

STRUCTURAL CONDITION ASSESSMENT
1800 Michigan Ave
Miami Beach, Florida

Prepared for

J. Luis Quintana

February 14, 2025

PREPARED BY



Youssef Hachem Consulting Engineering

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Date: 2025.02.18 07:17:08-05'00'

TABLE OF CONTENTS

I. Introduction	Page 3
II. Methodology	Page 4
III. Structural system	Page 5
IV. Site Observations	Page 6
V. Structural Evaluation	Page 13
VI. Recommendations	Page 17
Appendix A - Photos	
Appendix B – Concrete Testing Report	
Appendix C – Calculations	
Appendix D - Survey	

STRUCTURAL CONDITION ASSESSMENT for
1800 Michigan Ave
Miami Beach, Florida

I. INTRODUCTION

General

Per the request of Mr. Quintana, we have conducted a visual and structural condition assessment on the existing structure located at 1800 Michigan Avenue in Miami Beach, Florida.

The purpose of the inspections was to assess the structural condition of the property to determine the feasibility of preservation and lifting of the structure to comply with current building code requirements and safe habitability.

Structural System

The Structure is a two-story masonry building. The Building Structural System is as follows:

- First Floor:
 - Elevated wood floor framing, with wood
 - Exterior 3 cell concrete masonry unit walls
 - Interior wood load bearing stud walls
- Second Floor:
 - Wood floor framing, with wood planking
 - Exterior 3 cell concrete masonry unit walls
 - Interior wood load bearing stud walls

The components and cladding of the house, such as doors, windows and roof waterproofing are not addressed in this report. Moreover, Mr. Quintana should perform termite and asbestos testing on the building. The electrical and electrical systems are not part of this report, but essentially are non-existent in the building.

II. METHODOLOGY

This inspection was visual in nature from the exterior and interior of the building. Our office did not perform any destructive or non-destructive testing, however Mr. Quintana engaged a licensed material testing company, of their choosing to perform concrete core samples to test for:

- 1- Concrete compressive strength
- 2- Extent of Carbonation

Currently, there are several locations in the building that have decayed wood framing which made a full inspection in parts of the building challenging. Every attempt was made to access all portions of the building to observe all signs of distress in the structural members of the building, which includes masonry, wood, and concrete. Distress signs are cracking, spalling, water damage, and termite damage.

A structural analysis was performed on the building to determine the capacity of the structural systems. It is our opinion that the current structural system of the building does not comply Florida Building Code 2023, HVHZ (High Velocity Hurricane Zone) edition.

III. STRUCTURAL SYSTEMS

Based on Miami Dade County tax records, the structure was built in 1935 with an area of 4,092 square feet. The building is approximately 54 feet long (East-West direction) by 40 feet wide (North-South direction). The building's structural members are as follows:

Foundations: The building is built on shallow foundations about 24" wide x 12" thick. The foundations support a concrete stem walls (interior and exterior). The interior stem walls support the interior wood stud walls and the exterior stem walls support the exterior masonry walls.

Exterior Walls: The exterior walls of the building are made up of 3 cell concrete masonry unit ("CMU") block, which were common construction material in 1935. The walls have a 5/8" stucco smooth finish and rough finish.

Interior Walls: There are two types of interior walls, load bearing and non-load bearing. Both types are wood 2"x4" stud walls. The load bearing walls support the floor joists system extending from the exterior walls. These stud walls are in turn supported by the concrete stem walls and foundations.

Floors: The flooring system is typical on all floors. The wood floor joists are 2"x10" spaced at 16" on center and spanning North-South from the exterior Wood wall over the interior load bearing wood stud walls (running North-South). The joists system is supporting 1"x 6" wood planks make up the 1st and 2nd floor system. All wood joists are "Fire Cut" into the Wood wall, meaning the wood joists are resting in openings in the Wood wall and are not connected to the walls via strapping or any other mechanism.

Roof: Typical of 1935 construction, the actual roof deck is 2"x8" wood joists supporting 1"x6" wood planks. The roof deck is supported by wood knee wall made up of 2"x4" vertical studs. The knee wall in turn is supported by 2"x8" wood joists. The knee wall system is used to slope the actual roof deck for stormwater drainage.

IV. SITE OBSERVATIONS

We have inspected the structure on several occasions, and our summary of the evaluation of the existing conditions of the structural components are as follows:

Wood members: The roof of the structure has failed in multiple locations, and the moisture intrusion had caused severe and extensive damage to all the wood members of the building (please see photos below). There is moisture damage (rot) of wood, that has caused wood members to deflect, sag, fail, and multiple areas of total collapse. The wood members collapse in the building have created hazardous conditions within the building. The fact that the building has been vacant for some time now, and the moisture intrusion from the roof, door, and window openings had created an atmosphere for the wood to deteriorate severely.

Concrete: The concrete spalling and cracking is evident throughout the building (please see photos below). **Concrete columns and beams exhibit concrete spalling that is estimated at 40% of the area.** Stucco cracking is also evident throughout the building. Previous repairs are also present that exhibit failure and re-cracking.

Interior walls and Ceilings: The components and cladding elements of the building and accessories such as doors, windows, louvers, rails, are all in poor condition. Moreover, the roof waterproofing membrane is also in a poor condition (please see photos below). There are various areas with mold and water intrusion present, all exhibit varying levels of failure. Many areas have rotten wood present due to the water intrusion.

Based on the extent of damage and original construction, the structure would require extensive interior and exterior demolition and would not withstand the necessary shoring and lifting portions of the current building to today's standards.



Mold is present on the interior finishes



Mold and water intrusion is present



Skylights present show water intrusion.



Failures noted in the exterior wood canopy



Rotted wood is noticed on the wood joist that make up the second floor of the property



Water intrusion is present throughout the entire property



Water intrusion is present throughout the entire property

V. STRUCTURAL EVALUATION

There are several factors to be considered in the structural evaluation of this building.

