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Seismic Retrofit and Structural Assessment of a Modern Timber Building Re-Commissioned for Exterior Conditions as a Giant Gazebo

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ABSTRACT

Built in 2001, the former entrance pavilion of the Quebec City Zoological Garden features a glulam frame, light-frame wood shear walls and steel braces as its Seismic Force Resistance System (SFRS). The projected deconstruction of the exterior envelope and side wings, that were integral part of the existing SFRS, created several structural retrofit challenges. During this project, it was identified that there is a lack of permissive approaches and references regarding both the seismic retrofit of modern engineered wood buildings and the evaluation of complex structural diaphragms. This study describes the methodology used to address the structural challenges encountered during the modifications of the building.

The anticipated seismic behavior of the structure conditions after deconstruction was found to be deficient, mainly due to the diaphragm design not allowing redirection of lateral forces to the foundations. A dynamic analysis of the frame and shell elements was performed to identify stresses using a lower bound – upper bound approach. The desired model behavior was obtained following modeling refinements including structural modifiers applied to specific elements. The strengthening of the diaphragm was performed by modifying various criteria of the CSA O86-19 standard after literature and peer reviews. A finite element model allowed the engineers to assess the stresses and strains of the steel elements of the bracing. The unique shape of the building and complex angles brought the engineers and architects to entirely model the building and design new adapted connections. The main challenge was to ensure the building's structural integrity at all steps through a well-planned deconstruction and retrofit schedule.

Keywords: Wood structure, Seismic retrofit, Structural assessment, Diaphragm, Shear connector.

INTRODUCTION

In 2020, Quebec's government announced a new Mammoth program to build several new high schools within its educational public system. Amongst them, both new elementary and new high schools were to be built on the now closed site of the former Quebec City Zoological Garden. The numerous unnecessary buildings located on the site were all planned to be demolished. However, as the former entrance pavilion of the zoological garden was built in 2001 and stands out as a quality, esthetical, and well-preserved building, it was instead decided to re-commission it as a giant gazebo for exterior activities. This is seen as an investment as it will provide students an easier access to the surrounding nature. It is also seen as a sustainable development measure by the reuse of already produced construction materials.

STRUCTURAL SYSTEM

Structural components of the building are primarily composed of glulam beams and columns with two light-frame wood wings on each side. The building is 34 meters long and features two cantilever roof sections at both extremities with a span of 2.7 meters and 4.5 meters. At the largest point, width is 22 meters, and it varies down to a minimum width of 9.7 meters. The two wings are 7.2 meters wide by 11.4 meters long and are connected on glulam columns. The total height of the one-story structure is approximately 6 meters. Similar structural components can be found on each side of a symmetrical axis in the middle of the building, parallel to the long direction. Foundations are composed of a perimeter wall and footing supporting the envelope. Columns are supported on isolated footings. As shown in Figure 1, the curved roof is angled towards a central section and points to two opposite vanishing points with inclined angles of 9 and 14 degrees. The unconventional geometry of the glulam columns and curved beams fixed at precise angles brought stability challenges to the numerical modeling while performing the seismic analysis.

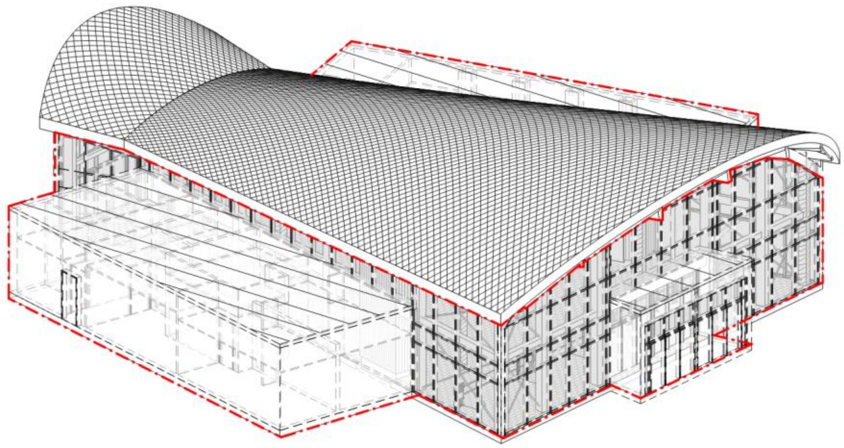


Figure 1. Existing glulam building before deconstruction.

