May 31, 2023

Keith Ziobron, PE
Associate Vice President
CHA
t. 678.787.9576
e. KZiobron@chacompanies.com

RE: Markwardt Building Structural Observation

MDS Project # 3422

Site Address: N 1st Ave. & W. Chocktoot Street
Chiloquin, OR 97624

Present: Britt Killian
MD Structural Engineering, Inc.
t. 360.433.9093

Cathy Stuhr
City of Chiloquin
e. cathy@cityofchiloquin.org

Dear Keith,

On April 6, 2023 I made a structural observation of the Markwardt Building in Chiloquin, OR at the above site address. The purpose of my visit was to make a general structural assessment of the existing structure and feasibility of its rehab.

The Markwardt building is a single-story open commercial building approximate interior dimensions of 106 ft x 78 ft in plan with a 30 ft x 78 ft second floor mezzanine at the west end of the building for a total of 10,800 sq ft of floor space. The ceiling height as defined by the bottom of the roof trusses and height of exterior masonry walls is approximately 10 ft. Based on the type and location of the building, it is estimated that the building was constructed some-time between 1920 and 1950.

The general condition of this building is very poor. The north, east and south walls are constructed of concrete masonry units (CMU) and lean outward at the top in varying amounts from 1 ½" to 3 ½" depending on location. The south, east and west walls consist mostly of large window or door openings while the north wall is almost completely without openings. There are multiple settlement/ shear cracks in the masonry walls throughout the eastern half of the building with the worst occurring in the northeast corner where the masonry has separated due to a large vertical settlement of the east wall. One of the roof trusses toward the east end of the building has failed and has been repaired with steel plating which has subsequently failed by buckling. The roof deck is leaking in both the northeast and southeast corners leading to some rotting wood rafter members. The building foundations were not exposed at the time of our visit therefore no observation of the foundations could be made. One masonry pilaster on the north wall has been cut through to make a door opening. The remaining CMU around that door is showing signs of stress evidenced by visible cracking in the adjacent mortar joints. Photos of the observations made and a map of their locations are attached in an appendix for reference.

The western end of the building appears to be in better shape. The structure of the west wall and the western 30 ft of the south wall are constructed from cast in place concrete beams and columns. No leaning of the exterior walls on the west end of the building was observed during our visit.

The roof consists of bowstring trusses with spacing that ranges from 12 ft to 16 ft & topped with 2x6 purlins at 24" OC covered with lumber planks for the roof deck. Roof purlin lumber appears to be in good condition with no real rot or degradation noted. Trusses are supported on the north wall by masonry pilasters integral to the masonry wall and on the south wall by the masonry wall alone without pilasters. Roof purlins are noticeably sagging in the 14 ft & 16 ft spans between trusses. Anchorage of the roof deck to the exterior walls was visible which we understand was from a prior seismic upgrade. Trusses themselves do not appear to be attached to the pilasters.

There is a second floor at the west end of the building which appeared to be constructed with 2x8 @ 16" OC spanning 14 ft between the truss bottom chords and 16 ft between the truss and west wall. The floor joists appeared to be in good condition. No cracking or sagging of the floor joists were noted.

The main floor of the building is a concrete slab on grade with various depressions that appears to be for service equipment used by past building tenants. Severe cracking and settling was noted at the east end of the building. Various cracks in the slab were also present throughout the balance of the building.

In general, the gravity-force-resisting system for the building appears to be laid out in a logical manner with a competent load path to the foundation. I have performed some preliminary calculations of the roof purlins and second floor members to get a feel for their adequacy. Those findings are listed below. I have not performed an analysis of the bowstring trusses at this time however based on past experience, bowstring trusses have performed poorly historically and we anticipate that repair/ reinforcing would be required if they were to be left in service.

1. I reviewed the 2"x6" roof purlins for the 12 ft, 14 ft and 16 ft spans observed assuming the lumber was a select structural Doug-Fire material based on the buildings perceived age. The 14 ft and 16 ft joists are overstressed 8% and 42% respectively when subjected to code applied minimum loading and all three spans are over the code allowed maximum deflections 22%, 94% and 290% respectively.

2. I reviewed the 2^{nd} floor joists for the 14 ft and 16 ft spans also assuming the lumber to be select structural assuming a code minimum office loading. Both of the spans indicate the lumber to be overstressed and over code allowed maximum deflections. The 14 ft joists are 14% overstressed and 77% over allowed deflections. The 16 ft joists are 49% overstressed and 265% over allowed deflections.

