



J.C.D. ARCHITECT , Inc.

JUAN C. DAVID R. A. AR-00015344

LEED ACCREDITED PROFESSIONAL
Design & Development

Architecture Interiors Planning Construction

1385 Coral Way, Suite 407 Miami, Florida 33145 Phone: (305) 285-4343 Fax: (305) 285-4330

PROPOSED NEW 50 APARTMENTS COMPLEX

3400 NW 191st Street
Miami Gardens, FL 33056

Owner: The Village at Miami Gardens

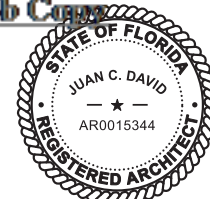
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STRUCTURAL CALCULATIONS 08202019.pdf



Digitally signed
by Juan C David
DN: c=US, o=JCD
Architect Inc,
ou=A01427E000
001677FD7566C
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cn=Juan C David
Date: 2019.08.22
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**J.C.D. ARCHITECT, Inc.**

JUAN C. DAVID R. A. AR-00013344
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Architecture Interiors Planning Construction
 1880 Coral Way, Suite 407 • Miami, Florida 33145 • Phone: (305) 285-4343 • Fax: (305) 285-4380

JOB MIAMI GARDENS TOWNHOUSES

ADDRESS _____

PROJECT No _____ SHEET No _____ OF _____

CALCULATED BY _____ DATE _____

CHECKED BY _____ DATE _____

+ DESIGN LOADS**ROOF**

ROOFING	
MECHANICAL EQUIP.	
CEILING & DUCTS	
AERIAL PRECAST SYSTEM	

PSF

DL

SUP.

PSF

LL

15

25

30

10

42

67

2ND FLOOR

FLOORING	
PARTITIONS	
CEILING & DUCTS	
AERIAL PRECAST SYSTEM	

16

11

10

42

79

37

40

1ST FLOOR

N/A

FLOORING	
PARTITIONS	
CONC. SLAB	

N/A

N/A

CONSTRUCTION LOAD

7" CONC. SLAB	
SHORING	
10" CONC. SLAB	

87

6

125

218

1ST FL2ND FL

30

BALCONY & TERRACE - SUPERIMPOSED	
STAIR - SUPERIMPOSED	

27

15

60

80

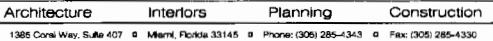
RAILING

50 PLF
 200 LB
 50 LB
 @ 1 SF

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E-19.83
P-22'



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JOB: MIAMI GARDENS TOWN HOUSES

ADDRESS _____

PROJECT No _____ SHEET No _____ OF _____

CALCULATED BY _____ DATE _____

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+ LOADS

DL

	INT. WF-24	EXT. WF-36	1ST FL. C.M.U.	2ND FL. C.M.U.
ROOF SYS. $\rightarrow (67 \frac{16}{16} \times 2) \times (\frac{24}{2} + \frac{23}{2}) \mid (\frac{24}{2})$	1,608	804	804	804
TB OR BM $\rightarrow (150 \frac{16}{16} \times 3) \times (0.67') \times (1')$	101	101	200	200
2ND F.W. $\rightarrow (55 \frac{16}{16} \times 2) \times (11') \mid (10')$	550	605	605	
2ND F.S. $\rightarrow (79 \frac{16}{16} \times 2) \times (\frac{24}{2} + \frac{23}{2}) \mid (\frac{24}{2})$	1,896	948	948	
	4,155	2,458	2,557	1,004

TB OR BM $\rightarrow (150 \frac{16}{16} \times 3) \times (0.67') \times (1')$

1ST F.W. $\rightarrow (55 \frac{16}{16} \times 2) \times (10')$

SOIL 5' W/F $\rightarrow (125 \frac{16}{16} \times 3) \times (2') \times (2') \mid (2.5')$

WDL

651	651	101	
4,806	3,109	2,658	1,004

LL

	WLL	WTL		
ROOF $\rightarrow (30 \frac{16}{16} \times 2) \times (\frac{24}{2} + \frac{23}{2}) \mid (\frac{24}{2})$	720	360	360	360
2ND F. $\rightarrow (40 \frac{16}{16} \times 2) \times (\frac{24}{2} + \frac{23}{2}) \mid (\frac{24}{2})$	960	480	480	
	1,680	840	840	360
	1,680	840	840	360

WL

PSF

PSF

PSF

AVERAGE

PSF

ZONE 4 = () $\times (0.6)$ =

ZONE 5 = () $\times (0.6)$ =

ZONE 1 = () $\times (0.6)$ =

ZONE 2 & 3 = () $\times (0.6)$ =

ZONE 1H = () $\times (0.6)$ =

ZONE 2H = () $\times (0.6)$ =

ZONE 3H = () $\times (0.6)$ =

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General Footing Analysis & Design

Page 1
miami_gardens.ecw:Footings

Description WF-24 (qs'=2.5 ksf)(2 Story-Exterior Wall)Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Allowable Soil Bearing	2,500.0 psf	Dimensions...	
Short Term Increase	1.330	Width along X-X Axis	2.000 ft
Seismic Zone	0	Length along Y-Y Axis	1.000 ft
Biaxial Applied Loads		Footing Thickness	12.00 in
Live & Short Term Combined		Col Dim. Along X-X Axis	8.00 in
f'c	3,000.0 psi	Col Dim. Along Y-Y Axis	12.00 in
Fy	60,000.0 psi	Base Pedestal Height	12.000 in
Concrete Weight	150.00 pcf	Min Steel %	0.0014
Overburden Weight	100.00 psf	Rebar Center To Edge Distance	3.50 in

Loads

Applied Vertical Load...

Dead Load	3.109 k	...ecc along X-X Axis	0.000 in
Live Load	0.840 k	...ecc along Y-Y Axis	0.000 in
Short Term Load	k		

Applied Moments...

	<u>Creates Rotation about Y-Y Axis</u> (pressures @ left & right)	<u>Creates Rotation about X-X Axis</u> (pressures @ top & bot)
Dead Load	k-ft	k-ft
Live Load	k-ft	k-ft
Short Term	k-ft	k-ft

Applied Shears...

	<u>Creates Rotation about Y-Y Axis</u> (pressures @ left & right)	<u>Creates Rotation about X-X Axis</u> (pressures @ top & bot)
Dead Load	k	k
Live Load	k	k
Short Term	k	k

Summary

Footing Design OK

2.00ft x 1.00ft Footing, 12.0in Thick, w/ Column Support 8.00 x 12.00in x 12.0in high
2.00ft x 1.00ft Footing, 12.0in Thick, w/ Column Suppo

	<u>DL+LL</u>	<u>DL+LL+ST</u>		<u>Actual</u>	<u>Allowable</u>
Max Soil Pressure	2,241.2	2,241.2 psf	Max Mu	0.647 k-ft per ft	
Allowable	2,500.0	3,325.0 psf	Required Steel Area		0.259 in2 per ft
"X" Ecc, of Resultant	0.000 in	0.000 in	Shear Stresses....	<u>Vu</u>	<u>Vn * Phi</u>
"Y" Ecc, of Resultant	0.000 in	0.000 in	1-Way	0.000	93.113 psi
X-X Min. Stability Ratio	No Overturning	1.500 :1	2-Way	2.260	186.226 psi
Y-Y Min. Stability Ratio	No Overturning				

Footing Design

Shear Forces	<u>ACI C-1</u>	<u>ACI C-2</u>	<u>ACI C-3</u>	<u>Vn * Phi</u>	
Two-Way Shear	0.00 psi	2.26 psi	1.45 psi	186.23 psi	
One-Way Shears...					
Vu @ Left	0.00 psi	0.00 psi	0.00 psi	93.11 psi	
Vu @ Right	0.00 psi	0.00 psi	0.00 psi	93.11 psi	
Vu @ Top	0.00 psi	0.00 psi	0.00 psi	93.11 psi	
Vu @ Bottom	0.00 psi	0.00 psi	0.00 psi	93.11 psi	
Moments	<u>ACI C-1</u>	<u>ACI C-2</u>	<u>ACI C-3</u>	<u>Ru / Phi</u>	<u>As Req'd</u>
Mu @ Left	0.65 k-ft	0.47 k-ft	0.30 k-ft	10.0 psi	0.26 in2 per ft
Mu @ Right	0.65 k-ft	0.47 k-ft	0.30 k-ft	10.0 psi	0.26 in2 per ft
Mu @ Top	0.00 k-ft	0.00 k-ft	0.00 k-ft	0.0 psi	0.02 in2 per ft
Mu @ Bottom	0.00 k-ft	0.00 k-ft	0.00 k-ft	0.0 psi	0.02 in2 per ft

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General Footing Analysis & Design

Page 2
miami_gardens.ecw:Footings

Description WF-24 (qs'=2.5 ksf)(2 Story-Exterior Wall)Miami Gardens

Soil Pressure Summary

Service Load Soil Pressures	Bottom-	Top-Rig	Bottom-	Top-Lef
DL + LL	2,241.17	2,241.17	2,241.17	2,241.17 psf
DL + LL + ST	2,241.17	2,241.17	2,241.17	2,241.17 psf
Factored Load Soil Pressures				
ACI Eq. C-1	3,263.63	3,263.63	3,263.63	3,263.63 psf
ACI Eq. C-2	2,395.22	2,395.22	2,395.22	2,395.22 psf
ACI Eq. C-3	1,594.05	1,594.05	1,594.05	1,594.05 psf

ACI Factors (per ACI 318-02, applied internally to entered loads)

ACI C-1 & C-2 DL	1.400	ACI C-2 Group Factor	0.750	Add'l "1.4" Factor for Seismic	1.400
ACI C-1 & C-2 LL	1.700	ACI C-3 Dead Load Factor	0.900	Add'l "0.9" Factor for Seismic	0.900
ACI C-1 & C-2 ST	1.700	ACI C-3 Short Term Factor	1.300		
....seismic = ST * :	1.100	Used in ACI C-2 & C-3			

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General Footing Analysis & Design

Page 1
miami_gardens.ecw:Footings

Description WF-30 (qs'=2.5 ksf)(2 Story-Exterior Wall)Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Allowable Soil Bearing	2,500.0 psf	Dimensions...	
Short Term Increase	1.330	Width along X-X Axis	2.500 ft
Seismic Zone	0	Length along Y-Y Axis	1.000 ft
Biaxial Applied Loads		Footing Thickness	12.00 in
Live & Short Term Combined		Col Dim. Along X-X Axis	8.00 in
f'c	3,000.0 psi	Col Dim. Along Y-Y Axis	12.00 in
Fy	60,000.0 psi	Base Pedestal Height	12.000 in
Concrete Weight	150.00 pcf	Min Steel %	0.0014
Overburden Weight	100.00 psf	Rebar Center To Edge Distance	3.50 in

Loads

Applied Vertical Load...

Dead Load	3.109 k	...ecc along X-X Axis	0.000 in
Live Load	0.840 k	...ecc along Y-Y Axis	0.000 in
Short Term Load	k		

Applied Moments...

	<u>Creates Rotation about Y-Y Axis</u> (pressures @ left & right)	<u>Creates Rotation about X-X Axis</u> (pressures @ top & bot)
Dead Load	k-ft	k-ft
Live Load	k-ft	k-ft
Short Term	k-ft	k-ft

Applied Shears...

	<u>Creates Rotation about Y-Y Axis</u> (pressures @ left & right)	<u>Creates Rotation about X-X Axis</u> (pressures @ top & bot)
Dead Load	k	k
Live Load	k	k
Short Term	k	k

Summary

Footing Design OK

2.50ft x 1.00ft Footing, 12.0in Thick, w/ Column Support 8.00 x 12.00in x 12.0in high
2.50ft x 1.00ft Footing, 12.0in Thick, w/ Column Suppo

	<u>DL+LL</u>	<u>DL+LL+ST</u>		<u>Actual</u>	<u>Allowable</u>
Max Soil Pressure	1,842.9	1,842.9 psf	Max Mu	0.979 k-ft per ft	
Allowable	2,500.0	3,325.0 psf	Required Steel Area		0.259 in2 per ft
"X" Ecc, of Resultant	0.000 in	0.000 in	Shear Stresses....	<u>Vu</u>	<u>Vn * Phi</u>
"Y" Ecc, of Resultant	0.000 in	0.000 in	1-Way	5.543	93.113 psi
X-X Min. Stability Ratio	No Overturning	1.500 :1	2-Way	2.906	186.226 psi
Y-Y Min. Stability Ratio	No Overturning				

Footing Design

Shear Forces	<u>ACI C-1</u>	<u>ACI C-2</u>	<u>ACI C-3</u>	<u>Vn * Phi</u>	
Two-Way Shear	0.00 psi	2.91 psi	1.87 psi	186.23 psi	
One-Way Shears...					
Vu @ Left	5.54 psi	4.06 psi	2.62 psi	93.11 psi	
Vu @ Right	5.54 psi	4.06 psi	2.62 psi	93.11 psi	
Vu @ Top	0.00 psi	0.00 psi	0.00 psi	93.11 psi	
Vu @ Bottom	0.00 psi	0.00 psi	0.00 psi	93.11 psi	
Moments	<u>ACI C-1</u>	<u>ACI C-2</u>	<u>ACI C-3</u>	<u>Ru / Phi</u>	<u>As Req'd</u>
Mu @ Left	0.98 k-ft	0.72 k-ft	0.46 k-ft	15.0 psi	0.26 in2 per ft
Mu @ Right	0.98 k-ft	0.72 k-ft	0.46 k-ft	15.0 psi	0.26 in2 per ft
Mu @ Top	0.00 k-ft	0.00 k-ft	0.00 k-ft	0.0 psi	0.02 in2 per ft
Mu @ Bottom	0.00 k-ft	0.00 k-ft	0.00 k-ft	0.0 psi	0.02 in2 per ft

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General Footing Analysis & Design

Page 2
miami_gardens.ecw:Footings

Description WF-30 (qs'=2.5 ksf)(2 Story-Exterior Wall)Miami Gardens

Soil Pressure Summary

Service Load Soil Pressures	Bottom-	Top-Rig	Bottom-	Top-Lef
DL + LL	1,842.93	1,842.93	1,842.93	1,842.93 psf
DL + LL + ST	1,842.93	1,842.93	1,842.93	1,842.93 psf
Factored Load Soil Pressures				
ACI Eq. C-1	2,680.91	2,680.91	2,680.91	2,680.91 psf
ACI Eq. C-2	1,968.68	1,968.68	1,968.68	1,968.68 psf
ACI Eq. C-3	1,320.24	1,320.24	1,320.24	1,320.24 psf

ACI Factors (per ACI 318-02, applied internally to entered loads)

ACI C-1 & C-2 DL	1.400	ACI C-2 Group Factor	0.750	Add'l "1.4" Factor for Seismic	1.400
ACI C-1 & C-2 LL	1.700	ACI C-3 Dead Load Factor	0.900	Add'l "0.9" Factor for Seismic	0.900
ACI C-1 & C-2 ST	1.700	ACI C-3 Short Term Factor	1.300		
....seismic = ST * :	1.100	Used in ACI C-2 & C-3			

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General Footing Analysis & Design

Page 1
miami_gardens.ecw:Footings

Description WF-36 (qs'=2.5 ksf)(2 Story-Interior Wall)Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Allowable Soil Bearing 2,500.0 psf
Short Term Increase 1.330
Seismic Zone 0
Biaxial Applied Loads
Live & Short Term Combined
f'c 3,000.0 psi
Fy 60,000.0 psi
Concrete Weight 150.00 pcf
Overburden Weight 100.00 psf

Dimensions...
Width along X-X Axis 3.000 ft
Length along Y-Y Axis 1.000 ft
Footing Thickness 12.00 in
Col Dim. Along X-X Axis 8.00 in
Col Dim. Along Y-Y Axis 12.00 in
Base Pedestal Height 12.000 in
Min Steel % 0.0014
Rebar Center To Edge Distance 3.50 in

Loads

Applied Vertical Load...