Initial Construction:

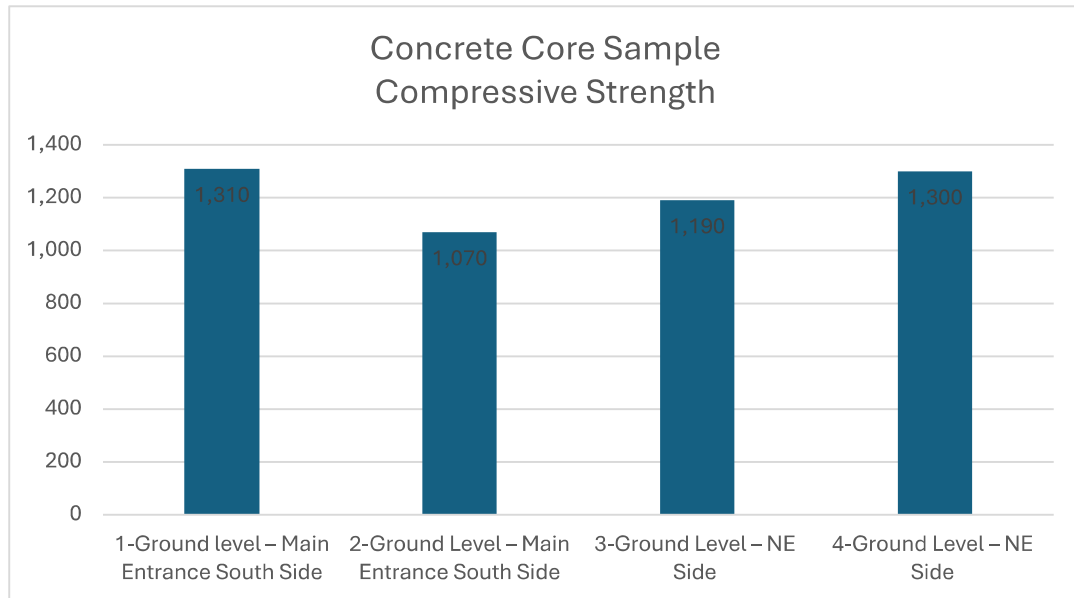
Building construction and standards of the 1930's are considered deficient in today's standards. This applies to this structure and other structures built in the 1930's. This building under current building code is deemed deficient. The structure's roof connections for wind uplift forces, and for wind lateral resistance are non-existent. Moreover, openings protection, and wood reinforcing is also non-existent. To rehabilitate this building, it has to undergo level III alteration of the Florida Building Code 2023 for existing structures. This means that the building has to be strengthened to comply with the current Florida Building Code. Which means that the roof connection tie downs have to be implemented to strengthen the roof, and lateral load structural systems have to be installed such as shearwalls. Wall openings such as doors and windows and the exterior wood walls have to be reinforced. Hence, the foundations also have to be strengthened to resist such lateral loads.

Concrete Testing Results:

Ownership engaged NV5, Inc. to conduct concrete laboratory testing on the building to obtain compressive strength, and carbonation depth. The laboratory extracted three (4) concrete core samples of sizes 3.00 in diameter by 6.00 in length approximately, which also were used to test for carbonation.

-Concrete compressive: the results of the testing for concrete strength are tabulated and charted as follows:

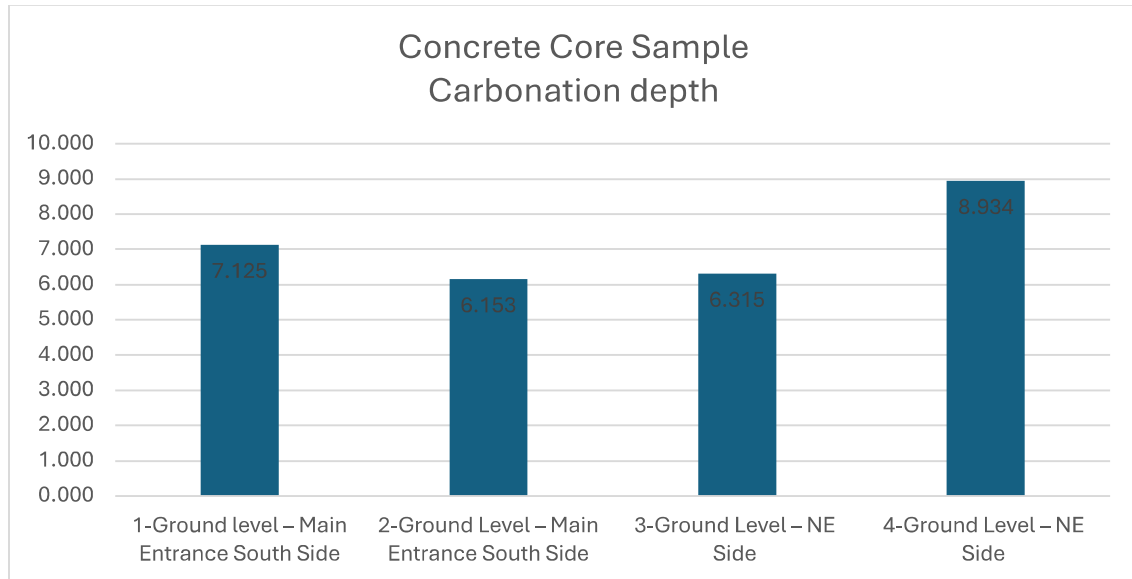
Core	Compressive Strength
Number	[PSI]
1-Ground level – Main Entrance South Side	1,310
2-Ground Level – Main Entrance South Side	1,070
3-Ground Level – NE Side	1,190
4-Ground Level – NE Side	1,300



The Concrete compressive strength ranged from 1,070 to 1,310 PSI. Per Florida Building Code the concrete strength should be 5,000 PSI.

-Carbonation depth: Carbon dioxide from air reacts with the calcium hydroxide in concrete to form calcium carbonate, this process is called carbonation. Carbonation, naturally starts from the exterior surface and progresses inwards. Carbonation actually increases the compressive strength of concrete; however, it also decreases alkalinity, which is essential for corrosion prevention of the reinforcement steel. The results of the testing for carbonation depth are tabulated and charted as follows:

Core	Carbonation depth
Number	[in]
1-Ground level – Main Entrance South Side	7.125
2-Ground Level – Main Entrance South Side	6.153
3-Ground Level – NE Side	6.315
4-Ground Level – NE Side	8.934



1

The carbonation found in the samples ranged between 6.153" – 8.934"

The carbonation is extensive and exposes the reinforcing rebars to corrosion

Site Conditions:

Based on the visual observation in the field, all the wood members of the building such as the roof, floor joists on all floors, and interior stud walls are in very poor and failing condition. Moreover, reinforcing rebars of the concrete members also are in poor condition.

Floor Elevation:

The First finish floor elevation is at 3.14' NAVD (1988), which is approximately 4.64' NGVD. Flood Elevation by FEMA flood maps is at 8.00' NGVD. Hence, the house is below flood. New construction is built at 9.00' NGVD (flood elevation + 1' flood freeboard).

Appendix C shows the wind loads applied to the house per the Florida Building Code, and also analyzes the roof beams of the house which are 4"x6" wood beams spaced at 30" o/c. The analysis shows that the roof beams will fail. When the wind loads applied on them, moreover, the roof beams also do not comply with the Florida Building Code in normal, non-hurricane conditions.

