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DRAFT 26 September 2017

Bourn Companies, LLC.
20 E. Congress, Suite 300
Tucson, AZ 85701

Attention: Matt Dickey

Reference: Indian Trading Post, Tucson

Dear Matt:

On Wednesday and Thursday, September 20 and 21, I visited the Indian Trading Post, originally the Rebeil Block, on Congress Street to observe the condition of the historic structural fabric of the building. I also met with Grenier Engineering, your Structural Engineer of Record (SER) for the project, and discussed the existing conditions and the required structural parameters of the design.

The following is a summary of my observations and my suggestions going forward. It should be stated for the record that Structures North is not the SER for this project, and any recommendations that might seem to be made in the report are actually, technically suggestions, for review and consideration by the project SER.

Building Description and Conditions

Original Construction and Historic Fabric-

The Rebeil Block was constructed in 1897/ 98 as a "high Victorian style" commercial building, and its subsequent history is well summarized in the "Historic Building Assessment Report", prepared by Vint and Associates Architects. There are two habitable floors in the building and a full basement.

The first and second floors are constructed with sawn Douglas fir lumber joists running in the east-west direction, between the east and west perimeter walls of the building and intermediately supported by a central, north-south running bearing line. The joists are 14" deep at the first and second floors and covered with a sawn plank subfloor. The central bearing line consists of multiple ply sawn lumber beams, fastened together somewhat sporadically with steel or wrought iron through bolts. The first floor beam is supported on brick piers, and the second floor beam is supported on solid wooden posts. The second floor joists are set into the exterior walls within individually cut pocket, with presumably fire-cut (tapered) ends. The first floor joists are pocketed into the stone foundation construction, and bear upon a continuous let-in ledger strip. In both cases, the breathability of the old lime mortar and brick materials (*please see below*), combined with the extremely dry desert climate appear to have prevented the end grain rot that can frequently occur in masonry-embedded wood in other parts of the country. The fact that the joists are composed of dense, likely old-growth Douglas fir also helps the situation.

The roof is constructed with board sheathed, east-west running wooden trusses that bear on the east and west walls. The basement floor is a cast-in-place concrete slab on grade.

The perimeter walls are constructed of multi-wythe unreinforced brick masonry bearing on stone foundations, forming an approximate rectangle in plan, with a 45-degree “clip” in the northwest corner at the main entrance. The north elevation and northeast corner were originally clad in ornamental cast iron as viewed from historic photographs. It is not known whether the cast iron was just an exterior treatment, or structurally supported the second floor over the window openings.

The exterior walls are 16” thick at the first floor, and then reduce to 12” thickness at the second. This originally transitioned to and ornate, corbelled parapet running around the top of the structure, however this no longer exists. There are 4” pilasters adorning the second floor exterior which corbel out of the wall just above the first floor, that once extended upward and helped support the corbelled parapet. The exposed building brick at both the interior and exterior are under-fired red clay units that are considerably taller than the standard brick units of today, however smaller than what would be considered a modern-day “jumbo” brick. These are set in a rather soft but well-bonded lime and sand mortar. There are regularly spaced header coursed weaving the wall together, and collar joints between wythes feel solid when tapped upon, revealing a relatively low strength but well-knitted composite mass.

The foundation walls step out toward the basement by 6”, resulting in a total thickness of at least 22”. Materially the walls are composed of pyroclastic stone units, chinked and semi-wet laid with the same lime and sand mortar as upstairs. These quarried stones were broken at relatively regular right angles, allowing in surprisingly consistent coursing for a rough-cut stone wall. The semi-wet laid nature of the construction is relatively common, and traditionally one of the primary purposes of having mortar at all was to water- and vermin-proof an otherwise dry-stacked assembly. Therefore, large voids exist within the inner portions of the wall, as viewed borescopically.

