Structural Analysis of Historic Absorption Building in Turner Valley, Alberta

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Abstract. Many historic structures in Canada are deemed unsafe and are closed or of limited access to the public. An "unsafe" steel and concrete heritage building rebuilt in 1930 has been analysed structurally. The building in question is the absorption building at the Turner Valley Gas Plant (TVGP), a National Historic site. Throughout the building lifespan the structural skeleton has been adapted to accommodate changes in the oil and gas processing. The TVGP was Alberta's first natural gas plant built and thus the birthplace of the energy sector in Western Canada. The absorption building housed the first ever absorption plant in Canada in 1914. The load path, effects of modified and missing members, and capacity of elements were assessed. Due to a lack of historical records, Non-destructive testing methods were used to determine building properties. Geometrical data was collected with laser scanners and ground penetrating radar systems. X-ray diffraction, scanning electron microscopy, hardness tests and tension/compression tests were used to determine material stiffness, strength, and chemical microstructure. Four finite element models were developed to conduct a linear-elastic analysis to assess the effects of changes in structural integrity which may have occurred due to structural member modifications. A load test was performed to validate the models. Results confirmed the load path and the effects of modifying members as an initial assessment towards a complete safety analysis. The research also exposed gaps within current standards and provided a guide to future engineers on structural interventions in heritage structures as standards are developed.

Keywords: Structural Analysis · Historical Structures · Finite Element Modelling · Non-destructive Testing · Heritage Conservation

1 Introduction

Historical structures are a key component in understanding past culture and engineering methods. The Canadian construction industry is shifting to reusing existing structures in an effort to reduce environmental impacts, like our European neighbors [1]. In the last 20 years more and more buildings have been designated as historical, with many of them closed to the public as they are considered unsafe. However, the amount of funding for conservation has not increased with the number of designations [2]. The lack of funding does not allow for structural assessments to be conducted on all buildings and

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therefore many are closed to the public due to safety concerns [2]. We conducted a structural assessment on a historical building located in Turner Valley, Alberta, Canada which is currently closed and designated as "unsafe". The building in question is the absorption building of the Turner Valley Gas Plant (TVGP), a National Historic site [3]. The structure was rebuilt in the early 1930s using steel, corrugated iron sheathing, and concrete when the original 1914 wooden building burnt down in 1920 [3]. The site is historically known as the birthplace of the energy sector in Western Canada and the absorption building was one of the first of its kind [3]. Due to adaptations in oil and gas processing the building's structural skeleton underwent many adaptations, i.e., structural members were removed or deformed to accommodate gas piping. The building requires a structural analysis to consider whether it can be designated as safe and opened to the public.

Conducting a structural analysis on a historical structure provides unique challenges. It is common that the structure will have no engineering records on the geometrical or material properties. The lack of records creates a need for geometrical, material, and structural data to be collected as these data are required to create a finite element model and conduct a structural analysis. Engineers must consider how the building is constructed to ensure the structure can safely withstand the loads to which it is subjected. Historical structures are protected by law against any damage during remediation, adding another layer of complexity to the analysis. Non-destructive and minor destructive methods were used to collect data. In Canada, the guidelines in the National Building Code of Canada (NBCC) and the Standards and Guidelines for the Conservation of Historic Places (SGCHP) from Parks Canada are used to set an expectation and roadmap in restoring structures based on internationally agreed principles [4, 5]. The NBCC is vague on how to conduct a structural assessment on historical structures and the SGCHP does not consider the technical process [4, 5]. Additionally, neither of these documents are mandatory when conducting an intervention - resulting in lack of guidance and inconsistent interventions. This poses a safety issue and public risk because engineers and the structures they are dealing with are not held to the same standards as applied to new structures.

Our objective was to determine if the building is safe to be open to the public, either in its current state or after an intervention. A structural analysis was conducted based on an applied test load. Multiple models were created to compare various characteristics of the building and determine their impact, i.e., the removal of members, and the effects of the corrugated iron sheathing surrounding the building.

2 Methodology

To conduct a structural analysis using FEM, the building's geometries and material properties need to be obtained as structural engineering records were not available for the absorption building at TVGP. The section below describes the various models that were developed to best represent and simulate the conditions seen on site at the absorption building.

Fig. 1. The absorption building at the Turner Valley Gas Plant.

