Basis of Design

Repair and Preservation of Matthew Jones House

Project #: HERT 20-2645

Fort Eustis, VA

100% Design Submission

May 15, 2025

Table of Contents

A. Basis of Design/Design Analysis

Design Scope of Work

The Matthew Jones House was built in 1725 as a one-and-a-half story timber framed structure with brick masonry chimneys on both side elevations. In 1730, the structure was improved with the addition of brick masonry walls, a two-story tower at its front elevation and a one-story shed structure along its rear elevation. In 1893, the roof of the house was raised to provide a full-height second floor, and the chimneys were extended.

The purpose of this design project is to make repairs and preservation efforts to stabilize and maintain the condition of the historic house. The basis of the repairs shall follow the documents physical survey (OPTION 2) of the Matthew Jones House, prepared by Mesick Cohen Wilson Baker Architects (MCWB) in collaboration with Resource Management Associates. The study presents prioritized recommendations for the on-going preservation of this important building, as well as the new abbreviated investigation to confirm the condition and ensure no new repairs are required.

This project shall have no adverse effect on the historic Matthew Jones House as defined in Code of Federal Regulations 36 CFR 800.5(b). In order to achieve that, the design must be consistent with the Secretary of the Interior's Standards for the Treatment of Historic Properties, Preservation found at 36 CFR 68.3(a). The Virginia State Historic Preservation Office shall be consulted throughout the development of the design.

Design Criteria

Applicable Codes & Standards

International Building Code (2021)
NFPA 101 -Life Safety Code (2015)
NFPA 72 - National Fire Alarm Code
UFC 1-200-01 - General Building Requirements
UFC 1-300-01 - Design Procedures
UFC 1-300-09N - Design Procedures
UFC 3-101-01 - Architecture
UFC 3-110-03 - Roofing
UFC 3-301-01 - Structural Engineering

Structural Design Loads:

The following design loads will be used per the 2021 International Building Code as adopted by the Virginia Uniform Statewide Building Code, 2021 Edition:

Design Wind Loads:	Ultimate Wind Speed	116 mph
	Risk Category	II
	Exposure Category	C
Design Snow Loads:	Ground Snow Load, Pg(asd)	15 psf
Design Seismic Loads:	Seismic Resisting System	Ordinary plain masonry shear walls

Basic Structural Systems:

Foundations: The west gable end wall and west portion of the north wall is to be underpinned with reinforced concrete. This is to provide additional load distribution at portions of the wall that appear to be out of plumb and bowed.

Roof Structures: The main roof structure is constructed with timber rafters that bear on a timber false plate. Connections from the rafters to the false plate and false plate to the ceiling joists are to be redesigned to provide additional thrust and uplift capacity.

Connections at both ends of the shed roof rafters are to be reinforced and detailed to provide additional load capacities.

Lateral Stability: A lateral system will be designed at the west gable end wall to provide lateral support where the 2nd floor system has been removed from the building. A lateral system will also be designed at roof level of the west gable end wall.

Accessibility: The stair structure to the entrance on the east side of the house will be redesigned. Architect or owner is to provide direction on what material is to be used for the stair structure.

Building Envelope: A masonry restoration plan will be developed for the exterior and interior walls of the home. This plan is to address masonry deterioration and weathering throughout the wall surfaces. Wall segment separation is to be addressed in concurrence with the masonry restoration plan.

B. Calculations

CALCULATION PACKAGE

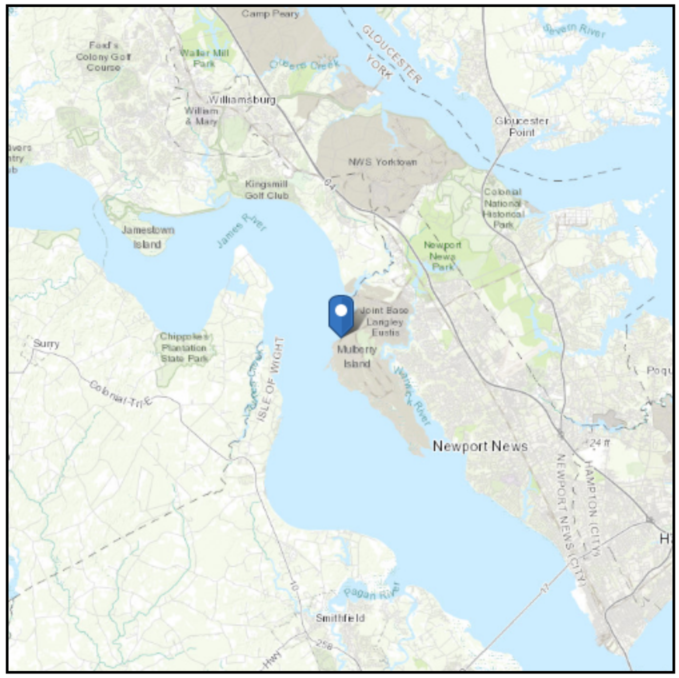
LOAD SUMMARY

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE Hazards Report

Address:
Harrison Rd
Fort Eustis, Virginia
23604

Standard:	ASCE/SEI 7-22	**Latitude:**	37.143778
Risk Category:	II	**Longitude:**	-76.61601
Soil Class:	D - Stiff Soil	**Elevation:**	5.05353108034816 ft (NAVD 88)



Wind

Results:

Wind Speed	117 Vmph
10-year MRI	77 Vmph
25-year MRI	85 Vmph
50-year MRI	93 Vmph
100-year MRI	99 Vmph
300-year MRI	109 Vmph
700-year MRI	117 Vmph
1,700-year MRI	126 Vmph
3,000-year MRI	129 Vmph
10,000-year MRI	139 Vmph
100,000-year MRI	159 Vmph
1,000,000-year MRI	180 Vmph

Data Source:	ASCE/SEI 7-22, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Wed Jan 08 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-22 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years). Values for 10-year MRI, 25-year MRI, 50-year MRI and 100-year MRI are Service Level wind speeds, all other wind speeds are Ultimate wind speeds.

Site is in a hurricane-prone region as defined in ASCE/SEI 7-22 Section 26.2. Glazed openings need not be protected against wind-borne debris.

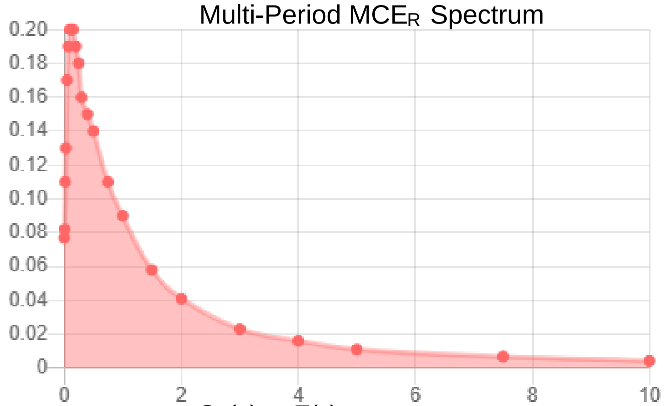
Site Soil Class: D - Stiff Soil

Results:

PGA_M :	0.069	T_L :	8
S_{MS} :	0.17	S_S :	0.13
S_{M1} :	0.09	S_1 :	0.042
S_{DS} :	0.11	V_{S30} :	260
S_{D1} :	0.06		

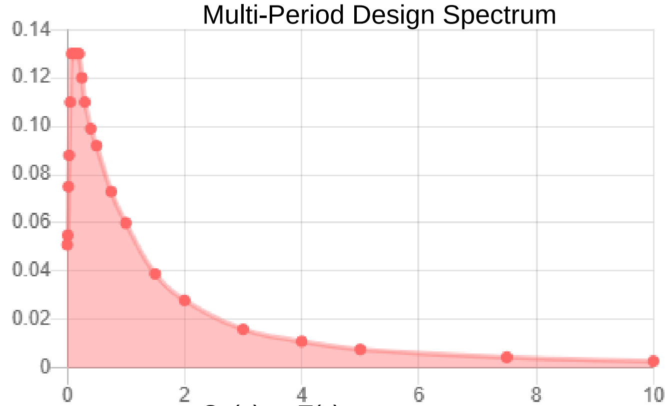
Seismic Design Category: A

Multi-Period MCE_R Spectrum



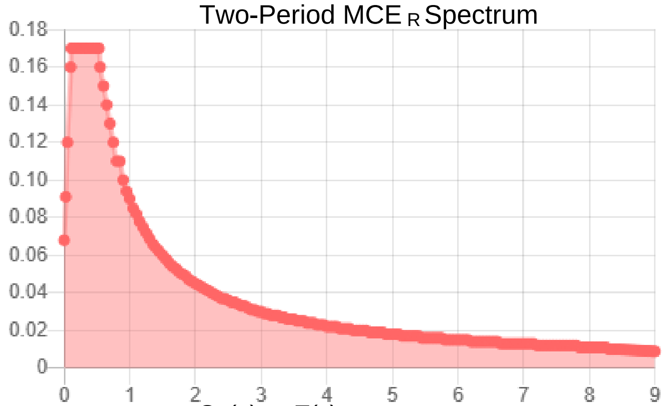
S_a(g) vs T(s)

Multi-Period Design Spectrum



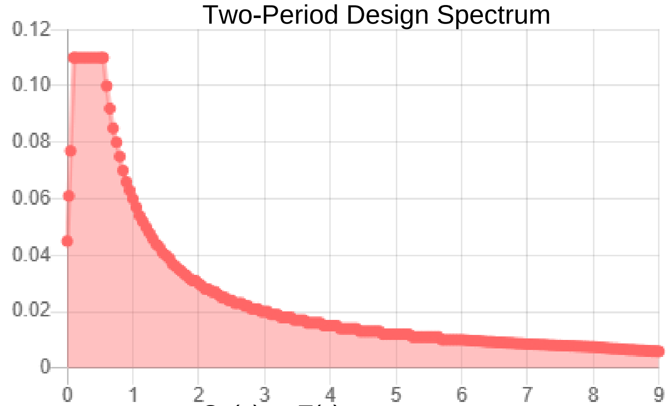
S_a(g) vs T(s)

Two-Period MCE_R Spectrum



S_a(g) vs T(s)

Two-Period Design Spectrum



S_a(g) vs T(s)

MCE_R Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Wed Jan 08 2025**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

Snow

Results:

Ground Snow Load, p_g :	36 lb/ft^2
20-year MRI Value:	9.44 lb/ft^2
Winter Wind Parameter:	0.45
Mapped Elevation:	3.9 ft
Data Source:	ASCE/SEI 7-22, Figures 7.6-1 and 7.6-2 A-D
Date Accessed:	Wed Jan 08 2025

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported ‘elevation’ and ‘mapped elevation’ differ significantly from each other.

Ground Snow Loads for IRC only, $p_{g(asd)}$: 25.2 lb/ft^2

MecaWind v2424

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Calculations Prepared by:
Date: Jan 09, 2025

File Location: N:\Projects\2024\MB246004\Working Files\Calculations-Analyses\MECAWIND.wnd

General:

Wind Load Standard	= ASCE 7-22	Basic Wind Speed	= 117.0 mph
Exposure Classification	= D	Risk Category	= II
Structure Type	= Building	Design Basis for Wind Pressures	= LRFD
Dynamic Type of Structure	= Rigid	Simple Diaphragm Building	= False
MWFRS Analysis Method	= Ch 27	C&C Analysis Method	= Ch 30 Pt 1
MWFRS Pressure Elevations	= Mean Ht	Topographic Effects	= None
Override Directionality Factor K_d	= False	Override the Gust Factor G	= False

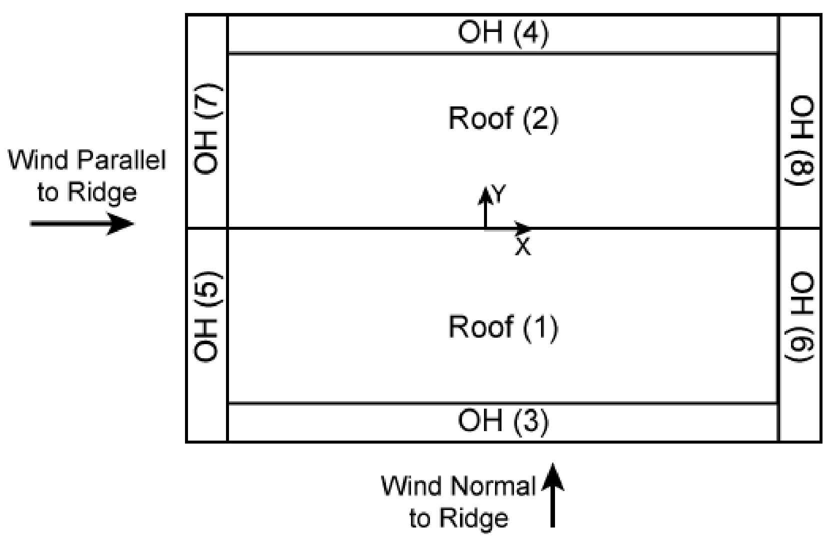
Building:

Roof	= Roof Type	= Gabled	Encl	= Enclosure Classification	= Enclosed
Help	= Help on Building Roof Type	= Help	Pitch	= Pitch of Roof	= 8.0 :12
θ	= Slope of Roof	= 33.68 Deg	R_{Ht}	= Ridge Height	= 23.581 ft
E_{Ht}	= Eave Height	= 16.417 ft	W	= Building Width	= 21.500 ft
L	= Building Length	= 31.083 ft	OH	= Type of Overhang	= None
Par	= Parapet Porosity	= None	HT_{over}	= Override Mean Roof Height	= False
Ht_{man}	= Mean Roof Height	= 19.999 ft	RA_{over}	= Override Roof Area	= False
IsElev	= Building is Elevated	= False			

Exposure Constants [Table 26.11-1]:

α = 3-s Gust-speed exponent	= 11.500	Z_g = Nominal Ht of Boundary Layer	= 1935.000 ft
â = Recipicol of α	= 0.087 ft	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	ε = Integral Length Scale Exponent	= 0.1250

Main Wind Force Resisting System (MWFRS) Wind Calculations per Ch 27



h	= Mean structure height	= 19.999 ft
K_z	= $2.41 \cdot (Z/Z_g)^{2/\alpha}$ [Table 26.10-1]	= 1.088
K_{zt}	= No Topographic feature specified	= 1.000
K_d	= Wind Directionality Factor per Table 26.6-1	= 0.85
$+GC_{pi}$	= Enclosed Positive Internal Pressure Table 26.13-1	= +0.18
$-GC_{pi}$	= Enclosed Negative Internal Pressure Table 26.13-1	= -0.18
LF	= Load Factor based upon STRENGTH Design	= 1.00
q_h	= $0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eqn 26.10-1]	= 38.13 psf

K_e	= Ground Elevation Factor: $e^{-0.0000362 \cdot Z_g}$ [Table 26.10-1]	= 1.000
RA	= Roof Area	= 803.08 ft^2
q_h	= $0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eqn 26.10-1]	= 38.13 psf
q_{in}	= Negative Internal Pressure: $q_h \cdot LF$	= 38.13 psf
q_{ip}	= Positive Internal Pressure: $q_h \cdot LF$	= 38.13 psf

MWFRS Wind Loads [Normal to Ridge]

h	= Mean Roof Height Of Building	= 19.999 ft
RHt	= Ridge Height Of Roof	= 23.581 ft
B	= Horizontal Dimension Of Building Normal To Wind Direction	= 31.083 ft
L	= Horizontal Dimension Of building Parallel To Wind Direction	= 21.500 ft
L/B	= Ratio Of L/B used For Cp determination	= 0.692
h/L	= Ratio Of h/L used For Cp determination	= 0.930
Slope	= Slope Of Roof	= 33.68 Deg

Gust Factor Calculation for Wind: [Normal to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method

G_1	= For Rigid Structures (Natural Frequency > 1 Hz) use 0.85	= 0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m	= Equiv Height of Struc: Max(0.6*Ht, Z_{min})	= 11.999 ft
I_{zm}	= Intensity of Turbulence at height Z_m: $c \cdot (33/Z_m)^{1/6}$ [Eqn 26.11-7]	= 0.178
L_{zm}	= Integeral Length Scale of Turbulence [Eqn 26.11-9]	= 572.788 ft
B	= Building Width Width Normal to Wind Direction	= 31.083 ft
Q	= $1/(1+0.63 \cdot [(B+Ht)/L_{zm}]^{0.63})$ [Eqn 26.11-8]	= 0.938
G_2	= $0.925 \cdot ((1+1.7 \cdot 3.4 \cdot I_{zm} \cdot Q)/(1+1.7 \cdot 3.4 \cdot I_{zm}))$	= 0.896

Gust Factor Used in Analysis

G	= Gust Factor: Min(G_1, G_2)	= 0.850

Cp_{WW}	= Windward Wall Coefficient (All L/B Values)	= 0.800
Cp_{LW}	= Leeward Wall Coefficient using L/B	= -0.500
Cp_{SW}	= Side Wall Coefficient (All L/B values)	= -0.700

Wind Pressures [Normal to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Elev ft	GC_{pi}	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
16.417	+0.18	38.13	1.051	1.000	36.85	15.46	-19.61	-25.12	35.07	16.00
16.417	-0.18	38.13	1.051	1.000	36.85	27.13	-7.94	-13.45	35.07	16.00

Notes Wall Pressures

K_z	=	$2.41 \cdot (Z/Z_g)^{2/\alpha}$	K_{zt}	=	No Topographic feature specified
GC_{pi}	=	Enclosed Internal Pressure Table 26.13-1	q_z	=	$0.00256 \cdot K_z \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eqn 26.10-1]
q_{ip}	=	Positive Internal Pressure: $q_h \cdot LF$	q_{in}	=	Negative Internal Pressure: $q_h \cdot LF$
Side	=	$q_h \cdot K_d \cdot G \cdot Cp_{SW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eqn 27.3-1	Leeward	=	$q_h \cdot K_d \cdot G \cdot Cp_{LW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eqn 27.3-1
Windward	=	$q_z \cdot K_d \cdot G \cdot Cp_{WW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eqn 27.3-1	Total	=	Windward - Leeward

- *Minimum Pressure: Para 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures for Positive & Negative Internal Pressure ($\pm GC_{pi}$) [Normal to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Reference	Description	Location	C_p Min	C_p Max	GC_{pi}	Pressure Min psf	Pressure Max psf
Roof	Roof Windward	1	0.210	-0.223	+0.18/-0.18	-11.97	11.63
Roof	Roof Leeward	2	-0.600	-0.600	+0.18/-0.18	-22.36	-10.70

Notes Roof Pressures based upon Ch 27:

Start = Start Dist from Windward Edge		End = End Dist from Windward Edge	
C_{p_min} = Smallest Coefficient Magnitude		C_{p_max} = Largest Coefficient Magnitude	
$Press_{Min} = q_h \cdot K_d \cdot G \cdot C_{p\_min} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eqn 27.3-1		$Press_{Max} = q_h \cdot K_d \cdot G \cdot C_{p\_max} - q_{in} \cdot K_d \cdot (-GC_{pi})$ Eqn 27.3-1	

- 0.826 Reduction Factor applied for h/L>=1 & 10<=Slope<=15 Deg
- The smaller uplift pressures due to Cp_Min can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7
- Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface

MWFRS Wind Loads [Parallel to Ridge]

h	= Mean Roof Height Of Building	= 19.999 ft
RHt	= Ridge Height Of Roof	= 23.581 ft
B	= Horizontal Dimension Of Building Normal To Wind Direction	= 21.500 ft
L	= Horizontal Dimension Of building Parallel To Wind Direction	= 31.083 ft
L/B	= Ratio Of L/B used For Cp determination	= 1.446
h/L	= Ratio Of h/L used For Cp determination	= 0.643
Slope	= Slope Of Roof	= 33.68 Deg

Gust Factor Calculation for Wind: [Parallel to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method

G_1	= For Rigid Structures (Natural Frequency > 1 Hz) use 0.85	= 0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m	= Equiv Height of Struc: Max(0.6*Ht, Z_{min})	= 11.999 ft
I_{zm}	= Intensity of Turbulence at height Z_m: $c \cdot (33/Z_m)^{1/6}$ [Eqn 26.11-7]	= 0.178
L_{zm}	= Integeral Length Scale of Turbulence [Eqn 26.11-9]	= 572.788 ft
B	= Building Width Width Normal to Wind Direction	= 21.500 ft
Q	= $1/(1+0.63 \cdot [(B+Ht)/L_{zm}]^{0.63})$ [Eqn 26.11-8]	= 0.945
G_2	= $0.925 \cdot ((1+1.7 \cdot 3.4 \cdot I_{zm} \cdot Q)/(1+1.7 \cdot 3.4 \cdot I_{zm}))$	= 0.899

Gust Factor Used in Analysis

G	= Gust Factor: Min(G_1, G_2)	= 0.850
Cp_{WW}	= Windward Wall Coefficient (All L/B Values)	= 0.800
Cp_{LW}	= Leeward Wall Coefficient using L/B	= -0.411
Cp_{SW}	= Side Wall Coefficient (All L/B values)	= -0.700

Wind Pressures [Parallel to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Elev ft	GC_{pi}	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
23.581	+0.18	38.13	1.120	1.000	39.24	16.85	-17.15	-25.12	34.00	16.00
19.999	+0.18	38.13	1.088	1.000	38.13	16.21	-17.15	-25.12	33.36	16.00
16.417	+0.18	38.13	1.051	1.000	36.85	15.46	-17.15	-25.12	32.62	16.00
23.581	-0.18	38.13	1.120	1.000	39.24	28.52	-5.49	-13.45	34.00	16.00
19.999	-0.18	38.13	1.088	1.000	38.13	27.87	-5.49	-13.45	33.36	16.00
16.417	-0.18	38.13	1.051	1.000	36.85	27.13	-5.49	-13.45	32.62	16.00

Notes Wall Pressures

$K_z = 2.41 \cdot (Z/Z_g)^{2/\alpha}$	K_{zt} = No Topographic feature specified
GC_{pi} = Enclosed Internal Pressure Table 26.13-1	$q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eqn 26.10-1]
q_{ip} = Positive Internal Pressure: $q_h \cdot LF$	q_{in} = Negative Internal Pressure: $q_h \cdot LF$
Side = $q_h \cdot K_d \cdot G \cdot Cp_{SW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eqn 27.3-1	Leeward = $q_h \cdot K_d \cdot G \cdot Cp_{LW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eqn 27.3-1
Windward = $q_z \cdot K_d \cdot G \cdot Cp_{WW} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eqn 27.3-1	Total = Windward - Leeward

- *Minimum Pressure: Para 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures for Positive & Negative Internal Pressure ($\pm GC_{pi}$) [Parallel to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Reference	Description	Location	Start Dist ft	End Dist ft	C_p Min	C_p Max	GC_{pi}	Pressure Min psf	Pressure Max psf
Roof	Roof (0 to h/2)	1,2	0.000	9.999	-0.950	-0.180	+0.18/-0.18	-32.01	0.88
Roof	Roof (h/2 to h)	1,2	9.999	19.999	-0.843	-0.180	+0.18/-0.18	-29.05	0.88
Roof	Roof (h to 2*h)	1,2	19.999	31.083	-0.557	-0.180	+0.18/-0.18	-21.19	0.88

Notes Roof Pressures based upon Ch 27:

Start = Start Dist from Windward Edge

End = End Dist from Windward Edge

C_{p_min} = Smallest Coefficient Magnitude

C_{p_max} = Largest Coefficient Magnitude

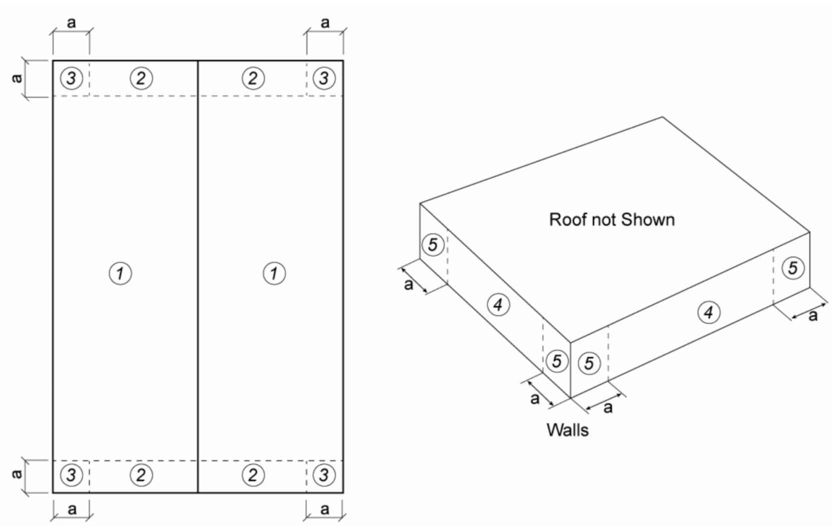
$Press_{Min}$ = $q_h \cdot K_d \cdot G \cdot C_{p\_min} - q_{ip} \cdot K_d \cdot (+GC_{pi})$ Eqn 27.3-1

$Press_{Max}$ = $q_h \cdot K_d \cdot G \cdot C_{p\_max} - q_{in} \cdot K_d \cdot (-GC_{pi})$ Eqn 27.3-1

- 0.826 Reduction Factor applied for h/L>=1 & (0 to h/2)
- The smaller uplift pressures due to Cp_Min can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7
- Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface

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

h/W	= Ratio of mean roof height to building width	= 0.930
h/L	= Ratio of mean roof height to building length	= 0.643
h	= Mean structure height	= 19.999 ft
K_z	= $2.41 \cdot (Z/Z_g)^{2/\alpha}$	= 1.088
K_{zt}	= No Topographic feature specified	= 1.000
K_d	= Wind Directionality Factor per Table 26.6-1	= 0.85
$+GC_{pi}$	= Enclosed Positive Internal Pressure Table 26.13-1	= +0.18
$-GC_{pi}$	= Enclosed Negative Internal Pressure Table 26.13-1	= -0.18
LF	= Load Factor based upon STRENGTH Design	= 1.00
q_h	= $0.00256 \cdot K_h \cdot K_{zt} \cdot K_e \cdot V^2 * LF$ [Eqn 26.10-1]	= 38.13 psf
K_e	= Ground Elevation Factor: $e^{-0.0000362 \cdot Z_g}$ [Table 26.10-1]	= 1.000
LHD	= Least Horizontal Dimension: Min(B, L)	= 21.500 ft
a_1	= $Min(0.1 \cdot LHD, 0.4 \cdot h)$	= 2.150 ft
a	= $Max(a_1, 0.04 \cdot LHD$, 3 ft [0.9 m])	= 3.000 ft
h/B	= Ratio of mean roof height to least horizontal dim: h/B	= 0.930



Wind Pressure Summary for C&C Roof & Wall based Upon Areas Ch 30 Pt 1 (Table 1 of 2)
All wind pressures include a Load Factor (LF) of 1.0

Zone	Figure	A ≤ 10.00 ft^2 psf	A = 20.00 ft^2 psf	A = 50.00 ft^2 psf
1	30.3-2D	35.01 -64.18	32.01 -54.42	28.04 -41.52
2	30.3-2D	35.01 -70.66	32.01 -63.16	28.04 -53.25
3	30.3-2D	35.01 -86.87	32.01 -75.62	28.04 -60.75
4	30.3-1	38.25 -41.49	36.52 -39.77	34.25 -37.49
5	30.3-1	38.25 -51.21	36.52 -47.77	34.25 -43.21

Wind Pressure Summary for C&C Roof & Wall based Upon Areas Ch 30 Pt 1 (Table 2 of 2)
All wind pressures include a Load Factor (LF) of 1.0

Zone	Figure	A = 100.00 ft^2 psf	A = 200.00 ft^2 psf	A > 500.00 ft^2 psf
1	30.3-2D	25.04 -31.76	22.04 -31.76	22.04 -31.76
2	30.3-2D	25.04 -45.75	22.04 -38.25	22.04 -38.25
3	30.3-2D	25.04 -49.50	22.04 -38.25	22.04 -38.25
4	30.3-1	32.52 -35.76	30.80 -34.04	28.52 -31.76
5	30.3-1	32.52 -39.77	30.80 -36.32	28.52 -31.76

* A is effective wind area for C&C: Span Length * Effective Width
* Effective width need not be less than 1/3 of the span length
* Maximum and minimum values of pressure shown.
* + Pressures acting toward surface, - Pressures acting away from surface
* Per Para 30.2.2 the Minimum Pressure for C&C is 16.00 psf [0.766 kPa] {Includes LF}
* Interpolation can be used for values of A that are between those values shown.

