SOMERSET HOUSE REHABILITATION

352 Somerset Street West, Ottawa, Ontario, K2P 0J9

2023 DESIGN INTENT REPORT

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Document Revision Index

Revision	Date	Description of Changes
00	June 23, 2023	Initial version

Table of Contents

Executive Summary

The purpose of the report is to outline ART Engineering Inc.'s (AEI's) intended design approach and methodology during the upcoming permitting process, specifically from a seismic design perspective, to ensure the structural rehabilitation of Somerset House is in full compliance with the requirements of 2012 Ontario Building Code (OBC) (2022 amended).

Located at the corner of Bank Street and Somerset Street West, the iconic Somerset House neighbours Somerset Street West to the north, a public parking lot to the east, Bank Street to the west and 297 Bank Street, a single storey commercial building, to the south. The original west wing of Somerset House was built in 1897-1899, while the east wing addition was constructed in the early 1900's. Only a portion of the full structure remains today, which includes the original heritage north, west and south facing brick masonry walls. The building has remained vacant for the past 15+ years with its interior fully gutted.

The general rehabilitation intent for 352 Somerset St. W involves seismically upgrading the existing building and constructing a new 3-storey addition to restore the previous east wing. The intent from a structural design perspective is to structurally separate the two buildings in accordance with the requirements of the Ontario Building Code. It is currently planned that the ground floor and basement levels will be commercial space and the second and third floors will be residential. The new addition will be designed in accordance with the full requirements of Part 4 of the Ontario Building Code while the existing building will enact Part 11 Compliance Alternative for full relief from Section 4.1.8. (Seismic Requirements).

A seismic performance objective review was conducted to determine the target building performance level and seismic loading to be considered in the structural analysis. Given the age and heritage nature of the building, a Limited Performance Objective (LPO) corresponding to a Life-Safety Performance requirement and a target seismic design load equivalent to a 10%/ 50-year probable exceedance was selected based on a risk assessment and the guidelines outlined in ASCE/SEI 41-17, "Seismic Evaluation and Retrofit of Existing Structures", as permitted by NBCC Commentary 'L' paragraph 45.

In-situ testing was conducted by Keller Engineering to determine mechanical properties of the multi-wythe brick masonry making up the exterior heritage walls. The mechanical properties of the brick masonry determined from the in-situ test results were used in the structural analysis of the building. Regular scheduled Field Investigations have been conducted by AEI over the past six (6) years, which has helped compile a comprehensive assessment of building condition and to establish overall building configuration.

A structural analysis of Somerset House was conducted to evaluate the building performance for the LPO target seismic design loads. Two-dimensional FEM non-linear static models were formulated using Etabs v2016, to evaluate the in-plane performance of each URM wall. Calculations, in accordance with CSA O86-19, were performed to evaluate the floor and roof diaphragm performance. Failure Line Method calculations, in accordance with CSA S304-14 (R2019) "Design of Masonry Structures", were used to evaluate the out-of-plane performance of the URM walls.

A deficiency list was compiled based on the results of the structural analysis which will need to be addressed as part of the upcoming seismic upgrade and rehabilitation works. Recommendations for seismic upgrade and rehabilitation are outlined in Section 10.0. Detailed design of proposed rehabilitation and seismic upgrade will be included as part of the Issued for Permit Structural Drawing Package.

1.0 Introduction

The purpose of the report is to outline ART Engineering Inc.'s (AEI's) intended design approach and methodology during the upcoming permitting process, specifically from a seismic design perspective, to ensure the structural rehabilitation of Somerset House is in full compliance with the requirements of 2012 Ontario Building Code (OBC) (2022 amended).

All calculations enclosed in this report represent the best judgement of AEI at the time of preparation and are solely for developing a general rehabilitation design philosophy and determining feasibility of the proposed approach.

2.0 General Background

2.1 Brief Recent Building History

Located at the corner of Bank Street and Somerset Street West, the iconic Somerset House neighbours Somerset Street West to the north, a public parking lot to the east, Bank Street to the west and 297 Bank Street, a single storey commercial building, to the south (Refer to Figure 1 for site location).

Figure 1 –Site Location

The original west wing of Somerset House was built in 1897-1899, while the east wing addition was constructed in the early 1900's. Significant renovation/ reconstruction of the west wing began with the current owner in early 2007 consisting of remediating exterior load bearing masonry walls, removing interior load bearing masonry walls and replacing with new interior structural steel beams and columns, installing new light-framed lumber floor systems, and underpinning sections of the foundations. Any renovation/ remediation works done prior to 2007 are unknown at the time of writing this report. On October 19, 2007, a partial collapse of the interior masonry common wall at gridlines 6/B-C occurred while renovation/ reconstruction works were underway. Building stabilization works progressed through to the end of December 2007, which involved demolishing a large portion of the east wing to grade, retaining the north and east facing brick façades. The north and east facing brick facades of the east wing addition were later removed circa 2015-2016.

This configuration remains largely unchanged since 2017 with the exception of some minor stabilization and remediation works completed based on the recommendations from regular scheduled site inspections.

2.2 Current Building Configuration

The extant building now consists of a portion of the original west wing, consisting of three exterior heritage brick masonry walls (north, west and south sides) (original construction) extending three storeys above grade (14 m approx.) and a single basement level, with approximate base plan dimensions of 17x17 m extending from gridlines A-C and 1 to 5 (Refer to Appendix C for 2023 As-Built Drawings). The exterior heritage brick masonry walls are supported on rubble stone foundation walls (original construction) that are underpinned (constructed 2007 or earlier). The floor framing at all levels generally consists of light-framed wood floor joists (constructed 2007) supported on wrought iron and steel beams and columns (constructed 2007 or earlier). Cast-in-place concrete columns support the ground floor framing (constructed 2007). Riveted, wrought iron roof trusses and rafter system (original construction) form the roof system. Temporary lumber stud walls enclose the east side of the building at all levels (constructed 2007).

In June 2022, some stabilization and remediation works were completed including installation of additional lateral braces in the basement, grouting of voids present in the north foundation wall, interior repointing of brick masonry wall and pier elements, and infilling of the northwest floor openings on the ground, second and third floor levels (Refer to Appendix 'A' for photos of current building configuration).

2.3 Relevant Project Documentation

The following related documentation are cited with respect to past recommendations for the building, provides detailed as-built information of the building and provides descriptions of the structural deficiencies identified:

Structural Documents, prepared by AEI (2007 to Current):

- 2023 As-Built Drawings, dated February 6, 2023 (Appendix C);
- Field Reviews #1 to #51, dated June 23, 2017 to June 14, 2023;
- 99% Complete Structural Drawings, dated June 6, 2023;
- Stabilization & Remediation Progress Reports #1 & #2, dated August 3, 2022, and August 15, 2022 respectively;
- Stabilization and Remediation Drawings, SR-1 to SR-6, revision 1, dated October 10, 2022;
- Excavation Shoring Drawings, SE-1 to SE-2, revision 1, dated December 7, 2022;
- Recommended Investigation & Testing Report, dated August 25, 2022;
- Feasibility Study & Structural Design Brief, dated February 3, 2020;
- 352 Somerset Street (Somerset House) Updated Design Intent Synopsis, dated November 1, 2017;
- 352 Somerset Street (Somerset House) Design Intent, dated August 8, 2017;
- 2007 As-Found Drawings, dated March 19, 2007;

Architectural Documents, prepared by Chmeil Architects (2017 to Current):

- 99% Complete Architectural Drawings, dated June 6, 2023;
- Architectural Drawings, Issued for Client Review, dated June 7, 2017;
- Somerset House Building Condition Assessment Ornamental Façade Metalwork Restoration, revision 1, dated March 8, 2017;

Geotechnical Documents, prepared by Patterson Group Ltd. (2017 to 2020):

- Geotechnical Investigation Report (No. PG4081-1), dated March 9, 2020;
- Geotechnical Investigation Report, dated February 28, 2017;

Material Testing Reports (2009 to 2023):

- 352 Somerset – In-Situ Measurement of Masonry Mortar Joint Shear Strength Index, prepared by Keller Engineering, dated January 20, 2023;
- 352 Somerset – In-Situ Measurement of Masonry Deformability Properties, prepared by Keller Engineering, dated January 20, 2023;
- Brick Masonry Review and Testing Report, prepared by EXP, dated June 6, 2019;
- Masonry Testing at 352 Somerset Street, Ottawa, prepared by Keller Engineering, dated November 13, 2009;

Heritage Reports (2022):

- A Cultural Heritage Impact Statement – Somerset House Draft, prepared by Commonwealth Historic Resource Management, dated April 2, 2022;

Third-Party Engineer Review Reports (2016 to 2021):

- Re: Peer Review of Engineering Reports Prepared by Art Engineering Inc. in Relation to the Building at 352 Somerset Street West in Ottawa, prepared by Ojdrovic Engineering, dated November 23, 2021;
- Re: Somerset House, 352 Somerset Street, Ottawa: Peer Review of Brick Masonry and Condition Analysis, prepared by Trevor Gillingwater Conservation Services Inc., dated September 19, 2019;
- Re: Peer Review of Engineering Reports Prepared by Art Engineering Inc. in Relation to the Building at 352 Somerset Street West in Ottawa, prepared by Ojdrovic Engineering, dated September 6, 2019;
- Re: Heritage Structural Review of 352 Somerset St. W., Ottawa, prepared by Ojdrovic Engineering, dated June 21, 2016;

2.4 Referenced and Related Publications

The following listed publications have been referenced as part of preparing this report:

American Society of Civil Engineers (ASCE):

- ASCE/SEI 41-17, "Seismic Evaluation and Retrofit of Existing Buildings", 2017;

Canadian Standards Association (CSA):

- CSA S306-14, "Design of Masonry Structures", 2014;

Canadian Masonry Symposium (7^{th} to 14^{th}):

- "In Situ and Laboratory Testing of the Canadian Parliament Building's Historic Masonry", M. Chase, D. Arnold, R. Lukic, D. Carson, 2021;
- "Advanced 3D Interface Model for Finite Element Analysis of Unreinforced Masonry Structures", B. Zeng, Y. Li, 2021
- "Out-of-Plane Lateral Capacity of Unreinforced Masonry Walls: A Predictive Analysis Before Experimentation", H. Scacco, L. Silva, G. Casconcelos, G. Milani, P. Lourenco, 2021;
- "Retrofit of Unreinforced Masonry Buildings: The State-of-the-Art", Y. Korany, R. Drysdale, S. Chidiac, 2001.

Federal Emergency Management Agency (FEMA):

- FEMA 274, "NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings", 1997;
- FEMA 306, "Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings", 1998;
- FEMA 356, "Seismic Rehabilitation of Buildings", 2000;
- FEMA 547, "Techniques for the Seismic Rehabilitation of Existing Building", 2006;

New Zealand Society for Earthquake Engineering (NZSEE):

- "The Seismic Assessment of Existing Buildings", Chapter C8 – Unreinforced Masonry Buildings, 2017;

3.0 Limitations

AEI does not assume liability for elements/components not included in the study. The following limitations apply to the structural analysis performed by AEI:

- All calculations represent the best judgement of AEI at the time of this report and are solely for developing a general rehabilitation design philosophy and determining feasibility of the proposed approach.
- The scope of this report covers seismic review of the main structural resisting elements only. No gravity or wind load cases were considered.
- The material properties used in the structural analysis are based on in-situ field testing and the visual site condition assessment of the structure.
- There are some areas of the structure which cannot be visually reviewed at the time of writing this report (eg: north-west corner column, exterior face of the foundation wall and concrete underpinning). Conservative assumptions were made for these areas for the purposes of the structural analysis and will be confirmed prior to construction.
- This report is only relevant for the rehabilitation intent outlined in Section 4.0. If the rehabilitation intent undergoes major change, the seismic structural review will need to be re-reviewed.

4.0 General Rehabilitation Intent

The general rehabilitation intent for 352 Somerset St. W involves the following:

1. The rehabilitation and seismic upgrade of the existing building, and;
2. The design and construction of a new 3-storey addition to restore the previous east wing.

The intent from a structural design perspective is to structurally separate the two buildings in accordance with clause 4.1.8.14(1)(a) of the 2012 OBC (2022 amended) using an expansion joint. It is currently planned that the ground floor and basement levels will be commercial space and the second and third floors will be of residential.

The new addition will be designed in accordance with the full requirements of Part 4 of the 2012 OBC (2022 amended). Given the age and heritage nature of the existing building, it would be economically unfeasible and detrimental to the building's heritage nature to detail the building's rehabilitation to meet the full seismic requirements of the current building code. Part 11 of 2012 OBC (2022 amended) is intended to facilitate the rehabilitation and preservation of heritage structures, without undue financial hardships, thereby encouraging owners to provide performance level improvements and ultimately extending the building's service life. Section 11.5.1.1 of 2012 OBC (2022 amended) provides compliance alternatives where the Chief Building Official (CBO) is satisfied that compliance to Section 4.1.8. (seismic requirements) of the code is impractical due to; (a) structural or construction difficulties, or; (b) detrimental to the preservation of a heritage building. As such, the intention is to request full relief of Section 4.1.8, as permitted in Table 11.5.1.1.C (C.A. Number C88), Table 11.5.1.1.D/E (C.A. Number DE81) and by the CBO.

The following sections (Section 5.0 & Section 6.0) outline AEI's approach for determining the target seismic performance level for the planned rehabilitation and seismic upgrade of the existing building.

5.0 Original Building Evaluation & Seismic Performance

5.1 Building Construction Overview

To ensure that the rehabilitated building will at least maintain or exceed the original seismic performance level, the original conditions of the building must be evaluated (the original building refers to the portion of 352 Somerset built in 1897-1899, excluding the east addition built in the early 1900s).

The building construction consists of 3-wythe unreinforced masonry (URM) perimeter bearing walls, laid in a running bond pattern with header courses spaced every 7 to 10 courses and collar joints left generally unfilled. The floors were constructed of rough-cut lumber joists and sheathing supported on interior, walls, posts and beams and embedded in exterior URM walls. The building was founded on stone rubble foundations (exterior) and multi-wythe URM brick masonry piers (interior). The building foundations are founded on Class C soils, in accordance with the latest Geotechnical Investigation Report (Refer to Section 2.0). This building construction type is common of heritage buildings across Canada and the United States and has consistently been shown to perform poorly in seismic events (ref. FEMA 547/ 2006 Edition).

URM bearing walls are generally quite rigid were there are no significant wall perforations. However, in the case of Somerset House, the North and West walls are heavily perforated, with large bay openings present at the ground floor level, which presents a Type 1, Type 3, Type 4 and Type 6 structural irregularities in the seismic-force-resisting system (SFRS). The east and south URM walls were largely unperforated at the time of original construction.

5.2 Lateral Analysis of Original Building

A structural analysis was conducted to estimate the upper-bound (U.B.) (highest probable) lateral seismic capacity of the original constructed, 1897-1899, building based on the information and documentation readily available at the time the analysis was conducted. A non-linear 3D FEM model of the original building was formulated, and the analysis methodology and results were outlined in the previous report titled, "Design Intent Report: 352 Somerset Street W", prepared by AEI, dated August 8, 2017. An updated report titled, "Updated Design Intent Synopsis: 352 Somerset Street W", prepared by AEI, dated November 1, 2017, was subsequently issued following discussions with the City of Ottawa Building Code Services Department (Both documents are included in Appendix 'E').

A summary of the results from the previously conducted structural analysis are included below for convenience:

- Some of the changes made to the existing building over the past 120+ years yield an improvement on the structure's performance level from a seismic point of view, while other items such as the removal of the east wall are considered to have a negative impact on the performance level;
- The U.B. seismic weight of the original structure was calculated to be 8.2 MN;
- The U.B. base shear capacity of the original structure was calculated to be only 6% of 2012 OBC Part 4 prescribed seismic load;

The reported upper-bound (highest probable) seismic lateral capacity of the original structure is expectedly low when compared to modern building design, measuring to only 6% of current Part 4 Seismic Loads. Therefore, it our opinion that life-safety risk in a seismic event remains high even if the building was seismically upgraded to meet the original building's upper-bound seismic performance level.

To further mitigate risk from a seismic event, a "Seismic Performance Objectives Review" was conducted (Section 6.0). The Performance Objectives Review will be used to evaluate the acceptable level of risk from a seismic event, to determine the corresponding seismic loads considered in design and to determine the extent of seismic upgrades required.

6.0 Seismic Performance Objectives Review

6.1 Performance Objectives Introduction

NBCC 2015 Structural Commentary 'L' "Application of NBC Part 4 of Division B for the Structural Evaluation and Upgrading of Existing Buildings" states that the intent of a seismic upgrade is to ensure that the Seismic Force Resisting System (SFRS) is compatible with the desired level of risk. To fully comply with a target upgrade level, the building must be able to withstand the seismic load for the target performance level and the drift imposed by the seismic load.