2.1 Geometrical Properties

A geometrical survey was conducted to obtain the geometrical properties of the building. The building is 6.1 m wide and 13 m long. The height of the apex is 5.2 m and the roof slopes 26–27° (Fig. 1). Five 1-m-wide absorption tanks run through the middle of the building. The tanks obscure the view of trusses and roof members. Additionally, the building has vaulted ceilings and missing or deformed members (Fig. 2).

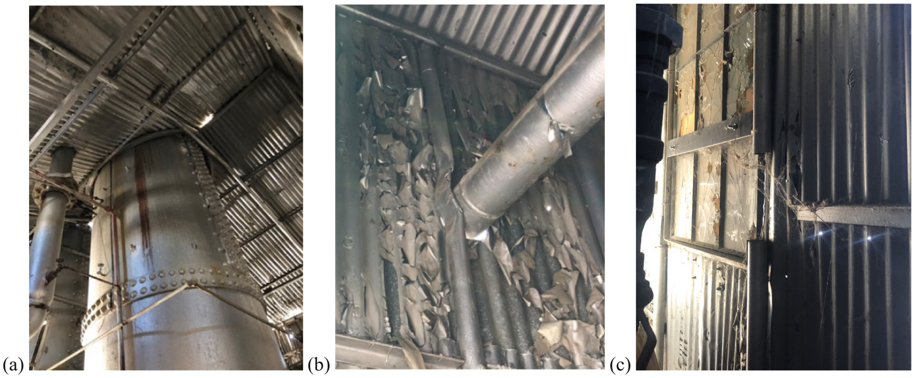


Fig. 2. a) Shows the vaulted ceilings and absorption tanks, b) Shows a member that was deformed to allow a pipe to run through the building, c) Shows a member that was removed which are typical throughout the building.

The complex building geometries and obstructions restricted the ability to use hand tools to determine measurements in various locations, especially in the roof [6]. Laser scanning was used to conduct a geometrical survey of the building and the results were validated against hand measurements [6]. The values obtained were within 5 mm of each other and therefore they were used in the modelling of the absorption building [6]. The cross sections of members were determined using calipers as the error present within the laser scanning cloud model was not reliable for such small measurements. Five structural members were used in the modelling of the building, i.e., pipe columns, pipe beams, roof angles, beam angles, and corrugated iron sheathing (typical cross-sections are shown in Fig. 3).

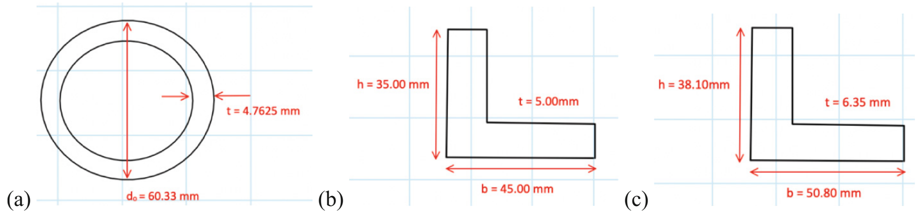


Fig. 3. Common cross sections within the absorption building. a) Cross section of all pipe members, b) Cross section of roof angle members, c) Cross section of beam angle members.

2.2 Material Properties

No records of material properties were available, i.e., the type of steel, whether piles were used, or if the concrete was structurally reinforced with steel. These structural and material properties were collected through various non-destructive or minor destructive tests. To determine the concrete strength a sample of the foundation was taken from a spalled area and tested until failure under compressive loads. A scaled version of the ASTM standards for compression tests was used to determine the height to diameter ratio for the samples as the concrete thickness available was limited [7]. Cylinders were created with a 2:1 height to diameter ratio (Fig. 4a). The compressive strength of concrete was determined to be 21 MPa. To determine the steel properties a 30 mm long section of pipe was cut from a modified member in the building and tested until failure under tension loads. A SNC machine was used to create a scaled version of the ATSM E6 standard dog-bone samples for testing [8] (Fig. 4b). The ultimate stress for the steel specimens was determined to be 390 MPa. Due to the size of the samples strain gauges were not applicable to the sample and Young's Modulus could not be verified. A value of 200 000 MPa was used [9].

Ground penetrating radar (GPR) was used to determine the foundation depth, if the concrete was reinforced, and if piles exist. The results found no reinforcement or piles were present within the concrete and the foundation was 15 cm thick. To test the chemical microstructure of the steel and concrete, X-ray diffraction, scanning electron microscopy, and hardness tests were used. The chemical composition results indicated that the concrete had a high cement ratio, and the steel was considered a mild steel with a high carbon content.