The existing seismic force resisting system (SFRS) is composed of tension-only steel braces oriented in both directions and located at one end of the building, as shown in Figure 2. After studying the existing structural drawings and the shop drawing details, it became evident that the light-frame wood walls connected to the glulam columns provided lateral resistance and stability to the structure. Therefore, the removal of the wings and envelope, for re-commissioning the building as a giant exterior gazebo, modified the diaphragm supports conditions, rendering it to a cantilever. This transformation made the gazebo very likely to perform poorly under large seismic demand, unless a retrofit was performed. In addition, the roof's diaphragm composition appeared to have limited capacity in regard with the shear forces coming form the seismic demand, as per the National Building Code of Canada (NBCC) [1]. Drawings and site survey reveled a design that did not allow for redirection of lateral forces to the foundation, which is critical in the case of a large seismic event.

Figure 2. Existing steel bracing system.

CHALLENGES

The main challenges for this project were identified early in the design process and were as follows:

- Ensuring the building's structural integrity through a well-planned deconstruction and retrofit schedule.
- The need to perform a seismic assessment and retrofit study due to the planned transformations.
- Lack of information on the connection pattern between the plywood diaphragms and the SFRS. On-site surveys of these connections were impossible as a strict command in the project was to keep the high-quality roof system intact.

It would have been necessary to partially demolish the roofing to gather information on the existing diaphragm and it was instead decided to take a conservative approach in the evaluation, and proceed to the retrofit if required.

- Lack of permissive approaches and references regarding both the seismic retrofit of modern engineered wood buildings and the evaluation of complex structural wood diaphragms.
- The unique shape of the building and complex angles brought the engineers and architects to entirely model the building in 3D and design new adapted connections. The shape also provided challenges for the structural analysis of the structure.
- The need for a complex finite element model to assess the behavior of the existing steel elements of the X braces middle connection formed with steel rings welded to a circular bent plate.
- Durability concerns and specific details.
- Thermal movements and loads.

DESIGN APPROACH

Seismic Performance criteria

In the province of Quebec, the construction legal document is the Quebec Construction Code [2], which now adopts NBCC 2015 [1] prescriptions, with modifications. One such modification, is the inclusion of clause 10.4.1.3 that greatly impacts the projects involving existing buildings. This clause essentially requests that when a building undergoes transformation, its ability to withstand seismic loads must meet the following:

1. **It must not be diminished by the effect of this transformation.**
2. Except for buildings whose structure has been designed in accordance with seismic design requirements of NBCC 2005 or NBCC 2010, it must be raised to at least 60% of the level of seismic protection that would be prescribed under Part 4 of NBCC 2015, if the transformation results in one of the following situations:
    a. In the case of a post-disaster building, more than 25% of all floor areas are stripped.
    b. The lateral load resistance system is modified by the effect of this transformation.
    c. An extension of more than 10% of the area of the building or of more than 150 m^2, except when the structure of this extension is distinct from that of the existing part and the movement of each structure in the event of an earthquake does not have impact on the adjacent structure.
    d. The transformation has the effect of increasing the permanent load of the building by more than 5% or of increasing the total of the live loads included in "W", as defined in paragraph 4.1.8.2. 1), by more than 5%.

In the project, both requirements 1. and 2.b. of clause 10.4.1.3 were not met as the light-frame wood walls connected to the glulam columns provided lateral resistance and stability to the structure and were planned to be demolished. Thus, the seismic capacity of the modified structure needed to be improved to at least 60 % of the NBCC 2015 (2% in 50 years – 1:2475 years return period) requirements. For this project, it was established that this targeted level of seismic protection was equivalent to aiming for 100 % capacity requirement under the 5% in 50 years (1:975 years return period) seismic ground motion.