MB246004

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Newport News: (757) 873 8858
Williamsburg: (757) 564 4434
Richmond: (804) 684 5850
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PROJECT #/ NAME MATTHEW JONES HOUSE
SUBJECT SNOW LOADING
SHEET ____ OF ____ BY ML DATE 01/09/25

$P_g = 36$ psf

MAIN BUILDING
SLOPE OF ROOF $= 7.5''/12'' \Rightarrow \theta = 32°$
REAR SHED $= 7''/12'' \Rightarrow \theta = 30°$
$\theta > 15°$; SLOPED ROOF SNOW LOADS, P_s

SLOPED ROOF SNOW LOAD $\Rightarrow P_s = C_s P_f$

EXPOSURE CATEGORY
SURFACE ROUGHNESS C
FULLY EXPOSED
$\therefore C_e = .9$

THERMAL FACTOR
UNHEATED STRUCTURE
$\therefore C_t = 1.2$

SLOPE FACTOR
USING FIGURE 7.4-1
$\therefore C_s = 1.0$

FLAT ROOF SNOW LOAD

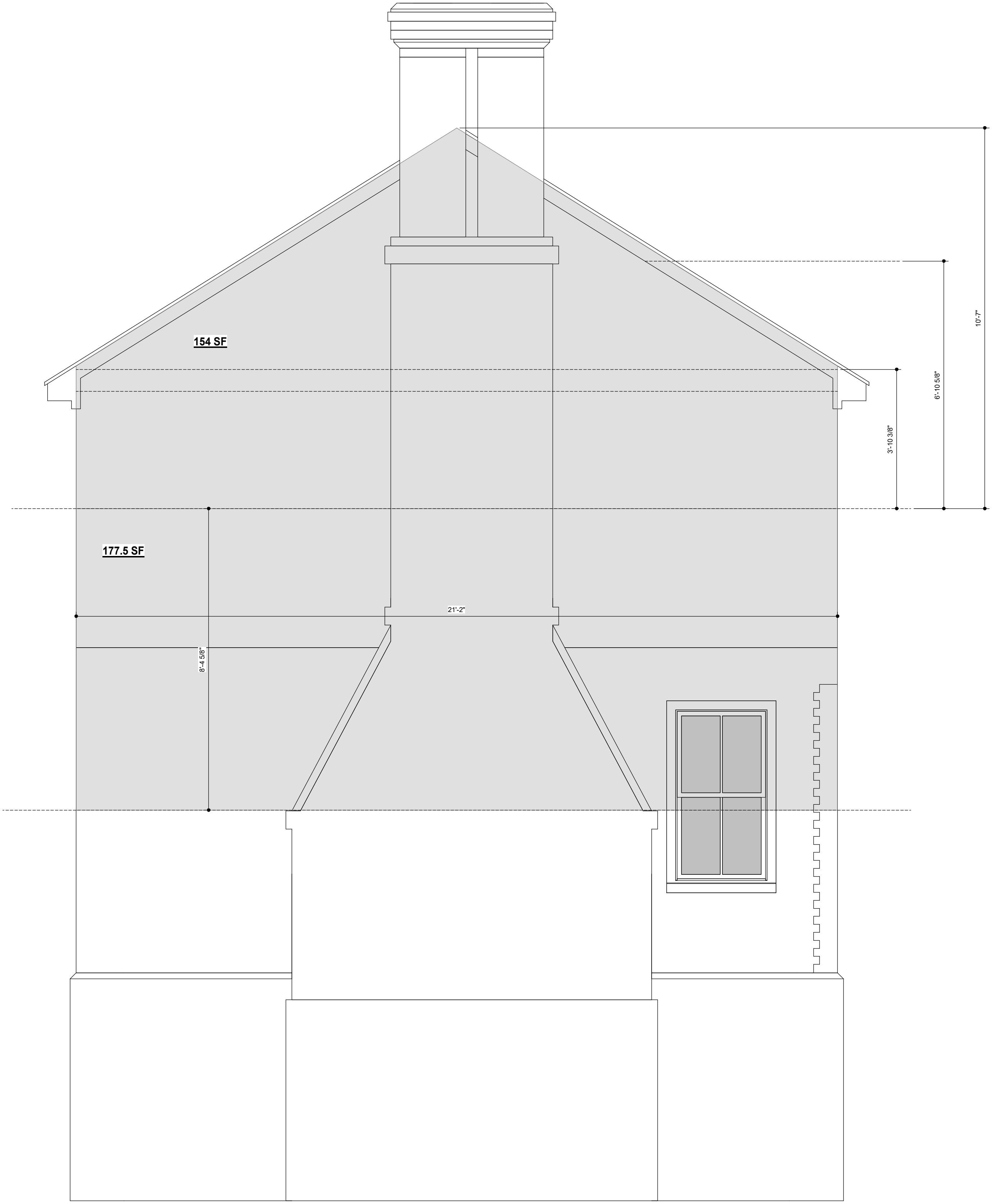
$P_f = .7(C_e)(C_t)P_g$
$= .7(.9)(1.2)(36\text{ psf})$
$= 27.2$ psf

SLOPED ROOF SNOW LOAD
$\rightarrow P_s = C_s P_f$

$P_s = 1.0(27.2\text{ psf}) = 27.2$ psf

2ND FLOOR LATERAL BRACE CALCULATIONS

WEST GABLE END - OUT OF PLANE
TRIB AREAS & SPANS



WEST GABLE END - IN PLANE
TRIB AREAS & SPANS

MB246004

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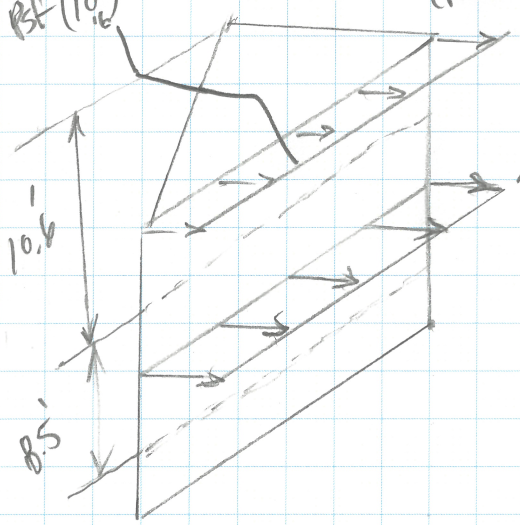
PROJECT #/ NAME MATTHEW JONES HOUSE
SUBJECT WEST GABLE END WALL
LATERAL BRACE @ 2ND FLOOR
SHEET 1 OF 3 BY MJL DATE 01/06/25

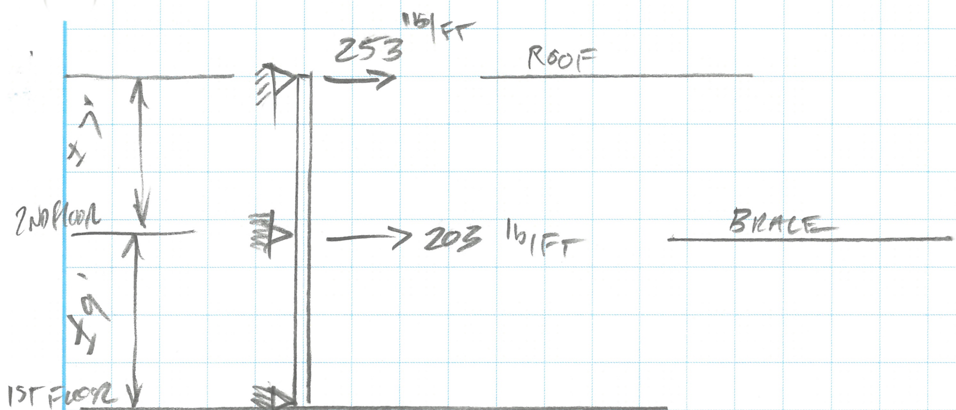
OUT OF PLANE LOADING

TRIB AREA = 154 SF
USING 100 SF → C:C ⇒ 39.77 psf
(LRFD)

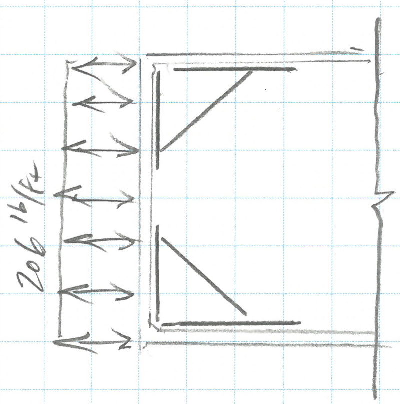
39.8 psf (10.6/2) = 422 lb/ft (.6)
= 253 lb/ft
(ASD)



39.8 PSF (8.5') = 340 lb/FT
(.6) = 203 lb/FT
(ASD)



SECTION



SEE ENERCALC FOR
FORCES IN SYSTEM

PLAN @ BRACE

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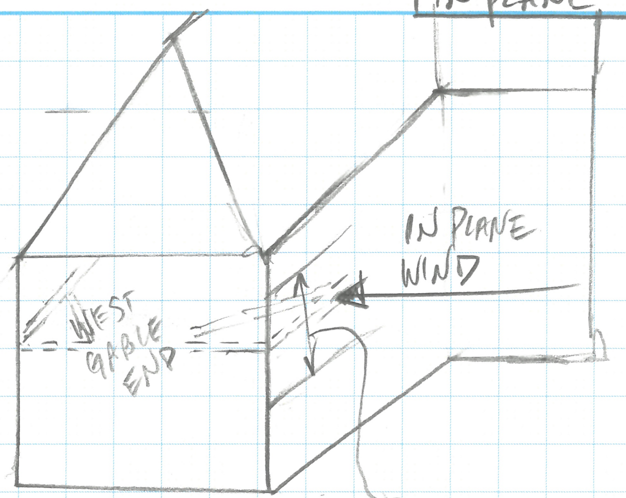
PROJECT #/ NAME MATTHEW JONES

SUBJECT LATERAL BRACE @ 2ND FL.

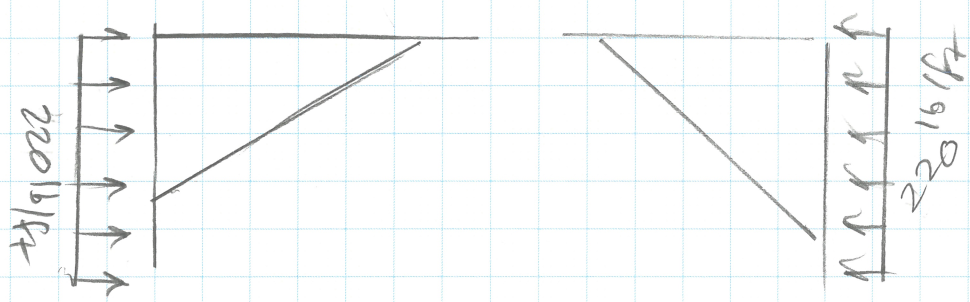
SHEET 2 OF 3 BY MJL DATE 01/06/25

IN PLANE LOADING



TRIB AREA = 80 SF
USE C&C 30 SF
→ ZONE 3 = 43.21 psf (LRFD)

INPLANE TRIB SPAN @ BRACE = 8.5'
LINE LOAD ALONG BRACE = 8.5(43.21) = 367 lb/ft (LRFD)
=> (.6) 367 = 220 lb/ft (ASD)



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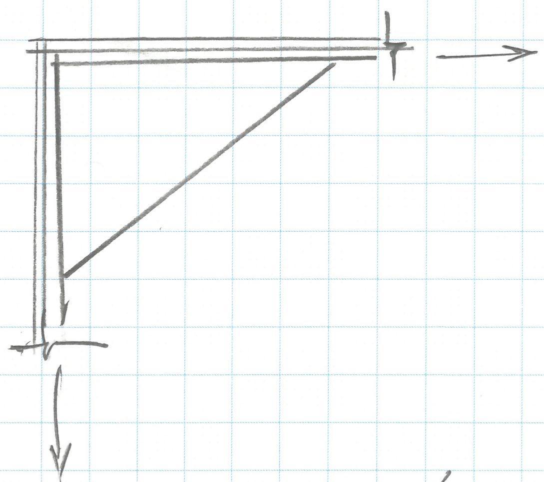
PROJECT #/ NAME: Matthew Jones
SUBJECT: Lateral Brace @ 2nd Floor
SHEET 3 OF 3 BY MJ DATE 01/06/25

Max Loads in System From Risa



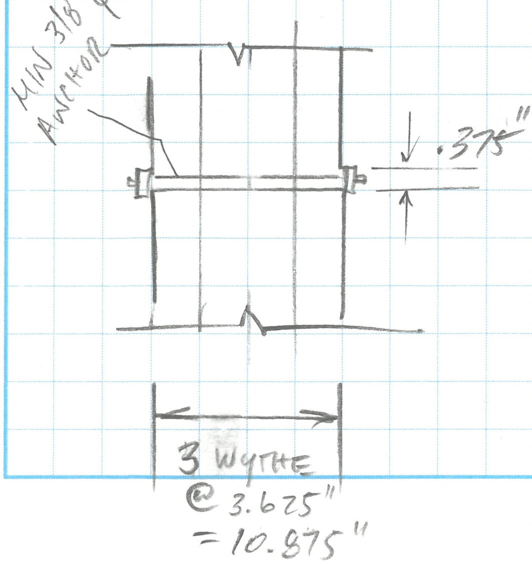
500 lb + 300 lb =
800 lb @ 8.25 ft = 97 lb/ft
IN PLANE

900 lb + 600 lb = 1500 lbs @ 6.5' = 231 lb/ft (CONTROLS)
OUT OF PLANE

Max Anchor Spacing

ONE @ EA END AND ONE IN MIDDLE : $\frac{6.5'}{2} = 3.25'$

$3.25' \, (231 \text{ lb/ft}) = \underline{751 \text{ lbs}}$ SHEAR FORCE IN ANCHOR



STRENGTH OF BRICK @ ANCHOR

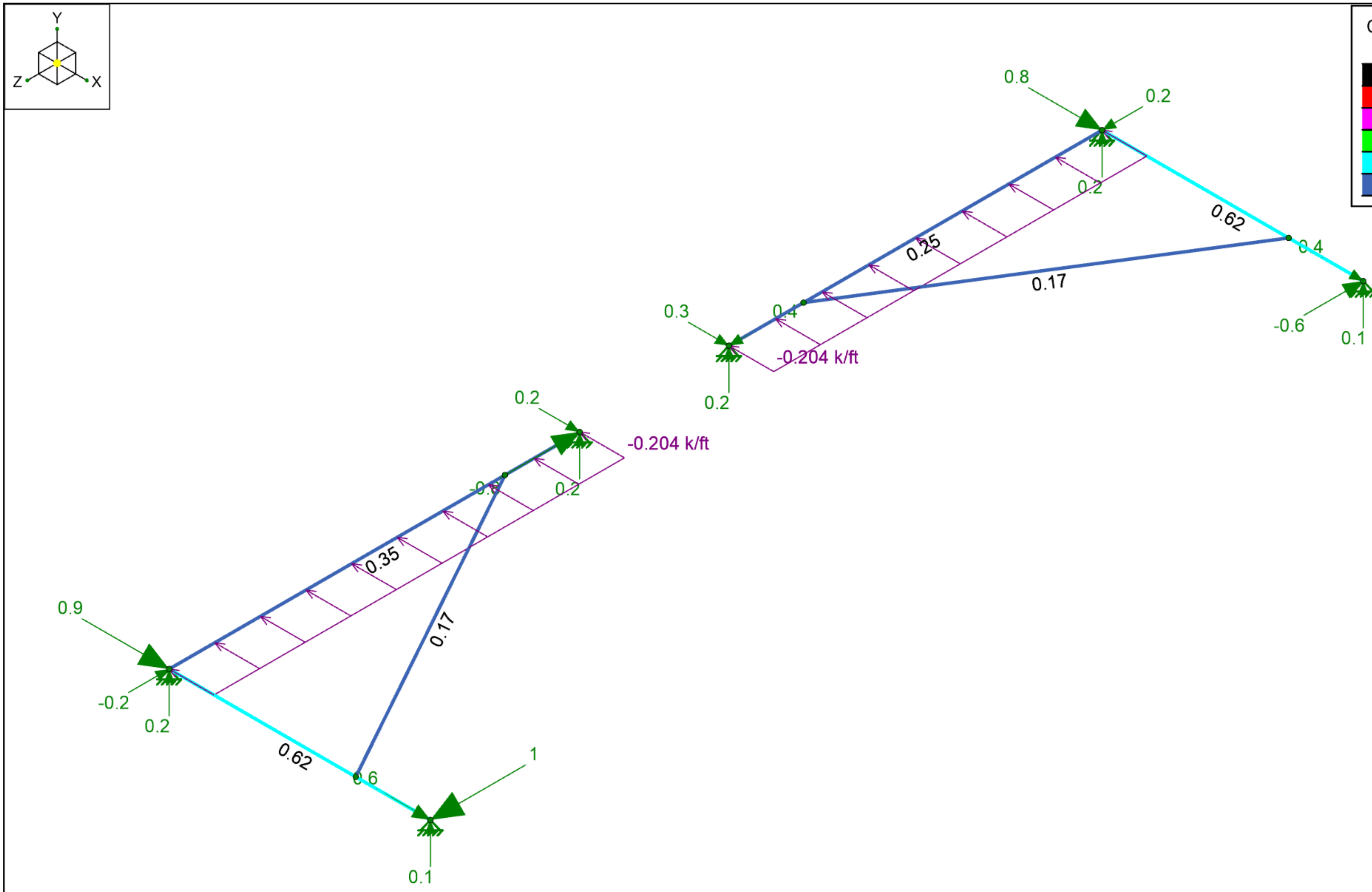
$.375'' \times 10.875'' \times 911 \frac{lb}{in^2} = 3720 \text{ lbs}$

> 751 lbs
OK
Good ✓

STRENGTH OF TYPICAL BRICK MOLDED MASONRY

$1822 \frac{lb}{in^2} (.5) = 911 \frac{lb}{in^2}$

SAFETY FACTOR

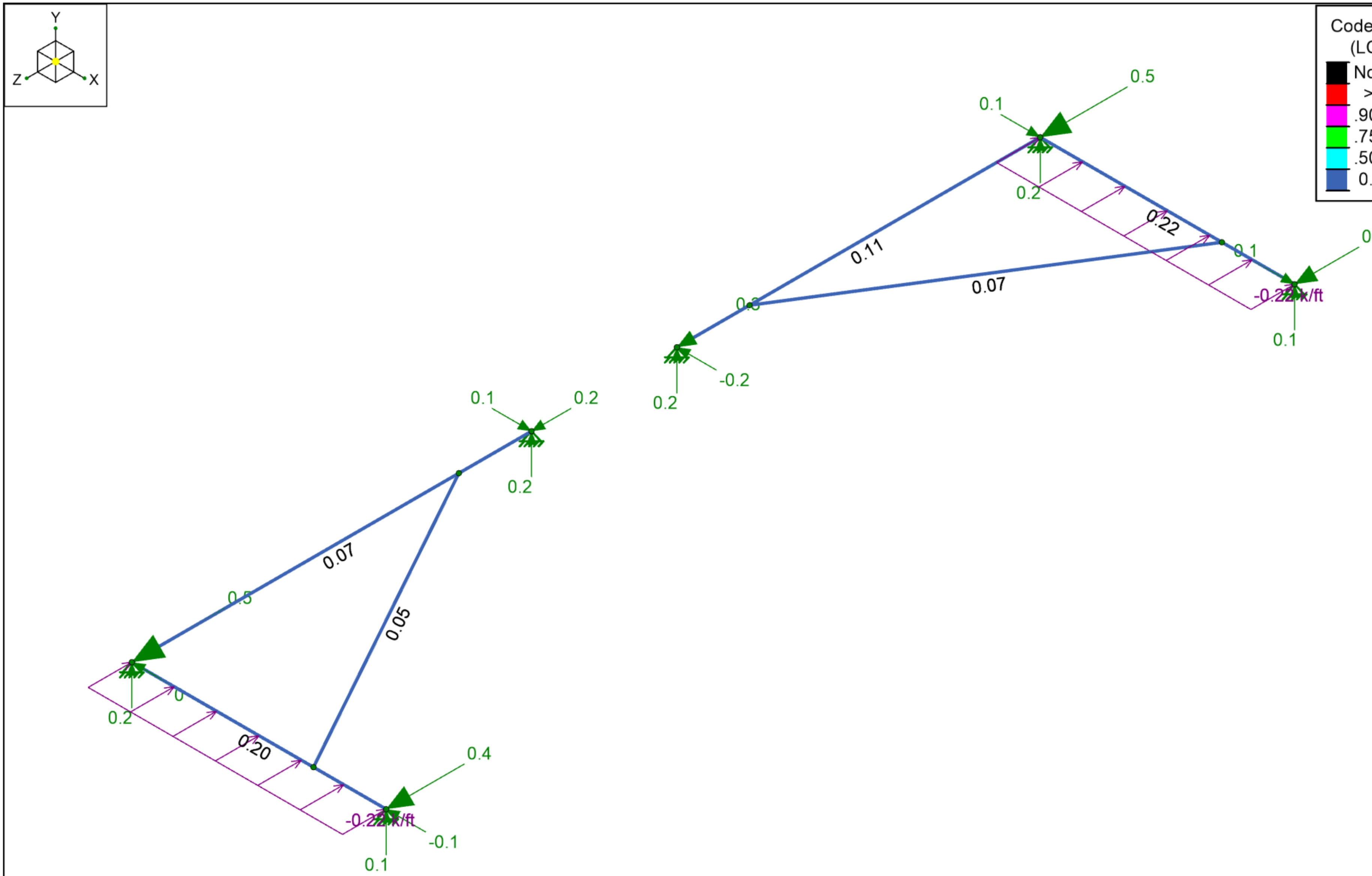


Code Check
(LC 10)

- No Calc
- > 1.0
- .90-1.0
- .75-.90
- .50-.75
- 0.-.50

Member Code Checks Displayed
Loads: LC 10, IBC 21/ASCE ASD 5 (a) (c)
Results for LC 10, IBC 21/ASCE ASD 5 (a) (c)
Reaction and Moment Units are kips and kip-ft

RISA A NEMETSCHEK COMPANY	<Licensed Company>		SK-1
	mjlucas		Jan 09, 2025 at 09:05 AM
			MB246004 - Matthew Jones House.r3d



Code Check
(LC 11)

