TAYLOR & VIOLA
STRUCTURAL ENGINEERS

JAMES S. TATE, P.E. JASON M. REEP, P.E. JOSHUA J. WINCHESTER, S. E. J. KIRK VIOLA, P.E. VANCE D. CARRIGAN, P.E.

January 18, 2022

Stephen Overcash
ODA Architecture
2010 S. Tryon St. Suite 1a
Charlotte, North Carolina

RE: Hotel Aiken Site Visit and Report

Dear Mr. Overcash,

A representative from Taylor and Viola Structural Engineers visited the Hotel Aiken site on January 4, 2022. The purpose of the site visit was to assess the condition of the existing structure and evaluate the building for future occupancy. Please find the requested structural assessment report attached for your use.

Should you have any questions regarding this report, our findings on site, or require further information, please feel free to contact our office.

Sincerely,
TAYLOR & VIOLA STRUCTURAL ENGINEERS, P.C.

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 21820 JAMES S. TATE

SOUTH CAROLINA TAYLOR & VIOLA STRUCTURAL ENGINEERS No. C00130 CERTIFICATE OF AUTHORIZATION

01.18.2022

James S. Tate, PE

P.O. BOX 2616 ▪ 308 15TH STREET SE ▪ HICKORY, NC 28603
PHONE: 828.328.6331 ▪ FAX: 828.322.1801 ▪ WWW.TAYLORVIOLA.COM

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Hotel Aiken - Structural Assessment Summary

As presented in the following structural assessment report, two types of structural deficiencies were noted for this building. The first type involves structural elements that are in generally poor condition. The second type involves existing structural elements which are in otherwise fair condition, but are sized inappropriately to meet current design standards. The International Existing Building Code (IEBC) would require remediation for a significant portion of this structure and/or load restrictions that would affect permitted use and occupancy. A brief summation is provided below and more detailed information may be found in the assessment report.

A number of structural elements were noted to be in poor condition. These elements have either degraded over time, have been modified excessively, or have suffered water or other environmental damage. For instance, exterior multi-wythe brick bearing walls appear to have settled and mortar joints have degraded over time leading to widespread cracking. Other examples include wood floor and wall framing that has been modified excessively to accommodate other systems, roof framing showing considerable water damage, and compromised soil bearing conditions.

Other framing elements were observed to be in generally fair condition but nevertheless raised concern due to improper sizing and/or bracing. Analysis revealed that both existing floor joists and existing 2x4 interior bearing walls are inadequate, regardless of condition, for the current configuration and intended use. Current codes also require exterior masonry walls to be tied to floor diaphragms for bracing against environmental loading. Such ties are not present and should be installed if the building is to be occupied in the future. A full lateral analysis of the building was not performed, but it is expected that diaphragm reinforcement and provisions for supplemental interior shear walls would be necessary to bring the building into compliance with current code design standards.

Much of the building can be preserved if desired, but it should be expected that preservation measures would be extensive in nature. In addition to repairing damaged structural elements, existing floors and interior wood bearing walls will require strengthening, and significant measures will need to be taken to stabilize existing exterior multi-wythe brick walls per IEBC standards.

P.O. BOX 2616 ▪ 308 15TH STREET SE ▪ HICKORY, NC 28603
PHONE: 828.328.6331 ▪ FAX: 828.322.1801 ▪ WWW.TAYLORVIOLA.COM

Hotel Aiken - Structural Assessment

235 Richland Ave. W, Aiken, SC

General Description and Comments

A representative from Taylor and Viola Structural Engineers visited the Hotel Aiken site on January 4, 2022. The purpose of the site visit was to assess the condition of the existing structure and evaluate the building for future occupancy.

According to public record the hotel was originally built in 1898 and has undergone several renovations over the years, including what appears to be a relatively recent partial renovation effort. The building is approximately 50 ft wide by approximately 180 ft. long and is three stories standing roughly 50 ft. tall. Exterior walls are multi-wythe brick which are load bearing along the longitudinal side walls of the building. Interior framing consists of rough cut 2x lumber assumed to be Southern Pine.