ASCE/SEI 41-17 "Seismic Evaluation and Retrofit of Existing Structures" was used as a guideline evaluate the acceptable level of risk and to determine the target building performance level (as permitted by the Commentary 'L' paragraph 45). ASCE/SEI 41 is intended to serve as a tool for the design professional, code officials and building owners undertaking seismic evaluation or retrofit of existing and heritage buildings. The standard is written based on experience-based judgement and is largely derived from observations of seismic impacted buildings.

Building performance is described qualitatively in terms of:

- The safety afforded to building occupants during and after a seismic event;
- The cost and feasibility of restoring the building to its pre-earthquake condition;
- The length of time the building is removed from service to conduct repairs;
- Economic, architectural, or historical effects on the larger community.

The target risk level selected will dictate the extent of seismic upgrades implemented, the cost and feasibility of the project, at the benefit of improved safety, reduction in property damage, and interruption of building use, in the event of future earthquakes.

6.2 Rational for Selecting Performance Objective

Given that Somerset House is a designated heritage building that is 125+ years old, a careful examination into the following were taken into consideration for purposes of selecting an appropriate target risk level:

- The original heritage elements of the building (URM walls, stone foundation, roof trusses) are 125+ years old and were not constructed with overall building lateral-load capacity in mind;
- The URM walls generally behave in a brittle nature (no reliable energy dissipating mechanism present). The capacity of these walls is limited by the low-strength lime-based heritage mortars used and stringent drift limits inherent of URM;
- The seismic upgrades implemented must not be detrimental to the preservation of the heritage attributes;
- The seismic upgrades implemented must not dictate an exorbitant level of construction effort to implement and be without undue financial hardship to the owner;
- The retrofitted building will have commercial occupancy at the basement and ground floor levels and residential occupancy at the second and third floor levels. The building's Importance Factor is selected as "NORMAL", as defined by 2012 OBC (2022 amended);
- Given the short-period spectral response at 0.2s and the long-period spectral response at 1.0s, the building location is classified to have a "LOW" Level of Seismicity, as defined by ASCE/SEI 41-17;
- The seismic upgraded building should retain a margin of safety against the onset of partial or total collapse when subjected to design seismic loads to minimize the life-safety risk of occupants and the general public.
- The seismic upgraded building should have a post-earthquake state that is able to support gravity loads for occupants to safely evacuate the building.

Given the above considerations, a "Limited Performance Objectives" (LPO), as defined in ASCE/SEI 41-17, has been selected as the most suitable target performance level for seismic upgrade of Somerset House. A LPO strikes a balance between producing a cost-effective seismic upgrade design, while not majorly affecting the heritage attributes of the building and affords building occupants with an increased level of safety during and after an earthquake event.

Buildings meeting a LPO are expected to experience some light-to-moderate levels of damage from the infrequent earthquakes that occur in Ottawa and be subject to high levels of damage and potential economic loss from rarer earthquakes in Ottawa. The level of damage and potential economic loss experienced by buildings rehabilitated to the LPO, is expected to be much greater than similar sized new constructed buildings but is expected to retain a margin of safety against partial or total collapse.

6.3 Target Building Performance Level (Seismic Design Loads)

The following seismic design loads have been selected based on the target risk level for a LPO building:

Table 1: Proposed Building Performance & Seismic Design Loads:

Performance Objective	*Building Performance Level	Seismic Hazard Level	**Design Base Shear at Hazard Level (kN)	Description of Performance Level
LPO	Life Safety Performance (S-3)	10%/ 50-year Probable Exceedance [475 years Mean Return Period]	586	Structure is damaged (possibly irreparable) but retains a margin of safety against the onset of partial or total collapse

*Building Performance Level as defined in ASCE/SEI 41-17.
**Design Base Shear calculated based on anticipated seismic weight of renovated existing building equal to 4.8 MN.

7.0 Investigations & Material Testing

A total of three (3) masonry testing programs have been undertaken within the past 15 years. A summary of the tests completed and of the results are included in the following sections for convenience. The test results capture material property change over time as well as regularly conducted visual inspections.

7.1 Brick Deformability and Shear Testing (Keller, 2023)

Two types of masonry tests were completed on December 22, 2022, by Keller Engineering. In-situ masonry deformability tests (ASTM C1197) were performed in three areas, Young's Modulus values were derived from these tests. In-situ masonry mortar joint shear tests (ASTM C1531) were performed in two areas, masonry shear strength values were derived from these tests. The tests were conducted as directed by AEI, as outlined in the Recommended Investigation & Testing Report, dated August 25, 2022. The summary of the test results is included in Table 2.

Table 2: Brick Masonry Test Results for ASTM C1197 & ASTM C1531 Procedures:

Location	Calculated Average Young's Modulus, E_m (MPa)	In-situ Shear Strength Index, τ_o (kPa)
3rd Floor, North Wall (Interior)	1049	540 to 590
2nd Floor, South Wall (Interior)	726	170 to 282
1st Floor, South Wall (Interior)	3220	N/A

7.2 Brick Unit Compressive Strength and Permeability Testing (EXP, 2019)

Brick samples were taken from four areas within the exterior walls. Absorption and compressive strength testing of the brick units was completed. A total of five brick units at each location were taken and testing in the laboratory. The following locations were sampled:

- Second Floor, interior wythe, north wall;
- Ground Floor, exterior wythe, north wall;
- Third Floor, exterior wythe, south wall;
- Second Floor, middle wythe, north wall;

Absorption and freeze-thaw durability tests were conducted on the sampled bricks in accordance with the requirements of CSA A82-14. In summary, all tested samples allowed 8.0% or greater water absorption percentages, and hence failed the 24-hour test.

Compressive strength tests were conducted on the sampled bricks in accordance with the requirements of CSA A82-14. The average compressive strength values reported for each location are summarized in Table 3.

Table 3: Brick Unit Compressive Strength Test Results:

Location	Compressive Strength of Brick Unit, f'_m (MPa)
2nd Floor, North Wall (Interior wythe)	20.3
1st Floor, North Wall (Exterior wythe)	28.9
3rd Floor, South Wall (Exterior wythe)	12.1
2nd Floor, North Wall (Middle wythe)	33.0

The test results indicate that two of the four areas failed the CSA A82-14 compressive test requirements.

7.3 Brick Deformability, Compressive, Shear and Pull-out Strength Testing (Keller, 2009)

Three types of masonry tests were completed on November 13, 2009, by Keller Engineering. In-situ masonry deformability tests (ASTM C1197) were performed in six areas, masonry compressive strength and Young's Modulus values were derived from these tests. In-situ masonry mortar joint shear tests (ASTM C1531) were performed in four areas, masonry shear strength values were derived from these tests. Anchor rod pull-out tests were performed in seven areas. The summary of the test results is as follows:

- The resulting average Young's Modulus was 482 MPa;
- The resulting average compressive strength of brick-mortar composition was 1314 kPa;
- The resulting average in-situ mortar joint shear index was 184 kPa;
- The resulting average anchor rod pull-out capacity was 4.7 kN;

7.4 Field Reviews and Structural Investigations (Art, 2017-2023)

Since 2017, AEI has been retained to provide regular scheduled site review of the building. Over the past six (6) years, AEI has provided recommendations for localized internal stabilizations and on-going repair and maintenance of the vacant building. Refer to Section 2.3 of this report for recent Field Reviews, Structural Investigations and other Studies prepared by AEI.

As-built drawings were recently prepared by AEI and are included in Appendix 'C'.

8.0 Structural Analysis

A structural analysis of Somerset House was conducted to evaluate the building performance under the LPO seismic design loads outlined in Section 6.0 and to determine the extent of seismic upgrades necessary.

Two-dimensional FEM non-linear static models were formulated using Etabs v2016, to evaluate the in-plane performance of each URM wall (Refer to Appendix 'B' for FEM Screenshots). Calculations, in accordance with CSA O86-19, were performed to evaluate the floor and roof diaphragm performance. Failure Line Method calculations, in accordance with CSA S304-14 (R2019) "Design of Masonry Structures", were used to evaluate the out-of-plane performance of the URM walls.

8.1 Material Properties

Material properties used in the structural analysis are consistent with results from the testing programs outlined in Section 7.0. Expected material properties considered for deformation-based failure modes were based on the average of the tested material properties. Lower-bound material properties considered for force-based failure modes were based on the average of the tested material properties minus one standard deviation. Table 4 outlines the material properties used for the structural analysis of the URM walls.

Table 4: URM Material Properties used in Structural Analysis:

Type	Youngs Modulus of Elasticity, E_m (MPa)	Shear Modulus, G_m (MPa)	Compressive Strength, f'_m (kPa)	Shear Strength Index, v_t (kPa)	Flexural Tensile Strength, f_t (kPa)
Expected	725	290	1310	396	198
Lower-Bound	558	223	1010	226	113

The shear modulus, G_m, was calculated using a poisson ratio of 0.25 for brick masonry. The compressive strength was calculated based on the Youngs Modulus of Elasticity, in accordance with CSA S304-14 (R2019). The shear strength, v_m, varies based on shear strength index, v_t, and the magnitude of axial load present in the element under consideration, in accordance with ASCE 41-17 and ASTM C1531. The flexural tensile strength, f_t, was conservatively taken equal to the shear strength, v_m, calculated with no axial load. The diagonal tension strength, f'_{dt}, was conservatively taken to be equal to the shear strength, v_m, of the element. Linear-elastic material properties were considered in the FEM model.

8.2 Loading Conditions

The seismic Ultimate Limit States (ULS) load cases considered in the analysis are in accordance with 2012 OBC (2022 amended) Part 4 requirements and are outlined in Table 5.

Table 5: Ultimate Limit States Load Cases:

Load Case	Load Combinations (ULS)
1	1.0DL + 1.0EQ
2	1.0DL + 1.0EQ + 0.25SL

*DL = dead load including superimposed dead load. EQ = earthquake load. SL = snow load.

The following gravity loading was considered in the structural analysis:

- Self-weight of brick masonry = 19 kN/m^3;
- Self-weight of structural steel or wrought iron components = 77 kN/m^3;
- Roof dead load = 1.50 kPa;

- Floor 3 dead load = 1.50 kPa;
- Floor 2 dead load = 2.00 kPa;
- Roof Snow load = 2.32 kPa + Drifts as shown on plans.

The base shear due to seismic loading was calculated in accordance with the equivalent static force procedure, as per clause 4.1.8.7.(c) of the building code. A 5% dampened response spectra for a 10%/50 year probability of exceedance seismic event was calculated using the 2015 NBC Seismic Hazard Calculator using the geodetic coordinates of Somerset House. The fundamental period of the building was calculated as 0.36 seconds in both principal directions using the simplified equations provided in the building code. Site Class 'C' was considered in calculation of the base shear, as reported in the Geotechnical Investigation Report, prepared by Patterson Group, dated March 9, 2020.

The calculated base shear was distributed to each diaphragm in proportion to the assigned floor mass and height in accordance with clause 4.1.8.11 of the building code. Seismic forces were considered to act in each principal axes and in each direction. Shear walls parallel to the load direction were considered to support their own self-generated seismic forces, whereas shear walls perpendicular to the load direction were considered to have their seismic generated forces transferred to the diaphragm based on tributary height.

Flexible diaphragms have been assumed at each floor level and at the roof for light-framed sheathed lumber diaphragms supported on URM shear walls. Based on flexible diaphragm assumption, loads have been distributed to each SFRS line based on tributary width. Accidental eccentricity of +/- 5% associated with flexible diaphragms in accordance with CSA O86-14 (R2019) was considered in the analysis.

Storey shears in perforated shear walls are distributed to wall piers in proportion to the relative lateral uncracked stiffness of each wall pier taking into consideration the rotational restraint provided by the bounding spandrels.

Ductility and overstrength values have been taken as unity (Rd = 1.0, Ro = 1.0) for URM shear walls in accordance with Table 4.1.8.9 of the building code. Brace forces and URM wall stresses were determined based on Rd & Ro factors of 1.0. Diaphragm and foundation forces were determined based on capacity-based design principals using an overstrength factor of 1.2.

8.3 Structural Element Modeling & Boundary Conditions

Shell elements were used to model the URM shear walls, piers and spandrels considering uncracked geometric properties. Line elements were used to model the wrought-iron and steel beams, columns and braced frame elements using equivalent geometric properties and pinned-end connections.

To model the soil-structure-interaction, vertical compression only springs were used at the base of URM walls to simulate the bearing resistance of foundation and horizontal springs have been used to model the bed joint interface between the brick wall and stone foundation. The spring stiffnesses was calculated using the shallow rigid footing assumption outlined in Section 8.4 of ASCE/SEI-41-17, assuming a firm-to-stiff unsaturated clay founding soil.

8.4 Non-Linear Staged Static Analysis

A non-linear staged analysis was considered necessary to accurately account for the already loaded masonry walls and the addition of new braced frame elements. P-delta effects have also been considered in the non-linear analysis. The following staging was considered in the structural model for the purposes of determining the wall stresses, braced frame forces and drifts.

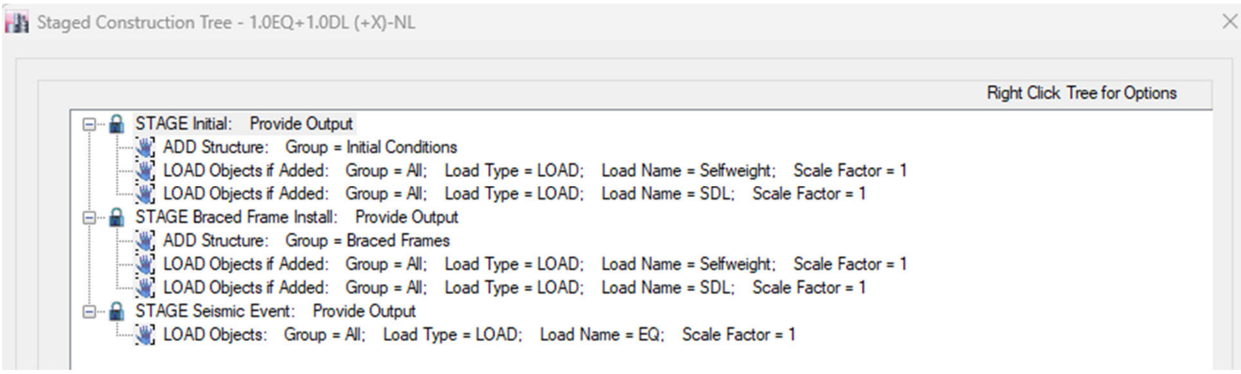


Figure 2: Non-Linear Staged Analysis Construction Tree

8.5 Seismic Response

The recommendations of NBCC Commentary ‘L’ provide the following insight into evaluating the seismic response of existing buildings:

> *“In doing a seismic review of a building or in determining its compliance with the current NBC, it is important to review both the capacity of the SFRS to carry seismic loads and the ability of the vertical-load-carrying system to accommodate the deformations that the seismic loading will impose. For brittle structures, such as buildings with multi-wythe brick walls or non-ductile concrete frames, drift requirements often govern. Information on the drift capacities of various types of construction can be found in ASCE/SEI 41.”*

URM buildings typically have flexible diaphragms and stiff walls. There is little relative dynamic amplification between the base and top of the URM walls in the direction parallel to the seismic force direction. Significant amplification instead occurs at the midspan of the flexible diaphragms as they are driven by in-plane motion of the end walls. This generates large out-of-plane forces on the connections between the diaphragm and exterior walls, causing the diaphragm to yield.

URM walls and piers have five primary in-plane failure modes. Deformation-controlled (ductile) in-plane failure modes include rocking and bed-joint sliding. Force-controlled (brittle) in-plane failure modes include toe crushing, diagonal tension and vertical compression. URM spandrels have two primary in-plane failure modes; deformation-controlled (ductile) shear failure and force-controlled (brittle) horizontal compression In-plane capacities were calculated in accordance with ASCE/SEI 41-17 Chapter 11.

Another failure mode is out of plane bending of URM wall, which is a force-controlled (brittle) failure mode. Out-of-plane (OOP) wall capacity was calculated in accordance with CSA S306 Failure Line Method. For OOP URM wall strength calculations, the collar joint was assumed to be void of mortar and therefore collar joint areas were not considered part of the effective thickness of the wall for out-of-plane behaviour.

Any structural steel braced frames added in the north or west URM walls are anticipated to be deformation-controlled, meaning that the braced frame design will be governed by stiffness rather than its capacity in order to provide meaningful engagement and load sharing characteristics with the URM walls. The capacity of the bracing will likely far exceed its load demand requirements.

9.0 Results

9.1 Results from In-Plane Analysis

The results of the structural analysis indicate that the north, south and west URM walls are overstressed and unstable when subjected to seismic loading. Additionally, the following structural irregularities were found to be present in accordance with clause 4.1.8.6 of the building code:

- Type 1 - Vertical Stiffness Irregularity (North & West Walls);
- Type 3 – Vertical Geometric Irregularity (North & West Walls);
- Type 4 – In-Plane Discontinuity in Vertical Lateral-Force-Resisting Element (North & West Walls):
- Type 6 – Discontinuity in Capacity – Weak Storey (North & West Walls);

The current building SFRS configuration also is lacking a secondary support line in the north-south load direction (East SFRS missing), causing instability.

