City of Lake City
City Hall Structural Assessment

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September 6, 2018

PRESENTED TO

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1.0 EXECUTIVE SUMMARY

Tetra Tech was retained by The City of Lake City to perform a detailed structural assessment of the City Hall building for lateral stability and strength with respect to the interior brick walls that were previously removed. This assessment report provides the City with the information it needs to evaluate whether they should have the building structurally retrofitted if they choose to repair and restore the exterior brick façade.

On August 1, 2018 Tetra Tech performed a visual inspection using non-destructive methods to obtain additional information that was not included in the scope of the first study of the brick façade.

Data was collected during the site visit by various methods including, but not limited to: measurements, photos, visual observation, and conversation with occupants. The exterior walls of the City Hall building are the original load bearing, multiwythe brick. The floors and roof are framed with wood joists and rafters but have undergone modification in several areas. Originally, there were interior load bearing brick walls. They have since been replaced by steel columns and girders. The member sizes and connection detailing for this steel framing appears to be just for gravity loads and does not indicate that they were designed as moment frames to resist lateral loads. So, there is no indication that the new steel framing is meant to replace the lateral stability and strength of the original brick walls. Furthermore, the new steel girders are not connected to the east and west brick walls, so no bracing support is provided by them.

The removal of the interior walls requires the wood floors to act as large diaphragms that transfer the lateral wind loads to the exterior walls. The problem is that the existing floors and roof decks are not detailed and constructed in a manner that is structurally adequate to meet this demand. Older buildings like this do not typically have significant lateral load path connections from the floor deck and framing to the perimeter of the building. It was confirmed onsite that there is not an identifiable load path for in and out-of- plane loads from the east and west walls to the floor diaphragm. The timber joists bearing on the north and south walls do offer some bracing due to the friction developed by the mortar and timber interface in the wall pocket. We also discovered some tie-back rods/anchors were installed to help stabilize the west wall, but it is unknown when these retrofit anchors were installed and what strength capacity they have.

One wall construction detail that could not be confirmed was how the three wythes of brick were tied together to act compositely. It is common for individual brick or rows of bricks to be turned perpendicular to the length of wall to act as headers that bridge and tie the multiple wythes together. This was not evident in the exterior brick pattern of the City Hall. It is assumed that the wythes are likely tied together by internal metal ties. The existence and condition of these ties needs to be confirmed through demolition if brick restoration occurs.

Wind loads used for analysis were based on the 2017 Florida Building Code (FBC) and ASCE 7-10. Specific criteria used were: Ultimate Wind Speed = 120 mph, Exposure B, Risk Category II, and Enclosed Building. Although wind loads were the driving force for lateral analysis, other gravity-based loads like material dead weight and live loads were also considered when applying load combinations.

Presently, the floor and roof diaphragms are irregular shaped with a re-entrant corner on the east side of the building and a large floor opening for a light-well just west of the re-entrant corner. Ties and collectors should have been designed during the 80's renovation to redistribute the diaphragm forces but were not. The diaphragms were analyzed to see what forces should be distributed to the shear walls around the building and the connection forces required to accomplish it. From this analysis it is certain that any future renovation should include structural retrofit of the diaphragm. The retrofit would include a shear collector running along grid 4.4 from grid A to K. Another shear collector is required along grid K, between grids 2 and 4. The shear collectors need to be specifically detailed to collect diaphragm shear forces along their length and transfer the sum of the forces to the shear wall via a specially detailed connection.

Diaphragm to wall anchors are also needed to transfer in-plane and out-of-plane loads. Therefore, a uniform arrangement of wall anchors along all sides of the building and all diaphragm levels is recommended. Other required improvements to the diaphragm strength include transfer straps and members around the large light-well opening to ensure the diaphragm stresses are properly transferred around the opening. New fasteners in the existing floor deck or another means of strengthening should also be installed.

The brick walls were evaluated for in-plane shear stresses around openings, through narrow piers, and the gross wall sections. In addition to shear, compressive and tensile stresses were also evaluated for the bending effect induced by lateral loads. For all the in-plane wind load cases, the walls were found to be compliant with the code required stress levels. ACI 530 allowable stresses for this building are: 37 psi for shear, 500 psi for compression, and 32 psi for tension. One major assumption used in our analysis is that the walls will be restored with all loose brick and cracks repaired appropriately.

Out-of-plane wall stresses were checked based on two conditions. The first assumption was that the individual wythes were not acting compositely since no header blocks were verified during the site visit. This resulted in tension stresses being significantly over allowable limits for all the walls. A second analysis was performed for out-of-plane wind forces assuming full composite action of all brick wythes. In this case, all walls were found to be in compliance with code required stresses.

Code compliance of the brick walls for out-of-plane wind loads is based on multi-wythe composite behavior assumptions and the masonry being in a structurally sound condition. Some of the highest stresses were found near openings where loose and cracked masonry exists. Therefore, the conclusion is that brick restoration and verification of masonry ties between the multi-wythes must occur for the brick walls to be deemed code compliant. If brick ties are not found within the brick walls to justify composite behavior, retrofit type helical tie anchors would need to be installed to correct this issue.

Based on our visual observations and structural analysis, Tetra Tech's opinion is that there are multiple issues with the integrity of the brick walls and lateral strength. The main concerns for the brick walls and building are their lateral stability without the three original interior brick walls, adequate diaphragm strength, perimeter diaphragm connections, diaphragm collectors, brick wythe ties, and brick deterioration above and around the windows. The building appears to be stable for dead and live loads associated with its current occupancy. But, the lateral stability and strength of the building in its current configuration are not adequate to withstand current wind design requirements.