Dead Load 4.806 k
Live Load 1.680 k
Short Term Load k

...ecc along X-X Axis 0.000 in
...ecc along Y-Y Axis 0.000 in

Applied Moments...

Dead Load k-ft
Live Load k-ft
Short Term k-ft

Creates Rotation about Y-Y Axis
(pressures @ left & right)

Creates Rotation about X-X Axis
(pressures @ top & bot)

Applied Shears...

Dead Load k
Live Load k
Short Term k

Creates Rotation about Y-Y Axis
(pressures @ left & right)

Creates Rotation about X-X Axis
(pressures @ top & bot)

Summary

Footing Design OK

3.00ft x 1.00ft Footing, 12.0in Thick, w/ Column Support 8.00 x 12.00in x 12.0in high
3.00ft x 1.00ft Footing, 12.0in Thick, w/ Column Suppo

	<u>DL+LL</u>	<u>DL+LL+ST</u>		<u>Actual</u>	<u>Allowable</u>
Max Soil Pressure	2,423.1	2,423.1 psf	Max Mu	2.184 k-ft per ft	
Allowable	2,500.0	3,325.0 psf	Required Steel Area		0.259 in2 per ft
"X" Ecc, of Resultant	0.000 in	0.000 in	Shear Stresses....	<u>Vu</u>	<u>Vn * Phi</u>
"Y" Ecc, of Resultant	0.000 in	0.000 in	1-Way	13.387	93.113 psi
X-X Min. Stability Ratio	No Overturning	1.500 :1	2-Way	5.928	186.226 psi
Y-Y Min. Stability Ratio	No Overturning				

Footing Design

Shear Forces	<u>ACI C-1</u>	<u>ACI C-2</u>	<u>ACI C-3</u>	<u>Vn * Phi</u>	
Two-Way Shear	0.00 psi	5.93 psi	3.55 psi	186.23 psi	
One-Way Shears...					
Vu @ Left	13.39 psi	9.89 psi	5.91 psi	93.11 psi	
Vu @ Right	13.39 psi	9.89 psi	5.91 psi	93.11 psi	
Vu @ Top	0.00 psi	0.00 psi	0.00 psi	93.11 psi	
Vu @ Bottom	0.00 psi	0.00 psi	0.00 psi	93.11 psi	
Moments	<u>ACI C-1</u>	<u>ACI C-2</u>	<u>ACI C-3</u>	<u>Ru / Phi</u>	<u>As Req'd</u>
Mu @ Left	2.18 k-ft	1.61 k-ft	0.97 k-ft	33.6 psi	0.26 in2 per ft
Mu @ Right	2.18 k-ft	1.61 k-ft	0.97 k-ft	33.6 psi	0.26 in2 per ft
Mu @ Top	0.00 k-ft	0.00 k-ft	0.00 k-ft	0.0 psi	0.02 in2 per ft
Mu @ Bottom	0.00 k-ft	0.00 k-ft	0.00 k-ft	0.0 psi	0.02 in2 per ft

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General Footing Analysis & Design

Page 2
miami_gardens.ecw:Footings

Description WF-36 (qs'=2.5 ksf)(2 Story-Interior Wall)Miami Gardens

Soil Pressure Summary

Service Load Soil Pressures	Bottom-	Top-Rig	Bottom-	Top-Lef
DL + LL	2,423.11	2,423.11	2,423.11	2,423.11 psf
DL + LL + ST	2,423.11	2,423.11	2,423.11	2,423.11 psf
Factored Load Soil Pressures				
ACI Eq. C-1	3,560.36	3,560.36	3,560.36	3,560.36 psf
ACI Eq. C-2	2,635.27	2,635.27	2,635.27	2,635.27 psf
ACI Eq. C-3	1,646.80	1,646.80	1,646.80	1,646.80 psf

ACI Factors (per ACI 318-02, applied internally to entered loads)

ACI C-1 & C-2 DL	1.400	ACI C-2 Group Factor	0.750	Add'l "1.4" Factor for Seismic	1.400
ACI C-1 & C-2 LL	1.700	ACI C-3 Dead Load Factor	0.900	Add'l "0.9" Factor for Seismic	0.900
ACI C-1 & C-2 ST	1.700	ACI C-3 Short Term Factor	1.300		
....seismic = ST * :	1.100	Used in ACI C-2 & C-3			

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Square Footing Design

Page 1
MIAMI_GARDENS.ECW:Footings

Description F-3x3x12 (15-5.2) (q's=2.5 ksf)Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Dead Load	15.000 k	Footing Dimension	3.000 ft
Live Load	5.200 k	Thickness	12.00 in
Short Term Load	0.000 k	# of Bars	4
Seismic Zone	0	Bar Size	5
Overburden Weight	100.000 psf	Rebar Cover	3.250
Concrete Weight	150.00 pcf	f'c	3,000.0 psi
LL & ST Loads Combine		Fy	60,000.0 psi
Load Duration Factor	1.330		
Column Dimension	0.00 in	Allowable Soil Bearing	2,500.00 psf

Note: Load factoring supports 2003 IBC and 2003 NFPA 5000 by virtue of their references to ACI 318-02 for concrete design.
Factoring of entered loads to ultimate loads within this program is according to ACI 318-02 C.2

Reinforcing

Rebar Requirement

Actual Rebar "d" depth used	8.438 in	As to USE per foot of Width	0.259 in2
200/Fy	0.0033	Total As Req'd	0.778 in2
As Req'd by Analysis	0.0011 in2	Min Allow % Reinf	0.0014
Min. Reinf % to Req'd	0.0014 %		

Summary

Footing OK

3.00ft square x 12.0in thick with 4- #5 bars

Max. Static Soil Pressure	2,494.44 psf	Vu : Actual One-Way	28.85 psi
Allow Static Soil Pressure	2,500.00 psf	Vn*Phi : Allow One-Way	93.11 psi
Max. Short Term Soil Pressure	2,494.44 psf	Vu : Actual Two-Way	109.49 psi
Allow Short Term Soil Pressure	3,325.00 psf	Vn*Phi : Allow Two-Way	186.23 psi

Mu : Actual	4.12 k-ft / ft
Mn * Phi : Capacity	14.94 k-ft / ft

Alternate Rebar Selections...

4 # 4's	3 # 5's	2 # 6's		
2 # 7's	1 # 8's	1 # 9's	1 # 10's	

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General Footing Analysis & Design

Page 1
MIAMI_GARDENS.ECW:Footings

Description F-3x5 (25-8)(q's=2.5 ksf)Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Allowable Soil Bearing 2,500.0 psf
Short Term Increase 1.330
Seismic Zone 0
Biaxial Applied Loads
Live & Short Term Combined
f'c 3,000.0 psi
Fy 59,500.0 psi
Concrete Weight 150.00 pcf
Overburden Weight 100.00 psf

Dimensions...
Width along X-X Axis 5.000 ft
Length along Y-Y Axis 3.000 ft
Footing Thickness 12.00 in
Col Dim. Along X-X Axis 42.00 in
Col Dim. Along Y-Y Axis 8.00 in
Base Pedestal Height 0.000 in
Min Steel % 0.0014
Rebar Center To Edge Distance 3.50 in

Loads

Applied Vertical Load...

Dead Load 25.000 k
Live Load 8.000 k
Short Term Load k

...ecc along X-X Axis 0.000 in
...ecc along Y-Y Axis 0.000 in

Applied Moments...

Dead Load k-ft
Live Load k-ft
Short Term k-ft

Creates Rotation about Y-Y Axis
(pressures @ left & right)

Creates Rotation about X-X Axis
(pressures @ top & bot)

Applied Shears...

Dead Load k
Live Load k
Short Term k

Creates Rotation about Y-Y Axis
(pressures @ left & right)

Creates Rotation about X-X Axis
(pressures @ top & bot)

Summary

Footing Design OK

5.00ft x 3.00ft Footing, 12.0in Thick, w/ Column Support 42.00 x 8.00in x 0.0in high
5.00ft x 3.00ft Footing, 12.0in Thick, w/ Column Suppo

	<u>DL+LL</u>	<u>DL+LL+ST</u>		<u>Actual</u>	<u>Allowable</u>
Max Soil Pressure	2,434.4	2,434.4 psf	Max Mu	2.189 k-ft per ft	
Allowable	2,500.0	3,325.0 psf	Required Steel Area		0.259 in2 per ft
"X" Ecc, of Resultant	0.000 in	0.000 in	Shear Stresses....	<u>Vu</u>	<u>Vn * Phi</u>
"Y" Ecc, of Resultant	0.000 in	0.000 in	1-Way	13.420	93.113 psi
X-X Min. Stability Ratio	No Overturning	1.500 :1	2-Way	23.185	151.276 psi
Y-Y Min. Stability Ratio	No Overturning				

Footing Design

Shear Forces	<u>ACI C-1</u>	<u>ACI C-2</u>	<u>ACI C-3</u>	<u>Vn * Phi</u>	
Two-Way Shear	23.19 psi	19.46 psi	11.98 psi	151.28 psi	
One-Way Shears...					
Vu @ Left	-0.14 psi	-0.11 psi	-0.09 psi	93.11 psi	
Vu @ Right	-0.14 psi	-0.11 psi	-0.09 psi	93.11 psi	
Vu @ Top	13.42 psi	10.06 psi	6.18 psi	93.11 psi	
Vu @ Bottom	13.42 psi	10.06 psi	6.18 psi	93.11 psi	
Moments	<u>ACI C-1</u>	<u>ACI C-2</u>	<u>ACI C-3</u>	<u>Ru / Phi</u>	<u>As Req'd</u>
Mu @ Left	0.90 k-ft	0.68 k-ft	0.42 k-ft	13.9 psi	0.26 in2 per ft
Mu @ Right	0.90 k-ft	0.68 k-ft	0.42 k-ft	13.9 psi	0.26 in2 per ft
Mu @ Top	2.19 k-ft	1.64 k-ft	1.01 k-ft	33.7 psi	0.26 in2 per ft
Mu @ Bottom	2.19 k-ft	1.64 k-ft	1.01 k-ft	33.7 psi	0.26 in2 per ft

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General Footing Analysis & Design

Page 2
MIAMI_GARDENS.ECW:Footings

Description F-3x5 (25-8)(q's=2.5 ksf)Miami Gardens

Soil Pressure Summary

Service Load Soil Pressures	Bottom-	Top-Rig	Bottom-	Top-Lef
DL + LL	2,434.44	2,434.44	2,434.44	2,434.44 psf
DL + LL + ST	2,434.44	2,434.44	2,434.44	2,434.44 psf
Factored Load Soil Pressures				
ACI Eq. C-1	3,568.22	3,568.22	3,568.22	3,568.22 psf
ACI Eq. C-2	2,676.17	2,676.17	2,676.17	2,676.17 psf
ACI Eq. C-3	1,711.00	1,711.00	1,711.00	1,711.00 psf

ACI Factors (per ACI 318-02, applied internally to entered loads)

ACI C-1 & C-2 DL	1.400	ACI C-2 Group Factor	0.750	Add'l "1.4" Factor for Seismic	1.400
ACI C-1 & C-2 LL	1.700	ACI C-3 Dead Load Factor	0.900	Add'l "0.9" Factor for Seismic	0.900
ACI C-1 & C-2 ST	1.700	ACI C-3 Short Term Factor	1.300		
....seismic = ST * :	1.100	Used in ACI C-2 & C-3			

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Square Footing Design

Page 1
MIAMI_GARDENS.ECW:Footings

Description F-4x4x12 (30-4)(q's=2.5 ksf) Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Dead Load	30.000 k	Footing Dimension	4.000 ft
Live Load	4.000 k	Thickness	12.00 in
Short Term Load	0.000 k	# of Bars	5
Seismic Zone	0	Bar Size	5
Overburden Weight	100.000 psf	Rebar Cover	3.250
Concrete Weight	150.00 pcf	f'c	3,000.0 psi
LL & ST Loads Combine		Fy	60,000.0 psi
Load Duration Factor	1.330		
Column Dimension	0.00 in	Allowable Soil Bearing	2,500.00 psf

Note: Load factoring supports 2003 IBC and 2003 NFPA 5000 by virtue of their references to ACI 318-02 for concrete design.
Factoring of entered loads to ultimate loads within this program is according to ACI 318-02 C.2

Reinforcing

Rebar Requirement

Actual Rebar "d" depth used	8.438 in	As to USE per foot of Width	0.259 in2
200/Fy	0.0033	Total As Req'd	1.037 in2
As Req'd by Analysis	0.0018 in2	Min Allow % Reinf	0.0014
Min. Reinf % to Req'd	0.0024 %		