Specific performance criteria were established in compliance with NBCC 2015 [1], CAN/CSA O86-19 Code [3], CAN/CSA A23.3-19 Code [4] and CAN/CSA S16-19 Code [5] requirements to perform performance-based seismic design. All criteria were studied for a high protection level ($I_E = 1.3$) at a seismic ground motion probability of exceedance of 5% in 50 years (1:975 years return period). The “high protection” level is mandatory due to the new occupancy of the structure. The main criteria used for design are summarized below:

- The service level of the building shall allow for the safe evacuation of the building.
- Repairable damages can be induced to the building’s structural components featuring some ductility (tension-only braces, selected glulam connections).
- Damages shall only necessitate localized, moderate interventions before restoring the building full service.

While the criteria established for this project are qualitative, they were assessed using stress and strains values that allowed for quantifying the damage sustained by the structural components following a performance-based approach.

Structural analysis and design

The CSi SAP2000 v22.2.0 software [6] was selected for the analysis and design of the existing structure due to the complex angles of the wooden elements. Multiple simplified models with curved beams were tested and compared with manual calculations to validate the numerical results. In comparison with discretized models composed of straight beam sections, no major variations were identified for that project. Therefore, a straight beam model, shown in Figure 3, allowed for the use of inclined thin shell elements as a diaphragm. Longitudinal beams were modeled at the same angle and transversal beams were split at strategic locations to create a proper curvature representing the actual structure. In regard of gravitational analysis, shell stiffness modifiers were used to fully redistribute the loads on the glulam beams. To have a proper load distribution, local axis of the shell element was angled towards the direction of the wood decking. Only the structural elements expected to be retained were included in the model, in order to capture the behaviour of the new giant gazebo.

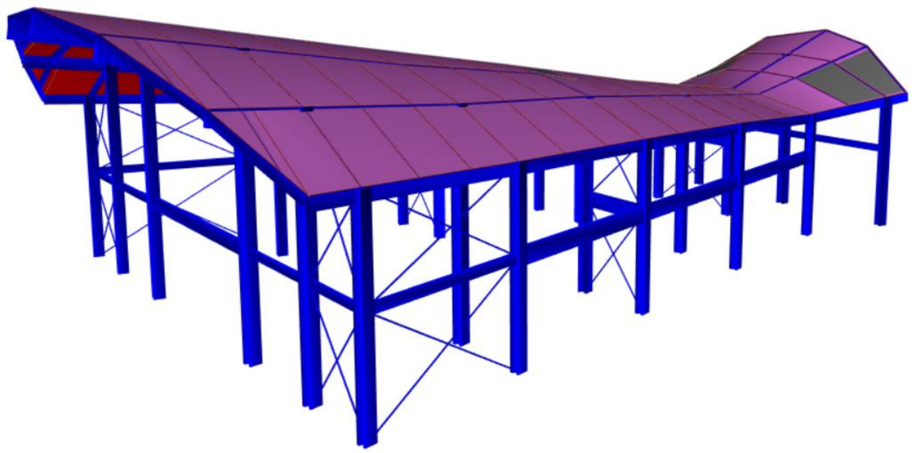


Figure 3. Numerical model of the wood building.

From the detailed analysis of the deconstructed building, the design's team assessed that the 5% in 50 years seismic event should result in:

- Yielding of the diaphragm connections or crushing due to concentrated stresses at support points. These damages are estimated to occur only in localized areas of the diaphragm.
- Disconnection of the diaphragm from the collector beam in the long direction due to high shear flow stress and deficient assembly design.
- Loss of stability and potential collapse of the structure due to the removal of the two side wings providing lateral resistance.

These damage levels did not allow the structure to meet the mandatory performance criteria described previously, therefore demonstrating the importance of the thorough seismic assessment. Consequently, the seismic retrofit was required.

Diaphragms

In regard with lateral loading, when analyzing the diaphragm, reductions of the in-plane properties in the two main local axes were applied. This allowed for a full utilization of the in-plane shear strength of the modeled elements of the diaphragm. The desired behavior was ultimately achieved by adding axial property modifiers to all the beams not considered as seismic force collectors.