The exposed brick façade on the west and south face need to be repaired to prevent further deterioration and instabilities for the building. This includes repairing cracks, sealing around the windows, and replacement or repair of cracked sills. The mortar joints need to be cleaned, repaired, and tuck pointed. Steel lintels should also be installed above the window openings at the second and third floors. Steel lintels will stabilize the sagging and cracking brick.

Adding diaphragm collectors, diaphragm to wall connections, and strengthening the diaphragm will require extensive demolition of the existing interior floor and ceiling finishes. With this level of work, also comes complications with construction scheduling if the building needs to remain occupied. Our assumption for pricing is that the building would not be occupied, and the contractor would be free to work on the whole building at once. All these factors drive the cost of the project and should be carefully considered.

The rough order of magnitude cost for restoring the brick façade and structurally retrofitting the walls and diaphragms for wind loads would be a minimum of $1,400,000, and as much as $3,000,000. The cost range is influenced by unknown factors like final design details, brick and mortar conditions behind the outer wythe, and if brick ties exist within the wall. The brick conditions could be better understood through select demolition if the City wants to pursue these repairs. Costs stated in this report reflect specifically listed unknowns, and so should any decision made regarding whether or not to proceed with a brick restoration effort.

2.0 INTRODUCTION

2.1 SCOPE OF WORK

The City of Lake City (the "City") retained Tetra Tech to perform a detailed structural analysis of the City Hall building located at 205 N. Marion Ave., Lake City, FL. The integrity of some of the brick façade is in question and was evaluated in the previous report dated: July 2, 2018. This study takes the next step to evaluate the building's resistance to lateral wind loads as it relates to the interior brick walls that were removed in the 1985 renovation. The City will then be able to determine whether they should have the building structurally retrofitted if they choose to repair and restore the exterior brick façade.

Tetra Tech's scope of work included: make one site visit with a Structural Engineering team to obtain specific details for the existing conditions of the lateral load resisting system, identify existing and missing load paths in the structural systems, analyze the lateral load resisting system for current building code, provide recommendations for repair with associated cost, and deliver an assessment report to summarize the findings. This assessment report will provide the City with the information it needs to evaluate the future use of the building.

2.2 BACKGROUND

The City purchased the building around 2005 and currently occupies all three floors. As-built drawings were not available to review for original construction details and building age. It is a three-story building, approximately 23,000 ft^2 with load-bearing multi-wythe brick walls, timber floors, and timber roof framing. An engraved stone on the building façade and historical pictures in the building indicate it was built circa 1911 (See ***Photo 1***).

The building was originally constructed as a bank and hotel but has undergone many different renovations and uses throughout its lifespan as noticed in ***Photo 2*** and ***Photo 3***. Most of the changes to the brick façade have been made along the first floor. The original arch shaped openings have since been restored, but the current condition varies from the original in that there is a recessed corridor and entry along the southwest corner of the building. Record drawings were provided for a major renovation and addition project that was performed around 1985. As part of that renovation three interior, load-bearing, multiwythe brick walls were removed and replaced with steel columns and beams. This is the focus of this study and report. The 1985 renovation also included a three-story reinforced cmu addition on east side of the building for a stairwell and elevator (See ***Figure 1***).

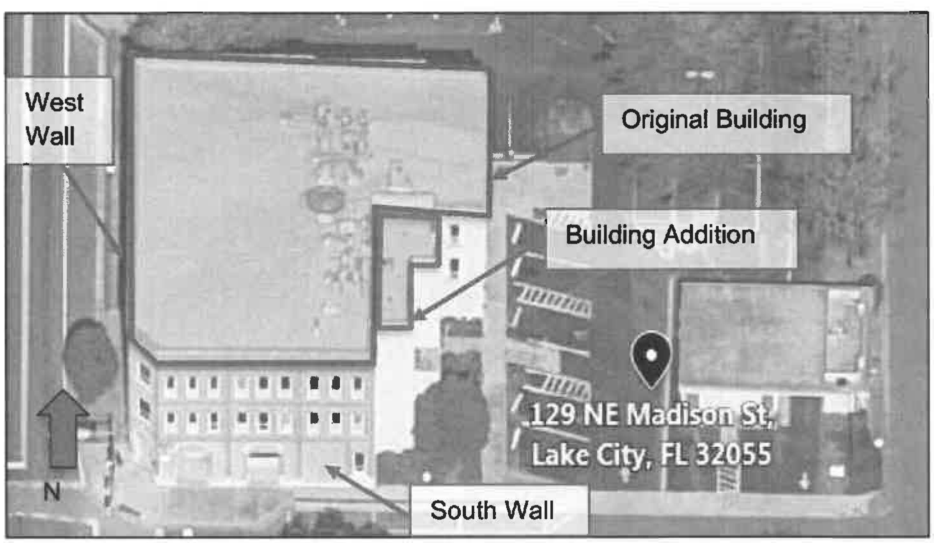

Figure 1: Building Aerial View (Looking North)

3.0 SITE INVESTIGATION AND FINDINGS

3.1 GENERAL

Tetra Tech visited the City Hall building on August 1, 2018 to collect additional structural information that was not part of the focus of the first study. The following personnel were in attendance from Tetra Tech: Jason Burkett, PE, SE and Justin Greenwell, PE. Steve Roberts from the City provided access throughout the building. The City's primary concern was to assess the lateral stability and strength of the building to determine if structural repairs need to be made if the City opts to move forward with brick restoration.

