loads (lbs) Uplift [5]	L1 [6]	L2 [6]
NVSTA-12H	NVTH16	12	5	6	1308	700	1049
NVSTA-14H	NVTH18	14	6	6	1426	760	1144
NVSTA-16H	NVTH20	16	7	6	1545	823	1239
NVSTA-20H	NVTH24	20	8	6	1664	887	1335
NVSTA-22H	NVTH28	22	9	6	1783	950	1430
NVSTA-24H	NVTH28	24					
NVSTA-28H	NVTH30	28					
NVSTA-32H	NVTH36	32					
NVSTA-44H	NVTH48	44					



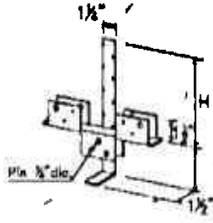
20G Seat

TABLE 7

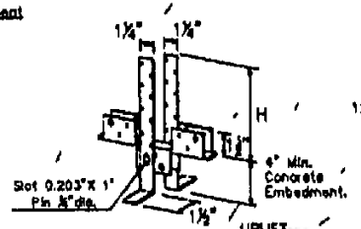
NVHTA–Heavy Duty Anchors with 14G Double NVTH Straps and 1 ply 20G seat

Assembly Product Code	14G Strap Product Code	Dimension H (inches)	Total No. of Fasteners in two Straps 10d x 1½"	Total No. of Fasteners in 20 GA. Seat 10d x 1½"	Allowable Loads (lbs) Uplift [5]	Uplift [5]	L1 [6]	L2 [6]
NVHTA-12H	NVTH16	12	10	6	1772	2078	1050	1450
NVHTA-14H	NVTH18	14	12	6	1994	2338	1161	1831
NVHTA-16H	NVTH20	16	14	6	2215	2598	1312	1812
NVHTA-20H	NVTH24	20	16	6	2437	2858	1444	1994
NVHTA-22H	NVTH28	22	18	6	2658	3117	1575	2175
NVHTA-24H	NVTH28	24						
NVHTA-28H	NVTH30	28						
NVHTA-32H	NVTH36	32						
NVHTA-44H	NVTH48	44						

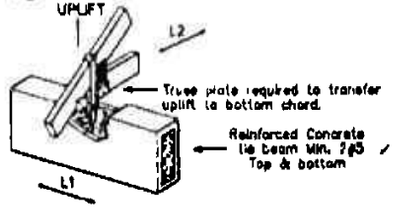
Notes:
1. Nails are necessary in straps and seat to achieve, desing loads.
2. See note 6, sheet 1 for combined loading.
3. Nails through chords shall not force the truss plates.
4. For general notes, see sheet 1.
5. For higher uplift loads, concrete shall be 3000 psi.
6. Based on min. 2500 psi concrete.

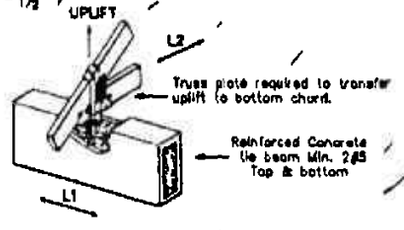


NVSTA
12H throght 44H



NVHTA
12H throght 44H

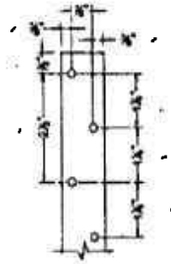




Approved as complying with the Florida Building Code
Date 12/23/04
NOA 04-1216.03
Miami-Dade Product Control Division
By

VIPIN N. TOLAT, PE (CIVIL)
FL. REG. # 12847
15123 LANTERN CREEK LANE
HOUSTON, TX 77068

11/3/04



14G NVTH

Side View
20 GA. Seat

Nu-Vue Industries, Inc.

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NVSTA & NVHTA HEAVY DUTY ANCHORS WITH SEAT

DWG #:	Sheet:	Date:	Revisions:
NU-5	4 of 4	April 15, 2004	Sept. 20, 2004

MIAMI-DADE

BUILDING CODE COMPLIANCE OFFICE (BCCO)
PRODUCT CONTROL DIVISION

MIAMI-DADE COUNTY, FLORIDA
METRO-DADE FLAGLER BUILDING
140 WEST FLAGLER STREET, SUITE 1603
MIAMI, FLORIDA 33130-1563
(305) 375-2901 FAX (305) 375-2908

NOTICE OF ACCEPTANCE (NOA)

Nu-Vue Industries, Inc.
1053-1059 East 29 Street
Hialeah, Florida 33013

SCOPE:

This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed by Miami-Dade County Product Control Division and accepted by the Board of Rules and Appeals (BORA) to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Division (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. BCPRC reserves the right to revoke this acceptance, if it is determined by Miami-Dade county Product Control Division that this product or material fails to meet the requirements of the applicable building code.
This product is approved as described herein, and has been designed to comply with the High velocity Hurricane Zone of the Florida Building Code.

DESCRIPTION: Wood Connectors NVTA, NVTAS, NVBH, NVUH & NVRT.

APPROVAL DOCUMENT: Drawing No. NU-2, sheets 1 through 3, titled "NVTA and NVTAS Anchors; NVBH 24and NVUH 26 Hangers & NVRT Flat and Twisted Rafter Ties," dated 02/13/03 with last revision on 07/07/03, prepared by Nu-Vue Industries, Inc. signed and sealed by V. N. Tolat, PE, bearing the Miami-Dade County Product Control Approval stamp with the Notice of Acceptance (NOA) number and approval date by the Miami-Dade County Product Control Division.

MISSILE IMPACT RATING: None

LABELING: Each unit shall bear a permanent label with the manufacturer's name or logo, city, state and following statement: "Miami-Dade County Product Control Approved", unless otherwise noted herein.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.
This revises NOA # 03-0327.14 and consists of this page 1 as well as approval document mentioned above.
The submitted documentation was reviewed by Candido F. Font PE.

01/13/05

PRODUCT CONTROL ACCEPTED

NOA No: 04-1202.01
Expiration Date: August 21, 2008
Approval Date: January 13, 2005
Page 1

NOTICE OF ACCEPTANCE: EVIDENCE SUBMITTED

(For File ONLY. Not part of NOA)

A DRAWINGS

1. Drawings prepared by Nu-Vue Industries, Inc., titled "NVTA and NVTAS Truss Anchors; NVBH24 and NVUH26 Hangers & NVRT Flat and Twisted Rafter Ties", drawing No. NU-2, sheets1 through 3, dated 02/13/03 with last revision on 07/07/03, signed and sealed by V. N. Tolat, PE.

B TEST

Test reports on wood connectors per ASTM D1761 by Product Testing, Inc., signed and sealed by C. R. Caudel, PE & S. E. Black, PE.

	Report No.	Wood Connector	Direction	Date
1.	PT 02-4073	NVTA	Upward	11/06/02
2.	PT 02-4075	NVTA	Upward	11/07/02
3.	PT 02-4074	NVTA	Upward	11/06/02
4.	PT 02-3938	NVTA	Upward	08/06/02
5.	PT 03-4177	NVRT36	Upward	02/03/03
6.	PT 03-4202	NVRT36-T	Upward	02/19/03
7.	PT 03-4271	NVRT36-T	Upward	03/27/03
8.	PT 03-4270	NVRT24-T	Upward	03/27/03
9.	PT 02-4095	NVUH26	Up & Downward	01/17/03
10.	PT 02-4096	NVBH24	Up &Downward	12/03/02
11.	31-22456.0002	NVTA & NVTAS	Lateral	07/06/02

C CALCULATIONS

Report of Design Capacities prepared by V. N. Tolat, PE

	Product Model	No. of Pages	Date	Signature
1.	NVBM24	7 through 8	05/05/03	V. N. Tolat, PE
2	NVRT	9 through 14	05/05/03	V. N. Tolat PE.
3.	NVTA & NVTAS	1 through 6	05/05/03	V. N. Tolat, PE
4.	NVTA & NVTAS	1 through 14	02/06/03	V. N. Tolat, PE
5.	NVRT	15 through 15	07/07/03	V. N. Tolat, PE

D STATEMENTS

1. No Financial Interest and code compliance letter issued by Vipin N. Tolat PE. on 03/26/03 signed and sealed by V. N. Tolat, PE.
2. Letter to Nu-Vue Industries issued by Building Code Compliance Office on 08/10/04and signed by C. F. Font PE.

01/13/05

Candido F. Font PE.
Sr. Product Control Examiner
NOA No 04-1202.01
Expiration Date: August 21, 2008
Approval Date: January 13, 2005

Do Not Use / USE NVSTA

This Pg

TABLE 1

Truss Anchors NVTA and Riveted Truss Anchors with Seat NVTAS

H Length (in)	Product Code		Gauge seat	Gauge strap	Fasteners 10d or 16d	Maximum Allowable Load (lbs) Uplift 10d or 16d	L1 10d or 16d	L2 10d or 16d
16	NVTA-16	NVTAS 212	20	14	6	646	250	500
					7	754		
					8	862		
18	NVTA-18	NVTAS 214	20	14	9	969	250	500
					10	1077		
20	NVTA-20	NVTAS 216	20	14	11	1185	250	500
					12	1293		
22	NVTA-22	NVTAS 218	20	14	13	1400	250	500
					14	1508		
24	NVTA-24	NVTAS 220	20	14	15	1616	250	500
					16	1724		
26	NVTA-26	NVTAS 222	20	14	17	1832	250	500
					18	1939		
28	NVTA-28	NVTAS 224	20	14	19	2047	250	500
					20	2155		
30	NVTA-30	NVTAS 226	20	14	21	2263	250	500
					22	2371		
36	NVTA-36	NVTAS 232	20	14	25	2694	250	500
48	NVTA-48	NVTAS 244	20	14	33	3557	250	500

TABLE 2

Truss Anchors NVTA and Riveted Truss Anchors with Seat NVTAS

H Length (in)	Product Code		Gauge seat	Gauge strap	Fasteners 10d x 1.5"	Maximum Allowable Load (lbs) Uplift	L1	L2
16	NVTA-16	NVTAS 212	20	14	6	890	350	500
					7	1038	505	585
					8	1187	520	630
18	NVTA-18	NVTAS 214	20	14	9	1335	520	630
					10	1483		
20	NVTA-20	NVTAS 216	20	14	11	1632	520	630
					12	1780		
22	NVTA-22	NVTAS 218	20	14	13	1928	520	630
					14	2077		
24	NVTA-24	NVTAS 220	20	14	15	2225	520	630
					16	2373		
26	NVTA-26	NVTAS 222	20	14	17	2522	520	630
					18	2670		
28	NVTA-28	NVTAS 224	20	14	19	2818	520	630
					20	2967		
30	NVTA-30	NVTAS 226	20	14	21	3115	520	630
					22	3263		
36	NVTA-36	NVTAS 232	20	14	25	3708	520	630
48	NVTA-48	NVTAS 244	20	14	33	4896	520	630

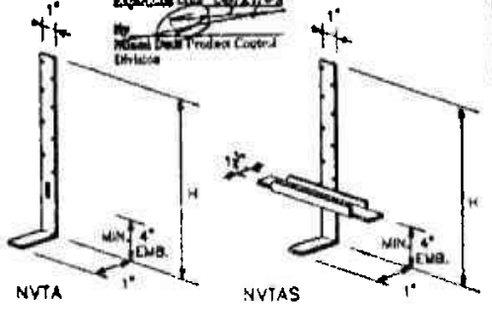
General Notes:

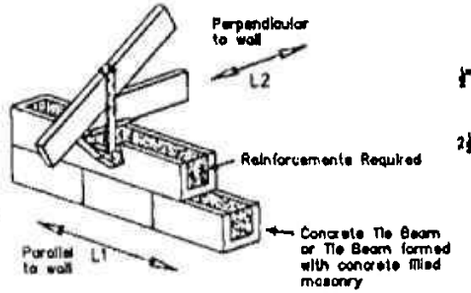
1. Steel shall conform to ASTM A653, structural grade 33 (min. yield 33 ksi) and a minimum galvanized coating of G 60 per ASTM A525.
2. Allowable loads are based on National Desing specifications (NDS) for wood construction, 1997 Edition.
3. Design loads are for Southern Pine species with a specific gravity of 0.55. Allowable loads for other species shall be adjusted accordingly.
4. Nail values are based on NDS table 12.3F, C=0.55 and have been reduced for Penetration Depth factor Cd.
5. Allowable loads for wind uplift have already been increased by a duration factor of 33% for uncharacteristic. This increase is not allowed for steel stress if dead load and wind loads are combined.
6. [redacted]
7. Allowable loads for more than one direction for a single connection cannot be added together. A design load which can be divided into components in the directions given must be evaluated as follows:

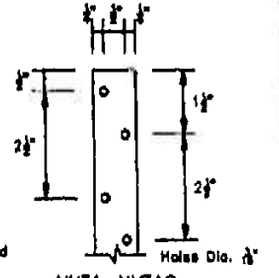
$$\frac{\text{Actual Uplift}}{\text{Allowable Uplift}} + \frac{\text{Actual L1}}{\text{Allowable L1}} + \frac{\text{Actual L2}}{\text{Allowable L2}} <= 1.0$$

8. Allowable loads are based on 1½" thick wood members unless otherwise noted.
9. All tie beams and grouted concrete masonry shall comply with chapter 21 of FBC. Concrete for tie beams and grout and mortar for concrete masonry shall be a min. of 2500 psi. Concrete masonry shall comply with ASTM C90.
10. All tests have been conducted in accordance with ASTM D-1761.

PRODUCT REVISED
as complying with the Florida Building Code
Acceptance No. 04-1208.01
Expiration Date 08/11/05
By
Miami Dade Product Control Division







NVTA, NVTAS

Approved as complying with the Florida Building Code
Date 07/21/03
NOA# 03-0527.18
Miami Dade Product Control Division
By

VIPIN N. TOLAT, PE
(CIVIL)
FL. REG. # 12547

7/21/03

Nu-Vue Industries, Inc.
1058-1059 East 29 Street
Hialeah, Florida 33013
(305) 694-0397
FAX: (305) 694-0398

NVTA AND NVTAS TRUSS ANCHORS

DWG #:	Sheet:	Date:	Revisions: July 7, 2003
NU-2	1 of 3	FEB. 13, 2003	

TABLE 3

NVBH 24 BUTTERFLY HANGER

SIZE	PRODUCT CODE	GAUGE	FASTENER SCHEDULE		ALLOWABLE LOADS (lbs.)	
			HEADER 8d	JOIST 8d	DOWNWARD GRAVITY LOADS 100%	WIND UPLIFT LOAD 133%
2x4	NVBH24	18	12	8	1113	364

Notes:

1. Use all specified fasteners in schedule to achieve values indicated.
2. Values are based on 1½" header and joist thickness.
3. See General Notes, Sheet 1.