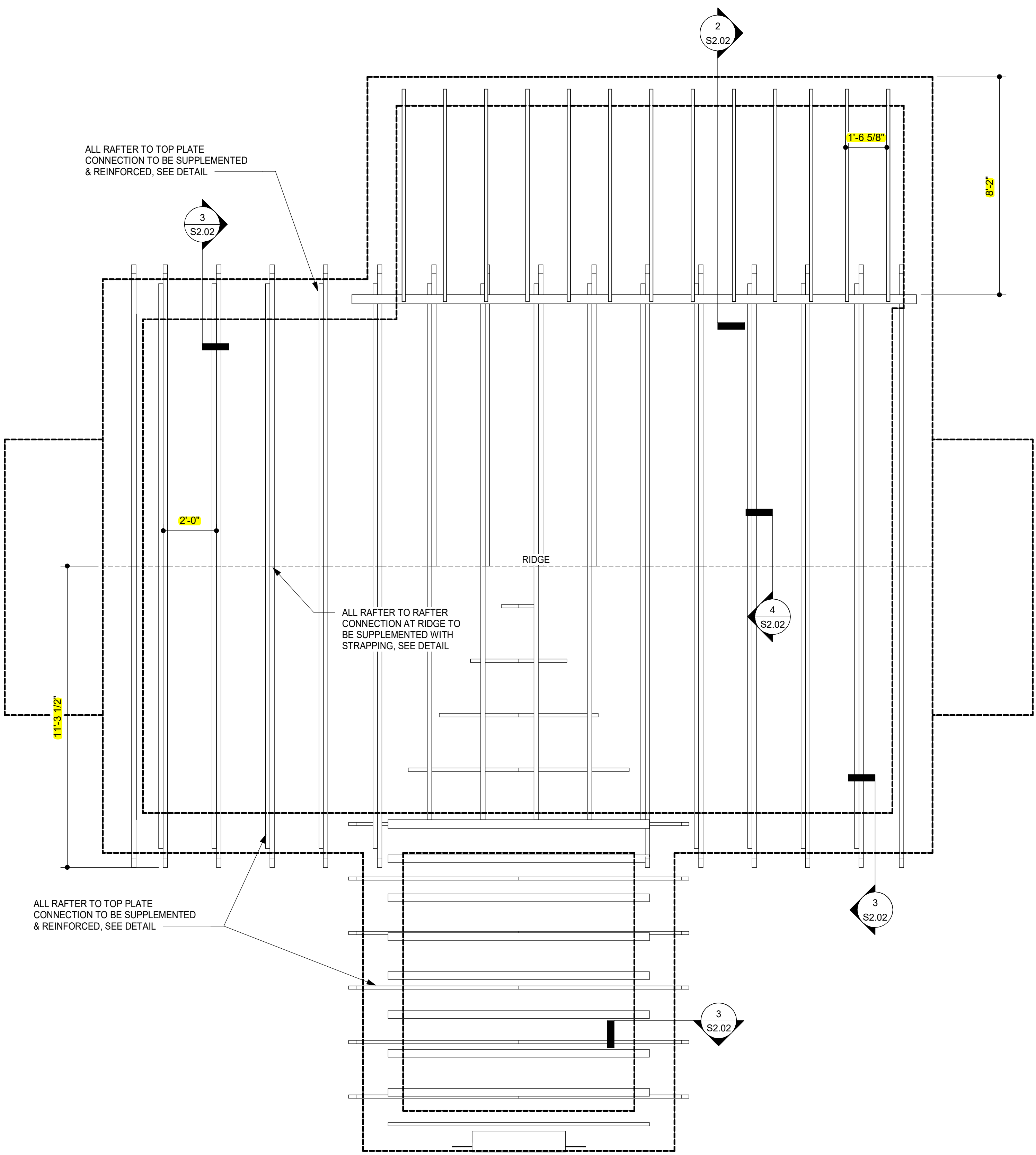
- No Calc
- > 1.0
- .90-1.0
- .75-.90
- .50-.75
- 0.-.50

Member Code Checks Displayed
Loads: LC 11, IBC 21/ASCE ASD 5 (a) (d)
Results for LC 11, IBC 21/ASCE ASD 5 (a) (d)
Reaction and Moment Units are kips and kip-ft

RISA A NEMETSCHEK COMPANY	<Licensed Company>	SK-2
	mjlucas	Jan 09, 2025 at 09:05 AM
		MB246004 - Matthew Jones House.r3d

FRAMING CALCULATIONS

ROOF FRAMING PLAN



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PROJECT #/ NAME MATTHEW JONES HOUSE
SUBJECT UPLIFT CALCS FOR ROOF RAFTERS
SHEET____ OF____ BY____ DATE____

MAIN ROOF RAFTER UPLIFT REACTIONS

RAFTER TRIB AREA = 11.25' x 2' = 22 SF
USING C;C ~ 20 SF (ZONE 1) => 54.42 psf (LRFD)
* .6 = 32.65 psf (ASD)

UPLIFT REACTION @ EACH END OF RAFTER

11.25' x 2' x 32.65 psf = 735 lbs

DEAD LOAD (.5) = 10 psf (11.25')(2)(.5) = 113 lbs

SAFETY FACTOR

NET UPLIFT => UPLIFT − .5(DEAD)
→ 735# − 113# = 622 lbs @ RAFTERS

RAFTER TO FALSE PLATE

> USE SIMPSON HL33 HEAVY ANGLE
→ UPLIFT CAPACITY = 740 lbs > 622 lbs ✓ OK Good

FALSE PLATE TO TOP OF WALL PLATE

> #10 SCREW ASSUMING SPRUCE PINE FIR
1¼" TO 3½" MAIN MEMBER
WITHDRAWAL CAPACITY = 314 lbs

$\frac{622 \text{ lbs}}{314 \text{ lbs}} = n = 1.98 \approx$ 2 SCREWS PER CEILING JOIST LOCATION

CEILING JOIST TO LOWER WALL PLATE

USE SIMPSON HL33

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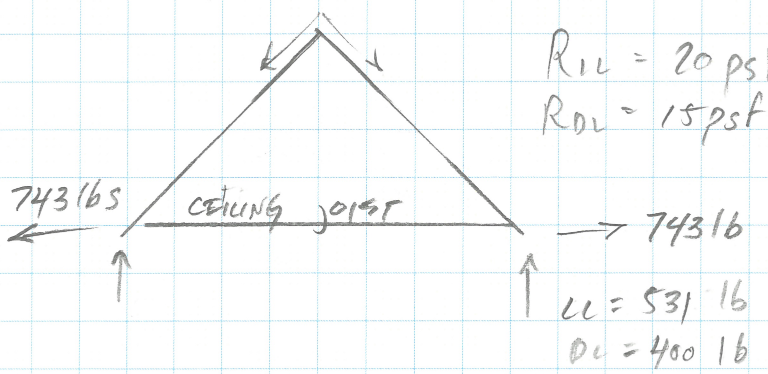
PROJECT #/ NAME Matthew Jones House
SUBJECT
Thrust @ Rafter Bearing
SHEET ____ OF ____ BY ____ DATE ____



Treating ceiling joist as collar tie, locked to top of wall

Analysis completed in Strucalc

Tension in collartie / ceiling joist = Thrust @ rafter roof bearing
→ Collartie tension = 743 lbs

Simpson heavy angle HL33, F_1 capacity = 1,040 lbs
Note 6, connectors are required @ both sides to achieve lateral loads in both directions.

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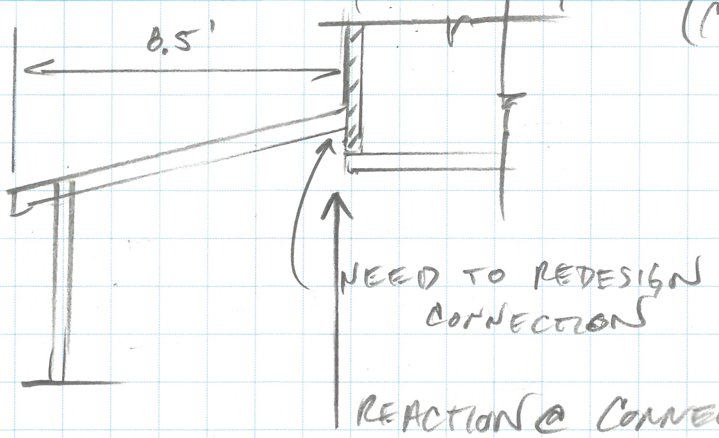
PROJECT #/ NAME Matthew Jones
SUBJECT Rear Shed Roof Framing
Rafter Connection @ North Wall
SHEET ______ OF ______ BY ______ DATE ______

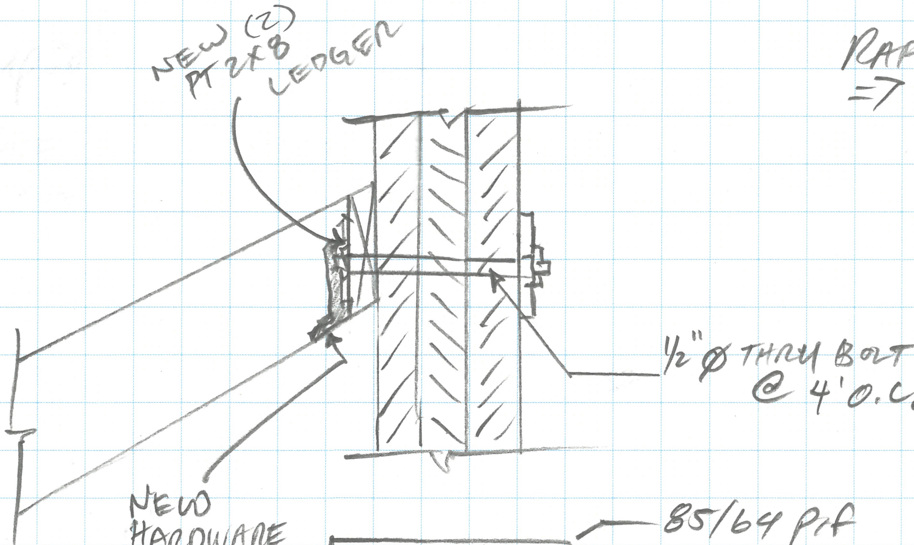
(Checking Gravity)

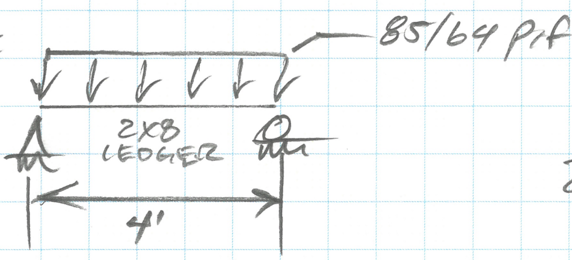
(Controls) Roof Snow = 27.2 psf
Roof Live = 20 psf
Roof Dead = 15 psf



Reaction @ Connection
=> 27.2 psf @ 8.5(.5) = SL = 116 plf
15 psf DL = 64 plf
TL = 180 plf

Rafters @ 2' o.c.
=> 180 plf (2') = 360 lb per rafter





Using Southern Pine No. 2

2x8 works, see StruCalc Rafter Ledger @ Rear Shed

Use Simpson A34 Framing Angle Rafter to Ledger

Simpson A34 Capacity = 415 lbs > 360 lbs
OK Good ✓

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PROJECT #/ NAME MATTHEW JONES
SUBJECT REAR SHED
RAFTER CONNECTION
SHEET____ OF____ BY____ DATE____

CHECKING UPLIFT

TRIB AREA $= 2' \times \frac{8.5'}{2} = 8.5$ SF USING C;C SF= $\Rightarrow$ 64.18 psf (LRFD) OR 38.5 psf (ASD)

UPLIFT $= 38.5\text{ psf}\left(\frac{8.5}{2}\right)(2') = 327$ lbs

DEAD $= 10\text{psf}\left(\frac{8.5}{2}\right)(2)(.5) = 43$ lbs

$\uparrow$ SAFETY FACTOR

NET UPLIFT = 327 − 43 lbs = 284 lbs

A_{34} Clip Capacity = 415 lbs > 284 lbs
OK Good ✓

Project: Matthew Jones Stucalc

Location: Main Building Rafter
Collar Tie
Collar Tie [2021 International Building Code(2018 NDS)
1.5 IN x 7.25 IN x 11.25 FT @ 24 O.C.
#2 - Southern Pine - Dry Use
1.5 x 9.25 Solid Sawn Lumber with minimum Ft = 575
Section Adequate By: 2.6%
Controlling Factor: Moment

The Vitruvius Project, Inc.

StruCalc Version 11.1.8.0 1/9/2025 9:41:28 AM

CAUTIONS

The design dead load deflection exceeds the default maximum of 1/4" on spans (2).

DEFLECTIONS

	Center	
Live Load	0.35	IN L/449
Dead Load	0.27	in
Total Load	0.62	IN L/257

Live Load Deflection Criteria: L/240 Total Load Deflection Criteria: L/180

RAFTER REACTIONS

	LOADS	REACTIONS
Lower Live Load @ A & B	266 plf	531 lb
Lower Dead Load @ A & B	199 plf	398 lb
Lower Total Load @ A & B	465 plf	929 lb
Collar Tie Tension		743 lb

RAFTER SUPPORT DATA

	B
Bearing Length	1.10 in

RAFTER DATA

	Interior
Span Length	11.25 ft
Unbraced Length-Bottom	13.27 ft
Rafter Pitch	7.5 :12
Collar Tie Location	7.36 ft
Roof Duration Factor	1.15
Peak Notch Depth	0.00
Base Notch Depth	0.00

RAFTER LOADING

Uniform Floor Loading

Roof Live Load:	LL =	20	psf
Roof Dead Load:	DL =	15	psf

Slope Adjusted Spans And Loads

Interior Span:	L-adj =	13.27	ft
Eave Span:	L-Eave-adj =	0	ft
Rafter Live Load:	wL-adj =	29	plf
Eave Live Load:	wL-Eave-adj =	29	plf
Rafter Dead Load:	wD-adj =	25	plf
Rafter Total Load:	wT-adj =	54	plf
Eave Total Load:	wT-Eave-adj =	54	plf

MATERIAL PROPERTIES

	Base Values		Adjusted	
Bending Stress:	Fb =	925 psi	Fb' =	1223 psi
	Cd=1.15 CF=1.00 Cr=1.15			
Shear Stress:	Fv =	175 psi	Fv' =	201 psi
	Cd=1.15			
Modulus of Elasticity:	E =	1400 ksi	E' =	1400 ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	565 psi	Fc - ⊥' =	565 psi

Controlling Moment: 1306 ft-lb
5.622 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: -787 lb
11.024 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	12.81 in3	13.14 in3
Area (Shear):	5.87 in2	10.88 in2
Moment of Inertia (deflection):	33.41 in4	47.63 in4
Moment:	1306 ft-lb	1340 ft-lb
Shear:	-787 lb	1459 lb

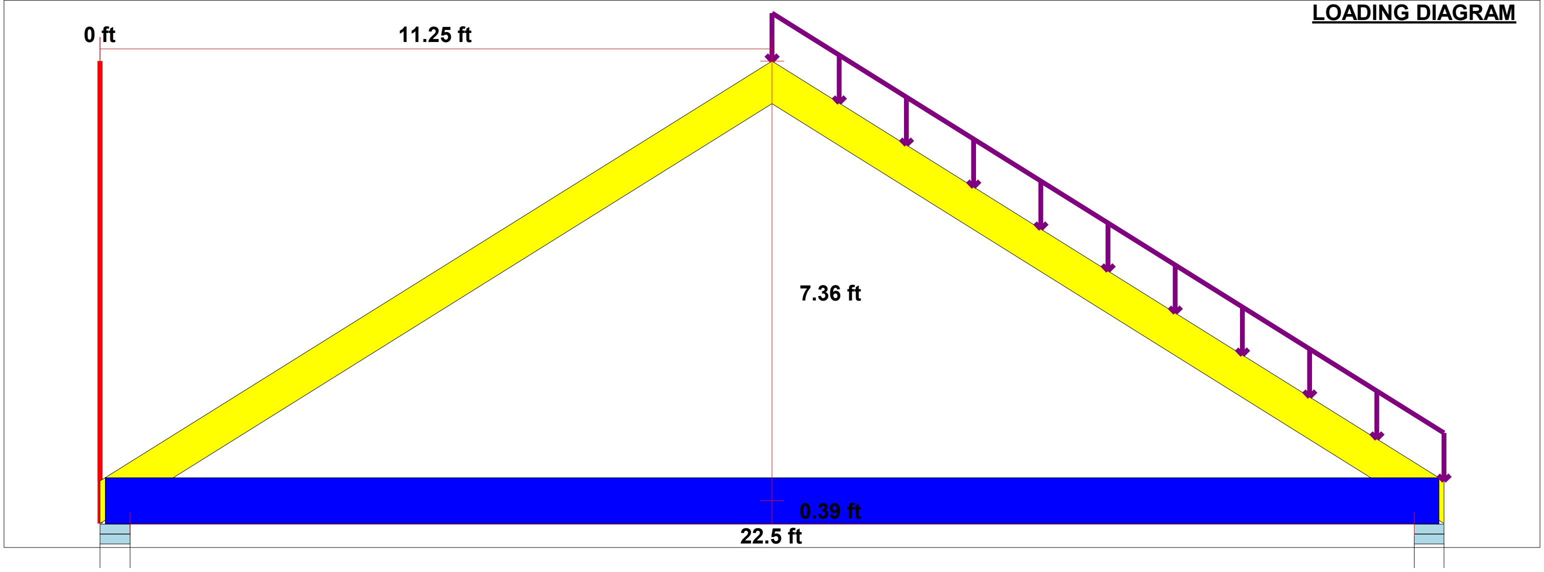
COLLAR TIE DESIGN

1.5 x 9.25 Solid Sawn Lumber with minimum Ft = 575

	Base Values		Adjusted
Tension Parallel to Grain	Ft =	575 psi	Ft' = 992 psi
	Cd=1.15 Cf=0.00		

Collar Tie Location	7.36 ft
Collar Tie Tension	743 lb
Collar Tie Capacity	13762 lb
Nailing Required @ Both Ends	
16d Common	5 Nails
16d Sinker	6 Nails
16d Box	7 Nails

LOADING DIAGRAM



Project: Matthew Jones Stucalc

Location: Rafter Ledger @ Rear Shed
Multi-Loaded Multi-Span Beam
Multi-Loaded Multi-Span Beam [2021 International Building Code(2018 NDS)
1.5 IN x 7.25 IN x 4.0 FT
#2 - Southern Pine - Dry Use
Section Adequate By: 234.1%
Controlling Factor: Moment

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page / of

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DEFLECTIONS	Center	
Live Load	0.01	IN L/6539
Dead Load	0.01	in
Total Load	0.01	IN L/3667

Live Load Deflection Criteria: L/360 Total Load Deflection Criteria: L/240

REACTIONS	A	B
Live Load	170 lb	170 lb
Dead Load	133 lb	133 lb
Total Load	303 lb	303 lb
Bearing Length	0.36 in	0.36 in

BEAM DATA	Center
Span Length	4 ft
Unbraced Length-Top	0 ft
Unbraced Length-Bottom	4 ft
Live Load Duration Factor	1.00
Notch Depth	0.00

MATERIAL PROPERTIES
#2 - Southern Pine

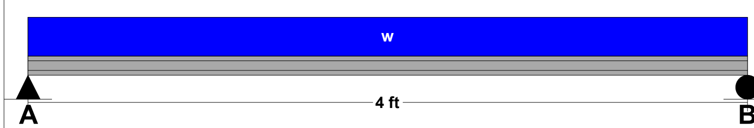
	Base Values		Adjusted	
Bending Stress:	Fb =	925 psi	Fb' =	925 psi
	Cd=1.00 CF=1.00			
Shear Stress:	Fv =	175 psi	Fv' =	175 psi
	Cd=1.00			
Modulus of Elasticity:	E =	1400 ksi	E' =	1400 ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	565 psi	Fc - ⊥' =	565 psi

Controlling Moment: 303 ft-lb
2.0 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Controlling Shear: 303 lb
At left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	3.93 in3	13.14 in3
Area (Shear):	2.6 in2	10.88 in2
Moment of Inertia (deflection):	3.12 in4	47.63 in4
Moment:	303 ft-lb	1013 ft-lb
Shear:	303 lb	1269 lb

LOADING DIAGRAM



UNIFORM LOADS	Center
Uniform Live Load	85 plf
Uniform Dead Load	64 plf
Beam Self Weight	3 plf
Total Uniform Load	152 plf

ConnectionCalc Results

Analysis Type:

Design Method:	Allowable Stress
Connection Loading:	Withdrawal
Fastener Type:	Wood Screw

Main Member Parameters:

Main Member material category:	Solid Lumber/Timber
Type:	Spruce-Pine-Fir
Main Member Thickness:	3-$^1/_2$ in
Load to Grain Angle:	90 deg

Side Member Parameters:

Side Member material category:	Solid Lumber/Timber
Type:	Spruce-Pine-Fir
Side Member Thickness:	1-$^1/_4$ in
Load to Grain Angle:	90 deg

Wood Screw Parameters:

Size:	No.10
Length:	3-$^1/_2$ in

Analysis Factors:

Load Duration (CD):	1.6
Wet Service (CM):	1
Temperature Factor (Ct):	1

Results

Adjusted ASD Capacity:	314

Notes

Tip length:	2x diameter
End grain installed screws:	End grain not applicable
Fastener pull through capacity:	Head pull through not addressed
Disclaimer:	While every effort has been made to insure the accuracy of the information presented, and special effort has been made to assure that the information reflects the state-of-the-art, neither the American Wood Council nor its members assume any responsibility for any particular design prepared from this Connection Calculator. Those using this Connection Calculator assume all liability from its use.

SIMPSON Strong-Tie®

HL

Heavy Angle and Gusset

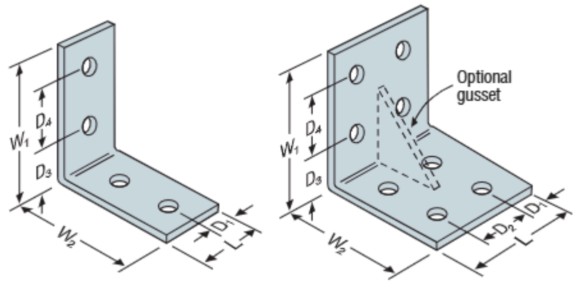
Versatile angle gussets and heavy angles promote standardization and construction economy, and are compatible with Simpson Strong-Tie structural hardware.

Finish: 7 gauge models — galvanized;
3 gauge models — Simpson Strong-Tie gray paint.
May be ordered HDG or black powder coat (add HDG or PC to model no.).

Options:

- Gussets may be added to HL models when L ≥ 5" (specify G after model number, as in HL46G).

Codes: See p. 13 for Code Reference Key Chart

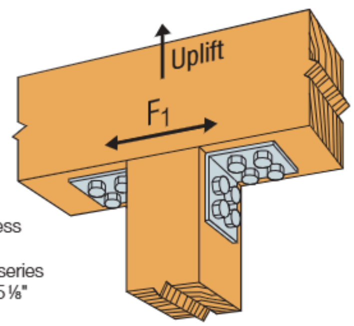


HL53 HL55

These products are available with additional corrosion protection. For more information, see p. 16.

Model No.	Ga.	Dimensions (in.)						Bolts (Total)		DF/SP Allowable Loads		Code Ref.
		W_1 and W_2	L	D_1	D_2	D_3	D_4	Qty.	Dia.	Uplift (160)	F_1 (160)	
Single Row Angles												
HL33	7	3¼	2½	1¼	—	2	—	2	½	740	1,040	—
HL35	7	3¼	5	1¼	2½	2	—	4	½	740	1,310	
HL37	7	3¼	7½	1¼	2½	2	—	6	½	740	1,310	
HL43	3	4¼	3	1½	—	2¾	—	2	¾	1,275	1,445	
HL46	3	4¼	6	1½	3	2¾	—	4	¾	1,275	1,680	
HL49	3	4¼	9	1½	3	2¾	—	6	¾	1,275	1,680	
Double Row Angles												
HL53	7	5¾	2½	1¼	—	2	2½	4	½	740	1,310	—
HL55	7	5¾	5	1¼	2½	2	2½	8	½	740	1,310	
HL57	7	5¾	7½	1¼	2½	2	2½	12	½	740	1,310	
HL73	3	7¼	3	1½	—	2¾	3	4	¾	2,445	2,885	
HL76	3	7¼	6	1½	3	2¾	3	8	¾	2,445	4,310	
HL79	3	7¼	9	1½	3	2¾	3	12	¾	2,445	4,310	

1. See pp. 276–277 for Straps and Ties General Notes.
2. For SPF/HF lumber, use 0.85 x DF/SP allowable loads.
3. Parts should be centered on the face of the member, which is at least as wide as the angle, to which they are attached.
4. Wood members for the "3" and "5" series must have a minimum thickness of 3½" for table loads to apply.
5. Wood members for the "4" and "7" series must have a minimum thickness of 5⅛" for table loads to apply.
6. Allowable loads are for a single connector. Uplift loads may be doubled when using two connectors. Connectors are required on both sides to achieve lateral loads in both directions. Lateral loads may not be doubled.
7. Lag screws of equal diameter (minimum 5" long) may be substituted for bolts in the beam with no reduction in load.
8. All references to bolts are for structural-quality through bolts (not lag screws or carriage bolts) equal to or better than ASTM A307, Grade A.



Typical HL55 Installation

Z

Clip

The Z clip secures 2x4 flat blocking between joists or trusses to support sheathing.

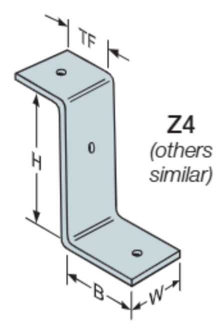
Material: See table

Finish: Galvanized

Installation:

- Use all specified fasteners; see General Notes.
- Z clips do not provide lateral stability. Do not walk on stiffeners or apply load until diaphragm is installed and nailed to stiffeners.