The following structural steel braced frames will need to be added to eliminate structural irregularities and reduce load-demand and limit drift on URM wall components:

- North Wall – HSS152x152x9.5 cross-bracing and columns added at the ground floor level in the two open bays between Gridlines 1 & 2A and Gridlines 2A & 3A;
- West Wall – HSS152x152x9.5 cross-bracing and columns added at the ground floor level in the two open bays between Gridlines A & B and Gridlines B & D;
- East Side – Full-height HSS127x127x6.4 braced frame was added at the east side building along Gridline 4A, between Gridlines B & C.

The braced frames were included in the model and the in-plane demands on the URM walls was found to be reduced to within acceptable limits, as outlined in Table 6 (Refer to Appendix ‘B’ for FEM model screenshots).

Table 6: Demand-Capacity Ratios (DCR) for URM Walls:

Wall	Rocking Drift Limit, DCR	Sliding Shear, DCR	Diagonal Tension, DCR	Spandrel Shear, DCR	Vertical Compression, DCR
North Wall	0.503	0.490	0.574	0.793	**1.00**
West Wall	0.414	0.416	0.572	**0.979**	0.853
South Wall	0.164	0.167	0.156	*N/A*	**0.506**

* Demand-Capacity Ratios (DCR) listed represent the maximum DCR of all structural components which make up the wall (piers, spandrels, walls) for the listed failure mode. Bolded items represent governing failure mode. Rocking drift limits were taken as 0.75%, per ASCE/SEI 41-17 Life-Safety Performance Level.

Previous water infiltration from exposed ends has caused the mortar to deteriorate in critical areas such as the bond courses. Wall remediation will be required to consolidate the wall to good working conditions to provide sufficient in-plane capacity consistent with the material and geometric properties used in the structural analysis.

9.2 Results from Out-of-Plane Analysis:

The north and south walls were found to have inadequate out-of-plane capacity at the third-floor level, where the walls span approximately 5.8 m to the roof level. An additional line of lateral support at the bottom of the steel trusses will need to be provided to satisfy out-of-plane requirements. It should be noted however that the previous rehabilitations have already provided lateral bracing to the west wall at the base of the truss elevation using structural steel angle cross-bracing and is found to be adequate to laterally support the west wall.

The south wall was found to have inadequate out-of-plane capacity at all floor levels between Gridlines 2 & 3, where a stair opening is present. The south wall was found to be unrestrained for its full height, over a length of 4 m, as the stair framing does not provide any lateral support to the wall.

Previous water infiltration from exposed ends has caused the mortar to deteriorate in critical areas such as bond courses. Wall remediation will also be required to consolidate the wall to good working conditions to provide sufficient out-of-plane capacity.

9.3 Results from Diaphragm Analysis:

The floor and roof diaphragms were analysed and found to have insufficient in-plane capacity to transfer shear loads into SRFS. Additionally, the diaphragms were found to have missing chord elements at the second and third floor levels along the north and south walls between Gridlines 1 & 5, and at the roof level along the north and south walls between Gridlines 1 & 5.

The floor stair openings in the second and third floor diaphragms were found to have insufficient capacity to transfer in-plane forces around the opening in accordance FTAO (Force Transfer Around Opening) method as described in CSA O86-19.

Floor-to-wall connections were generally found to be sufficient at the second and third floor levels, except where anchor connection to the wall are made utilizing wood ledgers. The wood ledgers will need to be reinforced to avoid cross-grain tension failure and subsequent loss of support. The roof-to-wall connections were found to be insufficient along the north and south walls between Gridlines 1 & 5. The roof-to wall connection was found to be sufficient at the west wall between Gridlines A & D.

The second floor, third floor and rood diaphragms were found to be too flexible to provide adequate lateral support to the URM walls when considering out-of-plane load conditions.

The second floor, third floor and roof diaphragms were found to have been fitted with adequate crossties to tie the outer URM walls together.

Previous water infiltration from the roof has caused critical wood diaphragm components to rot and deteriorate over the years and have become structurally compromised. Rotted wood components will need to be replaced to provide sufficient in-plane capacity to transfer shear loading.

9.4 Results from Foundation Wall Analysis:

The north foundation wall between gridlines 1 & 5, and west foundation wall between gridlines A & B found to be deteriorated and needing remediation to support anticipated vertical and lateral loading due to seismic.

The south foundation wall between gridlines 1 & 5, and the west foundation wall between gridlines B & D were recently remediated within the past 15 years and are generally found to have sufficient capacity to support anticipated vertical and lateral loading due to seismic.

New braced frames added above will be supported on new independent foundations, so additional loads will not be imposed to the existing stone foundation.

Existing underpinning along the north and west wall do not fully support the stone rubble foundation and is eccentrically loaded unless braced by the future raft slab. Construction of a raft slab and extension of the underpinning is required to eliminate eccentricity insufficiencies with the foundation.

10.0 Proposed Rehabilitation & Seismic Upgrades

Based on the results of the structural analysis and previous building condition assessments, the proposed seismic upgrades and remediations are as follows:

- Exterior URM wall improvements, including:
  - Rake out and repoint deteriorated header and bed mortar joints, reset loose units, and replace frost-damaged units from reserve brick supply;
  - Consolidate URM walls by installing helical ties or other proprietary tie anchors (eg: Cintec anchors), where the header course is deteriorated;
  - Eliminate structural irregularities and limit URM wall stresses and drifts by installing new HSS structural steel braced frames on the north, west and east sides of the building;
  - Infill north URM wall opening between gridlines 3A & 4, at the ground floor level, with compatible brick masonry tying in with the surround wall;
  - Establish continuous lateral support to the north and south URM wall at the roof diaphragm level and at the underside of truss elevation;
  - Establish a positive connection between the URM north and west walls and the supporting wrought-iron beams at the ground floor level;
  - Install structural steel support at south wall stair opening to provide lateral support of the south URM wall at the second and third floor levels;
  - Establish a positive connection between the existing northwest corner column and the floor beams and to the foundation.
- Diaphragm improvements, including:
  - Remove and replace deteriorated/ rotted components (floor joists, subfloor, roof rafters, roof deck boards, blocking, etc.);
  - Infill the northwest corner opening at the second and third floor levels (completed 2022);
  - Install new plywood subflooring to stiffen the floor diaphragms and improve its capacity at the second and third floor levels;
  - Install new plywood decking to stiffen the roof diaphragm and improve its capacity at the roof level;
  - Install lumber cross bracing, or plywood sheathing, to existing knee walls that sit above the roof trusses that act as load-transfer elements. Provide necessary additional anchorage between load-transfer elements;
  - Install new diaphragm chords along the north wall of the second and third floors;
  - Reinforce existing floor diaphragm ledger chords to avoid potential cross-grain tension failure, or install new floor-to-wall connections;
  - Install new roof diaphragm-to-wall anchors to adequately transfer diaphragm forces at the north and south roof level and underside of truss elevation;

- Interior framing improvements, including:
    - Establish a positive connection between the existing floor beams and columns;
    - Providing redundant columns to provide secondary support to masonry piers;

- Foundation improvements, including:
    - Complete construction of raft foundation system to eliminate eccentric loaded foundations;
    - Confine north and west stone rubble foundation wall with new reinforced concrete foundation to support all gravity and lateral loads;
    - Install new reinforced concrete pads and foundations to support newly installed braced frames above;
    - Rake out and repoint deteriorated mortar joints between stone units, and reset loose units for the north, south and west foundations;
    - Consolidate stone rubble foundation walls by installing helical ties or other proprietary tie anchors (eg: Cintec anchors), where the bond between exterior and interior face wythes no longer exists;
    - Installing new suspended reinforced concrete ground floor slab to provide continuous lateral support of the exterior foundations;

In accordance with the recommendations of NBCC Commentary 'L', the following non-structural improvements are also recommended, since non-structural building components have shown to pose a greater risk in recent earthquake events rather than the building structure itself:

- Non-structural improvements, including:
    - Re-establishing a positive connection from cornices, ornaments, and appendages to the URM wall;
    - Restoring deteriorated cornices, ornaments, and wall appendages to working condition;
    - Reconstruct the two (2) west facing bay windows, refasten to the URM walls supported by the new structural steel frame.

It should be noted that with the above proposed seismic upgrades and remediations, and any additional future recommendations proposed by AEI, the rehabilitated structure will meet, and in most instances exceed, the original structure's seismic performance level as well as achieving a seismic upgrade to the selected requirements of a Limited Performance Objective (LPO) in accordance with Section 6.0 of this report, while fully complying to all other non-seismic sections of 2012 OBC (2022 amended) Part 4.

11.0 Summary

In summary, the general rehabilitation intent for Somerset House is to construct a east addition which is structurally separated from the existing building and designed to full Part 4 Ontario Building Code requirements. The existing structure would be rehabilitated and seismically upgraded as outlined in this report. A seismic performance objective review was conducted to determine the seismic loads and building performance considered in the structural analysis. A Limited Performance Objective corresponding to a Life-Safety Performance with a target seismic design load equivalent to a 10%/ 50-year probable exceedance was selected.

A structural lateral analysis of the existing structure was conducted to help compile a deficiency list which will need to be addressed as part of the upcoming seismic upgrade and rehabilitation works. Recommendations for seismic upgrade and rehabilitation were outlined in Section 10.0. Detailed design of proposed rehabilitation and seismic upgrade will be included as part of the Issued for Permit Structural Drawing Package.

We trust that that above satisfies your requirements. Should you have any further questions, please do not hesitate to contact our office at (613) 836-0632.

Timothy Berg, P.Eng.
Senior Engineer & Team Lead

Hussein Makke, M.Eng., P.Eng.
Director, Buildings & Infrastructure

Appendix A: Site Photographs from Field Investigations

Figure A1 – West Exterior Wall

Figure A2 – North Exterior Wall

Figure A3 – East Side of the Building

Figure A4 – Ground Floor Facing East

Figure A5 – Basement North Foundation

Figure A6 – Basement South Foundation

Figure A7 – Second Floor Facing North

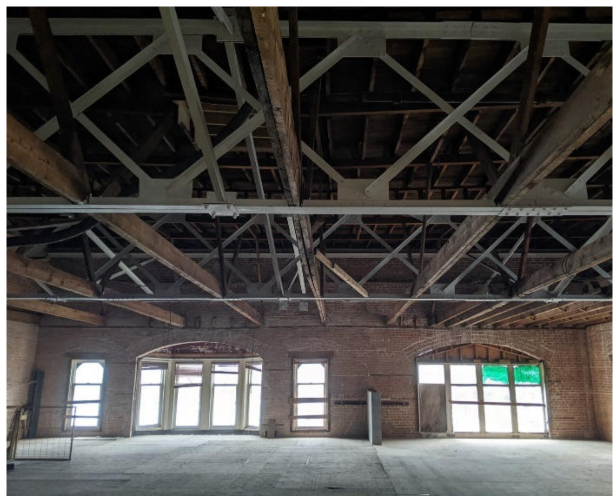


Figure A8 – Third Floor Facing West

Appendix B: FEM Models Screenshots

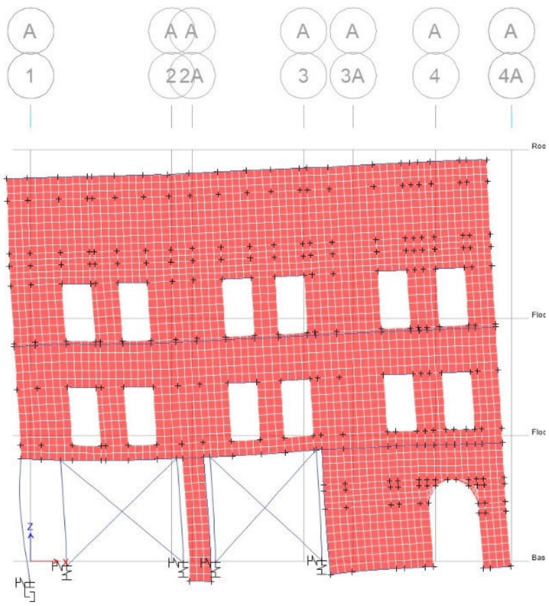


Figure B1 – North Wall Deflection (-X)

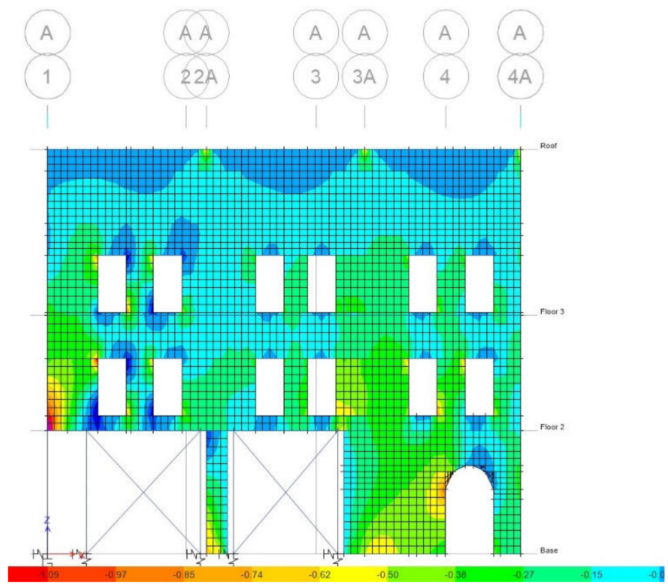


Figure B2 – North Wall Vertical Stress (S22)

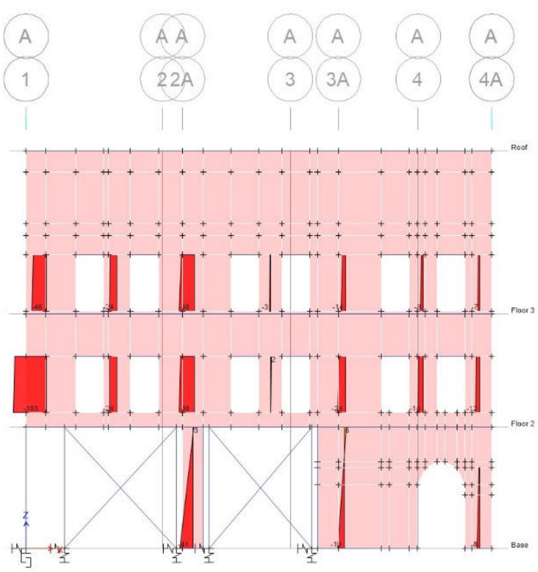


Figure B3 – North Wall Pier Shear (V22)

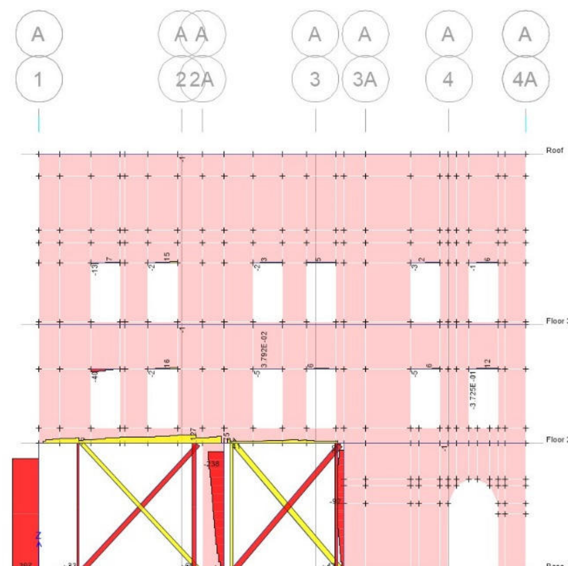


Figure B4 – North Wall Braced Frame Axial (-X)

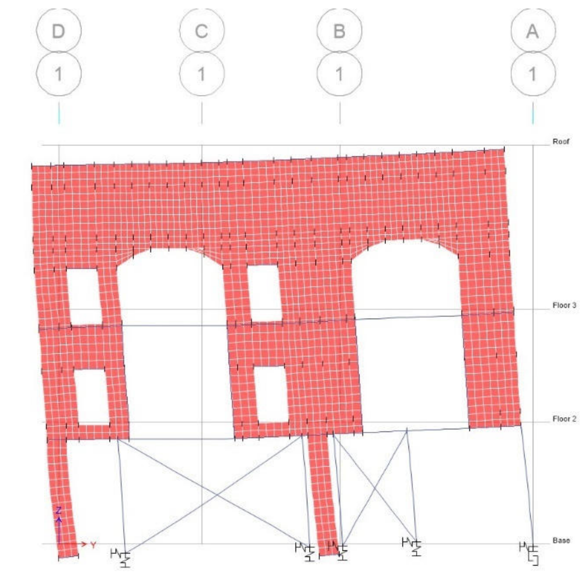


Figure B5 – West Wall Deflection (-Y)