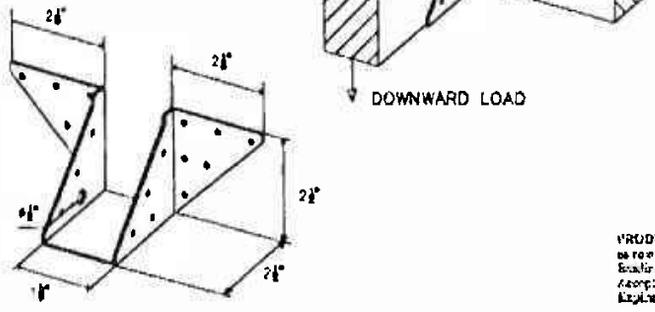
TABLE 4

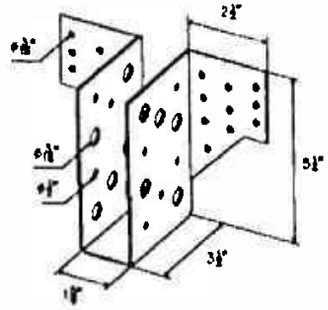
NVUH 26 JOIST HANGER

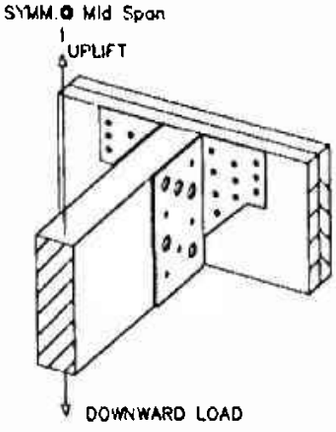
SIZE	PRODUCT CODE	GAUGE	FASTENER SCHEDULE		ALLOWABLE LOADS (lbs.)	
			HEADER 16d	JOIST 10d x 1½"	DOWNWARD GRAVITY LOADS 100%	WIND UPLIFT LOAD 133%
2x6	NVUH26	14	20	10	2233	1213

Notes:

1. Use all specified fasteners in schedule to achieve values indicated.
2. Values are based on 3" header thickness and 1½" joist thickness.
3. See General Notes, Sheet 1.







PRODUCT REVISED as complying with the Florida Building Code Accepted No. 04-1202.01 Expiration Date: 04/01/05

By: Miami-Dade Product Control Division

Approved as complying with the Florida Building Code Date: 5/21/03 NOA# 03-0331.01 Miami-Dade Product Control Division By:

VIPIN N. TOLAT, PE (CIVIL) FL. REG. # 12847

5/21/03

Nu-Vue Industries, Inc.
1063-1069 East 28 Street
Hialeah, Florida 33013
(305) 694-0397
FAX: (305) 694-0398

NVBH 24 AND NVUH 26 HANGERS

DWG #:	Sheet:	Date:	Revisions: July 7, 2003
NU-2	2 of 3	FEB. 18, 2003	

TABLE 5

NVRT Flat and Twisted Rafter Ties

Length (in)	Product Code	Gauge	16d Fasteners		Maximum Uplift Load (lbs)	
			TOTAL	In each member*	Flat Ties	Twisted Ties
12	NVRT-12	14	6	3	454	454
			8	4	605	605
16	NVRT-16	14	8	4	605	605
			10	5	756	756
			12	6	907	907
18	NVRT-18	14	10	5	756	756
			12	6	907	907
			14	7	1059	1059
20	NVRT-20	14	12	6	907	907
			14	7	1059	1059
			16	8	1135	1125
22	NVRT-22	14	14	7	1059	1059
			16	8	1135	1125
			18	9	1135	1125
24	NVRT-24	14	14	7	1059	1059
			16	8	1135	1125
			18	9	1135	1125
30	NVRT-30	14	14	7	1059	1059
			16	8	1135	1125
			18	9	1135	1125
36	NVRT-36	14	14	7	1059	1059
			16	8	1135	1125
			18	9	1135	1125
48	NVRT-48	14	14	7	1059	1059
			16	8	1135	1125
			18	9	1135	1125

Notes:

1. Specify "F" for Flat and "T" for Twisted when ordering.
2. Fastener values are based on a minimum 1½" thick wood members.
3. * Indicates no. of nails in each connected wood member.
4. See General Notes, sheet 1.

PRODUCT REVISED
as complying with the Florida Building Code
Acceptance No. 04-1209.01
Expiration Date: 04/21/08
By
Miami Dade Product Control Division

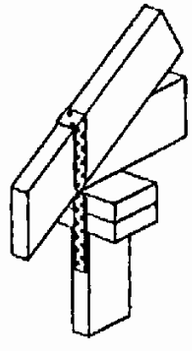
TABLE 6

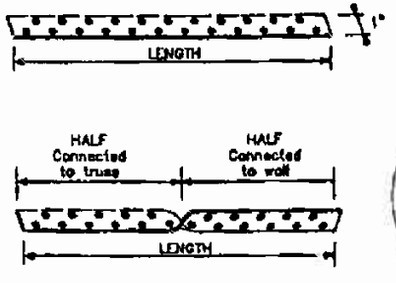
NVRT Twisted Rafter Ties to Concrete Tie Beams or Concrete Filled Masonry

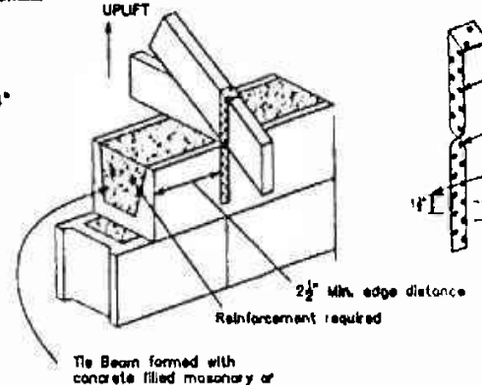
Length (in)	Product Code	Gauge	No. of 16d nails to Wood Framing	No. of ¼" diameter Tapcons to Concrete	Maximum Uplift Load (lbs)
12	NVRT-12	14	3	3	454
			4	3	525
16	NVRT-16	14	4	3	575
			5	4	704
			6	4	704
18	NVRT-18	14	5	4	704
			6	4	704
			7	5	880
20	NVRT-20	14	6	4	704
			7	5	880
			8	5	880
22	NVRT-22	14	7	5	880
			8	5	880
			9	5	880
24	NVRT-24	14	7	5	880
			8	5	880
			9	5	880
30	NVRT-30	14	7	5	880
			8	5	880
			9	5	880
36	NVRT-36	14	7	5	880
			8	5	880
			9	5	880
48	NVRT-48	14	7	5	880
			8	5	880
			9	5	880

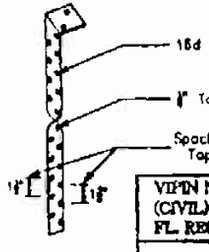
Notes:

1. ITW tapcons shall be embedded a minimum of 1¼" into concrete tiebeam or tiebeam formed with concrete filled masonry. ITW tapcons shall have a min. edge distance of 2½" and minimum spacing of 1½" as shown.
2. See General Notes, sheet 1.

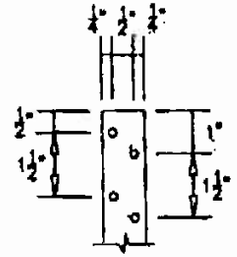








Approved as complying with the Florida Building Code
Date 7/21/03
NOA# 04-1209.01
Miami Dade Product Control Division
By



VIPIN N. TOLAT, PE
(CIVIL)
FL. REG. # 12847

7/21/03

Nu-Vue Industries, Inc.
1053-1059 East 29 Street
Hialeah, Florida 33013
(305) 694-0397
FAX: (305) 694-0398

NVRT FLAT AND TWISTED RAFTER TIES

DWG #:	Sheet:	Date:	Revisions: July 7, 2003
NU-2	3 of 3	FEB. 13, 2003	

MIAMI-DADE

BUILDING CODE COMPLIANCE OFFICE (BCCO)
PRODUCT CONTROL DIVISION

MIAMI-DADE COUNTY, FLORIDA
METRO-DADE FLAGLER BUILDING
140 WEST FLAGLER STREET, SUITE 1603
MIAMI, FLORIDA 33130-1563
(305) 375-2901 FAX (305) 375-2908

NOTICE OF ACCEPTANCE (NOA)

NU-VUE Industries Inc.
1055 East 29 Street.
Hialeah, Florida 33013

SCOPE:
This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed by Miami-Dade County Product Control Division and accepted by the Board of Rules and Appeals (BORA) to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Division (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. BCPRC reserves the right to revoke this acceptance, if it is determined by Miami-Dade county Product Control Division that this product or material fails to meet the requirements of the applicable building code.
This product is approved as described herein, and has been designed to comply with the High velocity Hurricane Zone of the Florida Building Code.

DESCRIPTION: Wood Connectors.

APPROVAL DOCUMENT: Drawing No.Nu-3, titled "NVJH Joist Supports, NVTP & NVTPH Plate Anchors NVSO 236 Joist Hanger, NVHC 43 & NVHC 42/2 Hurricane Clip,"sheet 1, 2 & 3 of 3, dated 07/10/03 with last revision on 12/10/03, prepared by Nu-Vue Industries Inc signed and sealed by V. N. Tolat, PE, bearing the Miami-Dade County Product Control Approval stamp with the Notice of Acceptance (NOA) number and approval date by the Miami-Dade County Product Control Division.

MISSILE IMPACT RATING: None

LABELING: Each unit shall bear a permanent label with the manufacturer's name or logo, city, state and following statement: "Miami-Dade County Product Control Approved", unless otherwise noted herein.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.
This NOA consists of this page 1, evidence page & approval document mentioned above.
The submitted documentation was reviewed by Candido E. Font PE.

01/01/04

NOA No: 03-0730.03
Expiration Date: January1, 2009
Approval Date: January 1, 2004

Nu-Vue Industries, Inc.

NOTICE OF ACCEPTANCE: EVIDENCE PAGE

A DRAWINGS

1. Drawings prepared by Nu-Vue Industries Inc, titled "NVJH Joist Support, NVTP & NVTPH Plate Anchors, NVSO 236 Joist Hanger, NVHC 43 & NVHC 43/2 Hurricane Clip", Drawing No. NU-3, sheets No. 1, 2 & 3 of 3, dated 07/10/03 with last revision on 12/10/03, signed and sealed by V. N. Tolat, PE.

B TEST

Test reports on wood connectors per ASTM D1761 by Product Testing Inc, signed and sealed by C. R. Caudel, PE.

	Report No.	Wood Connector	Direction	Date
1.	PT # 03-4303	NVTP4	Upward	04/21/03
2	PT # 03-4343	NVTP4H	Upward	05/05/03
3	PT # 03-4344	NVTP4	Upward	05/01/03
4.	PT # 03-4345	NVTP4H	Upward	05/02/03
5	PT # 03-4349	NVSO236	Up & Down	05/19/03
6.	PT # 03-4357	NVSO236	Up & Down	05/20/03
7.	PT # 03-4358	NVJH24	Up & Down	05/30/03
8.	PT # 03-4385	NVJH26	Up & Down	05/30/03
9.	PT # 03-4386	NVSO236	Up & Down	05/13/03
10	PT # 03-4387	NVJH28	Up & Down	05/30/03

C CALCULATIONS

Report of Design Capacities prepared by V. N. Tolat on 07/22/33, sheet 1 through 13, signed and sealed by V. N. Tolat PE.

D QUALITY ASSURANCE

1. Product Control Division

E STATEMENTS

1. No interest and code compliance letter issue by V. N. Tolat on 07/26/03 signed and sealed by V. N. Tolat, PE.

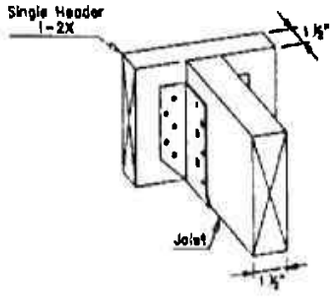
01/01/04

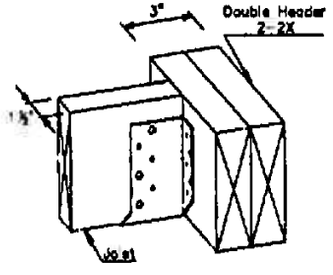
Candido F. Font PE.
Sr. Product Control Examiner
NOA No 03-0730.03
Expiration Date: January 1, 2009
Approval Date: January 1, 2004

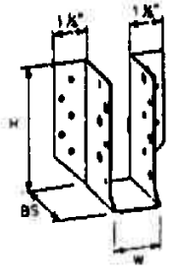
TABLE 1

JOIST SUPPORTS

'18 G NVJH JOIST SUPPORTS										Allowable Loads (lbs)		
Product Code	Dimension (inches)			Joist Size	Double Header Size	Single Header Size	Fasteners			Gravity Loads 100%		Uplift Loads 133%
	W	H	BS				Double Header	Single Header	Joists	Double Header	Single Header	Single & Double Headers
NVJH24	1⅝	3⅝	3	2x4 2x6	2-2x4 2-2x6	2x4 2x6	8-10d	8-10d x 1½"	4-10d x 1½"	726	613	493
NVJH26	1⅝	5	3	2x6 2x8	2-2x6 2-2x8	2x6 2x8	10-10d	10-10d x 1½"	6-10d x 1½"	1210	1022	818
NVJH28	1⅝	6⅜	3	2x8 2x10 2x12	2-2x8 2-2x10 2-2x12	2x8 2x10 2x12	14-10d	14-10d x 1½"	7-10d x 1½"	1694	1431	954







NVJH 28 as shown
NVJH 24 & 28
similar but with
different holes.

General Notes:

1. Steel shall conform to ASTM A653, structural grade 33 (Min. yield 33 ksi) and a minimum galvanized coating of G 60 per ASTM A525.

2. Allowable loads are based on National Desing specifications (NDS) for wood construction, 1997 Edition.

3. Design loads are for Southern Pine species with a specific gravity of 0.55. Allowable loads for other species shall be adjusted accordingly.

4. Nail values are based on NDS table 12.3F, G=0.55 and have been reduced for Penetration Depth factor Cd.

5. Allowable loads for wind uplift and lateral loads have already been increased by a duration factor of 33% for anchor nail. This increase is not allowed for steel stress if dead load and wind loads are combined.