Structural elements were found to be in fair-to-poor condition. According to the International Existing Building Code (IEBC), substantial structural damage is defined to have occurred where "vertical elements of the lateral force-resisting system have suffered damage such that the lateral load-carrying capacity of any story in any horizontal direction has been reduced by more than 33 percent from its pre-damage condition." Similar explanations of substantial structural damage are provided for vertical gravity load carrying elements. Due to the age and type of building construction, pre-damage capacity and likewise the degree of structural degradation is difficult to quantify. Nevertheless, it is apparent that damage has occurred, and per IEBC Section 405, damaged elements should be restored to their pre-damaged condition (also difficult to quantify). Therefore, if it is intended to occupy the exiting building, structural elements requiring repair should be restored in compliance with current standards rather than attempting to replicate outdated practices. Further, although damage is not clearly defined as being substantial for this building, it is encouraged to consider repairs to be substantial in nature and to retrofit the building for lateral demand per IEBC Section 405.2.3.3.

Information is provided in the "Structural Evaluation" section of this report to present structural concerns for reference and to assist with decision making. Evaluation of structural elements, regardless of condition, revealed deficiencies per current design standards. Such elements should be strengthened to meet current standards, or alternatively for gravity systems, placards may be posted in

areas of the building with deficient framing stating approved (reduced) live loads per Section 304 IEBC.

The full extents of planned structural alterations are unknown at this time and it was also not evident at the time of the site visit whether or not interior shear walls were present and diaphragms properly fastened to such elements. Therefore, a full lateral analysis of the building was not performed. However, according to Chapter 9 of the IEBC, "where work involves a substantial structural alteration, the lateral load-resisting system of the altered building shall be shown to satisfy the requirements of Sections 1609 and 1613 of the IBC." Details to comply with this code requirement are beyond the scope of this report, but a full lateral analysis and subsequent diaphragm and shear wall reinforcement to accommodate such measures should be considered in planning.

According to the 2021 International Existing Building Code (IEBC), three options exist for owners and design teams when dealing with alterations of existing buildings. Option 1 is the Prescriptive Compliance Method which essentially requires any addition or alteration to be compliant with the current building code. Option 2 is the Work Area Compliance Method which has different requirements depending upon the level or overall extent of alterations. Option 3 is the Performance Compliance Method which requires in-situ testing.

This report assumes Option 2, the Work Area Compliance Method, will be used for compliance with the IEBC. It is assumed that if the building is intended to function as a hotel in the future, a Level 3 alteration would be necessary for renovation. According to the IEBC, a Level 3 alteration consists of "adding or eliminating any door or window, the reconfiguration or extension of any system, or the installation of any additional equipment" where the work area exceeds 50 percent of the building area. Level 3 alterations must conform to Chapters 7-9 of the IEBC, and subsequent sections of this report take that into consideration.

Architectural features are beyond the scope of this report, but in general, all architectural elements/finishes/etc. were found to be in poor condition.

An outline of general existing conditions is presented below followed by a brief structural evaluation of existing structural elements. An exhaustive survey of the building, cost analysis, or full design and detailing of any necessary repairs is beyond the scope of this report.

Roof Framing

Roof framing spans perpendicular to the longitudinal walls of the building and consists of built-up trusses with 2x top and bottom chords and 1x web members. The roof diaphragm consists of 1x decking laid perpendicular to trusses. Generally, roof framing appeared to be in fair condition with the exception of framing along the south wall of the building and other isolated areas which indicated considerable water damage. Actual dimensions of roof framing were unable to be determined during the visit, but trusses are generally deeper at the North side of the building (Fig.1) and taper down towards the South to provide positive drainage to scuppers located on the South side of the building (Figs. 2-3). Evidence of water intrusion was noted along the south wall of the building (Figs. 2-3) suggesting ponding of rainwater along this wall. Significant water damage was also noted in isolated locations (Fig. 4).

It should further be noted that per IEBC Section 906, buildings assigned to Seismic Design Category C (this building) with a structural system that includes unreinforced masonry (URM) walls that the alteration work "shall include installation of wall anchors at the roof line…and at the floor lines of unreinforced masonry buildings." Per this same section, URM parapets are also required to be braced and/or supported by the roof framing and diaphragm. Repair and bracing of URM walls and parapets and associated reinforcement of roof framing to accommodate such measures should be considered in planning.