The lime mortar has almost completely degraded, and falls freely out of the wall face when even lightly disturbed, giving it the mortar little or no structural value. Along with the loose sifting mortar, some of the smaller chinking stones have also become loose. Even with the very poor to non-existent condition of the mortar and occasional loose chinkers, the foundation walls overall remain intact, as well as straight and true. Further, no evidence of major cracking nor sign of settlement can be seen in the wall structures above.

Subsequent Modifications-

Throughout much of the 20th century, the Rebeil Block underwent multiple renovations and changes in occupancy. This involved the following:

- Significant enlargement of the east wall’s window openings. This was done by brick removal and insertion of lintels spanning between the resulting brick piers that remained. The steel lintels appear to be functioning adequately, with no significant visible deflection or cracking above them.

- Removal of the cast iron at the north elevation and northeast corner and the insertion steel wide flange columns to support new lintels that almost entirely supported the brick at the second floor. The steel column and beam system appears to be functioning adequately, however the baseplates at the bottoms of the columns do not appear to be well founded and there are gaps below them.

- Cutting away a portion of the first floor to construct two-level vault structures of cast-in-place concrete and masonry, which run a little over half the way up through the first floor. The top surfaces of the vaults now support an enclosed mezzanine, accessed by a spiral stair. These are robustly constructed and it will be interesting to see whether they remain part of an eventual re-use plan.

- In a later modification, the large street-front openings were made smaller again, with brick infills and punched window openings.

- The original roof was replaced by a nailed together stick framed truss system that free-spans between the east and west walls, and supporting a wood framed ceiling system below it. The ceiling has recently been removed, exposing the trusses which are in an advanced state of failure.

Considerations for Adaptive Re-Use

Interior Floor Structures-

The first and second floor structures appear for the most part to be in good condition, however a multitude of joists have been deeply notched or cut. According to Grenier's analysis, the floor joists are sufficient to remain in service as long as the damaged joists are sistered with new ones to strengthen them. We concur with the assumption of "Select Structural" grade lumber used in their analysis.

According to Grenier's analysis the center beams running below the first and second floor joists are insufficient. We run into this frequently when analyzing archaic structures and I am not surprised. My personal theory is that floor structures used to be constructed more for moving concentrated loads than the uniformly distributed loads for which we design them today. Steel beams are slated to replace these members, which makes sense.

One suggestion if additional headroom is desired under the first floor would be to fill in the spaces between the new beam flanges with solid wooden blocking that is bolted through the web and providing a seat plate on the bottom that is wider than the lower flange. This way, a slot can be created by cutting the ends of the floor joists in the middle of floor and sliding the new beam into the slot. The joist end could be toe-nailed to the wood blocking and the bottoms of the joists could be seated on the lower plate, either directly or with blocking, depending upon the needed depths of the beams.

I understand that the large brick piers are to be replaced by steel columns at the basement, which makes sense form a space saving perspective. These can either be sat on new footings, or the existing footings can be utilized but with concrete caps that spread the concentrated loads from the new smaller columns out the same footprint of the piers in order not to increase bending moment on these unreinforced elements. With the reduction in load afforded by the removed brick piers, these footings should be more than sufficient to keep supporting the building structure above, based upon their long history of performance.

Between the first and second floors, the wooden posts are also slated to be replaced. These posts appear, visually, to be quite massive, and even though the wooden beams above them are apparently insufficient, it would be interesting to see if the posts structurally work. If so, one might consider keeping then and reinforcing the upper beams instead of pulling them out. Saving and featuring these elements architecturally could add to the historic authenticity of the first floor space.

For reasons discussed below (*please see "masonry walls"*), it would be advantageous to keep the joists bearing on the masonry walls, rather than supporting them on a supplemental beam system as currently proposed.

Roof Structure-

I concur that the roof structure is in a state of failure and should be replaced. The new roof should bear on the perimeter masonry walls and not on a supplemental interior frame. The reason for this is explained below.