6. Allowable loads for more than a single connetion cannot be added together. A design load which be divided into components in the direction given must be evaluated as follows:

$$\frac{\text{Actual Uplift}}{\text{Allowable Uplift}} + \frac{\text{Actual L1}}{\text{Allowable L1}} + \frac{\text{Actual L2}}{\text{Allowable L2}} \leq 1.0$$

7. Allowable loads are base on 1 ½" thick wood members unless otherwise noted.

8. All tests have been conducted in accordance with ASTM D-1761.

VIPIN N. TOLAT, PE (CIVIL)
FL. REG. # 12847
15123 LANTER CREEK LANE
HOUSTON, TX 77068

12/10/03

Approved as complying with the Florida Building Code
Date 3/9/04
NOA# 03-0910.05
Miami-Dade Product Control
Division
By

Nu-Vue Industries, Inc.
1058-1059 East 29 Street
Hialeah, Florida 33013
(305) 694-0397
FAX: (305) 694-0398

NVJH JOIST SUPPORTS

DWG #:	Sheet:	Date:	Revisions: (1) 11/10/03 (2) 12/11/03
NU-3	1 of 3	JULY 10, 2003	

TABLE 2

TOP PLATE ANCHORS
NVTP & NVTPH

Size	Product Code	Gauge	Dimensions (in)	
			A	B
2x4/4x4	NVTP4	20	3 1/8"	8"
2x6/4x6	NVTP6	20	5 1/8"	8"
2x8/4x8	NVTP8	20	7 1/8"	8"
2x4/4x4	NVTP4H	18	3 1/8"	8"
2x6/4x6	NVTP6H	18	5 1/8"	8"
2x8/4x8	NVTP8H	18	7 1/8"	8"

Product code	Total number of fasteners 10d x 1 1/2" Wind/Earthquake Max. Uplift Capacity 133% (lbs)			
	6	8	10	12
NVTP 4,6,8	818	1087	1346	1605
NVTP 4H,6H,8H	818	1090	1363	1636

Notes:

1. See General Notes, Sheet 1.
2. One half of all specified fasteners shall be used on each side of the stud to achieve tabulated values.

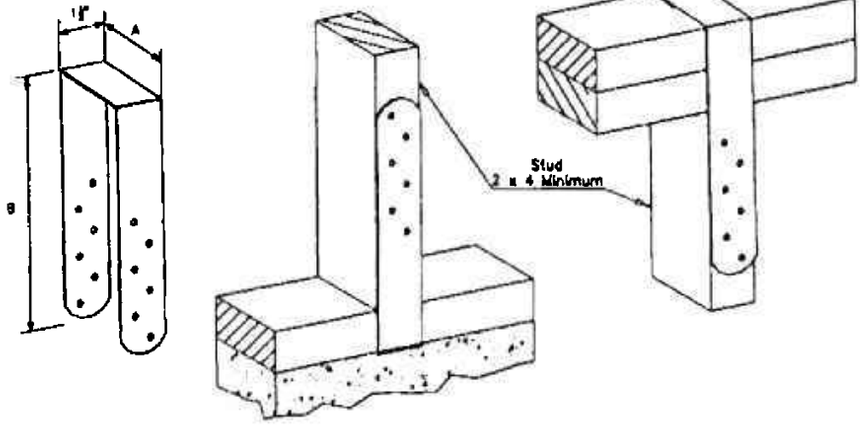


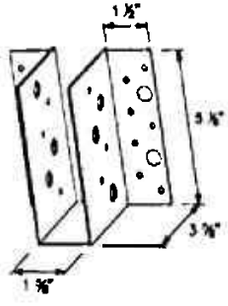
TABLE 3

NVSO 236, 16 GAUGE, HEAVY DUTY
FACE MOUNT JOIST HANGER

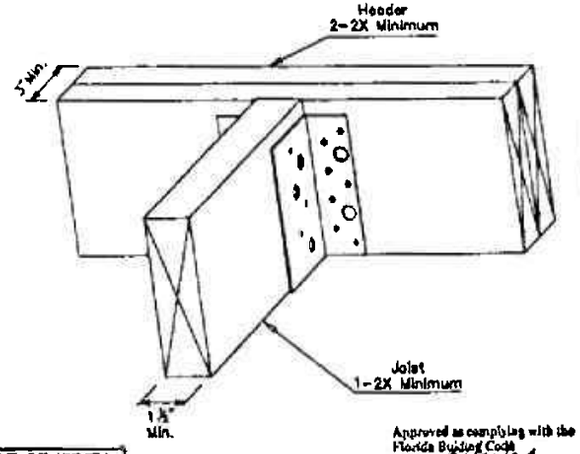
Joist Size	Header Size	Fasteners		Allowable Loads (Lbs.)	
		Header	Joist	GRAVITY 100%	Uplift 133%
2x8-6	2-2x8 2-2x10 2-2x12	14-10d	6-10d	1708	824
		14-16d	6-16d	1875	901
		2-1/4" x 3" Lag Screws	6-16d	1600	901

Notes:

1. See General Notes, Sheet 1.



BOLT HOLES 1/2" DIA.
NAIL HOLE 3/16" DIA.



Approved as complying with the Florida Building Code
Date 01/01/04
NOA# 03-0730.03
Miami-Dade Product Control
Window

VIPIN N. TOLAT, PE (CIVIL)
FL. REG. # 12847
15123 LANTER CREEK LANE
HOUSTON, TX 77068

12/14/03

Nu-Vue Industries, Inc.
1053-1059 East 29 Street
Hialeah, Florida 33013
(305) 694-0397
FAX: (305) 694-0398

NVTP & NVTPH PLATE ANCHORS, NVSO 236 JOIST HANGER

DWG #:	Sheet:	Date:	Revisions: (1) 11/10/03 (2) 12/11/03
NU-3	2 of 3	JULY 10, 2003	

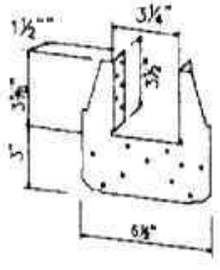
TABLE 4

18 Gauge NVHC 43 & NVHC 43/2 HURRICANE CLIP.

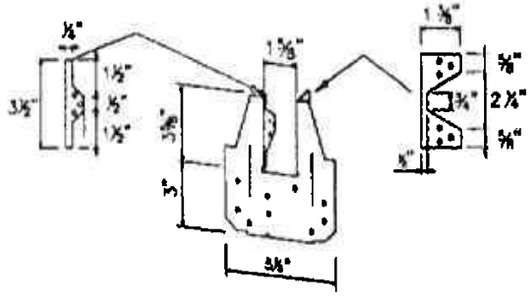
PRODUCT CODE	DESCRIPTION	FASTENERS		DESIGN LOADS (LBS)		
		HEADER	JOIST	UPLIFT	L1	L2
NVHC 43	Hurricane Clip - Wide	9-10d	9-10d	687	407	308
NVHC 43/2	Hurricane Clip - Widex2	10-10d	10-10d	917	547	432

Notes:

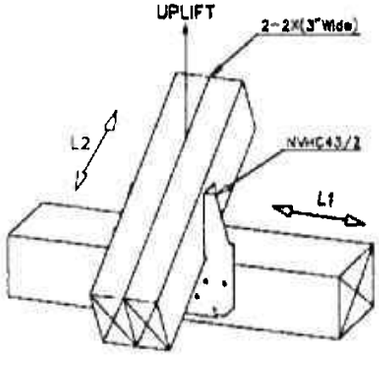
1. See General Notes, Sheet 1.

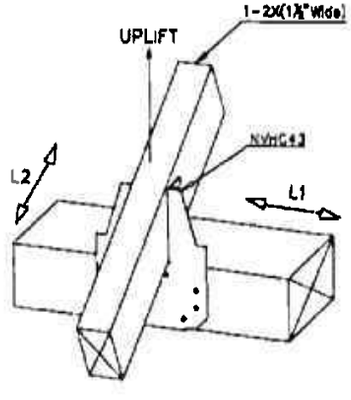


NVHC 43/2



NVHC 43





VIPIN N. TOLAT, PE (CIVIL)
FL. REG. # 12847
15123 LANTER CREEK LANE
HOUSTON, TX 77068

12/14/03

Approved as complying with the Florida Building Code
Date 12/12/03
FAC/A 03-0715.03
Miami-Dade Product Control Division
By

Nu-Vue Industries, Inc.
1058-1060 East 29 Street
Hialeah, Florida 33013
(305) 694-0397
FAX (305) 694-0398

NVHC 43 & NVHC 43/2 HURRICANE CLIP.

DWG #:	Sheet:	Date:	Revisions: (1) 11/10/03 (2) 12/11/03
NU-3	3 of 3	JULY 10, 2003	

MIAMI-DADE COUNTY, FLORIDA
METRO-DADE FLAGLER BUILDING

BUILDING CODE COMPLIANCE OFFICE
METRO-DADE FLAGLER BUILDING
140 WEST FLAGLER STREET, SUITE 1603
MIAMI, FLORIDA 33130-1563
(305) 375-2901 FAX (305) 375-2908

CONTRACTOR LICENSING SECTION
(305) 375-2527 FAX (305) 375-2558

CONTRACTOR ENFORCEMENT DIVISION
(305) 375-2966 FAX (305) 375-2908

PRODUCT CONTROL DIVISION
(305) 375-2902 FAX (305) 372-6339

PRODUCT CONTROL NOTICE OF ACCEPTANCE

United Steel Products Company
703 Rogers Drive (P. O. Box 80)
Montgomery ,MN 56069

Your application for Notice of Acceptance (NOA) of:
Face Mount Joist Hangers And Framing Angles
under Chapter 8 of the Code of Miami-Dade County governing the use of Alternate Materials and Types of Construction, and completely described herein, has been recommended for acceptance by the Miami-Dade County Building Code Compliance Office (BCCO) under the conditions specified herein.

This NOA shall not be valid after the expiration date stated below. BCCO reserves the right to secure this product or material at any time from a jobsite or manufacturer's plant for quality control testing. If this product or material fails to perform in the approved manner, BCCO may revoke, modify, or suspend the use of such product or material immediately. BCCO reserves the right to revoke this approval, if it is determined by BCCO that this product or material fails to meet the requirements of the South Florida Building Code.

The expense of such testing will be incurred by the manufacturer.

ACCEPTANCE NO.: 01-0417.11
EXPIRES: 07/09/2006

Raul Rodriguez
Chief Product Control Division

THIS IS THE COVERSHEET, SEE ADDITIONAL PAGES FOR SPECIFIC AND GENERAL CONDITIONS
BUILDING CODE & PRODUCT REVIEW COMMITTEE

This application for Product Approval has been reviewed by the BCCO and approved by the Building Code and Product Review Committee to be used in Miami-Dade County, Florida under the conditions set forth above.

Francisco J. Quintana, R.A.
Director
Miami-Dade County
Building Code Compliance Office

APPROVED: 06/28/2001

United Steel Products Company.

ACCEPTANCE NO: 01-0417.11

APPROVED: JUN 2 8 2001

EXPIRES: 07/09/2006

NOTICE OF ACCEPTANCE: SPECIFIC CONDITIONS

1. SCOPE

1.1 This renews the Notice of Acceptance No. 98-0421.06, which was issued on 07/09/98. It approves wood connectors; as described in Section 2 of this Notice of Acceptance, designed to comply with the South Florida Building Code (SFBC), 1994 Edition for Miami-Dade County. For the locations where the actual loads as determined by SFBC Chapter 23, do not exceed the design load indicated in the approved drawings.

2. PRODUCT DESCRIPTION

2.1 The **United Steel Wood Connectors** shall be fabricated and used in strict compliance with the following documents: Drawing No. JUS/JA/MP, sheet 1 of 1, titled "Face Mount Joist Hangers and Framing Angles", prepared by United Steel Products Company, dated 04/06/01 with no revisions. The drawings shall bear the Miami-Dade County Product Control Approval stamp with the Notice of Acceptance number and approval date by the Miami-Dade Product Control Division. These documents shall hereinafter be referred to as the approved drawings.

3. LIMITATIONS

3.1 Allowable loads are for Douglas Fir or better with a specific gravity of 0.50 and moisture content of 19% or less.

3.2 Allowable loads are based on testing per ASTM D1761 and calculations per National Design Specifications for Wood Construction 1991 Edition & 1993 Errata.

4. INSTALLATION

4.1 The wood connectors shall be installed in strict compliance with the approved drawings.

5. LABELING

5.1 Each wood connector shall bear a permanent label with the manufacturer's name or logo, city, state and the following statement: "Miami-Dade County Product Control Approved".

6. BUILDING PERMIT

6.1 Application for Building Permit shall be accompanied by copies of the following:

6.1.1 This Notice of Acceptance

6.1.2 Duplicate copies of the approved drawings as identified in Section 2 of this Notice of Acceptance, clearly marked to show the hangers and angles selected for the proposed installation.

6.1.3 Any other document required by the Building Official or the SFBC in order to properly evaluate the installation of these products.

Candido Font, PE, Sr. Product Control Examiner
Product Control Division

United Steel Products Company.

ACCEPTANCE NO.: 01-0417.11

APPROVED: JUN 2 8 2001

EXPIRES: 07/09/2006

NOTICE OF ACCEPTANCE STANDARD CONDITIONS

1. Renewal of this Acceptance (approval) shall be considered after a renewal application has been filed and the original submitted documentation, including test-supporting data, engineering documents, are no older than eight (8) years.

2. Any and all approved products shall be permanently labeled with the manufacturer's name, city, state, and the following statement: "Miami-Dade County Product Control Approved", or as specifically stated in the specific conditions of this Acceptance.

3. Renewals of Acceptance will not be considered if:
a) There has been a change in the South Florida Building Code affecting the evaluation of this product and the product is not in compliance with the code changes;
b) The product is no longer the same product (identical) as the one originally approved;
c) If the Acceptance holder has not complied with all the requirements of this acceptance, including the correct installation of the product;
d) The engineer who originally prepared, signed and sealed the required documentation initially submitted is no longer practicing the engineering profession.

4. Any revision or change in the materials, use, and/or manufacture of the product or process shall automatically be cause for termination of this Acceptance, unless prior written approval has been requested (through the filing of a revision application with appropriate fee) and granted by this office.

5. Any of the following shall also be grounds for removal of this Acceptance:
a) Unsatisfactory performance of this product or process.
b) Misuse of this Acceptance as an endorsement of any product, for sales, advertising or any other purpose.