Summary

Footing OK

4.00ft square x 12.0in thick with 5- #5 bars

Max. Static Soil Pressure	2,375.00 psf	Vu : Actual One-Way	43.55 psi
Allow Static Soil Pressure	2,500.00 psf	Vn*Phi : Allow One-Way	93.11 psi
Max. Short Term Soil Pressure	2,375.00 psf	Vu : Actual Two-Way	185.13 psi
Allow Short Term Soil Pressure	3,325.00 psf	Vn*Phi : Allow Two-Way	186.23 psi
Mu : Actual	6.80 k-ft / ft	Alternate Rebar Selections...	
Mn * Phi : Capacity	14.05 k-ft / ft	6 # 4's 4 # 5's 3 # 6's	
		2 # 7's 2 # 8's 2 # 9's 1 # 10's	

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Square Footing Design

Page 1
MIAMI_GARDENS.ECW:Footings

Description F-5x5x15 (42-13)(q's=2.5 ksf) Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Dead Load	42.000 k	Footing Dimension	5.000 ft
Live Load	13.000 k	Thickness	15.00 in
Short Term Load	0.000 k	# of Bars	6
Seismic Zone	0	Bar Size	5
Overburden Weight	100.000 psf	Rebar Cover	3.250
Concrete Weight	150.00 pcf	f'c	3,000.0 psi
LL & ST Loads Combine		Fy	60,000.0 psi
Load Duration Factor	1.330		
Column Dimension	0.00 in	Allowable Soil Bearing	2,500.00 psf

Note: Load factoring supports 2003 IBC and 2003 NFPA 5000 by virtue of their references to ACI 318-02 for concrete design.
Factoring of entered loads to ultimate loads within this program is according to ACI 318-02 C.2

Reinforcing

Rebar Requirement

Actual Rebar "d" depth used	11.438 in	As to USE per foot of Width	0.324 in2
200/Fy	0.0033	Total As Req'd	1.620 in2
As Req'd by Analysis	0.0016 in2	Min Allow % Reinf	0.0014
Min. Reinf % to Req'd	0.0022 %		

Summary

Footing OK

5.00ft square x 15.0in thick with 6- #5 bars

Max. Static Soil Pressure	2,487.50 psf	Vu : Actual One-Way	41.01 psi
Allow Static Soil Pressure	2,500.00 psf	Vn*Phi : Allow One-Way	93.11 psi
Max. Short Term Soil Pressure	2,487.50 psf	Vu : Actual Two-Way	167.52 psi
Allow Short Term Soil Pressure	3,325.00 psf	Vn*Phi : Allow Two-Way	186.23 psi
Mu : Actual	11.37 k-ft / ft	Alternate Rebar Selections...	
Mn * Phi : Capacity	18.53 k-ft / ft	9 # 4's 6 # 5's 4 # 6's	
		3 # 7's 3 # 8's 2 # 9's 2 # 10's	

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Square Footing Design

Page 1
MIAMI_GARDENS.ECW:Footings

Description F-6x6x16 (55-15.5)(q's=2.5 ksf) Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Dead Load	55.000 k	Footing Dimension	6.000 ft
Live Load	15.500 k	Thickness	16.00 in
Short Term Load	0.000 k	# of Bars	7
Seismic Zone	0	Bar Size	5
Overburden Weight	100.000 psf	Rebar Cover	3.250
Concrete Weight	150.00 pcf	f'c	3,000.0 psi
LL & ST Loads Combine		Fy	60,000.0 psi
Load Duration Factor	1.330		
Column Dimension	0.00 in	Allowable Soil Bearing	2,500.00 psf

Note: Load factoring supports 2003 IBC and 2003 NFPA 5000 by virtue of their references to ACI 318-02 for concrete design.
Factoring of entered loads to ultimate loads within this program is according to ACI 318-02 C.2

Reinforcing

Rebar Requirement

Actual Rebar "d" depth used	12.438 in	As to USE per foot of Width	0.360 in2
200/Fy	0.0033	Total As Req'd	2.157 in2
As Req'd by Analysis	0.0018 in2	Min Allow % Reinf	0.0014
Min. Reinf % to Req'd	0.0024 %		

Summary

Footing OK

6.00ft square x 16.0in thick with 7- #5 bars

Max. Static Soil Pressure	2,258.33 psf	Vu : Actual One-Way	43.29 psi
Allow Static Soil Pressure	2,500.00 psf	Vn*Phi : Allow One-Way	93.11 psi
Max. Short Term Soil Pressure	2,258.33 psf	Vu : Actual Two-Way	185.75 psi
Allow Short Term Soil Pressure	3,325.00 psf	Vn*Phi : Allow Two-Way	186.23 psi
Mu : Actual	14.81 k-ft / ft	Alternate Rebar Selections...	
Mn * Phi : Capacity	19.66 k-ft / ft	11 # 4's	7 # 5's
		4 # 7's	3 # 8's
		5 # 6's	3 # 9's
		2 # 10's	

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**J.C.D. ARCHITECT, Inc.**

JUAN C. DAVID R. A. AR-00015344

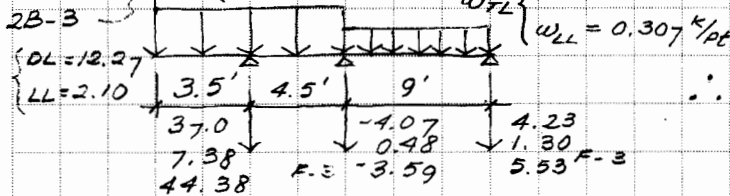
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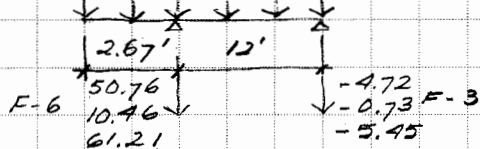
PROJECT No _____ SHEET No _____ OF _____

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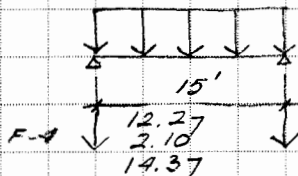
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CONCRETE BEAMS DESIGN2B-1 (8" x 24") $W_{DL} = 2.07 \text{ K/ft}$ $W_{TL} \left\{ \begin{array}{l} W_{LL} = 0.537 \text{ K/ft} \end{array} \right.$  $b = 8" \quad f'_c = 3.0 \text{ KSI, 1-LAYERS}$ $d = 24" - 2.31" = 21.75"$ $24" - 2.75" = 21.25"$ $\therefore$ PROVIDE 2#7 T. & 3#3 TIES,

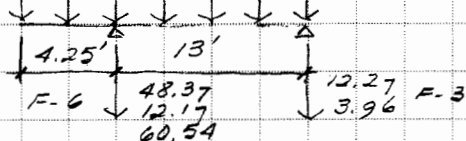
6 @ 4" O.C., BAL. @ 8" O.C.

2B-2 (8" x 24")
 $2B-1 \left\{ \begin{array}{l} DL = 37.0 \text{ W} \\ LL = 7.38 \end{array} \right. \left\{ \begin{array}{l} W_{DL} = 0.616 \text{ K/ft} \\ W_{LL} = 0.160 \text{ K/ft} \end{array} \right.$
 $W_{TL} \left\{ \begin{array}{l} W_{LL} = 0.160 \text{ K/ft} \end{array} \right.$  $b = 8" \quad f'_c = 3.0 \text{ KSI, 2-LAYERS}$ $d = 24" - 2.31" = 21.69" \quad 24" - 2.75" = 21.25"$ $\therefore$ PROVIDE 4#7 T. & 2#6 B. & 3#3 TIES

6 @ 4" O.C., BAL. @ 8" O.C.

2B-3 (8" x 24")
 $W_{TL} \left\{ \begin{array}{l} W_{DL} = 1.636 \text{ K/ft} \\ W_{LL} = 0.280 \text{ K/ft} \end{array} \right.$
 $b = 8" \quad f'_c = 3.0 \text{ KSI, 1-LAYER}$ $d = 24" - 2.31" = 21.69"$ $\therefore$ PROVIDE 2#7 B. & 2#6 T. & 3#3 TIES

6 @ 6" O.C., BAL. @ 8" O.C.

2B-4
 $2B-3 \left\{ \begin{array}{l} DL = 12.27 \text{ W} \\ LL = 2.10 \end{array} \right. \left\{ \begin{array}{l} W_{DL} = 2.804 \text{ K/ft} \\ W_{LL} = 0.840 \text{ K/ft} \end{array} \right.$
 $W_{TL} \left\{ \begin{array}{l} W_{LL} = 0.840 \text{ K/ft} \end{array} \right.$  $b = 8" \quad f'_c = 3.0 \text{ KSI, 2-LAYERS}$ $d = 24" - 2.31" = 21.69"$ $-2.75" = 21.25"$ $\therefore$ PROVIDE 2#7 B. & 4#7 TOP. & 3#3 TIES

6 @ 4" O.C., BAL. @ 5" O.C.

2B-5
 $W_{TL} \left\{ \begin{array}{l} W_{DL} = 1.636 \text{ K/ft} \\ W_{LL} = 0.280 \text{ K/ft} \end{array} \right.$
 $b = 8" \quad f'_c = 3.0 \text{ KSI, 2-LAYERS}$ $d = 24" - 2.31" = 21.69" \quad 24" - 2.75" = 21.25"$ $\therefore$ PROVIDE 4#7 B. & 2#6 T. & 3#3 TIES

8 @ 6" O.C., BAL. @ 8" O.C.

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Multi-Span Concrete Beam

Page 1
MIAMI_GARDENS.ECW:Beams

Description 2B-1 (8"x24")Conc. Beam (3-Spans 4'C+4.5'+9')Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy	60,000.0 psi	Spans Considered	Continuous Over Supports	ACI Dead Load Factor	1.40
f'c	3,000.0 psi	Stirrup Fy	60,000.0 psi	ACI Live Load Factor	1.70

Concrete Member Information

Description		S-1	S-2	S-3
Span	ft	3.50	4.50	9.00
Beam Width	in	8.00	8.00	8.00
Beam Depth	in	24.00	24.00	24.00
End Fixity		Free-Pin	Pin-Pin	Pin-Pin
Reinforcing	Center Area	1.20in2	1.20in2	1.20in2
	Bar Depth	21.69in	21.69in	21.75in
	Left Area	1.20in2	1.20in2	1.20in2
	Bar Depth	2.31in	2.31in	2.25in
	Right Area	1.20in2	1.20in2	1.20in2
	Bar Depth	2.31in	2.31in	2.25in

Loads

Using Live Load This Span ??		Yes	Yes	Yes
Dead Load	k/ft	2.100	2.100	0.906
Live Load	k/ft	0.537	0.537	0.307
Point #1 DL	k	12.270		
LL	k	2.100		
@ X	ft	0.000	0.000	0.000

Results

		Beam OK	Beam OK	Beam OK
Mmax @ Cntr	k-ft	0.00	0.70	18.48
@ X =	ft	0.00	4.50	4.44
Mn * Phi	k-ft	107.57	107.57	107.90
Max @ Left End	k-ft	0.00	-96.22	0.70
Mn * Phi	k-ft	107.57	107.57	107.90
Max @ Right End	k-ft	-96.22	0.70	0.00
Mn * Phi	k-ft	107.57	107.57	107.90
		Bending OK	Bending OK	Bending OK
Shear @ Left	k	0.00	30.21	7.98
Shear @ Right	k	34.23	12.87	8.13

Reactions & Deflections

DL @ Left	k	0.00	37.06	-4.07
LL @ Left	k	0.00	7.38	0.48
Total @ Left	k	0.00	44.45	-3.59
DL @ Right	k	37.06	-4.07	4.23
LL @ Right	k	7.38	0.48	1.30
Total @ Right	k	44.45	-3.59	5.53
Max. Deflection	in	-0.086	0.011	-0.006
@ X =	ft	0.00	1.83	4.50
Inertia : Effective	in4	3,670.45	3,670.45	9,216.00

Shear Stirrups

Stirrups @ Left	in	10.85	10.85	Not Req'd
Stirrups @ .4*L	in	10.85	10.85	Not Req'd
Stirrups @ .8*L	in	10.85	10.85	Not Req'd
Stirrups @ Right	in	10.85	10.85	Not Req'd

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Multi-Span Concrete Beam

Page 2
MIAMI_GARDENS.ECW:Beams

Description 2B-1 (8"x24")Conc. Beam (3-Spans 4'C+4.5'+9')Miami Gardens

Query Values

Location	ft	0.00	0.00	0.00
Moment	k-ft	0.0	-96.2	0.7
Shear	k	-20.7	30.2	8.0
Deflection	in	-0.0856	0.0000	0.0000

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Multi-Span Concrete Beam

Page 1
MIAMI_GARDENS.ECW:Beams

Description 2B-2 (8"x24") Concrete Beam (2-Spans 3'C+12')Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy	60,000.0 psi	Spans Considered	Continuous Over Supports	ACI Dead Load Factor	1.20
f'c	5,000.0 psi	Stirrup Fy	60,000.0 psi	ACI Live Load Factor	1.60

Concrete Member Information

Description		S-1	S-2
Span	ft	2.67	12.00
Beam Width	in	8.00	8.00
Beam Depth	in	24.00	24.00
End Fixity		Free-Pin	Pin-Pin
Reinforcing	Center	Area	1.20in2
		Bar Depth	21.25in
	Left	Area	2.40in2
		Bar Depth	2.75in
	Right	Area	2.40in2
		Bar Depth	2.75in

Loads

Using Live Load This Span ??		Yes	Yes
Dead Load	k/ft	0.616	0.616
Live Load	k/ft	0.160	0.160
Point #1 DL	k	37.000	
LL	k	7.380	
@ X	ft	0.000	0.000

Results

		Beam OK	Beam OK
Mmax @ Cntr	k-ft	0.00	0.00
@ X =	ft	0.00	0.00
Mn * Phi	k-ft	108.63	108.63
Max @ Left End	k-ft	0.00	-153.62
Mn * Phi	k-ft	205.14	205.14
Max @ Right End	k-ft	-153.62	0.00
Mn * Phi	k-ft	205.14	205.14
		Bending OK	Bending OK
Shear @ Left	k	0.00	18.77
Shear @ Right	k	58.87	6.83

Reactions & Deflections

DL @ Left	k	0.00	50.76
LL @ Left	k	0.00	10.46
Total @ Left	k	0.00	61.21
DL @ Right	k	50.76	-4.72
LL @ Right	k	10.46	-0.73
Total @ Right	k	61.21	-5.45
Max. Deflection	in	-0.226	0.144
@ X =	ft	0.00	4.88
Inertia : Effective	in4	2,726.76	2,726.76

