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The Use of ASCE 41 for the Retrofit of Rubble Wall Buildings: A Case Study in a Moderate Seismic Zone

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ABSTRACT

Unreinforced masonry buildings erected at the beginning of the XXth century are known to exhibit poor seismic performance during major earthquakes. This is due to the limited ductility of the system and to the methods and materials used in construction. These buildings are often of historic importance, thus demolition is to be avoided and their seismic retrofit, according to current codes requirements, is required. In this paper, a discussion of the seismic retrofit of heritage buildings made of rubble walls, using ASCE 41, is presented by means of an existing building case study. It is to note that rubble walls are not explicitly covered in this code, which was complemented with guidance from European codes and research literature. The seismic performance of the existing building is evaluated using ASCE 41 linear dynamic procedure (LDP), for the as-built condition. The evaluation procedure and methodology developed to assess the performance are discussed. The determination of the material properties is also reviewed. The governing collapse mechanisms of the building are determined, and critical structural elements are identified. A retrofit strategy of the structural system is proposed, following ASCE 41 recommendations.

Keywords: Seismic Retrofit, ASCE 41, Rubble Walls, Case Study, Unreinforced Masonry

INTRODUCTION

Many of the buildings erected at the beginning of the XXth century were built with unreinforced masonry (URM) which is now known to exhibit poor seismic performance during major earthquakes due to the limited ductility of the structural system and to the methods and materials used in construction. At that time, the seismic hazard was also not considered in the design [1]. Consequently, those older buildings are often considered seismically deficient according to current design codes, which can trigger their retrofit, specifically when repurposed or modified. However, these buildings often are of significant heritage importance, which requires careful consideration to the type of retrofit implemented and its impacts on the building's value.

In 2020, Arup was given the structural engineering mandate to convert a portion of Montreal's emblematic site of the Royal Victoria Hospital into an innovative and collaborative space for study, learning and research in the field of sustainability. [2]. An overview of the future project and the layout of the site is shown in Figure 1, which notably includes the retrofit of three heritage buildings, to be used mainly as study spaces and offices. A new building is also to be designed, housing mainly laboratory and research spaces, and built along with a forecourt featuring a circular underground classroom.

The site is emblematic for the city of Montreal for its location on the southern slopes of Mont Royal, making its buildings visible from downtown Montreal, as well as its historic importance as one of the city's main healing places for around 100 years. The specific buildings included in the New Vic project stand out because they are the original buildings of the site, the first open-plan wards in Canada, and because of the many groundbreaking medical achievements associated to them [3].

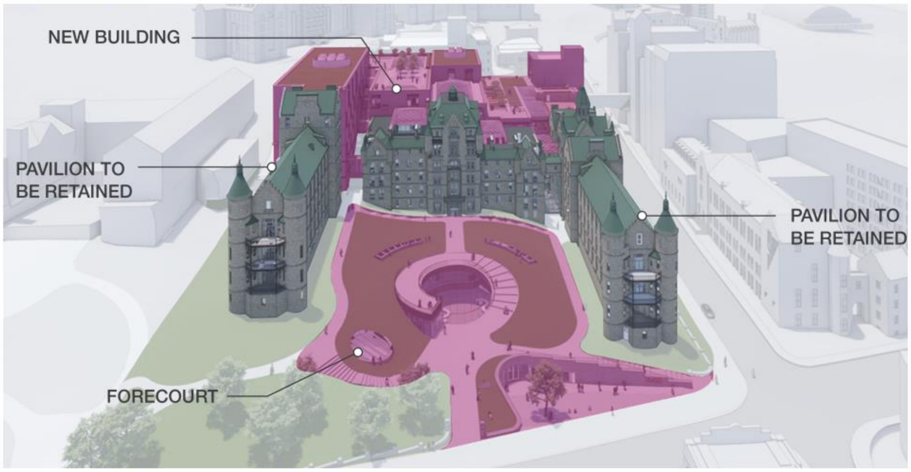

Figure 1. Overview of the New Vic Project, with the retained buildings, the new building and forecourt to be added.