6. The Notice of Acceptance number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the Notice of Acceptance is displayed, then it shall be done in its entirety.
7. A copy of this Acceptance as well as approved drawings and other documents, where it applies, shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at all time. The engineer need not reseal the copies.

8. Failure to comply with any section of this Acceptance shall be cause for termination and removal of Acceptance.

9. This Notice of Acceptance consists of pages 1, 2 and this last page 3.

Candido Font, PE, Sr. Product Control Examiner
Product Control Division

END OF THIS ACCEPTANCE

United Steel Products Company.

ACCEPTANCE NO: 01-0417.11

APPROVED: JUN 2 8 2001

EXPIRES: 07/09/2006

NOTICE OF ACCEPTANCE: SPECIFIC CONDITIONS
(For File ONLY. Not part of NOA)

A DRAWINGS

1. Drawings prepared by United Steel Products Company, titled "Face Mount Joist Hangers and Framing Angles", Drawing No. JUS/JA/MP, sheet 1 of 1, dated 04/06/01 with no revisions, signed and sealed by T. A. Kolden, PE.

B TEST

Test reports on wood connectors per ASTM D1761 by Maxim Technologies Inc., signed and sealed by S. L. Muschinske, PE.

	Report No.	Wood Connector	Direction	Date
1.	3013-71-5327	JUS Series	Upward & Down	03/06/98
2.	3013-71-5327	JA Series	Upward & Down	03/06/98
3.	3013-71-5327	MP Series	Upward & Down	03/06/98

C CALCULATIONS

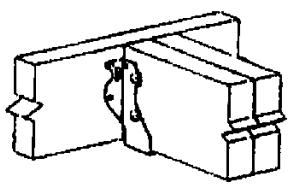
1 JUS Series Calculations prepared by T. A. Kolden, signed and sealed by T. A. Kolden PE on 04/10/2001.

D STATEMENTS

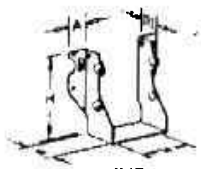
1. No Financial Interest & Code Compliance letter issued by Thomas A. Kolden, PE., signed by T.A. Kolden, PE. and notarized on 03/19/98 by M. E. Steinhouse.
2. Merger Documents issued by the State of Minnesota on 02/14/200 and signed by the Secretary of State, M. Kiffmeyer.

Candido Font, PE, Sr. Product Control Examiner
Product Control Division

JUS JOIST HANGERS



TYPICAL JUS INSTALLATION



JUS

PRODUCT CODE	STEEL GAUGE	DIMENSIONS (in.) V	H	D	A	DI	FASTENERS HEADER	JOIST	ALLOWABLE LOADS (lbs.) DOWN	UPLIFT
JUS24-2	18	2 1/8	3 1/2	2	1	1	4 - 16d	2 - 16d	715	NA
JUS26-2	18	2 1/8	5 1/4	2	1	1	4 - 16d	4 - 16d	1005	1215
JUS28-2	18	2 1/8	7 1/4	2	1	1	6 - 16d	4 - 16d	1278	1215
JUS210-2	18	2 1/8	9 1/4	2	1	1	8 - 16d	6 - 16d	1775	1820
JUS212-2	18	2 1/8	11 1/4	2	1	1	10 - 16d	6 - 16d	2040	1820
JUS44	18	3 5/8	3 1/4	2	1	1	4 - 16d	2 - 16d	715	NA
JUS46	18	3 5/8	5	2	1	1	4 - 16d	4 - 16d	1005	1215
JUS48	18	3 5/8	7	2	1	1	6 - 16d	4 - 16d	1278	1215
JUS410	18	3 5/8	9	2	1	1	8 - 16d	6 - 16d	1775	1820
JUS412	18	3 5/8	11	2	1	1	10 - 16d	6 - 16d	2040	1820
JUS24-3	18	4 5/8	2 3/4	2	1	1	4 - 16d	2 - 16d	715	NA
JUS26-3	18	4 5/8	4 1/2	2	1	1	4 - 16d	4 - 16d	1005	1215
JUS28-3	18	4 5/8	6 1/2	2	1	1	6 - 16d	4 - 16d	1278	1215
JUS210-3	18	4 5/8	8 1/2	2	1	1	8 - 16d	6 - 16d	1775	1820
JUS212-3	18	4 5/8	10 1/2	2	1	1	10 - 16d	6 - 16d	2040	1820

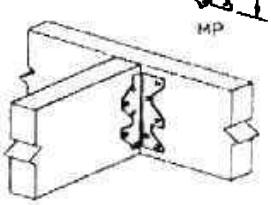
1 Minimum header thickness shall be 2 inches for 16d nails.

2 Specified joist nails shall be installed at 30 - 45 degrees horizontally such that they penetrate through the end of the joist and into the header.

MP & JA FRAMING ANGLE CONNECTORS

JA

MP



TYPICAL MP INSTALLATION
JA INSTALLATION SIMILAR

PRODUCT CODE	STEEL GAUGE	DIMENSIONS (in.) V	H	D	A	FASTENERS HEADER	JOIST	ALLOWABLE LOADS (lbs.) DOWN	UPLIFT
JAJG	14	---	3	2 1/2	2 1/2	4 16d	4 10d x 1 1/2	265	NA
JAGG	14	---	5	2 1/2	2 1/2	6 16d	6 10d x 1 1/2	550	735
MP34	18	---	2 13/16	1 3/8	1 3/8	3 8d x 1 1/2	3 8d x 1 1/2	265	NA
MP4J	18	---	4 1/2	1 3/8	1 3/8	5 8d x 1 1/2	5 8d x 1 1/2	445	NA

1 Minimum header thickness shall be 2 inches for 16d nails, and 1 1/2 inches for all others specified.

2 Load values are for Framing Angles installed singly, and may be doubled if installed in pairs.

3 The joist shall be a minimum of 1 1/2 inches thick.

APPROVED AS SUBMITTED PER 2000 IRC
SOUTH FLORIDA BUILDING CODE
DATE JUN 28 2001
BY
PRODUCT CONTROL
BUILDING CODE COMPLIANCE OFFICE
ACCEPTANCE NO 01-0911.11

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653, STRUCTURAL GRADE 33, U.O.N. AND A MINIMUM GALVANIZED COATING OF G-60
2) FASTENERS ARE COMMON WIRE NAILS UNLESS OTHERWISE NOTED
3) ALLOWABLE UPLIFT LOADS HAVE BEEN INCREASED BY A SHORT TERM DURATION FACTOR OF 33% FOR WIND LOAD CONDITION. NO FURTHER INCREASE ALLOWED.
4) ALLOWABLE DOWN LOADS ARE NOT INCREASED BY SHORT TERM DURATION FACTOR
5) ALLOWABLE LOADS ARE BASED ON THE NATIONAL DESIGN SPECIFICATION FOR WOOD CONSTRUCTION 1997 EDITION. FOR DOUGLAS FIR G= 0.50 OR BETTER) & TEST PERFORMED IN ACCORDANCE WITH ASTM D1761

UNITED STEEL PRODUCTS CO
703 ROGERS DRIVE
MONTGOMERY MN 56069

FACE MOUNT JOIST HANGERS AND FRAMING ANGLES

DATE: 04/06/01
DRAWING NUMBER: JUS/JAG/MP
SHEET NUMBER: 1 OF 1

5.24.2001

THOMAS A. KOLDEN, P.E.
CIVIL NUMBER FL#50899

MIAMI-DADE

BUILDING CODE COMPLIANCE OFFICE (BCCO)
PRODUCT CONTROL DIVISION

MIAMI-DADE COUNTY, FLORIDA
METRO-DADE FLAGLER BUILDING
140 WEST FLAGLER STREET, SUITE 1603
MIAMI, FLORIDA 33130-1563
(305) 375-2901 FAX (305) 375-2908

NOTICE OF ACCEPTANCE (NOA)

United Steel Products Co.
703 Rogers Drive.
Montgomery, MN 56069

SCOPE:

This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed by Miami-Dade County Product Control Division and accepted by the Board of Rules and Appeals (BORA) to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Division (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. BCPRC reserves the right to revoke this acceptance, if it is determined by Miami-Dade county Product Control Division that this product or material fails to meet the requirements of the applicable building code.
This product is approved as described herein, and has been designed to comply with the High velocity Hurricane Zone of the Florida Building Code.

DESCRIPTION: Wood Connectors UGTS Series & LUGT2

APPROVAL DOCUMENT: Drawing No.102-2, titled UGTS SERIES (Uplift Girder Tie down Short) & LUGT2 (Light Uplift Girder Tie down)" sheet 1 & 2, with revision No. 1, dated 04/16/03 & 02/07/03, prepared by United Steel Products Company signed and sealed by R. W. Lutz, PE, bearing the Miami-Dade County Product Control Revision stamp with the Notice of Acceptance (NOA) number and expiration date by the Miami-Dade County Product Control Division.

MISSILE IMPACT RATING: None

LABELING: Each unit shall bear a permanent label with the manufacturer's name or logo, city, state and following statement: "Miami-Dade County Product Control Approved", unless otherwise noted herein.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.
This NOA revises NOA # 02-0128.04 and consists of this page 1, evidence page & approval document mentioned above.
The submitted documentation was reviewed by Candido F. Font PE.

10/09/03

NOA No: 03-0219.02
Expiration Date: June 20, 2007
Approval Date: October 09, 2003

United Steel Products Company.

NOTICE OF ACCEPTANCE: EVIDENCE PAGE

A DRAWINGS

1. Drawings prepared by United Steel Products Company, titled "UGTS SERIES (Uplift Girder Tie down Short)& LUGT2 (Light Uplift girder Tie down)", Drawing No. 102-2, sheets No. 1 & 2, dated 04/16/03 & 02/07/03 with revision No. 1, signed and sealed by R. W. Lutz, PE.

B. TESTS

Laboratory Report	Test	Date	Engineer.
1. Stork 031205	ASTM D1761	12/30/01	J. D. Lee, PE.
2. Stork 031205	ASTM D1761	11/30/01	J. D. Lee, PE.
3. Stork 031205 Revised	ASTM D1761	04/18/02	J. D. Lee, PE.
4. Stork 301802-508833	ASTM D1761	11/19/02	J. D. Lee, PE

C. CALCULATION

1. Calculated capacities of UGTS/USC Rafter Ties, pages 1 and 2, dated 01/15/02, prepared by R. W. Lutz, PE.
2. Calculated capacities of LUGT2 Light Uplift Girder Tie, pages 1 through 3, dated 01/16/02, prepared by R. W. Lutz, PE.
3. Calculated capacities of UGTS Series Uplift girder Ties, pages 1 through 3, dated 01/31/03 signed and sealed by R. W. Lutz, PE.

D. QUALITY ASSURANCE.

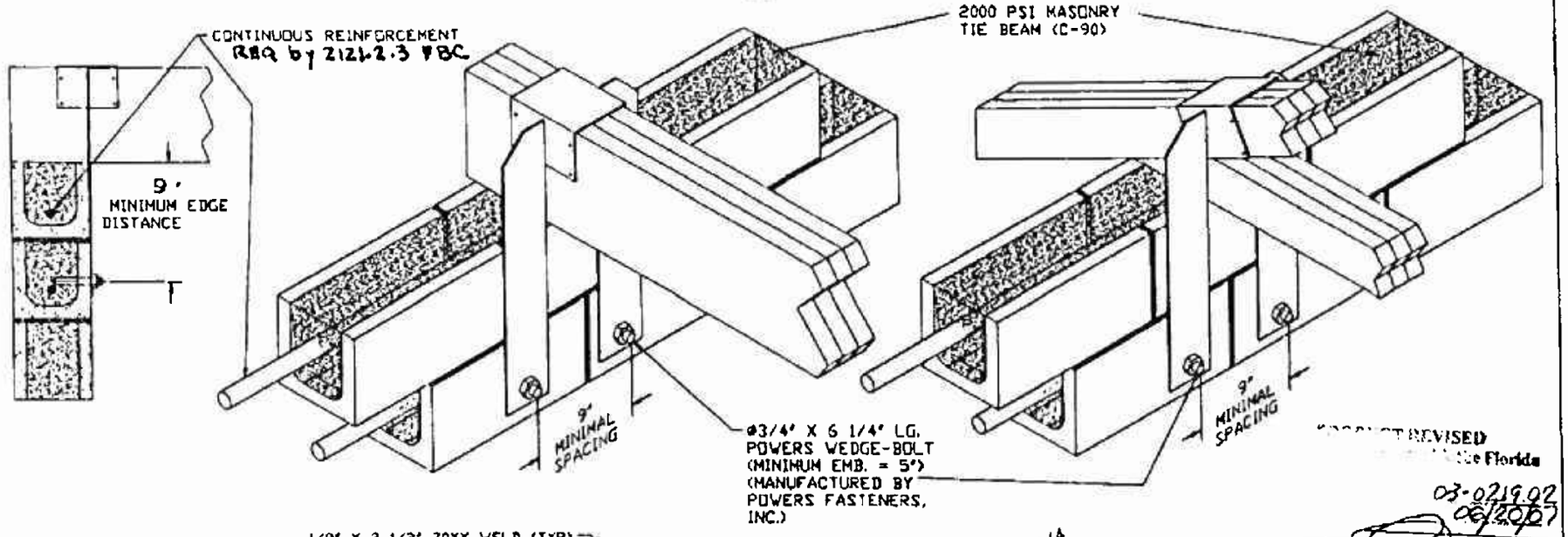
1. Miami-Dade County. Building Code Compliance Office.

E. STATEMENTS

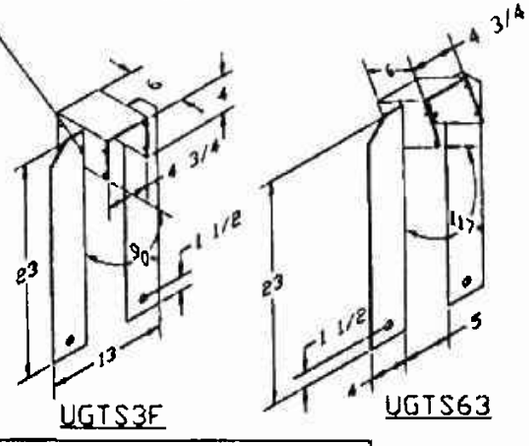
1. No-financial interest letter issued by USP Lumber Connectors on 01/18/02 and signed by R. W. Lutz, PE.
2. No-financial interest letter issued by Twin City Testing Corporation, on 11/30/01, signed by J. D. Lee, PE and notarized by C. L. Jacobson.
3. Code compliance letter, prepared by USP Lumber Connectors on 01/18/01,signed and sealed by R. W. Lutz, PE.
4. Confirmation letter verifying capacity of ¾ Power Wedge Anchor with 5" embedment, prepared by Power Fastener on 07/22/03 and signed by M. Ziegler.
5. Confirmation letter verifying letter of Powers Fasteners to United Steel Products, prepared by Stork Twin City Testing Corporation on 07/30/03, signed and sealed by J. D. Lee, PE.