Shear Stirrups

Spacing @ Left	in	5.31	10.63
Spacing @ .4*L	in	5.31	10.63
Spacing @ .8*L	in	5.31	Not Req'd
Spacing @ Right	in	5.31	Not Req'd

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Multi-Span Concrete Beam

Page 2
MIAMI_GARDENS.ECW:Beams

Description 2B-2 (8"x24") Concrete Beam (2-Spans 3'C+12')Miami Gardens

Query Values

Location	ft	0.00	0.00
Moment	k-ft	0.0	-153.6
Shear	k	-56.2	18.8
Deflection	in	-0.2260	0.0000

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Multi-Span Concrete Beam

Page 1
 MIAMI_GARDENS.ECW:Beams

Description 2B-3 (8"x24") Concrete Beam (1-Spans 15') Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy	60,000.0 psi	Spans Considered	Continuous Over Supports	ACI Dead Load Factor	1.40
f'c	3,000.0 psi	Stirrup Fy	60,000.0 psi	ACI Live Load Factor	1.70

Concrete Member Information

Description	S-1
Span	ft 15.00
Beam Width	in 8.00
Beam Depth	in 24.00
End Fixity	Pin-Pin
Reinforcing	Center
	Area 1.20in2
	Bar Depth 21.69in
	Left
	Area 0.88in2
	Bar Depth 2.31in
	Right
	Area 0.88in2
	Bar Depth 2.31in

Loads

Using Live Load This Span ??	Yes
Dead Load	k/ft 1.636
Live Load	k/ft 0.280

Results

Beam OK

Mmax @ Cntr	k-ft 77.80
@ X =	ft 7.50
Mn * Phi	k-ft 107.57
Max @ Left End	k-ft 0.00
Mn * Phi	k-ft 80.71
Max @ Right End	k-ft 0.00
Mn * Phi	k-ft 80.71
	Bending OK
Shear @ Left	k 20.75
Shear @ Right	k 20.75

Reactions & Deflections

DL @ Left	k 12.27
LL @ Left	k 2.10
Total @ Left	k 14.37
DL @ Right	k 12.27
LL @ Right	k 2.10
Total @ Right	k 14.37
Max. Deflection	in -0.175
@ X =	ft 7.50
Inertia : Effective	in4 3,990.79

Shear Stirrups

Stirrup Rebar Area	in2 0.220
Spacing @ Left	in 10.85
Spacing @ .2*L	in 10.85
Spacing @ .4*L	in Not Req'd
Spacing @ .5*L	in Not Req'd
Spacing @ .8*L	in 10.85
Spacing @ Right	in 10.85

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Multi-Span Concrete Beam

Page 2
MIAMI_GARDENS.ECW:Beams

Description 2B-3 (8"x24") Concrete Beam (1-Spans 15') Miami Gardens

Query Values

Location	ft	0.00
Moment	k-ft	-0.0
Shear	k	20.7
Deflection	in	0.0000

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Multi-Span Concrete Beam

Page 1
MIAMI_GARDENS.ECW:Beams

Description 2B-4 (8"x24") Concrete Beam (2-Spans 4.25'C+13')Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy	60,000.0 psi	Spans Considered	Continuous Over Supports	ACI Dead Load Factor	1.40
f'c	3,000.0 psi	Stirrup Fy	60,000.0 psi	ACI Live Load Factor	1.70

Concrete Member Information

Description		S-1	S-2
Span	ft	4.25	13.00
Beam Width	in	8.00	8.00
Beam Depth	in	24.00	24.00
End Fixity		Free-Pin	Pin-Pin
Reinforcing	Center	Area	0.88in2
		Bar Depth	21.25in
	Left	Area	2.40in2
		Bar Depth	2.75in
	Right	Area	2.40in2
		Bar Depth	2.75in

Loads

Using Live Load This Span ??		Yes	Yes
Dead Load	k/ft	2.804	2.804
Live Load	k/ft	0.840	0.804
Point #1 DL	k	12.270	
LL	k	2.100	
@ X	ft	0.000	0.000

Results

		Beam OK	Beam OK
Mmax @ Cntr	k-ft	0.00	53.96
@ X =	ft	0.00	8.49
Mn * Phi	k-ft	78.96	105.20
Max @ Left End	k-ft	0.00	-136.53
Mn * Phi	k-ft	191.35	191.35
Max @ Right End	k-ft	-136.53	0.00
Mn * Phi	k-ft	191.35	191.35
		Bending OK	Bending OK
Shear @ Left	k	0.00	44.90
Shear @ Right	k	43.50	23.90

Reactions & Deflections

DL @ Left	k	0.00	48.37
LL @ Left	k	0.00	12.17
Total @ Left	k	0.00	60.54
DL @ Right	k	48.37	12.27
LL @ Right	k	12.17	3.96
Total @ Right	k	60.54	16.22
Max. Deflection	in	-0.164	-0.065
@ X =	ft	0.00	8.06
Inertia : Effective	in4	2,617.91	3,289.73

Shear Stirrups

Stirrups @ Left	in	10.63	5.31
Spacing @ Left	in	10.63	5.31
Spacing @ .4*L	in	10.63	5.31
Spacing @ .8*L	in	5.31	10.63
Spacing @ Right	in	5.31	10.63

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Multi-Span Concrete Beam

Page 2
MIAMI_GARDENS.ECW:Beams

Description 2B-4 (8"x24") Concrete Beam (2-Spans 4.25'C+13')Miami Gardens

Query Values

Location	ft	0.00	0.00
Moment	k-ft	0.0	-136.5
Shear	k	-20.7	44.9
Deflection	in	-0.1636	0.0000

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Multi-Span Concrete Beam

Page 1
MIAMI_GARDENS.ECW:Beams

Description 2B-5 (8"x24") Concrete Beam (1-Spans 22') Miami Gardens

General Information

Code Ref: ACI 318-02, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Fy	60,000.0 psi	Spans Considered	Continuous Over Supports	ACI Dead Load Factor	1.40
f'c	3,000.0 psi	Stirrup Fy	60,000.0 psi	ACI Live Load Factor	1.70

Concrete Member Information

Description	S-1
Span	ft 22.00
Beam Width	in 8.00
Beam Depth	in 24.00
End Fixity	Pin-Pin
Reinforcing	Center
	Area 2.40in2
	Bar Depth 21.25in
	Left
	Area 0.88in2
	Bar Depth 2.75in
	Right
	Area 0.88in2
	Bar Depth 2.75in

Loads

Using Live Load This Span ??	Yes
Dead Load	k/ft 1.636
Live Load	k/ft 0.280

Results

Beam OK

Mmax @ Cntr	k-ft 167.37
@ X =	ft 11.00
Mn * Phi	k-ft 191.35
Max @ Left End	k-ft 0.00
Mn * Phi	k-ft 78.96
Max @ Right End	k-ft 0.00
Mn * Phi	k-ft 78.96
	Bending OK
Shear @ Left	k 30.43
Shear @ Right	k 30.43

Reactions & Deflections

DL @ Left	k 18.00
LL @ Left	k 3.08
Total @ Left	k 21.08
DL @ Right	k 18.00
LL @ Right	k 3.08
Total @ Right	k 21.08
Max. Deflection	in -0.609
@ X =	ft 11.00
Inertia : Effective	in4 5,307.60

Shear Stirrups

Stirrup Rebar Area	in2 0.220
Spacing @ Left	in 10.63
Spacing @ .2*L	in 10.63
Spacing @ .4*L	in Not Req'd
Spacing @ .5*L	in Not Req'd
Spacing @ .8*L	in 10.63
Spacing @ Right	in 10.63

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Multi-Span Concrete Beam

Page 2
MIAMI_GARDENS.ECW:Beams

Description 2B-5 (8"x24") Concrete Beam (1-Spans 22') Miami Gardens

Query Values

Location	ft	0.00
Moment	k-ft	-0.0
Shear	k	30.4
Deflection	in	0.0000

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Steel Column

Page 1
MIAMI_GARDENS.ECW:Columns

Description TS 4"x4"x5/16" Steel Column (14')(33-15)Miami Gardens

General Information

Code Ref: AISC 9th ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Steel Section	TS4X4X5/16	Fy	36.00 ksi	X-X Sidesway :	Restrained
		Duration Factor	1.330	Y-Y Sidesway :	Restrained
Column Height	9.000 ft	Elastic Modulus	29,000.00 ksi		
End Fixity	Pin-Pin	X-X Unbraced	9.000 ft	Kxx	1.000
Live & Short Term Loads Combined		Y-Y Unbraced	9.000 ft	Kyy	1.000

Loads

Axial Load...

Dead Load	55.00 k	Ecc. for X-X Axis Moments	0.000 in
Live Load	15.00 k	Ecc. for Y-Y Axis Moments	0.000 in
Short Term Load	k		

Summary

Column Design OK

Section : TS4X4X5/16, Height = 9.00ft, Axial Loads: DL = 55.00, LL = 15.00, ST = 0.00k, Ecc. = 0.000in

Unbraced Lengths: X-X = 9.00ft, Y-Y = 9.00ft

Combined Stress Ratios	<u>Dead</u>	<u>Live</u>	<u>DL + LL</u>	<u>DL + ST + (LL if Chosen)</u>
AISC Formula H1 - 1	0.7820	0.2133	0.9953	0.7483
AISC Formula H1 - 2	0.5840	0.1593	0.7433	0.5589
AISC Formula H1 - 3				

XX Axis : Fa calc'd per Eq. E2-1, $K^*L/r < C_c$

YY Axis : Fa calc'd per Eq. E2-1, $K^*L/r < C_c$

Stresses

Allowable & Actual Stresses	<u>Dead</u>	<u>Live</u>	<u>DL + LL</u>	<u>DL + Short</u>
Fa : Allowable	16.13 ksi	16.13 ksi	16.13 ksi	21.45 ksi
fa : Actual	12.61 ksi	3.44 ksi	16.06 ksi	16.06 ksi
Fb:xx : Allow [F1-6]	21.60 ksi	21.60 ksi	21.60 ksi	28.73 ksi
Fb:xx : Allow [F1-7] & [F1-8]	21.60 ksi	21.60 ksi	21.60 ksi	28.73 ksi
fb : xx Actual	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi
Fb:yy : Allow [F1-6]	21.60 ksi	21.60 ksi	21.60 ksi	28.73 ksi
Fb:yy : Allow [F1-7] & [F1-8]	21.60 ksi	21.60 ksi	21.60 ksi	28.73 ksi
fb : yy Actual	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi

Analysis Values

F'ex : DL+LL	28,131 psi	Cm:x DL+LL	0.60	Cb:x DL+LL	1.00
F'ey : DL+LL	28,131 psi	Cm:y DL+LL	0.60	Cb:y DL+LL	1.00
F'ex : DL+LL+ST	37,414 psi	Cm:x DL+LL+ST	0.60	Cb:x DL+LL+ST	1.00
F'ey : DL+LL+ST	37,414 psi	Cm:y DL+LL+ST	0.60	Cb:y DL+LL+ST	1.00
Max X-X Axis Deflection	0.000 in at	0.000 ft	Max Y-Y Axis Deflection	0.000 in at	0.000 ft

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Steel Column

Page 2
MIAMI_GARDENS.ECW:Columns

Description TS 4"x4"x5/16" Steel Column (14')(33-15)Miami Gardens

Section Properties TS4X4X5/16

Depth	4.000 in	Weight	14.81 #/ft	Values for LRFD Design....	
Thickness	0.313 in	Ixx	9.580 in4	J	16.100 in4
Width	4.000 in	Iyy	9.580 in4		0.00
		Sxx	4.790 in3	Zx	5.900 in3
Area	4.36 in2	Syy	4.790 in3	Zy	5.900 in3
Rt	2.000 in	Rxx	1.480 in		0.000
		Ryy	1.480 in		

Section Type = TS-Square

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Steel Column Base Plate

MIAMI_GARDENS.ECW:Studs & Joists

Description 12"x12"x3/4" Base Plate (Miami Gardens)

General Information

Code Ref : AISC 9th Ed ASD, 1997 UBC, 2003 IBC, 2003 NFPA 5000

Loads

Axial Load 50.00 k
X-X Axis Moment 0.00 k-ft

Plate Dimensions

Plate Length 12.000 in
Plate Width 12.000 in
Plate Thickness **0.750 in**

Support Pier Size

Pier Length 36.000 in
Pier Width 36.000 in

Steel Section

Section Length 4.000 in
Section Width 4.000 in
Flange Thickness 0.313 in
Web Thickness 0.000 in

TS4x4x5/16

Allowable Stresses

Concrete f'c 3,000.0 psi
Base Plate Fy 36.00 ksi
Load Duration Factor 1.330

Anchor Bolt Data

Dist. from Plate Edge 3.000 in
Bolt Count per Side 2
Tension Capacity 7.360 k
Bolt Area 0.442 in²

Summary

Baseplate OK

Concrete Bearing Stress **Bearing Stress OK**
Actual Bearing Stress 347.2 psi
Allow per ACI318-95, A3.1
= $0.3 \cdot f'_c \cdot \sqrt{A_2/A_1} \cdot LDF$ 2,394.0 psi
Allow per AISC J9 2,793.0 psi

Full Bearing : No Bolt Tension

Plate Bending Stress **Thickness OK**
Actual fb 35,851.9 psi
Max Allow Plate Fb 35,910.0 psi

Tension Bolt Force **Bolt Tension OK**
Actual Tension 0.000 k
Allowable 7.360 k

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Masonry Wall Design

Page 1
MIAMI_GARDENS.ECW:CMU

Description 8" Reinf. C.M.U. Wall Design-1st Floor (Zone-4)Miami Gardens

General Information

Code Ref: ACI 530-02

Wall Height	11.00 ft	Seismic Factor	0.0000	f'm	1,500.0 psi
Parapet Height	0.00 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.330	Special Inspection	
Thickness	8.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	5			Medium Weight Block	
Rebar Spacing	48 in			Equivalent	
Depth to Rebar	3.810 in @ Center			Solid Thickness	4.600 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	38.720 psf
Dead Load	2,658.000 #/ft	Dead Load	0.000 #/ft		
Live Load	840.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	0.000 in	Roof Load			
Floor Load					

