OCMULGEE ASSOCIATES, INC.
Consulting Structural Engineering
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February 16, 2001

Lynne Spencer, AIA
Claude Emanuel Menders, Architects
59 Commercial Wharf
Boston, Massachusetts 02110

Reference: Stonehurst, Paine Estate, Waltham, Massachusetts
OA File 20016.1

Dear Lynne:

I visited Stonehurst yesterday to meet Patrick Guthrie and you from CEM and Ann Clifford of Stonehurst in order to observe certain areas of distress noted in your conservation assessment report of May, 2000.

1. During our original inspection on January 27, 2000 we noted that the pony truss in the attic was built in a manner that resulted in the diagonal members intersecting the horizontal tie beam several feet away from the vertical supports. An engineering analysis indicated that the horizontal tie beam would be seriously overstressed at the intersection points. Therefore, we requested that a closet wall be opened so that one of the intersection points could be revealed and inspected for any damage or any reinforcement that might be present. This opening was made and yesterday we observed that a sloping column had been introduced by the builders between the second floor and attic, running from the main column up to the intersection point on the pony truss.

   Based on this information, a revised engineering analysis found that all of the structural members (pony truss elements, columns, second floor beam, tie rod) are stressed within allowable limits under the full dead and live loads required by the Massachusetts State Building Code.

2. A visible deflection is present in the second floor under the corridor wall at the southeast bedroom (Mrs. Paine's bedroom). The wall on the north side of the corridor is a continuous bearing wall from the basement to the attic but the wall on the south side of the corridor sits on the second floor and is present only between the second floor and attic. The engineering analysis that we made last year indicated that the joists were highly stressed under the loads required by the present building code but that the stresses are not necessarily unsafe. Insofar as our analysis was based on old drawings and written information from previous reports, we requested that an opening be made in the second floor so that the joists could be inspected and any signs of cracking, notching or other distress could be discovered. The joists were exposed at a hot air floor grate and yesterday we observed that the wood is a finely-grained pine (hard pine was specified in the original construction drawings) with small, tight, widely spaced knots. Checking is limited to a hairline crack at the midheight of the joists. The wood is probably equivalent to a modern Select Structural grade with a safe allowable stress approaching 1,600 psi. Except for a one inch deep notch in the bottom of the joists where they override the bearing wall, the joists are full depth at their supported ends.

   Setting up a computer model to represent the second floor joists, corridor wall and attic joists as a structural system acting together, we found that the attic framing more or less supports the dead load of the attic for the typical 27 foot distance across Mrs. Paine's bedroom and the corridor.

However, the bearing wall on the north side of the dressing room is shifted two feet to the north from the line of the typical north corridor bearing wall such that the attic joists must span 29 feet across Mrs. Paine's bedroom and the dressing room. This additional span doubles the attic floor load on the wall on the south side of the corridor. Although the attic floor load on the wall is relatively small, there are additional dead loads present to locally raise the stresses and deflections in a few joists near the entry door into Mrs. Paine's bedroom: the wall into the dressing room; and large wardrobes on both sides of the south wall. The observed deflection is further aggravated by the effects of creep under sustained dead loads; that is, the initial, calculated deflection will gradually double under sustained pressure even though the stresses remain the same. This appears to be what has occurred in the area we observed. One component of the aggravating dead load is the 1-1/2 inch thick sound deadening mortar seen between the joists on both the second and attic levels. This mortar represents about 40 percent of the dead load.

Based on our load and deflection analysis, we recommend that no live load, i.e. stored items, be permitted in the attic and no more than 15 people be allowed in Mrs. Paine's bedroom at any one time. The attic should be posted with signs warning that no loads should be placed east of the pony truss discussed in item 1, above.

3. The window wall of the south-facing shed dormer has leaned outward. The wall and the portion of the roof sitting on the wall are not supported on the attic floor but rather on a low 4x12 header beam under the windows. The beam frames into the rafters on both sides of the dormer. If the full 30 psf snow load is placed on the roof, the beam would be severely overstressed. As a practical matter, the snow probably slides off the metal seam roof before it can significantly accumulate.

   The tilt in the window wall is probably not related to the undersized beam. Because there is no ridge beam, the roof is unbalanced. The steeply sloping north roof leans against the shallowly sloped dormer roof and pushes it over. Theoretically, the roof framing is unstable at the shed dormer. However, the roof sheathing probably redistributes the loads and restrains further movement.