Data collected during the site visit was primarily obtained by visual observation of exposed surfaces and ceiling spaces. Comparing the available record drawings with the field investigation data, it is Tetra Tech's opinion that the building is generally constructed as indicated on the available as-built drawings and in accordance with typical methods for multiwythe construction from that era.

3.2 BRICK WALLS AND FOUNDATIONS

The exterior walls of the City Hall building are load bearing, multiwythe brick. The walls are nominally 13" thick and comprised of 3 wythes of clay brick. The west and south walls also have pilasters that add an additional 4" of thickness between every third window. This pattern creates 4 typical bays on the west side and 3 typical bays on the south wall, as shown in ***Photo 3***. The north and east walls are thought to be uniform in thickness, without pilasters, because they were separation walls for previously attached buildings and they are now covered in stucco.

When the building was originally constructed, there were three interior load bearing multiwythe brick walls that aligned with the pilasters on the west face of the building. However, the 1985 renovation removed these load bearing walls and replaced them with five new steel girder lines, see ***Figure 2***. The member sizes and connection detailing for this steel framing appears to be just for gravity loads and does not indicate that they were designed as moment frames to resist lateral loads. So, there is no indication that the new steel framing is meant to replace the lateral stability and strength of the original brick walls. Furthermore, the new steel girders are not connected to the east and west brick walls, so no bracing support is provided by them.

Older buildings like this do not typically have significant lateral load path connections from the floor deck and framing to the perimeter of the building. It was confirmed onsite that there is not an identifiable load path for in and out-of-plane loads from the east and west walls to the floor diaphragm. The timber joists bearing on the north and south walls do offer some bracing due to the friction developed by the mortar and timber interface in the wall pocket. We also discovered some tie-back rods/anchors were installed to help stabilize the west wall, but it is unknown when these retrofit anchors were installed. ***Photo 4*** shows a typical anchor and ***Photo 5*** shows an example where an anchor does not support the wall as intended due to missing blocking. There are 20 of these wall anchors installed on the west wall of the building, as shown in ***Photo 6***.

The brick walls on the south and west faces of the building were the primary focus of previous assessment due to their visible issues. Missing and loose bricks were discovered on the second and third floors by the City and caused concern for the safety of the building occupants and pedestrian traffic adjacent to the building. Details and recommendations regarding the current condition of the brick façade can be found in the report dated: July 2, 2018.

One wall construction detail that could not be confirmed was how the three wythes of brick were tied together to act compositely. It is common for individual brick or rows of bricks to be turned perpendicular to the length of wall to act as headers that bridge and tie the multiple wythes together. This was not evident in the exterior brick pattern of the City Hall. It is assumed that the wythes are likely tied together by internal metal ties. The existence and condition of these ties needs to be confirmed through demolition if brick restoration occurs.

Foundation construction is not known with certainty due to lack of record drawings and not being exposed above grade or by a basement. It is anticipated that the building foundation is comprised of either stone or concrete below grade. In a few places along the west side of the building façade, the multiwythe brick was sitting on what appeared to be a concrete stemwall. It is hard to say for sure since only a small sample was visible and it could have been from later repair or restoration work. Either way, the brick near grade level did not show any signs of cracking or differential settlement in the foundations. Our observation is that the foundations appear stable and are assumed adequate to transfer the loads considered in this structural assessment.