Design Values

E	1,350,000 psi	Rebar Area	0.078 in ²	np	0.03641	j	0.92137
n : Es / Em	21.481	Radius of Gyration	2.535 in	k	0.23590	2 / kj	9.20187
Wall Weight	49.000 psf	Moment of Inertia	354.810 in ⁴				
Max Allow Axial Stress = 0.25 f'm (1-(h/140r) ²) * Splnsp				323.14 psi			
Allow Masonry Bending Stress = 0.33 f'm * Splnsp =				495.00 psi			
Allow Steel Bending Stress =				24,000.00 psi			

Load Combination & Stress Details Summary

	Moment in-#	Axial Load lbs	Bending Stresses		Axial Compression
			Steel psi	Masonry psi	
Top of Wall					
DL + LL	0.0	3,498.0	0.0	0.0	63.37
DL + LL + Wind	0.0	3,498.0	0.0	0.0	63.37
DL + LL + Seismic	0.0	3,498.0	0.0	0.0	63.37
Between Base & Top of Wall					
DL + LL	0.0	3,767.5	0.0	0.0	68.25
DL + LL + Wind	7,027.7	3,767.5	25,831.6	371.2	68.25
DL + LL + Seismic	0.0	3,767.5	0.0	0.0	68.25

Summary

11.00ft high wall with 0.00ft parapet, Med Wt Block w/ 8.00in wall w/ #5 bars at 48.00ino.c. at center

Max. Bending Compressive Stress	439.49 OK
Allowable	658.35 OK
Max. Axial Only Compressive Stress	68.25 psi
Allowable	323.14 OK
Max Steel Bending Stress	25,831.64 psi
Allowable	31,920.00 OK

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Masonry Wall Design

Page 2
MIAMI_GARDENS.ECW:CMU

Description 8" Reinf. C.M.U. Wall Design-1st Floor (Zone-4)Miami Gardens

Final Loads & Moments

Wall Weight moment @ Mid Ht	269.50 lbs	Wind Moment @ Mid Ht	7,027.68 in-#
		Seismic Moment @ Mid Ht	0.00 in-#
Dead Load Moment @ Top of Wall	0.00 in-#		
Dead Load Moment @ Mid Ht	0.00 in-#	Total Dead Load	2,658.00 lbs
		Total Live Load	840.00 lbs
Live Load Moment @ Top of Wall	0.00 in-#		
LiveLoad Moment @ Mid Ht	0.00 in-#		
Maximum Allow Moment for Applied Axial Load =		6,529.37 in-#	
Maximum Allow Axial Load for Applied Moment =		17,837.11 lbs	

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Masonry Wall Design

Page 1
MIAMI_GARDENS.ECW:CMU

Description 8" Reinf. C.M.U. Wall Design-1st Floor (Zone-5)Miami Gardens

General Information

Code Ref: ACI 530-02

Wall Height	11.00 ft	Seismic Factor	0.0000	f'm	1,500.0 psi
Parapet Height	0.00 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.330	Special Inspection	
Thickness	8.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	5			Medium Weight Block	
Rebar Spacing	48 in			Equivalent	
Depth to Rebar	3.810 in @ Center			Solid Thickness	4.600 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	45.000 psf
Dead Load	2,658.000 #/ft	Dead Load	0.000 #/ft		
Live Load	840.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	0.000 in	Roof Load			
Floor Load					

Design Values

E	1,350,000 psi	Rebar Area	0.078 in ²	np	0.03641	j	0.92137
n : Es / Em	21.481	Radius of Gyration	2.535 in	k	0.23590	2 / kj	9.20187
Wall Weight	49.000 psf	Moment of Inertia	354.810 in ⁴				
Max Allow Axial Stress = 0.25 f'm (1-(h/140r) ²) * Splnsp				323.14 psi			
Allow Masonry Bending Stress = 0.33 f'm * Splnsp =				495.00 psi			
Allow Steel Bending Stress =				24,000.00 psi			

Load Combination & Stress Details Summary

	Moment in-#	Axial Load lbs	Bending Stresses		Axial Compression
			Steel psi	Masonry psi	psi
Top of Wall					
DL + LL	0.0	3,498.0	0.0	0.0	63.37
DL + LL + Wind	0.0	3,498.0	0.0	0.0	63.37
DL + LL + Seismic	0.0	3,498.0	0.0	0.0	63.37
Between Base & Top of Wall					
DL + LL	0.0	3,767.5	0.0	0.0	68.25
DL + LL + Wind	8,167.5	3,767.5	30,021.3	431.5	68.25
DL + LL + Seismic	0.0	3,767.5	0.0	0.0	68.25

Summary

11.00ft high wall with 0.00ft parapet, Med Wt Block w/ 8.00in wall w/ #5 bars at 48.00ino.c. at center

Max. Bending Compressive Stress	499.70 OK
Allowable	658.35 OK
Max. Axial Only Compressive Stress	68.25 psi
Allowable	323.14 OK
Max Steel Bending Stress	30,021.28 psi
Allowable	31,920.00 OK

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Masonry Wall Design

Page 2
MIAMI_GARDENS.ECW:CMU

Description 8" Reinf. C.M.U. Wall Design-1st Floor (Zone-5)Miami Gardens

Final Loads & Moments

Wall Weight moment @ Mid Ht	269.50 lbs	Wind Moment @ Mid Ht	8,167.50 in-#
		Seismic Moment @ Mid Ht	0.00 in-#
Dead Load Moment @ Top of Wall	0.00 in-#		
Dead Load Moment @ Mid Ht	0.00 in-#	Total Dead Load	2,658.00 lbs
		Total Live Load	840.00 lbs
Live Load Moment @ Top of Wall	0.00 in-#		
LiveLoad Moment @ Mid Ht	0.00 in-#		
Maximum Allow Moment for Applied Axial Load =		6,529.37 in-#	
Maximum Allow Axial Load for Applied Moment =		17,837.11 lbs	

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Masonry Wall Design

Page 1
MIAMI_GARDENS.ECW:CMU

Description 8" Reinf. C.M.U. Wall Design-2nd Floor (Zone-4)Miami Gardens

General Information

Code Ref: ACI 530-02

Wall Height	10.00 ft	Seismic Factor	0.0000	f'm	1,500.0 psi
Parapet Height	2.00 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.330	Special Inspection	
Thickness	8.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	5			Medium Weight Block	
Rebar Spacing	48 in			Equivalent	
Depth to Rebar	3.810 in @ Center			Solid Thickness	4.600 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	39.190 psf
Dead Load	1,004.000 #/ft	Dead Load	0.000 #/ft		
Live Load	360.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	0.000 in	Roof Load			
Floor Load					

Design Values

E	1,350,000 psi	Rebar Area	0.078 in ²	np	0.03641	j	0.92137
n : Es / Em	21.481	Radius of Gyration	2.535 in	k	0.23590	2 / kj	9.20187
Wall Weight	49.000 psf	Moment of Inertia	354.810 in ⁴				
Max Allow Axial Stress = 0.25 f'm (1-(h/140r) ²) * Splnsp				332.14 psi			
Allow Masonry Bending Stress = 0.33 f'm * Splnsp =				495.00 psi			
Allow Steel Bending Stress =				24,000.00 psi			

Load Combination & Stress Details Summary

	Moment in-#	Axial Load lbs	Bending Stresses		Axial Compression
			Steel psi	Masonry psi	psi
Top of Wall					
DL + LL	0.0	1,462.0	0.0	0.0	26.49
DL + LL + Wind	940.6	1,462.0	3,457.2	49.7	26.49
DL + LL + Seismic	0.0	1,462.0	0.0	0.0	26.49
Between Base & Top of Wall					
DL + LL	0.0	1,707.0	0.0	0.0	30.92
DL + LL + Wind	5,417.6	1,707.0	19,913.6	286.2	30.92
DL + LL + Seismic	0.0	1,707.0	0.0	0.0	30.92

Summary

10.00ft high wall with 2.00ft parapet, Med Wt Block w/ 8.00in wall w/ #5 bars at 48.00ino.c. at center

Max. Bending Compressive Stress	317.11 OK
Allowable	658.35 OK
Max. Axial Only Compressive Stress	30.92 psi
Allowable	332.14 OK
Max Steel Bending Stress	19,913.57 psi
Allowable	31,920.00 OK

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Masonry Wall Design

Page 2
MIAMI_GARDENS.ECW:CMU

Description 8" Reinf. C.M.U. Wall Design-2nd Floor (Zone-4)Miami Gardens

Final Loads & Moments

Wall Weight moment @ Mid Ht	343.00 lbs	Wind Moment @ Mid Ht	5,417.62 in-#
		Seismic Moment @ Mid Ht	0.00 in-#
Dead Load Moment @ Top of Wall	0.00 in-#		
Dead Load Moment @ Mid Ht	0.00 in-#	Total Dead Load	1,004.00 lbs
		Total Live Load	360.00 lbs
Live Load Moment @ Top of Wall	0.00 in-#		
LiveLoad Moment @ Mid Ht	0.00 in-#		
Maximum Allow Moment for Applied Axial Load =		6,529.37 in-#	
Maximum Allow Axial Load for Applied Moment =		18,333.97 lbs	

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Masonry Wall Design

Page 1
MIAMI_GARDENS.ECW:CMU

Description 8" Reinf. C.M.U. Wall Design-2nd Floor (Zone-5)Miami Gardens

General Information

Code Ref: ACI 530-02

Wall Height	10.00 ft	Seismic Factor	0.0000	f'm	1,500.0 psi
Parapet Height	2.00 ft	Calc of Em = f'm *	900.00	Fs	24,000.0 psi
		Duration Factor	1.330	Special Inspection	
Thickness	8.0 in	Wall Wt Mult.	1.000	Grout @ Rebar Only	
Rebar Size	5			Medium Weight Block	
Rebar Spacing	48 in			Equivalent	
Depth to Rebar	3.810 in @ Center			Solid Thickness	4.600 in

Loads

Uniform Load		Concentric Axial Load		Wind Load	45.910 psf
Dead Load	1,004.000 #/ft	Dead Load	0.000 #/ft		
Live Load	360.000 #/ft	Live Load	0.000 #/ft		
Load Eccentricity	0.000 in	Roof Load			
Floor Load					

Design Values

E	1,350,000 psi	Rebar Area	0.078 in ²	np	0.03641	j	0.92137
n : Es / Em	21.481	Radius of Gyration	2.535 in	k	0.23590	2 / kj	9.20187
Wall Weight	49.000 psf	Moment of Inertia	354.810 in ⁴				
Max Allow Axial Stress = 0.25 f'm (1-(h/140r) ²) * Splnsp				332.14 psi			
Allow Masonry Bending Stress = 0.33 f'm * Splnsp =				495.00 psi			
Allow Steel Bending Stress =				24,000.00 psi			

Load Combination & Stress Details Summary

	Moment in-#	Axial Load lbs	Bending Stresses		Axial Compression
			Steel psi	Masonry psi	psi
Top of Wall					
DL + LL	0.0	1,462.0	0.0	0.0	26.49
DL + LL + Wind	1,101.8	1,462.0	4,050.0	58.2	26.49
DL + LL + Seismic	0.0	1,462.0	0.0	0.0	26.49
Between Base & Top of Wall					
DL + LL	0.0	1,707.0	0.0	0.0	30.92
DL + LL + Wind	6,346.6	1,707.0	23,328.2	335.3	30.92
DL + LL + Seismic	0.0	1,707.0	0.0	0.0	30.92

Summary

10.00ft high wall with 2.00ft parapet, Med Wt Block w/ 8.00in wall w/ #5 bars at 48.00ino.c. at center

Max. Bending Compressive Stress	366.19 OK
Allowable	658.35 OK
Max. Axial Only Compressive Stress	30.92 psi
Allowable	332.14 OK
Max Steel Bending Stress	23,328.19 psi
Allowable	31,920.00 OK

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Masonry Wall Design

Page 2
MIAMI_GARDENS.ECW:CMU

Description 8" Reinf. C.M.U. Wall Design-2nd Floor (Zone-5)Miami Gardens

Final Loads & Moments

Wall Weight moment @ Mid Ht	343.00 lbs	Wind Moment @ Mid Ht	6,346.60 in-#
		Seismic Moment @ Mid Ht	0.00 in-#
Dead Load Moment @ Top of Wall	0.00 in-#		
Dead Load Moment @ Mid Ht	0.00 in-#	Total Dead Load	1,004.00 lbs
		Total Live Load	360.00 lbs
Live Load Moment @ Top of Wall	0.00 in-#		
LiveLoad Moment @ Mid Ht	0.00 in-#		
Maximum Allow Moment for Applied Axial Load =		6,529.37 in-#	
Maximum Allow Axial Load for Applied Moment =		18,333.97 lbs	

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WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

Description: **Miami Gardens T.** Company Name: **J.C.D.**
 Analysis by: **O.G.**

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	175	mph
Struc Category (I, II, III, or IV)	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.3	:12
Slope of Roof (Theta)	1.2	Deg
Type of Roof	Monoslope	
Kd (Directionality Factor)	0.85	
Eave Height (Eht)	19.83	ft
Ridge Height (Rht)	20.17	ft
Mean Roof Height (Ht)	20.00	ft
Width Perp. To Wind Dir (B)	36.00	ft
Width Paral. To Wind Dir (L)	187.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.56
Flexible Structure	No

Calculated Parameters		
<i>Hurricane Prone Region (V>100 mph)</i>		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Gust Factor Category I: Rigid Structures - Simplified Method			
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85	
Gust Factor Category II: Rigid Structures - Complete Analysis			
Zm	Zmin	15.00	ft
Izm	$Cc * (33/z)^{0.167}$	0.2281	
Lzm	$l * (zm/33)^{Epsilon}$	427.06	ft
Q	$(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^{0.63}))^{0.5}$	0.9225	
Gust2	$0.925 * ((1+1.7*Izm*3.4*Q)/(1+1.7*3.4*Izm))$	0.8842	
Gust Factor Summary			
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85	

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	0.18	-0.18

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WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev ft	Kz	Kzt	qz lb/ft ²	Pressure (lb/ft ²)					Shear (Kip)	Moment (Kip-ft)
				Windward Wall*		Leeward Wall		Total		
				+GCpi	-GCpi	+GCpi	-GCpi	+/-Gcpi		
20.17	0.90	1.00	60.21	30.12	51.76	-21.04	0.60	51.16	0.31	0.03
20	0.90	1.00	60.10	30.05	51.69	-21.04	0.60	51.09	0.63	0.08
19.83	0.90	1.00	59.99	29.98	51.61	-21.04	0.60	51.01	9.50	21.55
15	0.85	1.00	56.57	27.65	49.29	-21.04	0.60	48.68	35.79	384.10

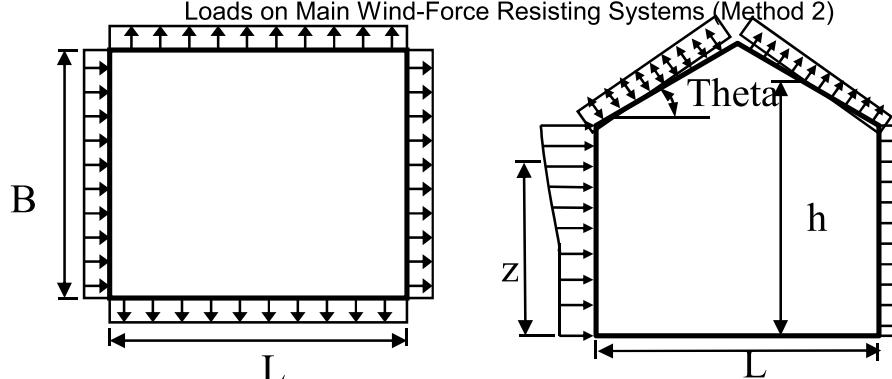
Note: 1) Positive forces act toward the face and Negative forces act away from the face.