10/09/03

Candido F. Font PE.
Sr. Product Control Examiner
NOA No 03-0219.02
Expiration Date: June 20, 2007
Approval Date: October 09, 2003



1/8" X 3 1/2" 70XX WELD (TYP)



UGTS3F UGTS63

ALL UGTS PRODUCTS ARE 10 GAUGE MATERIAL.

TRUSS/RAFTER PITCH RANGE BETWEEN 4/12 & 8/12

PRODUCT CODE	PITCH	FASTENERS SCHEDULE			ALLOWABLE LOADS
		RAFTER/ TRUSS	CONCRETE/ MASONRY WALL	WIDTH	UPLIFT 133%
UGTS3F	0/12	(8) 10d	(2) 3/4	4 3/4	6390
UGTS4F	0/12	(8) 10d	(2) 3/4	6 1/2	6390
UGTS43	4/12	(8) 10d	(2) 3/4	4 3/4	6390
UGTS44	4/12	(8) 10d	(2) 3/4	6 1/2	6390
UGTS53	5/12	(8) 10d	(2) 3/4	4 3/4	6390
UGTS54	5/12	(8) 10d	(2) 3/4	6 1/2	6390
UGTS63	6/12	(8) 10d	(2) 3/4	4 3/4	6390
UGTS64	6/12	(8) 10d	(2) 3/4	6 1/2	6390
UGTS73	7/12	(8) 10d	(2) 3/4	4 3/4	6390
UGTS74	7/12	(8) 10d	(2) 3/4	6 1/2	6390
UGTS83	8/12	(8) 10d	(2) 3/4	4 3/4	6390
UGTS84	8/12	(8) 10d	(2) 3/4	6 1/2	6390

Robert W. Lutz
10 Jun 03

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653 STRUCTURAL GRADE 33, AND A MINIMUM GALVINIZED COATING OF G90.
2) FASTENERS ARE COMMON WIRE NAILS UNLESS OTHERVISE NOTED.
3) ALLOWABLE LOADS HAVE BEEN INCREASED BY A SHORT TERM DURATION FACTOR OF33% FOR WIND LOAD CONDITION. NO FURTHER INCREASE IS ALLOWED
4) ALLOWABLE LOADS ARE BASED ON THE NATIONAL DESIGN SPECIFICATIONS FOR WOOD CONSTRUCTION 1997 EDITION FOR SOUTHERN YELLOW PINE (G= 0.55 OR BETTER AND MOISTURE CONTENT OF 19% OR BETTER). ALL TESTS PERFORMED IN ACCORDANCE WITH ASTM D1761.

PRODUCT REVISED
to Florida
03-021902
06/20/03

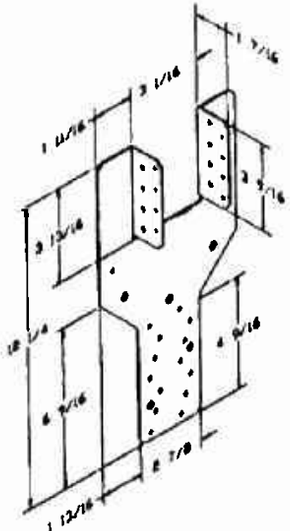
UNITED STEEL PRODUCTS COMPANY
703 ROGERS DRIVE, MONTGOMERY, MN 56069 PHONE (507) 364-7333

NAME: UGTS SERIES
(UPLIFT GIRDER TIEDOWN SHORT)

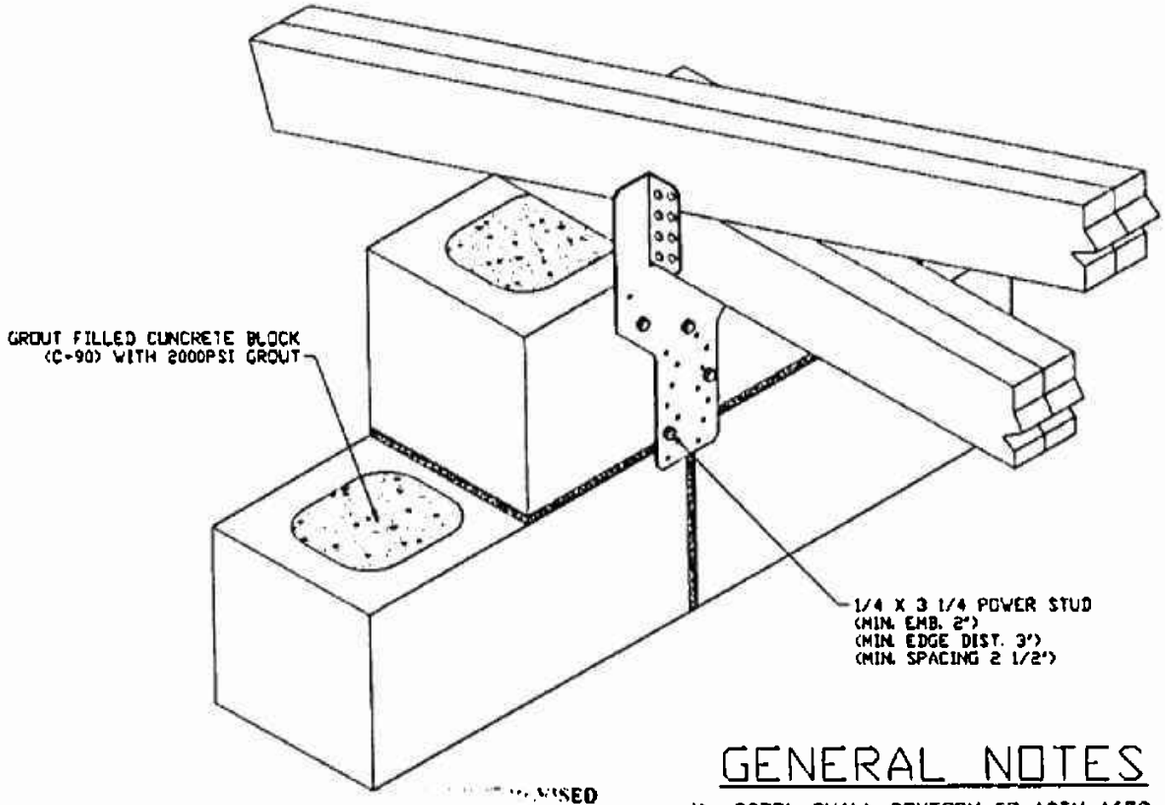
DATE: 4/16/03

ROBERT W. LUTZ
PROFESSIONAL ENGINEER (STRUCTURAL)
FLORIDA REG. NO. 55409

SHEET:	DRAWING NO.:	REVISION NO.:	
1 OF 2	102-2	1	MDADE



LUGT2



PRODUCT CODE	GA	FASTENERS SCHEDULE			ALLOWABLE LOADS 100%		
		RAFTER/ TRUSS	PLATE	CONCRETE	L1	L2	UPLIFT
LUGT2	14	(16) 10d	(2) 10d	(5) 1/4" DIA. POWER STUD	1220	460	1850

NOTES

1) L1 ARE LOADS APPLIED PARALLEL TO BEARING WALL

2) L2 ARE LOADS APPLIED PERPENDICULAR TO BEARING WALL.

3) ALLOWABLE LOADS FOR UPLIFT, L1 & L2 ARE NOT TO BE COMBINED.

...ISED
...te Florida
03-0219.02
06/20/05
...ida Product Control
Division

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653 STRUCTURAL GRADE 33, AND A MINIMUM GALVINIZED COATING OF G90.
2) FASTENERS ARE COMMON WIRE NAILS UNLESS OTHERWISE NOTED.
3) ALLOWABLE LOADS ARE BASED ON THE NATIONAL DESIGN SPECIFICATIONS FOR WOOD CONSTRUCTION 1997 EDITION FOR SOUTHERN YELLOW PINE (G= 0.55 OR BETTER AND MOISTURE CONTENT OF 19% OR LESS) ALL TESTS PERFORMED IN ACCORDANCE WITH ASTM D1761.

Robert W. Lutz
10 JUN 03

UNITED STEEL PRODUCTS COMPANY
703 ROGERS DRIVE, MONTGOMERY, MN. 56069 PHONE (507) 364-7333

NAME: LUGT2 (LIGHT UPLIFT GIRDER TIEDOWN)

DATE: 2/7/03

ROBERT W. LUTZ
PROFESSIONAL ENGINEER (STRUCTURAL)
FLORIDA REG. NO. 55409

SHEET: 2 OF 2

DRAWING NO: 102-02

MDADE

MIAMI-DADE

BUILDING CODE COMPLIANCE OFFICE (BCCO)
PRODUCT CONTROL DIVISION

MIAMI-DADE COUNTY, FLORIDA
METRO-DADE FLAGLER BUILDING
140 WEST FLAGLER STREET, SUITE 1603
MIAMI, FLORIDA 33130-1563
(305) 375-2901 FAX (305) 375-2908

NOTICE OF ACCEPTANCE (NOA)

United Steel Products Co.
703 Rogers Drive.
Montgomery, MN 56069

SCOPE:

This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed by Miami-Dade County Product Control Division and accepted by the Board of Rules and Appeals (BORA) to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Division (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. BCPRC reserves the right to revoke this acceptance, if it is determined by Miami-Dade county Product Control Division that this product or material fails to meet the requirements of the applicable building code.
This product is approved as described herein, and has been designed to comply with the High velocity Hurricane Zone of the Florida Building Code.

DESCRIPTION: Wood Connectors PAU & C Series.

APPROVAL DOCUMENT: Drawing No.100-02, titled "PAU SERIES (Post Anchors) & C SERIES (Post Caps)" sheet 1 & 2, with no revisions, dated 04/30/03, prepared by United Steel Products Company signed and sealed by R. W. Lutz, PE, bearing the Miami-Dade County Product Control Approval stamp with the Notice of Acceptance (NOA) number and approval date by the Miami-Dade County Product Control Division.

MISSILE IMPACT RATING: None

LABELING: Each unit shall bear a permanent label with the manufacturer's name or logo, city, state and following statement: "Miami-Dade County Product Control Approved", unless otherwise noted herein.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.
This NOA revises NOA #02-0128.03 and consists of this page 1, evidence page & approval document mentioned above.
The submitted documentation was reviewed by Candido F. Font PE.

9/25/03

PRODUCT CONTROL ACCEPTED

NOA No: 03-0508.05
Expiration Date: May 02, 2007
Approval Date: September 25, 2003

United Steel Products Company.

NOTICE OF ACCEPTANCE: EVIDENCE PAGE

A. DRAWINGS

1. Drawing No. 100-02, dated 04/30/03, sheets 1 & 2, with no revisions, prepared by United Steel Products Company, titled "PAU SERIES (Post Anchors) & C SERIES (Post Caps)" signed and sealed by R. W. Lutz, PE.

B. TESTS

Test reports on wood connectors per ASTM D1761, signed and sealed by C.R. Caudel, PE.

	Laboratory Report	Wood Connector	Direction	Date
1.	PTI # 01-3345	PAU 66	Up & Down	10/31/01.
2.	PTI # 01-3344	PAU 44	Up & Down	10/29/01
3.	PTI # 01-3348	C 66	Up & Perpdclr	10/08/01.
4.	PTI # 01-3347	C 46	Up & Perpdclr	10/05/01.
4.	PTI # 01-3346	C 44	Up & Perpdclr	10/02/01
5.	PTI # 02-3921	C 88	Parllel & Perpdclr	01/17/03
6.	PTI # 03-4301	C 88	Upward	04/15/03

C. CALCULATION

Calculated Capacities prepared by R. W. Lutz PE.

	Product Model	No. of Pages	Date	Signature
1.	C44, 46 & 66	1 through 5	12/20/01	R. W. Lutz, PE.
2.	PAU 44	1 through 5	12/21/01	R. W. Lutz, PE.
3.	C88	1 through 3	04/09/03	R. W. Lutz, PE

C. MATERIAL CERTIFICATIONS

N/A

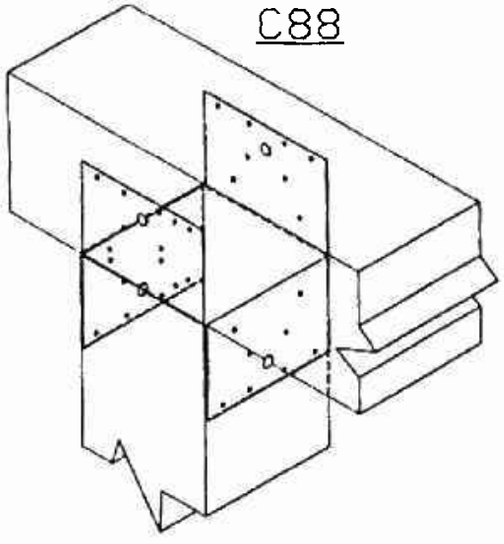
D. STATEMENTS

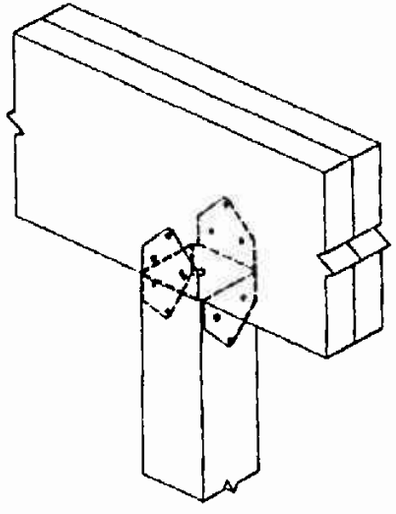
1. No Financial Interest letter issued by USP Lumber Connectors, dated 01/18/02 signed by R. W. Lutz, PE. and notarized by R. Radabaugh
2. Code Compliance letters issued by USP Lumber Connectors, dated 01/18/02 & 04/30/03 signed and sealed by R. W. Lutz, PE.
3. No Financial Interest letter issued by UPS Lumber Connectors, dated 04/30/03,
4. signed by R.W. Lutz, PE and notarized by G. P. Gunkel.