Figure 1: Roof Framing at North Wall – Generally Fair Condition

Figure 2: Roof Decking at South Wall – Generally Poor Condition

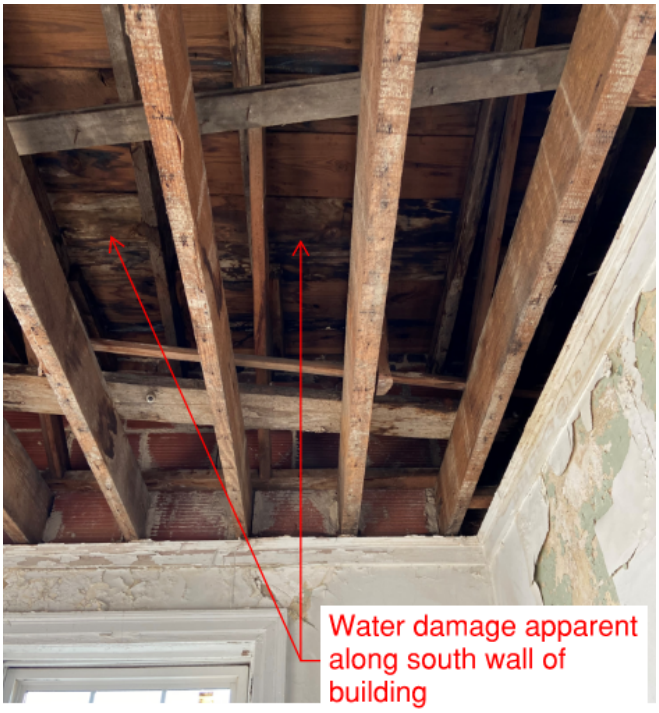

Figure 3: Roof Decking at South Wall – Generally Poor Condition

Figure 4: Roof Framing – Localized Substantial Water Damage

Floor Framing

Floor framing is generally 2x12 or 3x12 rough cut joists varying from 16”-19”oc spanning from exterior multi-wythe brick walls to interior corridor walls except at the central stair location where joists span from demising wall to demising wall. The floor diaphragm consists of a 1x diagonal slat subfloor nailed to floor joists. In general, floor framing appeared to be in fair condition (Fig. 5); however, floor framing has been severely compromised in numerous locations, most notably at existing bathroom locations adjacent to corridors (Figs. 6-7). Further, due to excessive spans of up to approximately 24’-6”, floor joists are inadequate to meet current design standards (see Structural Evaluation section).

It should further be noted that per IEBC Section 906, buildings assigned to Seismic Design Category C (this building) with a structural system that includes unreinforced masonry (URM) walls that the alteration work “shall include installation of wall anchors at the roof line…and at the floor lines of unreinforced masonry buildings.” Repair and bracing of URM at each floor level and associated reinforcement of floor diaphragms to accommodate such measures should be considered in planning.

Figure 5: Floor Framing – Generally Fair Condition

Figure 6: Floor Framing – Severely Compromised Adjacent to Corridors

Figure 7: Floor Framing –Compromised Adjacent to Corridors

Interior Walls

Stud wall framing was found to be rough cut 2x4s at 16”oc typical at all demising and corridor walls full height. Exposed studs at the time of visit were found to be generally in fair condition (Fig. 8) with localized areas that have been severely compromised (Figs. 9-11). Considering stud size and floor-to-floor heights, existing walls were determined to be inadequate to meet current design standards (see Structural Evaluation section). Localized areas of exposed studs were noted where substantial water damage (Fig. 9) and excessive modifications (Fig. 10-11) have severely compromised stud wall framing.

Figure 8: Interior Stud Wall Framing – Generally Fair Condition

At least one interior wall adjacent to the main stair in the center of the building was found to be multi-wythe brick. This wall has been furred out with wood framing and plaster and it was therefore not possible to assess its condition at the time of our site visit.