Figure 6.5.12.2.4 Parapets

Type of Parapet	Enclosed	
Top Elevation of Parapet	22.00	ft
Parapet-Windward Pressure	110.38	lb/ft ²
Parapet-Leeward Pressure	-67.45	lb/ft ²

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	$2.01 \cdot (Ht/zg)^{2/\alpha}$	0.90	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	$.00256 \cdot (V)^2 \cdot Kh \cdot Kht \cdot Kd$	60.10	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.90	
Qhcc	$.00256 \cdot V^2 \cdot Khcc \cdot Kht \cdot Kd$	60.10	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00
Calculations for Wind Normal to 36 ft Face	Cp Pressure (psf)

WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

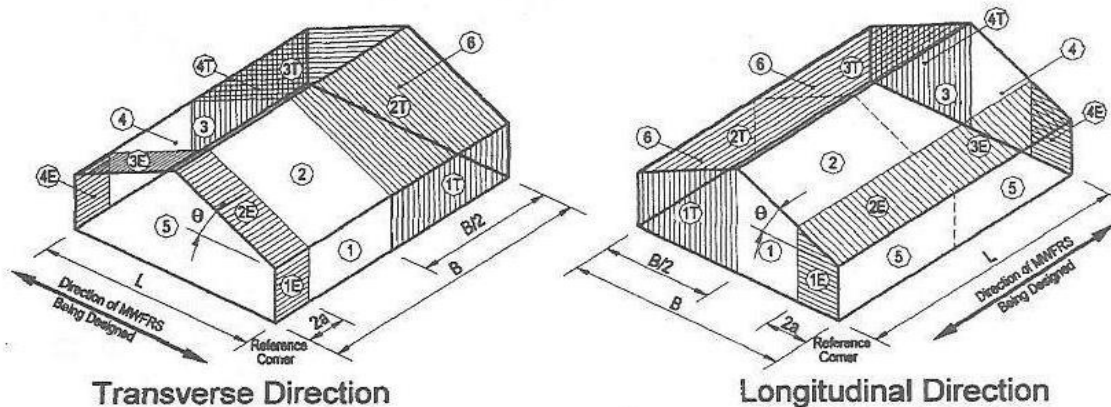
Additional Runs may be req'd for other wind directions		+GCpi	-GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.20	-21.04	0.60
Leeward Walls (Wind Dir Normal to 187 ft wall)	-0.50	-36.36	-14.72
Side Walls	-0.70	-46.58	-24.94
Overhang Bottom (Applicable on Windward only)	0.80	40.80	40.80
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 40 ft - Max Cp	-0.18	-20.01	1.62
Dist from Windward Edge: 0 ft to 10 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 20 ft to 36 ft - Min Cp	-0.50	-36.36	-14.72
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 187 ft face			
Dist from Windward Edge: 0 ft to 40 ft - Max Cp	-0.18	-20.01	1.62
Dist from Windward Edge: 0 ft to 10 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.88	-55.66	-34.02
Dist from Windward Edge: 20 ft to 36 ft - Min Cp	-0.52	-37.50	-15.86

* Horizontal distance from windward edge

Figure 6-10 - External Pressure Coefficients, GCpf

Loads on Main Wind-Force Resisting Systems w/ Ht <= 60 ft

Kh =	2.01*(Ht/zg)^(2/Alpha)	=	0.90
Kht =	Topographic factor (Fig 6-2)	=	1.00
Qh =	0.00256*(V)^2*Kh*Kht*Kd	=	60.10
Theta =	Angle of Roof	=	1.2 Deg



Torsional Load Cases

Wind Pressures on Main Wind Force Resisting System						
Surface	GCpf	+GCpi	-GCpi	qh (psf)	Min P (psf)	Max P (psf)
1	0.40	0.18	-0.18	60.10	13.22	34.86
2	0.40	0.18	-0.18	60.10	13.22	34.86
3	-0.37	0.18	-0.18	60.10	-33.06	-11.42
4	-0.29	0.18	-0.18	60.10	-28.25	-6.61

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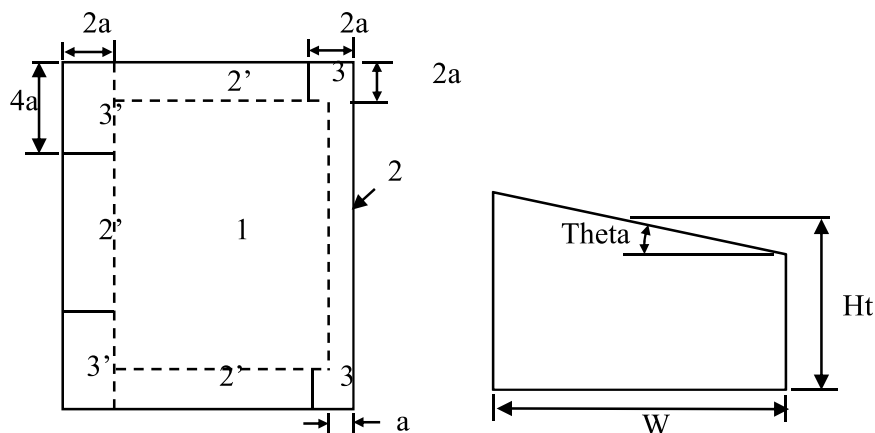
Detailed Wind Load Design (Method 2) per ASCE 7-10

5	-0.45	0.18	-0.18	60.10	-37.86	-16.23
6	-0.45	0.18	-0.18	60.10	-37.86	-16.23
1E	0.61	0.18	-0.18	60.10	25.84	47.48
2E	-1.07	0.18	-0.18	60.10	-75.13	-53.49
3E	-0.53	0.18	-0.18	60.10	-42.67	-21.04
4E	-0.43	0.18	-0.18	60.10	-36.66	-15.03

* $p = qh * (GC_{pf} - GC_{pi})$

Figure 6-11 - External Pressure Coefficients, GC_p

Loads on Components and Cladding for Buildings w/ $H_t \leq 60$ ft



Note: The image shows a Gabled roof, but Fig 6-11 also applies to some monoslope cases

$a = 3.6 \implies 3.60 \text{ ft}$

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft ²)	Zone	GCp		Wind Press (lb/ft ²)	
					Max	Min	Max	Min
Wall	1	11	40.33	4	0.80	-0.89	59.12	-64.53
Wall	1	11	40.33	5	0.80	-1.07	59.12	-74.98
			0.00					
Wall	1	10	33.33	4	0.82	-0.91	59.92	-65.32
Wall	1	10	33.33	5	0.82	-1.09	59.92	-76.56
			0.00					
Roof	10	1	10.00	1	0.30	-1.00	28.85	-70.92
Roof	10	1	10.00	2	0.30	-1.80	28.85	-119.00
Roof	10	1	10.00	3	0.30	-2.80	28.85	-179.10
			0.00					
OH	10	1	10.00	1H	0.30	-1.70	18.03	-102.17
OH	10	1	10.00	2H	0.30	-1.70	18.03	-102.17
OH	10	1	10.00	3H	0.30	-2.80	18.03	-168.28
			0.00					
			0.00					
			0.00					
			0.00					
			0.00					
			0.00					

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WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

			0.00				
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Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

Description: **Miami Gardens T.** Company Name: **J.C.D.**
 Analysis by: **O.G.**

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	175	mph
Struc Category (I, II, III, or IV)	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.3	:12
Slope of Roof (Theta)	1.2	Deg
Type of Roof	Monoslope	
Kd (Directionality Factor)	0.85	
Eave Height (Eht)	19.83	ft
Ridge Height (Rht)	20.17	ft
Mean Roof Height (Ht)	20.00	ft
Width Perp. To Wind Dir (B)	36.00	ft
Width Paral. To Wind Dir (L)	164.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.56
Flexible Structure	No

Calculated Parameters		
<i>Hurricane Prone Region (V>100 mph)</i>		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Gust Factor Category I: Rigid Structures - Simplified Method		
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85
Gust Factor Category II: Rigid Structures - Complete Analysis		
Zm	Zmin	15.00 ft
Izm	$Cc * (33/z)^{0.167}$	0.2281
Lzm	$l * (zm/33)^{Epsilon}$	427.06 ft
Q	$(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^{0.63}))^{0.5}$	0.9225
Gust2	$0.925 * ((1+1.7*Izm*3.4*Q)/(1+1.7*3.4*Izm))$	0.8842
Gust Factor Summary		
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	0.18	-0.18

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WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev ft	Kz	Kzt	qz lb/ft ²	Pressure (lb/ft ²)					Shear (Kip)	Moment (Kip-ft)
				Windward Wall*		Leeward Wall		Total		
				+GCpi	-GCpi	+GCpi	-GCpi	+/-GCpi		
20.17	0.90	1.00	60.21	30.12	51.76	-21.04	0.60	51.16	0.31	0.03
20	0.90	1.00	60.10	30.05	51.69	-21.04	0.60	51.09	0.63	0.08
19.83	0.90	1.00	59.99	29.98	51.61	-21.04	0.60	51.01	9.50	21.55
15	0.85	1.00	56.57	27.65	49.29	-21.04	0.60	48.68	35.79	384.10

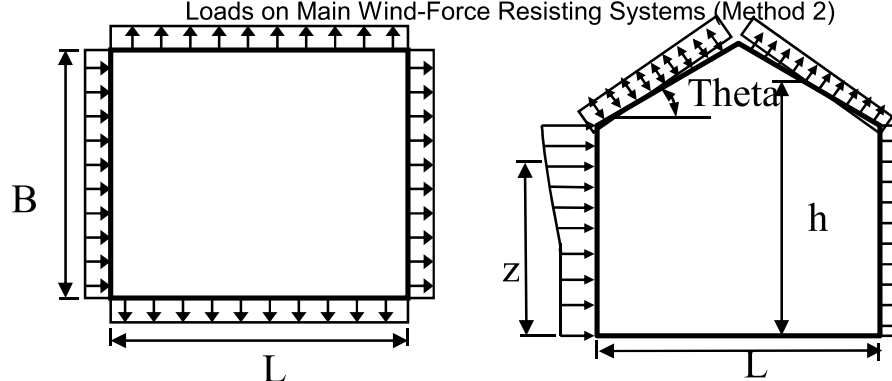
Note: 1) Positive forces act toward the face and Negative forces act away from the face.

Figure 6.5.12.2.4 Parapets

Type of Parapet	Enclosed	
Top Elevation of Parapet	22.00	ft
Parapet-Windward Pressure	110.38	lb/ft ²
Parapet-Leeward Pressure	-67.45	lb/ft ²

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	$2.01 \cdot (Ht/zg)^{(2/\alpha)}$	0.90	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	$.00256 \cdot (V)^2 \cdot Kh \cdot Kht \cdot Kd$	60.10	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.90	
Qhcc	$.00256 \cdot V^2 \cdot Khcc \cdot Kht \cdot Kd$	60.10	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00
Calculations for Wind Normal to 36 ft Face	Cp Pressure (psf)

WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

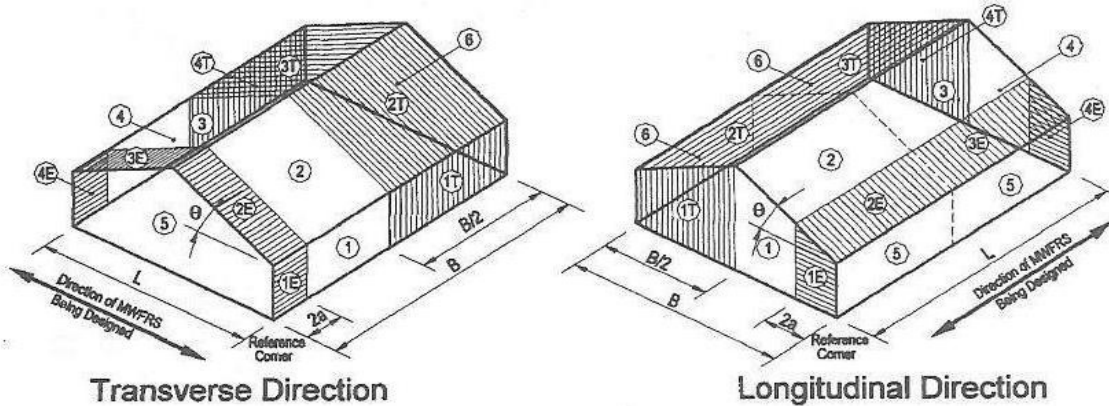
Additional Runs may be req'd for other wind directions		+GCpi	-GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.20	-21.04	0.60
Leeward Walls (Wind Dir Normal to 164 ft wall)	-0.50	-36.36	-14.72
Side Walls	-0.70	-46.58	-24.94
Overhang Bottom (Applicable on Windward only)	0.80	40.80	40.80
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 40 ft - Max Cp	-0.18	-20.01	1.62
Dist from Windward Edge: 0 ft to 10 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 20 ft to 36 ft - Min Cp	-0.50	-36.36	-14.72
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 164 ft face			
Dist from Windward Edge: 0 ft to 40 ft - Max Cp	-0.18	-20.01	1.62
Dist from Windward Edge: 0 ft to 10 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.88	-55.66	-34.02
Dist from Windward Edge: 20 ft to 36 ft - Min Cp	-0.52	-37.50	-15.86