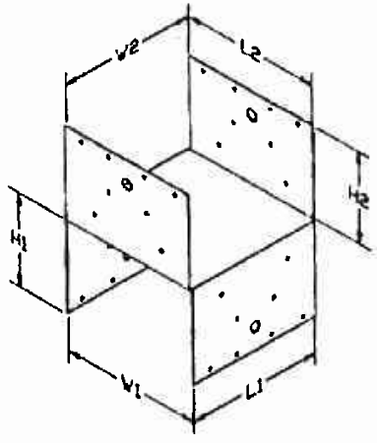
9/25/03

Candido F. Font PE.
Sr. Product Control Examiner
NOA No 03-0508.05
Expiration Date: May 02, 2007
Approval Date: September 25, 2003

C88

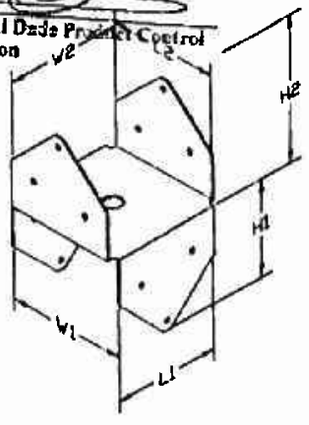






C88

PRODUCT REVISED
to comply with the Florida
Application No. 03-0508.05
Expiration Date: 03/02/07
By:
Miami Dade Product Control
Division



C POST CAP SERIES
18 GAUGE GALVANIZED (TYP)

PRODUCT CODE	DESCRIPTION	DIMENSIONS						FASTENER SCHEDULE		ALLOWABLE LOADS (lbs.) (133%)		
		W1	W2	H1	H2	L1	L2	POST	BEAM	F1	F2	UPLIFT
C44	4 x 4 POST CAP	3-9/16	3-9/16	2-7/8	2-7/8	3-1/4	3-1/4	(6) 16d	(6) 16d	860	860	820
C46	4 x 6 POST CAP	3-9/16	5-1/2	2-7/8	2-7/8	5-3/8	3-1/4	(10) 16d	(6) 16d	950	950	745
C66	6 x 6 POST CAP	5-1/2	5-1/2	2-7/8	2-7/8	5-1/4	5-1/4	(10) 16d	(10) 16d	1690	1690	870
C88	8 x 8 POST CAP	7 1/2	7 1/2	5	5	7 3/8	7 3/8	(16) 16d	(16) 16d	2390	3070	1125

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653 STRUCTURAL GRADE 33, AND A MINIMUM GALVINIZED COATING OF G90.
2) FASTENERS ARE COMMON WIRE NAILS UNLESS OTHERWISE NOTED.
3) ALLOWABLE UPLIFT LOADS HAVE BEEN INCREASED BY A SHORT TERM DURATION FACTOR OF 33% FOR WIND LOAD CONDITION. NO FURTHER INCREASE IS ALLOWED.
4) ALLOWABLE LOADS ARE BASED ON THE NATIONAL DESIGN SPECIFICATIONS FOR WOOD CONSTRUCTION 1997 EDITION FOR SOUTHERN YELLOW PINE (G= 0.55 OR BETTER & MOISTURE CONTENT OF 19% OR LESS. ALL TESTS PERFORMED IN ACCORDANCE WITH ASTM A1761.
5) PENETRATION ASSUMED TO BE 2" INTO WOOD.
6) F1 LOADS PARALLEL TO WALL.
7) F2 LOADS PERPENDICULAR TO WALL.
8) ALLOWABLE LOADS FOR UPLIFT, F1 & F2 ARE NOT TO BE COMBINED.

Robert W. Lutz
30 Apr 03

UNITED STEEL PRODUCTS COMPANY
703 ROGERS DRIVE, MONTGOMERY, MN. 56069 PHONE (507) 364-7333

NAME: C SERIES (POST CAPS)

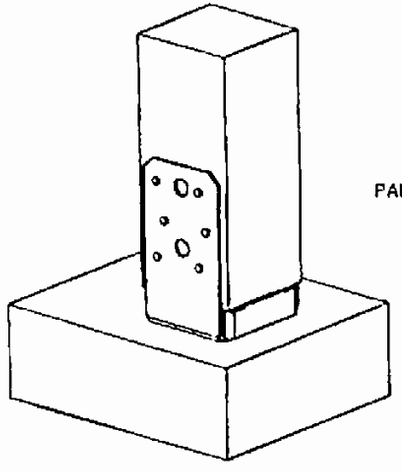
DATE: 4/30/03

ROBERT W. LUTZ
PROFESSIONAL ENGINEER (STRUCTURAL)
FLORIDA REG. NO. 55409

SHEET: 2 OF 2

DRAWING NO: 100-02

MDADE



PAU POST ANCHOR W/ BOLT SERIES

16 OR 12 GAUGE STAND-OFF
10 GAUGE SUPPORT COVER
10 GAUGE WASHER

PRODUCT REVISED
Florida
03-0505.05
Missouri Dade Product Control
Division

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653 STRUCTURAL GRADE 33, AND A MINIMUM GALVINIZED COATING OF G90.
2) FASTENERS ARE COMMON WIRE NAILS UNLESS OTHERWISE NOTED.
3) ALLOWABLE UPLIFT LOADS HAVE BEEN INCREASED BY A SHORT TERM DURATION FACTOR OF 33% FOR WIND LOAD CONDITION. NO FURTHER INCREASE IS ALLOWED
4) ALLOWABLE DOWN LOADS ARE NOT INCREASED BY SHORT TERM DURATION FACTOR.
5) ALLOWABLE LOADS ARE BASED ON THE NATIONAL DESIGN SPECIFICATIONS FOR WOOD CONSTRUCTION 1997 EDITION FOR SOUTHERN YELLOW PINE (G= 0.55 OR BETTER AND MOISTURE CONTENT OF 19% OR LESS). ALL TESTS PERFORMED IN ACCORDANCE WITH ASTM D1761.
6) PENETRATION IS ASSUMED TO BE 2" INTO WOOD.

PRODUCT CODE	POST/COLUMN SIZE	DIMENSIONS				FASTENER SCHEDULE				ALLOWABLE LOADS (Lbs.)	
							POST		BEARING	UPLIFT (133%)	
		L1	L2	H	L	ANCHOR BOLT	NAILS	BOLTS	100%	NAILS	BOLTS
PAU44	4 x 4	3-9/16	3	5-7/16	3	(1) 5/8	(12) 16d	(2) 1/2	6885	1825	1825
PAU66	6 x 6	5-9/16	5	2-7/8	2-7/8	(1) 5/8	(12) 16d	(2) 1/2	14,300	2425	2425

Robert W. Lutz
30 Apr 03

UNITED STEEL PRODUCTS COMPANY
703 ROGERS DRIVE, MONTGOMERY, MN. 56069 PHONE (507) 364-7333

NAME: PAU SERIES (POST ANCHOR)

DATE: 4/30/03

ROBERT W. LUTZ
PROFESSIONAL ENGINEER (STRUCTURAL)
FLORIDA REG. NO. 55409

SHEET: 1 OF 2

DRAWING NO: 100-02

MDADE

MIAMI-DADE

BUILDING CODE COMPLIANCE OFFICE (BCCO)
PRODUCT CONTROL DIVISION

MIAMI-DADE COUNTY, FLORIDA
METRO-DADE FLAGLER BUILDING
140 WEST FLAGLER STREET, SUITE 1603
MIAMI, FLORIDA 33130-1563
(305) 375-2901 FAX (305) 375-2908

NOTICE OF ACCEPTANCE (NOA)

United Steel Products Co.
703 Rogers Drive.
Montgomery, MN 56069

SCOPE:

This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed by Miami-Dade County Product Control Division and accepted by the Board of Rules and Appeals (BORA) to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Division (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. BCPRC reserves the right to revoke this acceptance, if it is determined by Miami-Dade county Product Control Division that this product or material fails to meet the requirements of the applicable building code.
This product is approved as described herein, and has been designed to comply with the High velocity Hurricane Zone of the Florida Building Code.

DESCRIPTION: Wood Connectors EHUH & UH Series.

APPROVAL DOCUMENT: Drawing No.**MD0603A**, titled "EHUH SERIES & UH SERIES" sheet 1 & 2, with no revisions, dated 06/09/03, prepared by United Steel Products Company signed and sealed by R. W. Lutz, PE, bearing the Miami-Dade County Product Control Approval stamp with the Notice of Acceptance (NOA) number and approval date by the Miami-Dade County Product Control Division.

MISSILE IMPACT RATING: None

LABELING: Each unit shall bear a permanent label with the manufacturer's name or logo, city, state and following statement: "Miami-Dade County Product Control Approved", unless otherwise noted herein.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.
This NOA revises NOA #00-0822.01 and consists of this page 1, evidence page & approval document mentioned above.
The submitted documentation was reviewed by Candido E. Font PE.

10/2/3

PRODUCT CONTROL ACCEPTED

NOA No: 03-0611.05
Expiration Date: August 21, 2008
Approval Date: October 2, 2003

United Steel Products Company.

NOTICE OF ACCEPTANCE: EVIDENCE PAGE

A DRAWINGS

1. Drawings prepared by United Steel Products Company, titled "EHUH SERIES & UH SERIES", Drawing No. MD0603A, sheets No. 1 & 2, dated 06/09/03 with no revisions, signed and sealed by R. W. Lutz, PE.

B TEST

Test reports on wood connectors per ASTM D1761 by Kleinfelder, signed and sealed by C. R. Caudel, PE.

	Report No.	Wood Connector	Direction	Date
1.	PT # 96-0904	UH 26	Down & Up	09/30/96
2	PT # 96-0905	UH 28	Down & Up	09/30/96
3	PT #96-0907	UH 210	Down & Up	10/02/96
2.	PT # 96-0901	ST 201	Down & Up	09/30/96
3.	PT # 96-0902	ST 201	Down & Up	11/05/96
4.	PT # 96-0903	ST2-202	Down & Up	11/06/96
5.	PT # 96-0908A	EHUH 26	Down & Up	11/06/96
5.	PT # 96-1009A	EHUH 28	Down & Up	01/29/97
6	PT # 96-1015	EHUH 210	Down & Up	03/04/97

C CALCULATIONS

Report of Design Capacities prepared for Hughes Manufacturing., Inc.

	Product Model	No. of Pages	Date	Signature
1.	EHUH Series	1 through 24	02/28/96	T. F. Devening, PE
2	UH Series	1 through 27	12/15/96	T. F. Devening, PE.
3	ST Series	1 through 15	12/15/96	T. F. Devening, PE

D STATEMENTS

1. No Change letter issued by United Steel Products Company. on 04/26/00 and signed by R. C. Brunson.
2. No interest letter issued by Thomas F. Devening Consulting Engineer, on 01/28/97, signed and sealed by T. F. Devening, PE.
3. Certificate of Merger between Hughes Manufacturing Inc. and United Steel Products Co. by The Secretary of the State of Minnesota, on 02/14/2000

10/2/3

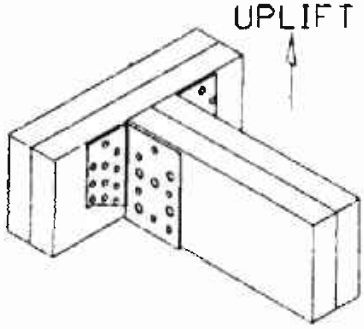
Candido F. Font PE.
Sr. Product Control Examiner
NOA No 03-0611.05
Expiration Date: August 21, 2008
Approval Date: October 2, 2003

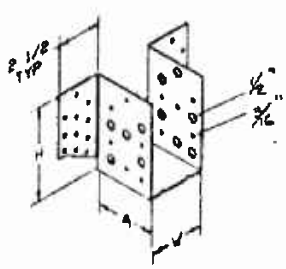
EXTRA HEAVY CONNECTOR 14 GAUGE

PRODUCT CODE	JOIST SIZE	DIMENSIONS W	H	A	FASTENER HEADER	JOIST	DESIGN LOADS DOWN	UPLIFT
EHUH26[2]	2X6	1 5/8	5 1/4	3 1/2	20-16d	10-10d X 1 1/2	2385	1200
EHUH26N[2]	2X6	1 5/8	5 1/4	3 1/2	20-HN20A	10-HN20A	3160	1730
EHUH28[2]	2X8	1 5/8	7 1/4	3 1/2	28-16d	14-10d X 1 1/2	3470	1845
EHUH28N[2]	2X8	1 5/8	7 1/4	3 1/2	28-HN20A	14-HN20A	4110	2515
EHUH210[2]	2X10	1 5/8	9 1/4	3 1/2	32-16d	16-10d X 1 1/2	3965	2110
EHUH210N[2]	2X10	1 5/8	9 1/4	3 1/2	32-HN20A	16-HN20A	4700	2875
EHUH26-2	(2) 2X6	3 1/8	5 3/8	3 1/2	22-16d	11-10d X 1 1/2	2875	1690
EHUH26-2N	(2) 2X6	3 1/8	5 3/8	3 1/2	22-HN20A	11-HN20A	3285	2195
EHUH28-2	(2) 2X8	3 1/8	7 3/8	3 1/2	32-16d	14-10d	4185	2155
EHUH28-2N	(2) 2X8	3 1/8	7 3/8	3 1/2	32-HN20A	14-HN20A	4775	2790
EHUH28-3	(2) 2X8	5 1/8	6 13/16	3 1/2	40-16d	16-10d	5190	2645
EHUH28-3N	(2) 2X8	5 1/8	6 13/16	3 1/2	40-HN20A	16-HN20A	5335	2850
EHUH210-2	(2) 2X10	3 1/8	9 3/8	3 1/2	40-16d	16-10d	5190	1645
EHUH210-2N	(2) 2X10	3 1/8	9 3/8	3 1/2	40-HN20A	16-HN20A	5335	2850
[illegible]H46	4X6	3 9/16	5 1/8	3 1/2	22-16d	11-10d	2875	1690
[illegible]H46N	4X6	3 9/16	5 1/8	3 1/2	22-HN20A	11-HN20A	3285	2195
[illegible]H48	4X8	3 9/16	7 1/8	3 1/2	32-16d	14-10d	4185	2155
EHUH48N	4X8	3 9/16	7 1/8	3 1/2	32-HN20A	14-HN20A	4775	2790
EHUH410	4X10	3 9/16	9 1/8	3 1/2	40-16d	16-10d	5190	2715
EHUH410N	4X10	3 9/16	9 1/8	3 1/2	40-HN20A	16-HN20A	5335	2850