This paper focusses on the analysis and retrofit of the structural elements of one of the heritage buildings, Pavilion L, a rubble wall structure, following ASCE 41 [4] and using seismic hazard compatible with NBCC 2010 [5]. This resulted in innovative retrofit strategies to preserve their heritage significance. Furthermore, in addition to the seismic evaluation according to ASCE 41, it is to note that all structural elements were also checked against gravity load combinations, but these will not be discussed.

RELEVANT CODES

Based on the location of the project and the start date of the design and construction, the applicable code is the NBCC 2010 as modified for Quebec, CCQ 2015 [6]. It is noteworthy that the CCQ 2015 requires the building to be retrofitted for at least 60% of the seismic design level of a new building.

The NBCC is a building code intended for new structures and has limitations when applied to the retrofit of existing buildings. It recognizes that other codes and guidelines, as well as engineering judgment, are required for the retrofit of structures. For the New Vic project, this is particularly true since the lateral load resisting system, rubble walls, is an archaic system not covered by most codes. Given that the more recent NBCC 2015 [7] specifically suggests the use of ASCE 41 for existing buildings, this approach was retained. However, ASCE 41 does not address stone rubble walls specifically, Arup's extensive international experience with heritage buildings of this type, as well as guidance from other codes, was used as a complement.

The ASCE 41 standard, using a performance-based seismic design approach, contains a comprehensive procedure for the evaluation and rehabilitation of seismically deficient building structures. The main procedure is composed of three different tiers, each one allowing a building to be deemed safe if all checks are passed, avoiding further study:

- Tier 1 is a screening procedure intended to rapidly evaluate a building based on checklists and simplified calculations.
- Tier 2 is a deficiency-based evaluation intended to further investigate all the potential deficiencies identified in the Tier 1 screening and to choose retrofit designs where applicable.
- Tier 3 is a systematic evaluation of the whole building used for evaluation and/or retrofit.

It is suggested to start with a Tier 1 screening to understand the building better, moving to a more thorough evaluation as required. However, Tier 3 is always accepted and includes linear and nonlinear, static and dynamic analyses.

BUILDING DESCRIPTION

The studied Pavilion L is shown in Figure 2. It has a narrow rectangular plan view, around 10 metres wide and 50 metres long. Its 4-storey high central portion, originally an open plan hospital wing, is completed by two round towers at the south end of the building, and a rectangular tower to the north which is 6-storeys high.

The lateral load resisting system relies on the perimeter stone walls. These are rubble walls built integrally with the stone façade, with cut stones on visible faces. They are typically between 600 and 800 mm wide, and this thickness is constant over the height of the building. The walls rest directly on competent rock. Diaphragms are mainly composed of wooden elements, and all will be replaced to accommodate the new use and fire rating requirement of the spaces. Note that horizontal diaphragms are present under the gabled roofs as well. New diaphragms will rely on concrete on steel deck and steel beams in all cases.

Figure 2. Current photo of Pavilion L as seen from the South-East.

In the early stages of the project, a more traditional retrofit approach was explored, relying on new concrete shear walls as vertical elements of the seismic force resisting system. As can be seen in Figure 3, this approach risked to seriously compromise the heritage value of the building, in particular its characteristic open plan. For this reason, alternative solutions were explored, relying on the capacity of the existing rubble stone walls, as well as their retrofit where necessary.

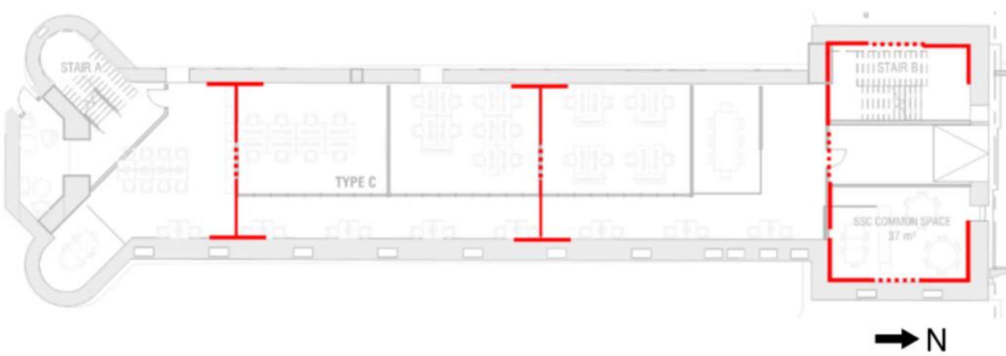

Figure 3. Preliminary design using concrete shear walls (in red).