Buildings are designed to resist both gravity loads and lateral loads. Lateral loads are resisted by a Lateral Force Resisting System (LFRS). The LFRS is comprised of the building elements that resist wind and seismic forces. At the time the Markwardt building was constructed, lateral forces were not generally considered in the design and construction of buildings. It is now considered a critical part of structural design and building safety. The lateral force resisting system for the Markwardt building consists of unreinforced masonry walls with a large wood roof diaphragm. The walls on the south, east and west sides largely consist of window or door openings leaving very little masonry or concrete to resist lateral loads. Roof diaphragms transfers out of plane wind and seismic forces to walls which in turn transfers those loads to the foundation. The roof diaphragm consists of plank sheathing which performs poorly as a diaphragm and would require upgrading. Unreinforced masonry is one of the poorest performing building materials for seismic design due to its significant weight and brittle nature. In its current condition, it is very likely the building will suffer major damage and has a high likelihood of partial or total collapse in a design seismic event due to the brittle nature of the construction and poor condition.

I understand that this building is intended for use in the future as a community event space. Due to the expected change in occupancy type, the building would be required to be brought up to current code with regard to safe vertical and lateral force resisting systems.

It is my opinion that if this building is to be put into service as intended, several major items must be addressed. Those items are the following:

1. Due to the many large openings through the walls on the east, south and west sides of the building, the poor condition of the masonry and its brittle nature, a new lateral force resisting system (LFRS) would be required to be designed and constructed inside the existing building envelope leaving the existing masonry to serve only as a façade.
2. A new foundation system designed to resist the forces from the new lateral force resisting system would need to be constructed.
3. Additional roof purlins would need to be installed or the existing roof purlins would need to be strengthened to bring their working stresses and deflections within code permitted limits.
4. Damaged or failed trusses would need to be repaired.
5. New supports for the existing timber trusses would need to be installed since the existing pilasters/ walls are currently out of plumb.

Based on experience with past renovation projects, I would estimate the cost to make the minimum repairs/ improvements outlined above to be between $750,000 and $1,000,000.

If you have any questions regarding this letter of require clarification of any items, please do not hesitate to contact me at our office.

Best Regards,

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
75801PE
OREGON
JANUARY 12, 2010
BRITT KILLIAN

Britt Killian, SE 5/31/23

RENEWAL DATE: 6-30-2023

APPENDIX

MD Structural Engineering
500 BROADWAY ST. STE. 460
VANCOUVER, WA. 98660
PHONE: 360-433-9093
EMAIL: MIKE@MDSTRUCTURAL.COM

Project Name: *Markwardt Building*	
Project No: *3422*	Sheet No. -
Date: 5/19/23	Engineer: *BK*