A dynamic analysis of the frame and shell elements was performed using a lower bound - upper bound design approach. This was deemed necessary as the existing diaphragms were composed with the following elements forming a complex composite section:

- 12.7 mm plywood layers featuring an unidentified nail connection pattern to the wood deck.
- 38 mm wood deck with specified screws on the glulam beams.
- Glulam beams.

The expected complex behaviour was captured by a lower bound - upper bound approach to evaluate a realistic diaphragm rigidity. Iterations were performed until convergence with the expected yielding mechanisms were obtained between the governing components in the diaphragm composite section. Along the process, when looking for literature on the subject in order to support the analysis, the authors noted a lack of scientific tests results or former studies concerning similar diaphragms.

The analyses were achieved by generating a response-spectrum analysis (RSA) measuring the contribution from each natural mode of vibration to indicate the likely maximum shear stresses in the diaphragm as shown in Figure 4. Based on the results from the RSA and available existing drawings, multiple deficiencies were identified that required numerous retrofit solutions. An axial load, and shear flow analysis through designated collector beams was also performed.

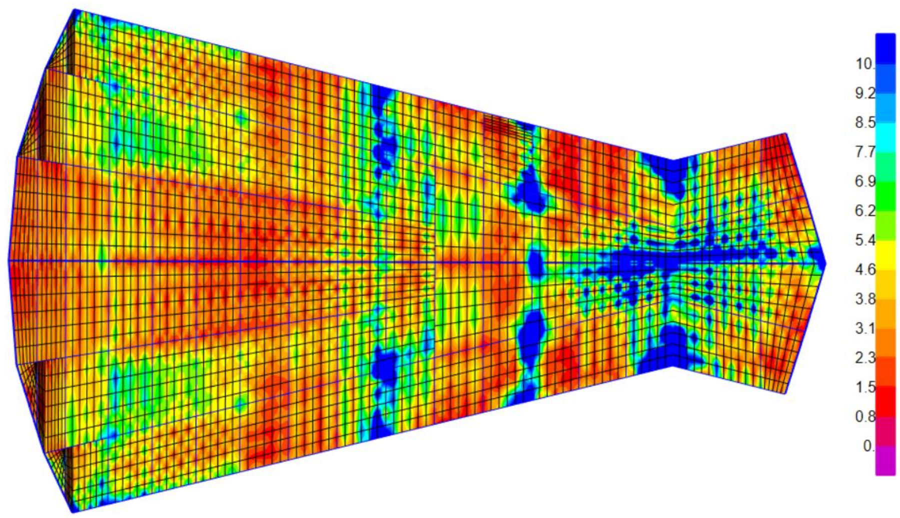


Figure 4. Maximum stress in the diaphragm.

One of the most important deficiencies identified was the lack of a rigid connection between the diaphragm and the longitudinal collector beams of the vertical bracings. The analyses showed that torsional and lateral out-of-plane moments were induced in the principal beams, see Figure 5.