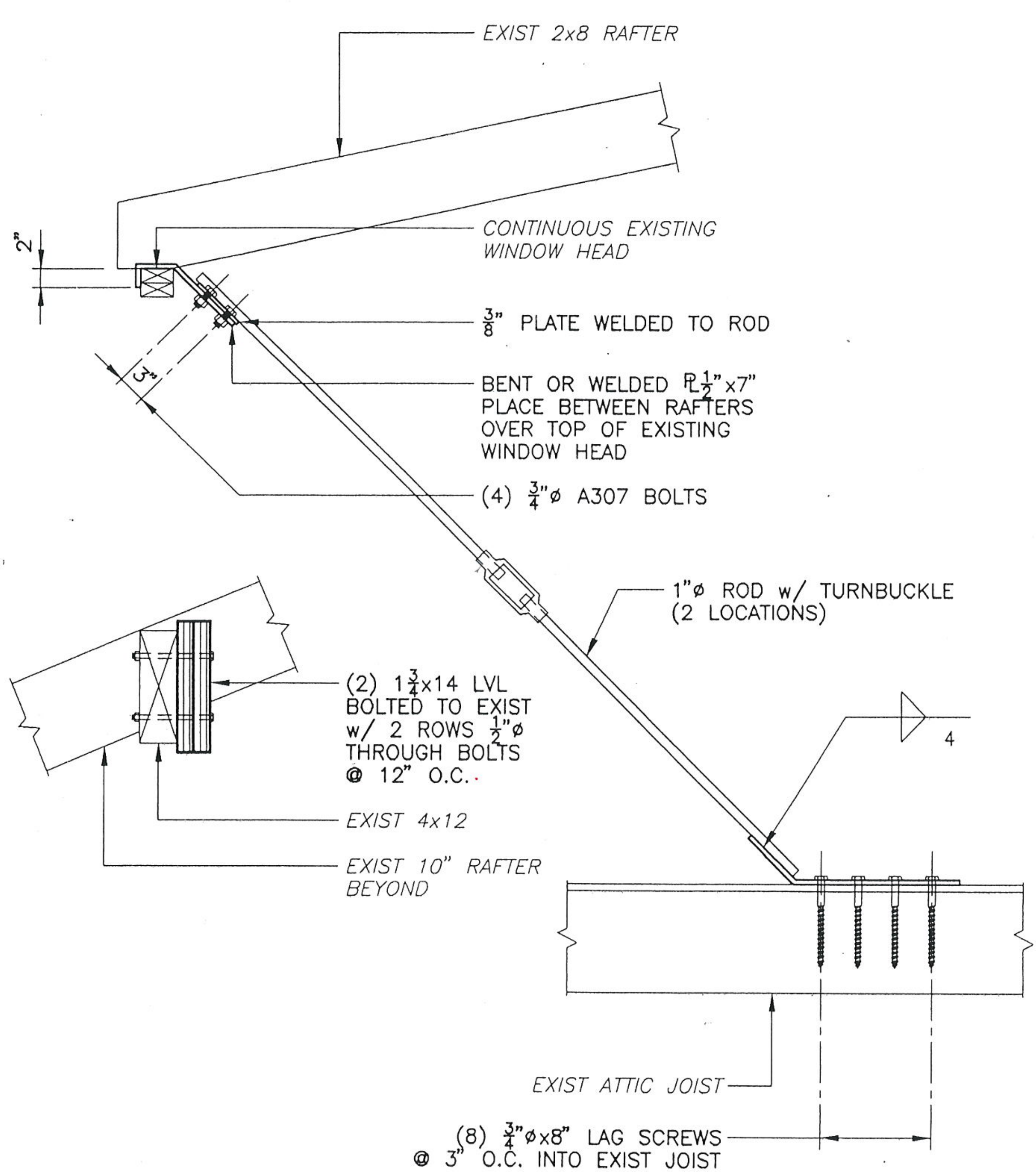
   We recommend that the 4x12 beam under the windows be reinforced with two 1-3/4"x14" sisters and that diagonal steel tie-rods be added in the attic space to brace the window wall. The tie rods would go from the top of the window wall down to the attic floor at about a 45 degree angle.