Figure 9: Severe Water Damage

Figure 10: Compromised Stud Walls – Excessive Modifications

Figure 11: Compromised Stud Walls – Excessive Modifications

Exterior Walls

Exterior walls are multi-wythe brick unreinforced masonry (URM), and are load bearing along the longitudinal sides of the building. It was noted that exterior walls are thicker at the base and step in at upper levels of the building. Exterior walls were found to be in generally fair-to-poor condition (Fig. 12). Over time, mortar joints have degraded and the building has settled which has caused widespread cracking. This building would fall into Seismic Design Category C, and current codes require masonry walls to be tied into and braced against lateral environmental forces by building diaphragms. Such bracing was not found to be present (Figs. 13-14), which was expected considering the building's age; nevertheless, exterior walls should be mechanically fastened and braced at each floor level to restrain lateral forces should it be desired to occupy the building in the future. Outside the building, cracking present at numerous wall openings (Figs. 17-18) and severe cracking was noted in the Southeast corner of the building due presumably to excessive settlement (Figs 15-16).

Figure 12: General Exterior Wall Conditions – Fair-to-Poor Condition

Figure 13: General Exterior Load Bearing Multi-Wythe Brick Wall

Figure 14: General Exterior Load Bearing Multi-Wythe Brick Wall

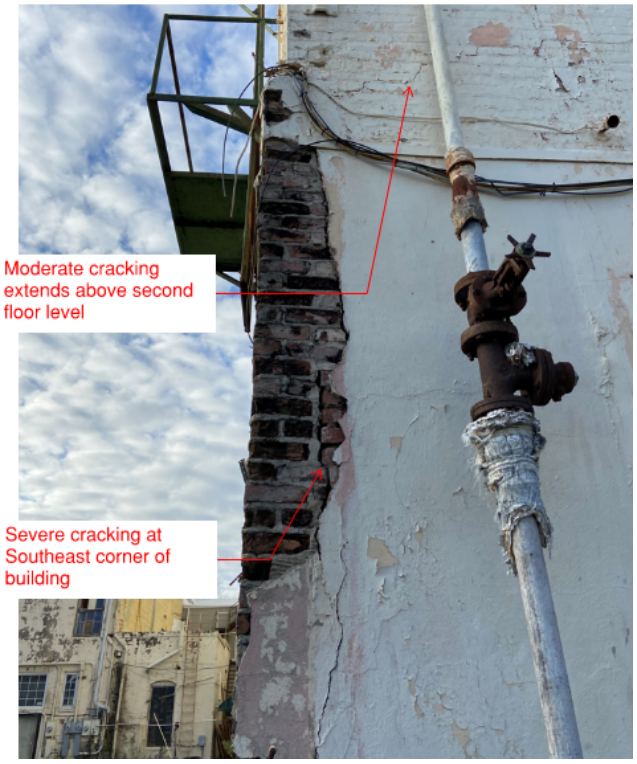

Figure 15: Southeast Building Corner – Severe Cracking

Figure 16: Southeast Building Corner – Severe Cracking

Figure 17: Crack Propagation at Window Openings

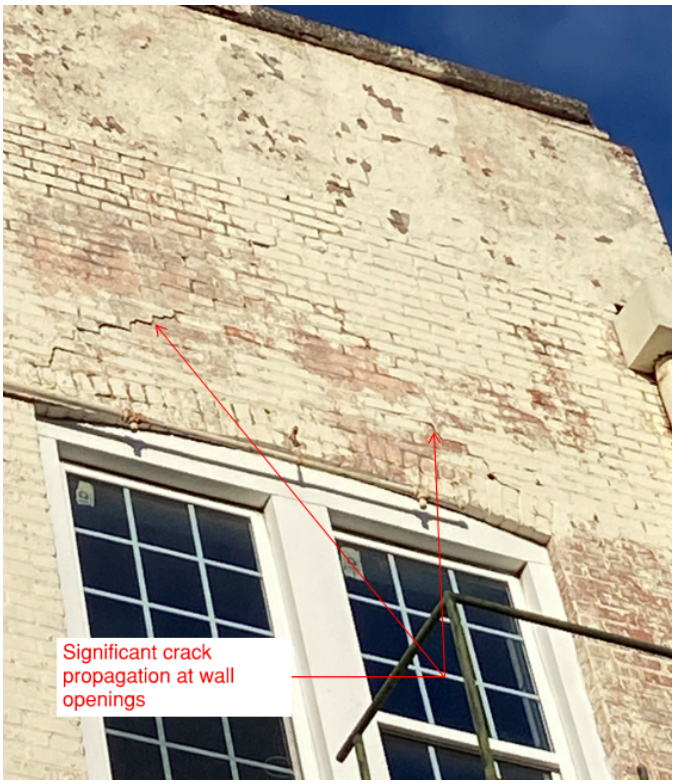

Figure 18: Crack Propagation at Window Openings

Foundations

Existing footings (if present) were not exposed in most locations. Evidence suggests, however, that the multi-wythe brick walls thicken at the base of the wall to form a wider soil bearing area (Fig.19). This would be consistent with our experience with buildings of similar age and construction. There were also locations that indicate a mudsill was poured under masonry walls (Fig.20). Where observed, mudsills were 4-6 inches thick and appeared to be unreinforced (Figs 19-20). These bearing conditions, brick bearing directly on soil, or unreinforced mudsills are highly susceptible to differential settlement. As previously discussed, differential settlement is apparent in the Southeast corner of the building (Figs. 15-16) where significant cracking has occurred. It is likewise also probable that differential settlement due to inadequate foundations is responsible for generally poor wall condition along the longitudinal bearing sides of the building (Fig 21).