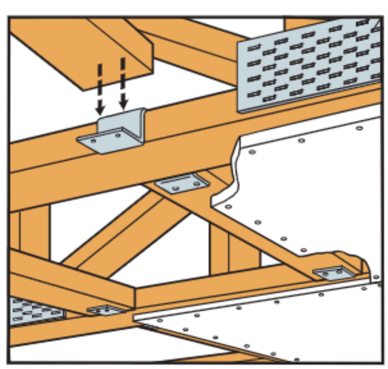
Codes: See p. 13 for Code Reference Key Chart



Z4 *(others similar)*

Model No.	Ga.	Dimensions (in.)				Fasteners[1] (Total) (in.)	DF/SP Allowable Download (100/115/125/160)	Code Ref.
		W	H	B	TF			
Z2	20	2⁵⁄₁₆	1½	1⅜	1⅜	(4) 0.148 x 1½	420	IBC®, FL, LA
Z4	12	1½	3½	2⅛	1¾	(2) 0.162 x 3½	420	
Z28	28	2⁵⁄₁₆	1½	1⅜	1⅜	0.148 x 1½[1]	—	—
Z38	28	2⁵⁄₁₆	2½	1⅜	1⅜	0.148 x 1½[1]	—	
Z44	12	2½	3½	2	1⅜	(4) 0.162 x 3½	775	IBC, FL, LA

1. Z28 and Z38 do not have nail holes. Fastener quantity and type shall be per designer.
2. Z4 loads apply with a nail in the top and a nail in the seat.
3. For SPF/HF lumber, use 0.86 x DF/SP allowable loads.
4. **Fasteners:** Nail dimensions are listed diameter by length. See pp. 23–24 for fastener information.


Typical Z2 Installation

LTP4/LTP5/A34/A35

SIMPSON Strong-Tie

Framing Angles and Plates

The larger LTP5 spans subfloor at the top of the blocking or rim board. The embossments enhance performance.

The LTP4 lateral tie plate transfers shear forces for top plate-to-rim board or blocking connections. Nail holes are spaced to prevent wood splitting for single and double top-plate applications. May be installed over plywood sheathing.

The A35 angle's exclusive bending slot allows instant, accurate field bends for all two- and three-way ties. Balanced, completely reversible design permits the A35 to secure a great variety of connections.

Material: LTP4/LTP5 — 20 gauge; all others — 18 gauge

Finish: Galvanized. Some products available in stainless steel or ZMAX® coating.

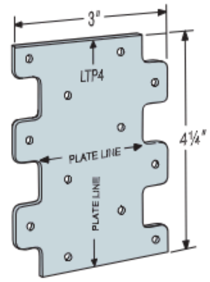
Installation:

- Use all specified fasteners; see General Notes
- A35 — Bend one time only

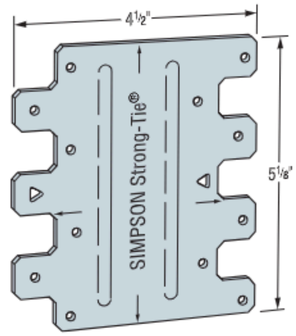
Codes: See p. 13 for Code Reference Key Chart

Web Applications:
Visit **app.strongtie.com/rws** to access our Roof-to-Wall Selector web application.

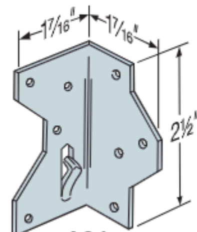
RWS Roof-to-Wall Selector



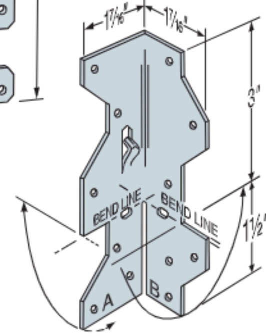
LTP4



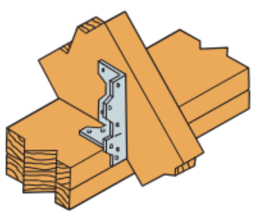
LTP5



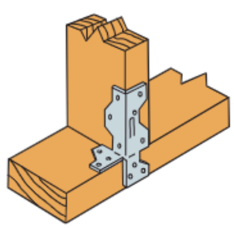
A34



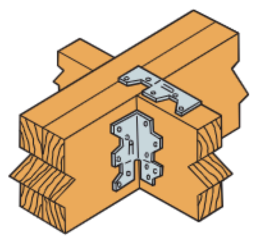
A35



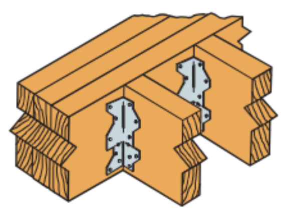
Joists to Plate with A Leg Inside



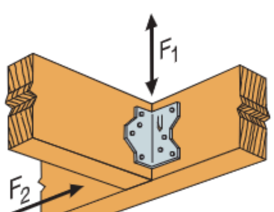
Studs to Plate with B Leg Outside



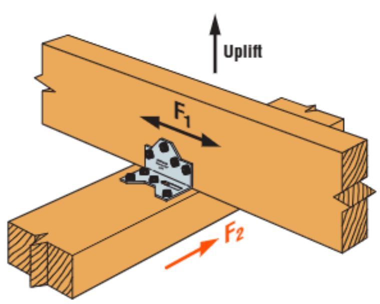
Joists to Beams



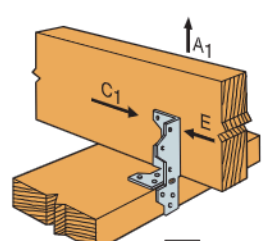
Ceiling Joists to Beam



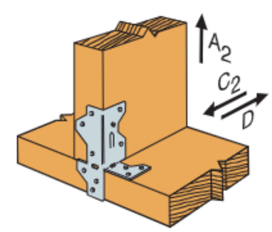
1 A34



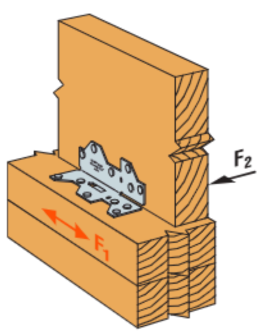
1 A34 Installed with SD Screws



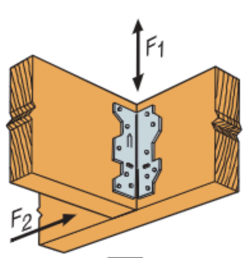
2 A35



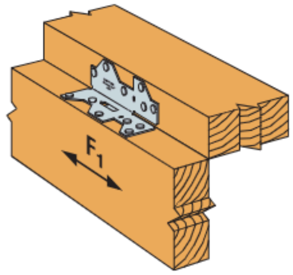
3 A35



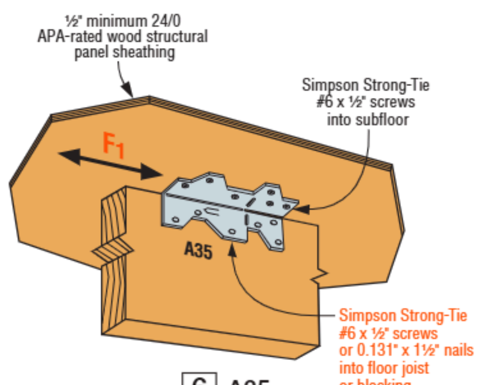
4 A35



4 A35



5 A35



6 A35

Framing Angles and Plates (cont.)

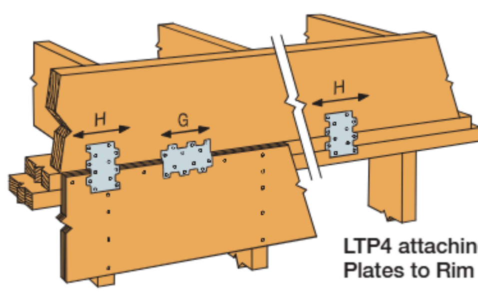
These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

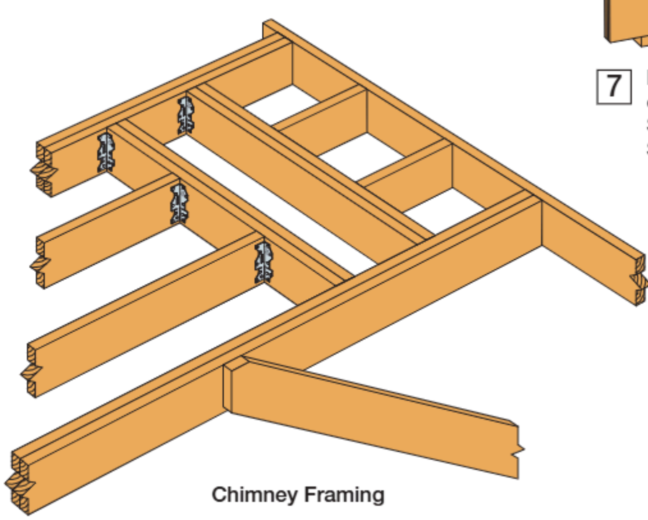
Model No.	Configuration Type of Connection	Fasteners (in.)	Direction of Load	DF/SP Allowable Loads Floor (100)	DF/SP Allowable Loads Roof (125)	DF/SP Allowable Loads (160)	SPF/HF Allowable Loads Floor (100)	SPF/HF Allowable Loads Roof (125)	SPF/HF Allowable Loads (160)	Code Ref.
SS A34	1	(8) 0.131 x 1½	F_1	395	480	545	340	415	480	IBC®, FL, LA
			F_2[6]	395	430	430	340	370	370	
		(8) #9 x 1½" SD	F_1	640	640	640	550	550	550	
			F_2[6]	495	495	495	425	425	425	
			Uplift	240	240	240	170	170	170	—
SS A35	2	(9) 0.131 x 1½	A_1	295	350	350	255	300	300	IBC, FL, LA
			E	295	360	385	255	310	330	
			C_1	185	185	185	160	160	160	
	3	(12) 0.131 x 1½	A_2	295	325	325	255	280	280	
			C_2	295	330	330	255	285	285	
			D	225	225	225	195	195	195	
	4	(12) 0.131 x 1½	F_1	590	650	650	510	560	560	
			F_2[6]	590	670	670	510	575	575	
	5	(12) 0.131 x 1½	F_1	555	555	555	475	475	475	
	6	(12) PH612I	F_1	420	420	420	360	360	360	—
LTP4	7	(12) 0.131 x 1½	G	580	715	715	500	615	615	IBC, FL, LA
			H	525	525	525	450	450	450	
LTP5	8	(12) 0.131 x 1½	G	565	565	565	485	485	485	
			H	490	490	490	420	420	420	

1. Allowable loads are for one angle. When angles are installed on each side of the joist, the minimum joist thickness is 3".
2. Some illustrations show connections that could cause cross-grain tension or bending of the wood during loading if not reinforced sufficiently. In this case, mechanical reinforcement should be considered.
3. LTP4 can be installed over ⅜" wood structural panel sheathing with 0.131" x 1½" nails and achieve 0.72 of the listed load, or over ½" sheathing and achieve 0.64 of the listed load. 0.131" x 2½" nails will achieve 100% load.
4. LTP4 satisfies the IRC® continuously sheathed portal frame (CS-PF) framing anchor requirements when installed over raised wood floor framing per Figure R602.10.6.4.
5. The LTP5 may be installed over wood structural panel sheathing up to ½" thick using 0.131" x 1½" nails with no reduction in load.
6. Connectors are required on both sides to achieve F_2 loads in both directions.
7. A34 and A35 installed with 0.131" x 1½" nails onto 1¼" LSL material will achieve 0.90 of the listed F_1 and F_2 loads.
8. **Fasteners:** Nail dimensions are listed diameter by length. SD screws are Simpson Strong-Tie Strong-Drive SD Connector screws. PH612I is a pan-head #6 x ½" screw available from Simpson Strong-Tie. See pp. 23–24 for other nail sizes and information.

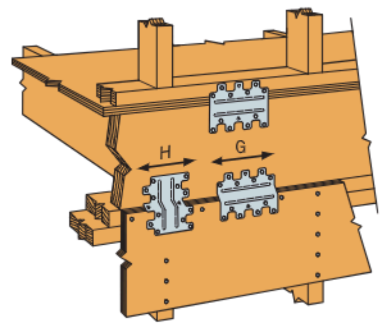


LTP4 attaching Top Plates to Rim Board

7 LTP4 Installed over Wood Structural Panel Sheathing



Chimney Framing



8 LTP5 Installed over Wood Structural Panel Sheathing or Attaching Plate to Rim Board

HRS/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI/ST

SIMPSON Strong-Tie

Strap Ties

Straps are designed to transfer tension loads in a wide variety of applications.

HRS — Heavy strap designed for installation on the edge of 2x members. The HRS416Z installs with Strong-Drive® SDS Heavy-Duty Connector screws.

HTP — Heavy tie plate designed for installation on the side of 2x4 or larger members.

LSTA and MSTA — Designed for use on the edge of 2x members, with a nailing pattern that reduces the potential for splitting.

LSTI and MSTI — Light and medium straps that are suitable where pneumatic-nailing is necessary through diaphragm decking and wood chord open-web trusses.

MST — High-capacity strap that can be installed with either nails or bolts. Suitable for double 2x member connections or greater.

MSTC — High-capacity strap that utilizes a staggered nail pattern to help minimize wood splitting. Nail slots have been countersunk to provide a lower nail head profile.

ST — Light and medium precut straps for quick installation.

Finish: Galvanized. Some products are available in stainless steel, ZMAX® coating or black powder coat (add PC to SKU).

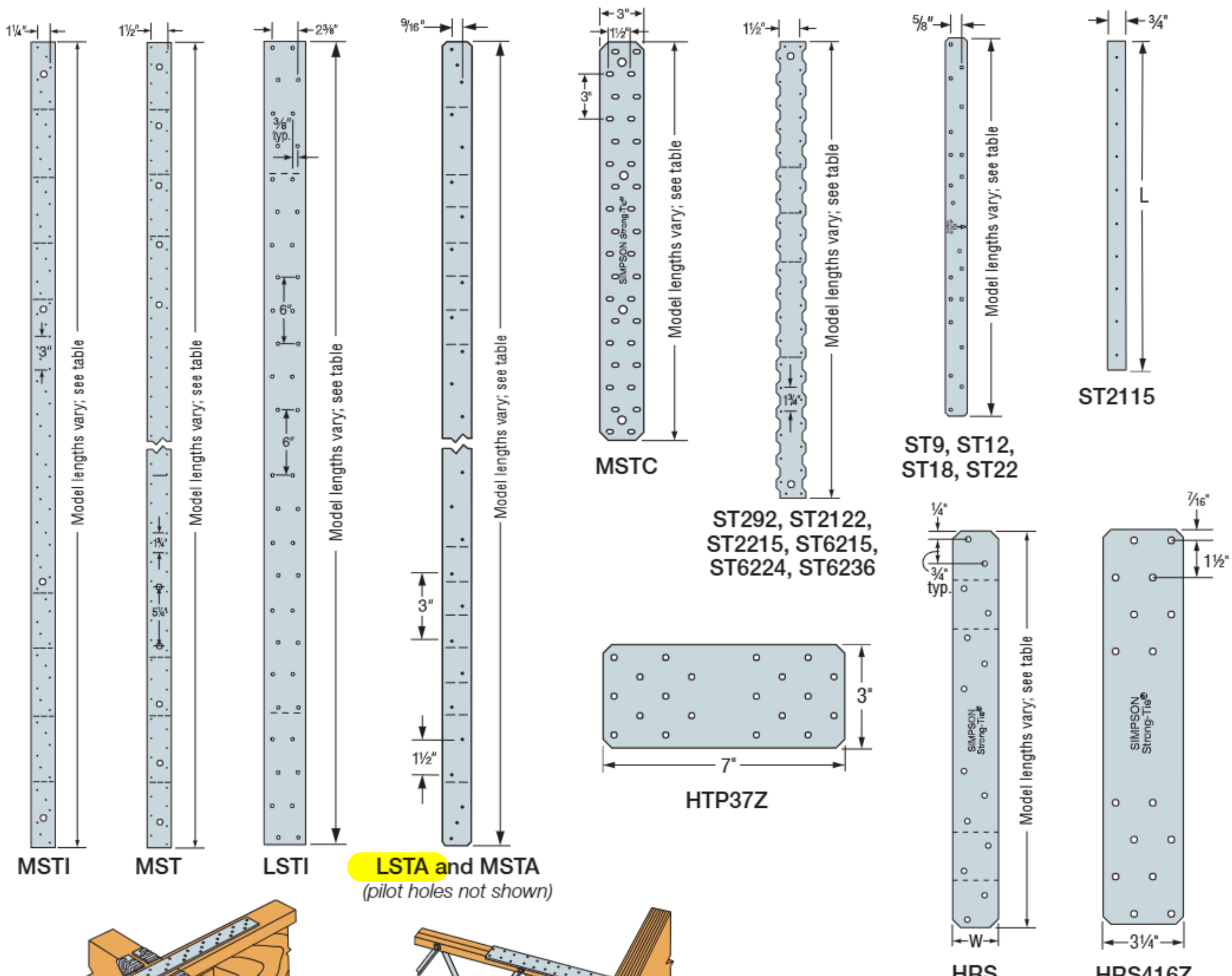
Installation: Use all specified fasteners; see General Notes.

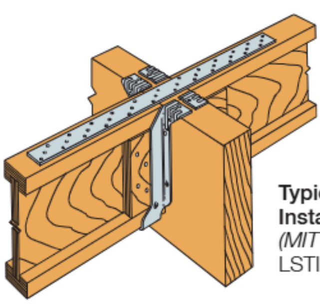
Options: Special sizes can be made to order; contact Simpson Strong-Tie

Codes: See p. 13 for Code Reference Key Chart

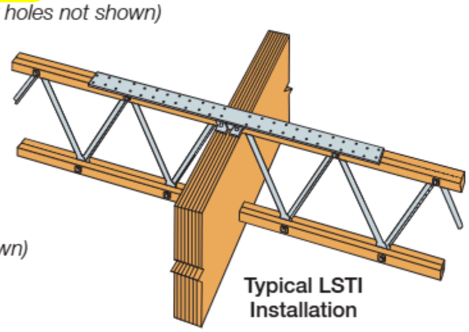
MSTC and RPS meet code requirements for reinforcing cut members (16 gauge) at top plate and RPS at sill plate.
International Residential Code® — 2012 /2015/2018/2021 R602.6.1
International Building Code® — 2012 2308.9.8; 2015/2018/2021 2308.5.8

(For RPS, refer to p. 321. For CTS218 compression and tension strap, see p. 319.)

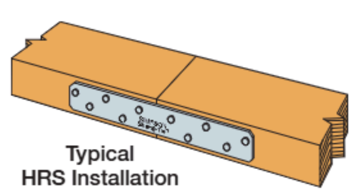




Typical MSTI Installation
(MIT hanger shown)
LSTI similar



Typical LSTI Installation



Typical HRS Installation

HRS/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI/ST

SIMPSON Strong-Tie

Strap Ties (cont.)

Codes: See p. 13 for Code Reference Key Chart

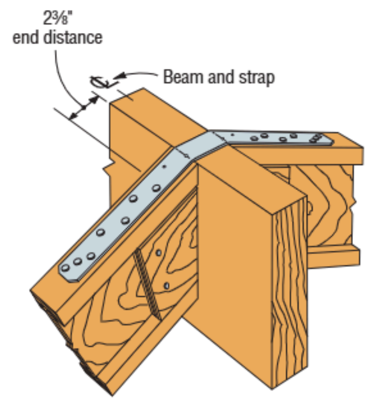
These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

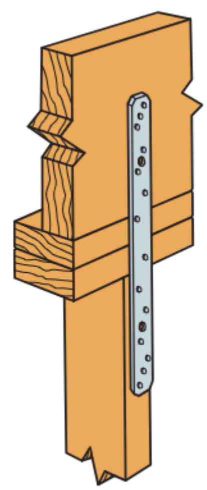
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

	Model No.	Ga.	Dimensions (in.)		Fasteners (Total) (in.)	DF/SP Allowable Tension Loads	SPF/HF Allowable Tension Loads	Code Ref.
			W	L		(160)	(160)	
	ST2115	20	¾	16 5/16	(10) 0.162 x 2½	660	660	IBC®, FL, LA
	LSTA9	20	1¼	9	(8) 0.148 x 2½	740	635	IBC®, FL, LA
	LSTA12	20	1¼	12	(10) 0.148 x 2½	925	795	IBC®, FL, LA
	LSTA15	20	1¼	15	(12) 0.148 x 2½	1,110	955	IBC®, FL, LA
	LSTA18	20	1¼	18	(14) 0.148 x 2½	1,235	1,115	IBC®, FL, LA
	LSTA21	20	1¼	21	(16) 0.148 x 2½	1,235	1,235	IBC®, FL, LA
	LSTA24	20	1¼	24	(18) 0.148 x 2½	1,235	1,235	IBC®, FL, LA
	LSTA30	18	1¼	30	(22) 0.148 x 2½	1,640	1,640	IBC®, FL, LA
	LSTA36	18	1¼	36	(24) 0.148 x 2½	1,640	1,640	IBC®, FL, LA
	MSTA9	18	1¼	9	(8) 0.148 x 2½	750	650	IBC®, FL, LA
SS	MSTA12	18	1¼	12	(10) 0.148 x 2½	940	810	IBC®, FL, LA
	MSTA15	18	1¼	15	(12) 0.148 x 2½	1,130	970	IBC®, FL, LA
SS	MSTA18	18	1¼	18	(14) 0.148 x 2½	1,315	1,135	IBC®, FL, LA
	MSTA21	18	1¼	21	(16) 0.148 x 2½	1,505	1,295	IBC®, FL, LA
SS	MSTA24	18	1¼	24	(18) 0.148 x 2½	1,640	1,460	IBC®, FL, LA
	MSTA30	16	1¼	30	(22) 0.148 x 2½	2,050	1,825	IBC®, FL, LA
SS	MSTA36	16	1¼	36	(26) 0.148 x 2½	2,050	2,050	IBC®, FL, LA
	MSTA49	16	1¼	49	(26) 0.148 x 2½	2,020	2,020	IBC®, FL, LA
	ST9	16	1¼	9	(8) 0.162 x 2½	885	765	IBC®, FL, LA
	ST12	16	1¼	11⅝	(10) 0.162 x 2½	1,105	955	IBC®, FL, LA
	ST18	16	1¼	17¾	(14) 0.162 x 2½	1,420	1,335	IBC®, FL, LA
	ST22	16	1¼	21⅝	(18) 0.162 x 2½	1,420	1,420	IBC®, FL, LA
	HRS6	12	1⅜	6	(6) 0.148 x 2½	605	530	IBC®, FL, LA
	HRS8	12	1⅜	8	(10) 0.148 x 2½	1,010	880	IBC®, FL, LA
	HRS12	12	1⅜	12	(14) 0.148 x 2½	1,415	1,230	IBC®, FL, LA
	ST292	20	2 1/16	9 5/16	(12) 0.162 x 2½	1,260	1,120	IBC®, FL, LA
	ST2122	20	2 1/16	12 13/16	(16) 0.162 x 2½	1,530	1,510	IBC®, FL, LA
	ST2215	20	2 1/16	16 5/16	(20) 0.162 x 2½	1,875	1,875	IBC®, FL, LA
	ST6215	16	2 1/16	16 5/16	(20) 0.162 x 2½	2,090	1,910	IBC®, FL, LA
	ST6224	16	2 1/16	23 5/16	(28) 0.162 x 2½	2,535	2,535	IBC®, FL, LA
	ST6236	14	2 1/16	33 13/16	(40) 0.162 x 2½	3,845	3,845	IBC®, FL, LA
	MSTI26	12	2 1/16	26	(26) 0.148 x 1½	2,745	2,380	IBC®, FL, LA
	MSTI36	12	2 1/16	36	(36) 0.148 x 1½	3,800	3,295	IBC®, FL, LA
	MSTI48	12	2 1/16	48	(48) 0.148 x 1½	5,070	4,390	IBC®, FL, LA
	MSTI60	12	2 1/16	60	(60) 0.148 x 1½	5,070	5,070	IBC®, FL, LA
	MSTI72	12	2 1/16	72	(72) 0.148 x 1½	5,070	5,070	IBC®, FL, LA
	HTP37Z	16	3	7	(20) 0.148 x 1½	900	690	IBC®, FL, LA
	MSTC28	16	3	28¼	(36) 0.148 x 3¼	3,460	2,990	IBC®, FL, LA
	MSTC40	16	3	40¼	(52) 0.148 x 3¼	4,735	4,315	IBC®, FL, LA
	MSTC52	16	3	52¼	(62) 0.148 x 3¼	4,735	4,735	IBC®, FL, LA
	MSTC66	14	3	65¾	(68) 0.148 x 3¼	5,850	5,850	IBC®, FL, LA
	MSTC78	14	3	77¾	(76) 0.148 x 3¼	5,850	5,850	IBC®, FL, LA
	HRS416Z	12	3¼	16	(16) ¼ x 1½ SDS	2,835	2,305	—
	LSTI49	18	3¾	49	(32) 0.148 x 1½	2,970	2,560	IBC, FL, LA
	LSTI73	18	3¾	73	(48) 0.148 x 1½	4,205	3,840	IBC, FL, LA

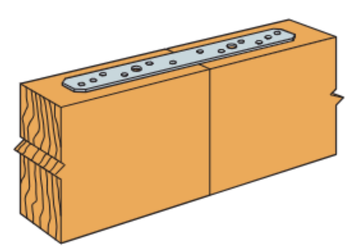
1. See pp. 276–277 for Straps and Ties General Notes.
2. **Fasteners:** Nail dimensions are listed diameter by length. SDS screws are Simpson Strong-Tie Strong-Drive SDS Heavy-Duty Connector screws. See pp. 23–24 for fastener information.



Typical LSTA Installation
(hanger not shown)
Bend strap one time only, max. 12/12 joist pitch.