4. The brick over the fireplace opening in the northeast bedroom is supported by two visibly sagging ½" x 1-1/2"iron or steel bars. Although there is some surface corrosion on the bars, the stresses due to a triangular shaped load of brick above the bars is relatively low.

Please call to discuss the documentation that will be needed to reinforce the shed dormer structure.

Sincerely,
Ocmulgee Associates, Inc.

Wayne C. King, P.E.



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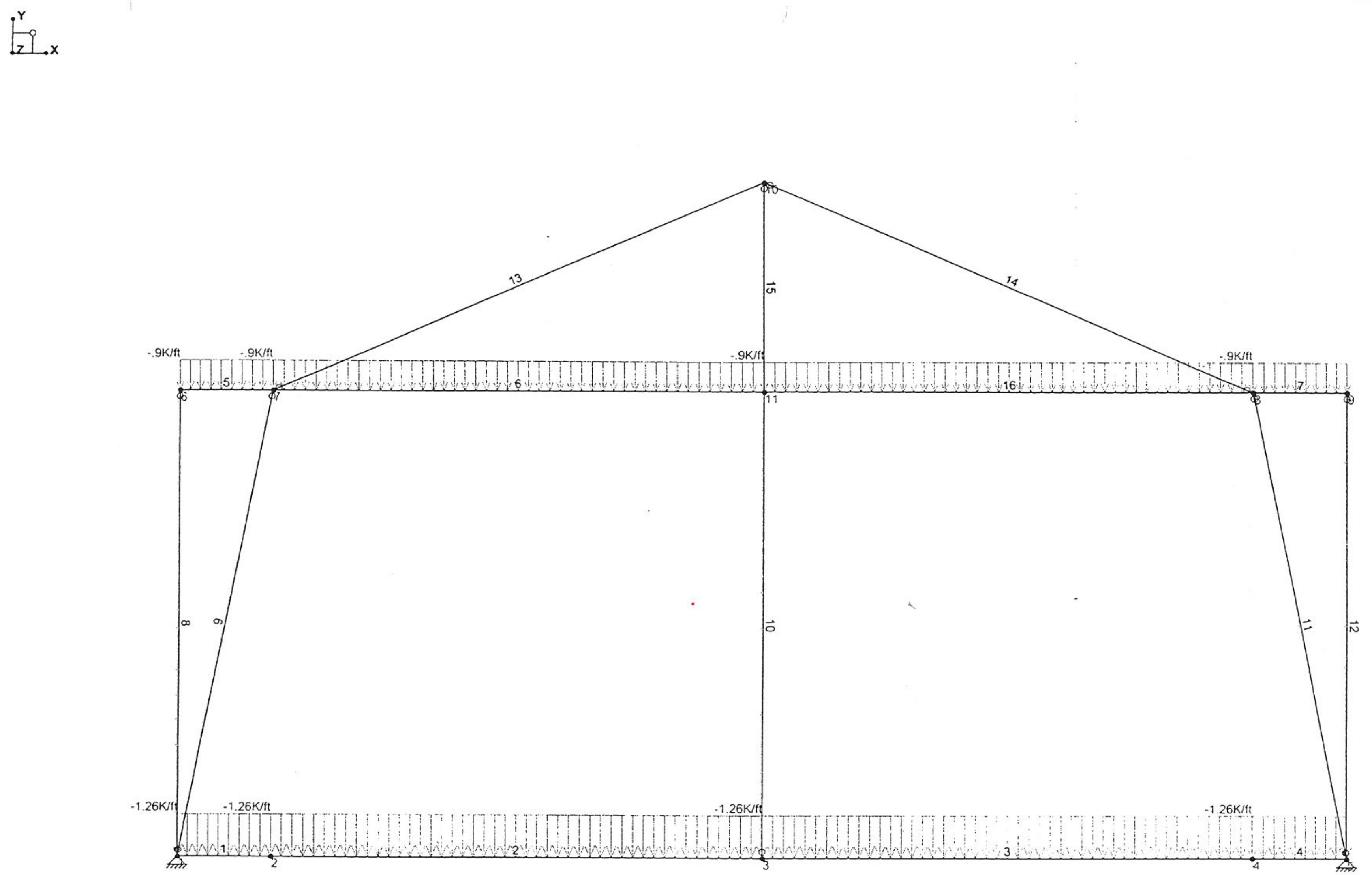
Project: **PAINE ESTATE**
Stonehurst