VI. RECOMMENDATIONS

Based on the site observations of the conditions of structural members of the building and level III alteration required by the Florida Building Code, the structural members of this replaced rather than repaired. Hence, in order to do so, these structural members need to be demolished.

It is evident that portions of the structure were built illegally and without permits when built, they were not built up to standards to support loading conditions.

The structure is in moderate to bad condition, leading to deficient structural conditions. The structural members which are mainly wood are deteriorated, moisture damaged and rotting. Most of the structural members cannot be replaced.

The structure is well below flood elevation and to raise the house to comply with FEMA flood rules, the house has to be lifted mechanically. This feat cannot be guaranteed successful based on the deteriorated and damaged structural members of the building.

Based on the concrete testing which averaged 1,217 PSI concrete compressive strength (new construction to comply with current building code concrete is designed for 5,000 PSI) shows that the lifting process will cause serious damage to the foundations.

Furthermore, the carbonation extends deep into the concrete (more than the 2" cover) indicates that all the reinforcing rebars have lost their alkaline protection layer and are exposed to corrosion.

Even if the roof members were in conditions, they do not comply with the requirements of the Florida Building Code. Therefore, the entire roof structure will require demolition and reconstruction at the new required elevation.

We are not confident that the replacement process will not damage the structure, even furthermore due to the connectivity between the members.

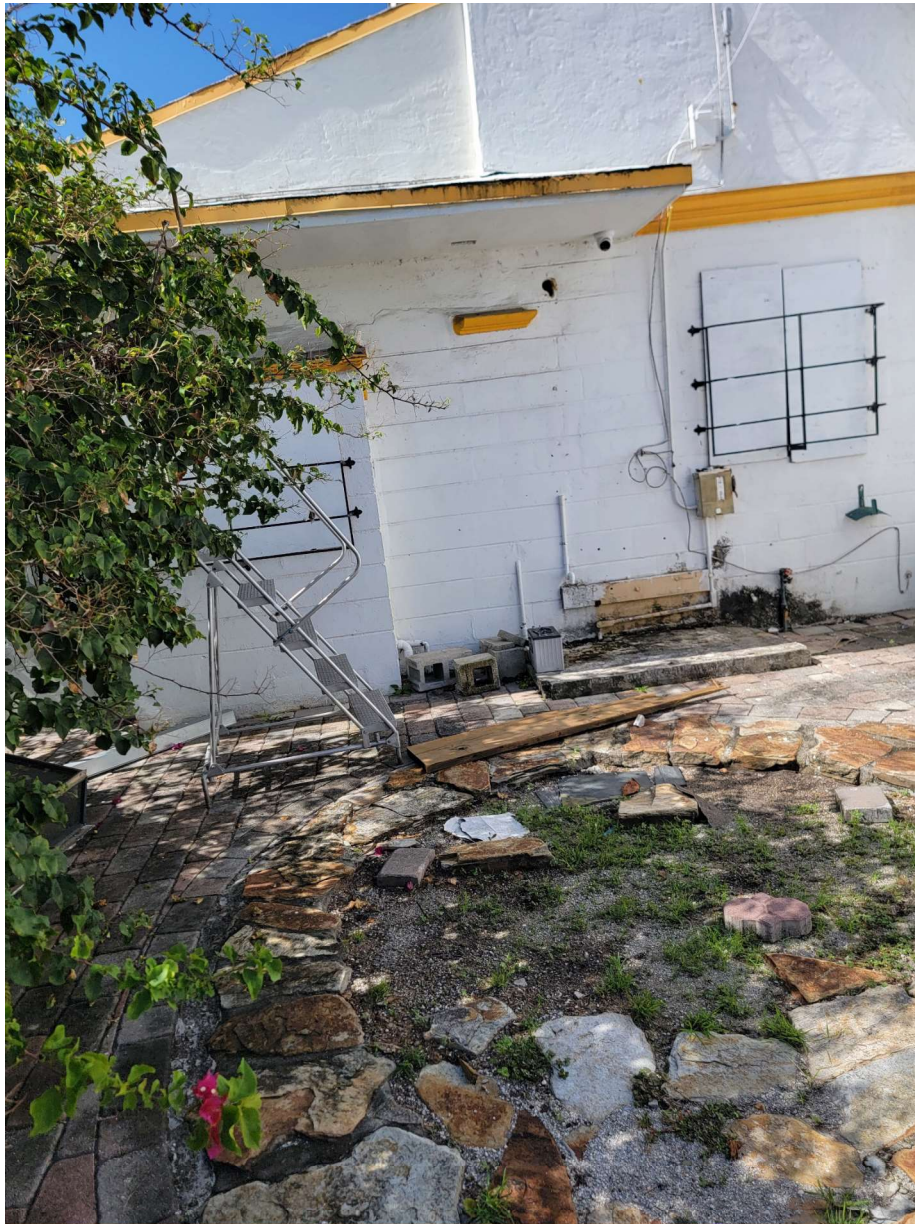
Structure does not comply with today's building code, and even when certain parts of it were built. The materials of the structural systems are compromised and cannot support the loads imposed on it, we recommend demolition of the structure.