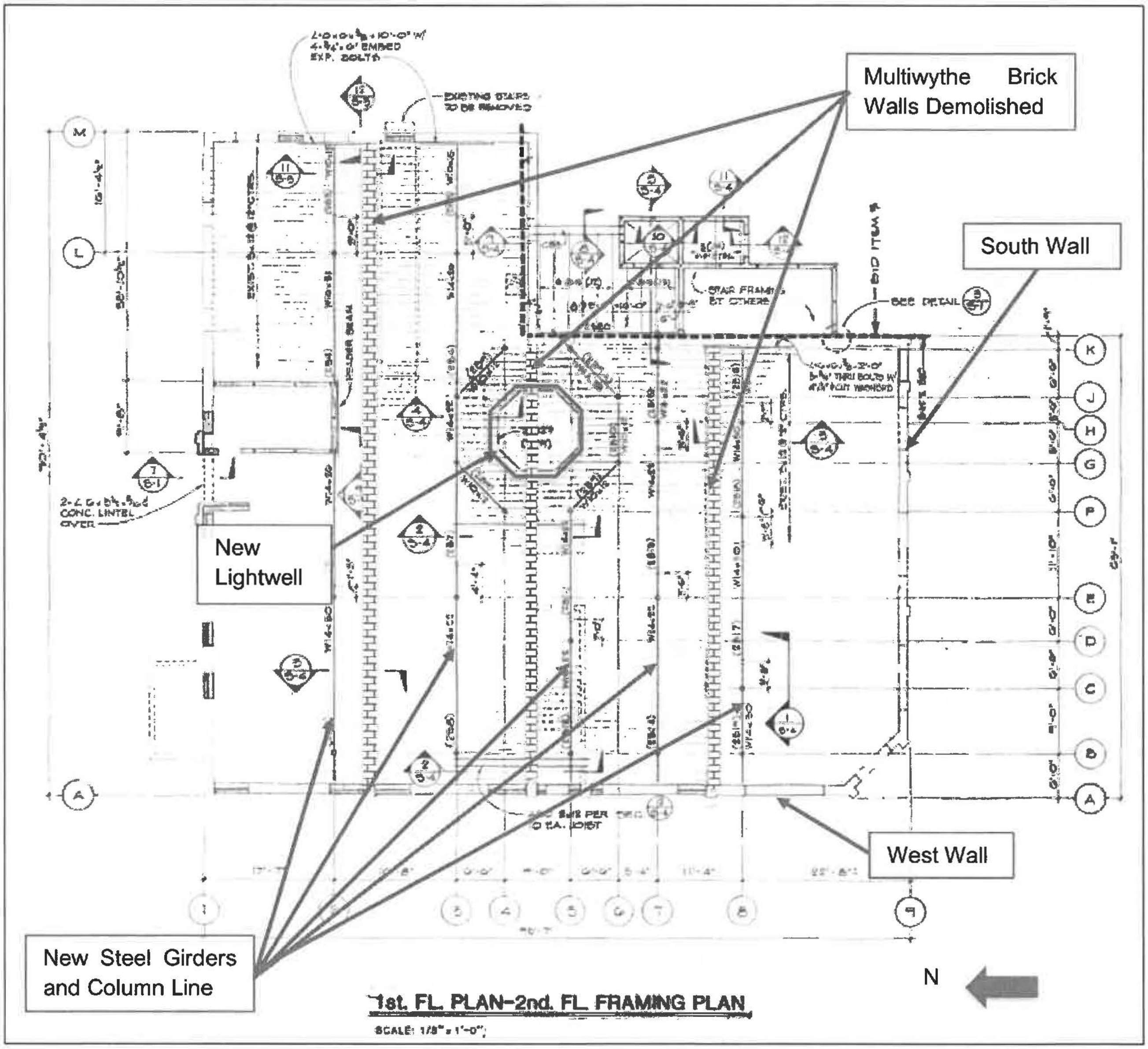

Figure 2: 1985 Renovation - Demolition Plan

3.3 FLOORS

As mentioned in the previous section, the second and third floor framing is constructed of timber. Floor joists are typically 3x12 or 2x12, spanning in the north-south direction, and spaced at 12 inches on center. Joists were originally designed as simple/single spans between the exterior and interior multiwythe brick walls which provided a 22 to 27ft clear span for the various bays. The floor decking consists of the tongue and groove boards approximately 1 inch thick and nailed to the supporting framing members. The tongue and groove boards nailed to the rafters act as a flexible diaphragm that can distribute lateral wind loads to the walls.

The 1985 renovations made significant changes to the support of the timber framed floors that are detailed in the previous report. Some of the joists now have single or double cantilever extensions, but this has not been identified as a structural concern and was not part of this lateral analysis. The main changes that affect the transfer of lateral loads are the new octagonal light well and missing support from interior bearing walls. The effect of these changes is discussed in further detail in the analysis section.

3.4 ROOF

Like the floors, roof framing consists of timber framing but of various types and span directions. There are a combination of beams, trusses, rafters, and knee walls. There was a considerable amount of rework in the 1985 renovation. Like the floor framing, the gravity load bearing members are not a structural concern and are not part of this study. It was noted that hurricane clips had been installed on the roof rafters at some point, but the timeframe is unknown. The hurricane clips on the roof rafter primarily serve to resist wind uplift forces so they were not considered in this study. Roof decking consists of plywood nailed down to the timber framing. Overall, the roof diaphragm was assumed to behave similar to the floors for load distribution purposes.

4.0 STRUCTURAL ANALYSIS AND RESULTS

4.1 LOADS

Since the primary focus of this study was lateral strength and stability, wind loads were the governing force used to analyze the building. Seismic analysis has not historically been required in Florida due to the low probability and risk. Wind loads used for analysis were based on the 2017 Florida Building Code (FBC) and ASCE 7-10. Specific criteria used were: Ultimate Wind Speed = 120 mph, Exposure B, Risk Category II, and Enclosed Building. Although wind loads were the driving force for lateral analysis, other gravity-based loads like material dead weight and live loads were also considered when applying load combinations.

Main Wind Force Resisting System (MWFRS) pressures were used for calculating forces on shear walls and diaphragms since they provide stability and support for the overall building. These wind forces are generated by wind acting on the surface of the building and then being transferred by an assemblage of components like walls, diaphragms, framing members, and connections. MWFRS pressures were applied as positive pressures on the windward side of the building and negative to the leeward and evaluated from two primary directions of East-West and North-South.

Component and Cladding pressures were used for calculating wind loads when evaluating the out-of-plane strength and stability of the walls since the wind acts directly on the component being evaluated. Component and cladding pressures act both positively and negatively(suction) against the wall surface, with suction forces usually being greater.

4.2 FLOOR AND ROOF DIAPHRAGMS

Removal of interior brick walls during the 1985 renovation significantly changed how the diaphragms send load to the shear walls. Previously, the overall diaphragm could be generally described as an assembly of sub-diaphragms bounded by lines of resistance along all sides. Presently, the diaphragm is irregular with a re-entrant corner to the east and a large floor opening west of the reentrant corner. Ties and collectors should have been designed during the 80's renovation to redistribute the diaphragm forces but were not.