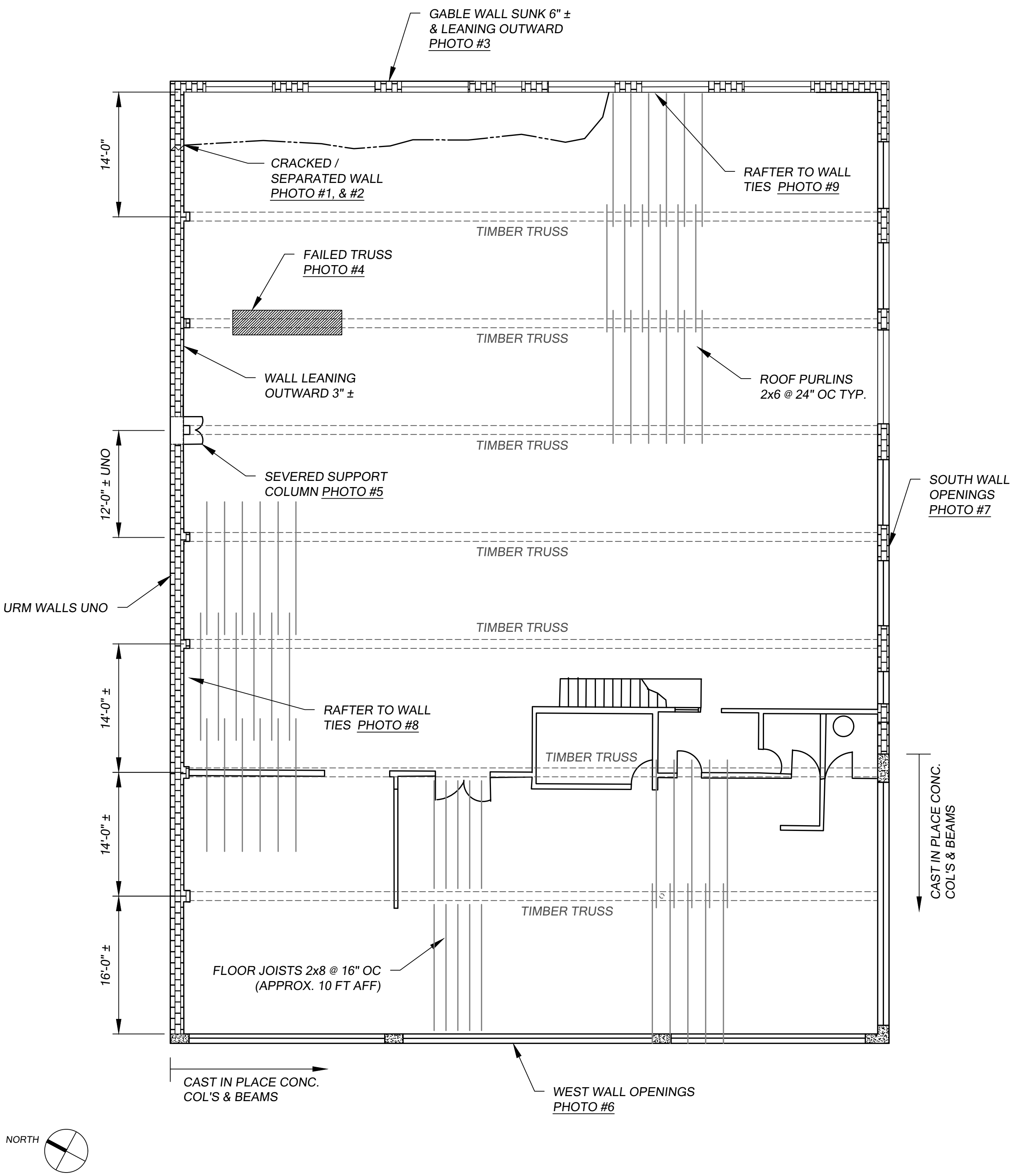

PHOTO MAP

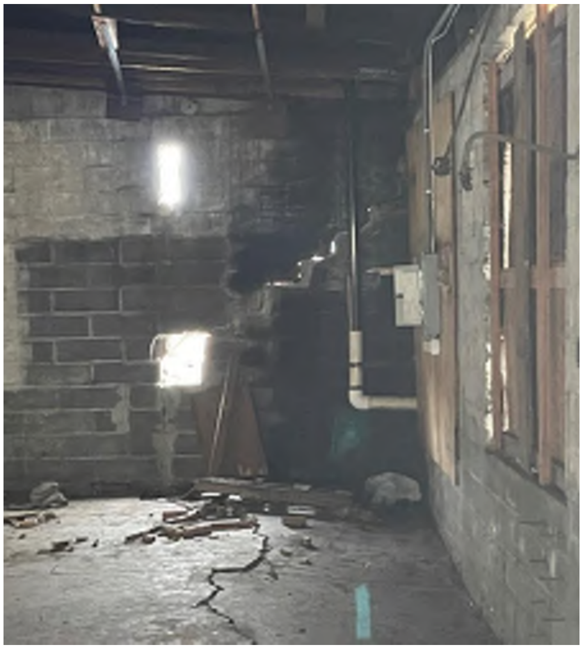

Photo #1 (Cracked/ separated Wall, east side)

Photo #2 (Leaning URM corner)

Photo #3 (West wall openings)

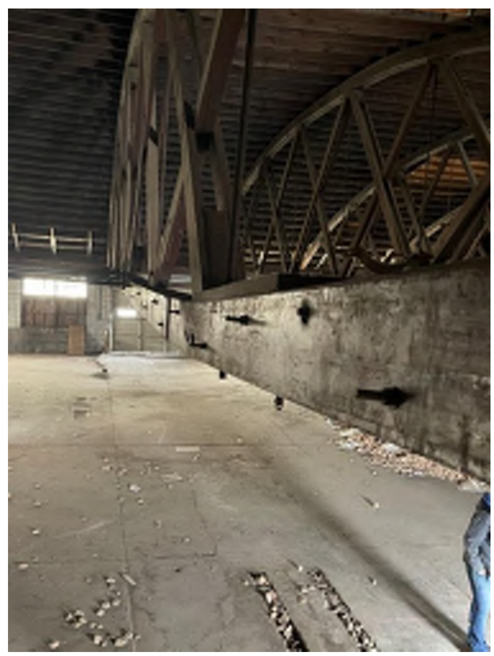

Photo #4 (Failed timber truss)

Photo #5 (Door through pilaster)

Photo #6 (West wall openings)

Photo #7 (South wall openings)

Photo #8 (Rafter to wall seismic ties)

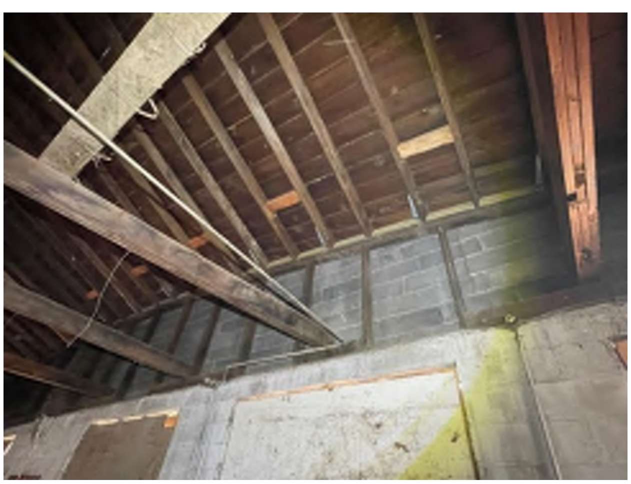

Photo #9 (Rafter to wall seismic ties)