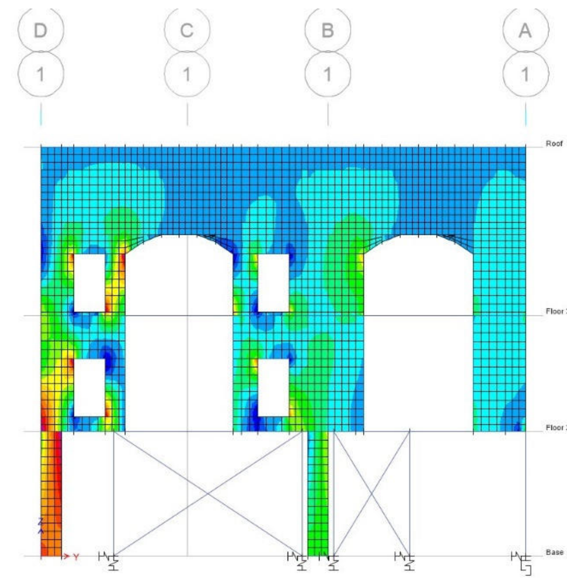


Figure B6 – West Wall Vertical Stress (S22)

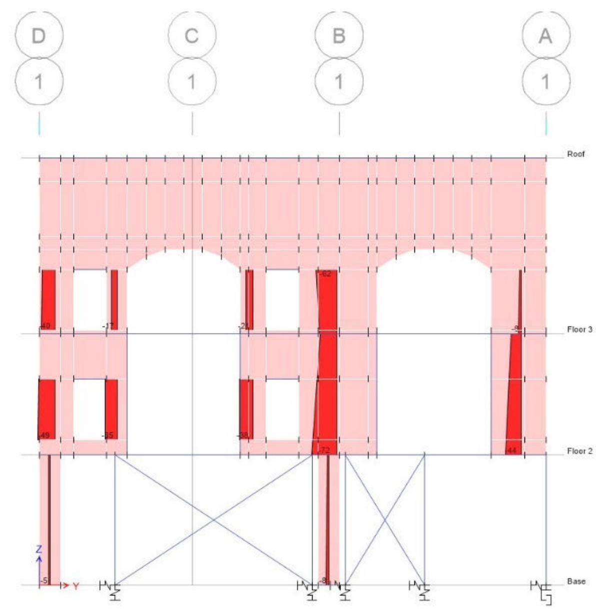


Figure B7 – West Wall Pier Shear (V22)

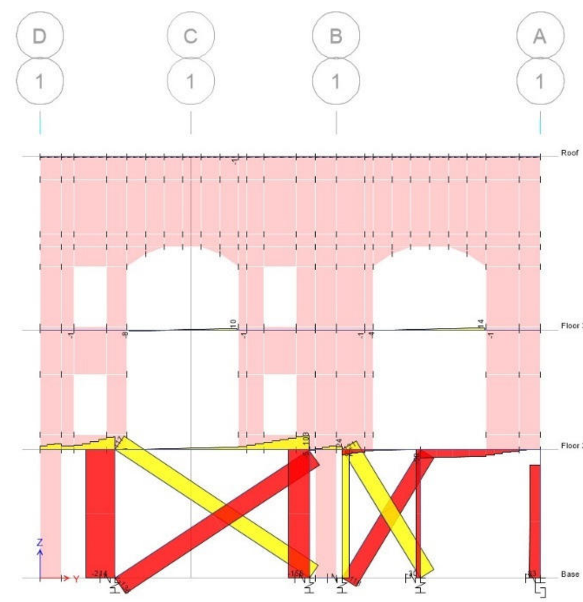


Figure B8 – West Wall Braced Frame Axial (-Y)

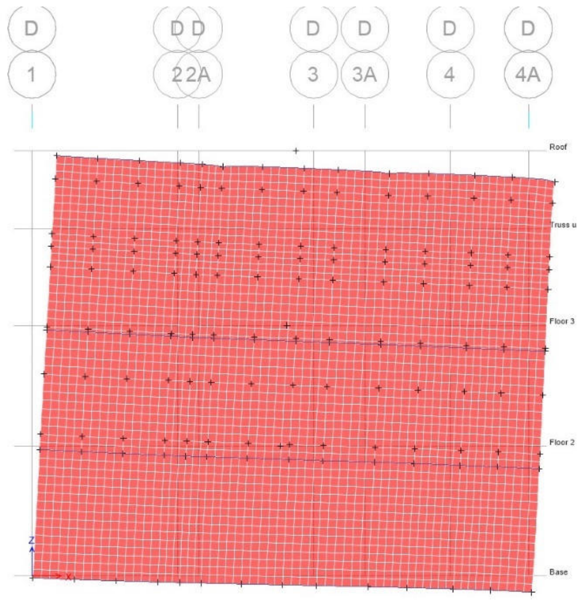


Figure B1 – South Wall Deflection (+X)

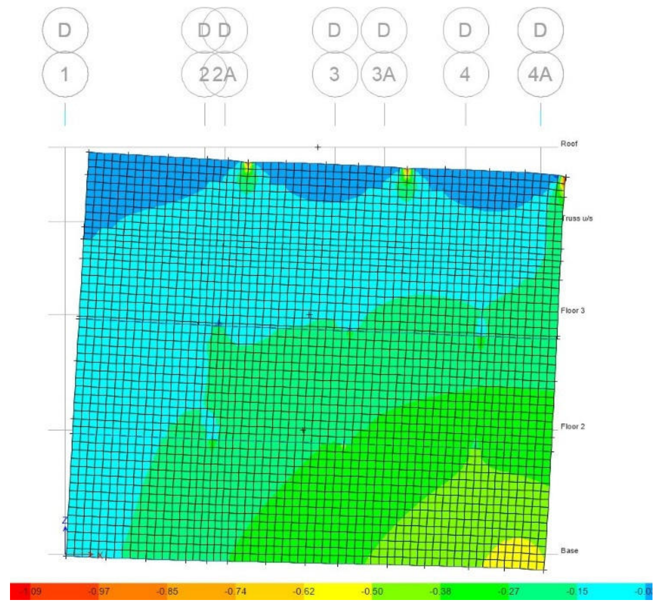
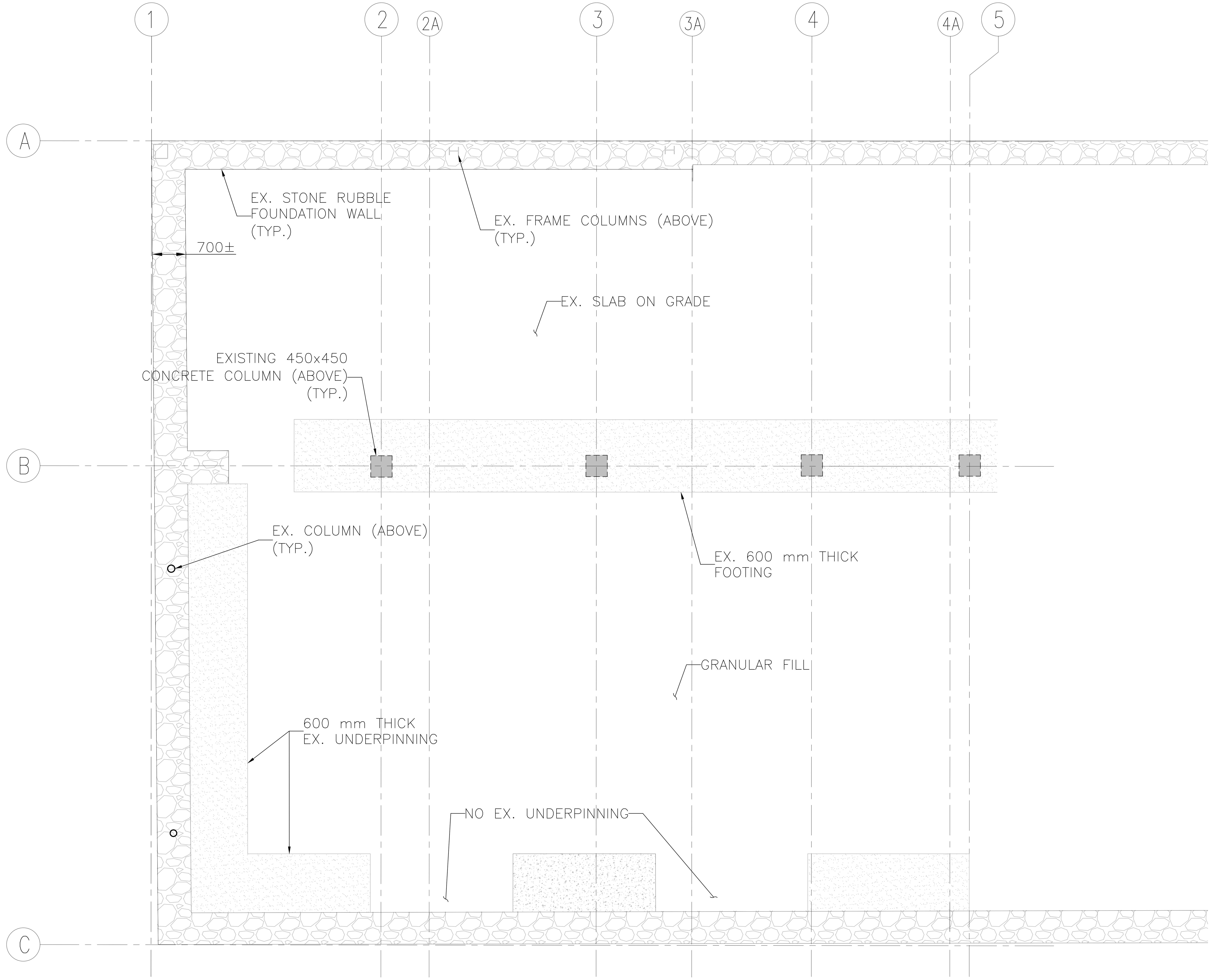


Figure B2 – South Wall Vertical Stress (S22)

Appendix C: 2022 As-Built Drawings (Art, 2023)



client

T.K.S. HOLDINGS INC.

project

AS–BUILT

352 SOMERSET STREET WEST
OTTAWA, ON

art engineering inc.
171 Walgreen Road
Carp - Ontario - K0A 1L0 - Canada
(613) 836-0632 - Fax: (613) 836-1226
www.artengineering.ca

A detail no.
no. de détail
B location drawing no.
sur dessin no.

scale 1:50

revisions		date
0	AS–BUILT	06–02–2023

drawing

BASEMENT

stamp

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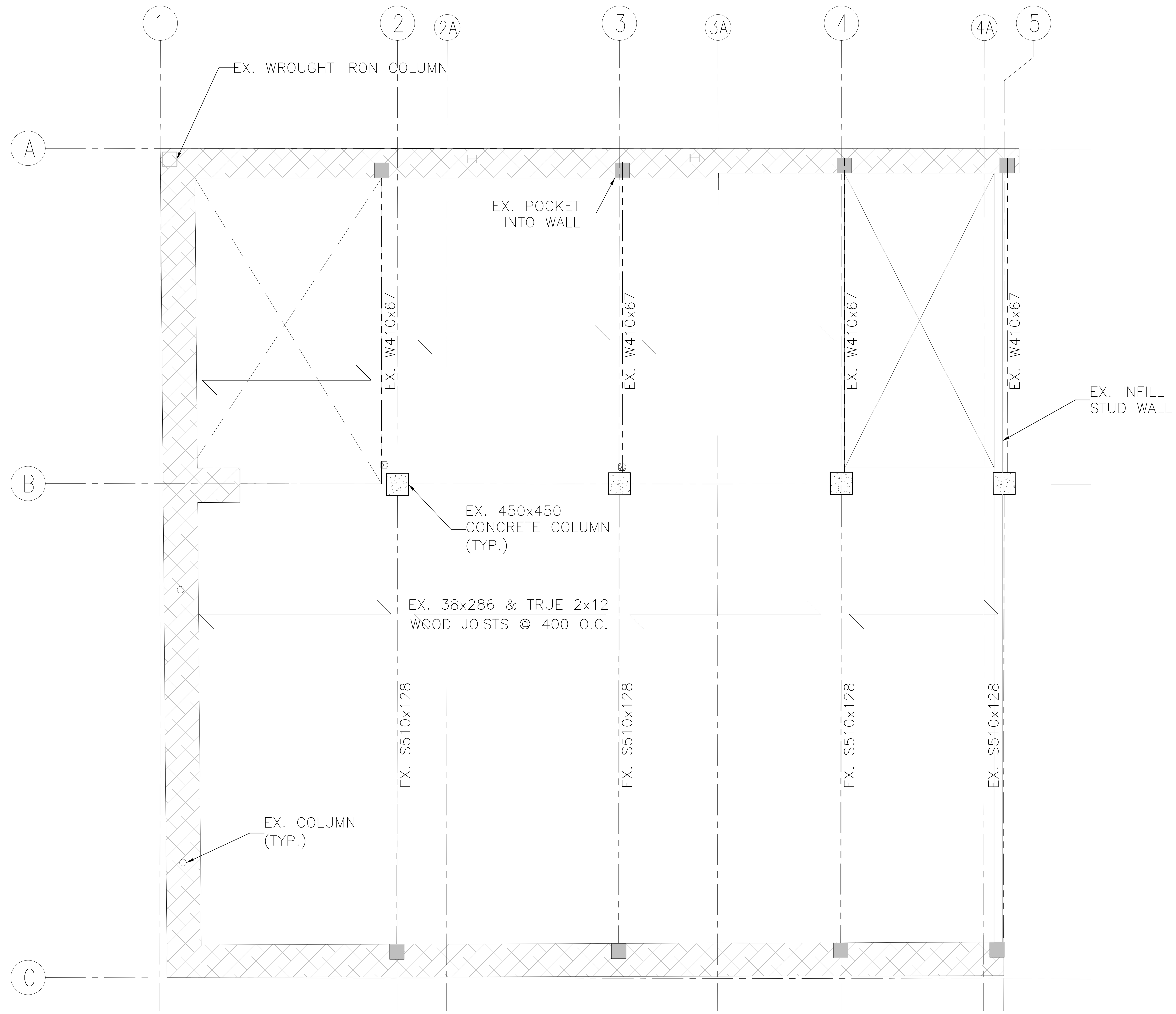
	T.B.		

date	project number	drawing number
February 6, 2023	0653	AB–1



client

T.K.S. HOLDINGS INC.

project

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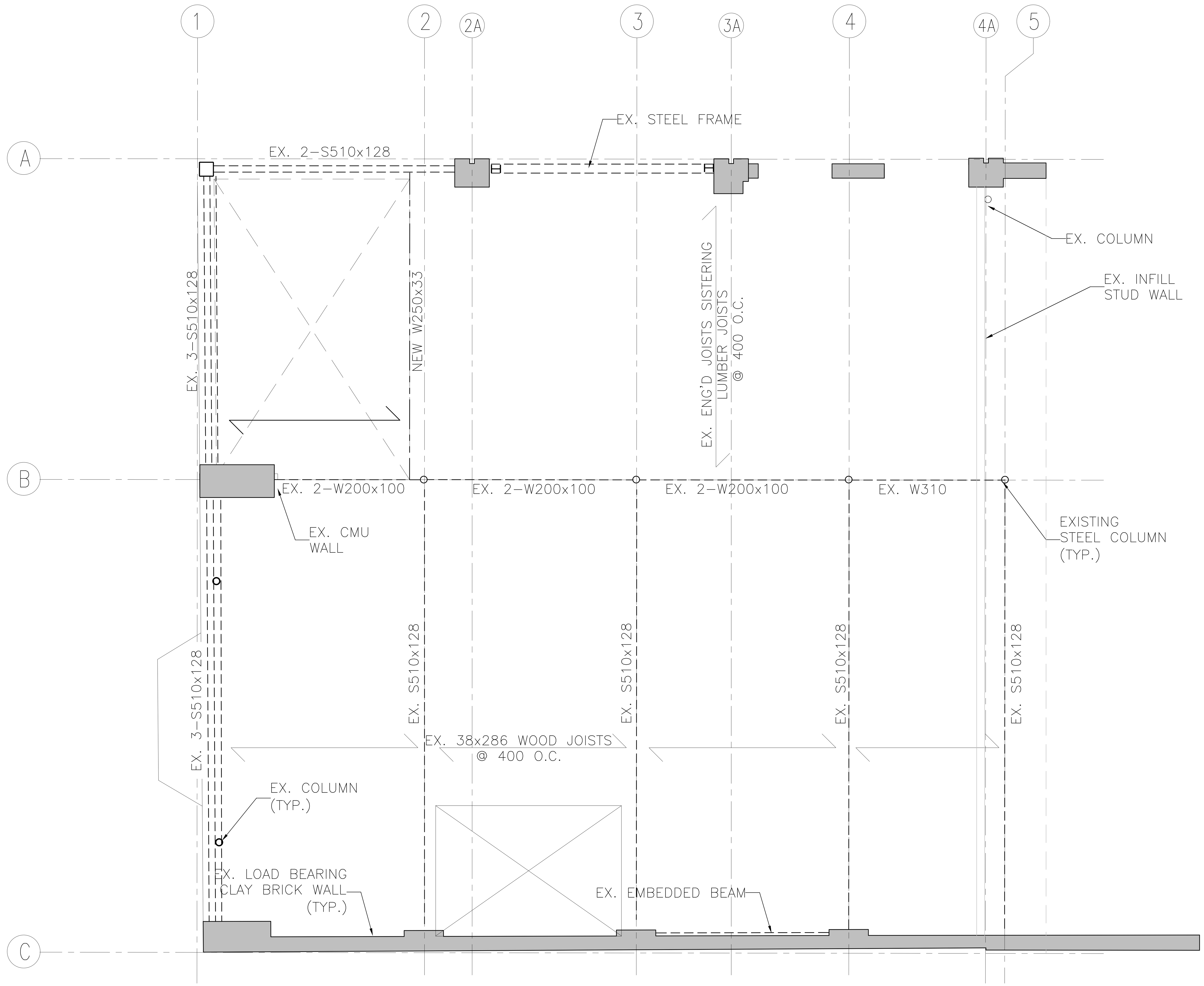

client

T.K.S. HOLDINGS INC.