The floor and roof decking were assumed to act as flexible diaphragms that transfer laterally applied wind loads based on tributary area and not stiffness. Usually, flexible diaphragms only transfer wind loads to walls that are parallel to the direction of the applied wind load. For wind loads in the E-W direction the lines of lateral resistance are exterior brick walls along grids 1,4.4, and 9, as shown in ***Figure 2***. Wind loads in the N-S direction are transferred to walls along grids A, K, and M.

To calculate diaphragm forces, the overall floor and roof diaphragms were broken down into two simple sub diaphragms. Namely, one between grids K and M, and one large one between K and A. For this analysis they were assumed to act independently and flexible, which affected the amount of tributary load assigned to each line of resistance. There are no interior walls to contribute to lateral resistance.

Due to the uncertainties in diaphragm fastening patterns and lack of ties and collectors within the diaphragms, internal stress analysis was not performed on the diaphragms. The diaphragms were analyzed to see what forces should be distributed to the shear walls around the building and the connection forces required to accomplish it. From this analysis it is certain that any future renovation should include retrofit of the diaphragm. The retrofit would include a shear collector running along grid 4.4 from grid A to K. Another shear collector is required along grid K, between grids 2 and 4. The shear collectors need to be specifically detailed to collect diaphragm shear forces along their length and transfer the sum of the forces to the shear wall via a specially detailed connection.

Diaphragm to wall anchors are also needed to transfer in-plane and out-of-plane loads. There are some locations that already have some mechanical anchors installed. But, the east wall appears to be the most susceptible to out-of-plane wind load due to unknown connection to the floor and roof diaphragm. While the existing joist bearing pockets on the north and south walls can transfer some force, it is preferred to have a mechanical connection that is more reliable and able to be accurately quantified. Therefore, a uniform arrangement of wall anchors along all sides of the building and all diaphragm levels is recommended.

Other required improvements to the diaphragm strength include transfer straps and members around the large light-well opening to ensure the diaphragm stresses are properly transferred around the opening. Lastly, new fasteners in the existing floor deck or another means of strengthening should also be installed. The current diaphragm fastening patterns were not able to be observed, but they are likely nails which tend to work themselves loose over many years like this building has existed. The diaphragm shear strength values of wood planks are also relatively small, so depending on the arrangement and details of other retrofits different means of diaphragm strengthening may need to be explored.

4.3 BRICK WALLS

With the diaphragm load transfer assumptions discussed previously, the in-plane shear loads were then applied to each exterior shear wall for the respective wind load cases. The brick walls were evaluated for in-plane shear stresses around openings, through narrow piers, and the gross wall sections. In addition to shear, compressive and tensile stresses were also evaluated for the bending effect induced by lateral loads. For all the in-plane wind load cases, the walls were found to be compliant with the code required stress levels of ACI 530. Allowable stresses used in the analysis were: 37 psi for shear, 500 psi for compression, and 32 psi for tension. One major assumption used in our analysis is that the walls will be restored with all loose brick and cracks repaired appropriately.

Out-of-plane wall stresses were checked based on two conditions. The first assumption was that the individual wythes were not acting compositely since no header blocks were verified during the site visit. For this case the bending moment was divided by the number of wythes (3) and bending stresses were calculated based on the width of one brick. This resulted in tension stresses being significantly over allowable limits for all the walls. Even the axial compression from floor dead and live loads were not enough to bring the stresses back into code compliance. The calculations assumed continuous vertical span conditions. Flexural stresses governed, while axial compressive stresses were low and a non-issue. A second analysis was performed for out-of-plane wind forces assuming full composite action of all brick wythes. In this case, all walls were found to be in compliance with code required stresses.

Code compliance of brick walls for out-of-plane wind loads is based on multi-wythe composite behavior assumptions and the masonry being in a structurally sound condition. Some of the highest stresses were found near openings where loose and cracked masonry exists. Therefore, the conclusion is that brick restoration and verification of masonry ties between the multi-wythes must occur for the brick walls to be deemed code compliant. If brick ties are not found within the brick walls to justify composite behavior, retrofit type helical tie anchors would need to be installed to correct this issue.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Based on our visual observations and structural analysis, Tetra Tech's opinion is that there are multiple issues with the integrity of the brick walls and lateral strength. The main concerns for the brick walls and building are their lateral stability without the three original interior brick walls, adequate diaphragm strength, perimeter diaphragm connections, diaphragm collectors, brick wythe ties, and brick deterioration above and around the windows. The building appears to be stable for dead and live loads associated with its current occupancy. But, the lateral stability and strength of the building in its current configuration are not adequate to withstand current wind design requirements.

The exposed brick façade on the west and south face need to be repaired to prevent further deterioration and instabilities for the building. This includes repairing cracks, sealing around the windows, and replacement or repair of cracked sills. The mortar joints need to be cleaned, repaired, and tuck pointed. There is a lot of missing, cracked, and deteriorated mortar that provides opportunity for water intrusion into the wall and building. Steel lintels should also be installed above the window openings at the second and third floors. Steel lintels will stabilize the sagging and cracking brick. Unfortunately, repairing the brick and installing the lintels is a tedious process that is hard to quantify before the work begins due to the complexity and unknowns that may be encountered. Costs stated in the next section will reflect this uncertainty, and so should any decision made regarding a brick restoration effort.