Typical LSTA18 Installation



Typical MSTA15 Installation

C. MCWB Survey Report

The Matthew Jones House
Joint Base Langley-Eustis:

An Update on Its Condition, with Recommendations

Prepared by

Mesick Cohen Wilson Baker Architects
Albany, New York and Williamsburg, Virginia

for

Resource Management Associates
Onancock, Virginia

December 2018

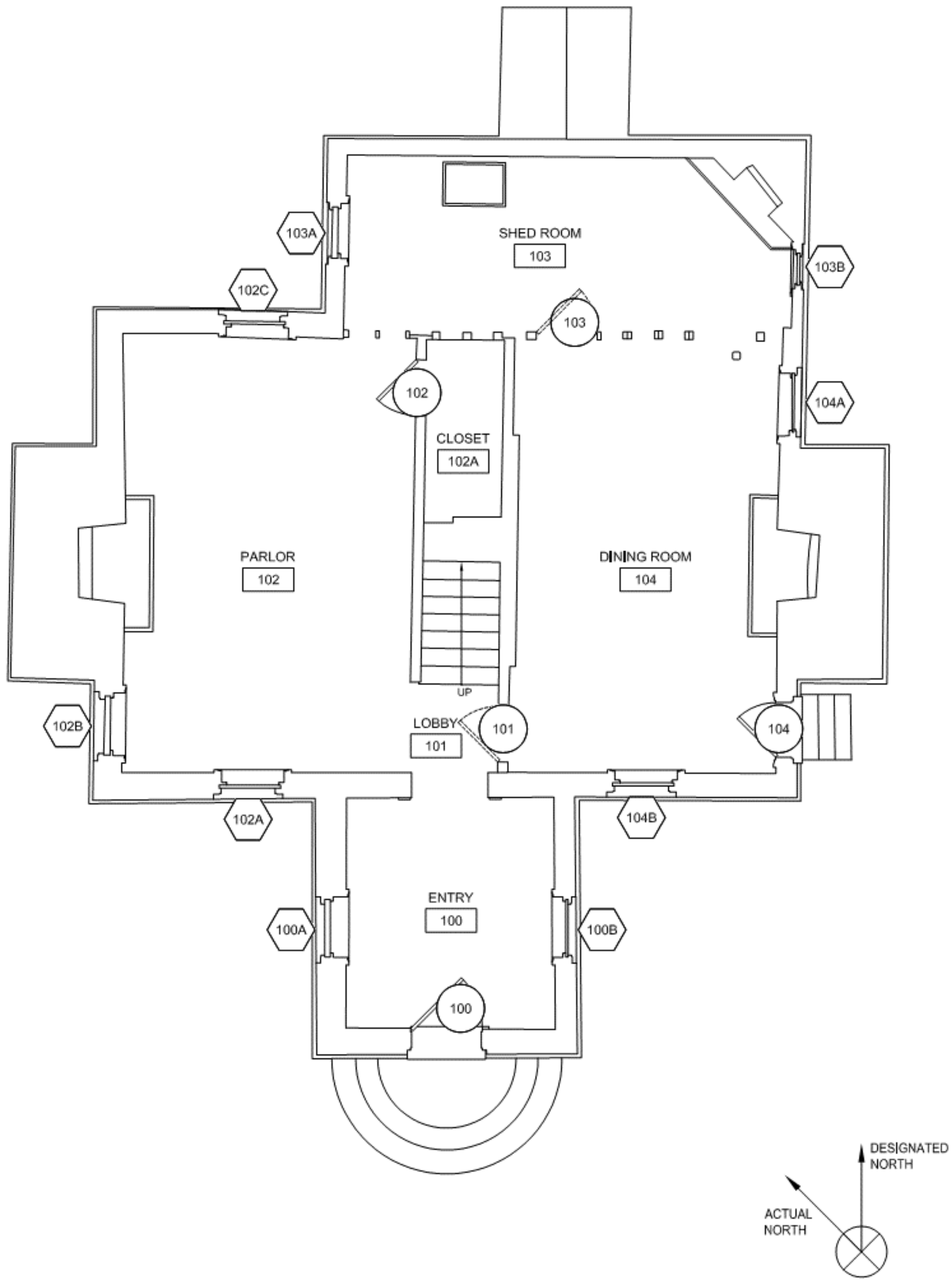
INTRODUCTION

This report documents a physical survey of the Matthew Jones House, an early eighteenth-century brick dwelling situated on the northwestern margin of Mulberry Island, now a part of Joint Base Langley-Eustis, near Newport News Virginia.

Prepared by Mesick Cohen Wilson Baker Architects (MCWB) in collaboration with Resource Management Associates, the study presents prioritized recommendations for the on-going preservation of this important building. Under direction of the U.S. Army Corps of Engineers, Norfolk District, the structure was treated in 1993 by the National Center for Preservation Technology and Training, National Park Service, Williamsport, MD (NPCTT). This effort followed a 1991 report on the building's history by the William and Mary Center for Archaeological Research.

Since that time, the house has been the subject of three condition reports, one in 2014, by the U. S. Army Corps of Engineers Engineer Research and Development Center (EDRC), another, more limited study by the U.S. Army Corps of Engineers in 2015, and finally, an extensive report by the NCPTT, also in 2015. The latter document is a comprehensive, detailed study. None of the recommendations presented in these reports have yet been implemented. The purpose of the present report is to update the 2015 NCPTT study, offering prioritized recommendations for immediate measures, and also for work to be completed in the longer term.

Having reviewed past reports, Jeff Baker, Eric Kuchar and Mark Wenger of MCWB visited the site on September 26, 2018 to examine the building, with HABS plans, and the 2015 NCPTT report in hand. Also present were Michael Ryan of Resource Management Associates, Sal DiPietro, of Springpoint Structural, and Chris McDaid, Archaeologist and Cultural Resource Manager at Joint Base Langley- Eustis. Later, Chris Keefe and Brianna Baker of MCWB visited the site to perform laser scanning of the building, inside and out.



ORIENTATION

The ground-floor doorway to the "Entry" or tower (the dwelling's original "front door") faces southwest. For purposes of discussion, this side the building is assumed to be the present front and is designated as facing *south*. Thus, the shed is assumed to stand at the rear of the building, and is designated as facing *north*. Accordingly, the two gable ends are designated east and west.

AN APPROACH

The structure known today as the Matthew Jones House embodies a succession of building campaigns. In the following section, we describe why the early brick house bound up in today's structure is so important to the history of Virginia architecture. Preserving that house is deeply important, a conviction informs all the recommendations that follow.

Many of the structural problems now evident at the Matthew Jones House can be traced back to the structure's earliest years. Dendrochronology indicates that the first house was built c. 1725 as a timber-frame structure with a large brick chimney at each gable end. On the death of Matthew Jones in 1728, title to the property passed to his minor son, Scervant Jones, for whom the house seems to have been modified extensively. Dendrochronology indicates that new framing was incorporated into the extant building in 1730.[1] Presumably, this was when the house was wrapped in brick. During that process, the wooden frame of the lower story was removed entirely. However, the frame's upper half (wall plates, joists, and rafters) remained and was encapsulated by the new brickwork. The evidence for this is clearest at the west gable end, just north of the chimney, where a first-period joist and its associated wall plate are still present.[2]

The 1730 brickwork merely abutted the earlier brick chimneys, and so did not bond into them. That is to say, the new brick walls remained essentially independent of the chimneys. And though the brick gables appear to be eleven inches thick throughout, they are substantially diminished where the masonry passes over first-period end joists. Each of these conditions seriously weakens the structure.

In 1893, the present second story was added to the house then extant. Early photographs and physical evidence indicate that the northwest corner of the structure had already settled approximately two inches by that time. As a consequence of this movement, the 1730, single-story corner rotated outward to the northwest, cracking the walls at their weakest points, above the north window and in the west gable, where the masonry wrapped around an early knee wall stud. An original north window in the 1730s west wall has since been filled with brickwork, possibly in an effort to arrest this movement.[3]

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[1] Heikkenen, Herman J. et al. *The Last Year of Tree Growth for Selected Timbers within the Tower and Attic of the Matthew Jones House as Derived by the Key-Year Dendrochronology Technique.* (Blacksburg, VA: American Institute of Dendrochronology, 1986), pp. 7-10. Heikkenen used the "Key-Year" method of dating, focused on years of significant agreement among available samples, based on each year's growth, measured relative to the previous year. Heikkenen measured the degree of correlation between the building samples and the "area pattern" (drawn from standing trees of known date), using the X^2 and K statistical tests. He found significant correlation between the two data sets in each instance—6.43 to 13.39 and .88 to 1.00, respectively. See pp. 17-24.

[2] *A Historic Preservation Plan for the Matthew Jones House, Fort Eustis, Virginia…*," Williamsburg, Virginia: The College of William and Mary Center for Archaeological Research, 1991), pp. 43-72.

[3] *Ibid*, pp. 72-88.

As the 1893 masons raised the house to two full stories, they attempted to plumb the displaced corner, flushing up their first course with the old wall below, then curved their work inward incrementally, to attain a cumulative adjustment of two inches at the top of the new wall. As a result, the entire wall now appears to bulge slightly outward at the middle, the lower, earlier wall leaning out, while the later, upper wall leans in. Consequently, the wall is eccentrically loaded, with a hinge point in the center. Finally, it seems that the added loads imposed on the northwest corner at this time accelerated settlement at that point, so that existing cracks, previously limited to the earlier masonry, now telegraphed upward into the new work. Once again, these cracks appeared at (and propagated from) the weakest locations in the wall, particularly where the 1728 brick gable had encapsulated a first-period knee-wall stud.

If the Matthew Jones House is to be preserved sustainably, its independent masonry systems must be induced to act as one. It will be equally important to exclude water from the building, which threatens the integrity of the masonry while attacking valued wood and plaster components. The following options have been formulated to serve these priorities.

SUMMARY RECOMMENDATIONS:

TWO OPTIONS

Believing that the 1893 alterations amplified the problems of this building significantly, we have developed TWO OPTIONS for its protection.

OPTION 1

> Restore the brick building to its late 1730s appearance, removing the later second story to mitigate the problems it has produced. This choice reflects a conviction that the building's significance lies primarily in the once-ubiquitous, earth-fast building tradition it represents. That type, prevalent in the Chesapeake colonies through the second quarter of the 18^{th} century, has since vanished from Virginia entirely. If late-19^{th} and 20^{th}-century building systems complicate or jeopardize this all-important aspect of the building, we believe their removal can be justified, despite the valid and competing importance of later work. To say it another way, many Virginia buildings tell a post-1883 story, but here in Virginia, only the Matthew Jones House tells the earlier, earth-fast story *above ground*.

OPTION 2

> Preserve the structure as it exists, treating extant problems in the context of current building systems. This approach acknowledges the expanded house as reflecting a later, dynamic era in the history in Warrick County and the Lower Peninsula. De-populated after the Civil War, the region eventually experienced an extraordinary economic expansion, initiated and sustained by the coming of the C&O Railroad in 1883, by Collis P. Huntington's development of what is now Newport News, and by the growing presence of the nation's armed forces during and after World War I. The Matthew Jones House attained its present form in 1893, when a full-height upper story was created, and all interior finishes were renewed. These changes reflected the region's transformed economy. The house really tells two stories. Option 2 serves both.

To assist in choosing between the two preservation options, the following section explores each alternative in greater detail. WHICHEVER THE OPTION CHOSEN, IT WILL BE NECESSARY TO RE-ESTABLISH, AS SOON AS POSSIBLE, THE LATERAL BRACING LOST WHEN THE FLOOR JOISTS OVER ROOM 102 WERE REMOVED. IT IS RECOMMENDED THAT A TEMPORARY BRACING SOLUTION BE

IMPLEMENTED IN THE IMMEDIATE FUTURE, WHILE DECISION-MAKING AND CONSTRUCTION DRAWINGS ARE BEING PREPARED FOR ONE OF THE TWO OPTIONS.

In each case, it will also be necessary to integrate the various masonry components. These pieces were conceived and built separately, and they have long performed separately, sometimes at cross purposes. Finally, it will be important, in each case, to exclude water from the structure. Water threatens the integrity of the masonry foundations and their bearings, while hastening the deterioration of most other building elements, including miraculous wooden survivors from the first-period building.

OPTION 1 - RESTORATION TO 1728

In terms of workmanship, the dwelling's 1893 fabric is unremarkable. Moreover, this later work complicates any effort to preserve the fragile remains of an utterly unique structure that preceded it. The 1893 walls have doubled the weight bearing on the lower, earlier walls. Neither these, nor the soil below, were intended to carry such loads. Most concerning, continued structural movement is slowly pulling the building apart.

We have seen that the original dwelling was a frame structure, built c. 1725. Around 1730, this frame was wrapped in brick, leaving the front and rear wall plates, the upper floor joists, and the roof frame of the original house intact. The result was a fashionable new house, reflecting the latest trends in domestic design.

Of the original framing members, only the wall plates and a single tie beam at the west gable survived the 1893 expansion. Nonetheless, the original framed house is still knowable. Mortises in the bottoms of the remaining wall plates locate the original doors and windows of the longitudinal walls, while mortar oozing from the 1730 masonry gables created casts of now-vanished framing and mortises.

The original frame is believed to have been an earth-fast, post-in-the-ground structure, based in part on the open mortise for a lap-joined up-brace on the rear wall of the main range.[4] No standing example of such a house from this early period survives in Virginia; only archaeological examples remain. As a result, our knowledge of such dwellings, once a prevalent form of construction in this region, is merely diagrammatic. That makes this house very precious. Whatever the significance of the 1893 house, we argue that the survival of the 1730 brick house, with the ghost of its frame predecessor, is paramount.

FOR THESE REASONS, OUR RECOMMENDATION IS THAT THE MATTHEW JONES HOUSE BE RESTORED, EXTERNALLY, TO THE APPEARANCE IT PRESENTED BETWEEN OVER THE 175-YEAR PERIOD BETWEEN C. 1730 AND 1892.

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[4] *A Historic Preservation Plan…*, p. 52.

RESTORATION - A SCOPE OF WORK

RE-ESTABLISHING THE LATERAL SUPPORT PREVIOUSLY AFFORDED BY THE UPSTAIRS FLOOR FRAMING IS A PRIORITY. BY ITS NATURE, RESTORATION WOULD REQUIRE RE-INSTATEMENT OF THE FIRST-PERIOD JOISTS FOR THE UPPER FLOOR. SEE ROOF FRAME AND JOISTS, below.

PREPARATIONS

- Erect a shelter over building.
- Shore masonry the walls and chimney.
- Remove existing foundation drainage system.
- Complete archaeology in the undisturbed zone around the brick foundations.

DEMOLITION

- Remove the existing roof and the 1893 masonry walls of upper story.
- Remove the wooden interior partitions of the upper story.
- (Preserve the front tower and its roof, the first-period brick gables and the chimneys).
- Underpin NW corner of building with a new concrete footing.

FOUNDATION DRAINAGE

- Trench and waterproof the foundation.
- Install new foundation drainage system and backfill.

MASONRY

- Re-integrate walls & chimneys w/ stainless steel ties; inject w/ grout where needed.
- Close up or adjust incorrect masonry openings--windows and doorways.
- Restore all fireplaces.
- Restore exterior masonry and secure against water/air infiltration.

ROOF FRAME AND JOISTS

PROVIDE NEW ROOF FRAME AND NEW JOISTS FOR THE SECOND FLOOR

RE-ESTABLISH LATERAL SUPPORT PREVIOUSLY AFFORDED BY 2ND-FLOOR JOISTS

- Modern elements (stainless steel tie rods, etc.) to securely brace exterior walls (assuring that the use of dissimilar materials is carefully accounted for in the design).
- Install flooring of reclaimed southern yellow pine, both stories.
- Flooring to be laid tight and double-nailed to enhance diaphragm action of system.
- Floor joists of upper rooms to remain exposed.
- White oak
- Worked green

- Hewn and pit-sawn
- Front cornice derived from ghost on side of tower
- Undersides of upper floor boards to be hand-planed.

Re-frame the shed roof, allowing early wall plate to remain

- White oak rafters
- Worked green
- Hewn and pit-sawn
- No soffit at rear cornice

Lath and shingle the new roof (main slopes and shed) with cypress or cedar.

Reframe period II (c. 1730) partitions.

- White oak
- Worked green
- Hewn and pit-sawn
- Between Hall and Chamber
- Between Chamber and Back [Shed] Room
- New, exposed collars of white oak to carry second-floor ceiling.
- Lower edges of collars molded
- Riven white oak flooring, feathered edges, drawn smooth, eased/lapped ends

DOORS & WINDOWS

Restore Exterior Doorways

- New exterior doors and frames, according to period designs.
- Reproduction hardware for doors, according to period designs.
- New cellar entry frame and doors, according to period designs.
- Reproduction hardware, according to period designs.

Restore Windows

- New window frames and sashes, according to period designs and archaeology (lead, glass, etc.).

ACCESS

Rebuild front and side steps.

- Subject to previous archaeological findings.
- Period-appropriate design and materials.

Rebuild cellar steps and enclosure.

- Subject to previous archaeological findings.
- Period-appropriate design and materials.

CELLAR

- Remove parging at NW corner of cellar.
- Remove concrete floor slab in cellar.
- Install new cellar floor of dry-laid brick pavers on rock-dust.
- Provide gravel margin around cellar paving

SYSTEMS

Provide HVAC system to control humidity.

Provide basic utilities.

SITE

Site work, visitor infrastructure.

OPTION 2 — REPAIR

A SCOPE OF WORK

The later history of the Matthew Jones House and its connection to contemporary events in the region are arguably as important and valid as for any earlier time. The colonial and antebellum periods have long held our imaginations in thrall, but the decades around 1900 were also part of history's continuum. They explain the times that came before and helped direct those which followed. It is difficult to say then, that the 1893 upper floor and interior finishes of the Matthew Jones House had less to do with producing our present circumstances than the building's earliest elements.

We have prioritized our recommendations for the repair of this extant building, though, of course, certain items of the proposed work, though less important in themselves, are listed early in the process for reasons of rational sequencing.

PRIORITY ONE

CLEARING

Remove all items from house

- Provide for off-site storage of all materials from the house.
- Vacuum all spaces.
- Clear and cover all HVAC floor registers.
- Protect extant wooden floors and trim.

STRUCTURAL MOVEMENT

SHORE BUILDING

- West gable-end wall.
- North wall from corner to rear shed.

RE-ESTABLISH LATERAL SUPPORT AFFORDED BY 2ND FLOOR JOISTS

- New floor joists over Room 102 to match 1893 work, but with some modern elements (stainless steel tie rods, etc.) to securely brace exterior walls (assuring that the use of dissimilar materials is carefully accounted for in the design).

- As part of second floor re-framing, provide load transfer mechanism to minimize “hinge” effect between original brick walls and upper floor brick walls which has occurred owing to removal of deterioration of original wall plates.
- Further bracing (assume galvanized steel angles) of the gable end wall between ceiling and attic framing may also be required due to significant deformations which were observed from the exterior but not accessible from the attic.
- Floor boards to be reclaimed, long-leaf southern yellow pine.
- Lay flooring tight and double-nail to enhance diaphragm effect.

ROOF FRAME REPAIRS

- Repair attic joist-to-rafter connection to form proper roof truss behavior (thrust resistance at eaves) over added second floor.
- Provide modern elements (stainless steel tie rods, etc.) to securely brace exterior walls.
- Repair rafter-to-top plate connection at top of shed roof.

FOUNDATION REPAIRS

- Remove existing foundation drain system.
- Perform archaeology around the building perimeter.
- Underpin NW corner, between west chimney and rear shed.

MASONRY REAPAIRS

- Insert stainless steel ties across cracks in west gable-end wall.
- Inject west wall with grout where necessary.
- Repoint chimney and west gable-end wall inside and out.
- Provide low-profile, vented copper cap on both of the main chimney stacks.

INTERIOR CHIMNEY REPAIRS

- Rebuild first-floor firebox of west chimney using 19th-century firebricks.
- Remove stove thimble from of west chimney and repair breast.

MOISTURE

Remove and replace existing foundation drainage system.

- After archaeology is complete, remove existing system.
- Trench foundation.
- Seal penetration for water line in cellar.
- Waterproof exterior of basement foundation.
- Provide new French drain.
- Connect new system to clear outlet.

Remove all damaged or moldy drywall.

Repair ruptured plumbing that serves second-floor bath.

Replace extant roof covering.

- Remove existing shingles and felt underlayment.
- Repair/replace deck as necessary with 1" reclaimed southern yellow pine.
- Replace associated trim in accoya, detailed to properly protect framing.
- Provide copper drip edges at rakes and eaves.
- Renew flashings in lead, secured with lead wool.
- Repair masonry as required.
- Provide copper gutters and rain leaders to get water away from building.
- Lay shingles on cedar breather & VaproShield underlayment.
- Shingles to display a 6" exposure.
- Secure shingles with stainless steel, ring-shank nails.
- Shingles to be Alaskan yellow cedar, square butts.
- Shingles to be 3" to 5" wide x 18" long x 5/8" to 3/4" thick at the butt.

Replace doors, frame and steps at cellar entry

- All material to be accoya.
- Exterior design to be appropriate for 1893.

HVAC

- Re-habilitate system to ventilate building and control moisture.
- Reposition supply registers above FFL.
- Reconnect system.

PRIORITY TWO

STRUCTURAL MOVEMENT

- Remove thimbles in chimney breasts and rebuild masonry.

MOISTURE

Repair exterior door sill – front tower.

- Plug holes of front face.
- Fill with flexible epoxy.

Replace exterior door sill — SE corner

- New sill to be reclaimed, long-leaf SYP, treated.

Remove parging in NW corner of basement.

Saw-cut and remove margin of basement floor slab, replacing with gravel.

FINISHES

Remove all late plaster.

Re-install drywall ceilings previously damaged or removed.

- Provide insulation.
- Co-ordinate fire detection systems and lighting in west room.

ACCESS

Rebuild front steps in present configuration.

- Perform archaeology below doorway.
- Provide concrete footing.
- Clean and re-use original bricks.
- Use natural cement mortar.

Provide new steps for east doorway.

- Perform archaeology in front of doorway.
- Provide concrete footing.
- Rebuild steps in brick.

FIELD OBSERVATIONS

ROOF:

Flashings:

- The roof appears to be flashed with lead-coated copper where it abuts the rear wall of the second floor.

Covering:

- The entire roof is covered with square-butt shingles of white oak. These date from the 1993 repairs. They were laid on roofer's felt applied over the present deck, having a very short exposure.

- Nail evidence indicates that the earlier exposure was actually 6".

- The roof has failed in several locations where moisture stains and mold are visible on the drywall ceiling of the second floor.

- On the exterior, many of the shingles have curled, opening the substrate to the elements, and the entire roof bears a heavy covering of lichens, moss and other growth. These tend to retain moisture, accelerating deterioration of the shingles.

Underlayment:

- Felt was laid at eaves, at raking edges, and also in valleys to secure the roof's most vulnerable boundaries. This underlayment remains visible from the interior of the building where the underside of the framing remains partly exposed. Owing to its limited use and dark color, this material is not obtrusive.

Deck:

- The deck is composed irregularly spaced "skip" sheathing, made of inch-thick SYP, of varied width and spacing. All of this material is circular-sawn, dating from the 1893 raising of the roof—or later. In recent times, a few pieces have been replaced around the truncated stack of the chimney serving the rear shed.

Framing

- Slender pine rafters date from the raising of this roof to create a house of two full stories. This framing remains in generally in good condition, though some deterioration has occurred where the 1993 shingles have failed.

Recommendations:

- We concur with the previous recommendation that that present roof covering should be replaced with wooden shingles.

- Among the species suggested by the NCPTT, we believe that Alaskan Yellow Cedar is preferable, according to data published by the U. S. Forest Products Lab. Stain gray with TWP, “Total Wood Preservative,” to simulate weathered color.

- Certain areas of the deck are fractured and would be further broken up by another application of wooden shingles.

- Because these areas are not extensive, we recommend that the affected sheathing be replaced in kind, using inch-thick, circular-sawn southern yellow pine.

- Deteriorated framing should be “sistered” using like material, using wire nails.

STRUCTURE:

Northwest Corner

- The present gables each represent two periods of construction. The lower portion of each is laid in English bond w/ glazed headers ascending the rakes. These lower gables belong to the brick exterior that wrapped an earlier house frame c.1730. The mortar “snots” oozing out of the interior faces of this masonry captured the form of and location of lap mortises for the early framing that preceded the masonry. For that reason, the interior, gable-end exposures are among the most important features of the house, modest as they may appear.

- The upper portion of each gable, laid in 1:7 bond, dates from 1893 when the roof and gables were raised to create a full-height second story.

- DURING THE 1990S INTERVENTION, MOREOVER, THE FLOOR JOISTS OF THE NORTHWEST SECOND-FLOOR ROOM WERE REMOVED, ELIMINATING THE RESTRAINTS THEY HAD PREVIOUSLY PROVIDED FOR THE C.1730 NORTH WALL AND FOR THE 1893 WALL SUPERIMPOSED ON IT. THE HORIZONTAL PLANE WHERE THE UPPER WALL BEARS IS CLEARLY A HINGE POINT, AND SO DEMANDS RESTRAINT.

- Prior to 1993, a diagonal crack had appeared on the west gable descending from the cornice end board at the building’s NW corner. This was pointed up in 1993, but subsequent movement has reopened it approximately 3/16” to ¼”. Moreover, a 1/8” space has opened up between the 1990s pointing and the adjacent, second-floor window frame of the north wall. The *vertical* displacement at all of these cracks is minimal, suggesting an outward rotation of the corner, with the center of rotation at the bottom of the wall.

- Early photos of the building, together with measurements inside the two adjacent walls indicate that the corner had dropped about 2” prior to the addition of the upper story, and that cracks initially confined to the lower, earlier wall eventually telegraphed into the added 1893 masonry above.

West Gable

- In 2015, the NCPTT called attention to the restraint imposed by the mass of the west chimney, suggesting that the crack was a consequence of thermal movement, or that removal of the joists had released all restraint on the front wall, allowing to move away from the gable.

- As an explanation for the rupture, thermal movement seems less likely than the fact that the brick gable has bellied inward, shortening it, and thus pulling it way from the front corner.

Recommendations

- Refer to Option 1/Option 2 recommendation sections.