APPENDIX A

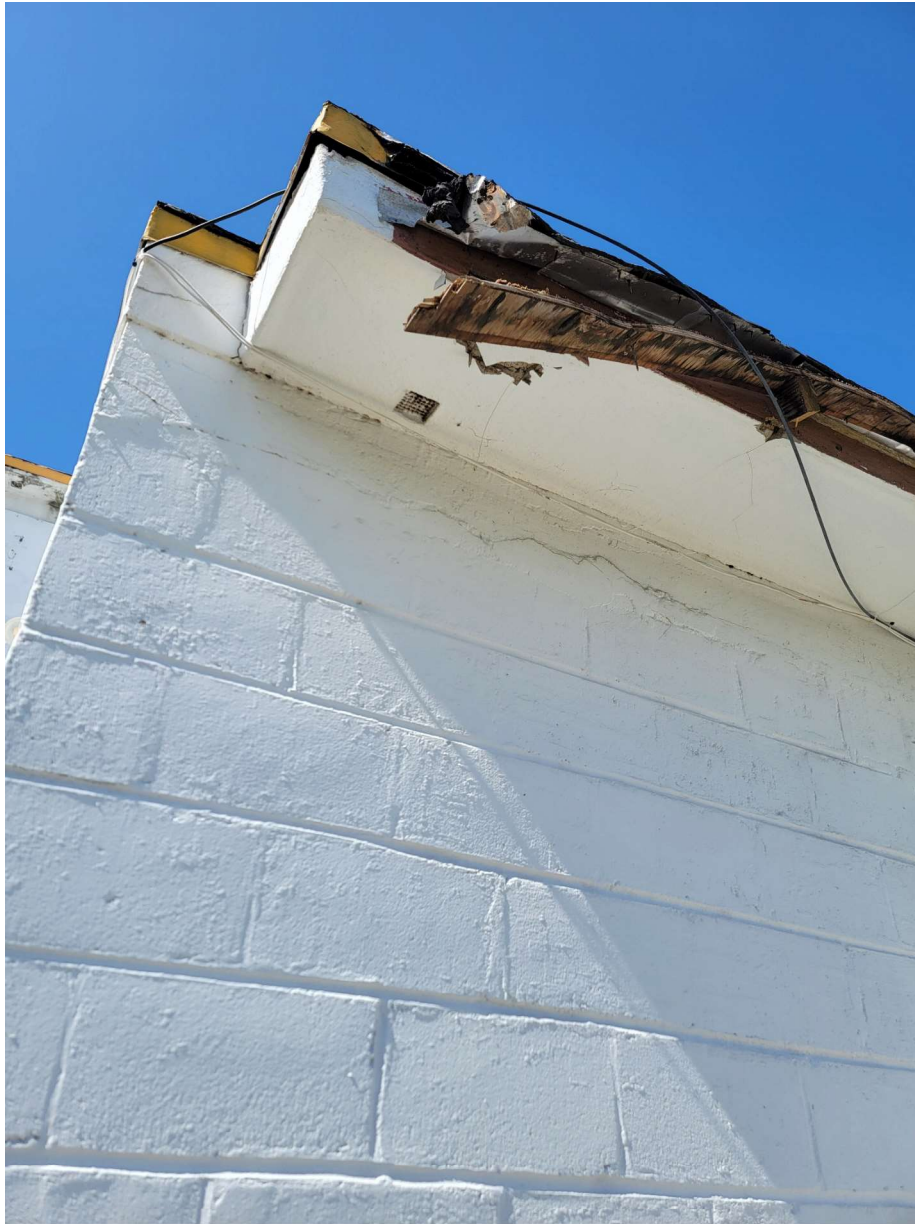
PHOTOS



Spall and delaminated stucco present



Spall and delaminated stucco present



Rotted wood present on fascia



Cracks noted leading to foundations



Termite damages noticed on wood floors



Water intrusion damages noted



Skylights failure noticed



Deflections noticed on foundations



Mold present in interior finishes due to water intrusion



Water intrusion thru windows



Water intrusion noticed coming from roof

Appendix B
Concrete testing report



January 28, 2025

IRRS 1800 Michigan, LLC

Attn: Ms. Emily Balter
7375 Collins Avenue
Sunny Isles Beach, Florida 33160

Re: Report of Concrete Core Extraction & Testing
1800 Michigan Ave Concrete Core Test
1800 Michigan Avenue
Miami Beach, Florida
NV5 Project No. 18715

Dear Ms. Balter:

NV5, Inc. submits this report in fulfillment of the scope of services described in our proposal 24-0977 dated December 10, 2024. This report describes our understanding of the project, presents our field and laboratory testing results.

This report should be read in its entirety.

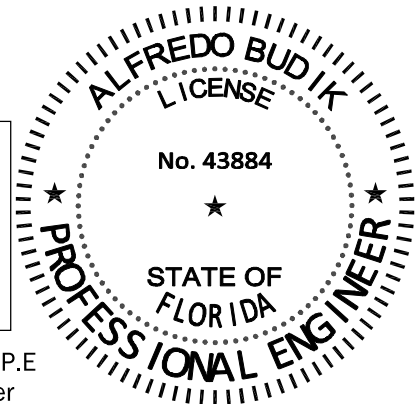
Sincerely,
NV5, Inc.

Leon R. Habr
Project Manager

This document has been digitally signed
and sealed by:

Printed copies of this document are not
considered signed and sealed, and the
signature must be verified on any
electronic copies

Alfredo Budik, P.E.
Senior Engineer
Florida License No. 43884



Distribution: 1 Copy to Addressee via Email
1 Copy to NV5 File

f:\doc\nv5 reports\18715_1800 michigan ave concrete core test_1800 michigan avenue_miami beach_florida_irrs 1800 michigan, llc_concrete core extraction and testing report_1-28-2025.doc

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
1.0 SITE AND PROJECT INFORMATION	1
2.0 PURPOSE AND SCOPE OF WORK	1
3.0 FIELD WORK	1
3.1 GPR (GROUND PENETRATING RADAR)	1
3.2 CONCRETE CORES EXTRACTION	1
4.0 LABORATORY TESTING RESULTS	2
4.1 CONCRETE CORES FOR COMPRESSIVE STRENGTH TESTING	2
4.2 CONCRETE CORES FOR CHEMICAL ANALYSIS (DEPTH OF CARBONATION)	2
5.0 REPORT LIMITATIONS	3
6.0 CLOSURE	3

APPENDICES

Appendix A	Concrete Core Locations (2 pages)
Appendix B	Concrete Core Compressive Strength Test Results (1 page)

1.0 SITE AND PROJECT INFORMATION

The project site is located at 1800 Michigan Avenue in Miami Beach, Florida. The site is bounded by a single-family home to the north, by 18th Street to the south, by Michigan Avenue to the east and by a vacant lot to the west. According to Miami-Dade Property Appraiser page the site corresponds to Folio No. 02-3234-004-0120. NV5 has been requested to perform concrete core compressive strength and carbonation tests. NV5 was informed that a total of six (6) cores will be extracted from various locations as specified by the structural engineer (YHCE) or other designated personnel. Please note that the dates for the concrete pour were not provided.

2.0 PURPOSE AND SCOPE OF WORK

The purpose of our services is to perform a Ground Penetrating Radar (GPR) scan, extract concrete cores and test the cores extracted in the laboratory for compressive strength and depth of carbonation.

3.0 FIELD WORK

3.1 GPR (Ground Penetrating Radar)

The selected locations of the south wall (Main entrance wall) and east wall were scanned with a ground penetrating radar (GPR) to determine the steel reinforcing configuration and to avoid damaging the reinforcement while performing the core extraction. Ground Penetrating Radar (GPR) is a geophysical locating method that uses radio waves to capture images below the surface of the ground or concrete in a non-destructive manner. A GSSI radar system with 350 Mega-hertz (MHz) antenna was used to identify possible steel reinforcement of the slab.

3.2 CONCRETE CORES EXTRACTION

Four (4) core samples were extracted by NV5 per client's request from the south wall (Main entrance wall) and east wall. The core sample locations were determined by the structural engineer (YHCE).

Concrete coring was performed in general accordance with ASTM C42-18, Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete. Immediately upon completion of the coring process, the core samples were bagged within the time limit indicated in ASTM C42-18 and transported to our laboratory for testing.