Figure 19: Base of Multi-Wythe Exterior Walls

Figure 20: Mudsill Along South Wall Adjacent to Excessive Wall Settlement

Figure 21: Inadequate Bearing Conditions

Structural Evaluation

This section of the report is intended to briefly present analysis results for primary structural elements. Please note that all framing conditions were not clearly visible or accessible at the time of visit. For instance, it was obvious that floor joists are bearing on exterior multi-wythe brick walls, but interior bearing conditions of the same floor joists at corridors or bathroom walls was not clearly observable (Fig.22). Framing conditions described in this section of the report should be verified prior to making any decisions to reuse or replace framing elements.

Two framing conditions in particular raised concerns during our visit. One was the relatively long spans for floor joists, and another was the presence of 2x4 full height bearing walls. A subsequent analysis revealed that these elements are inadequate, regardless of condition, for the current configuration and intended use. Analysis results for these elements are presented below. Also, an abbreviated section for lateral resistance is provided for reference.

Figure 22: Floor Framing Appears to Fully Span from Exterior Walls to Corridor Walls

Floor Joists:

A calculation summary is presented below for 3x12 rough cut joists at 16”oc spanning 24’-6” (Fig. 23). In the calculations below, dead load is considered to be 25 psf, and live load is 40 psf. Floor framing as described above is considered to be more than 20% overstressed per current design standards.

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Wood Beam

Lic. # : KW-06001053

DESCRIPTION: Floor Joists - 3x12

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-05

Material Properties

Analysis Method : Allowable Stress Design
Load Combination ASCE 7-05

Wood Species :
Wood Grade :

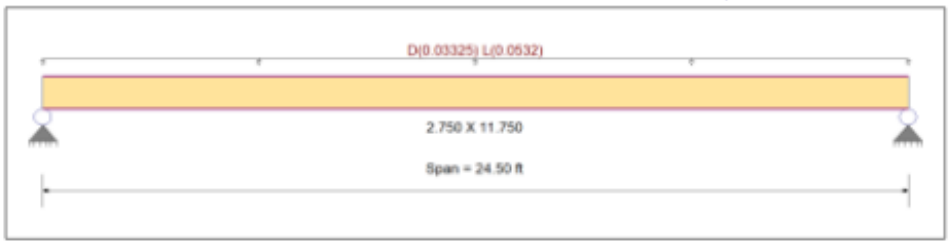
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,000.0 psi
Fb -	1,000.0 psi
Fc - Prll	1,000.0 psi
Fc - Perp	1,000.0 psi
Fv	65.0 psi
Ft	65.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	1,300.0 ksi
Density	34.0 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.0250, L = 0.040 ksf, Tributary Width = 1.330 ft