The corrugated iron sheathing was measured to be 1.5 mm thick with a corrugated pitch of 73.6 mm. When compared to standard sheet sizes, an appropriately similar

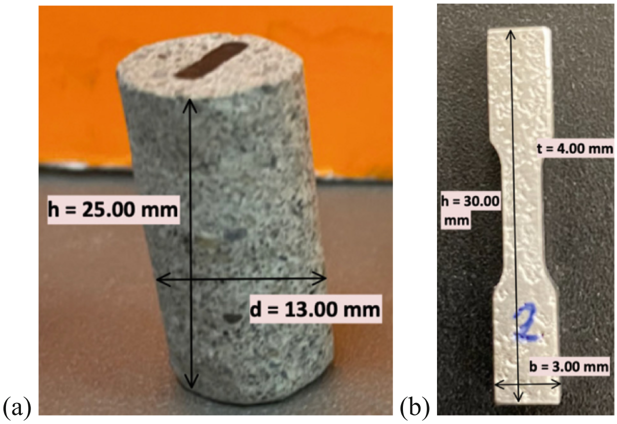


Fig. 4. a) A concrete test sample for compression tests, b) A steel test sample for tension tests.

industry size is a 1.5-mm-thick sheet with a depth of 19 mm and corrugated pitch of 75 mm [10]. Therefore, the standard sheet size was carried in further calculations.

2.3 Load Path

To understand the load path, we must understand the construction of the building. The building consists of 7 main frames – two exterior and five interiors. Of the five interior frames, all are repetitive except for one (Fig. 5).

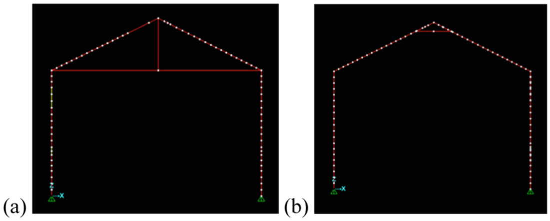


Fig. 5. a) Interior repetitive section, b) Interior section used at one location.

The gravitational load system consists of the roof sheathing tied to the roof angles, and roof angles tied to the angled pipe columns, which are simply supported on the top of the wall. The walls consist of corrugated sheathing tied to both horizontal beam angles and column pipes as seen in Fig. 6a. The column pipes were connected to a horizontal pipe beam at their base, tying all the frames together. For extra rigidity the horizontal pipe beam was cast into a concrete ledge. The ledge then carried the load to the ground. Figure 6b depicts pipe columns encased in the concrete ledge. The corrugated iron sheathing had begun corroding in various areas, but the corrosion was not significant to raise concerns (Fig. 6c). A goal of the modeling was to determine if the sheathing is carrying loads and if the corrosion and connections should be addressed.

Fig. 6. a) Depicts the corrugated iron sheathing wall connected to both a pipe column using bolts and an angle beam using ties on an interior repetitive section, b) Depicts the concrete ledge in which a pipe beam is encased, c) Depicts corrosion that is seen throughout the sheathing.

2.4 Connections

The connections within the building lack consistency and vary from welds, to ties and bolt connections. The exact method used to create the tie connections is unknown, however they are best described as bent nails. Figure 6a shows tie connections and bolt connections. The connection strength was not tested in this case study. However, it was noted that the connections between the sheathing and the columns at the base of the walls were beginning to fail in various locations.

2.5 Finite Element Model

The finite element method in the linear elastic range was used to conduct a structural analysis of the absorption building. The results of the GPR scanning showed that piles were not present, rebar did not exist, and the foundation was about 15 cm thick. Based on these results, we decided modelling the foundation was not beneficial. To account for the foundation and concrete ledge, fixed boundary conditions were used at the base of the wall in all models. All FEMs were analyzed in SAP2000 and Abaqus. The steel material properties used were that of typical mild steel as historical values were not obtainable (Young's Modulus was set to 200000 MPa, and Poisson's Ratio to 0.3) [9]. All cross sections previously mentioned in the geometrical properties section were used in the creation of the framework.

The sinusoidal profile of the 1.5-mm-thick corrugated iron was converted to an equivalent rectangular block shell element, as we were interested in the addition of sheathing to overall stiffness and not the structural analysis of the sheathing itself. The effective stiffness was calculated to represent that of the sheathing. The equivalence properties based on industry sheet sizes is listed in Table 1 [10]. The ratio of the moment of inertia for the corrugated sheathing and equivalent rectangular shell section was found to be 168.6. This value was used as a multiplier in the FEM material properties, i. e., the strong axis moment of inertia, and bending, to account for corrugation of the sheathing. The resulting equivalently stiff rectangular shell element was 1.8 mm thick. The equivalent stiffness was calculated for the shell elements and is reported in the last row of Table 1 demonstrating the equivalent stiffness to the corrugated iron sheathing.