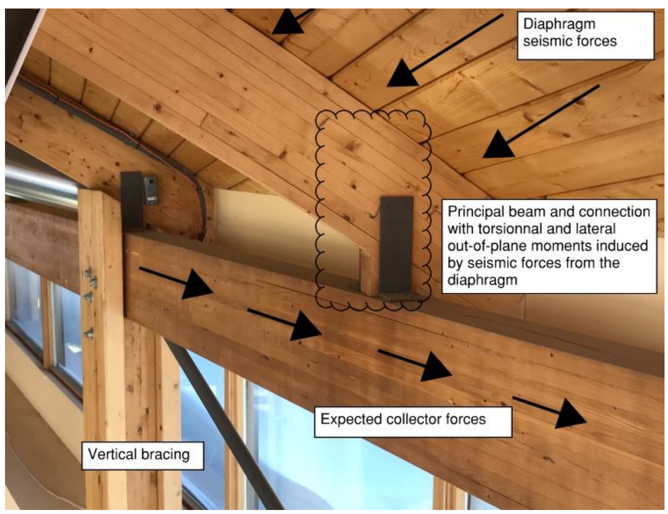
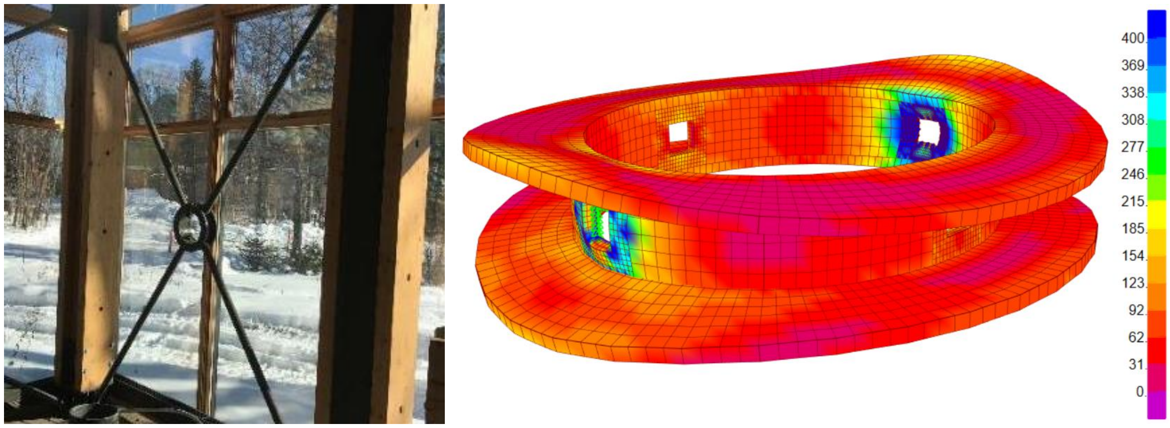


Figure 5. Existing diaphragm deficiency

Detailed analysis of bracings

Tension-only concentric steel bracing systems are often used as the preferred SFRS in wood building due to their well-known seismic behavior, low capacity-design loads, and low material cost. In the case of the studied existing timber building and in the context of the structural assessment, the identified SFRS as per the NBCC was a conventional construction braced frame ($R_d = 1.5, R_o = 1.3$). The principles of capacity design [7] were used to ensure proper energy dissipation through the vertical bracing system therefore protecting the wood elements and the wood-to-steel assemblies.

The major requirement for the bracing system was to obtain proper seismic behaviour while keeping the existing custom design. The middle connection of the X braces is formed with steel rings welded to a circular bent plate, shown in Figure 6. This connection ensures that the tension force is transferred from one tension element to another. Through all the methods taken to study this special component, the most efficient analysis method used was solid modeling and detailed finite element analysis (FEA). Using the CSI SAP2000 software and seismic forces in the bracing rings resulting from a capacity design approach, Von Mises stresses [8] were analyzed in the steel element. No major deficiencies were reveled by this study. The existing rings were therefore deemed adequate to sustain their design loads. These design loads were found to induce stresses reaching approximately 80% to 90% of the yield capacity of the material. FEA analysis led to the conclusion that tension-only steel rods are the energy dissipating component in this system.



(a) *(b)*

Figure 6. Bracings connector ring: (a) existing steel rings, (b) FEA model and results.

RETROFIT

Specific objectives

The main objective of the seismic retrofit was to make the new giant gazebo safe in the event of a major seismic event. Since the construction site was already mobilized, accessibility and supplementary restrictions were not considered as major issues. However, the identified challenges led the structural design team to apply many iterations and consult with specialized professionals from a consulting program and a university to ensure proper application of the CSA O86-19 standard [3].

The following specific objectives were therefore defined to overcome the project challenges:

- Allow for a design-build project delivery method to ensure proper coordination due to the unique shape and angles of the building.
- Allow flexibility to the client and the architect along the retrofit solutions selection process.
- Find proper documentation to understand the impact of using wood in exterior conditions to include numerous specific durability measures to the project.
- Do not deconstruct the high-quality roof, try to keep most the existing materials, and preserve the original design.
- Provide sufficient bracing to allow for the forces to travel along proper load paths and obtain a proper seismic behavior.

These objectives allowed for the comparison and selection of a preferred retrofit solution.