The core samples were used for compressive strength testing. The same cores were tested right after the compression test for depth of carbonation.

Concrete core locations are presented in Appendix A.



Table 1 – Core Locations and Tests Performed

Core Number	Core Location	Test Performed
1	South Wall	Compressive Strength- Depth of Carbonation
2	South Wall	Compressive Strength- Depth of Carbonation
3	East Wall	Compressive Strength- Depth of Carbonation
4	East Wall	Compressive Strength- Depth of Carbonation

Concrete core extraction was performed on January 17, 2025. The results and locations of the concrete core laboratory testing are summarized in Section 4.0.

Concrete cores respective lengths are shown in Appendix B of this report.

4.0 LABORATORY TESTING RESULTS

4.1 CONCRETE CORES FOR COMPRESSIVE STRENGTH TESTING

The compressive strength testing of the concrete core samples was performed in general accordance with ASTM C42-18. The core samples were trimmed and later subjected to compressive strength testing. The compressive strength of the concrete cores ranged between 1,070 and 1,310 pounds per square inch (psi).

Details of the compressive strength results of the core samples are presented in Appendix B.

4.2 CONCRETE CORES FOR CHEMICAL ANALYSIS (Depth of Carbonation)

Right after the compressive strength testing the same core samples were tested for chemical analysis (Depth of Carbonation). The depth of carbonation test was performed in accordance with ASTM C856, Standard Practice for Petrographic Examination of Hardened Concrete to determine the depth of carbonation. The concrete samples were freshly fractured to expose a clean surface, and a pH indicator solution (phenolphthalein) was applied to the exposed area. The indicator reacts to the pH levels within the concrete, turning pink in areas where the pH is above 9, indicating uncarbonated concrete, and remaining colorless in areas where the pH is below 9, indicating carbonation. The depth of carbonation was measured as the distance from the surface of the concrete to the boundary where the color change occurred. This method identified the extent of carbonation and provided data on the concrete's susceptibility to carbonation-related durability concerns.

The depth of carbonation ranged between approximately 6.153 and 8.934. A summary of the test results is summarized below in table 2 and illustrated in Figure 1 through 4.

Table 2 – Core Locations and Tests Performed

Core Number	Core Location	Depth of Carbonation (in)
1	South Wall	7.125
2	South Wall	6.153
3	East Wall	6.315
4	East Wall	8.934



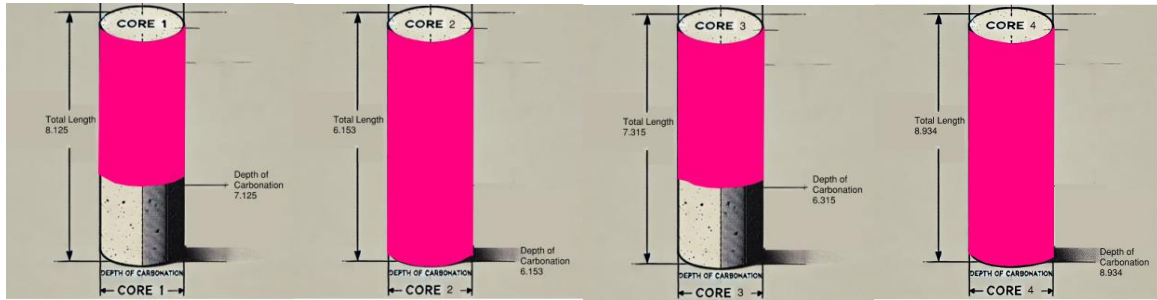


Figure 1- Depth of Carbonation (Core 1) Figure 2- Depth of Carbonation (Core 2) Figure 3- Depth of Carbonation (Core 3) Figure 4- Depth of Carbonation (Core 4)

5.0 REPORT LIMITATIONS

This report has been prepared pursuant to our proposal 24-0977 dated December 10, 2024. This report should be read in its entirety. NV5 is not responsible for misinterpretations arising from only reading sections of the report.

This report has been prepared for the exclusive use of the Owner and other members of the design/construction team for the specific site and project discussed in this report. This report is not applicable to any other site or project.

The tests were performed in general accordance with the procedures described above and the results presented in this report are representative of the in-situ conditions only at the specific locations tested. The structural engineer should evaluate these results accordingly.

6.0 CLOSURE

We appreciate the opportunity to provide specialized engineering services on this project and look forward to an opportunity to participate in construction related aspects of the development. If you have questions about the information contained in this report, contact the writer on 305.666.3563.