* Horizontal distance from windward edge

Figure 6-10 - External Pressure Coefficients, GCpf

Loads on Main Wind-Force Resisting Systems w/ Ht <= 60 ft

Kh =	2.01*(Ht/zg)^(2/Alpha)	=	0.90
Kht =	Topographic factor (Fig 6-2)	=	1.00
Qh =	0.00256*(V)^2*Kh*Kht*Kd	=	60.10
Theta =	Angle of Roof	=	1.2 Deg



Torsional Load Cases

Wind Pressures on Main Wind Force Resisting System						
Surface	GCpf	+GCpi	-GCpi	qh (psf)	Min P (psf)	Max P (psf)
1	0.40	0.18	-0.18	60.10	13.22	34.86
2	0.40	0.18	-0.18	60.10	13.22	34.86
3	-0.37	0.18	-0.18	60.10	-33.06	-11.42
4	-0.29	0.18	-0.18	60.10	-28.25	-6.61

WIND10 v3-20

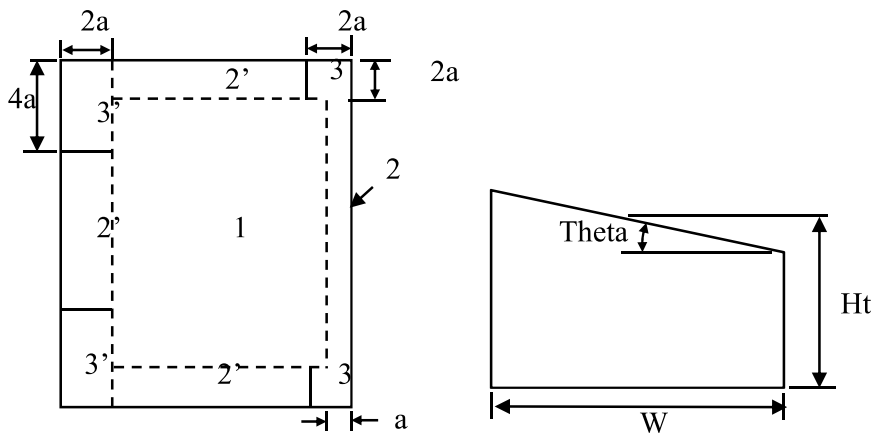
Detailed Wind Load Design (Method 2) per ASCE 7-10

5	-0.45	0.18	-0.18	60.10	-37.86	-16.23
6	-0.45	0.18	-0.18	60.10	-37.86	-16.23
1E	0.61	0.18	-0.18	60.10	25.84	47.48
2E	-1.07	0.18	-0.18	60.10	-75.13	-53.49
3E	-0.53	0.18	-0.18	60.10	-42.67	-21.04
4E	-0.43	0.18	-0.18	60.10	-36.66	-15.03

$$* p = qh * (GCpf - GCpi)$$

Figure 6-11 - External Pressure Coefficients, GCp

Loads on Components and Cladding for Buildings w/ Ht ≤ 60 ft



Note: The image shows a Gabled roof, but Fig 6-11 also applies to some monoslope cases

$$a = 3.6 \implies 3.60 \text{ ft}$$

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft ²)	Zone	GCp		Wind Press (lb/ft ²)	
					Max	Min	Max	Min
Wall	1	11	40.33	4	0.80	-0.89	59.12	-64.53
Wall	1	11	40.33	5	0.80	-1.07	59.12	-74.98
			0.00					
Wall	1	10	33.33	4	0.82	-0.91	59.92	-65.32
Wall	1	10	33.33	5	0.82	-1.09	59.92	-76.56
			0.00					
Roof	10	1	10.00	1	0.30	-1.00	28.85	-70.92
Roof	10	1	10.00	2	0.30	-1.80	28.85	-119.00
Roof	10	1	10.00	3	0.30	-2.80	28.85	-179.10
			0.00					
OH	10	1	10.00	1H	0.30	-1.70	18.03	-102.17
OH	10	1	10.00	2H	0.30	-1.70	18.03	-102.17
OH	10	1	10.00	3H	0.30	-2.80	18.03	-168.28
			0.00					
			0.00					
			0.00					
			0.00					
			0.00					
			0.00					

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WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

			0.00				
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Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

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WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

Description: **Miami Gardens T.** Company Name: **J.C.D.**
 Analysis by: **O.G.**

User Input Data		
Structure Type	Building	
Basic Wind Speed (V)	175	mph
Struc Category (I, II, III, or IV)	II	
Exposure (B, C, or D)	C	
Struc Nat Frequency (n1)	1	Hz
Slope of Roof	0.3	:12
Slope of Roof (Theta)	1.2	Deg
Type of Roof	Monoslope	
Kd (Directionality Factor)	0.85	
Eave Height (Eht)	19.83	ft
Ridge Height (Rht)	20.17	ft
Mean Roof Height (Ht)	20.00	ft
Width Perp. To Wind Dir (B)	36.00	ft
Width Paral. To Wind Dir (L)	141.00	ft

Calculated Parameters	
Type of Structure	
Height/Least Horizontal Dim	0.56
Flexible Structure	No

Calculated Parameters		
<i>Hurricane Prone Region (V>100 mph)</i>		
Table 6-2 Values		
Alpha =	9.500	
zg =	900.000	
At =	0.105	
Bt =	1.000	
Bm =	0.650	
Cc =	0.200	
l =	500.00	ft
Epsilon =	0.200	
Zmin =	15.00	ft

Gust Factor Category I: Rigid Structures - Simplified Method			
Gust1	For rigid structures (Nat Freq > 1 Hz) use 0.85	0.85	
Gust Factor Category II: Rigid Structures - Complete Analysis			
Zm	Zmin	15.00	ft
Izm	$Cc * (33/z)^{0.167}$	0.2281	
Lzm	$l * (zm/33)^{Epsilon}$	427.06	ft
Q	$(1/(1+0.63*((Min(B,L)+Ht)/Lzm)^{0.63}))^{0.5}$	0.9225	
Gust2	$0.925 * ((1+1.7*Izm*3.4*Q)/(1+1.7*3.4*Izm))$	0.8842	
Gust Factor Summary			
G	Since this is not a flexible structure the lessor of Gust1 or Gust2 are used	0.85	

Fig 6-5 Internal Pressure Coefficients for Buildings, Gcpi

Condition	Gcpi	
	Max +	Max -
Open Buildings	0.00	0.00
Partially Enclosed Buildings	0.55	-0.55
Enclosed Buildings	0.18	-0.18
Enclosed Buildings	0.18	-0.18

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WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

6.5.12.2.1 Design Wind Pressure - Buildings of All Heights

Elev ft	Kz	Kzt	qz lb/ft ²	Pressure (lb/ft ²)					Shear (Kip)	Moment (Kip-ft)
				Windward Wall*		Leeward Wall		Total		
				+GCpi	-GCpi	+GCpi	-GCpi	+/-Gcpi		
20.17	0.90	1.00	60.21	30.12	51.76	-21.25	0.39	51.37	0.31	0.03
20	0.90	1.00	60.10	30.05	51.69	-21.25	0.39	51.30	0.63	0.08
19.83	0.90	1.00	59.99	29.98	51.61	-21.25	0.39	51.23	9.54	21.64
15	0.85	1.00	56.57	27.65	49.29	-21.25	0.39	48.90	35.94	385.74

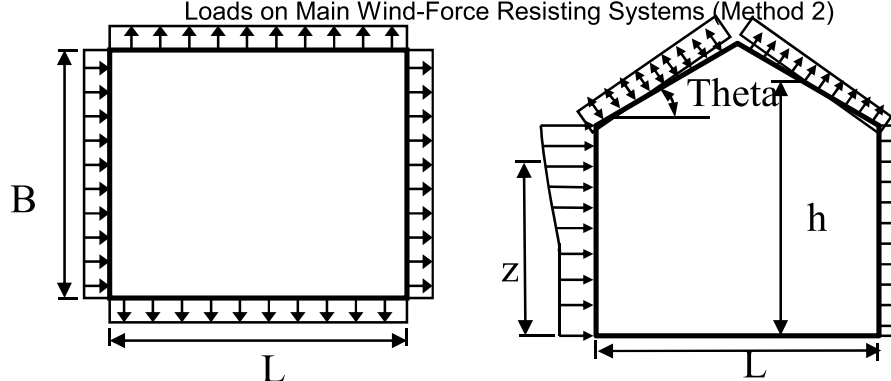
Note: 1) Positive forces act toward the face and Negative forces act away from the face.

Figure 6.5.12.2.4 Parapets

Type of Parapet	Enclosed	
Top Elevation of Parapet	22.00	ft
Parapet-Windward Pressure	110.38	lb/ft ²
Parapet-Leeward Pressure	-67.45	lb/ft ²

Figure 6-6 - External Pressure Coefficients, Cp

Loads on Main Wind-Force Resisting Systems (Method 2)



Variable	Formula	Value	Units
Kh	$2.01 \cdot (Ht/zg)^{2/\alpha}$	0.90	
Kht	Topographic factor (Fig 6-4)	1.00	
Qh	$.00256 \cdot (V)^2 \cdot Kh \cdot Kht \cdot Kd$	60.10	psf
Khcc	Comp & Clad: Table 6-3 Case 1	0.90	
Qhcc	$.00256 \cdot V^2 \cdot Khcc \cdot Kht \cdot Kd$	60.10	psf

Wall Pressure Coefficients, Cp	
Surface	Cp
Windward Wall (See Figure 6.5.12.2.1 for Pressures)	0.8

Roof Pressure Coefficients, Cp	
Roof Area (sq. ft.)	-
Reduction Factor	1.00
Calculations for Wind Normal to 36 ft Face	Cp Pressure (psf)

WIND10 v3-20

Detailed Wind Load Design (Method 2) per ASCE 7-10

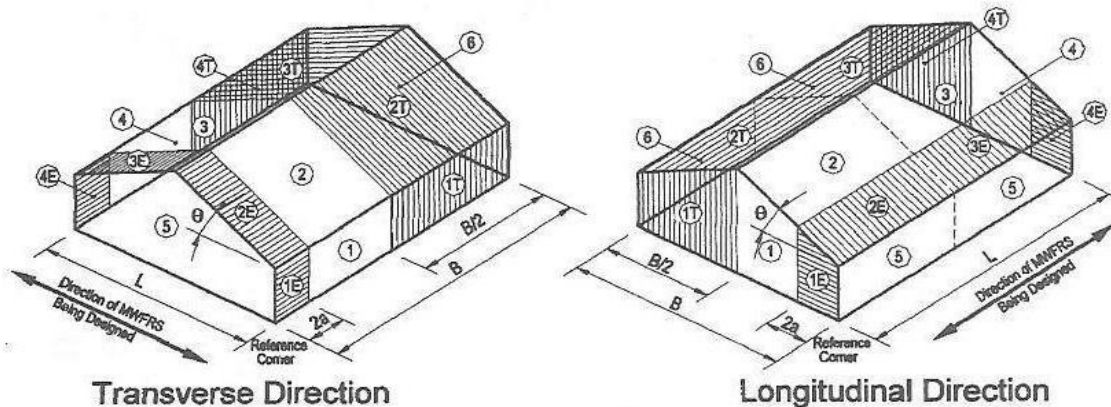
Additional Runs may be req'd for other wind directions			
		+GCpi	-GCpi
Leeward Walls (Wind Dir Normal to 36 ft wall)	-0.20	-21.25	0.39
Leeward Walls (Wind Dir Normal to 141 ft wall)	-0.50	-36.36	-14.72
Side Walls	-0.70	-46.58	-24.94
Overhang Bottom (Applicable on Windward only)	0.80	40.80	40.80
Roof - Wind Normal to Ridge (Theta<10) - for Wind Normal to 36 ft face			
Dist from Windward Edge: 0 ft to 40 ft - Max Cp	-0.18	-20.01	1.62
Dist from Windward Edge: 0 ft to 10 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 20 ft to 36 ft - Min Cp	-0.50	-36.36	-14.72
Roof - Wind Parallel to Ridge (All Theta) - for Wind Normal to 141 ft face			
Dist from Windward Edge: 0 ft to 40 ft - Max Cp	-0.18	-20.01	1.62
Dist from Windward Edge: 0 ft to 10 ft - Min Cp	-0.90	-56.80	-35.16
Dist from Windward Edge: 10 ft to 20 ft - Min Cp	-0.88	-55.66	-34.02
Dist from Windward Edge: 20 ft to 36 ft - Min Cp	-0.52	-37.50	-15.86

* Horizontal distance from windward edge

Figure 6-10 - External Pressure Coefficients, GCpf

Loads on Main Wind-Force Resisting Systems w/ Ht <= 60 ft

Kh =	$2.01 \cdot (Ht/zg)^{2/\alpha}$	=	0.90
Kht =	Topographic factor (Fig 6-2)	=	1.00
Qh =	$0.00256 \cdot (V)^2 \cdot Kh \cdot Kht \cdot Kd$	=	60.10
Theta =	Angle of Roof	=	1.2 Deg



Torsional Load Cases

Wind Pressures on Main Wind Force Resisting System						
Surface	GCpf	+GCpi	-GCpi	qh (psf)	Min P (psf)	Max P (psf)
1	0.40	0.18	-0.18	60.10	13.22	34.86
2	0.40	0.18	-0.18	60.10	13.22	34.86
3	-0.37	0.18	-0.18	60.10	-33.06	-11.42
4	-0.29	0.18	-0.18	60.10	-28.25	-6.61

WIND10 v3-20

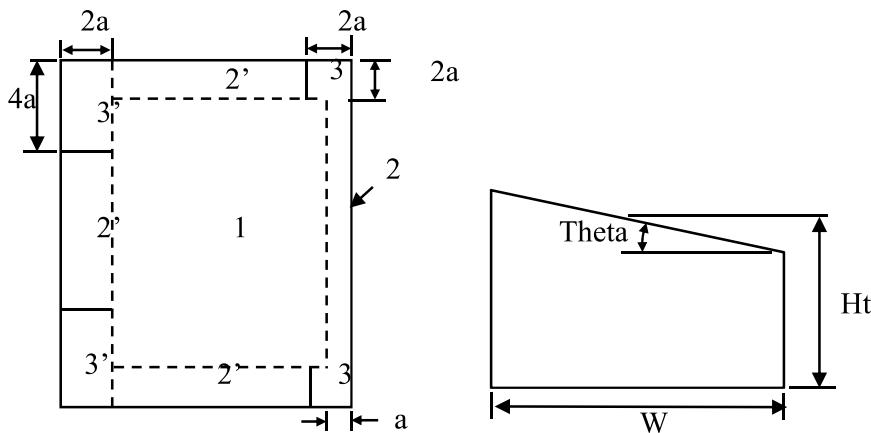
Detailed Wind Load Design (Method 2) per ASCE 7-10