Adding diaphragm collectors, diaphragm to wall connections, and strengthening the diaphragm will require extensive demolition of the existing interior floor and ceiling finishes. Until proven non-existent, the effort and cost of adding brick ties between the multi-wythes will not be considered due to the likelihood of them being in place. Another cost consideration it that the retrofitting work would likely trigger other miscellaneous interior renovations. With this level of work, also comes complications with construction scheduling if the building needs to remain occupied. Our assumption for pricing is that the building would not be occupied and the contractor would be free to work on the whole building at once. All these factors drive the cost of the project and should be carefully considered.

5.2 OPINION OF PROBABLE COST

The Rough Order of Magnitude (ROM) costs associated with repairing and restoring the brick façade and making structural retrofit repairs at City Hall, including design fees and contingency, is described in the following and summarized in ***Figure 3***. This represents a Class 4 cost estimate based on a feasibility study, which has an expected accuracy range of from -30% to +50%. Assumptions for brick repair and restoration take into consideration partial removal of existing and installation of new components. Structural retrofits are assumed to occur in one phase with the building being vacated for the duration of construction. The costs are rounded up to the nearest $1,000 and are based on experience and R.S. Means Cost Estimating Manuals.

The rough order of magnitude cost for restoring the brick façade and structurally retrofitting the walls and diaphragms for wind loads would be a minimum of $1,400,000, and as much as $3,000,000. The cost range is influenced by unknown factors like final design details, brick and mortar conditions behind the outer wythe, and if brick ties exist within the wall. The brick conditions could be better understood through select demolition if the City wants to pursue these repairs.

CITY HALL BRICK FAÇADE AND STRUCTURAL RESTORATION	
Brick Restoration	
Window Sealant and Sill Replacement	$50,000
Waterproofing Spray	$60,000
Brick Repair and Mortar Tuck Pointing	$100,000
Brick Resetting and Window Lintels	$185,000
Wall Anchor Repairs	$20,000
Subtotal	$415,000
30% Contingency	$125,000
Design Fees	$42,000
Total	$582,000
Structural Retrofitting	
Diaphragm Strengthening	$75,000
Diaphragm Perimeter Connections	$525,000
Diaphragm Collectors	$150,000
Flooring Demo and Replacement	$88,000
Ceiling Demo and Replacement	$42,000
Roofing Demo and Replacement	$96,000
Subtotal	$976,000
30% Contingency	$293,000
Design Fees	$196,000
Total	$1,465,000
Retrofit Total	$1,391,000
Contingency Total	$418,000
Design Fees Total	$238,000
TOTAL CONSTRUCTION COST	**$2,047,000**

Figure 3: Rough Order of Magnitude Cost Summary

6.0 DISCLAIMER

The observations, recommendations and conclusions offered in this report are based on limited visual observation made during the site visit and record drawings made available. The general assumption is that structural members and connections that were not observed, were constructed in a typical manner throughout the building. Nothing in this report should be construed as a warranty for how the building was constructed or the future performance of this building, and none is offered. Proper interpretation of this report is the responsibility of the persons who authorized this report. No inferences other than those included herein should be made without first contacting Tetra Tech for written concurrence.