PERFORMANCE OBJECTIVE AND SEISMICITY

A performance objective (defined in terms of seismic hazard level and seismic performance level) was selected in consultation with the client. The BPOE performance objective was retained: this corresponds to a collapse prevention performance level for an earthquake with a probability of occurrence of 5% in 50 years. It was demonstrated that this seismic hazard level is equivalent to the minimum retrofit level required by the CCQ 2015, 60% of values for new construction for the site in question by comparison of spectral acceleration values. Hence, this hazard level satisfies both local regulations and the client need.

Combining different codes should always be done with care, given potential compatibility issues. In this case, ASCE 41 was used, together with its companion codes. ASCE 41 refers to ASCE 7 [8] to determine the seismic ground motion values. However, seismic hazard maps available in the ASCE 7 do not cover Canadian seismicity. NBCC 2010 was used as a reference (note that spectral acceleration values for NBCC 2015 would have been slightly lower at the project's location, so this approach is conservative). A comparative study of acceleration values at several border locations where both ASCE 7 and NBCC 2015 values are available was performed to ensure compatibility of the Canadian and American ground motion values. From this comparison, it was demonstrated that the Canadian spectral accelerations, together with Canadian soil class modifiers can be used as inputs for the US codes. The soil class of the site is A, hard rock, and local soil modifiers were obtained by a bespoke geotechnical investigation. Then, using the definition of the level of seismicity given in ASCE 41, and the site-specific spectral accelerations, the 5% damped response spectrum used during the analysis was calculated, with short and long period accelerations, S_{D1} = 0.036 g and S_{DS} = 0.252 g. It is noteworthy that Montreal is generally classified as a moderate seismic zone, but the good local soil conditions allow the designation to be defined, according to ASCE 41, as low seismicity.

MATERIAL PROPERTIES

In order to evaluate seismically deficient URM buildings, ASCE 41 requires the use of two sets of specific material properties given by the characteristic and expected values. Characteristic properties are based on mean values of tested material properties minus one standard deviation. Properties specified in construction documents are taken as characteristic values unless stated otherwise. Expected material properties are based on mean values of tested material properties. ASCE 41 provides factors to convert from characteristic to expected values for each material covered in the code. The mechanical properties to be considered in the analyses depend on the type of action, either force-controlled or deformation-controlled, see the following sections.

Extensive mortar and stone testing allowed to identify the rubble walls as Montreal Limestone, of an average tested compressive capacity of 100 MPa or more, bonded primarily with non-hydraulic lime mortar, although some later repairs were made using Portland cement mortar mixes. ASCE 41 does not provide specific material properties for this type of structure. Furthermore, testing of assemblies is not practical, given the difficulty to replicate the on-site conditions. In order to determine the overall assembly characteristics, an extensive literature review [9-21], combined with the findings of the component testing, was used. Recognizing the inherent variability of this historic material, upper and lower bound characteristic strengths were retained for the analysis and design, as indicated in Table 1. Tabulated values for masonry for ASCE 41 were used to determine expected strengths from characteristics strengths when called for in the procedure. Figure 4 shows an example of the interior face of the rubble wall.

Table 1. Mechanical properties used for rubble walls

Rubble wall mechanical properties	Scenario A - Lower bound	Scenario B – Upper bound
Compressive strength - f'_m	7.0 MPa	12.0 MPa
Youngs Modulus - E	3.5 GPa	12 GPa
Shear Modulus - G	2.8 GPa	4.8 GPa

In both cases the shear strength is assumed to be 0.05 + 0.5σ, where σ is the normal compressive stress in MPa. The maximum assumed value for the shear strength was 1.4 MPa. The unit weight of the masonry was taken as 26.5 kN/m^3.

Figure 4. View of the interior face of a rubble wall observed on site.