MOISTURE AND DRAINAGE:

General:

- The house shows clear signs of ongoing moisture issues since 1993, and they persist today.

The Roof

- The roof currently has no gutters or rain leaders. As a result, it routinely discharges water in a concentrated zone around the perimeter of the foundation.

- The ceilings of the upper story display discoloration from water and mold growth, both attributable to roof leaks directly above.

- Upstairs and downstairs, finished floors display dampness and discoloration from the leaking roof.

The Foundation Drain

- In other cases, mortar in the perimeter walls has deteriorated alarmingly, leaving deep piles of sand aggregate on the basement floor below.

- The scope and intensity of this condition suggest a failure of the foundation drainage system introduced in 1993 to remove excess water from around the foundation. As a result, the walls suffer from the continuing effects of rising damp.

- The NCPTT report noted that filter cloth on the drain had separated from the foundation in a particular location.

- A more likely explanation of the difficulties described above, also noted by NPCTT, is that the sewage treatment plant to which the drain originally connected, has since been removed. Now, it is unclear where the water in the drain now goes, or whether this conduit even has an outlet.

- The NCPTT's 1993 treatment report mentions that footings were inserted below the interior face of the perimeter foundation walls, and also below the interior foundation between the two basement rooms. This suggests that the water moistening the walls is coming from the exterior of the house.

The Cellar

- The floor framing of the lower rooms was replaced entirely in 1993. However, incipient fungal growth and rusting nails are evident on certain of the new members. Both are clear signs of persistent moisture, surely from the roof leaks mentioned earlier.

- The cellar walls reflect a continuing presence of excess moisture. In most cases the moisture content measured 40 WME, 19 being the upper end of the acceptable range.

- Below the east chimney, the moisture readings attained 90 WME, at (or exceeding) the upper limit of the WME scale. The hearth directly above this section of wall has been saturated by water coming down the chimney.

- In this dampest location, the wall had been repointed at some early date with a hard mortar. Much of the softer material behind this has leached out, leaving voids behind a brittle skim riddled with holes.

- The present cellar entry incorporates a sloping pair of doors, intended to shed water while providing external access to the basement. However, the doors are a make-do installation, involving two pieces of T-111 plywood. These are badly deteriorated, with some sections completely gone. Tarps and sandbags are the current fix, but the protection these afford is not complete. Consequently, the wooden steps immediately below these doors are deteriorating, and the floor below is quite wet.

- Below these steps is a floor drain to receive condensate from the adjacent air handler. It may have been blocked by debris when the cellar was recently filled with standing water. This occurred when plumbing on the second floor failed, inundating the cellar with a foot of standing water.

- Near the SE corner of the cellar, water is slowly entering between a PVC wall sleeve and the water supply line.

Recommendations

- Install half-round copper gutters and round copper rain leaders to collect water from the roof and move and it well away from the building.

- Remove and replace the present foundation drain, installing it in conjunction with an impervious membrane against the brick foundation walls of the excavated spaces.

- Establish a permanent outlet for the new foundation drainage system.

- Care must be taken not to undermine shallow foundations where the cellar and the unexcavated crawl spaces adjoin.

- Rebuild wooden components of the cellar entry, according to proper details from the 19th century, the finished product to be a plausible design for 1893.

- Provide ventilated caps on the chimneys to prevent water from entering the flues.

- Replace with the concrete floor slab of the cellar with dry-laid brick on rock dust, or saw-cut and remove the outer margin of the slab, replacing it with crushed stone, so that the foundations can breathe.

- Open the condensate drain in the cellar floor.

- Seal off the slow infiltration of water around the wall sleeve of where the water supply enters the cellar.

HVAC

Equipment:

- HVAC equipment appears to have been inactive for an extended period.

- Electrical service to the compressors appears to have been disabled.

- Cover of junction box for west compressor is missing.

Distribution:

- Floor-level slot diffusers filled with debris.

- Ducts dirty, and possibly inoculated with mold.

Recommendations

- Re-habilitate HVAC system to control extremes of moisture and humidity.

- Vacuum ducts to remove mold.

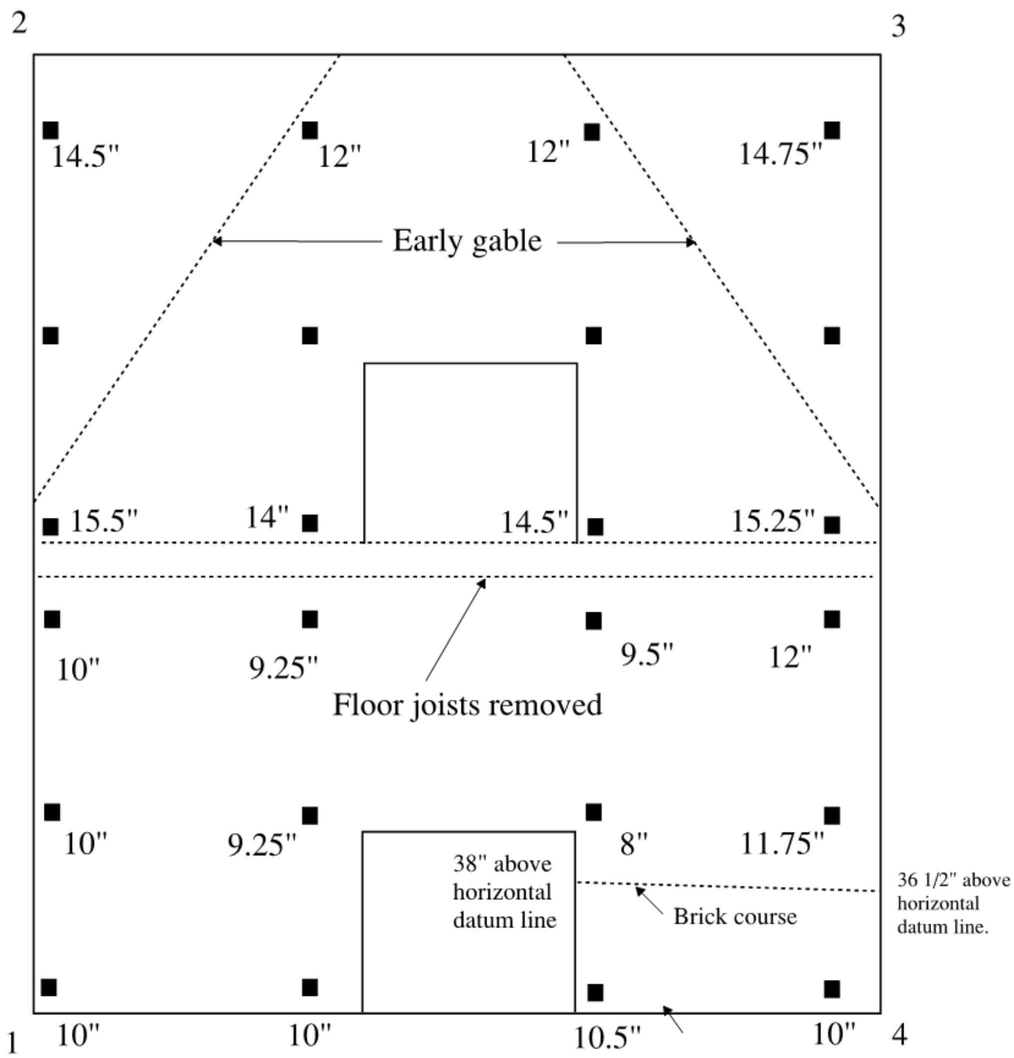
- Raise supply registers above FFL.

- Provide new thermostat to be located in west, ground-floor room.

- Re-insulate ducts.

Deformations - West, Gable-End Wall

(No Scale)

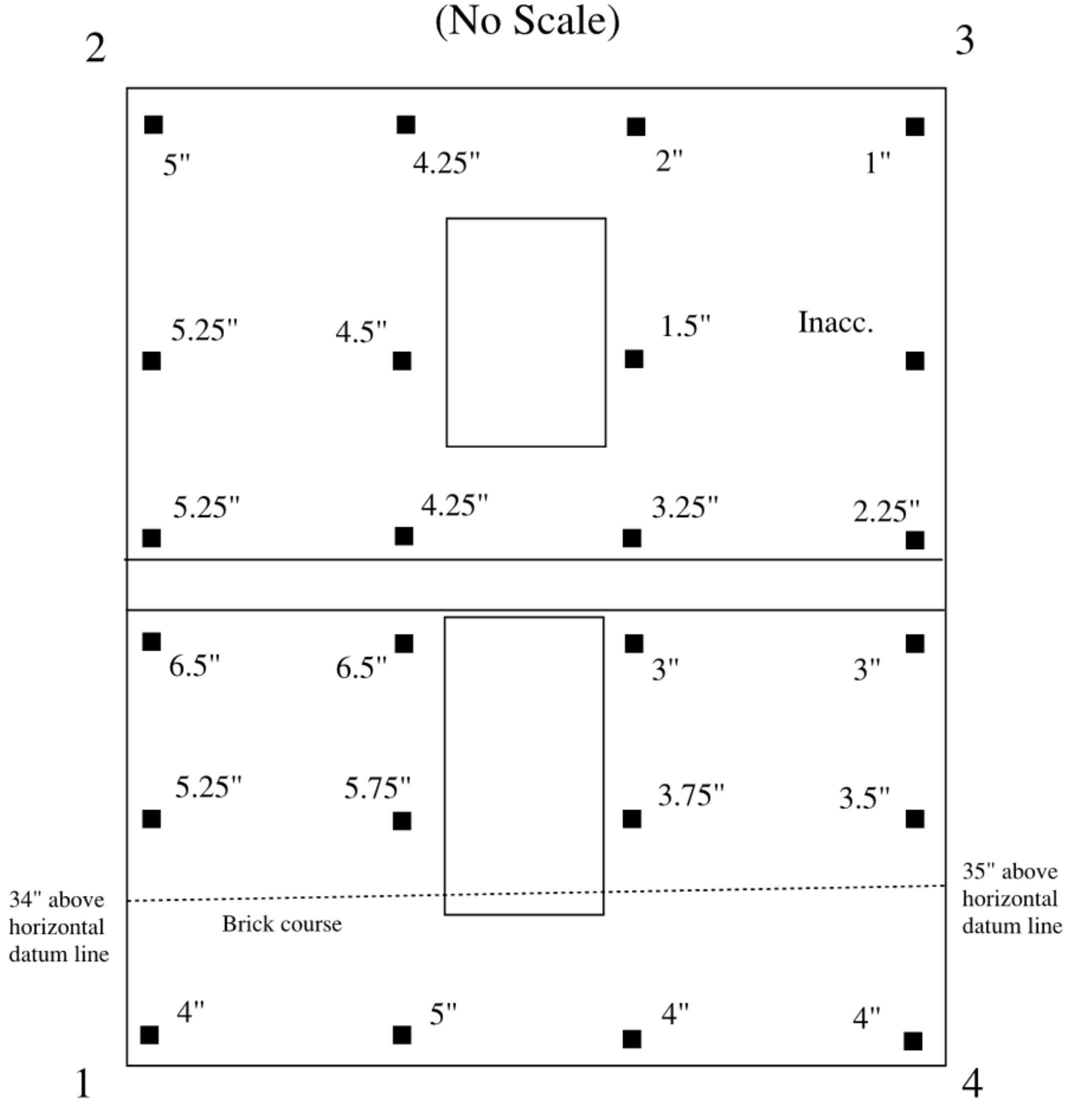


Vertical datum plane 1 2 3 4 at west, gable-end wall.

Datum plane stands 10" from the wall at points 1 and 4.

All dimensions measured from wall out to datum plane.

Deformation of Rear Wall
Between NW Corner and Rear Shed
(No Scale)



Vertical datum plane 1 2 3 4 at rear wall
Plane stands 4" from rear wall at 1 and 4
All dimensions measure from wall out to datum plane

ESTIMATED COSTS – OPTION 1

Matthew Jones House	Harrison Road	Newport News	VA 23604
Option 1 Restoration Project Phases	**Amount**		
Masonry Restoration			
Removals	$ 10,000.00		
Archaeology	$ 145,964.00		
Shoring	$ 4,235.00		
Site work	$ 30,000.00		
Underpinning	$ 50,000.00		
Scaffolding	$ 42,897.00		
Drainage	$ 25,000.00		
Masonry (Disassembly of 2d floor	$ 300,000.00		
Stitching, Repointing).			
Removal of basement concrete floor	$ 5,000.00		
New brick floor on stone dust	$ 50,000.00		
Total	**$ 653,096.00**		
Carpentry Restoration			
Removals (Roof and 1893 addition)	$ 9,000.00		
New Roof Framing	$ 98,175.00		
New Deck and Shakes	$ 225,369.00		
New Front Cornice	$ 9,125.00		
New Second Floor Joists	$ 13,860.00		
New Second Floor Flooring	$ 18,480.00		
New Cockloft	$ 7,508.00		
New Door sill and Threshold	$ 8,432.00		
New Period Doors and Frmaes	$ 15,593.00		
New Period Windows	$ 35,508.00		
New Interior Partitions in Correct Loci	$ 14,322.00		
New Int. Doors and Frames	$ 10,626.00		
New Wood Cellar Entry	$ 4,851.00		
Total	**$ 470,849.00**		
MEP's			
HVAC	$ 65,000.00		
Plumbing	$ 5,000.00		
Electric	$ 5,000.00		
Total	**$ 75,000.00**		

General Conditions			
Temp Toilet	$ 124.00	Per Month	
36' Lull	$ 3,215.00	Per Month	
20 yd Dumpster	$ 500.00	Per Month	
Contingency	$ 118,389.00		
MCWB A&E Fees	$ 177,583.00		
Storage Container	**By Owner**		
Mobile Office	**By Owner**		
Temp Building	**By Owner**		
Project Total	**$ 1,504,861.00**		

ESTIMATED COSTS – OPTION 2

Matthew Jones House	Harrison Road	Newport News	VA 23604
Option 2 Stabilization Project Phases	**Amount**		
Masonry Restoration			
Removals	$ 10,000.00		
Archaeology	$ 145,964.00		
Shoring	$ 4,235.00		
Site work	$ 30,000.00		
Underpinning	$ 50,000.00		
Scaffolding	$ 42,897.00		
Drainage	$ 25,000.00		
Struct. Steel	$ 90,000.00		
Masonry (Stabilization of chimneys,	$ 200,000.00		
Stitching, Repointing).			
Cutting of verge around basement floor	$ 15,000.00		
(Replace with crushed stone)			
Total	**$ 603,096.00**		
Carpentry Stabilization			
Removals	$ 6,000.00		
Second Floor Joists	$ 13,200.00		
Second Floor Flooring	$ 18,480.00		
Roof Deck and Shingle	$ 70,235.00		
Doors and Windows	$ 17,820.00		
New Wood Cellar Entry	$ 4,620.00		
Total	**$ 124,355.00**		
MEP's			
HVAC	$ 75,000.00		
Plumbing	$ 10,000.00		
Electric	$ 9,000.00		
Total	**$ 94,000.00**		
General Conditions			
Temp Toilet	$ 124.00	Per Month	
36' Lull	$ 3,215.00	Per Month	
20 yd Dumpster	$ 500.00	Per Month	
Contingency	$ 79,302.00		

MCWB A&E Fees	$ 118,953.00		
Storage Container	**By Owner**		
Mobile Office	**By Owner**		
Temp Building	**By Owner**		
Project Total	**$ 1,016,272.00**		

RECOMMENDATIONS – PREVIOUS AND CURRENT COMPARED

BIBLIOGRAPHY

1940

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1993

Williamsport Preservation Training Center, National Park Service, U.S. Department of the Interior. *Record of Treatment: Matthew Jones House Preservation/Rehabilitation Project*, by Chris Robinson, (Williamsport, MD: U.S. Department of the Interior, 1993).

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2015

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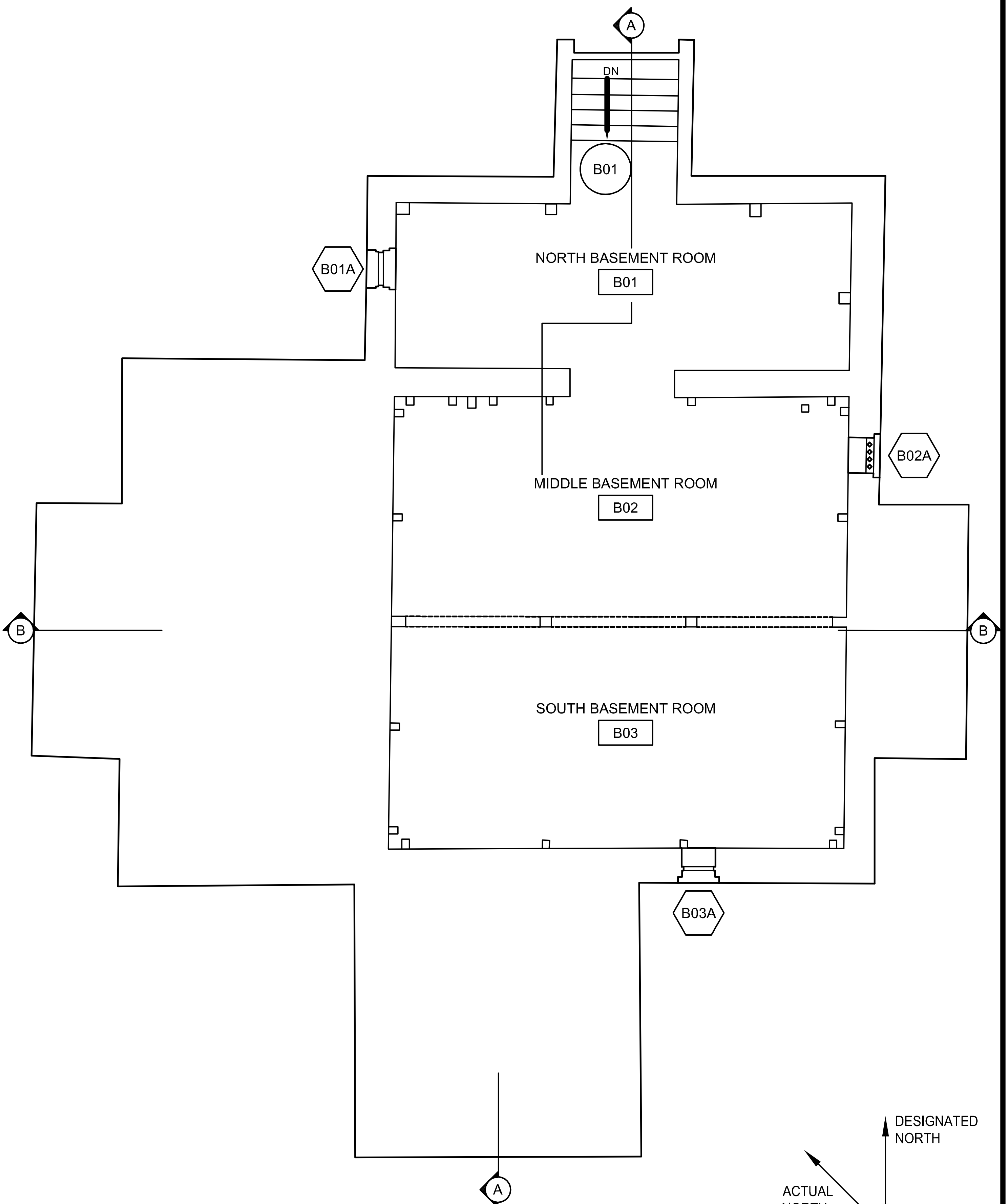
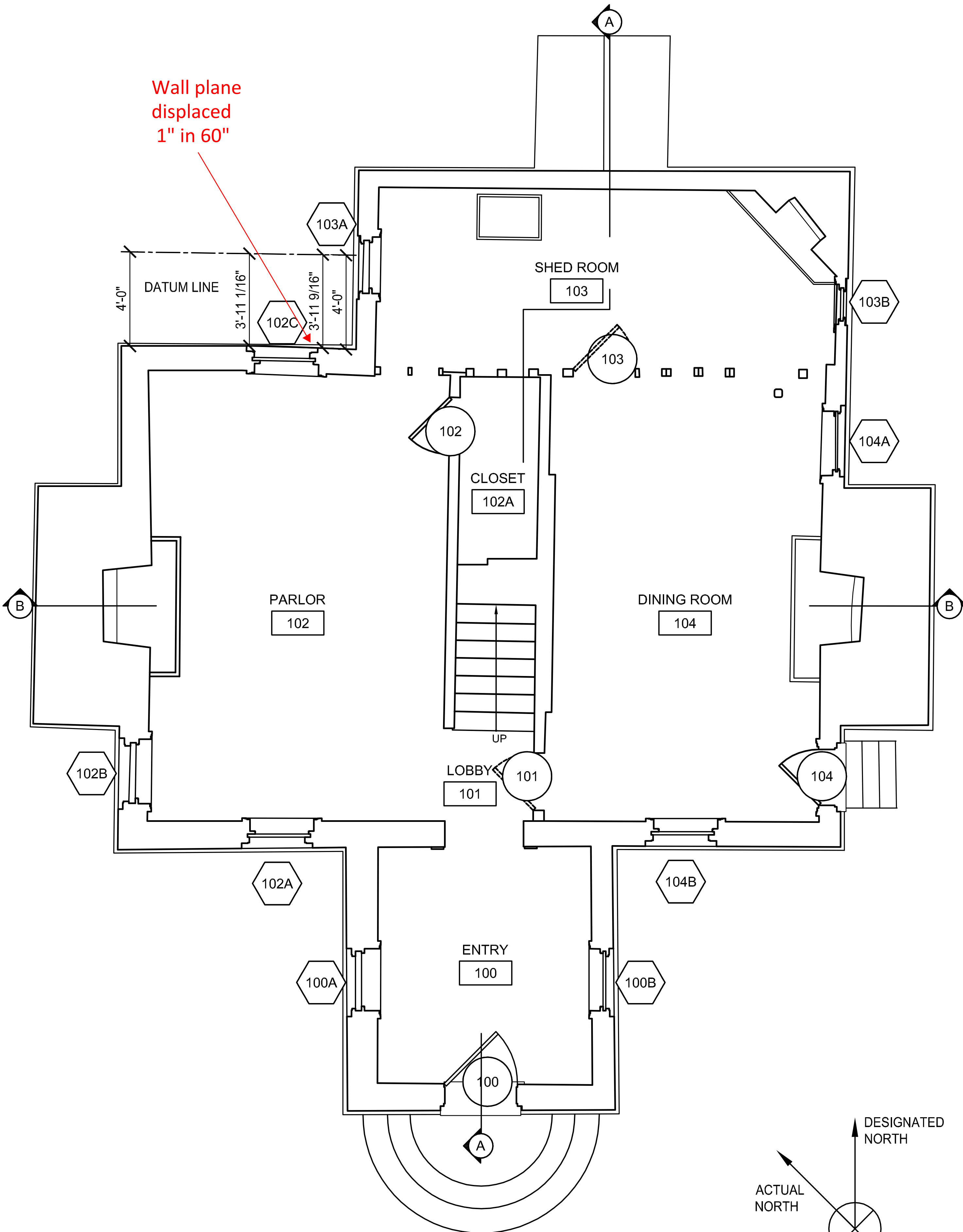
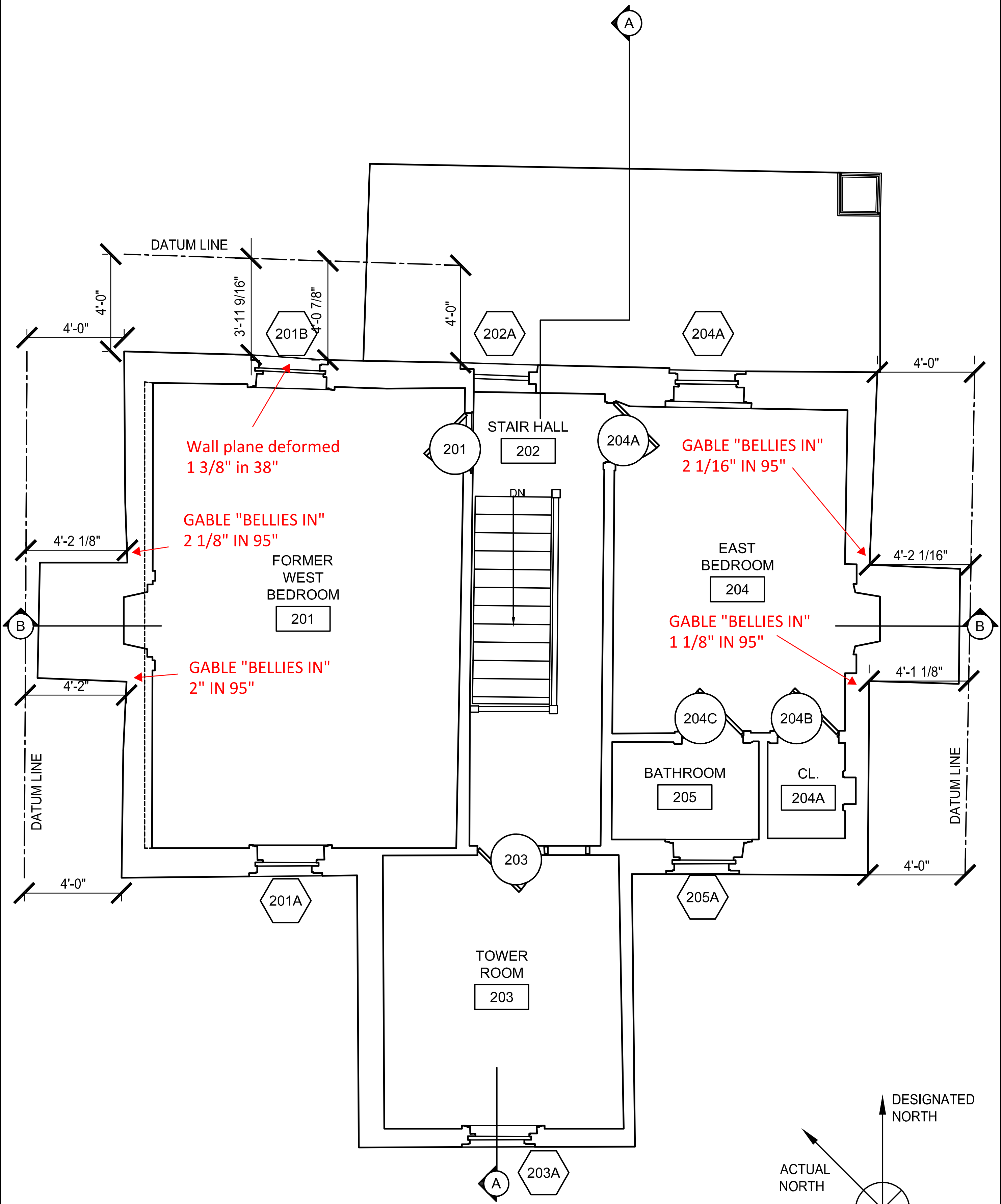