APPENDIX: PHOTOS

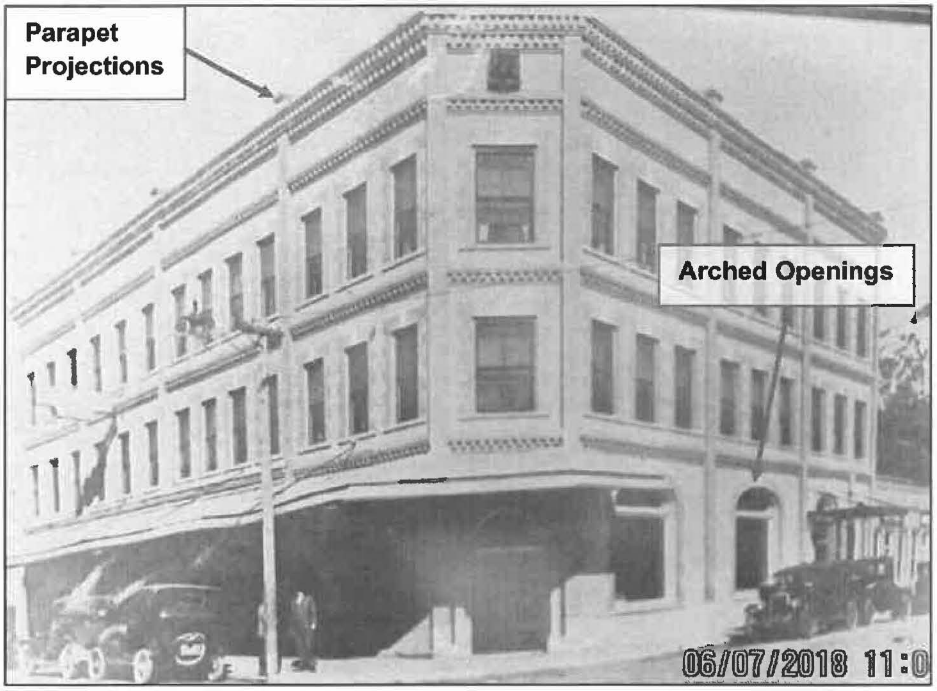

Photo 1: Original Building

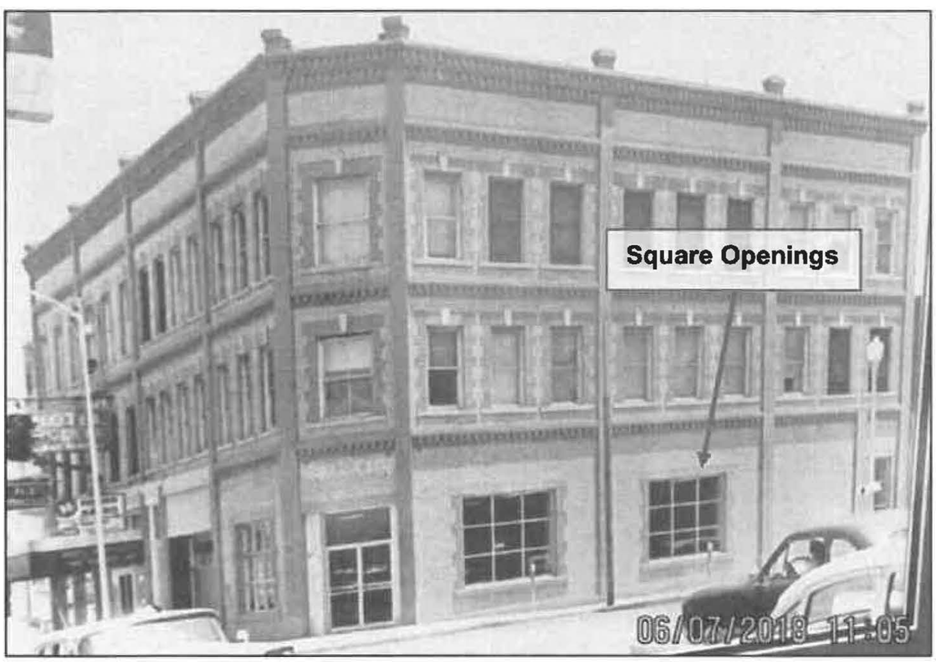

Photo 2: Building with Modifications

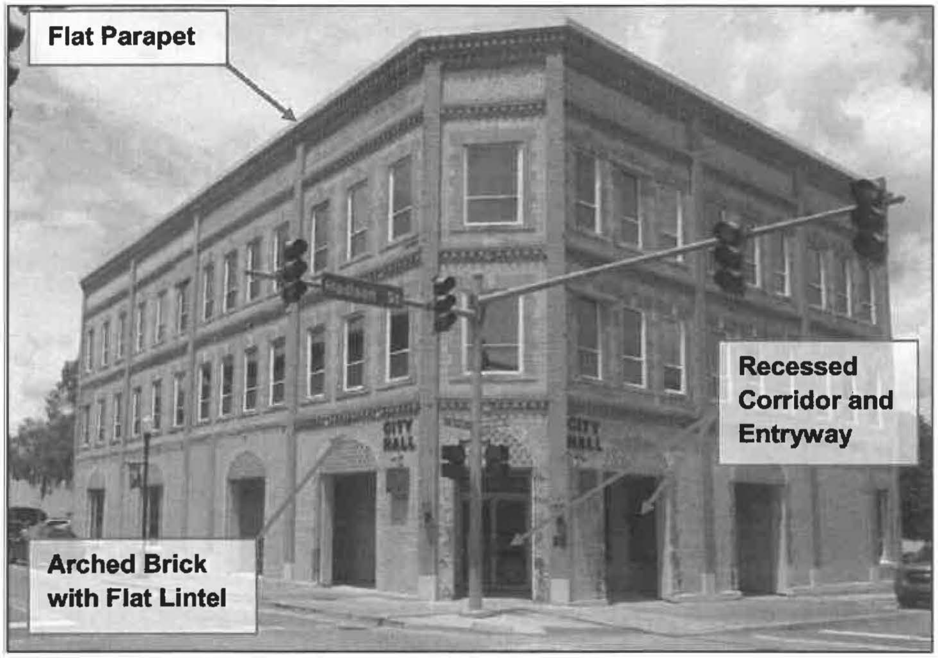

Photo 3: Building - Present Day