Appendix A

Concrete Core Locations



main entrance / South wall

13ft

Text

6ft

Core #1

8 inches above
ground

Existing holes

Core #2

8 inches above
ground

East wall located
north east corner

1800

2

Core #4



4ft

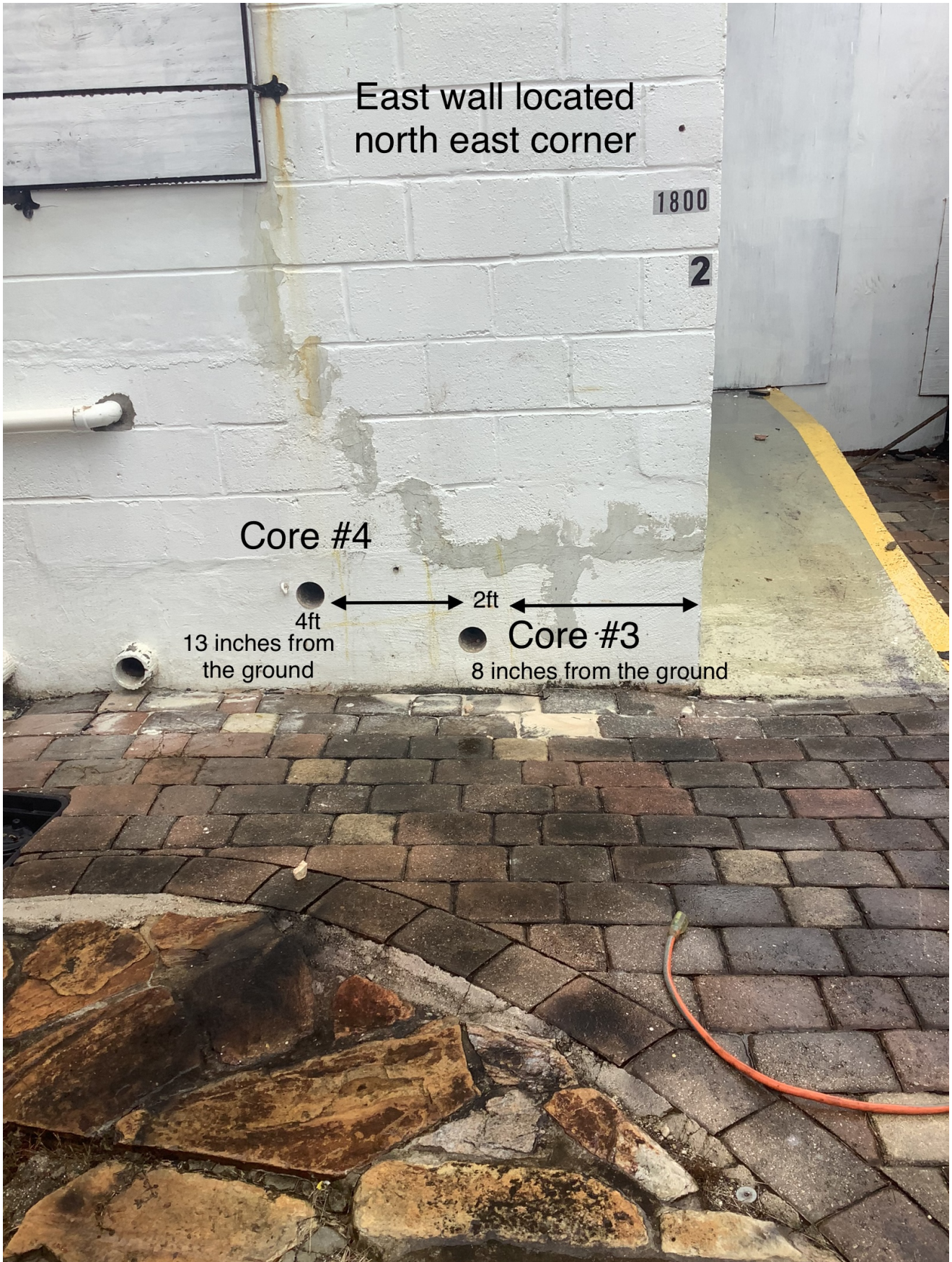
13 inches from
the ground

2ft



Core #3

8 inches from the ground



Appendix B

Concrete Core Compressive Strength



CORES COMPRESSIVE STRENGTH REPORT
NV5, INC.
14486 COMMERCE WAY, MIAMI LAKES FL 33016
TELEPHONE NO. 305-666-3563 FAX NO.: 305-666-3069

PROJECT NAME: 1800 Michigan Ave Concrete Core Test
CLIENT: IRRS 1800 Michigan, LLC
CONTRACTOR: YHCE
TEST METHOD: In general accordance with ASTM C42-20

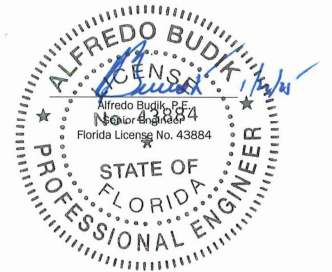
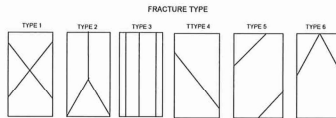
PROJECT NUMBER: 18/135
SAMPLE BY: NV5
SPECIFIED STRENGTH: Not Provided
CONCRETE SUPPLIER: Not Provided

DATE: 1/22/2025
SET NO.: 1
PAGE NO.: 1

Core	Core Location	Structural Element	Core Dimensions					Compressive Strength				Fracture Type	Maximum Nominal Aggregate Size	Pour Date	Core Date	Preparation Date	Test Date	Core Weight (lbs.)	Core Unit Weight (lbs./ft³)
			Diameter (inches)	Lengths			Cross Sectional Area (sq.inches)	Maximum Load (lbs.)	L/D	Correction Factor	Approx. Compressive Strength (psi)								
				Original (inches)	w/o cap (inches)	with cap (inches)													
1	Ground Level - Main Entrance South Side	slab	3.149	8.875	5.401	N/A	7.79	10443	1.72	0.98	1,310	3	# 57	N/A	1/17/2025	1/18/2025	1/22/2025	3.02	124.21
2	Ground Level - Main Entrance South Side	slab	3.145	6.153	4.904	N/A	7.77	8602	1.56	0.96	1,070	3	# 57	N/A	1/17/2025	1/18/2025	1/22/2025	2.70	122.29
3	Ground Level - Northeast Side of Residence	slab	3.149	7.315	4.221	N/A	7.79	9862	1.34	0.94	1,190	3	# 57	N/A	1/17/2025	1/18/2025	1/22/2025	2.39	125.52
4	Ground Level - Northeast Side of Residence	slab	3.145	8.934	4.810	N/A	7.77	10480	1.53	0.96	1,300	3	# 57	N/A	1/17/2025	1/18/2025	1/22/2025	2.71	125.39

Notes

- According to AGI 31.8 and Note 4 of ASTM C42, "The concrete represented by the cores is considered structurally adequate if the average strength of three cores is at least 85% of the specified strength and no single core strength is less than 75% of the specified strength". Compressive strength results should be reviewed by the Engineer of Record for acceptance.
- According to ASTM C42-20 -- "Allow the cores to remain in the sealed plastic bags or nonabsorbent containers for at least 5 days after last being wetted and before testing, unless stipulated otherwise by the specifier of tests".
- Direction of load application is Parallel and moisture condition is bagged.
- Due to tightly spaced reinforcing steel, the core diameter had to be reduced under the recommended diameter described in ASTM C42 with the approval of the structure Engineer
- All cores were trimmed and grinded prior to compressive strength testing



Appendix C
Calculations Roof Beams

MecaWind v2485

Developed by Meca Enterprises Inc., www.mecaenterprises.com, Copyright © 2025

Calculations Prepared by:

Date: Feb 13, 2025

File Location: U:\2024\MISC\H241230 (1800 Michigan)\DESIGN PHASE\Calculations\
WIND CALC'S\house.wnd

General:

Reference Abbreviations: T: Table, F: Figure, E: Equation, S: Section

Wind Load Standard	=	FBC 2023	Basic Wind Speed	=	175.0 mph
Exposure Classification	=	D	Risk Category	=	II
Structure Type	=	Building	Design Basis for Wind Pressures	=	ASD
MWFRS Analysis Method	=	None	C&C Analysis Method	=	Ch 30 Pt 1
Dynamic Type of Structure	=	Rigid	Show Advanced Options	=	False

Wind Speed Basis to be used in calculations:

V = Convert to ASD wind speed: $V_{ult} \cdot 0.6^{0.5}$ [Eqn 16-17] = 135.6 mph

Building:

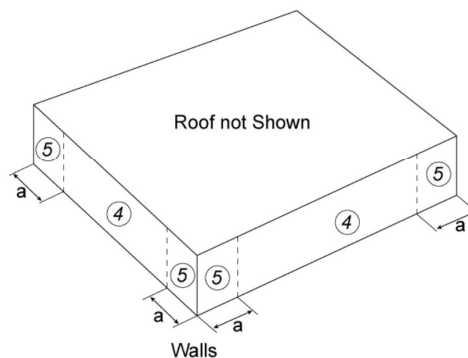
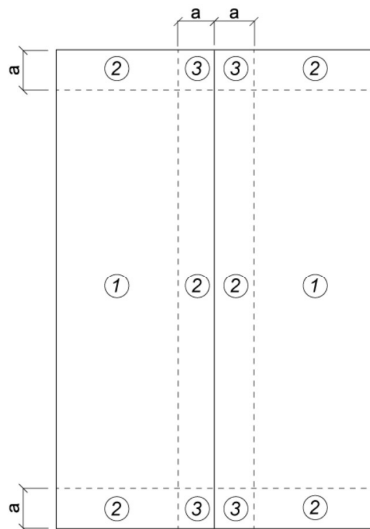
Roof = Roof Type	=	Gabled	Encl = Enclosure Classification	=	Enclosed
Help = Help on Building Roof Type	=	Help	W = Building Width	=	28.600 ft
L = Building Length	=	110.000 ft	R _{ht} = Ridge Height	=	24.767 ft
E _{ht} = Eave Height	=	20.000 ft	Pitch = Pitch of Roof	=	4.0 :12
θ = Slope of Roof	=	18.435 Deg	OH = Overhang Configuration	=	All None
Par = Parapet	=	None	HT _{over} = Override Mean Roof Height	=	False
HT _{man} = Mean Roof Height	=	22.383 ft	RA _{over} = Override Roof Area	=	False
GC _{pi_o} = Override GC _{pi} value	=	False	IsElev = Building is Elevated	=	False

Exposure Constants [T:26.11-1]:

α = 3-s Gust-speed exponent	=	11.500	Z _g = Nominal Ht of Boundary Layer	=	1935.000 ft
â = Reciprocal of α	=	0.087	b = 3 sec gust speed factor	=	1.090
α _m = Mean hourly Wind-Speed Exponent	=	0.125	b _m = Mean hourly Windspeed Exponent	=	0.780
c = Turbulence Intensity Factor	=	0.150	e = Integral Length Scale Exponent	=	0.1250

Components and Cladding (C&C) Wind Loads per Ch 30 Pt 1 Roof & Wall

FBC 2023 refers to ASCE 7-22 for these calculations.



h = Mean structure height	=	22.383 ft	K _h = 2.41 • (Z/Z _g) ^{2/α}	=	1.110
K _{zt} = No Topographic Feature	=	1.000	K _d = Directionality Factor T:26.6-1	=	0.85
GC _{pi} = ± Internal Press Coef T:26.13-1	=	±0.18	LF = ASD Load Factor	=	0.60
K _e = Ground Elev Factor T:26.10-1	=	1.000	q _h = 0.00256 • K _h • K _{zt} • K _e • V ² • LF E:26.10-1	=	52.20 psf

$\theta = \text{Slope of Roof}$
 $a = \text{Max}(a_1, 0.04 \cdot B, 3 \text{ ft } [0.9 \text{ m}])$

$= 18.43^\circ$
 $= 3.000 \text{ ft}$

$a_1 = \text{Min}(0.1 \cdot B, 0.4 \cdot h)$
 $= 2.860 \text{ ft}$

Wind Pressures for C&C Ch 30 Pt 1 Roof & Wall
 All wind pressures include a Load Factor (LF) of 0.6

Description	Zone	Width ft	Span ft	Area ft ²	1/3 Rule	Figure	GC _{pi}	GC _{pd}	GC _{pu}	P _{down} psf	P _{uplift} psf
	1	1.00001	1.0000	1.00	No	30.3-2B	±0.18	0.60	-2.00	34.61	-96.72
	2	1.00001	1.0000	1.00	No	30.3-2B	±0.18	0.60	-2.70	34.61	-127.78
	3	1.00001	1.0000	1.00	No	30.3-2B	±0.18	0.60	-3.60	34.61	-167.71

GC_{pd}
 P_{down}
 $+Press$
 $\$30.2.2$
 $Width$
 $Area$
 GC_{pi}

= Down (+) External Coefficient
 = Down Press: $q_h \cdot K_d \cdot [GC_{pd} - GC_{pi}]$ E:30.3-1
 = Pressure Acting Toward Surface
 = C&C Min Pressure = 9.60 psf
 = Width of Component
 = Span • Width
 = +Internal Coef T:26.13-1

GC_{pu}
 P_{uplift}
 $-Press$
 $Zone$
 $Span$
 $1/3 \text{ Rule}$
 $Figure$

= Uplift (-) External Coefficient
 = Uplift Press: $q_h \cdot K_d \cdot [GC_{pu} - GC_{pi}]$ E:30.3-1
 = Pressure Acting Away from Surface
 = Applicable Zone per Figure
 = Span of Component
 = Width limited to Span/3
 = Applicable Figure from Standard