Foundation Walls-

The multi course, multi-wythe stone foundation walls are generally plumb, planar and true, even with the total decomposition of mortar. If the bearing loads are not changed on the walls and mortar conditions do not worsen, they could be expected to keep performing in the same manner, however the question would be at what factor of safety. Further, these essentially dry-stacked walls have never yet needed to be analyzed for their seismic resistance.

Because the mortar that was present has failed, the shear strength, and to a limited extent the bearing strength, of the wall construction has been reduced from its originally as-built condition. Therefore, we suggest that the existing construction be strengthened where it is critically supporting load.

This can be done in-situ by injection grouting with a pozzolan-lime material that would entirely replace the decomposed mortar. As an independent venture, I have teamed up with several scientists and conservators to develop a system called VoidSpan (www.voidspan.com) that can do this in a relatively efficient and economical way.

The steps, as illustrated in the attached *Installation Guide*, would be as follow:

1. Evaluate stone foundation wall, re-set loose or missing stones. Several of the stones have become loose or have been removed to create places for piping and conduits.

2. Wedge and/or brace remaining wall face. The decomposed mortar, while not a primary load carrying element, helps hold some of the outer stones in place. Stones that would be loosened by its removal should be supplementarily braced added chinking stones or wedges.

3. Jet clean joints and internal cavities. This can be done by inserting 90 degree jet cleaning wands into the open wall joints and jetting the friable mortar out. Additional flushing can be done via drilled holes through some of the inner stones, if the wands can't reach all the way to the back of the wall.

4. Point exposed wall faces, remove wedges. Pointing mortar should be lime and cement or pozzolan lime based and should visibly match the original material. The joint faces should be recessed.

5. Insert sealed plastic grouting tubes or leave open ports. These should be located at deep gaps or drilled holes in order to provide pathways for the grout to travel as far as possible into the backs of the walls.

6. Grout wall starting from bottom. This should be done in a low-lift, low-pressure manner in order to minimize the chance of blow-out in the wall.

7. Continue to grout fill wall. Grouting should proceed upward to the bottom of the brickwork.

8. Remove grouting tubes. They should be clamped after they are filled and then removed after the grout's initial set.
9. Patch holes in pointing. Patching should be done to blend with the surrounding joint appearance.

The end result would be the well-bonded equivalent of a wet-laid stone structure. Ultimate 120-day VoidSpan PHLc grout material strengths would be as follow:

Ultimate Compression = 1877 psi
Ultimate Tensile Bond = 48 psi
Ultimate Lap Shear = 156 psi

We recommend a factor of safety of four, and an assumed bonding efficiency of 75%, to account for potential gaps in the grouting process. The resulting low-end material strength estimate, accounting only for grout and negating the stone-on-stone bearing that is holding the wall in place now, we would arrive at the following allowable stresses:

Working Compression Stress = 352 psi
Working Tensile Bond Stress = 9 psi
Working Lap Shear Stress = 29 psi

When applied to the 22" thickness of the wall this would result in the following allowable resistances per running foot:

Compression Resistance = 93,000 plf (not adjusted for slenderness)

Out-of-plane Bending Resistance = 726 foot-lbs per foot (based on tension at basement face of wall, not including arching action)

In-plane Shear Resistance = 7,656 plf

We suggest that the foundations be analyzed based upon the above stabilization assumptions in order to avoid using the presently recommended interior shotcrete wall face and supplementary basement columns. If columns are necessary for braced frames and/or moment frames due to the above-grade wall openings, we suggest these be landed at the first floor where the foundation wall steps out by 6". The column flanges can be nested into slots cut in the brick wall construction with their webs tight against the inner wall faces. Slots can be cut into the upper portion of the foundation to accept cast-in-place concrete bearing blocks and drag struts that can dissipate the column and frame loads as line loads into the foundations.

Where there are significant uplift loads on the columns, the webs can be bolted into the brick walls into which they are nested using adhesive anchors or sock anchors. If there is not sufficient overburden from the brick construction, Cintec sock anchors (www.cintec.com) can be installed as tie-downs into the foundation.