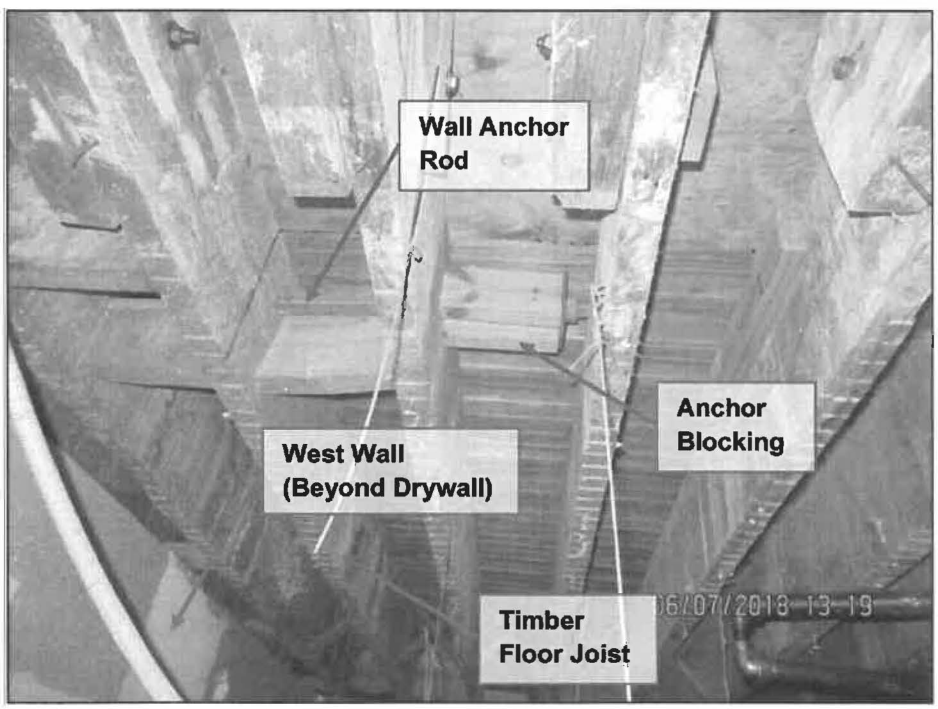

Photo 4: Retrofit Wall Anchor (West Wall)

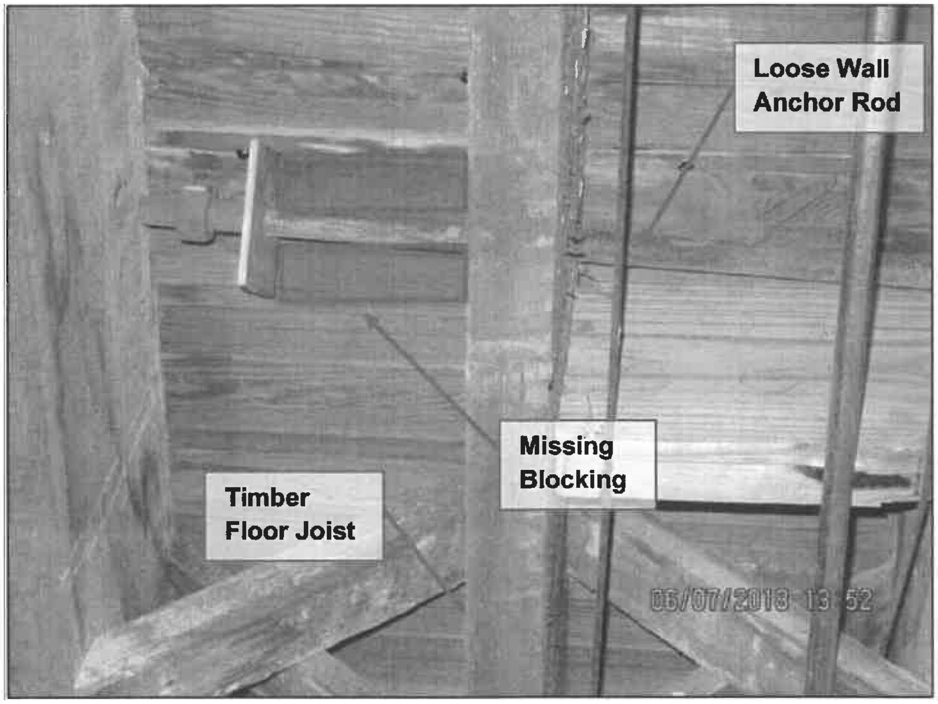

Photo 5: Loose Wall Anchor

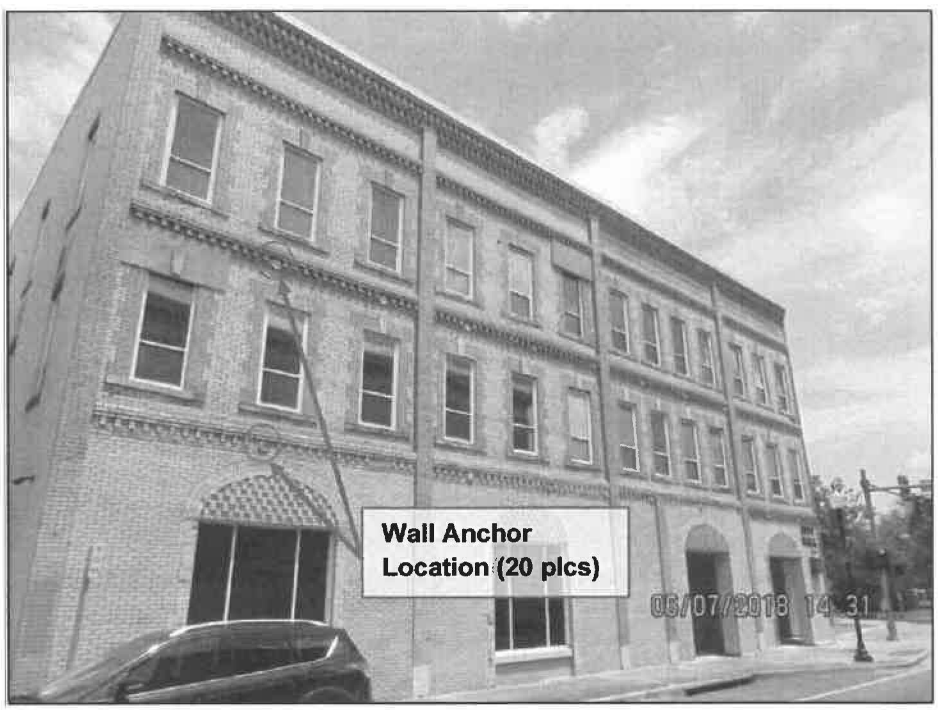

Photo 6: West Facade Anchor Locations