5	-0.45	0.18	-0.18	60.10	-37.86	-16.23
6	-0.45	0.18	-0.18	60.10	-37.86	-16.23
1E	0.61	0.18	-0.18	60.10	25.84	47.48
2E	-1.07	0.18	-0.18	60.10	-75.13	-53.49
3E	-0.53	0.18	-0.18	60.10	-42.67	-21.04
4E	-0.43	0.18	-0.18	60.10	-36.66	-15.03

$$* p = qh * (GCpf - GCpi)$$

Figure 6-11 - External Pressure Coefficients, GCp

Loads on Components and Cladding for Buildings w/ Ht ≤ 60 ft



Note: The image shows a Gabled roof, but Fig 6-11 also applies to some monoslope cases

$$a = 3.6 \implies 3.60 \text{ ft}$$

Double Click on any data entry line to receive a help Screen

Component	Width (ft)	Span (ft)	Area (ft ²)	Zone	GCp		Wind Press (lb/ft ²)	
					Max	Min	Max	Min
Wall	1	11	40.33	4	0.80	-0.89	59.12	-64.53
Wall	1	11	40.33	5	0.80	-1.07	59.12	-74.98
			0.00					
Wall	1	10	33.33	4	0.82	-0.91	59.92	-65.32
Wall	1	10	33.33	5	0.82	-1.09	59.92	-76.56
			0.00					
Roof	10	1	10.00	1	0.30	-1.00	28.85	-70.92
Roof	10	1	10.00	2	0.30	-1.80	28.85	-119.00
Roof	10	1	10.00	3	0.30	-2.80	28.85	-179.10
			0.00					
OH	10	1	10.00	1H	0.30	-1.70	18.03	-102.17
OH	10	1	10.00	2H	0.30	-1.70	18.03	-102.17
OH	10	1	10.00	3H	0.30	-2.80	18.03	-168.28
			0.00					
			0.00					
			0.00					
			0.00					
			0.00					
			0.00					

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Detailed Wind Load Design (Method 2) per ASCE 7-10

			0.00				
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Note: * Enter Zone 1 through 5, or 1H through 3H for overhangs.

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DYNATECH ENGINEERING CORP.

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Miami, July 23, 2018

Mr. Berny Kopel
AFFORDABLE RESIDENTIAL 3400, LLC.
1930 Harrison Street, Suite 304
Hollywood, FL 33020

Re: The Village At Miami Gardens @
3400 NW 191st Street
Miami Gardens, FL

Dear Mr. Kopel:

Pursuant to your request, DYNATECH ENGINEERING CORP. (DEC) completed Percolation Tests on July 23, 2018 at the above referenced project. The purpose of our investigation was to help determine the hydraulic conductivity for storm drainage design.

The above hydraulic conductivity represents an ultimate value. The designer should decide on the required safety factor. This value is based on the existing soils at the location of the test. In the event the test location is changed or the soil removed and replaced; the test results will need to be re-evaluated.

Groundwater was measured immediately at the completion of each boring and was found at an average depth of approximately 6' below existing ground surface at the time of drilling. Existing ground surface elevation was not provided to us at the time of drilling. Design engineers must verify existing ground elevations as well as FEMA Flood and County highest and lowest groundwater elevation for their design. Fluctuation in water level **is anticipated** due to seasonal variations and run off as well as varying ground elevations, construction dewatering and pumping activities in the area and global warming. Site contractor must familiarize himself with site conditions in the event groundwater controls and dewatering is needed during construction. Surface flooding may result under hurricane conditions and should be taken into consideration in the design of the project. The contractor shall monitor and make sure that groundwater levels on adjacent properties are not adversely impacted due to the contractors dewatering activities. Specialty groundwater and water proofing contractors shall be consulted for all work below the groundwater level.

In case of existing structures, existing footings, new foundations and proposed drainage lines, provisions shall be made by the structural engineer, the civil engineer, and site contractor to protect all footings from future erosion, undermining and exposure. The geotechnical engineer shall be notified of these conditions to evaluate the applicability of his recommendations. The drainage system installation depth and dimension must be verified in the field during construction.

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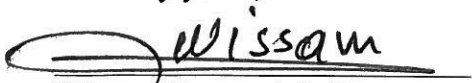
Re: 3400 NW 191st Street, Miami Gardens, FL

This report was prepared in compliance with the 2017 Florida Building Code, 6th edition. Site elevations were not provided to us for the test locations. Depths reported on the field boring logs represent the depth below existing ground surface as they existed on the date of drilling. In the event of subsequent filling, excavations or site work, the reported depths must be adjusted to represent proper depths.

The boring log (s) attached present (s) a detailed description of the soils encountered at test location (s). The soil stratification shown on the boring log (s) is based on the examination of the recovered soil samples and interpretation of the driller's field log (s). It indicates only the approximate boundaries between soil types. The actual transitions between adjacent soil types may be gradual. 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. In accepting and using this report the client understands and accepts that all data from the borings are strictly for foundation analysis only and are not to be used for excavation or back filling estimates and pricing. Owner and site contractor must familiarize themselves with site conditions prior to bidding. Client recognizes that actual conditions in areas not tested by DEC may differ from those anticipated in DEC's report. Client understands and accepts that this can significantly increase the cost of construction for its future projects. Client agrees that DEC shall not be responsible or liable for any variations in the actual conditions of areas not tested by DEC. This report is not a Phase I and/or Phase II Environmental Site Assessments. 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 scope of services performed in the execution of this investigation may not be appropriate to satisfy the needs of other users, and use or re-use of this document or the findings, conclusions, or recommendations is at the risk of said user. Said user must contact DEC in writing to verify applicability of this report for their use. All work must be conducted under the supervision of our geotechnical engineer. 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. All work shall be conducted in compliance with the Florida Building Code FBC and OSHA workers protection rules and all applicable Federal, State, County and City rules and regulations. In the event, changes, challenges and other value engineering opportunities occur without our knowledge, our recommendations may become compromised and geotechnical related issues may be misconstrued. Therefore, all geotechnical work shall be performed under our supervision to verify compliance with the intent of our recommendations.

It has been a pleasure working with you and look forward to do so in the near future.

Sincerely yours,



Wissam Naamani, P. E.

DYNATECH ENGINEERING CORP.

Florida Reg. No. 39584

Special Inspector No. 757

Certificate of Authorization No.: CA 5491

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PERCOLATION TEST

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PERCOLATION TEST ACCORDING TO S.F.W.M.D. D.O.T. STANDARD TEST

DATE : July 19, 2018
CLIENT : AFFORDABLE RESIDENTIAL 3400, LLC.
PROJECT : The Village at Miami Gardens @
PROJECT LOCATION : 3400 NW 191st Street, Miami Gardens, FL
LOCATION OF TEST : SAS
DIAMETER OF HOLE : 7"
TEST NO. : P-1

TEST DEPTH (feet) 0-15'
AVERAGE FLOW (GPM) 9.6
AVERAGE K (CFS/Sq. Ft-Ft Head) 3.55x10⁻⁴

* The above hydraulic conductivity represents an ultimate value. The designer should decide on the required safety factor. This value is based on the existing soils at the location of the test.

Water Table 6' Below existing ground surface.

SUBSURFACE INVESTIGATION

Depth Below Ground Surface

0'-0" to 0'-6"
0'-6" to 1'-0"
1'-0" to 3'-0"
3'-0" to 8'-0"
8'-0" to 15'-0"

Soil Description

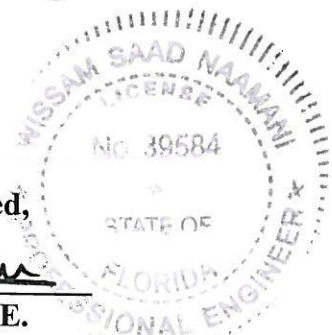
Light tan medium sand w/rock fragments
Brownish fine sand
Light tan medium sand
Brownish medium sand
Tanish medium sand

Field Tech: A.S.

Respectfully submitted,



Wissam Naamani, P. E.
DYNATECH ENGINEERING CORP.
Florida Reg. No. 39584
Certificate of Authorization No.: CA 5491



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PERCOLATION TEST ACCORDING TO S.F.W.M.D. D.O.T. STANDARD TEST

DATE : July 19, 2018
CLIENT : AFFORDABLE RESIDENTIAL 3400, LLC.
PROJECT : The Village at Miami Gardens @
PROJECT LOCATION : 3400 NW 191st Street, Miami Gardens, FL
LOCATION OF TEST : SAS
DIAMETER OF HOLE : 7"
TEST NO. : P-2

TEST DEPTH (feet) 0-15'
AVERAGE FLOW (GPM) 10.9
AVERAGE K (CFS/Sq. Ft-Ft Head) 4.04x10⁻⁴

* The above hydraulic conductivity represents an ultimate value. The designer should decide on the required safety factor. This value is based on the existing soils at the location of the test.

Water Table 6' Below existing ground surface.

SUBSURFACE INVESTIGATION

Depth Below Ground Surface

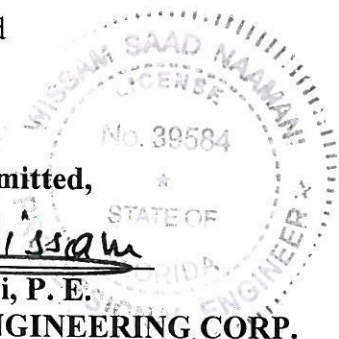
Soil Description

0'-0" to 0'-6"	Topsoil and grass
0'-6" to 1'-6"	Light brownish medium sand
1'-6" to 4'-0"	Whiteish medium sand
4'-0" to 5'-0"	Dark brown medium sand
5'-0" to 6'-0"	Brown medium sand
6'-0" to 15'-0"	Tan brown medium sand

Field Tech: A.S.

Respectfully submitted,


Wissam Naamani, P. E.
DYNATECH ENGINEERING CORP.
Florida Reg. No. 39584
Certificate of Authorization No.: CA 5491



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PERCOLATION TEST ACCORDING TO S.F.W.M.D. D.O.T. STANDARD TEST

DATE : July 19, 2018
CLIENT : AFFORDABLE RESIDENTIAL 3400, LLC.
PROJECT : The Village at Miami Gardens @
PROJECT LOCATION : 3400 NW 191st Street, Miami Gardens, FL
LOCATION OF TEST : SAS
DIAMETER OF HOLE : 7"
TEST NO. : P-3

TEST DEPTH (feet) 0-15'
AVERAGE FLOW (GPM) 10.7
AVERAGE K (CFS/Sq. Ft-Ft Head) 3.96x10⁻⁴

* The above hydraulic conductivity represents an ultimate value. The designer should decide on the required safety factor. This value is based on the existing soils at the location of the test.

Water Table 6' Below existing ground surface.

SUBSURFACE INVESTIGATION

Depth Below Ground Surface

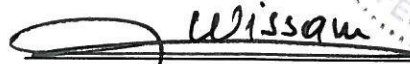
0'-0" to 0'-3"
0'-3" to 1'-6"
1'-6" to 3'-6"
3'-6" to 4'-6"
4'-6" to 15'-0"

Soil Description

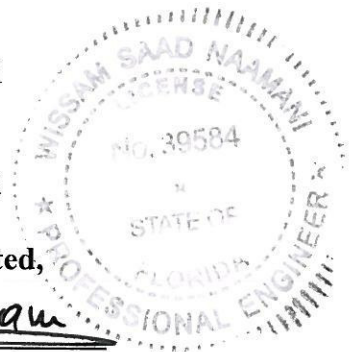
Topsoil and grass
Light brownish medium sand
Light tan medium sand
Brown medium sand
Light brownish medium sand

Field Tech: A.S.

Respectfully submitted,



Wissam Naamani, P. E.
DYNATECH ENGINEERING CORP.
Florida Reg. No. 39584
Certificate of Authorization No.: CA 5491



* As a mutual protection to the clients, the public and ourselves, all reports are submitted as the confidential property of clients, and authorization for publication of statement conclusions or extracts from or regarding our reports is reserved pending on our written

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SITE PLAN

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○ P-#: Percolation Test Location

N



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Client: Affordable Residential 3400, LLC.

Scale: N.T.S.

Project: 1400 Nations Dr, Miami Gardens, FL

Date: July 23, 2018

GENERAL NOTES

Soil borings on unmarked vacant property should be considered preliminary with further boring(s) to be drilled after building pad(s) are staked out.

Soil borings on existing structures that are to be demolished should be considered preliminary and additional borings would need to be performed after the structure(s) has been demolished and proposed new building staked out.

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KEY CLASSIFICATION & SYMBOLS

Correlation of Penetration Resistance With Relative Density and Consistency

	<u>Cone Penetration Tests (Kg/cm³)</u>	<u>Standard Penetration (Blows/ft.)</u>	<u>Relative Density</u>
Sands	0-16	0-4	Very Loose
	17-40	5-10	Loose
	41-80	11-20	Firm
	81-120	21-30	Very Firm
	Over 120	31-50	Dense
Silts & Clay	0-3	0-2	Very Loose
	4-9	3-4	Soft
	10-17	5-8	Firm
	18-31	9-15	Stiff
	32-60	16-30	Very Stiff
	Over 60	31-50	Hard

Particle Size

Boulders	> 12 in.
Cobble	3 in. to 1 in.
Gravel	4.76 mm to 3 in.
Sand	0.07 mm to 4.67 mm.
Silt	0.005 mm. to 0.074 mm
Clay	< 0.005 mm

Modifiers

5% - 10 %	Slightly Silty or Clayey
10% - 30%	Silty or Clayey
30% - 50%	Very Silty or Very Clayey
0% - 5%	Slightly Trace
6% - 10%	Trace
11% - 20%	Little
21% - 35%	Some
> 35% And	

Rock Hardness Description

Soft	Rock core crumbles when handled
Medium	Can break with your hands.
Moderate Hard	Thin edges or rock core can be broken with fingers
Hard	Thin edges or rock core cannot be broken with fingers
Very Hard	Rock core rings when struck with hammer (cherts)

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