NOTE 1.) PENETRATION OF FASTENER IN HEADER IS 2" FOR 16d NAIL & 1 3/4" FOR HN20A

NOTE 2.) PENETRATION OF FASTENER IN JOIST IS 1 1/2"





EHUH26-2 SHOWN
NTS

Approved as complying with the Florida Building Code
Date: 10/02/03
NOA: 03-0801.05
Miami-Dade Product Control Division
By:

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653 STRUCTURAL GRADE 33, G 0 N
2) FASTENERS ARE COMMON WIRE NAILS UNLESS OTHERWISE NOTED.
3) ALLOWABLE UPLIFT LOADS HAVE BEEN INCREASED BY A SHORT TERM DURATION FACTOR OF 33% FOR WIND LOAD CONDITIONS. NO FURTHER INCREASE IS ALLOWED.
4) ALLOWABLE DOWN LOADS ARE NOT INCREASED BY SHORT TERM DURATION FACTOR

Robert W. Lutz
9 JUN 03

UNITED STEEL PRODUCTS COMPANY
703 ROGERS DRIVE, MONTGOMERY, MN. 56069 PHONE (507) 364-7333

NAME: EHUH SERIES

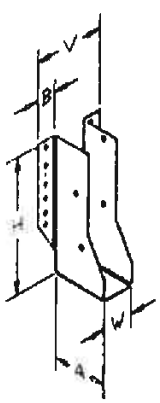
DATE: 6/9/03

ROBERT W. LUTZ
PROFESSIONAL ENGINEER (STRUCTURAL)
FLORIDA REG. NO 55409

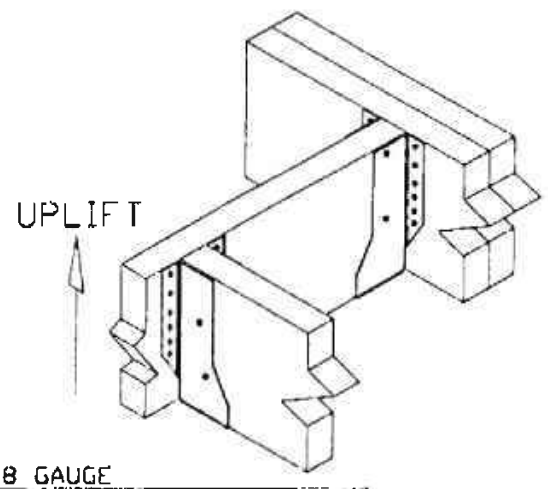
SHEET: 1 OF 2

DRAWING NO. MD0603A

MDADE



UH28 SHOWN



JOIST HANGER U-TYPE 18 GAUGE

PRODUCT CODE	JOIST SIZE	DIMENSIONS					FASTENERS		DESIGN LOAD			
		A	B	H	W	V	HEADER	JOIST	16d 100%	16d UPLIFT	20d 100%	20d UPLIFT
UH210	2X10	3	1	9 1/4	1 5/8	3 3/4	16	5	2320	900	2900	1440
UH24	2X4	3	1	3 1/4	1 5/8	3 3/4	4	2	580	----	700	----
UH26	2X6	3	1	5 1/4	1 5/8	3 3/4	8	3	1160	720	1400	785
UH28	2X8	3	1	7 1/4	1 5/8	3 3/4	12	4	1740	735	2100	1110
UH36	3X6	3	1	4 3/4	2 5/8	4 3/4	8	3	1160	----	1400	700
UH46	4X6	3	1	5 1/4	3 5/8	5 3/4	10	3	1450	----	1750	700

Approved as complying with the Florida Building Code
Date 10/2/03
NOA# 03-0611.05
Miami-Dade Product Control Division
By

NOTES: 1. NAILS ON JOIST SHALL BE LONG ENOUGH TO BE DRIVEN THROUGH BOTH PLATES & CLINCHED.
2 PENETRATION OF FASTENERS IN HEADER & JOIST SHALL BE 2" FOR 16d NAILS & 2 3/8" FOR 20d NAILS

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653 STRUCTURAL GRADE 33, AND A MINIMUM GALVANIZED COATING OF G60.
2) FASTENERS ARE COMMON WIRE NAILS UNLESS OTHERWISE NOTED
3) ALLOWABLE LOADS ARE BASED ON THE NATIONAL DESIGN SPECIFICATIONS FOR WOOD CONSTRUCTION 1997 EDITION FOR SOUTHERN YELLOW PINE (G= 0.55 OR BETTER AND MOISTURE CONTENT OF 19% OR LESS) ALL TESTS PERFORMED IN ACCORDANCE WITH ASTM D1761.
4) ALLOWABLE UPLIFT LOADS HAVE BEEN INCREASED BY A DURATION FACTOR OF 33% FOR WINDLOAD CONDITION NO FURTHER INCREASE IS ALLOWED.

Robert W. Lutz
9 Jun 03

UNITED STEEL PRODUCTS COMPANY
703 ROGERS DRIVE, MONTGOMERY, KY. 56069 PHONE (507) 364-7333

NAME: UH SERIES

DATE: 6/9/03

ROBERT W. LUTZ
PROFESSIONAL ENGINEER (STRUCTURAL)
FLORIDA REG. NO. 55409

SHEET: 2 OF 2

DRAWING NO. MD0603A

MDADE

MIAMI-DADE

BUILDING CODE COMPLIANCE OFFICE (BCCO)
PRODUCT CONTROL DIVISION

MIAMI-DADE COUNTY, FLORIDA
METRO-DADE FLAGLER BUILDING
140 WEST FLAGLER STREET, SUITE 1603
MIAMI, FLORIDA 33130-1563
(305) 375-2901 FAX (305) 375-2908

NOTICE OF ACCEPTANCE (NOA)

United Steel Products Company.
703 Rogers Drive.
Montgomery, MN 56069

SCOPE:

This NOA is being issued under the applicable rules and regulations governing the use of construction materials. The documentation submitted has been reviewed by Miami-Dade County Product Control Division and accepted by the Board of Rules and Appeals (BORA) to be used in Miami Dade County and other areas where allowed by the Authority Having Jurisdiction (AHJ).

This NOA shall not be valid after the expiration date stated below. The Miami-Dade County Product Control Division (In Miami Dade County) and/or the AHJ (in areas other than Miami Dade County) reserve the right to have this product or material tested for quality assurance purposes. If this product or material fails to perform in the accepted manner, the manufacturer will incur the expense of such testing and the AHJ may immediately revoke, modify, or suspend the use of such product or material within their jurisdiction. BCPRC reserves the right to revoke this acceptance, if it is determined by Miami-Dade county Product Control Division that this product or material fails to meet the requirements of the applicable building code.
This product is approved as described herein, and has been designed to comply with the Florida Building Code including the High velocity Hurricane Zone.

DESCRIPTION: GTU Series and HJH Series. Wood Connectors.

APPROVAL DOCUMENT: Drawing No. MD0404, titled "HJH Series (Hip & Jack Heavy Connectors) and GTU Series (Girder Hanger Heavy)"sheet 1 and 2, dated 03/10/05 and 01/05/05 (Revision #1) , prepared by United Steel Products Company signed and sealed by R. W. Lutz, PE, bearing the Miami-Dade County Product Control Revision stamp with the Notice of Acceptance (NOA) number and Expiration date by the Miami-Dade County Product Control Division.

MISSILE IMPACT RATING: None

LABELING: Each unit shall bear a permanent label with the manufacturer's name or logo, city, state and following statement: "Miami-Dade County Product Control Approved", unless otherwise noted herein.

RENEWAL of this NOA shall be considered after a renewal application has been filed and there has been no change in the applicable building code negatively affecting the performance of this product.

TERMINATION of this NOA will occur after the expiration date or if there has been a revision or change in the materials, use, and/or manufacture of the product or process. Misuse of this NOA as an endorsement of any product, for sales, advertising or any other purposes shall automatically terminate this NOA. Failure to comply with any section of this NOA shall be cause for termination and removal of NOA.

ADVERTISEMENT: The NOA number preceded by the words Miami-Dade County, Florida, and followed by the expiration date may be displayed in advertising literature. If any portion of the NOA is displayed, then it shall be done in its entirety.

INSPECTION: A copy of this entire NOA shall be provided to the user by the manufacturer or its distributors and shall be available for inspection at the job site at the request of the Building Official.
This NOA revises NOA # 04-0427.04 and consists of this page, evidence page & approval document mentioned above.
The submitted documentation was reviewed by Candido F. Font PE.

PRODUCT CONTROL ACCEPTED

04/01/05

NOA No: 05-0204.01
Expiration Date: December 2, 2006
Approval Date: April 7, 2005

United Steel Products Company.

NOTICE OF ACCEPTANCE: EVIDENCE PAGE

A DRAWINGS

1. Drawings prepared by United Steel Products Company, titled "HJH Series (Hip & Jack Heavy Connectors) and GTU Series (Girder Hanger Heavy)", Drawing No. MDO0404, sheets No. 1 and 2, dated 03/10/05 and 01/05/05 (Revision # 1), signed and sealed by R. W. Lutz, PE.

B TEST

Test reports on wood connectors per ASTM D1761 by Product Testing Inc, signed and sealed by C. R. Caudel, PE.

	Report No.	Wood Connector	Direction	Date
1.	PT # 98-971447	HJH 26L/R	Up & Down	10/29/98
2.	PT # 98-971448	HJH 28L/R	Up & Down	10/31/98
3.	PT # 98-971456	HSU 8016	Up & Down	07/15/98
4.	PT # 98-971457	HSU 4016	Up & Down	07/05/98

C CALCULATIONS

1. Report of Design Capacities of HJH prepared by Thomas F. Devening on November 1997, sheet 1 through 11, signed and sealed by T. F. Devening, PE.
2. Report of Design Capacities of HSU prepared by Thomas F. Devening on November 1998 sheet 1 through 10 signed and sealed by T. F. Devening, PE.

D QUALITY ASSURANCE

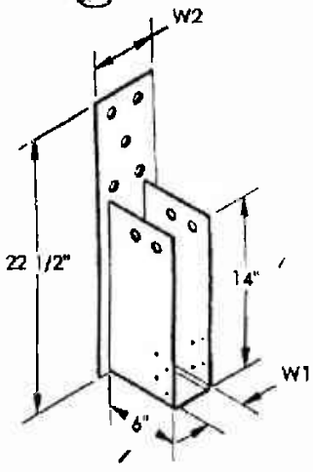
1. Product Control Division

E STATEMENTS

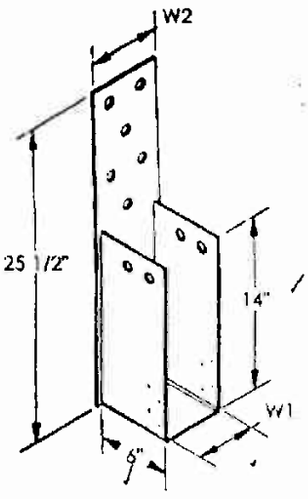
1. Minor Change letter issue by USP Lumber Connector on 04/26/00 and signed by R. C. Brunson.
2. Merger documents issued by the State of Minnesota on 02/14/00 signed by the Secretary of State M. Kiffmeyer.
3. No financial interest letter issued by The Dore' Group on 11/20/98, signed and sealed by T. F. Devening PE.
4. Name Change only letter issued by USP Structural Connectors on 03/10/05 signed and sealed by R. W. Lutz.

04/07/05

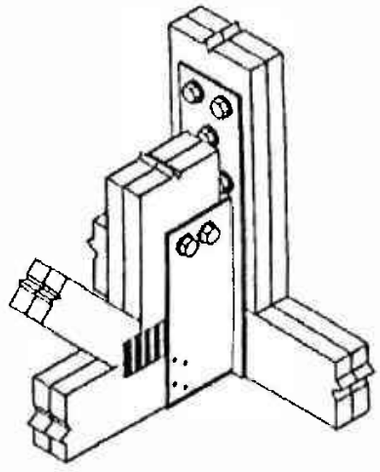
Candido F. Font PE.
Sr. Product Control Examiner
NOA No 05-0204.01
Expiration Date: December 2, 2006
Approval Date: April 7, 2005



GTU40



GTU80



PRODUCT CODE	DESCRIPTION	DIMENSIONS		FASTENERS			DESIGN LOADS (lbs.)	
				BOLTS		NAILS	NORMAL	UPLIFT
		W_t	H	GIRDER	BLOCK	CARRIED MEMBER	100%	133%
GTU40	GIRDER HGR (HVY.)	3 1/4"	23"	(5) 3/4"	(2) 3/4"	(8) 10d	7040	5185
GTU80	GIRDER HGR (HVY.)	4 7/8"	23 1/2"	(6) 3/4"	(2) 3/4"	(8) 10d	8445	7950

NOTES: 1. MATERIAL: 7 GA. (3/16") ASTM A36.
2. ALL 3/4: BOLTS REQUIRE 3: PENETRATION AND MINIMUM WOOD VERTICAL WIDTH OF 5 1/2".
3. BOLTS ASTM A307 GRADE A.
4. NAILS ON CARRIED MEMBER SHALL HAVE A MINIMUM PENETRATION OF 1 3/4"

PRODUCT REVISED
as complying with the Florida Building Code
Acceptance No. 05/0204.01
Expiration Date 04/07/05
By
Miami Dade Product Control Division

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653 STRUCTURALGRADE 33, AND A MINIMUM GALVANIZED COAT OF G90.
2) FASTENERS ARE COMMON WIRE NAILS UNLESSOTHERWISE NOTED.
3) ALLOWABLE LOADS HAVE BEEN INCREASED BY A SHORT TERM DURATION FACTOR OF 33% FOR WINDLOAD CONDITIONS. NO FURTHER INCREASE IS ALLOWED.
4) ALLOWABLE LOADS ARE BASED ON THE NATIONALDESIGN SPECIFICATIONS FOR WOOD CONSTRUCTION 1997 EDITION FOR SOUTHERN YELLOW PINE (G= 0.55 OR BETTER. ALL TESTS PERFORMED IN ACCORDANCE WITH ASTM D1761.
5) MINIMUM 2000 PSI GROUT.
6) PENETRATION IS ASSUMED TO BE 1 1/2" INTO WOOD. BETWEEN RAFTER & STUD.