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	1.230 : 1	Maximum Shear Stress Ratio	=	0.701 : 1
Section used for this span		2.750 X 11.750	Section used for this span		2.750 X 11.750
fb: Actual	=	1,230.07 psi	fv: Actual	=	45.57 psi
Fb: Allowable	=	1,000.00 psi	Fv: Allowable	=	65.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	12.250 ft	Location of maximum on span	=	23.606 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.898 in	Ratio =	327 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.561 in	Ratio =	524 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	Fb	Shear Values V	fv	Fv
D Only													0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.526	0.300	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.49	473.11	900.00	0.38	17.53	58.50
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	1.230	0.701	1.00	1.000	1.00	1.00	1.00	1.00	1.00	6.49	1,230.07	1000.00	0.98	45.57	65.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.378	0.216	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.49	473.11	1250.00	0.38	17.53	81.25
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.411	0.234	1.15	1.000	1.00	1.00	1.00	1.00	1.00	2.49	473.11	1150.00	0.38	17.53	74.75
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.633	0.475	1.25	1.000	1.00	1.00	1.00	1.00	1.00	5.49	1,040.83	1250.00	0.83	38.56	81.25
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.905	0.516	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.49	1,040.83	1150.00	0.83	38.56	74.75
+D+W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.296	0.169	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.49	473.11	1600.00	0.38	17.53	104.00

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DESCRIPTION: Floor Joists - 3x12

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	Fb	Shear Values V	fv	Fv
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.296	0.169	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.49	473.11	1600.00	0.38	17.53	104.00
+D+0.750Lr+0.750L+0.750W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.651	0.371	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.49	1,040.83	1600.00	0.83	38.56	104.00
+D+0.750L+0.750S+0.750W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.651	0.371	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.49	1,040.83	1600.00	0.83	38.56	104.00
+D+0.750Lr+0.750L+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.651	0.371	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.49	1,040.83	1600.00	0.83	38.56	104.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.651	0.371	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.49	1,040.83	1600.00	0.83	38.56	104.00
+0.60D+W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.177	0.101	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.50	283.86	1600.00	0.23	10.52	104.00
+0.60D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 24.50 ft	1	0.177	0.101	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.50	283.86	1600.00	0.23	10.52	104.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
L Only	1	0.8976	12.339		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.059	1.059
Overall MINimum	0.896	0.896
D Only	0.407	0.407
+D+L+H	1.059	1.059
+D+Lr+H	0.407	0.407
+D+S+H	0.407	0.407
+D+0.750Lr+0.750L+H	0.896	0.896
+D+0.750L+0.750S+H	0.896	0.896
+D+W+H	0.407	0.407
+D+0.70E+H	0.407	0.407
+D+0.750Lr+0.750L+0.750W+H	0.896	0.896
+D+0.750L+0.750S+0.750W+H	0.896	0.896
+D+0.750Lr+0.750L+0.5250E+H	0.896	0.896
+D+0.750L+0.750S+0.5250E+H	0.896	0.896
+0.60D+W+H	0.244	0.244
+0.60D+0.70E+H	0.244	0.244
D Only	0.407	0.407
L Only	0.652	0.652
D Only	0.407	0.407
+D+L+H	1.059	1.059
+D+Lr+H	0.407	0.407
+D+S+H	0.407	0.407
+D+0.750Lr+0.750L+H	0.896	0.896
+D+0.750L+0.750S+H	0.896	0.896
+D+W+H	0.407	0.407
+D+0.70E+H	0.407	0.407
+D+0.750Lr+0.750L+0.750W+H	0.896	0.896

Figure 23: 3x12 Floor Framing Calculation Summary

It should be noted that the longer span of 24’-6” was only observed at guestrooms on the north side of the building. A shorter span of approximately 18’-6” was observed at guestrooms on the south side of the building. 3x12 floor framing at 16”oc for this shorter span, 18’-6”, was determined to be acceptable for intended loads assuming that joists have not been cut or modified (Figs. 6-7). It appears, however, that 2x12 floor joists (Fig. 22) were provided in places rather than 3x12 (Fig. 22). 2x12 rough cut joists were determined to be 10% overstressed for design loads at a span of 18’-6”.

2x4 Bearing Wall Framing:

A calculation summary is presented below for 2x4 rough cut studs at the second story spaced at 16”oc with a floor-to-floor height of 13’-4” (Fig. 24). These studs support the 3rd floor and are assumed to support roof framing. In the calculations below, dead load is considered to be 25 psf, floor live load is 40 psf, and roof live load is 20 psf. Wall framing as described above is considered to be more than 10% overstressed per current design standards.