Retrotit solutions studied

The location of the added vertical tension-only braces was chosen at strategic locations by using an iterative approach to reduce the in-plane stress in the diaphragm. The approach started with the identification of the high stress regions, following model adjustments, to obtain proper diaphragm behavior. New bracings were then added to ensure the redistribution of internal efforts to the foundations. Based on the results, a feasibility evaluation of the proposed solution allowed the designers to choose an assembly design. In the case the proposed location did not satisfy the design requirements, the first step was repeated jointly with the architectures needs and design tolerance. Once the bracing layout was settled, proper analysis of the existing system with new conditions could be performed.

As the diaphragm capacity was deemed insufficient, the following solutions were studied:

- Due to the lack of precise information on the diaphragm's connection pattern, a study with new horizontal roof tension-only bracings under the roof was performed. A conservative approach to fully dismiss the shell element's contribution to a minimum realistic value by applying stiffness modifiers was used.
- To sustain shear forces, it was studied to upgrade the composite action between the plywood and the wood deck through a new nailing pattern performed from the underneath, see Figure 7. This is the opposite of a normal plywood diaphragm as it is normally nailed from the top of the plywood. As this case is not specifically covered by CSA O86-19 standard [3], many numerical tests were performed, many discussions were held, and specialized professional's opinions were obtained. The results of these can be summarized with the following recommendations:
  - Ideally the tip of the nail should exceed from the plywood.
  - It is important to respect minimal thicknesses in respect with nail diameter as illustrated with figure 12.17 of CSA O86-19. For a 12.7 mm thickness plywood, the maximum nail diameter is 2.54 mm.

- In the equations of clause 12.9.3.2 of CSA O86-19 standard [3], use the value of f2 in place of f3. f3 allows for some membrane effects in the equations d, e, and g. This membrane effect is more significant when the nail is nailed directly into the plywood as it is a thin element.
- There is a risk that the tip of the nail chip the plywood a little bit when it will come out of the plywood surface, which will reduce its contact length (value of t2). As a precaution, it would be recommended to subtract the thickness of a ply in the calculation of t2. In the case of 12.7 mm (4 ply) plywood, this represented an approximate value of t2 = 9.5 mm.

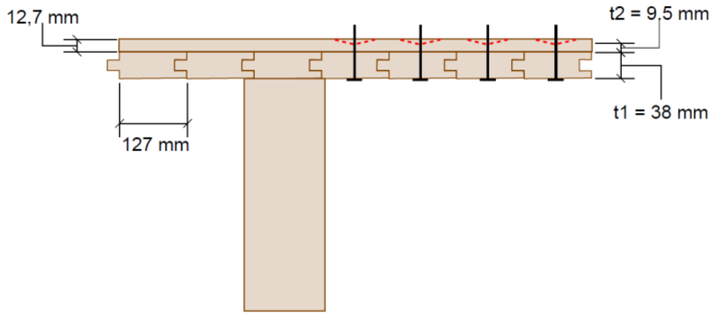


Figure 7. New nailing pattern performed from underneath of the roof structure.

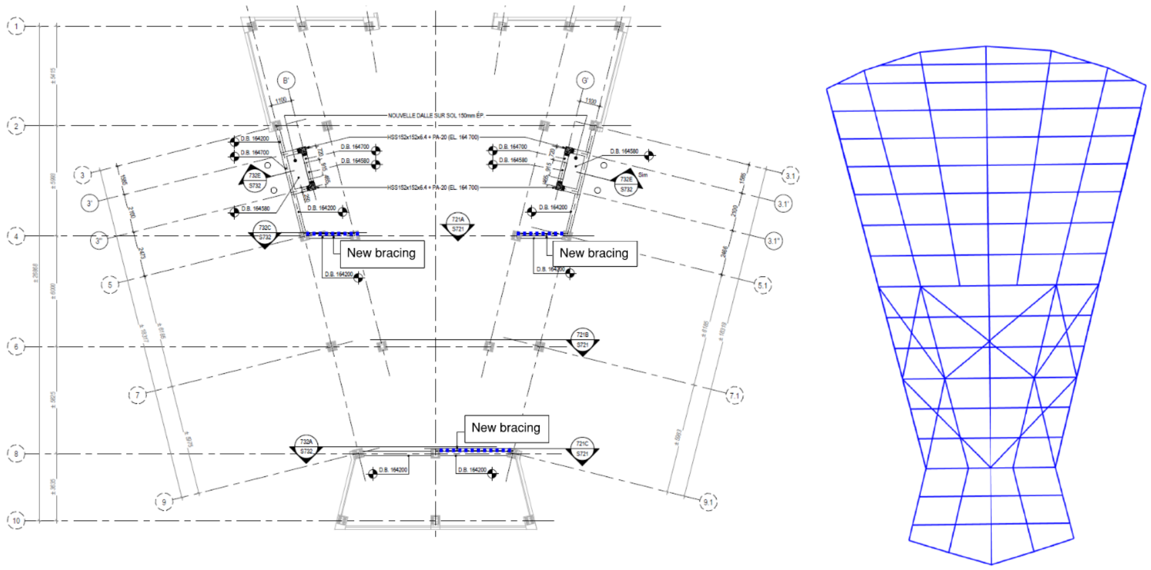
It should be noted that no experimental tests were performed to support these recommendations. They are based on specialized professionals understanding of current standards and on engineering judgement. This composite action, and the impact of nailing from the underneath, should be investigated by an experimental research program.