Table 1. Equivalent structural properties of rectangular shell element and corrugated sheathing [9].

	Corrugated Sheathing	Equivalent Rectangular Shell Element
Cross-Sectional Area (mm2 (in2))	1974 (3.06)	---
Moment of Inertia (mm4 (in4))	81 935 (0.06)	486
EI (Stiffness) (N-mm2 x1010)	1.6387	1.6388*

*This value was determined by multiplying the equivalent rectangular shell element moment of inertia by the equivalence ratio of 168.6 and by Young's Modulus

Four different FEMs were created with the same material properties and overall geometry but with differences in structural arrangement and boundary conditions of the members.

Fully Fixed Framework Model. This model used the skeleton of the building with all members in place and all connections fully fixed (Fig. 7). Fully fixed constraints mean translation and rotational degrees of freedom are fully linked between members at all joints. Moments, shear, and axial forces are transferable at all connections. The model was constructed using beam elements and had no sheathing and no missing members. This was done to mimic the model once construction had been completed and to act as a baseline for all other models. The displacement results obtained from this model would be the minimum displacement allowed for the building. Thus, creating a limit for any validation testing. This model was also created to determine if the sheathing was affecting the structural integrity of the building. The remaining models used this model as a base.

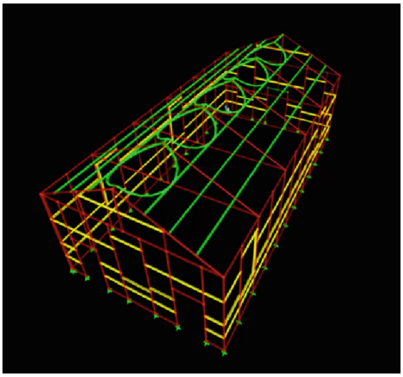


Fig. 7. Depicts the Fully Fixed Framework Model. The green members depict roof angle sections, yellow depicts angle beam sections, and red depicts both pipe column and beam sections.

Fully Fixed Shell Model. Shell elements were added to the Fully Fixed Framework Model on all faces of the building with continuity at all edges (Fig. 8). The shell elements were designed to account for the sheathing. This model, like the framework model, acts as an idealized model of the post original construction version of the building to assess a potential structural role of the sheathing. Additionally, this model was used in further models as a base.

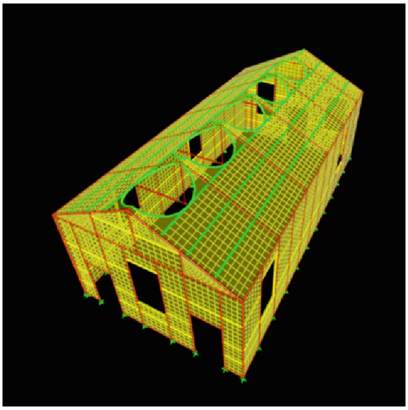


Fig. 8. Depicts the Fully Fixed Shell Model. The shell elements shown in yellow were added to the original framework shown in Fig. 7.

Partially Fixed Shell Model. In the Partially Fixed Shell Model, connections were modified to simulate the reality of the connections on site. All beam connections in the Fully Fixed Framework Model were converted to pinned connections as the angle beams were usually cut to surround the column and join to the pipe. This is shown in Fig. 6a. After further observations on site, it was discovered that the sloped roof pipes were only semi-welded to the beam at the top of the wall. The shell edges at the top of the walls and bottom of the roof were released to pinned connections as the roof sheathing is not connected to the wall sheathing on site. Additionally, the shell edges at the bottom of the wall and bottom of the columns were released to pinned connections to simulate the failing connections between the sheathing and framework as observed. Figure 9 highlights the shell elements that were released in green. As the connections on site were not tested, we were limited in the number of connections we could release. The shell edges, beam to column connections, and sloped beam weld connections were released as they were visually observed as disconnected or partially connected. The pin connections allow shear and axial loads to transfer through the building, but not moments and torsion.