project

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A detail no.
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B location drawing no.
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0	AS–BUILT	06–02–2023

drawing

SECOND

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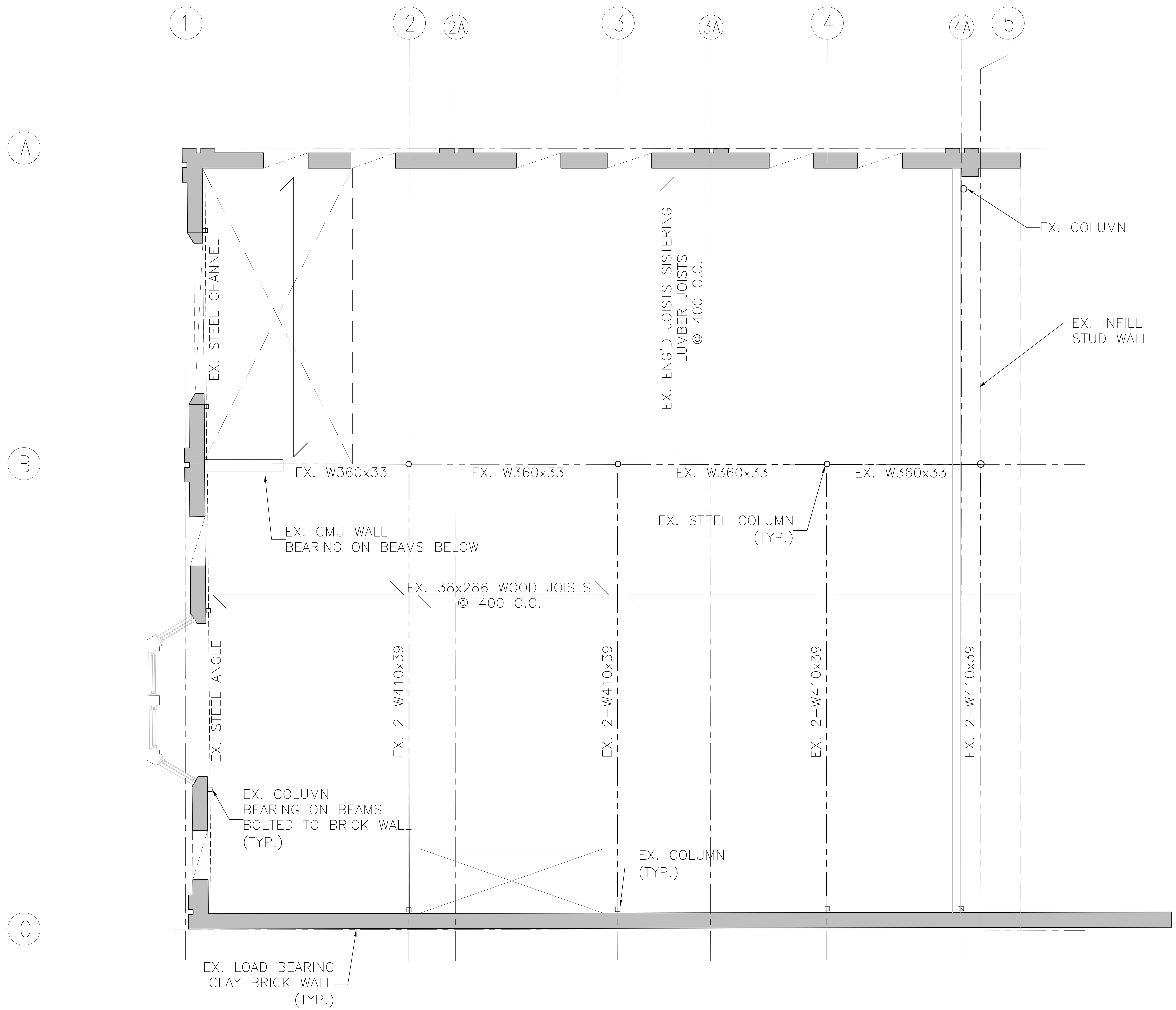
	T.B.		

date	project number	drawing number
February 6, 2023	0653	AB–3

ANSI D (22.00" x 34.00")



client

T.K.S. HOLDINGS INC.

project

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A detail no.
no. de détail
B location drawing no.
sur dessin no.

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revisions		date
0	AS-BUILT	06-02-2023

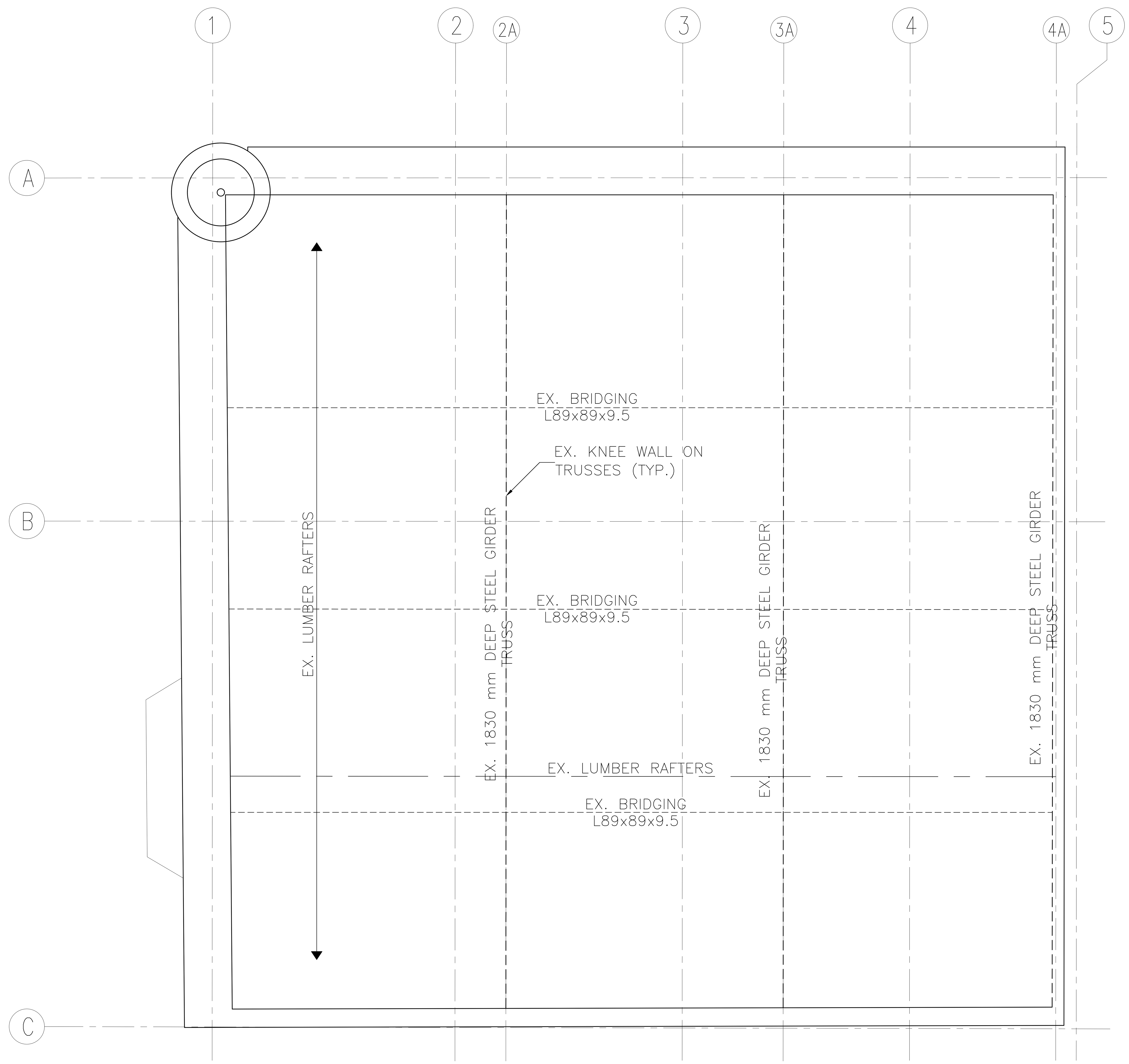
drawing

THIRD

stamp

designed	drawn	reviewed	approved
	T.B.		

date	project number	drawing number
February 6, 2023	0653	AB-4



client

T.K.S. HOLDINGS INC.

project

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A detail no. / no. de détail
B location drawing no. / sur dessin no.

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revisions		date
0	AS–BUILT	06–02–2023

drawing

ROOF

stamp

designed	drawn	reviewed	approved
	T.B.		
date		project number	drawing number
February 6, 2023		0653	AB–5

Appendix D: Material Testing Report (Keller, 2023)

500-885 MEADOWLANDS DR.
OTTAWA, ONTARIO, K2C 3N2
TEL. 613-224-1594
FAX. 613-224-1642
www.kellerengineering.com

2023 01 20 1220687

Art Engineering
171 Walgreen Rd,
Carp, ON
K0A 1L0

Attn: Tim Berg, P.Eng.

352 SOMMERSET – IN-SITU MEASUREMENT OF MASONRY DEFORMABILITY PROPERTIES

TEST OBJECTIVES

To determine the average in-situ bed joint shear strength 352 Sommerset in Ottawa, ON, using ASTM C1197-20e1.

Our measurements were taken on December 22, 2023.

TEST RESULTS

Calculated Average Young's Modulus:

Location 1: 3rd Floor, North Elevation – **1049 MPa** – Early failure (gauge pressure 2.76 MPa)
Location 2: 2nd Floor, South Elevation – **726 MPa** – Early failure (gauge pressure 1.72 MPa)
Location 3: 1st Floor, South Elevation – **3220 MPa** – Point of failure (gauge pressure 3.79MPa)

TEST LOCATIONS

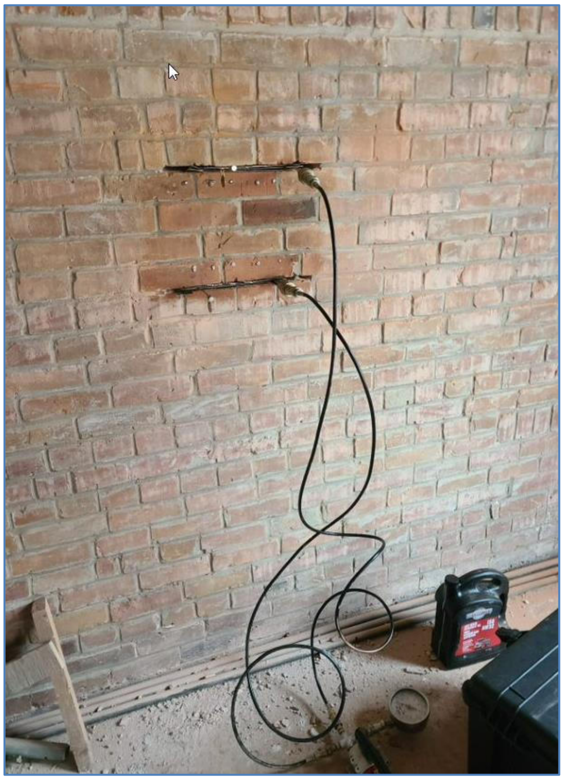
Testing was performed at locations assumed to be representative of the building. These locations are:

Location 1: 3rd Floor, North Elevation – Clear area between windows (4 gauges)
Location 2: 2nd Floor, South Elevation – Base of stairs (3 gauges)
Location 3: 1st Floor, South Elevation – Pier (2 gauges)

SHEAR STRENGTH TESTING

Testing Procedure

Two flat jack tests were performed at the locations indicated on Figure 1. Tests were performed in accordance with ASTM C1197-20e1 – Standard Test Method for In Situ Measurement of Masonry Deformability Properties Using the Flat Jack Method using circular flat jacks. The test arrangement is shown in Image 1. Load deformation measurements were taken during the loading process using a 200 mm Demec digital extensometer.



Img 1. Typical Aparatus assembly

Visual Observations

Deformations of the masonry behaved as expected for the initial loading conditions. Brick units however began failing under load at low values, early into the tests. On release of the load, most joints within the test area cracked.

Limitations and Bias

The failure of the brick units and surrounding masonry so early into the loading has limited the amount of data points that are available during the test.

Results from laboratory investigations conducted on old brick masonry have shown that variations between tests may be as great as 24%. This variation may be considered to be within acceptable limits for old masonry.

Experimental and analytical investigations indicate that the in situ deformability test typically over estimate the average compressive modulus of the masonry by up to 15%.

Trusting the above satisfies your current requirements. Please feel free to contact us if you have any questions regarding the above.

Sincerely,

Steve Christison, P.Eng.

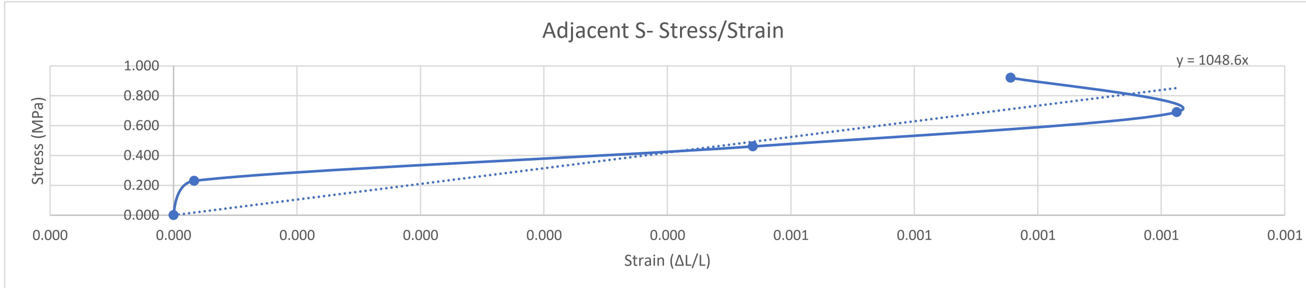
LICENSED PROFESSIONAL ENGINEER
S. D. CHRISTISON
100166147
2023 01 20
PROVINCE OF ONTARIO

APPENDIX A
DATA

ASTM C1197 - Test Results

Project	**1220687 - 352 Sommerset**
Date:	**2022-12-22**
Location:	**1 - 3rd Floor - North Elevation**
Tester:	SC

Km (Jacks) = 0.76
Ka = 0.44

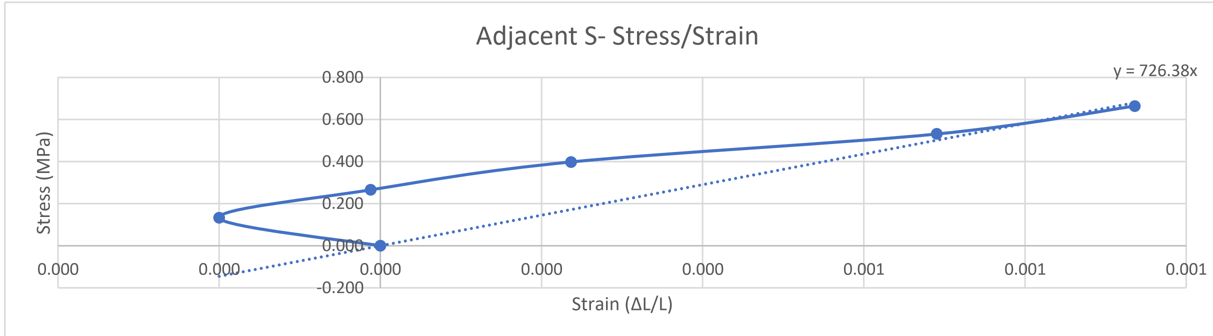


	Guage 1 Reading		Guage 2 Reading		Guage 3 Reading		Gauge 4 Reading		Guage 1 $\varepsilon_{m1}=(\Delta L/L)$	Guage 2 $\varepsilon_{m2}=(\Delta L/L)$	Guage 3 $\varepsilon_{m3}=(\Delta L/L)$	Gauge 4 $\varepsilon_{m3}=(\Delta L/L)$	Strain ε_{mAvg}	Stress $f_m=K_mK_ap$	Tangent Modulus $E_t=f_m/\varepsilon_m$
Initial Measurements	2.166 2.166 2.166	2.166	2.496 2.496 2.500	2.497	2.165 2.165 2.165	2.165	2.436 2.432 2.432	2.433					0.000	0.000	
p= 100 psi = 0.69 MPa	2.134 2.139 2.134	2.136	2.498 2.498 2.498	2.498	2.175 2.173 2.173	2.174	2.434 2.443 2.446	2.441	0.000152	-0.000003	-0.000043	-0.000038	0.00002	0.230	13814.093
p= 200 psi = 1.38 MPa	1.953 1.953 1.953	1.953	2.355 2.355 2.356	2.355	2.159 2.159 2.159	2.159	2.419 2.419 2.419	2.419	0.001065	0.000710	0.000030	0.000072	0.00047	0.460	981.463
p= 300 psi = 2.07 MPa	1.852 1.853 1.852	1.852	2.287 2.287 2.285	2.286	2.139 2.139 2.139	2.139	2.334 2.334 2.334	2.334	0.001568	0.001055	0.000130	0.000497	0.00081	0.691	850.098
p= 400 psi = 2.76 MPa	1.776 1.776 1.776	1.776	2.257 2.255 2.255	2.256	2.436 2.432 2.432	2.433	2.253 2.255 2.255	2.254	0.001950	0.001209	-0.001342	0.000895	0.00068	0.921	1358.068
p= 500 psi = 3.45 MPa															