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: roof.ec6

LIC# : KW-06016439, Build:20.23.08.30

YOUSSEF HACHEM CONSULTING ENGINEERING INC

(c) ENERCALC INC 1983-2023

DESCRIPTION: roof beam

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : IBC 2021

Wood Species : Southern Pine

Wood Grade : No.2: 2"-4" Thick: 5"-6" Wide

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

Fb + 1,000.0 psi

Fb - 1,000.0 psi

Fc - Prll 1,400.0 psi

Fc - Perp 565.0 psi

Fv 175.0 psi

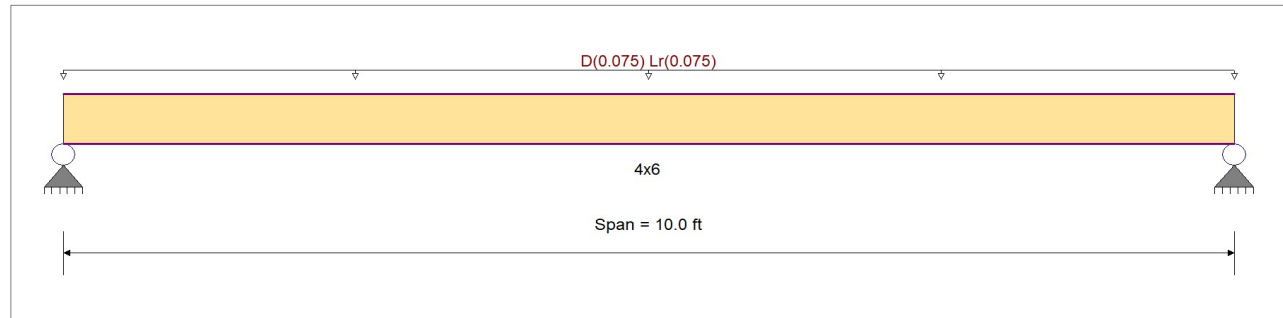
Ft 600.0 psi

E : Modulus of Elasticity

Ebend- xx 1,400.0 ksi

Eminbend - xx 510.0 ksi

Density 34.330pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.0750, Lr = 0.0750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	1.020	1	Maximum Shear Stress Ratio	=	0.244	1
Section used for this span		4x6		Section used for this span		4x6	
fb: Actual	=	1,275.09psi		fv: Actual	=	53.32 psi	
F'b	=	1,250.00psi		F'v	=	218.75 psi	
Load Combination		+D+Lr		Load Combination		+D+Lr	
Location of maximum on span	=	5.000ft		Location of maximum on span	=	0.000 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.250 in	Ratio =	480	>=360	Span: 1 : Lr Only		
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a		
Max Downward Total Deflection	0.500 in	Ratio =	240	>=180	Span: 1 : +D+Lr		
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a		

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0	
Length = 10.0 ft	1		0.708	0.169	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.94	637.5	900.0	0.34	26.7	157.5
+D+Lr						1.00	1.00	1.00	1.000	1.00	1.00	1.00		0.0	0.00	0.0	0.0	
Length = 10.0 ft	1		1.020	0.244	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.88	1,275.1	1,250.0	0.68	53.3	218.8
+D+0.750Lr						1.00	1.00	1.00	1.000	1.00	1.00	1.00		0.0	0.00	0.0	0.0	
Length = 10.0 ft	1		0.893	0.213	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.64	1,115.7	1,250.0	0.60	46.7	218.8
+0.60D						1.00	1.00	1.00	1.000	1.00	1.00	1.00		0.0	0.00	0.0	0.0	
Length = 10.0 ft	1		0.239	0.057	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	382.5	1,600.0	0.21	16.0	280.0

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: roof.ec6

LIC# : KW-06016439, Build:20.23.08.30

YOUSSEF HACHEM CONSULTING ENGINEERING INC

(c) ENERCALC INC 1983-2023

DESCRIPTION: roof beam

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.4997	5.036		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.750	0.750
Max Upward from Load Combinations	0.750	0.750
Max Upward from Load Cases	0.375	0.375
D Only	0.375	0.375
+D+Lr	0.750	0.750
+D+0.750Lr	0.656	0.656
+0.60D	0.225	0.225
Lr Only	0.375	0.375

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: roof.ec6

LIC# : KW-06016439, Build:20.23.08.30

YOUSSEF HACHEM CONSULTING ENGINEERING INC

(c) ENERCALC INC 1983-2023

DESCRIPTION: wind uplift roof beam

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

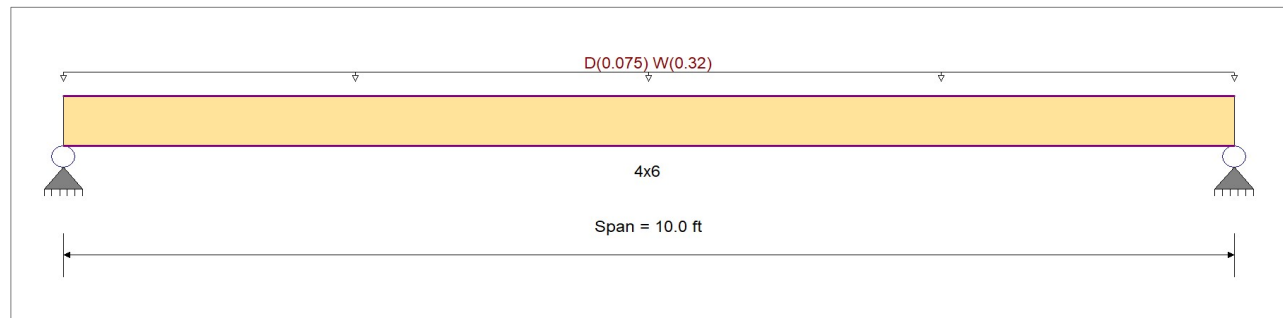
Analysis Method : Allowable Stress Design
Load Combination : IBC 2021

Fb + 1,000.0 psi
Fb - 1,000.0 psi
Fc - Prll 1,400.0 psi
Fc - Perp 565.0 psi
Fv 175.0 psi
Ft 600.0 psi

E : Modulus of Elasticity
Ebend- xx 1,400.0 ksi
Eminbend - xx 510.0 ksi
Density 34.330 pcf

Wood Species : Southern Pine
Wood Grade : No.2: 2"-4" Thick: 5"-6" Wide

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



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.0750, W = 0.320, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	1.419	1	Maximum Shear Stress Ratio	=	0.339	1
Section used for this span		4x6		Section used for this span		4x6	
fb: Actual	=	2,269.66 psi		fv: Actual	=	94.91 psi	
F'b	=	1,600.00 psi		F'v	=	280.00 psi	
Load Combination		+D+0.60W		Load Combination		+D+0.60W	
Location of maximum on span	=	5.000 ft		Location of maximum on span	=	0.000 ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	1.066 in	Ratio =	112 < 360	Span: 1 : W Only			
Max Upward Transient Deflection	0 in	Ratio =	0 < 360	n/a			
Max Downward Total Deflection	0.889 in	Ratio =	134 < 180	Span: 1 : +D+0.60W			
Max Upward Total Deflection	0 in	Ratio =	0 < 180	n/a			

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only																	
Length = 10.0 ft	1	0.708	0.169	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.94	637.5	900.0	0.00	0.0	0.0
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.34	26.7	157.5
Length = 10.0 ft	1	1.419	0.339	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.34	2,269.7	1,600.0	1.22	94.9	280.0
+D+0.450W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	1.164	0.278	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.74	1,861.6	1,600.0	1.00	77.9	280.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	1.259	0.301	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.96	2,014.6	1,600.0	1.08	84.2	280.0
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.239	0.057	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	382.5	1,600.0	0.21	16.0	280.0

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: roof.ec6

LIC# : KW-06016439, Build:20.23.08.30

YOUSSEF HACHEM CONSULTING ENGINEERING INC

(c) ENERCALC INC 1983-2023

DESCRIPTION: wind uplift roof beam

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	1.0660	5.036		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.600	1.600
Max Upward from Load Combinations	1.335	1.335
Max Upward from Load Cases	1.600	1.600
D Only	0.375	0.375
+D+0.60W	1.335	1.335
+D+0.450W	1.095	1.095
+0.60D+0.60W	1.185	1.185
+0.60D	0.225	0.225
W Only	1.600	1.600

Appendix D

Survey