SELECTED EVALUATION PROCEDURE

As a first step, the Pavilion L was evaluated for the Tier 1 requirements at the beginning of the project, and determined to have five deficiencies: geometric, mass, and torsion irregularities, unknown/missing information regarding the diaphragms and connections, and the quick shear stress check of ASCE 41 was not passing for all URM walls. Based on these findings and limiting conditions for the use of the Tier 2 procedure, a Tier 3 detailed evaluation was then performed, using the linear dynamic procedure as well as further knowledge of the site gained through exploratory openings and testing.

To perform a Tier 3 evaluation, an acceptance criterion for each structural element is calculated. Failure mechanisms of structural elements are to be categorized as deformation-controlled actions (i.e., ductile behaviour failure mechanisms) and force-controlled actions (i.e.: non-ductile behaviour failure mechanisms). For deformation-controlled actions, the acceptance criterion is given by:

$$\mathrm{m}\kappa Q_{\mathrm{CE}} = Q_{\mathrm{UD}} \tag{1}$$

where m is a ductility factor that reflects the permissible ductility of the components, which depends on the component properties and the performance level, κ is the knowledge factor, Q_{CE} is the component expected lateral strength, and Q_{UD} is the deformation-controlled action caused by gravity loads and earthquake forces, calculated in accordance with ASCE 41. The m values for unreinforced masonry given in ASCE 41 were retained for the analysis. A knowledge factor of 1.0 is assumed because of the extensive examination and testing to be completed on the building before final design.

For force-controlled actions, the acceptance criterion is given by:

$$\kappa Q_{\mathrm{CL}} = Q_{\mathrm{UF}} \tag{2}$$

where Q_{CL} is the component lower-bound lateral strength, and Q_{UF} is the force-controlled action caused by gravity loads in combination with earthquake forces. Values of Q_{UD} and Q_{UF} on each structural element were computed following ASCE 7 load combinations.

STRUCTURAL ELEMENTS SPECIFIC EVALUATION PROCEDURE

ASCE 41 defines structural members as those that affect the lateral stiffness or distribution of the forces, or are loaded because of lateral deformations. Elements need to be considered even if they are not intended to be part of the seismic force resisting system. They are classified as primary or secondary as follows:

- Primary: elements required to resist the seismic forces and accommodate seismic deformations. Need to be evaluated against seismic and gravity forces, as well as deformations.

- Secondary: elements required to accommodate the seismic deformations, but which do not resist forces. Need to be evaluated against seismic deformations with gravity loads.

The stiffness and resistance of only the primary components need to be included in the mathematical model for linear methods. However, when classifying elements as secondary, it needs to be verified that the stiffness of secondary elements does not exceed 25% of the stiffness of the primary components.

Wall piers

Rubble walls are evaluated against five primary in-plane actions according to ASCE 41. Deformation-controlled actions are rocking and bed-joint sliding (initial and final strength). Force-controlled actions are toe crushing, diagonal tension, and vertical compression. Figure 5 shows illustrations of the mentioned failure modes, except for the vertical compression failure. It is to note that bed-joint sliding includes stair-step cracking through head and bed joints and can also present itself as a diagonal crack. This type of failure also includes two checks. The initial bed joint sliding strength is based on the mortar bond and the friction, while the final capacity accounts for degradation of the wall and is based only on friction.

It is to note that given the performance objective and the height to thickness ratios of the walls, out-of-plane failure of the walls was deemed not to be critical. Furthermore, the contribution to the in-plane capacity of wall flanges was ignored in the calculations as a simplification. It has been demonstrated that flanges contribute positively to the capacity [22], and as such, this approach is conservative.

One of the important parameters in the capacity equations for the wall failure mechanisms is the effective height (h_{eff}). This parameter represents the height at which the shear from seismic forces is applied. It can vary within the same wall assembly based on geometry since it depends on opening locations. The definition of the effective height is further explained in the commentary of ASCE 41, and a way to calculate it can be summarized in Figure 6, approach used for the present study.