Title: **ATTIC SHED ROOF WALL REINF DETAIL**

Sheet No: **SKS-1**

Scale: 3/4" | Date: 03.07.01 | Drawn: RGC | Checked: WCK



Loads: BLC 1, TOTAL LOAD

Ocmulgee Associates, Inc.	Paine Estate, Stonehurst Pony Truss	March 3, 2001
Wayne C. King		3:34 PM
20016.1		PainePonyTruss.r2d

Basic Load Case Data

BLC No.	Basic Load Case Description	Category Code	Category Description	Gravity X	Gravity Y	Load Type Totals Nodal	Point	Dist.
1	TOTAL LOAD	None						8

Boundary Conditions

Joint Label	X Translation (K/in)	Y Translation (K/in)	Rotation (K-ft/rad)
1	Reaction	Reaction	
5	Reaction	Reaction	

Member Distributed Loads, Category : None, BLC 1 : TOTAL LOAD

Member Label	I Joint	J Joint	Load Pattern Label	Pattern Multiplier
1	1	2	UNIFORMY	1.26
2	2	3	UNIFORMY	1.26
3	3	4	UNIFORMY	1.26
4	4	5	UNIFORMY	1.26
5	6	7	UNIFORMY	.9
6	7	11	UNIFORMY	.9
7	8	9	UNIFORMY	.9
16	11	8	UNIFORMY	.9

Materials (General)

Material Label	Young's Modulus (Ksi)	Shear Modulus (Ksi)	Poisson's Ratio	Thermal Coef. (per 10^5 F)	Weight Density (K/ft^3)	Yield Stress (Ksi)
STL	29000	11154	.3	.65	.49	36
WOOD	1600		0	0	0	0

Joint Coordinates

Joint Label	X Coordinate (Ft)	Y Coordinate (Ft)	Joint Temperature (F)
1	0	0	0
2	2	0	0
3	12.5	0	0
4	23	0	0
5	25	0	0
6	0	10	0
7	2	10	0
8	23	10	0
9	25	10	0
10	12.5	14.5	0
11	12.5	10	0

Joint Displacements, LC 1 : TOTAL LOAD

Joint Label	X Translation (In)	Y Translation (In)	Rotation (radians)
1	0.000	0.000	-0.006
2	0.000	-0.143	-0.006
3	0.000	-0.408	0.000
4	0.000	-0.143	0.006
5	0.000	0.000	0.006

Joint Displacements, LC 1 : TOTAL LOAD, (continued)

Joint Label	X Translation (In)	Y Translation (In)	Rotation (radians)
6	-0.043	0.006	-0.001
7	-0.043	-0.041	-0.005
8	0.043	-0.041	0.005
9	0.043	0.006	0.001
10	0.000	-0.257	-0.002
11	0.000	-0.319	0.000

Reactions, LC 1 : TOTAL LOAD

Joint Label	X Force (K)	Y Force (K)	Moment (K-ft)
1	4.506	27	0.000
5	-4.506	27	0.000
Reaction Totals :	0.000	54	