ASTM C1197 - Test Results

Project **1220687 - 352 Sommerset**
Date: **2022-12-22**
Location: **2 - 2nd Floor - South Elevation**
Tester: SC

Km (Jacks) = 0.76
Ka = 0.51

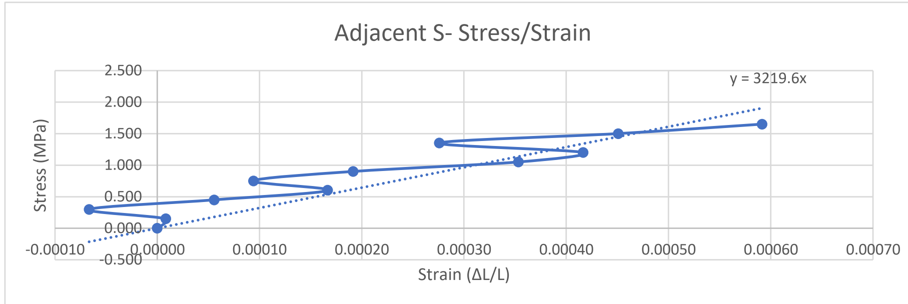


	Guage 1 Reading		Guage 2 Reading		Guage 3 Reading		Guage 1 $\varepsilon_{m1}=(\Delta L/L)$	Guage 2 $\varepsilon_{m2}=(\Delta L/L)$	Guage 3 $\varepsilon_{m3}=(\Delta L/L)$	Strain ε_{mAvg}	Stress $f_m=K_mK_ap$	Tangent Modulus $E_t=f_m/\varepsilon_m$
	-2.799		-2.705		-2.466							
Initial Measurements	-2.797	-2.796	-2.705	-2.705	-2.466	-2.466				0.000	0.000	
	-2.793		-2.705		-2.466							
	-2.702		-2.683		-2.461							
p= 50 psi = 0.34 MPa	-2.702	-2.702	-2.684	-2.684	-2.461	-2.462	-0.00047	-0.00011	-0.00002	0.000	0.133	-663.923
	-2.702		-2.684		-2.463							
	-2.763		-2.705		-2.491							
p= 100 psi = 0.69 MPa	-2.765	-2.763	-2.705	-2.705	-2.493	-2.492	-0.00016	0.00000	0.00013	0.000	0.266	-21728.396
	-2.762		-2.705		-2.491							
	-2.815		-2.755		-2.544							
p= 150 psi = 1.03 MPa	-2.808	-2.810	-2.755	-2.755	-2.544	-2.544	0.00007	0.00025	0.00039	0.000	0.398	1683.186
	-2.808		-2.755		-2.544							
	-2.890		-2.860		-2.635							
p= 200 psi = 1.38 MPa	-2.890	-2.890	-2.860	-2.860	-2.630	-2.632	0.00047	0.00077	0.00083	0.001	0.531	769.147
	-2.890		-2.860		-2.630							
	-2.959		-2.908		-2.662							
p= 250 psi = 1.72 MPa	-2.959	-2.959	-2.908	-2.908	-2.662	-2.662	0.00081	0.00102	0.00098	0.001	0.664	709.235
	-2.959		-2.908		-2.662							
p= 300 psi = 2.07 MPa												

ASTM C1197 - Test Results

Project	**1220687 - 352 Sommerset**
Date:	**2022-12-22**
Location:	**1st Floor - South Elevation - Pier**
Tester:	SC

Km (Jacks) = 0.80
Ka = 0.55
L = 200



	Guage 1		Guage 2		Guage 1	Guage 2	Strain	Stress	Tangent Modulus
	Reading		Reading		$\varepsilon_{m2}=(\Delta L/L)$	$\varepsilon_{m3}=(\Delta L/L)$	ε_{mAvg}	$f_m=K_mK_ap$	$E_t=f_m/\varepsilon_m$
	-2.250		-2.455						
Initial Measurements	-2.250	-2.250	-2.450	-2.452			0.00000	0.000	
	-2.250		-2.450						
	-2.230		-2.475						
p= 50 psi = 0.34 MPa	-2.230	-2.230	-2.475	-2.475	-0.000100	0.000117	0.00001	0.150	18015.236
	-2.230		-2.475						
	-2.278		-2.397						
p= 100 psi = 0.69 MPa	-2.278	-2.278	-2.397	-2.397	0.000140	-0.000273	-0.00007	0.300	-4503.809
	-2.278		-2.397						
	-2.298		-2.426						
p= 150 psi = 1.03 MPa	-2.298	-2.298	-2.426	-2.426	0.000240	-0.000128	0.00006	0.450	8066.524
	-2.298		-2.426						
	n/a		-2.485						
p= 200 psi = 1.38 MPa	n/a	#DIV/0!	-2.485	-2.485	#DIV/0!	0.000167	0.00017	0.601	3603.047
	n/a		-2.485						
	-2.311		-2.430						
p= 250 psi = 1.72 MPa	-2.311	-2.311	-2.425	-2.428	0.000305	-0.000117	0.00009	0.751	7971.344
	-2.311		-2.430						
	-2.330		-2.445						
p= 300 psi = 2.07 MPa	-2.330	-2.330	-2.450	-2.448	0.000400	-0.000017	0.00019	0.901	4699.627
	-2.330		-2.450						
	-2.355		-2.488						
p= 350 psi = 2.41 MPa	-2.355	-2.355	-2.488	-2.488	0.000525	0.000182	0.00035	1.051	2974.214
	-2.355		-2.488						
	-2.347		-2.524						
p= 400 psi = 2.76 MPa	-2.349	-2.348	-2.517	-2.520	0.000490	0.000343	0.00042	1.201	2882.438
	-2.348		-2.520						
	-2.376		-2.440						
p= 450 psi = 3.10 MPa	-2.370	-2.372	-2.440	-2.440	0.000610	-0.000058	0.00028	1.351	4898.403
	-2.370		-2.440						
	-2.376		-2.506						
p= 500 psi = 3.45 MPa	-2.376	-2.376	-2.506	-2.506	0.000630	0.000272	0.00045	1.501	3329.988
	-2.376		-2.506						
	-2.400		-2.525						
p= 550 psi = 3.79 MPa	-2.420	-2.413	-2.525	-2.525	0.000817	0.000367	0.00059	1.651	2791.093
	-2.420		-2.525						
p= 600 psi = 4.14 MPa									

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Art Engineering
171 Walgreen Rd,
Carp, ON
K0A 1L0

Attn: Tim Berg, P.Eng

352 SOMMERSET – IN-SITU MEASUREMENT OF MASONRY MORTAR JOINT SHEAR STRENGTH INDEX

TEST OBJECTIVES

To determine the average in-situ bed joint shear strength 352 Sommerset in Ottawa, ON, using Method C of ASTM C1531-15.

Our measurements were taken on December 22, 2023.

TEST RESULTS

In-situ shear strength indices (τ_o) using coefficient of friction for masonry (μ) as 0.3 -1.6

Location 1: 3rd Floor, North Elevation – **540 kPa - 590 kPa** (estimated normal compressive stress (σ_v): 39 kPa)

Location 2: 2nd Floor, South Elevation – **170 kPa - 282 kPa** (estimated normal compressive stress (σ_v): 86 kPa)

TEST LOCATIONS

Testing was performed at locations assumed to be representative of the building. These locations are:

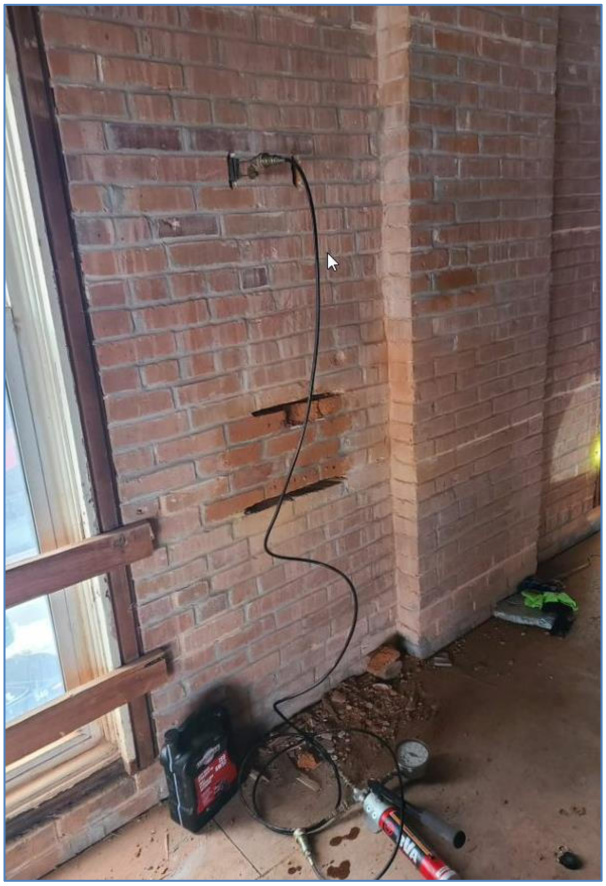
Location 1: 3rd Floor, North Elevation – Clear area between windows
Location 2: 2nd Floor, South Elevation – West of stairs

SHEAR STRENGTH TESTING

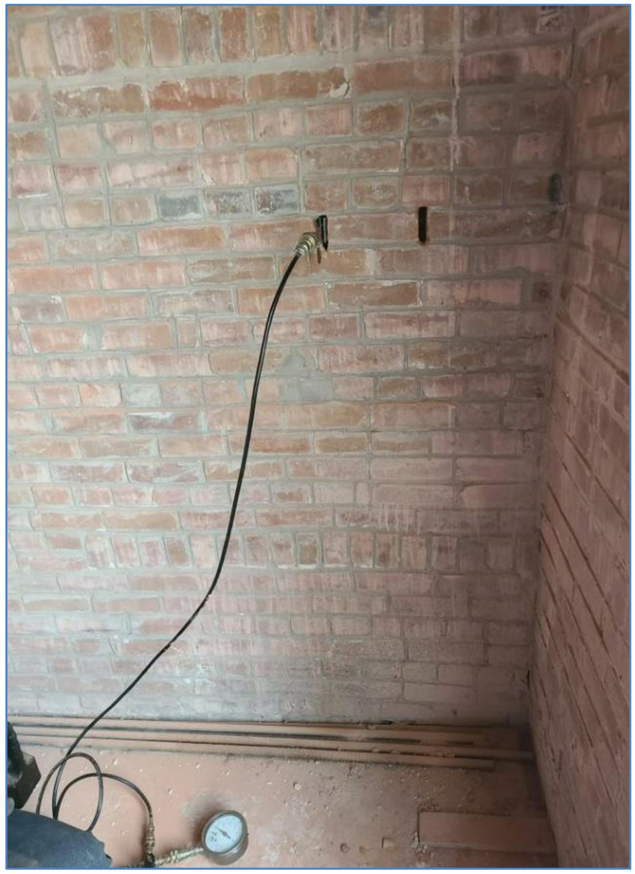
Testing Procedure

At each test location, the head joint mortar was removed on each side of the masonry unit to be tested. Special care was taken not to disturb the bed joint mortar bond. 2 flat jack tests were performed.

Testing was performed in conformance with Method C of ASTM C1531-15 – Standard Test Method for In Situ Measurement of Masonry Mortar Joint Shear Strength Index using rectangular flat jacks. The test arrangement is shown in Image 1. During the load application, close-up observations were made to identify the maximum shear load at the initial mortar bond failure



Img 1. Test Location 1



Img. 2. Test Location 2

Visual Observations

Cracks were observed at the bed joints of Locations 1 and 2 on failure at gauge pressure 5.86 MPa and 3.1 MPa, respectively.

Limitations and Bias

Insufficient data exists to correlate the joint shear strength index measured with the in situ test to the actual shear strength index of the masonry. In situ measurement of bed joint shear strength and coefficient of friction may be affected by workmanship, the quality of the collar joint an the inaccuracies in determining normal compressive stress, whether estimated or controlled during testing using flatjacks

Laboratory studies have shown that the in-situ bed joint shear strength index test will generally overestimate the actual shear strength index of a wall panel; however, insufficient data currently exists to provide a reliable bias statement.

Trusting the above satisfies your current requirements. Please feel free to contact us if you have any questions regarding the above.

Sincerely,

Steve Christison, P.Eng.

LICENSED PROFESSIONAL ENGINEER
S. D. CHRISTISON
100166147
2023 01 20
PROVINCE OF ONTARIO

Appendix E: 2017 Design Intent Report (Art, 2017)

art engineering inc.
171 Walgreen Road • Carp • Ontario • K0A 1L0 • Canada
(613) 836-0632 • Fax: (613) 836-1226
www.artengineering.ca Project No: 0653

352 Somerset Street (Somerset House) Design Intent Document

DATE: **August 08, 2017**
Client/Owner: **T.K.S. Holdings Inc.**
Architect: **Rick Kemp - Chmiel Architects**
Attention: **Elizabeth Kisilewicz, P. Eng., Building Code Engineer**

ART Engineering Inc. (AEI) has been retained by T.K.S. Holdings Inc. as the structural engineer of record for the rehabilitation and reconstruction of 352 Somerset Street, located in Ottawa, Ontario. The purpose of the following document is to outline AEI's intended design approach and methodology during the upcoming permitting process, specifically from a seismic design perspective, in order to ensure the structural rehabilitation and reconstruction of Somerset House is in full compliance with all relevant sections of the latest applicable Ontario Building Code (OBC 2012).

It should be noted that all preliminary calculations enclosed in this report represent the best judgment of AEI at the time of preparation and are solely for developing a general rehabilitation design philosophy and determining feasibility of the proposed approach. Additional refined methods of analysis and field investigations/testing will be conducted while preparing the "For Permit" drawings to further develop the design in a more detailed manner, as discussed in a later section of this report.

General Background (Current):

Located at the corner of Bank Street and Somerset Street West, the existing structure at 352 Somerset St. W. currently neighbours a parking lot along the east side of the property limits, Somerset Street West along the north, Bank Street along the west and an adjacent structure along the south. The original corner building of Somerset House was built in 1897-1899 (west wing), while the eastern wing of the building was added in the early 1900's. AEI was previously retained by T.K.S Holdings in 2007 to perform an extensive field investigation and produce as-built structural drawings of the (previously) existing conditions, prior to the partial collapse of the structure's eastern section, along with a portion of the original western wing, in October of 2007. Following the partial collapse, the remaining northeast brick wall was temporarily supported with a steel frame shoring system. Both the northeast brick wall and shoring system have since been removed. The current remaining portion of the original structure, which extends approximately 17000 mm along the north/south faces, consists of three bays and three storeys above ground, in addition to one basement level below grade. If the top of ground subfloor level is taken at a reference elevation of 0.000 m, the respective elevation of the floors are as follows:

Top of basement level: -3.700 m +/-;
Top of ground level subfloor: 0.000 m +/-;
Top of second level subfloor: 4.300 m +/-;
Top of third level subfloor: 8.310 m +/-;
Underside of riveted roof trusses: 11.41 m +/-;
Underside of wooden roof deck: 13.98 m +/-.