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Wood Column
Lic. # : KW-06001953

DESCRIPTION: Typical Corridor Wall - 2nd-3rd

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities Top & Bottom Pinned
Overall Column Height 13.33 ft
(Used for non-slender calculations)

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

Fb +	1,100.0 psi	Fv	175.0 psi
Fb -	1,100.0 psi	Ft	675.0 psi
Fc - Prll	1,450.0 psi	Density	34.330 pcf
Fc - Perp	565.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,400.0	1,400.0	1,400.0 ksi
Minimum	510.0	510.0	

Wood Section Name 2x4
Wood Grading/Manuf. Graded Lumber
Wood Member Type Sawn

Exact Width	1.750	in
Exact Depth	3.750	in
Area	6.563	in^2
Ix	7.690	in^4
Iy	1.675	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 13.33 ft, K =

Applied Loads

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

Column self weight included : 20.857 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 13.330 ft, D = 0.740, L = 0.890 k

DESIGN SUMMARY

Bending & Shear Check Results

FAIL Max. Axial+Bending Stress Ratio = 1.131 : 1
Load Combination +D+L+H
Governing NDS Formula Comp Only, fc/Fc'
Location of max above base 0.0 ft
At maximum location values are . . .
Applied Axial 1.651 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 222.346 psi

PASS Maximum Shear Stress Ratio = 0.0 : 1
Load Combination +0.60D+0.70E+0.60H
Location of max above base 13.330 ft
Applied Design Shear 0.0 psi
Allowable Shear 280.0 psi

Maximum SERVICE Lateral Load Reactions . .
Top along Y-Y 0.0 k Bottom along Y-Y 0.0 k
Top along X-X 0.0 k Bottom along X-X 0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .
Bending Compression Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.900	0.170	0.5238	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+L+H	1.000	0.153	1.131	FAIL !	0.0 ft	0.0	PASS	13.330 ft
+D+Lr+H	1.250	0.124	0.5174	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+S+H	1.150	0.134	0.5188	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+0.750Lr+0.750L+H	1.250	0.124	0.9713	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+0.750L+0.750S+H	1.150	0.134	0.9739	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+0.60W+H	1.600	0.097	0.5141	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+0.70E+H	1.600	0.097	0.5141	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+0.750Lr+0.750L+0.450W+H	1.600	0.097	0.9650	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+0.750L+0.750S+0.450W+H	1.600	0.097	0.9650	PASS	0.0 ft	0.0	PASS	13.330 ft
+D+0.750L+0.750S+0.5250E+H	1.600	0.097	0.9650	PASS	0.0 ft	0.0	PASS	13.330 ft
+0.60D+0.60W+0.60H	1.600	0.097	0.3084	PASS	0.0 ft	0.0	PASS	13.330 ft

Figure 24: Second Story 2x4 Bearing Wall Calculation Summary

Lateral Resistance:

According to Chapter 9 of the IEBC, "where work involves a substantial structural alteration, the lateral load-resisting system of the altered building shall be shown to satisfy the requirements of Sections 1609 and 1613 of the IBC." However, as previously mentioned, it was not evident at the time of the site visit whether or not interior shear walls were present and diaphragms properly fastened to such elements. Therefore, a full lateral analysis of the building was not performed.

For general information, we typically find with modern wood diaphragms in buildings of similar size is that shear walls are needed at approximately 40-60 ft. intervals. Considering the length of this building, approximately 180 ft., it would be expected to need two-to-three interior lines of shear walls for lateral resistance at a minimum. Should it be desired to occupy the existing building in the future it would be necessary to verify the presence of interior shear walls and confirm lateral capacity or design and install new shear walls as required.

Also, as mentioned in previous sections of this report, current codes require masonry walls to be tied into and braced against lateral environmental forces by building diaphragms. Should it be desired to preserve exterior masonry walls, proper restraint should be provided to resist lateral demand and associated reinforcement of floor diaphragms to accommodate such measures should be considered in planning.

Closing

This report is intended to present structural findings, bring attention to potential structural code requirements, and evaluate the building for future occupancy. Recommendations for future use are beyond the scope of this report, but in general, structural elements were found to be in fair-to-poor condition as presented in previous sections, many of which will require repair per the IEBC. Much of the building can be preserved if desired, but it should be expected that preservation measures would be extensive in nature.