Center of Gravity Coords (X,Y) (Ft) :	12.5	4.167

Member Section Forces, LC 1 : TOTAL LOAD

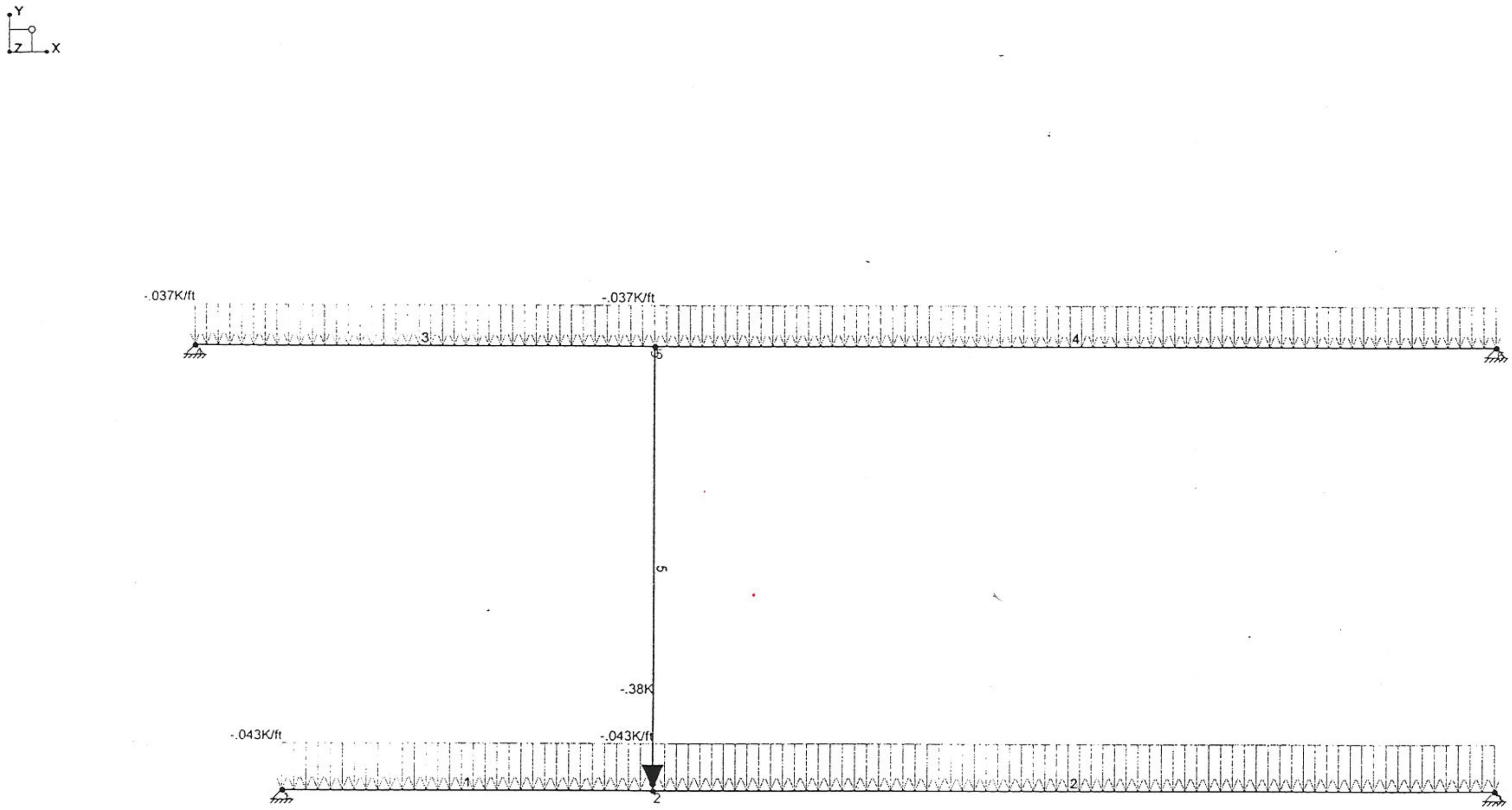
Member Label	Section	Axial (K)	Shear (K)	Moment (K-ft)
1	1	0	7.243	0
	2	0	6.613	-3.464
	3	0	5.983	-6.613
	4	0	5.353	-9.447
	5	0	4.723	-11.966
2	1	0	4.723	-11.966
	2	0	1.416	-20.024
	3	0	-1.892	-19.399
	4	0	-5.199	-10.092
	5	0	-8.507	7.898
3	1	0	8.507	7.898
	2	0	5.199	-10.092
	3	0	1.892	-19.399
	4	0	-1.416	-20.024
	5	0	-4.723	-11.966
4	1	0	-4.723	-11.966
	2	0	-5.353	-9.447
	3	0	-5.983	-6.613
	4	0	-6.613	-3.464
	5	0	-7.243	0
5	1	0	-2.773	0
	2	0	-3.223	1.499
	3	0	-3.673	3.223
	4	0	-4.123	5.173
	5	0	-4.573	7.347
6	1	-26.451	4.689	7.347
	2	-26.451	2.327	-1.862
	3	-26.451	-0.036	-4.87
	4	-26.451	-2.398	-1.676
	5	-26.451	-4.761	7.72
7	1	0	4.573	7.347

Member Section Forces, LC 1 : TOTAL LOAD, (continued)