Existing Building Shell Model. This model was developed to simulate the building in its current form with missing members and appropriate connections. Twenty-nine members were missing or deformed and therefore unable to carry load, and these connections were completely removed from this model. The most affected faces of the building are depicted in Fig. 10. The remaining connections were simulated the same as in the Partially Fixed Shell Model. This model sought to determine if the removed

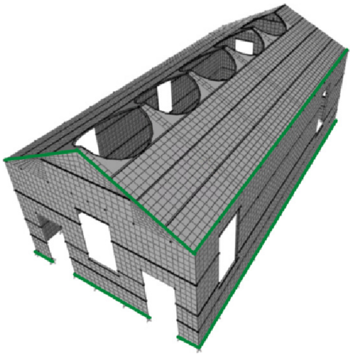
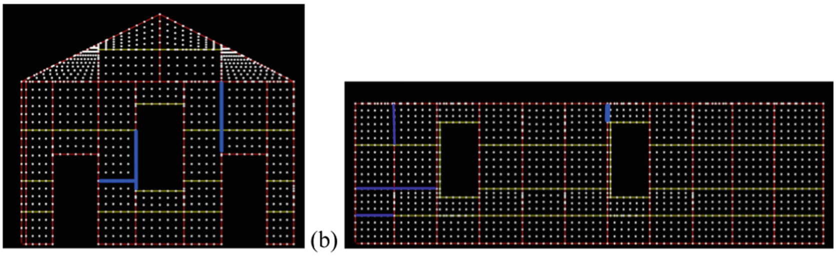


Fig. 9. Depicts the shell edges in green that were released to pin connections.

members affected the structural integrity of the building and was also used to compare to load test results conducted on the building.



(a) (b)

Fig. 10. Depicts an example of the missing or deformed members in blue that were removed from the existing building model. a) Depicts the front of the absorption building, and b) Depicts the side profile of the building.

2.6 Validation

A load test was conducted and measured on site at the absorption building. The same test load was applied at the same location to all the above-described models. This was done to confirm and validate the model results. The test load location was strategically selected for feasibility both on site and in the model's space. The load was applied to the bottom chord of the interior frame closest to the back wall as shown in Fig. 11. The geometry of this frame does not distribute the loads as well as the other interior frames, therefore displacements will be most visible at this location. Additionally, this location had no obstructions and thus provided the best access on site. The length of the loaded member was 1124 mm and using the limit of L/360 the allowable deflection is 3.12 mm [4].

The load test was conducted on a still spring morning with no snow load and wind load present. A 55-kg weight (532.7 N) was applied at the above specified location. Dial

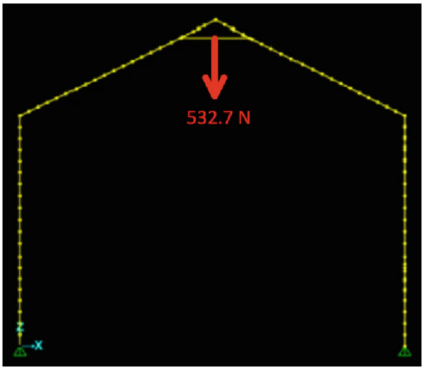
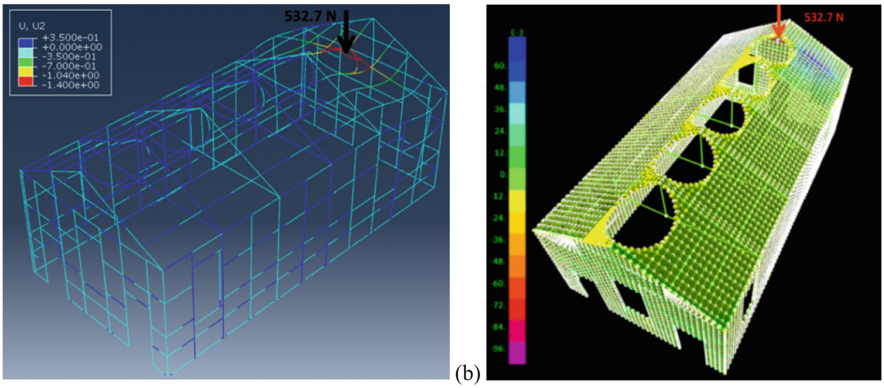


Fig. 11. Depicts the interior rare section with a load of 532.7 N

gauges were set at the top of the columns and at the location of load application. The dial gauges have a minimal resolution of one thousandth of an inch (0.0254 mm) A reading was taken immediately after loading and a half hour after loading with the loading still applied.

3 Results

The following results were obtained when the same test load was applied on site and in the models. Figure 12 depicts the vertical displacements of the Framework model compared to the Shell model for the same test load (Table 2).