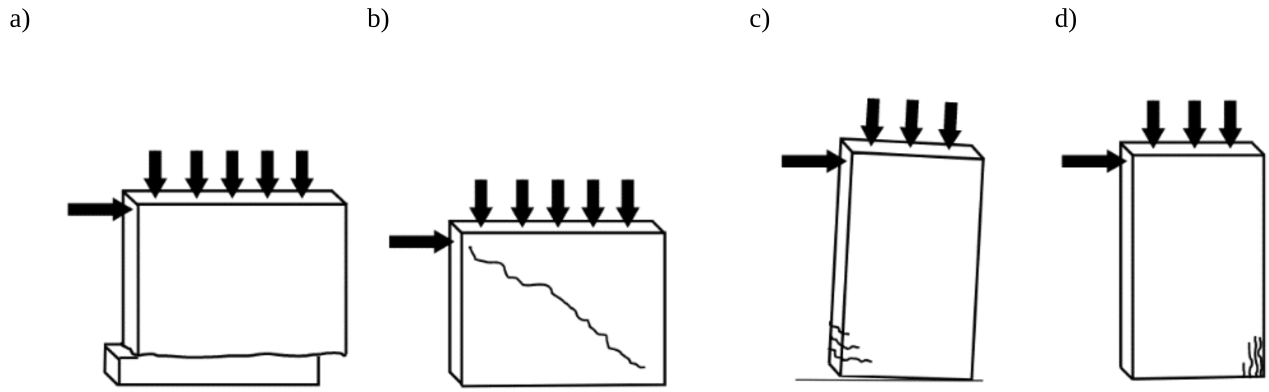

Figure 5. In-plane failure modes of a laterally loaded URM wall: a) bed-joint sliding; b) diagonal-tension; c) rocking; and d) toe-crushing.

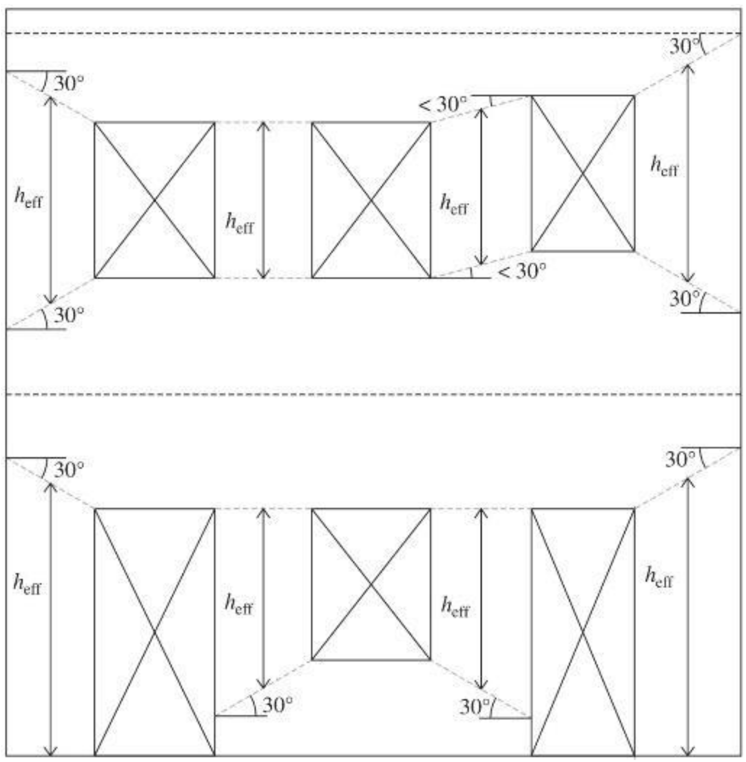

Figure 6. Figure C11-5 from ASCE 41 demonstrating the effective heights of wall piers. [4]

Spandrels

ASCE 41 lists four failure modes for spandrels: each spandrel is analyzed against its expected capacity, which is the lesser of its flexural strength and shear strength. For both cases, there is the peak strength (prior to spandrel cracking) and the residual strength (after the spandrel is cracked). Two types of spandrels are identified for the project: rectangular spandrels, typically with wood lintels, and spandrels with shallow arches.