The remaining structure generally consists of rubble stone masonry foundation walls below grade along the exterior building perimeter (with the exception of the east face) and load bearing clay brick masonry walls bearing on the foundation walls above grade. The floor framing for all levels, with the exception of the roof, generally consists of wooden floor joists (new/recent) supported on wrought iron and steel beams. Cast in place concrete columns support level (1), cast iron columns extend between the ground floor and the underside of level 2, while steel columns support level (3). Riveted, wrought iron roof trusses complete with wood decking (original construction) form the roof support system. Temporary lumber stud walls have been installed along the east face of the structure on all levels (basement to roof), to provide weather protection. Salamander heaters have been installed in the basement which regulate the temperature during the winter. A more detailed description of the existing structural system can be found in the "Structural Field Review", dated June 23, 2017, prepared by AEI.

It should be noted that the current available lateral load resisting system is comprised of the exterior brick masonry walls. The north and west brick masonry walls on the first floor were originally built with large openings, as was common practice at the time of construction for similar structures, resulting in a soft storey and vertical irregularity effects, as defined in today's modern building code, OBC 2012 table 4.1.8.6. The current configuration of the masonry brick walls is as follows:

Main Floor (Wall Between Top of Ground and Underside of Level 2):

North Wall: Opening along the west corner [+/-4500 mm (W) x +/-3100 mm (H) to u/s steel], currently infilled with concrete masonry units (CMUs); opening in mid wall section [+/-4800 mm (W) x +/-3100 mm (H) to u/s steel], supported by a steel lintel and HSS posts; an arch window opening along the east corner [+/-1700 mm (W) x +/-3100 mm (H)]; percentage of wall length along north face with current and previously existing openings, excluding the northwest cast iron column [65%].

West Wall: Opening along the south side [+/- 8850 mm (W) x +/-3100 mm (H) to u/s steel], currently infilled with lumber studs and supported by original wrought iron lintels and circular columns/brick masonry pier; an opening along the north side [+/- 6200 mm (W) x +/-3100 mm (H) to u/s steel] supported by original wrought iron lintels bearing on existing corner iron column/brick masonry pier and infilled with lumber to protect against the elements; percentage of wall length along the west face with current and previously existing openings, excluding the northwest cast iron column [90%]. The west wall at the north corner is not connected to the floor diaphragms throughout due to an existing opening.

South Wall: Previously existing opening located near the east end [+/- 3700 mm (W) x +/- 3700 mm (H)], currently infilled with clay brick; percentage of wall length along south face with current and previously existing openings [21%].

East Wall: Non-existent, weather enclosure lumber stud wall currently erected.

Second Floor (Wall Between Top of Level 2 and Underside of Level 3):

North Wall: Two openings in west, middle and east bays, each measuring [+/- 1000 mm (W) x +/- 2000 mm (H)] to u/s floor], supported by arch brick lintels; percentage of wall length along north face with current and previously existing openings [35%].

West Wall: Two large window bay openings each measuring [+/-3700 mm (W) x +/-6300 mm (H) extending over 2 storeys], supported by arch brick lintels on the third floor; two smaller window openings also exist each measuring [+/-1100 mm (W) x +/-1850 mm (H)], supported by arch brick lintels on the second floor; percentage of wall length along the west face with current and previously existing openings [57%].

South Wall: No openings.

East Wall: Non-existent, weather enclosure lumber stud wall currently erected.

Third Floor (Wall Between Top of Level 3 and Underside of Roof):

North Wall: Two openings in west, middle and east bays, each measuring [+/- 1000 mm (W) x +/- 2000 mm (H)], supported by arch brick lintels; percentage of wall length along north face with current and previously existing openings [35%].

West Wall: Two large window bay openings each measuring [+/-3700 mm (W) x +/-3200 mm (H) above the third floor], supported by arch brick lintels; two smaller window openings also exist each measuring [+/-1100 mm (W) x +/-1850 mm (H)], supported by arch brick lintels on the third floor; percentage of wall length along the west face with current and previously existing openings [57%].

South Wall: No openings.

East Wall: Non-existent, weather enclosure lumber stud wall currently erected.

An engineering survey of the load bearing clay brick walls along Bank Street and Somerset Street West has been conducted by AEI's survey team*.* The survey outlines out of plane position at each level with respect to the second floor, indicating no current major out of plane movement of the walls, considering the age of the structure.

General Rehabilitation Intention:

The intent of the design and subsequent work to be carried out includes the rehabilitation and upgrade of the remaining heritage structure at 352 Somerset (western portion), along with the design and construction of a new structure to replace the previously existing (collapsed) eastern portion. The intent is to separate both structures in accordance with clause 4.1.8.14 (1) of OBC 2012 by the square root of the sum of the squares of each structure's individual lateral deflection, calculated in accordance with clause 4.1.8.13 (1) from a linear elastic analysis (including torsion) and magnified by RdR_o to account for deformations due to system ductility and over strength .

The new structure to be constructed in the eastern portion of the property limits will be designed in accordance with OBC 2012 part 4 for both gravity and lateral loads. Once the intended use and occupancy has been determined, along with the preferred structural system, details will be provided in the drawings issued "For Permit". Ductility and over strength related force modification factors will be provided in accordance with table 4.1.8.9 of OBC 2012 to calculate the actual storey drifts/lateral deflection and provide sufficient separation to prevent the risk of seismic pounding (elastic deflections to be magnified by ductility and over strength factors).

Given the age and heritage nature of the remaining portion of 352 Somerset, it would be economically unfeasible to detail the structure's rehabilitation to meet the full requirements of the current building code (part 4), namely from a seismic perspective. Due to the brittle nature of unreinforced masonry, the building would be required to remain fully elastic under earthquake loading according to section 4.1.8 of OBC 2012. In addition, the structure adjacent to 352 Somerset on the south end has been constructed against the south brick wall, and separation per clause 4.1.8.14 has not been provided. If the structure at 352 Somerset is required to meet the full demand requirements of section 4.1.8 of OBC 2012, additional stiffness and strength improvements measures would also be required to account for the mass and stiffness of the adjacent existing structure. It is in AEI's opinion that part 11 of OBC 2012 is intended to facilitate the rehabilitation and preservation of heritage structures, without undue financial hardships, thereby encouraging owners to provide performance level improvements and ultimately extending the life of heritage structures. Section 11.5 of OBC 2012 provides compliance alternatives where the chief building official is satisfied that compliance with other noted sections of the code is impractical due to (a) structural or construction difficulties, or; b) detrimental to the preservation of a heritage building. It is AEI's opinion that applying the full requirements of section 4.1.8 of OBC 2012 would both be impractical and detrimental to the heritage nature of 352 Somerset. As such, the intention is to request relief of full compliance to section 4.1.8, as indicated in Table 11.5.1.1.D/E (Compliance Alternatives for Business/Mercantile Occupancies) from the chief building official, while maintaining or exceeding the original structure's seismic performance level as detailed in later sections of this report. Full wind loads, gravity loads (dead, live, snow) and other loads (permanent horizontal earth, ground settlement, temperature changes, etc..) will be used to upgrade the structure in accordance with the requirements OBC part 4.

Original Conditions:

In order to ensure that the rehabilitated structure will at least maintain or exceed the original seismic performance level, the original conditions of the structure must be considered for comparative and evaluation purposes. For the remaining portion of this report, the original structure refers to the portion of 352 Somerset built in the late 1800s, excluding the addition (early 1900s).

The openings along the north and west faces of the structure are part of the original construction, as indicated by available documentation from 1920 (***Appendix A***). As such, a weak storey effect and vertical irregularities (defined in Table 4.1.8.6 of OBC 2012), in both principal directions of the SFRS system (clay brick exterior masonry walls), have existed as part of the structural system since original construction. It is widely agreed upon that the presence of a weak storey effect, particularly on the main floor, is unfavorable during a seismic event. As such, composite steel bracing (HSS filled with concrete) directly connected to the diaphragm and masonry walls will be provided to increase the original performance level. Additional details regarding the methodology behind the design of these braces are provided in the ***Original and Proposed Structure Capacities*** and ***Proposed Preliminary Changes*** sections.

The structure's flooring system, in its original as-built condition, consisted of rough-cut heavy timber joists and diagonal ship lap planking. The flooring system was not adequately connected to the masonry walls as outlined from pictures obtained in 2007 (***Appendix A***). The flooring system was also built with a large opening at the northwest corner which provided virtually no out-of-plane restraint, as evidenced by the previous out of plane movement of the west wall (attached photographs from 2007 in ***Appendix A***). Without adequate connections to the diaphragms, the masonry walls are vulnerable to out of plane failure as detailed in the "Seismic Design Guide for Masonry Buildings - Chapter 2" explanatory notes of CSA S304.1-14. The original flooring system had also experienced fire damage as indicated in pictures obtained from 2007 (***Appendix A***). Suggested diaphragm upgrades are discussed in the ***Proposed Preliminary Changes*** section.

Given the age of the 352 Somerset structure, and the common practices at the time of construction, clay brick partition walls, similar to what is currently present, were used as outlined from pictures obtained in 2007 (***Appendix A***). The use of such partition walls, which bear on steel beams, does not provide any structural performance gain and only adds to the overall weight and therefore inertial forces for seismic design. These walls have since been removed, thereby enhancing the seismic performance level since original construction by reducing inertia forces. It should also be noted that due to the large movement of flexible and semi-flexible diaphragms relative to the unreinforced masonry walls, early failure of the heavy partitions and subsequent failure of the flooring system has been noted in similar structures.

The previously existing east wall (part of the original construction in the late 1800s) later became a party wall following the addition of the eastern wing in the early 1900s. The eastern addition, a four storey structure with the same overall height as the structure built in the late 1800s, had lower floor heights which framed into the east party wall. This configuration resulted in floor misalignment, which is usually unfavorable and a cause for out of plane inter-storey shear failure. The east party wall and a portion of the north and south walls have since been demolished, however documentation and an as-built drawing outlining the configuration prior to collapse are available and provided in ***Appendix A***. Openings between the eastern and western wings were added and extended on all levels, including the basement. Since the eastern wall has been removed, the effects of doing so on the overall original seismic performance level should be investigated to ensure that the new construction and structural system has a performance level meeting or exceeding the original configuration. Details for the methodology behind obtaining original seismic capacity are outlined in the ***Original and Proposed Structure Capacities*** and ***Proposed Preliminary Changes*** sections. The exterior west masonry wall does not appear to have been altered.

Brick Piers previously existed in the basement and were used to carry gravity loads to the founding level. Pictures showing the brick piers have been included in ***Appendix A***. These piers have since been replaced with concrete columns doweled into a raft slab. It should be noted that at the time of replacement, available documentation indicated that the brick piers had experienced significant deterioration and mortar leaching due to moisture infiltration. The brick piers have since been replaced with 400x400 concrete columns having a lower overall mass.

The foundation walls consist of stone rubble masonry extending the depth of the basement level and dating back to the time of original construction. Portions of the north and west foundation walls still require repointing and resetting which will be addressed in the "For Permit" drawings. The exact original configuration of the ground level flooring system (framed to the top of the foundation walls) is unknown, since documentation or picture evidence of the original flooring system is not available. It is assumed that the flooring system also consisted of heavy timber and ship lap planking supported on wrought iron beams, similar to the original configuration of other levels. The original framing orientation is unknown, however the current ground flooring system will be modified to provide lateral support at the top of all existing foundation walls.

The original (and current) roofing system consists of wrought iron riveted trusses spanning north-south complete with lumber joists and planks. Timber spanning the east-west direction between the bottom chords of the trusses existed previously, as indicated in photographs from 2007 outlined in ***Appendix A.*** The timber has since been replaced with steel angle cross bracing fastened to a steel angle continuously bolted into the top of the west wall.

Noted Changes to Original Structure:

The existing structure at 352 Somerset has experienced the following changes from its original condition:

- New flooring systems consisting of modern sawn lumber and engineered joists, complete with 19 mm sheathing have been installed on all levels. Joists have been grouted into the unreinforced exterior masonry walls or fastened to a perimeter angle in some locations. Some upgrades are still required to the wooden diaphragms to increase in plane capacity and provide additional out of plane restraint, however they are a net improvement to the original system in their current configuration.
- Pattress plates have been installed locally along the edge of the existing north wall, providing additional out of plane restraint.
- Interior brick partition walls have been removed, reducing the overall inertia forces and potential for failure due to the movement of the wood diaphragms during a seismic event.
- The original brick piers in the basement have been replaced with reinforced concrete columns.
- The load bearing brick walls have been partially repointed and the west wall has been re-aligned. Additional masonry rehabilitation work and testing is required, which will be outlined in the "For Permit" drawings.
- The east party wall and a portion of the north and south walls and associated flooring/roof have been removed. The seismic mass of the structure has decreased (65% of original mass remains); however, the associated stiffness and capacity of these wall portions have also been removed from the system.
- Steel angle bracing has been provided to the west wall at the roof level in lieu of the original timber rafters.
- A partial raft slab and footing underpinning have been constructed, which will be structurally assessed and completed upon approval of the "For Permit" drawings.

Some of the changes noted above are considered a net improvement on the structure's performance level from a seismic point of view, while items such as the removal of the east wall are considered to have a negative impact on the performance level. In an effort to ensure that the performance level of the rehabilitated structure is an all around net improvement to the original conditions, various performance level criteria of the original configuration have been assessed and compared to the proposed preliminary rehabilitated state, described in the sections below.

Original and Proposed Structure Capacities:

In order to ensure the performance level of the rehabilitated heritage structure at 352 Somerset meets or exceeds the original as-built performance level from a seismic perspective, the capacity of the original structure has been determined in accordance with OBC 2012, as summarized in this section.

Two approaches have been used to determine the capacity of the original structure, including manual calculations and a non-linear finite element model accounting for the compression, tension and shear stress limits of unreinforced masonry, and the associated stress redistribution after local failure (an upper limit analysis to determine absolute failure limits). A similar model has also been used for the preliminary sizing of bracing members to ensure stiffness compatibility between structural elements (i.e. interaction between steel bracing and masonry) and a desired stress redistribution profile. As noted in the calculations and throughout this section, the calculated original capacity has purposely been exaggerated to provide a conservative seismic design value that will be used for the design of the rehabilitation of 352 Somerset, in addition to all other loading criteria outlined in part 4 of OBC 2012.

The equivalent static force procedure has been used in accordance with section 4.1.8.7 (C) for both the original and proposed preliminary structure rehabilitation. Detailed seismic mass calculations are provided in ***Appendix B***, which have been verified against the FEM model. The model has in turn also been calibrated to account for additional masses not explicitly forming part of the structural system. The east party wall, which had numerous openings as indicated in the available as-built documentation, has been treated as a solid wall for stiffness and capacity contribution, but its seismic mass has been reduced to account for the openings. The current structure has a seismic weight of 5300 kN and a total base shear of 2995 kN, while the original structure had a seismic weight of 8200 kN and a total base shear of 4635 kN (35% reduction based on the same fundamental period). The fundamental period of the structure in both cases (original vs rehabilitated) has been taken as 0.36 seconds in both directions, since OBC 2012 does not permit the use of a higher period under section 4.1.8.11. It should be noted that the introduction of steel bracing in the north-south direction, and the removal of the east wall, results in an increase in the calculated fundamental period due to the increase in system flexibility, which is considered a net improvement from a seismic perspective. This increase in fundamental period has conservatively not been accounted for in the calculations presented in ***Appendix B***.

In determining the capacity of the original structure, the following assumptions and parameters have been used:

- Per CSA 086 and ASCE7-16 clauses 12.3.1.1 to 12.3.1.3, wood diaphragms bearing on load bearing masonry structures are treated as flexible diaphragms. For the purpose of calculating an exaggerated original capacity, a rigid diaphragm assumption has been used to purposely shift the loading from the west wall (soft storey) towards the east wall (assumed solid) as noted in ***Appendix B***. The discontinuity in the floor diaphragms at the northwest corner has been ignored.
- Torsional sensitivity has been neglected and the OBC 2012 required accidental eccentricity has been set to 0, as this created additional demand on the walls and it is conservative for the purpose of overestimating original capacity.
- The walls have been assessed based on an in-plane capacity analysis only, neglecting out of plane failure in the original structure configuration. Out of plane failure will be calculated and addressed in the "For Permit" drawings based on the design criteria outlined in this document.
- No inter-storey loads on the east wall due to the addition in the early 1900s have been assumed and inter-storey shear failure has been neglected in determining original capacity.
- High strength clay brick for the time of construction and full, unleached mortar strength model parameters have been used (i.e. assume not deteriorated). The clay brick compressive strength

has been taken as 20 MPa and a mortar design compressive strength of 4 Mpa have been used in the analysis.
- Material reduction factors in calculating the strength have been ignored (set equal to 1).
- Clay brick partition walls have not been included in the seismic mass and assumed to never have existed in the original structure.
- All loads distributed to each floor diaphragm have been distributed in proportion to the assigned floor mass and height, in accordance with OBC 4.1.8.11.
- The structural models have been calibrated and center of mass has been verified against manual calculations.
- non-linear compression only springs have been used to model the bearing resistance against the foundation walls and linear lateral springs have been used to model bed joint failure, neglecting any onset of uplift (tension or no compression) in reducing the bed joint sliding capacity.
- The actual fundamental period in both directions has been calculated in accordance with the dynamic analysis procedure requirements of OBC for comparative purposes and to ensure that the fundamental period is not decreased in the rehabilitated configuration.
- In accordance with OBC 4.1.8.3 (5), the columns would normally be required to have adequate capacity for supporting the floor loads under the drifts and loads associated with a design seismic event, without failure or the reliance on friction to do so. The original cast iron columns were not built with any positive attachment to the flooring system (mainly the wrought iron beams). This requirement has been neglected in determining the original structure's seismic capacity.