(a) (b)

Fig. 12. a) Depicts the vertical displacements of the Fully Fixed Framework Model with displacements shown magnified by a factor of 500, b) Depicts the vertical displacements of the Existing Building Shell Model with displacements magnified by a factor of 500 as well illustrating the significantly reduced displacements.

Based on the above, the following observations were noted:

- The fully fixed shell model predicted the smallest vertical displacement.

Table 2. Load test results for vertical displacement at point of load application.

Model Names	Vertical Displacement (mm)
Fully Fixed Shell	0.1036
Partially Fixed Shell	0.1059
Existing Building	0.1061
Fully Fixed Framework	1.3154
On-site Data[a]	0.0508–0.12701

[a]The tabulated value for on-site data is a range from instantaneous deflection to post half an hour deflection

- The shell models all predicted very similar displacements.
- The fully fixed framework model predicted the largest displacement and when compared to the on-site data it is the only value that does not fall within the range.
- The on-site deflection is within the range of allowable deflections based on the simply supported member.

4 Discussion

Based on our analyses and experimental measurements, the shell models were found to be accurate within marginal error. The displacements measured during the on-site load test are under the allowable 3.12 mm based on the NBCC [4]. This result is a step in the right direction to determining if the building is safe to be reopened to the public. The fully fixed framework was intended to act as a minimum, which was found not to be the case. The corrugated sheathing (here modelled with shell elements) appears to be carrying a significant portion of the load and needs to be considered in the structural analysis of the building. The results between the framework and shell model show that the sheathing adds stiffness to the walls, reducing horizontal displacements and adds stiffness to the roof which reduces vertical displacements. Figure 11 b and c show the deformed structure with and without shell elements for the same scale. It is visible that the sheathing is acting as a support to the framework by preventing displacement of the framework itself.

It was observed that connections at the base of the sheathing were failing. This is of concern as the sheathing is carrying load. If the load is unable to transfer through the sheathing the displacements will increase and could result in failure of the building or entire walls. The connections should be monitored and tracked as a preventative measure. Proper maintenance of the sheathing and connections would prolong the service life of the building. Further studies should be conducted on the connections, their design, strength, and constructability. Additionally, material tests should be run on the corroded corrugated iron sheathing to determine the rate at which its structural capacity is diminishing. Based on the results obtained, the model has been validated and now can be used to model other potential load cases that will determine if the building can be opened to the public. Potential load cases would include wind and snow loads based off survey data from Turner Valley.

The Fully Fixed Shell Model is acting as a minimum. The 29 insufficient members in the Existing Building Model accounts for the difference with the Partially Fixed Shell Model but their absence has not significantly affected the overall structural response of the building. However, locally, there could be a more significant effect on adjacent members which will have to be considered. Complexities of the building's structural joints are difficult to fully incorporate in the model, possibly resulting in variations between the shell models and the on-site data: further consideration for better simulati0n of the types of connections used may be needed. Systemic errors from the use of sensitive dial gauges with a resolution of 0.0254 mm, material and geometrical data collection, and the setup used for load testing the building were present. Future studies should include more test locations and the effects of horizontally loading the building.

As previously mentioned, no code exists for conducting structural reviews on historical structures in Canada. Recommendations are in place based on the Parks Canada Guidelines [5]; however, they are not enforced, and the document does not aid in the technical portion of the review. For the current study, the guidelines were followed as applicable with the non-destructive testing and minimal impact on the building. However, the tests completed were not specified and methods for conducting studies are open to interpretation which can lead to inconsistences in evaluations. This case study could be used as a reference for the load testing conducted and the appropriate material, geometrical, and structural studies conducted. A set of structural guidelines should be created and enforced for historical structures in Canada.

5 Conclusion

Two main conclusions from this study are: (1) the sheathing around the building is carrying loads, and (2) the missing and deformed members in the building had minimal structural effect on the vertical load-displacement response of the building. Intervention methods on the absorption building should take the sheathing into consideration and be careful in the event of removal or replacement. The Parks Canada Guidelines and National Building Code of Canada were considered throughout this study [4, 5]. Material, geometrical, and structural properties were all collected in a non-destructive or minor-destructive manner. The on-site experimental testing conducted for model validation was done with minimal weight to ensure the structure was not overloaded or caused permanent damages. The minimal differences between the displacements measured on site and in the FEM predictions provide confidence to the validation of the models for use in future studies. This research exposed gaps within current standards and provided a guide to future engineers on how to conduct a structural analysis on a steel structure as standards are developed.

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