- Depending on the selected retrofit solutions studied, strategic collector beams were identified and designated to transfer the lateral loads from the diaphragm to the vertical bracing system.
- To retrofit the identified deficiency concerning the lack of a rigid connection between the diaphragm and the longitudinal collector beams, three solutions were studied:
  - Adding customized C shape connections between the collector beam and the wood deck.
  - Adding light-frame wood shear walls between the collector beam and the roof the wood deck.
  - Adding stiffeners and plates to the existing steel connections.

Implemented retrofit solution

The final implemented retrofit solution featured the following interventions:

- The add of three new vertical tension-only X bracings bays, see Figure 8.
- Customized C shape connections between the longitudinal collector beams and the wood deck to provide a rigid link.
- New foundation walls and footings under the new vertical tension-only X bracings to provide stability and shear capacity.
- New horizontal tension-only bracings under the roof, see Figure 8. For theses elements, it was especially important to assess the thermal stresses, as temperature in Quebec City features annual variations between -30°C and 30°C. These stresses were added to specified initial tension in the braces, to be added during the construction process, to ensure their proper activation during an earthquake event. The simultaneity of the initial tension and thermal stresses with seismic stresses was also studied.
- Some parts of the diaphragm are upgraded by increasing the composite action between the plywood and the wood deck through a new nailing pattern performed from the underneath.
- Some new collectors' elements are added and connected to the structure.