Brick Bearing Walls-

As noted above, the brick walls are of generally sound, well bonded condition although of materially low strength, with few signs of damage or even weathering. Other than for seismic loading, I see no reason why they could not continue to support floor and roof loads as they have successfully for more than a century.

The added vertical stress created by the floor and roof dead loads actually helps increase the lateral stability of the walls through floor-to-floor arching action, as well as the shear capacities by friction. We suggest that the supplemental column system be eliminated except where and it needed for seismic loading.

With the removal of the second floor ceiling, there are lines of disused tension ties at the east and west walls. These walls should be checked, laterally, for their unbraced lengths and wind loading without the effective tie at the ceiling, and additionally braced if necessary. There are no signs of there having been ceiling ties at the north or south walls.

There are a few discrete areas where brickwork has been cracked or weakened due to improper modifications, and these should be addressed by localized re-setting and/or pinning and injection grouting. Mortar and grout should be of moderate to low strength, and should be compatible with the existing masonry. A 1:3:12 (cement : lime : sand) or a pozzolan lime and sand mortar meeting *ASTM C1713: Standard Specification for Mortars for the Repair of Historic Masonry* would be appropriate (I happen to be chair of the C1713 committee).

Where new enlarged openings are shown on the east side of the building, I suggest that these be limited to the spans of the existing lintels that run just below the second floor level, with the openings being created by simply removing the infills. This will avoid expensively needling and suspending the upper portion of the building while longer lintels are inserted. This will also maintain the former pier widths between the openings, and avoid overstressing the masonry.

If there continues to be concern over the in-situ compressive strength of the brick wall assemblies, this can be determined by carefully cutting out cubic “prism” samples from the wall with a deep concrete saw or wet masonry cutting chainsaw. These can then be compression tested in accordance with *ASTM C1314: Standard Test Method for Compressive Strength of Masonry Prisms*. Using the test results, the wall can then be analyzed in accordance with Brick Institute guidelines for unreinforced brick masonry.

<u>*Seismic Considerations*</u>-

Assuming the foundation is stabilized and the floor plates repaired, the remaining “weak link” in the lateral seismic resistance of the structure might be the brick masonry above the ground floor, along with the diaphragm connections. The walls would need to be analyzed as unreinforced masonry, which is assigned specific factors in the International Existing Building Code that increases the seismic design force to account for the brittle, non-ductile nature of the construction.

The existing brickwork can be tested in place in accordance with ASTM C1531: Standard Test Methods for In Situ Measurement of Masonry Mortar Joint Shear Strength Index. This test will determine the shear capacity of the masonry, which combined with the compressive strength, geometry and overburden can be used to determine the sufficiency of the construction.

If the walls are determined to be insufficient, structural steel braced frames and/or moment frames can be added and partially inserted into the interior masonry, with suggested locations as shown on the accompanying plan. These will need to be anchored to the foundation, as previously described, and tied into the floor and roof diaphragms.

Floor and roof boundaries will need to be tied to the perimeter walls, both laterally and in plane. Lateral ties to the brick can be achieved by adhesive anchored rods and/or rods and external plates. In-plane shear can be achieved by engaging embedded joist ends in side-bearing by adding lines of blocking just in from the wall faces and nailing it from the floor deck. Where joists are parallel to the walls, new rim joists can be added against the brickwork and tied to it with adhesive anchors.

At the first floor level the joists rest upon an embedded wooden ledger that reduces the shear strengths of the walls. These should be removed and replaced with small wooden bearing blocks under the joists and the remaining slot filled with mortared stone chinking.

Thank you for the opportunity to look at this interesting and historically important structure. Please contact me if you have any questions or we can be of further assistance.

Respectfully Yours,

John M. Wathne, PE (MA), President
Structures North Consulting Engineers, Inc.