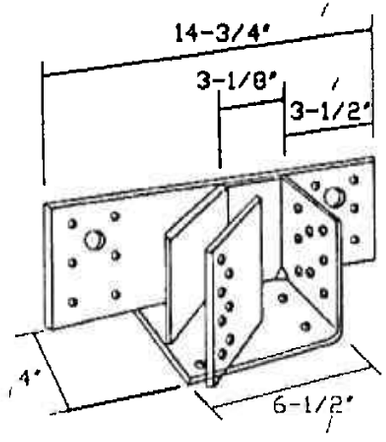
Robert W Lutz
2 Feb 05

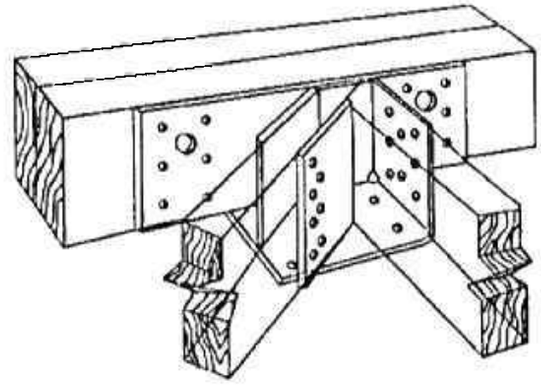
REV. #1

UNITED STEEL PRODUCTS COMPANY
703 ROGERS DRIVE, MONTGOMERY, MN. 56069 PHONE (507) 364-7333

NAME: GTU SERIES (GIRDER HANGER HEAVY)

DATE		
1-5-05	ROBERT W LUTZ PROFESSIONAL ENGINEER (STRUCTURAL) FLORIDA REG. NO. 55409	
SHEET: 2 OF 2	DRAWING NO.# MD0404	MDADE





HIP & JACK CONNECTORS (HEAVY)

Product Code	Description	Dimensions	Fasteners				Both Combined Design Loads (lbs.)	
			Header Nails	Header Bolts	Hip Truss	Jack Truss	Normal 100%	Uplift 133%
		H						
HJH 26L/R	Hip/Jack Hanger 2X6 Right or Left	5-1/4"	12-16d	None	7-16d	8-16d	1920	1890
HJH 28L/R	Hip/Jack Hanger 2X6 Right or Left	7-1/4"	16-16d	None	7-16d	8-16d	2560	2185

Note: 1. Material: 7 gauge (3/16") ASTM A36.
2. Nail on the header shall have a minimum penetration of 2"

PRODUCT REVISED
as complying with the Florida Building Code
Acceptance No. 05-0204.01
Expiration Date 04/07/05
By
Miami Dade Product Control Division

GENERAL NOTES

1) STEEL SHALL CONFORM TO ASTM A653 STRUCTURAL GRADE 33, AND A MINIMUM GALVINIZED COATING OF G90.
2) FASTENERS ARE COMMON WIRE NAILS UNLESS OTHERWISE NOTED.
3) ALLOWABLE LOADS HAVE BEEN INCREASED BY A SHORT TERM DURATION FACTOR OF 33% FOR WIND LOAD CONDITION. NO FURTHER INCREASE IS ALLOWED.
4) ALLOWABLE LOADS ARE BASED ON THE NATIONAL DESIGN SPECIFICATIONS FOR WOOD CONSTRUCTION 1997 EDITION FOR SOUTHERN YELLOW PINE (G= 0.55 OR BETTER). LL TEST PERFORMED IN ACCORDANCE WITH ASTM D1761.
5) MINIMUM 2000 PSI GROUT.
6) PENETRATION IS ASSUMED TO BE 1 1/2" INTO WOOD. BETWEEN RAFTER & STUD.

Robert W. Lutz
10 Mar 05

UNITED STEEL PRODUCTS COMPANY
703 ROGERS DRIVE, MONTGOMERY, MN. 56069 PHONE (507) 364-7333

NAME: HJH SERIES (HIP & JACK HEAVY CONNECTORS)

DATE: 3/10/05

ROBERT W. LUTZ
PROFESSIONAL ENGINEER (STRUCTURAL)
FLORIDA REG. NO. 55409

SHEET: 1 OF 2

DWG. #: MD0404

MDADE

SIDDIQ KHAN & ASSOCIATES, INC
CONSULTING ENGINEERS AND PLANNERS
7400 S.W. 50TH TERRACE, SUITE 105
MIAMI, FLORIDA 33155

(305) 662-2301
FAX: (305) 661-3962
www.ska-engineering.com

Gainor Residence
5800 North Bay Road
City of Miami Beach, Florida

Copy of Geotechnical Engineer's Report

GEOTECHNICAL
ENVIRONMENTAL
HYDROGEOLOGY
ASBESTOS

TESTING LABORATORIES
DRILLING SERVICES
INSPECTION SERVICES
ROOFING

DYNATECH ENGINEERING CORP.

Miami, April 5, 2005

Mr. Thomas F. Weber
THOMAS WEBER INC.
KEY EXECUTIVE BUILDING
104th Crandon Boulevard
Key Biscayne, FL 33149

Re: Gainor Residence @
5800 North Bay Road
Miami Beach, FL

Dear Mr. Weber:

Pursuant to your request; DYNATECH ENGINEERING CORP., D.E.C. completed a Preliminary Subsoil Investigation on April 5, 2005 at the above referenced project. The purpose of our investigation was to verify subsoil conditions relative to foundation design of the proposed additions.

A total of (2) standard penetration boring tests were performed according to ASTM-D 1586 down to an average depth of 35' below existing ground surface.

The following graph was developed as a general condition for the subject site: (Refer to field boring logs for exact locations and soil description):

Depth From	To	Description
0'- 0"	1'- 0"	Topsoil and grass
1'- 0"	4'- 0"	Silty beach sand
4'-0"	5'-0"	Peat
5'-0"	15'-0"	Gray silty beach sand
15'-0"	18'-0"	Tan medium sand with rock fragment
18'-0"	35'-0"	Tan sandy lime rock

Groundwater table elevation was measured immediately at the completion of each boring and was found at an average depth of 5' below existing ground surface. Fluctuation in water level should be anticipated due to seasonal variations and run off as well as varying ground elevations construction dewatering pumping activities in the area.

750 West 84 Street Hialeah FL 33014-3618 • Phone (305) 828-7499 • Fax (305) 828-9598

Based on our understanding of the proposed structure and our field boring logs; it is evident that the deep foundation system are needed to support the proposed addition without detrimental settlement to the structure.

Deep foundation systems shall consist of one of the following alternatives:

Alternatives Pile Foundation	Approximate Pile Depth	Size	Pile Capacity in Tons Compression	Pile Capacity in Tons Tension	Allowable Lateral Capacity in Tons
Pin Piles	To Refusal	3 Inch	5 Tons	2 Tons	1 Ton
Pin Piles	To Refusal	4 Inch	8 Tons	3 Tons	1 Ton
Type A of B Helical Piles	25' BLS	4 Inch	10 Tons	2 Tons	1 Ton
Auger Cast Piles	25' BLS	12 Inch	25 Tons	7 Tons	2 Tons
Auger Cast Piles	25' BLS	14 Inch	35 Tons	10 Tons	4 Tons
Precast Concrete Piles	25' BLS	10"x 10"	17 Tons	5 Tons	1 Tons
Precast Concrete Piles	25' BLS	12"x 12"	25 Tons	7 Tons	2 Tons
Precast Concrete Piles	25' BLS	14"x 14"	35 Tons	10 Tons	4 Tons

BLS: Below Existing Land Surface

Estimated Lateral Load for a pile Top Deflection of ¼ inch. The proposed pile length is based on the existing ground elevation at the time of drilling. Pile length may vary depending on proposed grade beam elevation and soil profile.

In the case of the Pin, Helical, or Precast piles a minimum of 4 piles shall be driven to determine production pile length. All work shall be performed in accordance with the applicable building code.

In case of existing structures in the vicinity of the pile driving operation, care shall be taken not to create excessive vibration. Vibration levels shall be monitored to verify compliance with county regulations. Step must be taken to prevent excessive vibrations. The minimum center to center of piles or adjacent foundation shall be not less than twice the average diameter for round piles or 1 – ¾ times the diagonal dimensions o rectangular piles, but in no case less than 30 inches.

Regardless of the thoroughness of a geotechnical exploration there is always the possibility that conditions may be different from those of the test locations; therefore, DYNATECH ENGINEERING CORP., does not guarantee any subsoil conditions between the bore test holes. The data from the soil boring is for foundation analysis only. It is not to be used for excavating or back filling estimates. As a mutual protection to clients, the public and ourselves, all reports are submitted as the confidential property of clients, and authorization for publication of statements, conclusions or extracts from or regarding our reports is reserved pending our written approval. The discovery of any site or subsurface conditions during construction which deviate from the information obtained from our subsoil investigation is always likely and should be reported to us for our evaluation.

It has been a pleasure working with you and look forward to do so in the near future. Please feel free to contact us if we may be of further service to you.

Sincerely yours,

Wissam Naamani, P.E.
DYNATECH ENGINEERING CORP.
Florida Reg. No. 39584
Special Inspector No. 757
WN/sk

DYNATECH ENGINEERING CORP.

750 WEST 84TH STREET
HIALEAH, FL 33014
(305) 828-7499

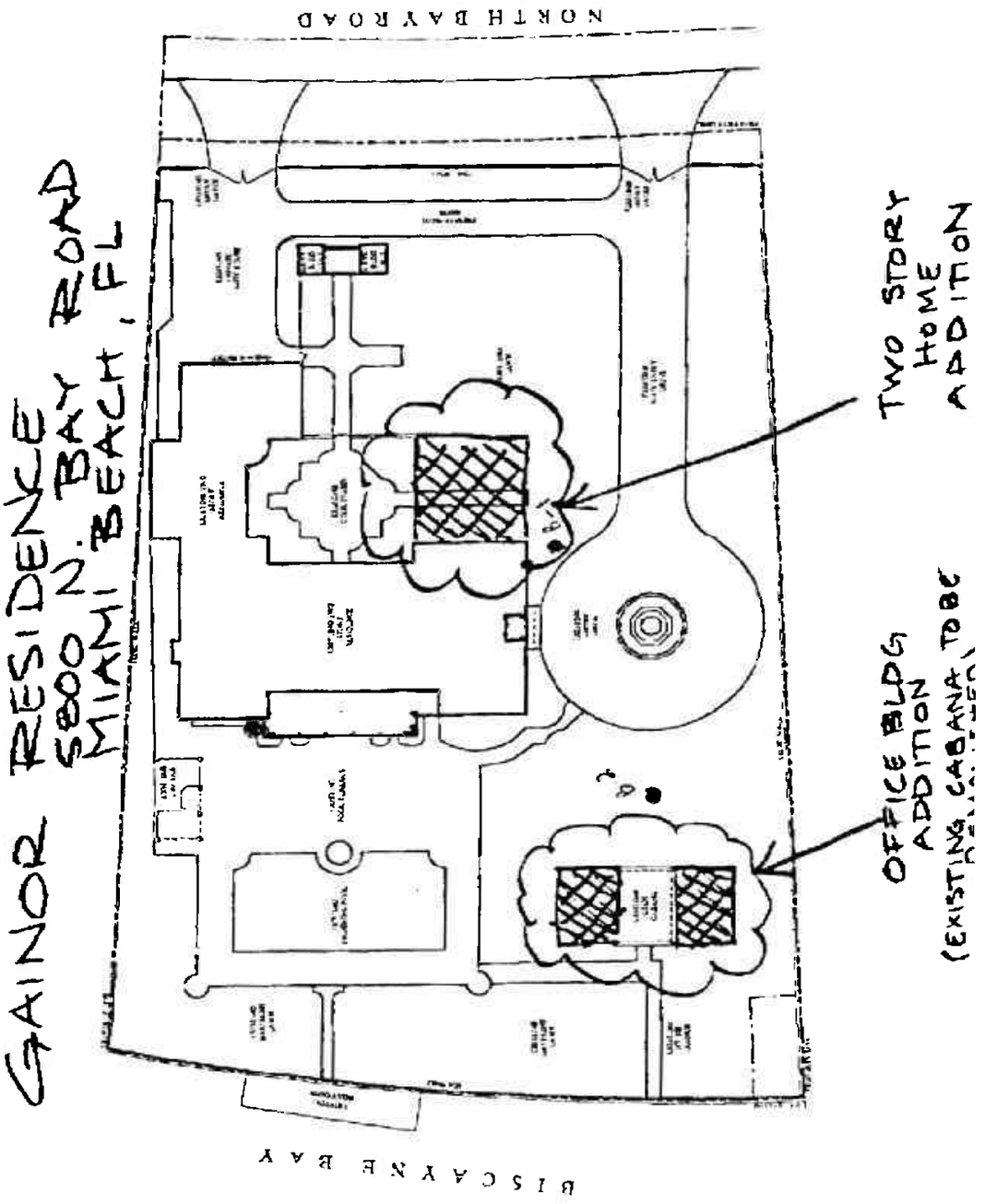
TEST BORING REPORT

CLIENT : Thomas Weber, Inc.
PROJECT : Gainor Residence @
ADDRESS : 5800 North Bay Road, Miami Beach, FL.
LOCATION : See attached plan

DATE: 04-04-05
HOLE NO.: B-2
DRILLER: ASLD

DEPTH	DESCRIPTION OF MATERIALS	SAMPLE NO.	HAMMER BLOWS ON SAMPLER	"N"
1		2	Hand	H
2				
3	+0'- 0" to 1'-0" TOPSOIL AND ROOTS AND GRASS	4	Hand	H
4				
5		6	1 2 / 2 1	4
6	1'-0" to 2'- 0" TAN BEACH SAND			
7		8	1 1 / 1 1	2
8	2'-0" to 4'-0" GRAY SILTY BEACH SAND			
9		10	2 2 / 1 1	3
10	4'-0" to 5'-0" PEAT			
11		12	2 2 / 1 1	3
12				
13	5'-0" to 15'-0" GRAY SILTY BEACH SAND	14	1 1 / 1 2	2
14				
15		16	A	A
16	15'-0" to 18'-0" TAN MEDIUM SAND WITH ROCK FRAGMENTS			
17		18	A	A
18	18'-0" to 35'-0" TAN SANDY LIME ROCK			
19		20	A	A
20				
21		22	20 23 / 24 23	47
22				
23		24	A	A
24				
25		26	A	A
26				
27		28	A	A
28				
29		30	24 25 / 27 24	52
30				
31		32	A	A
32				
33		34	A	A
34				
35		36	A	A
36				
37		38		
38				

Water Level: 5' Below Surface


NORTH BAY ROAD
BISCAYNE BAY
GAINOR RESIDENCE
5800 N. BAY ROAD
MIAMI BEACH, FL
TWO STORY HOME ADDITION
OFFICE BLDG ADDITION
(EXISTING CABANA TO BE