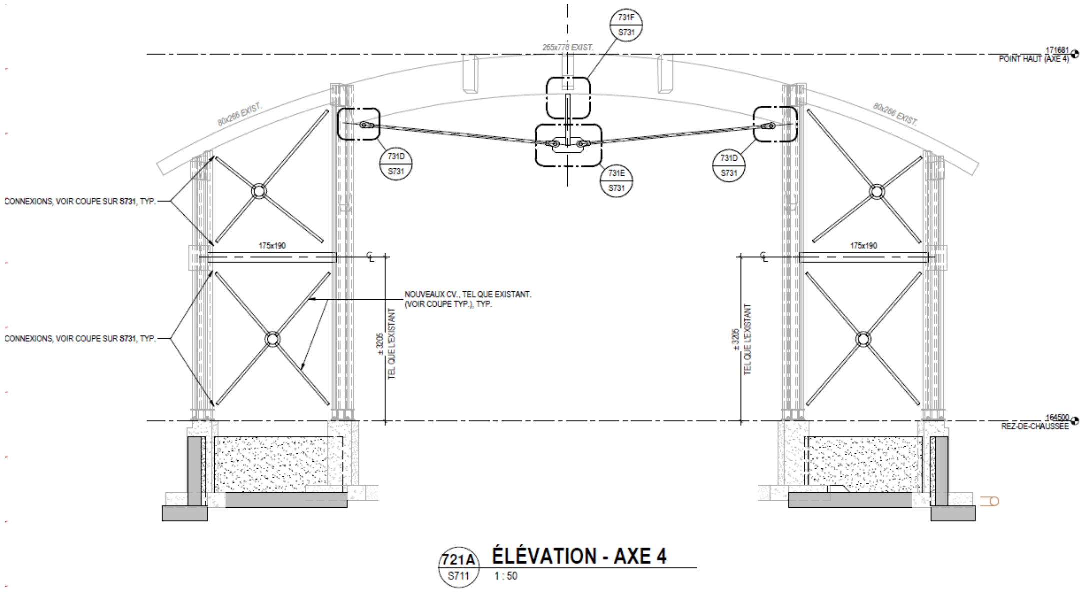
(a) *(b)*

Figure 8. Implemented seismic retrofit: (a) New vertical tension-only X bracings, (b) New horizontal tension-only bracings layout under the roof.

The seismic load redistribution from the diaphragm through the existing and new bracing system was ensured with multiple retrofit solutions for existing and new steel assemblies.

Design and modelling of the connections

The construction schedule would not allow for contractor's designed connections, as it would usually be the case in steel and glulam projects. Because of that, and because the existing timber frame had many specificities to consider, the structural engineer designed and modeled all the needed types of connections for this retrofit, see Figure 9.



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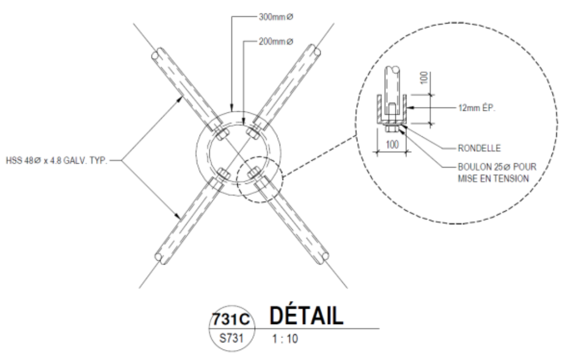

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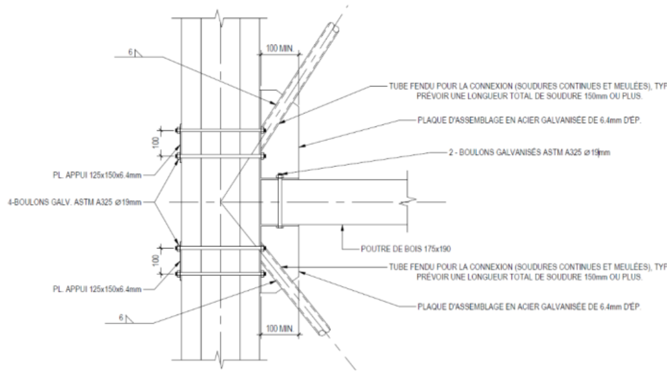

(c)

Figure 9. Detailed design and modelling of the connections (a) New vertical tension-only X bracings layout, (b) Steel rings, (c) New vertical tension-only X bracings center beam and column connection.

CONCLUSIONS

The retrofit solution adopted to meet the project's specific performance objectives at a "high protection" level is the integration of new vertical and horizontal tension-only bracings as well as various connections and diaphragm strengthening interventions. Challenges for the re-commissioning of this existing glulam building for exterior conditions as a giant gazebo were identified early in the design process and numerous structural analyses were performed to assess the compliance with the performance objectives established. This allowed the development of robust solutions and provided the best value for the Client. An innovative diaphragm retrofit solution by upgrading the composite action between the plywood and the wood deck through a new nailing pattern performed from the underneath was studied and designed. As this solution is not covered explicitly by Code, Standards or scientific data, specific design recommendations were made for future study and use.

It is to be noted that the authors found that there is a lack of support of scientific experimental results or former studies concerning complex modern wood diaphragms analysis and retrofit solutions. An experimental research program, including numerous testing protocols, would be interesting to test the composite action of wood diaphragms when paired with steel horizontal bracings members. Also, the specific composite action between plywood and wood decking, with different nailing patterns and beam conditions, should be investigated through thorough research and testing to better capture the behavior and capacity under seismic loads. This would provide support for engineering professionals in future retrofit projects.

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