BASEMENT FLOOR PLAN



FIRST FLOOR PLAN

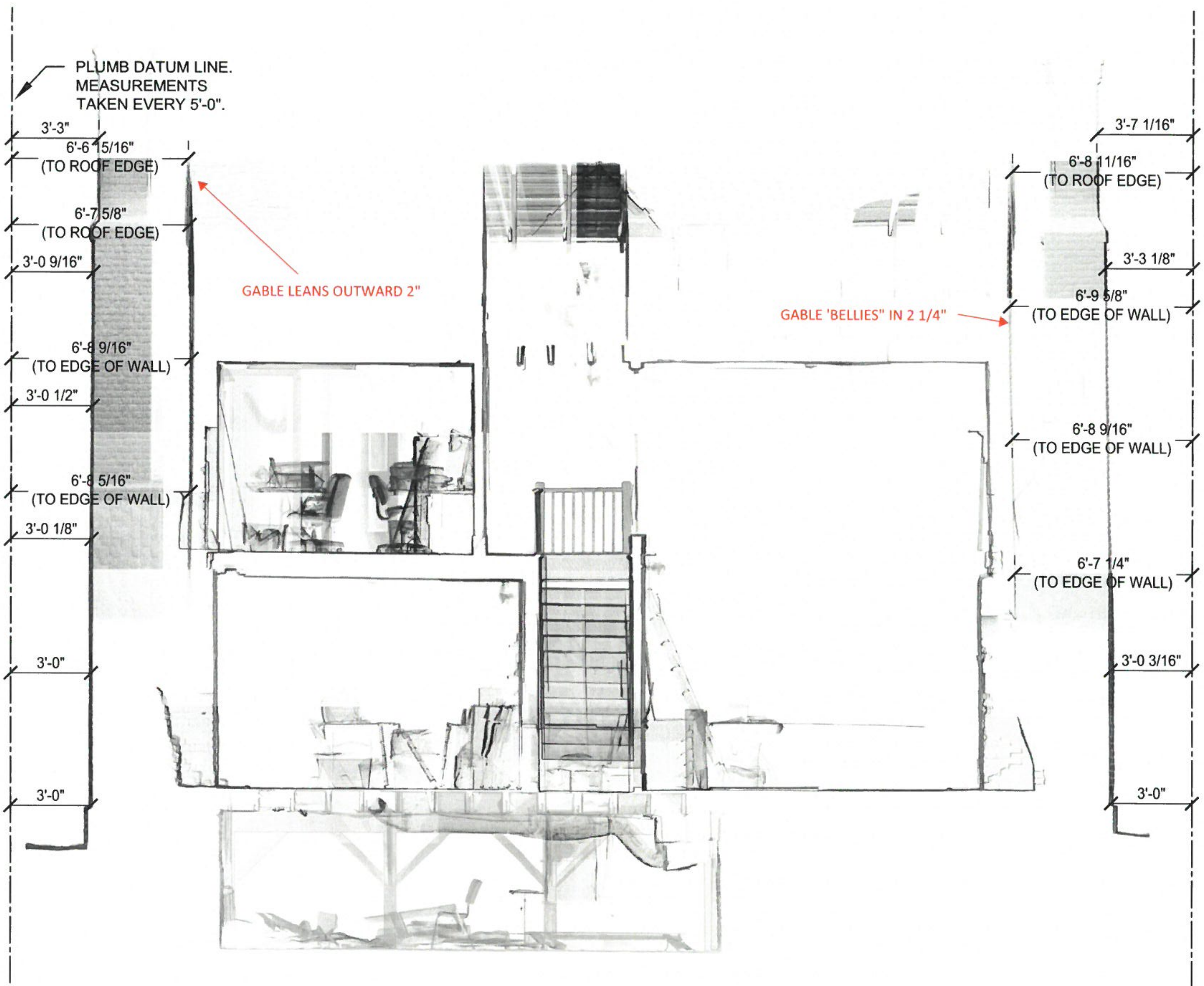


SECOND FLOOR PLAN

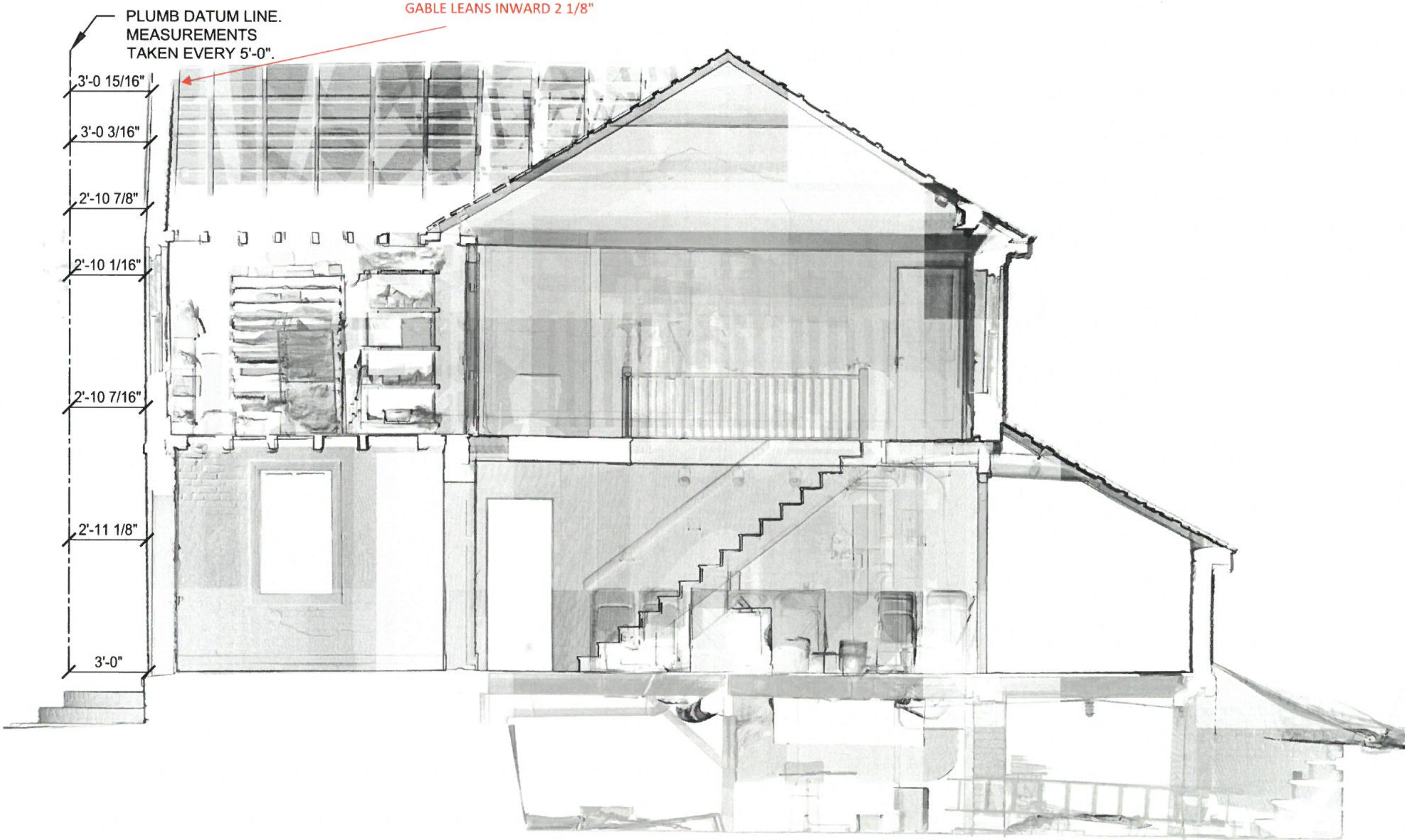
SOUTH ELEVATION

EAST ELEVATION

WEST ELEVATION



LONGITUDINAL SECTION - B



TRANSVERSE SECTION - A

Newport News: 757.873.8858
Williamsburg: 757.564.4434
Richmond: 804.684.5850

www.tamconsultants.com
www.terracon.com

October 22, 2024

Michael Creasy, AIA
GuernseyTingle
4350 New Town Avenue
Williamsburg, VA 23188
Phone: 757-220-0220
Email: mcreasy@guernseytingle.com

Subject: JBLE Matthew Jones House Abbreviated Investigation
Harrison Road, Fort Eustis, Newport News, VA 23604
TAM Project No. MB246004

Dear Michael Creasy:

In accordance with your request and the project requirements, TAM Consultants has completed an abbreviated investigation at the Matthew Jones House located on the grounds of Fort Eustis in Newport News, Virginia.

The purpose of this investigation is to determine whether conditions at the house have changed since it was reviewed in 2018 and to review the options for repairs in the project scope of work.

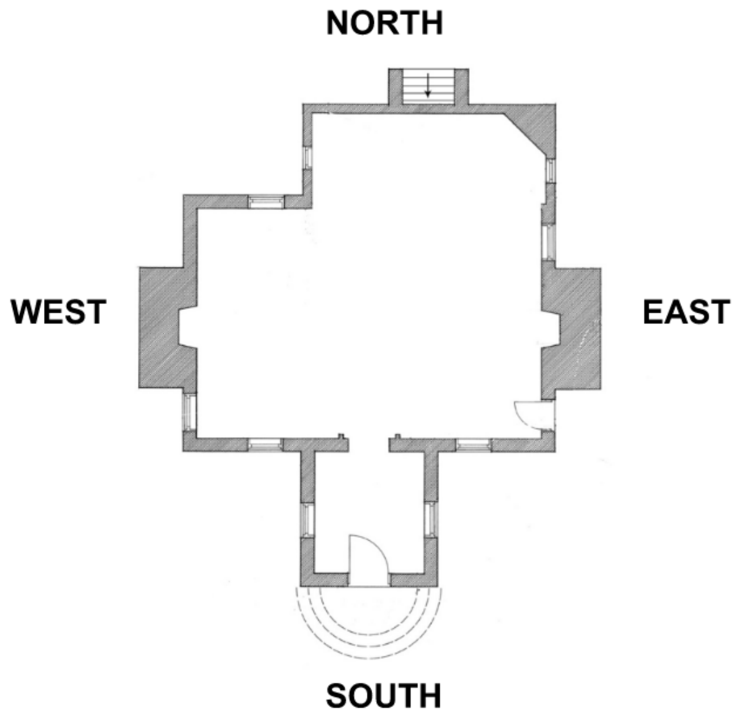
The scope of work for this project is based on Option 2 from the 2018 report titled "An Update on Its Condition, with Recommendations" prepared by Mesick Cohen Wilson Baker Architects. The 2018 report was an update of an earlier report prepared in 2015 by the National Center for Preservation Technology and Training and titled "Matthew Jones House Recommendations for Treatment". The findings of both reports were compared with the current existing conditions of the building at the time of our investigation. Note that this report is limited to the structural aspects described in Option 2 of the 2018 report.

Our site work was performed on October 4 & 11, 2024 and included a visual observation of the exterior and interior masonry walls, the exposed interior portions of the structural roof elements, and visible structural members or elements inside the house. Dimensions and measurements were recorded on provided plans and extensive photographic recording was completed to document the existing conditions of the structure.

Background:

The Matthew Jones House was built in 1725 as a one-and-a-half story timber framed structure with brick masonry chimneys on both side elevations. In 1730, the structure was improved with the addition of brick masonry walls, a two-story tower at its front elevation and a one-story shed structure along its rear elevation. In 1893, the roof of the house was raised to provide a full-height second floor and the chimneys were extended. In 1992, efforts were made to stabilize the house to allow for occupancy and for the home to be used as an architectural study museum.

For convention in this report, the following plan will be used to reference each elevation and to provide general orientation when describing our observations.



The structural components of the home consist of load-bearing brick masonry exterior walls that support timber roof rafters, attic floor joists and second floor joists. The second floor structural system has been partially removed at the west side of the structure. Partial timber end joists from the 1725 construction are embedded into the mid height of the masonry walls. A timber beam spans across the opening between the rear shed addition and the main house that provides support for the second floor structural system and the north brick masonry wall above.

Timber stud walls at the first and second floor separate the interior spaces. The east side and center hall area of the home sit above a vaulted cellar. The first floor structural system inside the cellar was rebuilt during the 1992 stabilization campaign. The 1992 reconstruction included the installation of independent lumber supports for the first floor system. The remaining portion of the first floor system at the west side of the home has no accessibility and its condition is concealed and unknown.

OBSERVATIONS

EXTERIOR WALL

- The brick masonry units original to the 1730 construction are laid in a Flemish bond with glazed header bricks. This original brick is distinguishable and creates a perimeter at the original gable end wall outlining the original masonry work prior to the addition of the second floor. Above the perimeter of the original brick work is brick masonry laid in common bond that encloses the second floor addition. Masonry trim work constructed of light red brick is present on the building in the form of window jambs, window headers, wall corners, and a small two course masonry band at the second floor level of the main entrance tower.
- The bricks in the exterior walls generally appear to be in serviceable condition although hairline cracking and minor surface degradation was observed in individual bricks throughout the building elevations.
- At the west elevation, we noted a significant step crack in brick masonry joints running from the northwest corner of the house and extending diagonally across the wall north of the chimney. This step crack transitions into a vertical crack where it crosses to the original brickwork and compromises the brick masonry units in that area. This crack was documented in the 2015 report and was noted to have been previously repaired but has since recurred.
- We noted that the upper portion of the west gable end wall is bowing or deflecting toward the interior of the house. This condition is visually apparent when viewing along the face of the wall parallel to the west elevation. This condition was previously noted in the 2015 and 2018 report. Displacements along this wall were documented as part of the 2018 report. Measurements in the field do not indicate significant changes in these displacements.
- At the west end of the north wall, we noted an area where the brick wall bows outward. In general, the condition is apparent from the exterior at the first floor level. A 6' level was used to measure the difference in vertical planes of the wall. The bowed portion of the wall was measured to be at least 2" out of plumb with the rest of the wall. Displacements along this wall were documented in the 2018 report. Measurements in the field do not indicate significant changes in these displacements.
- At the north wall above the lower shed roof, the second floor brick wall was found to be deflected downward. The deflection is visually apparent when viewing along the face of the wall parallel to the north elevation. Step cracking in brick units and mortar joints at the upper corners of the north elevation appear to be caused by this brick masonry deflection. These cracks have been repaired and do not appear to have recurred in the time since the repair.
- The mortar joints throughout the exterior elevations have a grapevine finish and vary in color and composition from one masonry construction period to the next. Brick masonry mortar joints at many of the building's exterior wall surfaces were observed to be in serviceable condition. It is apparent that repointing on the building has been completed in the past. Several areas of brick masonry mortar joints were observed to be severely

- Efflorescence and organic matter on the brick masonry were observed at scattered locations throughout the building's exterior elevations. These conditions were most prominent at the horizontal or sloped surfaces of the chimneys and adjacent to the chimney attached to the shed at the rear of the building.

INTERIOR WALLS

- The center hall walls and second floor structural system at the east end of the home have been finished with drywall that conceal their interior framing members.

- The brick masonry wall surfaces on the interior of the building's west and east end are partially concealed by existing plaster. The plaster is in a degraded state and crumbles to the touch. Much of the plaster has either been removed or has fallen from the wall exposing areas of the interior brick masonry wall surface.

- The exposed interior brick and mortar joints throughout the house were found to be in poor condition, most notably at the west gable end wall.

    - The west wall exhibits step cracking and vertical cracking at several locations. Step crack locations observed on the interior surface of the west gable end wall are consistent with the step cracks observed at the wall's exterior surface.

    - Brick masonry units and mortar joints were found to be in a severe state of deterioration where pieces of brick and mortar were seen to be loose and, in some cases, missing.

- Significant vertical cracking was observed at the interior surfaces of the west and east walls of the second floor room in the main entry tower. These cracks are located where the south exterior walls intersect the main entry tower walls.

- We observed vertical joints between sections of the masonry built at varying times over the life of the structure. In particular, we noted large vertical joints where the entry tower and chimney walls meet the main house walls.

TIMBER FRAMING

- The main roof of the house is gabled and constructed with timber roof rafters with a span of approximately 13'-9".

    - The rafters were measured to be approximately 1.75"x3.5" and are spaced at a max of 26" on center.
    - The rafters span from the brick masonry walls to the roof ridge and are aligned with the rafters on the other side of the roof. The rafters are supported at the eave by a wood false plate that sits on top of the ceiling joists.
    - Timber decking runs perpendicular on top of the rafters and are spaced out to provide a base for the wood roof shingles.
    - A large portion of the building's main roof is concealed by a drywall finish and could not be visually observed.

- The shed roof of the rear addition on the north side of the building consists of timber rafters with a span of approximately 8’-8”.
    - These rafters were measured to be 1.75”x3.5” and are spaced at a max of 26” on center.
    - The bottom end of the rafters are notched to seat on the bearing timber plate. They are also attached to wood brackets that sit on the timber plate. The plate was observed to have significant wood rot and insect damage over most of its length.
    - The upper ends of the rafters frame up to the face of the brick masonry wall along the north elevation. Embedded metal brackets/pins extend out from the mortar joints and fasten the rafters to the brick masonry. The bracket/pins were observed to be corroded and were missing at every other rafter.

- The majority of the second floor structural system is concealed and only a small portion of the system can be viewed from inside the shed at the rear of the building. The second-floor structural system at the west end of the building was removed at some point in the past. Open masonry joist pockets are present along the north and south elevation that show the locations where the floor framing members used to be supported.

- The timber beam supporting the second floor and north brick wall located above the opening between the rear shed and the main house was observed to be severally cracked at its midspan. The beam also appears to have evidence of advanced wood rot.

- Supplemental steel framing was observed at the west end of the home that provides support for the brick masonry at the second floor. The supplemental framing includes steel angles that are embedded in the masonry walls .

- A supplemental steel framing system has also been installed to provide new support for the wall, floor, and roof above the opening between the rear shed and the main house. This framing includes steel angles attached to the existing second floor structural members, a steel beam in the attic spanning along the north end of the main roof, tube columns that run from the cellar to the steel attic beam, and steel rods that hang the steel angles from the attic beam.

CONCLUSIONS & RECOMMENDATIONS

EXTERIOR WALLS

1. In our opinion, cracked and deteriorated individual bricks on the building’s exterior elevations are most likely caused by general weathering and aging. Cracking and degradation in individual bricks can be addressed by either removing and replacing the affected bricks or by carefully routing the crack and filling the void with a repair mortar that matches the brick’s original color. It is our opinion that routing, preparing and repairing the brick is the best option as it will better preserve the historic aesthetic of the façade.

2. In our opinion, step cracking observed at the exterior elevations of the building is most likely caused by either building settlement or displacement of the masonry walls.

   a. Settlement can be caused by several possible factors including initial settlement upon loading, soil compaction over time or uneven sloped soils.
   b. Displacements due to building movements can be the indirect result of settlement, the direct result of failures of the structure that provide support, or the result of expansion and/or shrinkage due to thermal differences.

   Prior to repairing instances of step cracking, the cause or causes must be determined and investigated. Once the cause has been addressed, step cracking should be addressed by the installation of crack stitching. Crack stitching involves the removal of the existing mortar bed joints on both sides of the crack and installing helical stainless-steel bars in the masonry bed joints. The open mortar joints are then repointed with a mortar that matches the original. Cracked bricks are routed and patched with a repair mortar.

3. In our opinion, the bowed west gable end wall is most likely caused by a combination of factors. In general, the cracking patterns noted both inside and outside are consistent with cracks that would result from inward movement at the top of the chimney and the second-floor fire box. The wall deflections noted in the 2018 report are similarly consistent with this movement. Contributing factors causing this movement are as follows.

   a) Settlement of the northwest corner of the building and the settlement of the interior side of the chimney has led to an inward lean of the overall chimney.
   b) The fact that the chimney is not continuous with or tied into the adjacent main walls of the building that would potentially provide additional stability.
   c) The removal of the second floor at the west end of the building has taken away the lateral support that would have been previously provided. The fact that the condition has worsened since the 1993 restoration work is most likely attributed to this removal.
   d) The removal of the floor system has resulted in walls that are twice as tall as they originally were with a weak area that effectively amounts to a hinge at the wall's mid height (due to the embedded wood end joists and plates).

   In order to address the inward movement and to effectively eliminate the issues associated with the weak point at the mid height, we recommend the installation of a bracing system that will serve to stabilize the overall wall. Note that we do not believe that repairs can be completed to restore the plumbness of the wall due to the severity of the conditions. This recommendation is generally consistent with the Option 2 scope of work from the 2018 report and would include the following tasks:

   a) Provide underpinning of the west gable end wall to prevent further settlement and/or rotation of the chimney and wall.
   b) Install a steel bracing system that will support the wall at mid height and at the attic floor level to prevent rotation and provide lateral support. Design of the system will determine whether the system needs to be tied into the rest of the second floor or if it can be installed independent of itself.
   c) An alternative to installing a steel frame would be to restore the second floor system. The acceptability of this would need to be determined by the owner.

4. In our opinion, the outward bow of the north wall (at the northwest corner of the building) is most likely caused by settlement that predates the addition of the second floor. This conclusion is based on the observation that the added second floor walls are essentially plumb while the first floor brick coursing visibly slopes downward at the corner. The inward bow of the west gable end wall may also be a contributing factor causing this condition. It is our opinion that this wall area cannot be made plumb without complete rebuilding of the masonry. We recommend that the wall be stabilized by underpinning and supporting it with the west end bracing system noted above.
5. The significant downward deflection of the second floor brick wall along the north elevation appears to have been caused by the failure of the timber beam that previously supported the wall. We did not observe signs of significant brick cracking in this wall that would indicate further movement or displacement of this section of wall. At the time of the investigation, the supplemental steel framing that was installed in 1993 appeared to be supporting the load of the wall, floor and roof.
6. We noted areas around the building exterior where brick mortar joints require repointing to maintain watertightness. Repointing work on the building will need to be performed in a manner that is sensitive to the historic nature of the original masonry. Mortar removal is recommended to be completed by hand and the use of grinders or mechanical equipment will most likely be prohibited. A mortar mix will be developed to match the original mortar color and hardness based on a petrographic examination of an original mortar sample. The mortar will be applied at test locations to allow for approval by the architect or owner prior to widespread use. The mortar is to be hand packed in lifts and struck with a grapevine finish to match the neighboring joints.
7. Efflorescence and the buildup of organic matter at the exterior brick masonry surfaces is caused by natural elements and the presence/buildup of excessive moisture. Design of repairs regarding adequate waterproofing and rainfall navigation should be completed and installed to create watertight conditions and to direct excess water away from the building. Waterproofing and building enclosure design is not included in our scope of work and is to be completed by the architect. The removal of efflorescence and organic matter on the brick masonry surfaces will be addressed by direction in our contract drawings with a specified formula safe to use on historic masonry.

INTERIOR WALLS

1. The existing plaster on the interior wall surfaces of the building has become deteriorated and is beyond its service life. We are unsure what the project intentions are for the plaster and whether the remaining plaster is to be left in place for historical purposes.
2. A portion of the interior surfaces of the masonry walls are currently concealed by drywall finishes. These concealed conditions are primarily located on the second floor at the east end of the home. The brick masonry behind the drywall could not be assessed at the time of the investigation and may contain conditions that are of concern to the structure of the building. Masonry repairs to these areas will not be included in the project scope of work unless the drywall is removed for further evaluation.

3. Many similar brick masonry conditions observed at the exterior of building were also observed at the interior walls. Similar repairs to these conditions are to be completed including step/vertical crack stitching, brick masonry unit repairs and masonry mortar joint re-pointing.

4. We noted that the gable end wall construction consists of brick masonry installed at different periods over the life of the building and that there are joints between these parts of the wall, most significantly between the chimneys and the original brick walls. We recommend that these wall sections be tied together with helical anchors and that the joints be solidly grouted to allow the wall segments to function together as a unit and to help resist future movement of the wall.

TIMBER FRAMING

1. In our opinion, the exposed rafters and ceiling joists that make up the main roof of the home appear to be in serviceable condition. We propose to evaluate the existing connection at the base of the rafters and design a reinforced connection to provide adequate resistance to lateral thrust. The revised connections will utilize fasteners, clips, and/or straps as required to provide additional capacity to that of the original construction. Note that during our investigation, much of the roof structure was concealed by a drywall ceiling and could not be directly observed or assessed. If structural deterioration or other structural concerns are revealed upon removal of the ceiling finish additional repairs may be required.

2. The shed roof structure at the rear of the building was fully visible at the time of the visit. Multiple structural concerns were observed at the shed roof that require attention. The upper connection of the roof rafters at the north masonry wall is to be redesigned to establish an adequate means of attachment. The rafter's lower attachment at the shed knee wall and timber top plate is to be either reinforced or replaced to provide additional capacity. A portion of a rafter member was found to have severe wood rot and/or insect damage and is to be replaced or supplemented to restore its structural integrity. The timber plate on top of the masonry knee wall was found to be severally deteriorated with insect damage and is to be either replaced or supplemented.