Member Label	Section	Axial (K)	Shear (K)	Moment (K-ft)
	2	0	4.123	5.173
	3	0	3.673	3.223
	4	0	3.223	1.499
	5	0	2.773	0
8	1	-2.773	0	0
	2	-2.773	0	0
	3	-2.773	0	0
	4	-2.773	0	0
	5	-2.773	0	0
9	1	22.976	0	0
	2	22.976	0	0
	3	22.976	0	0
	4	22.976	0	0
	5	22.976	0	0
10	1	-17.014	0	0
	2	-17.014	0	0
	3	-17.014	0	0
	4	-17.014	0	0
	5	-17.014	0	0
11	1	22.976	0	0
	2	22.976	0	0
	3	22.976	0	0
	4	22.976	0	0
	5	22.976	0	0
12	1	-2.773	0	0
	2	-2.773	0	0
	3	-2.773	0	0
	4	-2.773	0	0
	5	-2.773	0	0
13	1	33.68	0	0
	2	33.68	0	0
	3	33.68	0	0
	4	33.68	0	0
	5	33.68	0	0
14	1	33.68	0	0
	2	33.68	0	0
	3	33.68	0	0
	4	33.68	0	0
	5	33.68	0	0
15	1	-26.535	0	0
	2	-26.535	0	0
	3	-26.535	0	0
	4	-26.535	0	0
	5	-26.535	0	0
16	1	-26.451	4.761	7.72
	2	-26.451	2.398	-1.676
	3	-26.451	0.036	-4.87
	4	-26.451	-2.327	-1.862
	5	-26.451	-4.689	7.347

Member Data

Member Label	I Joint	J Joint	Rotate (degrees)	Section Set	End Releases I-End AVM	End Releases J-End AVM	End Offsets I-End (In)	End Offsets J-End (In)	Inactive Code	Member Length (Ft)
1	1	2		SEC1						2
2	2	3		SEC1						10.5
3	3	4		SEC1						10.5
4	4	5		SEC1						2
5	6	7		SEC2						2
6	7	11		SEC2						10.5
7	8	9		SEC2						2
8	1	6		SEC3	PIN	PIN				10
9	1	7		SEC3	PIN	PIN				10.198
10	3	11		SEC4	PIN					10
11	5	8		SEC3	PIN	PIN				10.198
12	5	9		SEC3	PIN	PIN				10
13	7	10		SEC5	PIN					11.424
14	10	8		SEC5	PIN	PIN				11.424
15	11	10		SEC4		PIN				4.5
16	11	8		SEC2						10.5



Loads: BLC 1, DEAD LOAD ONLY

Ocmulgee Associates, Inc.	Paine Estate, Stonehurst Mansion	March 3, 2001
Wayne C. King		3:46 PM
20016.1		PaineEstate.r2d

Basic Load Case Data

BLC No.	Basic Load Case Description	Category Code	Category Description	Gravity X	Gravity Y	Load Type Totals Nodal	Point	Dist.
1	DEAD LOAD ONLY	None				1		4
2	LIVE LOAD ONLY	None						2

Boundary Conditions

Joint Label	X Translation (K/in)	Y Translation (K/in)	Rotation (K-ft/rad)
1	Reaction	Reaction	
3	Reaction	Reaction	
4	Reaction	Reaction	
6	Reaction	Reaction	

Member Distributed Loads, Category : None, BLC 1 : DEAD LOAD ONLY

Member Label	I Joint	J Joint	Load Pattern Label	Pattern Multiplier
1	1	2	UNIFORMY	.043
2	2	3	UNIFORMY	.043
3	4	5	UNIFORMY	.037
4	5	6	UNIFORMY	.037

Member Distributed Loads, Category : None, BLC 2 : LIVE LOAD ONLY

Member Label	I Joint	J Joint	Load Pattern Label	Pattern Multiplier
1	1	2	UNIFORMY	.038
2	2	3	UNIFORMY	.038

Materials (General)

Material Label	Young's Modulus (Ksi)	Shear Modulus (Ksi)	Poisson's Ratio	Thermal Coef. (per 10^5 F)	Weight Density (K/ft^3)	Yield Stress (Ksi)
WOOD	1440		0	0	0	0

Joint Coordinates

Joint Label	X Coordinate (Ft)	Y Coordinate (Ft)	Joint Temperature (F)
1	0	0	0
2	8.25	0	0
3	27	0	0
4	-2	10	0
5	8.25	10	0
6	27	10	0