In determining the global capacity of the existing structure's proposed rehabilitated configuration (preliminary) and bracing, the ultimate base shear (distributed to each level) which would have caused the onset of failure in the original structure has been used to assess demand:capacity ratios. The following model parameters have been used:

- The wooden diaphragms behave as flexible diaphragms, similar to the original structure configuration. The diaphragms will be detailed and designed to have superior gravity and lateral load capacity than the original system, while ensuring shear force transfer around openings and providing continuous lateral restraint to the masonry walls. A preliminary analysis of the wooden diaphragm system deflection compared to the steel bracing indicates that a flexible diaphragm meeting the criteria of ASCE7-16 CL 12.3.1.3 is feasible.
- The mortar and brick compressive strengths are similar to that used in the original structure model, ignoring any upgrades which will take place.
- Brace forces are determined based on Rd & Ro factors of 1. Given the stiffness requirements of the bracing to provide meaningful engagement and load distribution characteristics compared to the existing masonry, the actual capacity of the bracing far exceeds the demand requirements obtained based on the original structure's capacity.
- Stiffness compatibility between masonry and braces has been accounted for. Anchorage and brace connection details will be outlined in the "For Permit" drawings.
- Bracing stiffness has been calibrated to provide an increased performance level in terms of storey shear capacity, while ensuring the fundamental period is not decreased in comparison to the original configuration.
- Accidental eccentricity associated with flexible diaphragms has been accounted for in determining bracing forces and ensuring the stresses induced in the existing masonry do not exceed the capacity limits, despite that the eccentricity was not accounted for in calculating the original structure's capacity.

The above noted parameters result in a total base shear capacity of 300 kN in the north/south direction and a total base shear capacity of 670 kN in the east/west direction for the original structure. As outlined in ***Appendix B*,** the seismic resistance capacity of the rehabilitated structure in both the east/west and north/south directions conservatively exceeds the original capacity, and is limited by the existing masonry walls, despite having an overall lower seismic weight to account for and a higher or equivalent fundamental period in both directions.

Proposed Preliminary Rehabilitation Measures:

The following proposed rehabilitation measures are based on the preliminary assessment results summarized in ***Appendix B:***

- Diaphragm improvements will be carried out, including the design of a ring beam to restrain the northwest corner on all levels. The diaphragm design will conform to section 4.1.8.15 of OBC 2012, with additional reserve capacity beyond the expected wall failure loads of 20% (conservative).
- Pattress plates will be provided, locations to be determined, in order to provide restraint to the north and south wall edges originally restrained by the east wall. A survey post installation will also be conducted to ensure wall plumbness within acceptable tolerances.
- Although it is typically favorable to detail braces and other SFRS elements to undergo ductile behavior during a seismic event to absorb energy, doing so in this scenario would not provide an advantage due to the brittle nature of the unreinforced masonry. Since the overall global capacity is limited by the existing masonry, which cannot sustain excessive deformations without failure, the braces will be steel bracing or steel concrete composite braces with adequate stiffness for compatibility with the adjacent masonry.
- Structural steel bracing will be provided along the east end of the existing structure at 352 Somerset St. The bracing will be detailed with sufficient stiffness such that the diaphragm is considered flexible, however the lower overall stiffness from the original east wall results in an increase in the structure's fundamental period (a net gain in seismic performance level). Diagonal bracing connections will be designed with additional reserve capacity beyond the expected diaphragm failure loads of 20%.
- The existing raft slab foundation will be extended to provide the required support and anchorage for the new steel braces and will be detailed in accordance with the requirements of CSA A23.3-14.
- The bracing currently provided to the west wall at the roof level will be assessed for adequacy against the outlined design criteria.
- All OBC 2012, part 4 loading criteria (other than full seismic requirements) will be used to assess the existing configuration. All proposed bracing will also be verified against wind loading criteria and the design may require iteration so that both seismic and wind loading design targets are satisfied.
- The corner turret and window bays will be replicated with lightweight material (likely light gauge steel), meeting the full requirements of OBC 2012 part 4.
- Column to beam positive connections will be detailed.
- The seismic separation gap between the existing structure and proposed new construction will be provided based on full design seismic forces, despite the existing structure's lack of capacity to sustain such forces.

It should be noted that with the above proposed upgrades, and any additional future recommendations proposed by AEI upon completion of the detailed site investigation, the rehabilitated structure will meet, and in most instances exceed, the original structure's seismic performance level, while fully complying to all other sections of OBC 2012 part 4.

Conclusion

As mentioned in the first section, the purpose of the design intent document is to outline the general design philosophy which will be used to develop the "For Permit" drawings. All preliminary calculations and sizing of members were completed to determine the feasibility of the proposed approach and upgrades, prior to preparation of the "For Permit" drawings, so that both the design team and The City of Ottawa are in general agreeance with the design intention. We trust that the above satisfies your requirements and addresses the comments previously issued in an email on March 14, 2017, with the comments and respective responses summarized below:

1. It shall be demonstrated that as a result of the work under the building permit, the performance level of the existing building has not been reduced.

   Response:
   Agreed. In our opinion, the design philosophy outlined in this document will result in a rehabilitated structure that, at a minimum meets and in most instances exceeds, all structural performance level criteria of the original structure.

2. Design principles of subsection 4.1.8 must be followed (RdRo values, distribution of lateral loads, connection design, etc.).

   Response:
   Agreed. The design principles of subsection 4.1.8 will be used and have been utilized in the preliminary calculations. As outlined in this design intent, a capacity based design approach will be used to ensure that the diaphragm and brace connections can sustain loads of 1.2 and 1.44 times the existing capacity of the masonry walls, respectively.

3. The consultant shall clarify how the lateral loads for the design of the new steel braces would be determined.

   Response:
   The design loads have been determined based on the exaggerated capacity of the original structure, to ensure that the capacity of the rehabilitated structure exceeds the original performance level. Once a conservative design base shear was determined, lateral loads were distributed to the walls based on the requirements of subsection 4.1.8. Loads were distributed to each line of SFRS based on a flexible diaphragm assumption and distributed to each element within the SFRS line based on stiffness.

4. Lateral forces would be transferred to the braces only if the braces are more rigid than the masonry walls, otherwise the braces would not be engaged in supporting lateral loads. Also, the weak storey at the ground floor would not be eliminated

   Response:
   Lateral loads will be transferred to the bracing despite the bracing being more flexible than the masonry walls, in a manner proportional to their respective stiffness. The bracing will not sustain more load than the existing masonry walls, however the stiffness of the braces adds to the overall capacity by limiting drift and therefore the loads imparted to the existing masonry walls. Stiffness compatibility has been accounted for in the preliminary design, resulting in braces that have a capacity far exceeding the demand, but such sections are required to ensure load sharing.

   The weak storey effect is part of the original configuration, and as such, forms part of the original performance level from a seismic perspective. Braces have been proposed to increase the shear capacity of the main floor and provide adequate means to resist wind loads and increase the overall seismic performance level considerably.

5. Compatibility between new braces and existing masonry walls shall be investigated and preliminary design of new braces carried out.

Response:

Agreed. Stiffness compatibility is possible to increase the performance level and the results are summarized in *Appendix B.*

6. Please comment on how the lateral loads would be distributed to the SFRS elements, proportional to the stiffness of these elements or proportionally to the tributary area. Justify.

   Response:

   The proposed wood diaphragms, which are similar to the original flooring system except having additional in plane capacity and restraining capabilities, can be treated as flexible diaphragms. Determination of what constitutes a flexible vs. rigid diaphragm depends on the in plane stiffness of the diaphragm relative to the in plane stiffness of the SFRS elements. Criteria/limits of when a diaphragm is considered flexible vs. rigid are not provided in OBC 2012, however guidance is available in CSA 086-14 (Engineering design in wood) and ASCE7-16 clauses 12.3.1.1 to 12.3.1.3. Both indicate that wooden diaphragms are to be treated as flexible elements, and ASCE7-16, clause 12.3.1.3 provides general guidance criteria for flexible diaphragms summarized as Δdiaphragm>= Δ(average) of SFRS elements, in order for the flexible diaphragm assumption to apply. Preliminary design calculations indicate that the braces along the east face can be designed with relative ease such that this criteria is satisfied.

   A semi rigid diaphragm assumption, having in plane stiffness inversely proportional to the expected diaphragm deflection, will also be used in preparing the "For Permit" drawings and is part of the additional refined analysis methods to be carried out. The redistribution of forces based on a semi-rigid diaphragm assumption is expected to alter the preliminary results slightly, but not in a manner to significantly impact the overall design approach.

7. ... For combination of different types of SFRS acting in the same direction in the same storey, Rd Ro shall be taken as the lowest of Rd Ro corresponding to these systems.

   Response:

   Agreed. Rd and Ro factors of 1 have been utilized in the analysis and preparation of the preliminary design. The buckling restrained brace frames and rods (proposed by others) have been eliminated in the design. Analysis results indicate that the rod bracing (tension only elements) would not contribute any significant capacity due to their relative flexibility compared to the masonry walls. The buckling restrained braces have also been replaced with a braced frame (potentially steel - concrete composite braces) and a capacity based design approach. In principal, it is favorable to have a ductile lateral resisting system capable of sustaining large deformations and absorbing energy, however, when combined with a brittle system (masonry), the benefits of such a system are not used. Instead, providing a braced system with critical elements, such as diaphragms and connections, purposely overdesigned results in a favorable behavior not compromising the original structure's seismic performance level.

Should you have any further questions, please do not hesitate to contact our office at (613) 836-0632.

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Appendix A

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Project No: 0653

Figure 1: Original Configuration (1920) Showing Openings

Figure 2: Previously Existing Condition Showing Eastern Addition

Figure 3: Previously Existing Condition Brick Partition Walls

Figure 4: Previously Existing Condition Flooring System and Fire Damage

Figure 5: Previously Existing Condition Flooring System and Perimeter Connections

Figure 6: Previously Existing Condition Flooring System and End Bearing Details

Figure 7: Previously Existing Condition Outlining Brick Piers in Basement

Figure 8: Previously Existing Condition Outlining Movement of West Wall

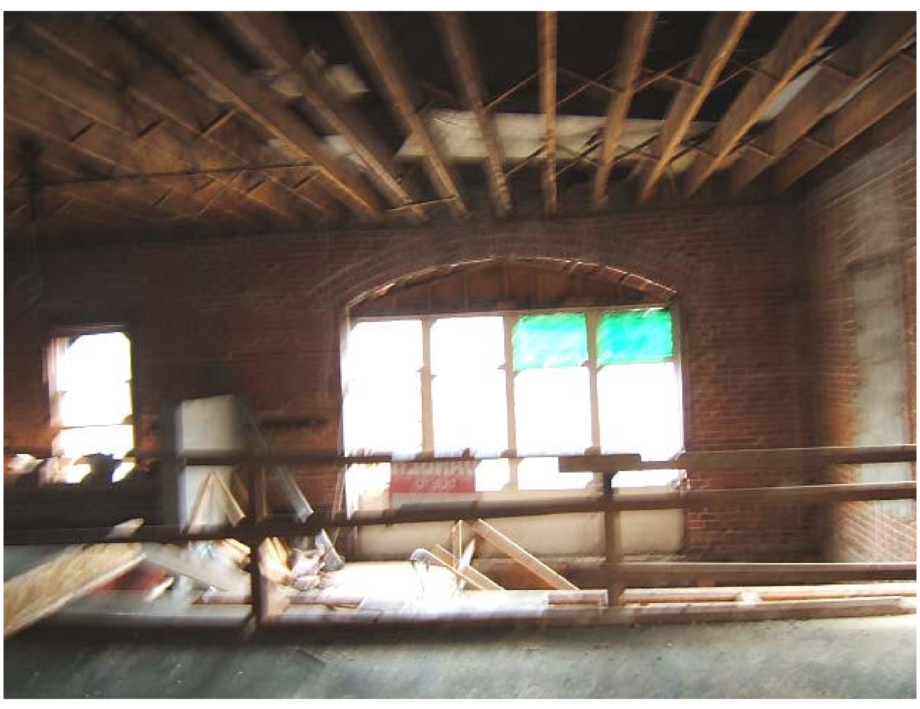
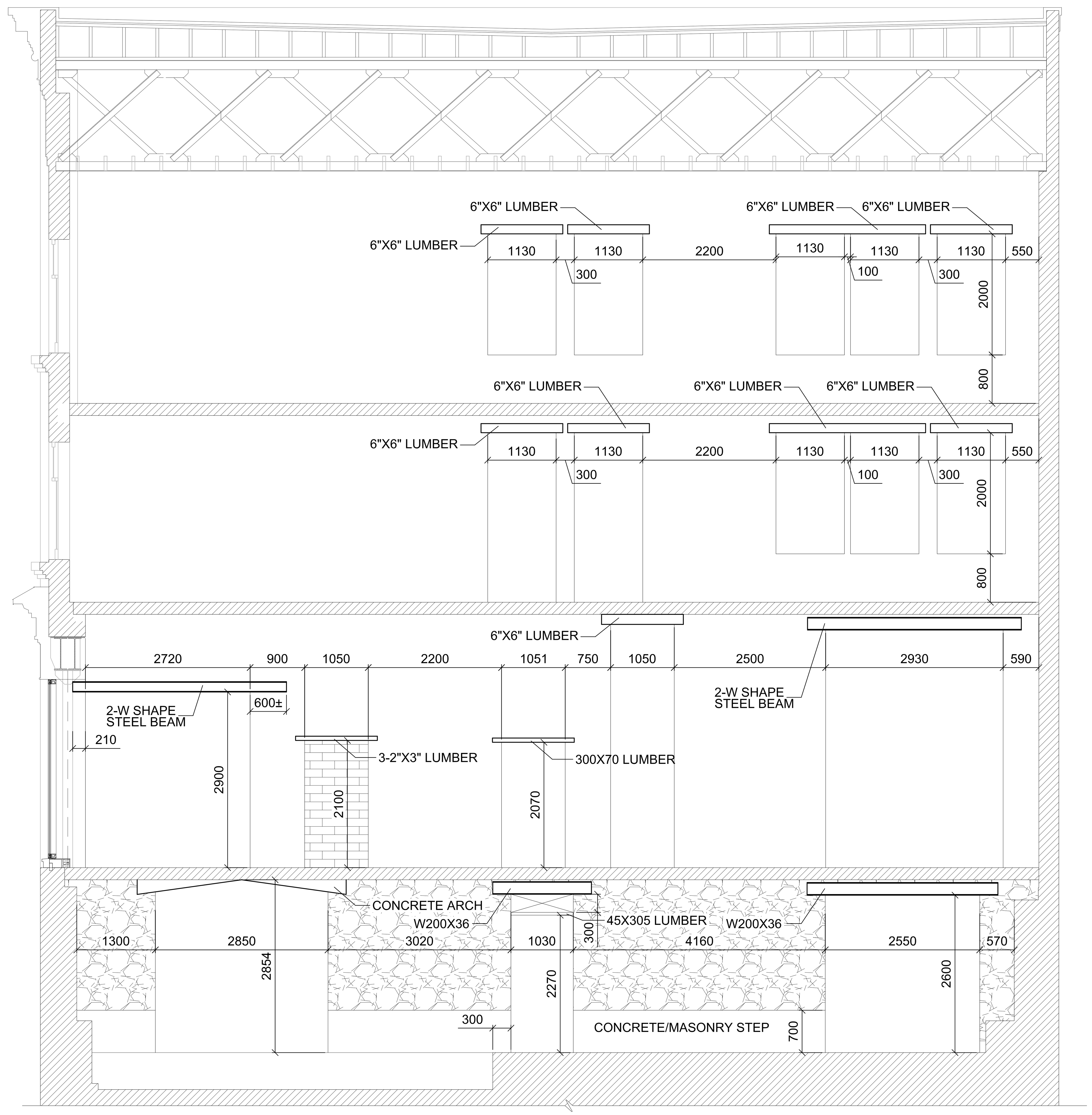


Figure 9: Previously Existing Condition Outlining Roof Diaphragm

Figure 10: Frame of Reference for Figure 10

Figure 11: Previously Existing East Wall As-Built



client

T.K.S HOLDINGS INC.

art engineering inc.

scale AS SHOWN

1.	FOR CLIENT'S REVIEW	May .07, 2007
revision		date

A detail no. / no. de détail
B location drawing no. / sur dessin no.

project

325 SOMERSET RECONSTRUCTION

drawing

AS–BUILT SECTION

designed A. Ivantchouk

drawn H. Ghassemi

date May 07, 2007

project number 0653

drawing no. A5