3. Most of the second floor structural system is currently concealed by drywall or the floor decking over the east and center portions of the house and could therefore not be assessed at the time of the investigation. If evaluation of these members is required drywall removal will be needed to allow the second floor framing members and their connections to be visually assessed.

4. The cracked timber beam that spans along the rear shed opening appears to have originally supported the second floor structural members, the shed roof rafter members, and the brick masonry wall above. It appears that throughout the building's history the loads on the beam became excessive and caused the beam to fail. The installation of steel members at the second floor framing system have created a condition in which much of the load has been removed from the affected timber beam. At the time of the visit, it appeared that the installed steel framing was sufficiently supplementing the loads that the beam originally supported.

In general, TAM Consultants agrees with the “Option 2” repair work provided in the 2018 report prepared by Mesick Cohen Wilson Baker Architects. Our scope of work for this project is limited to the design of structural repairs within “Option 2” and are listed below:

- Provide temporary shoring plan for the west gable end wall and the portion of the north wall west of the shed.
- Re-establish the lateral support afforded by the second floor system that had been removed. Options are to be provided to the owner including the installation of a new floor system or steel braced frame.
- Supplement or redesign the attic joist-to-rafter connection to resist lateral thrust.
- Supplement or redesign both rafter connections at the rear shed roof.
- Provide design for the foundation underpinning requirements of the project.
- Develop a masonry repair plan showing required work tasks on building elevations.
- If required, design a concrete footing for the rebuilding of the front entry steps.
- Design new stairs for the steps at the east doorway.

<u>DISCLAIMERS:</u>

Please note that our review was limited to the portions of the building or structure discussed in this report and may not include other detrimental conditions that may exist. Our observations and comments are limited to the conditions noted and those that were readily visible at the time of our visit. We make no claim either stated or implied that all conditions were observed, or that a detailed analysis of the building or structure was performed. Our opinions do not represent engineering design, as we have not calculated loads or validated adequacy of any of the structural members.

Conclusions drawn in this report are based on visual observations and on information available, known, and declared in our report on the date of our site visit and/or the time of preparation of this report. Should additional information be uncovered or made available, we retain the right to revise or supplement our report accordingly.

This report does not provide any warranty or guarantee for any portion of the property. Noted conditions may change. If observed conditions indicate that other distress to the building or structure may have occurred, we should be contacted so the condition can be evaluated.

This report is furnished as privileged and confidential to the addressee. Release to any other company, concern, or individual is solely the responsibility of the addressee.

We appreciate the opportunity to provide professional services to you. If you have any questions or need additional information or investigation into this matter, please call us at (757) 564-4434.

Sincerely,

COMMONWEALTH OF VIRGINIA
MICHAEL J. LUCAS
Lic. No. 0402065019
10/22/2024
PROFESSIONAL ENGINEER

Michael J. Lucas, PE
Project Engineer

Timothy P. Jester, PE
Senior Structural Engineer


Photo 1: General view of the south (front) elevation.


Photo 2: General view of the west (left) elevation.

Photo 3: General view of the north (rear) elevation.

Photo 4: General view of the east (right) elevation.

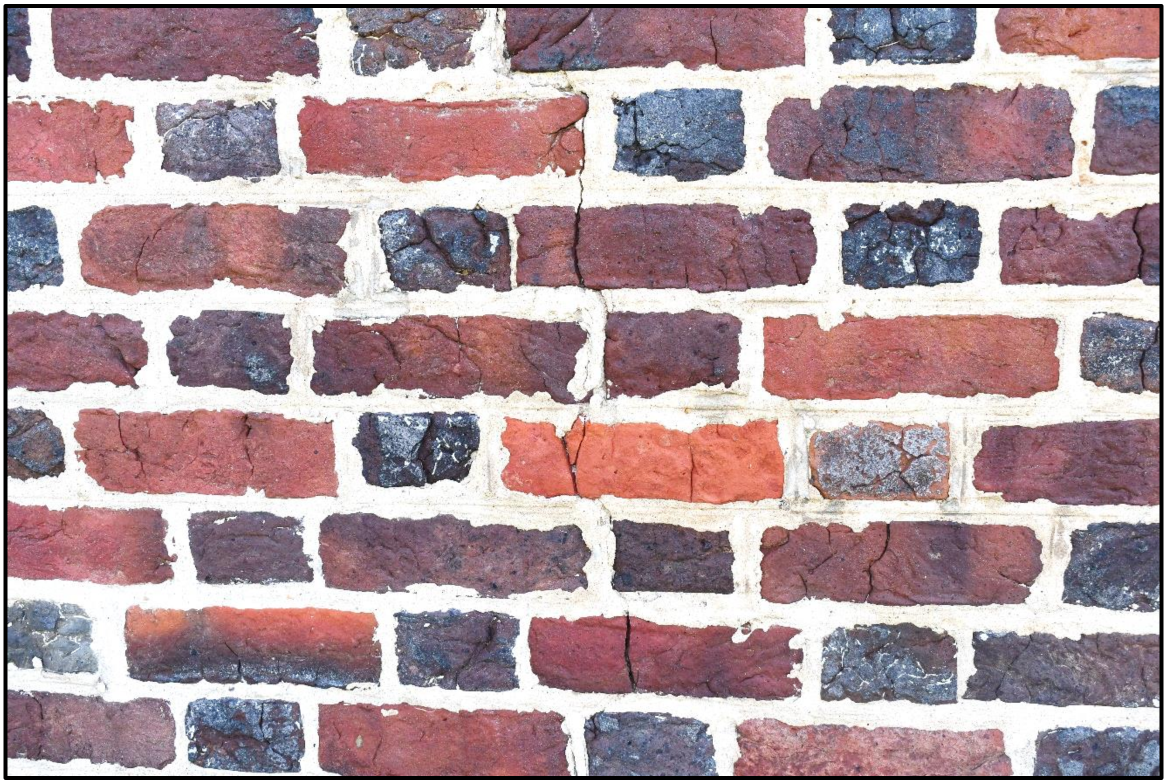


Photo 5: Representative view of cracking in brick masonry units observed at scattered locations throughout the building's façade.

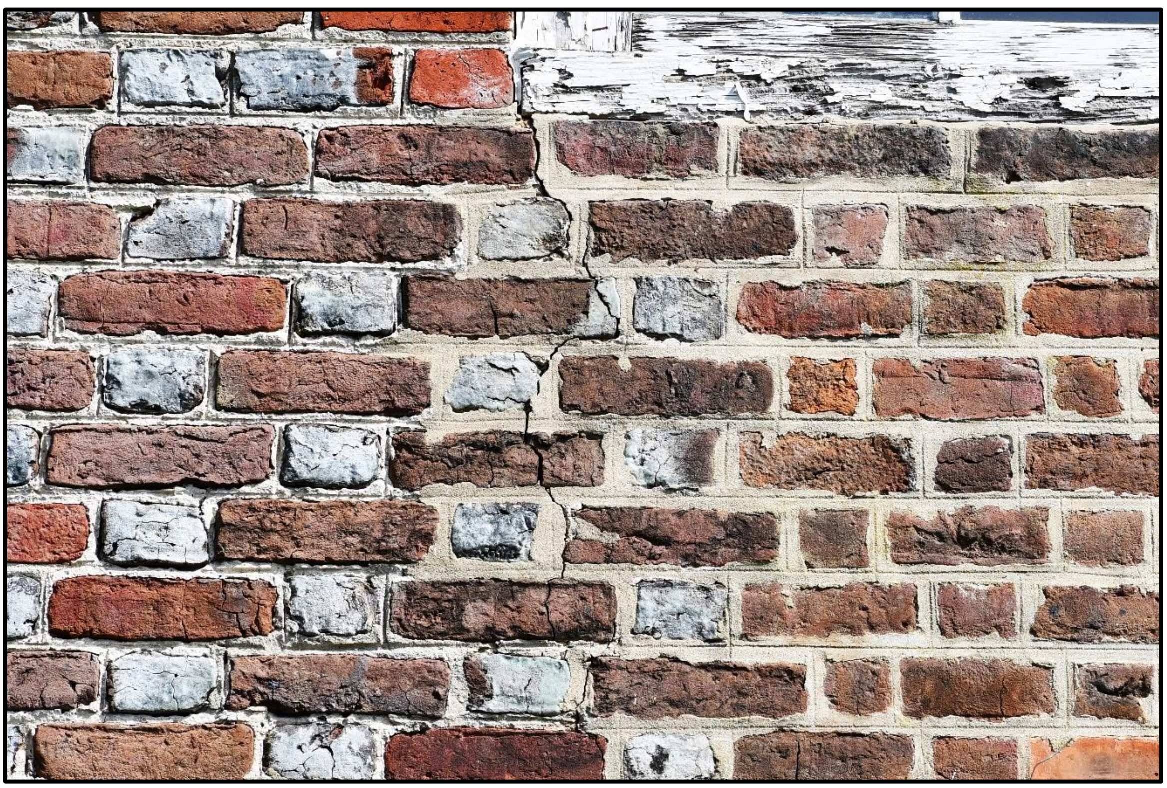


Photo 6: Vertical cracking in masonry observed emanating from the second floor window located above the main entrance along the south elevation.

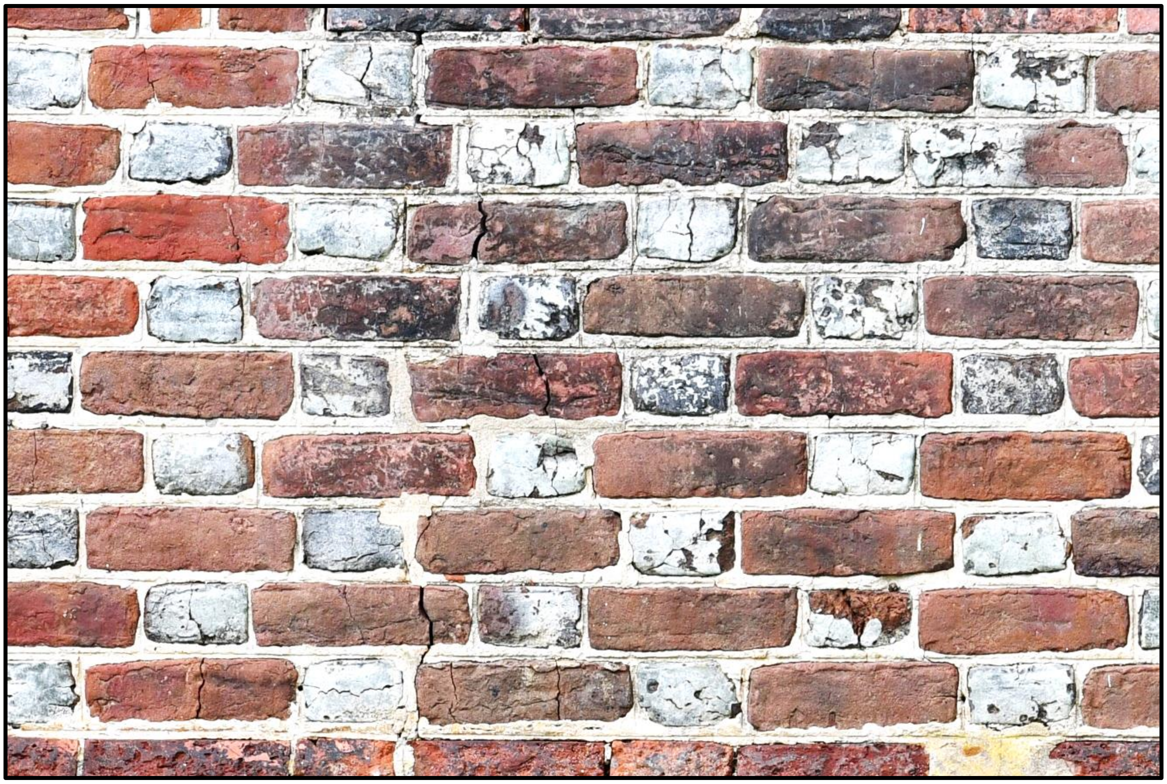


Photo 7: Vertical cracking in brick masonry units at a wall area located on the east elevation south of the chimney.

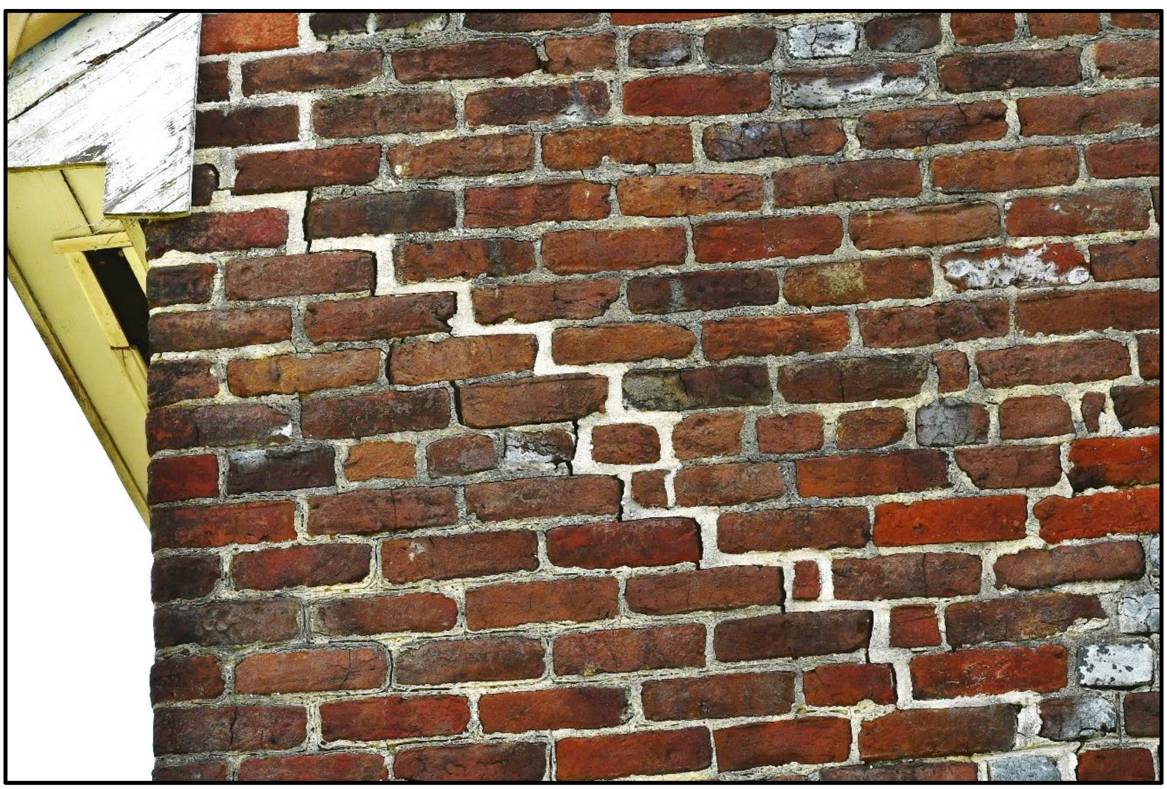


Photo 8: Step crack emanating from the upper northwest corner of the building on the west elevation.

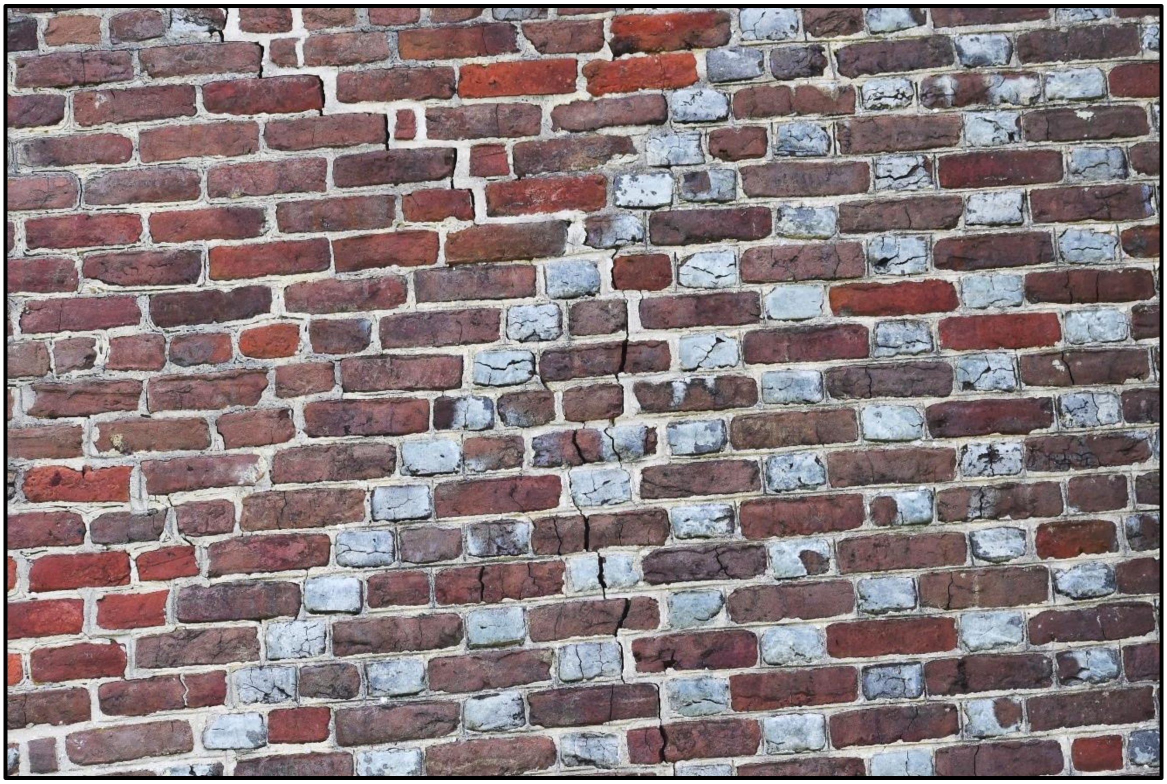


Photo 9: Step crack at the northwest corner of the building transitioning into a vertical crack through the homes original brick masonry.


Photo 10: Photograph taking at a view parallel to the west elevation that exhibits the bow in the west elevation gable wall.

Photo 11: Out of plumb measurement taken at the mid height of the bowed wall area on the north elevation west of the rear shed.

Photo 12: Deflected brick masonry above the shed structure viewed along the north elevation.


Photo 13: Open brick masonry mortar joints located at the southeast corner of the building.


Photo 14: Deteriorated and weathered brick masonry mortar joints located above the cellar door on the north elevation.

Photo 15: Deteriorated/open brick masonry mortar joints and signficant presence of organic growth located at the northeast corner of the building.

Photo 16: Significant presence of organic growth located at the sloped surface of the east elevation chimney.

Photo 17: General condition of the existing plaster on the interior walls located at the southwest corner of the home.

Photo 18: Step crack at the interior surface of the west gable end wall consistent with the location of the exterior crack exhibited in photograph #8 & #9.

Photo 19: Typical condition of the masonry located on the interior surface of the west gable end wall. Note the joints between the varying wall segments.

Photo 20: Vertical crack observed in the interior west wall located on the second floor of the main entry tower.

Photo 21: Vertical crack observed in the interior east wall located on the second floor of the entry tower. Note the daylight seen through the crack.

Photo 22: View of the west side of the home showing where the 2nd floor system had been removed.

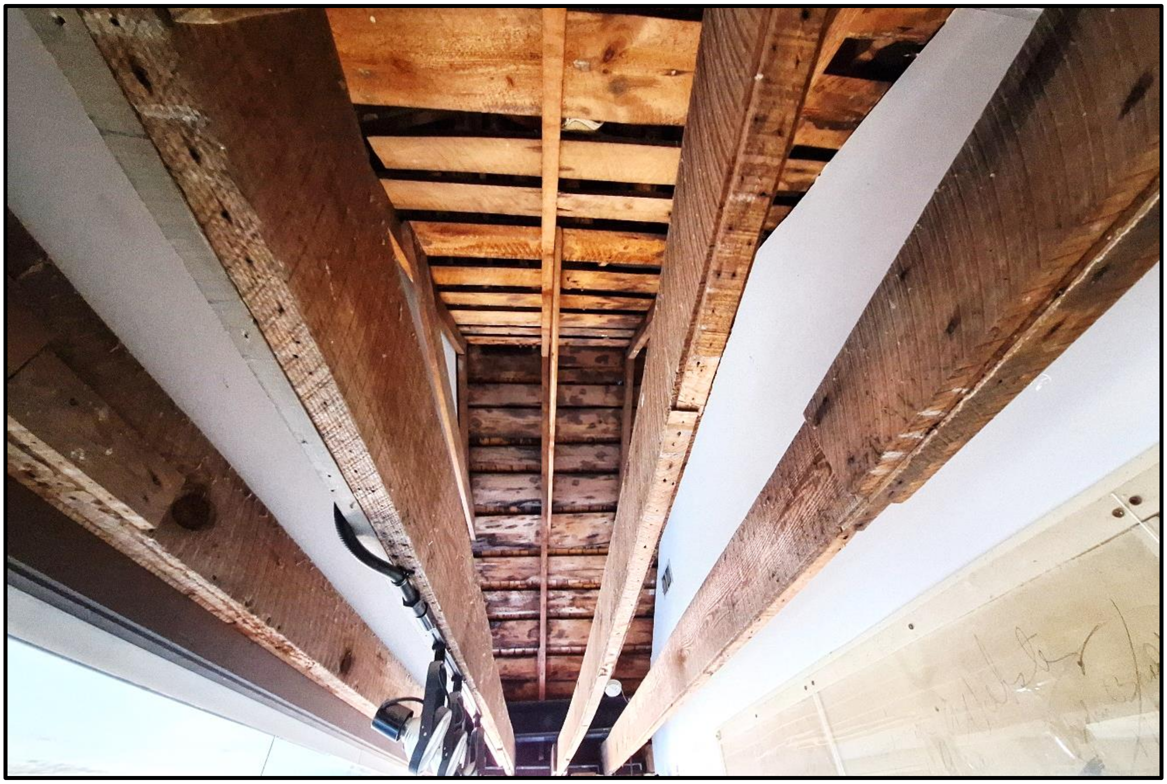


Photo 23: General view of the main roof and ceiling structure located in the hallway of the second floor.

Photo 24: Main roof rafter and attic floor joist bearing condition located at the north wall of the home.

Photo 25: Upper connection of the rear shed roof rafter. Note the corrosion on the original framing hardware.

Photo 26: Lower connection of the rear shed roof rafter. Note the deteriorated condition of the timber plate.


Photo 27: Wood rot or insect damage on a rafter located at the rear shed roof.

Photo 28: Portion of the second floor system visible from inside the rear shed. Note the supplemental steel angles adjacent to the floor joists.

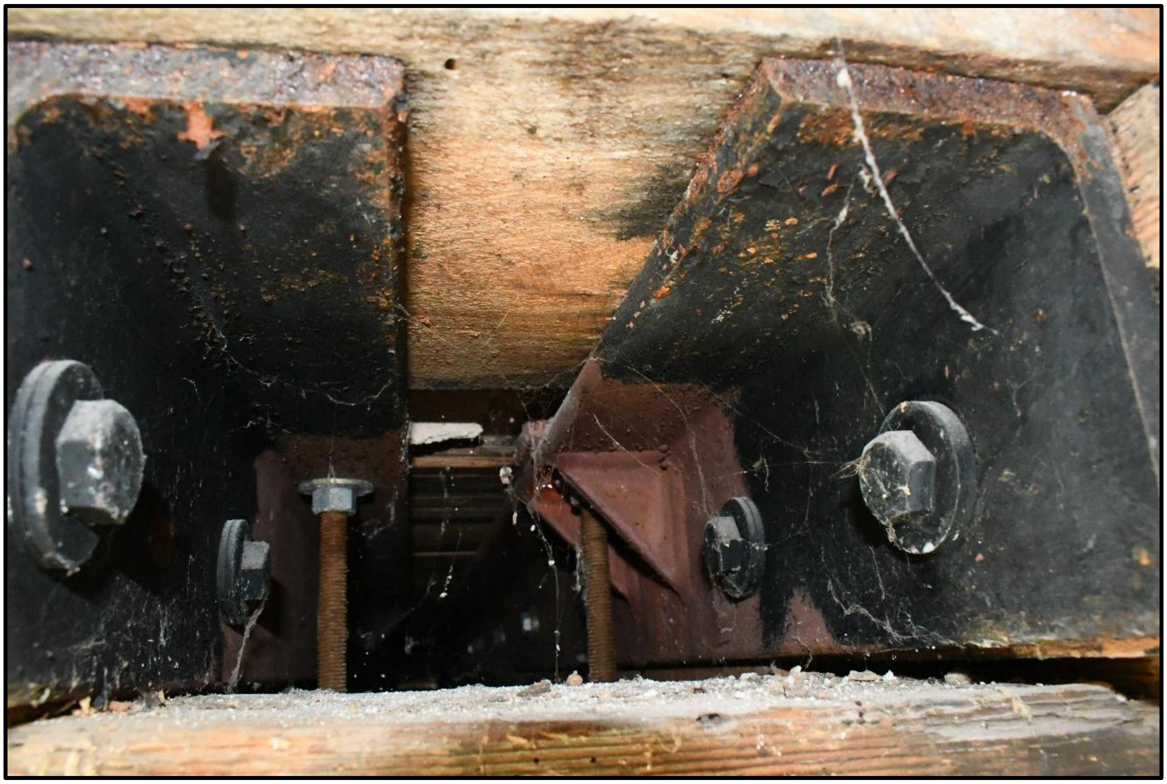


Photo 29: Steel angles installed adjacent to the second floor joists and hung from the steel beam located above the main roof attic joists.

Photo 30: Steel beam located above the main roof attic floor joist located along the north wall of the home.

D. Abbreviated Investigation