Joint Displacements, LC 1 : DL ONLY

Joint Label	X Translation (In)	Y Translation (In)	Rotation (radians)
1	0.000	0.000	-0.015
2	0.000	-1.251	-0.008
3	0.000	0.000	0.014
4	0.000	0.000	-0.013
5	0.000	-1.253	-0.006
6	0.000	0.000	0.014

Joint Loads/Enforced Displacements, Category : None, BLC 1 : DEAD LOAD ONLY

Joint Label	[L]oad or [D]isplacement	Direction	Magnitude (K, K-ft, In, rad)
2	L	Y	-.38

Reactions, LC 1 : DL ONLY

Joint Label	X Force (K)	Y Force (K)	Moment (K-ft)
1	0.000	1.06	0.000
3	0.000	0.792	0.000
4	0.000	0.336	0.000
6	0.000	0.427	0.000
Reaction Totals :	0.000	2.614	

Center of Gravity Coords (X,Y) (Ft) :	12.326	4.105

Member Deflections, LC 1 : DL ONLY

Member Label	Section	x-Translation (In)	y-Translation (In)	(n) L/y Ratio
1	1	0	0	NC
	2	0	-0.372	1680.244
	3	0	-0.719	1061.51
	4	0	-1.019	1230.813
	5	0	-1.251	NC
2	1	0	-1.251	NC
	2	0	-1.464	427.827
	3	0	-1.262	353.361
	4	0	-0.728	542.451
	5	0	0	NC
3	1	0	0	NC
	2	0	-0.385	1723.821
	3	0	-0.735	1130.356
	4	0	-1.029	1381.961
	5	0	-1.253	NC
4	1	0	-1.253	NC
	2	0	-1.452	439.815
	3	0	-1.291	338.793
	4	0	-0.766	497.221
	5	0	0	NC
5	1	-1.251	0	NC
	2	-1.252	0	NC
	3	-1.252	0	NC
	4	-1.253	0	NC
	5	-1.253	0	NC

Member Section Forces, LC 1 : DL ONLY

Member Label	Section	Axial (K)	Shear (K)	Moment (K-ft)
1	1	0	1.06	0
	2	0	0.971	-2.095
	3	0	0.883	-4.007

Member Section Forces, LC 1 : DL ONLY, (continued)

Member Label	Section	Axial (K)	Shear (K)	Moment (K-ft)
	4	0	0.794	-5.737
	5	0	0.705	-7.283
2	1	0	0.015	-7.283
	2	0	-0.187	-6.879
	3	0	-0.388	-5.531
	4	0	-0.59	-3.238
	5	0	-0.792	0
3	1	0	0.336	0
	2	0	0.241	-0.739
	3	0	0.146	-1.234
	4	0	0.051	-1.487
	5	0	-0.044	-1.496
4	1	0	0.267	-1.496
	2	0	0.094	-2.342
	3	0	-0.08	-2.374
	4	0	-0.253	-1.594
	5	0	-0.427	0
5	1	0.311	0	0
	2	0.311	0	0
	3	0.311	0	0
	4	0.311	0	0
	5	0.311	0	0

Member Data

Member Label	I Joint	J Joint	Rotate (degrees)	Section Set	End Releases I-End AVM	J-End AVM	End Offsets I-End (In)	J-End (In)	Inactive Code	Member Length (Ft)
1	1	2		SEC1						8.25
2	2	3		SEC1						18.75
3	4	5		SEC2						10.25
4	5	6		SEC2						18.75
5	2	5		SEC3	PIN	PIN				10

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February 3, 2000

Lynne Spencer
Claude Emanuel Menders, Architects
59 Commercial Wharf
Boston, Massachusetts 02110

Reference: Stonehurst, Paine Estate, Waltham, Massachusetts
OA File 20016

Dear Lynne:

Ocmulgee Associates was asked by Claude Emanuel Menders to review a November 15, 1999 structural conditions report prepared by TBA Architects at 241 Crescent Street in Waltham. Your firm sent a copy of this report, which included framing plans prepared by TBA, and original 1883 drawings and specifications prepared by H. H. Richardson. On Thursday, January 27, I met you and Bill Finch of Finch and Rose (architectural conservators) at Stonehurst and I personally inspected the building in order to understand the issues raised in the TBA report.

The most important structural issues were:

1. Lateral drift of a large, decorative summer beam in the ceiling of the Great Hall and its three columns.
2. Improper fabrication of the king rod pony truss in the attic; a steel hanger rod from this truss supports the decorative Great Hall summer beam at its midspan.
3. Distress in the brick basement pier that supports one of the three columns in the Great Hall.
4. Excessive deflection in the second floor corridor and southeast bedroom.
5. Distress at most of the hearths and adjacent floors.
6. Distress in the first floor beams supporting the bearing wall between the Summer Parlor and Mr. Paine's study.

<u>Lateral Drift of the Great Hall Beam and Columns</u>

Column Plumbness. Using a six foot level, we observed that the northernmost column and the one at the foot of the steps were out of plumb about 1/4 inch in six feet (height/288) while the one at the intermediate landing was perfectly plumb. Except for the buckling in the spool screen adjacent to the northernmost column, there was no sign of distress or distortion in the ceiling or in the handrails and moldings at the stairs.

Distortions in Timbers. The timber beam and columns were identified as southern pine, based on the light and dark grain pattern and knowledge of its common use in the late 1800's. The six foot level revealed bowing in the two out-of-plumb columns and twisting was observed in stress resulting from the diagonal members of the pony truss not converging on the columns.

Brick Pier Distress

Cracking at the Top. The 20 inch square brick pier that supports the first floor beams and the northernmost column of the Great Hall that was discussed above has hairline vertical cracks stepping down about four to six courses. While vertical cracking is usually associated with splitting from excessive compressive forces, the observed cracks were outside the area of the piers actually receiving pressure from the beam and column. That is, the cracks are not under the beam or column and are in a zero stress zone; simply removing the bricks beyond the cracks would not affect the load-carrying capacity of the pier.

Because the column is doubly notched to fit over and distribute its load into the floor beams, it is implausible that any twisting in the column could have caused the brick to split, especially since there is a thin wood shim under the column that can absorb and redistribute any differential pressures. An engineering analysis found that the direct stress under the beam and column is about 120 psi and only 65 psi on the gross area of the pier, well below the allowable 25 percent of masonry strength. We assumed that the masonry strength, fm' is conservatively about 600 to 800 psi; it could be higher considering that cement mortars were specified by H. H. Richardson.

It should be noted that another 20 inch square brick pier with a similar but higher loading condition did not have cracks.

We also noted that the base of the pier was not damaged from moisture. There may be minor rising damp action occurring insofar as the whitewash coating has flaked off in the lower courses, but there has not been any significant mortar erosion.

Excessive Deflections in Second Floor

There are visible distortion in the second floor corridor and southeast bedroom, especially near the north end under the bedroom-corridor wall and built-in wardrobes. We performed an engineering analysis of the 3x12 joists and found that they are stressed to about 1,500 psi under the dead loads of the second floor and attic and live loads of 30 psf on the second level and 20 psf in the unfinished attic. While this calculated stress level with full code live load present is greater than the allowable stress of 1,200 psi it is well below failure and is not unusual for wood buildings of this age.

The engineering analysis found that the dead load deflection plus long term creep is about 2 inches, confirming the deflections that were visually observed.

Distress Around Hearths

deflections may be symptoms of localized failures of some joists. For example, if the joists are notched into the supporting beams, long horizontal shear splits may have developed that have weakened the joists' bending capacity. Therefore, the ceiling of the Summer Parlor should be opened up under the second floor corridor in order to verify the condition of the joists. Alternatively, access may be achieved by removing the flooring and underlayment boards of the second floor in the bedroom and corridor.

5. No structural remedial action is needed at the hearths.

6. Add pressure treated timber or steel lally columns between the brick piers and under the timber beams in the basement that support the bearing wall between the Summer Parlor and Mr. Paine's study.

<u>Other Issues</u>

The TBA report discusses other issues but these have either been addressed or are being addressed. These include the addition of the bearing studs in the basement and the acknowledgment by the house staff that rotted joist ends in the basement under Mr. Paine's study need to be propped with additional bearing studs. Insofar as this review deals with structural issues, we have not addressed any issues related to water infiltration, such as at the chimneys, or related to aesthetic issues, such as how to straighten the spool screen.

If you have any questions or comments, please call me.

Sincerely,
Ocmulgee Associates, Inc.

Wayne C. King

Wayne C. King, P.E.