One critical value in the determination of the capacity of unreinforced masonry spandrels is p_{sp}, the axial stress in the spandrel under seismic loading [23]. As the walls are well distributed throughout the building and diaphragm reinforcement results in minimal collector beam forces, the primary source of axial stress in the spandrels in this structure was determined to be the horizontal component of compressive struts transferring shear between piers under seismic displacements. Although cracking of spandrels caused by deformation can lead to an increase in axial stresses from elongation confined by adjacent walls, this effect was conservatively neglected due to the difficulty in determining precise values under seismic loading. As a conservative approach, p_{sp} was neglected for peak flexure and shear capacities (before cracking could lead to the development of confinement forces) and calculated from the horizontal component of diagonal compressive struts transferring shear for residual behaviours. An upper limit corresponding to the diagonal tension failure of the compressive strut was also applied as proposed in ASCE 41.

Diaphragms

ASCE 41 defines the different elements that compose a diaphragm, including chords, collectors and ties, as well as the retrofit procedures for each of those elements, depending on the material they are made of. However, for the studied pavilion, all floors required to be replaced, so no evaluation was needed and design followed standards for new construction. Forces for the design of diaphragms and their connections, however, were taken from ASCE 41. In the case of performing a LDP, results in diaphragm forces can be obtained directly, with a minimal value of 80% of forces that would be obtained using the linear static procedure. Minimum values are also given for out-of-plane anchorage of walls to the new diaphragm. It is noted that, as is typical in construction of this period, no positive connection between the diaphragm and walls was present in the original design. This was remediated in the design of the new floors.

MODELLING

The seismic analysis of the building was performed with the commercial computer software CSI ETABS [24]. Figure 7 shows the geometry of the model for Pavilion L, with a focus on wall piers and spandrels (floors hidden for clarity).

Two finite element models were used for the lateral analysis and design of the existing wall piers, using the upper and lower bound material properties previously defined. Forces were extracted from the upper bound model and confirmed to be conservative since this model generates the highest forces due to the stiffer structure, something that was not offset by the higher masonry compressive strength used for this case. The lower bound model was used to determine the building drifts since it is more flexible. For both cases, limiting conditions for the use of the LDP procedure were verified as required, given the building's irregularities, and found to be satisfactory.

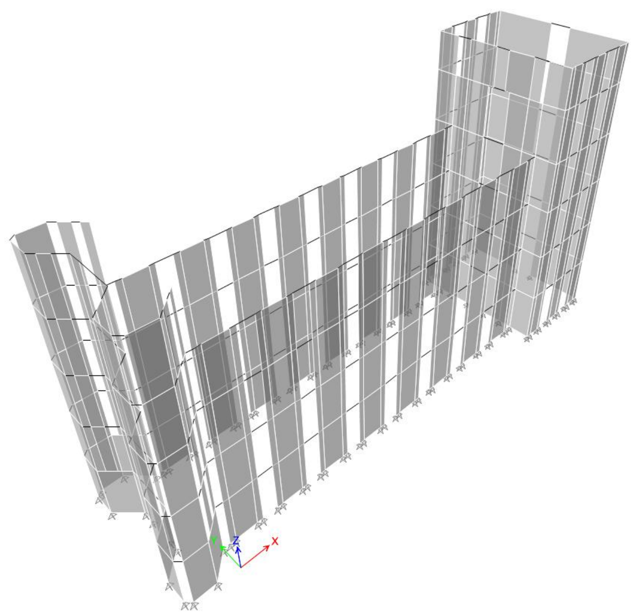

Figure 7. 3D view of the model proposed to perform the seismic analysis of the building.

In both models, the URM walls were modelled as finite element shells neglecting their out-of-plane stiffness. The URM spandrels, typically above window or door openings, were modelled as beam elements with fixed ends connecting to the wall piers. Floor slabs were included as a non-composite floor system, on which superimposed gravity loads were applied as uniform loads.

In agreement with ASCE 41, the effective seismic weight was calculated as the total dead load and 20% of the snow load. The foundations of the buildings were modelled as fixed (no soil-structure interaction was considered) as the rock on which the building is erected is of high quality. This approach is accepted by ASCE 41 for linear models. Finally, viscous damping corresponding to 2% of the critical damping was specified.

As the building has some irregularities, multidirectional seismic effects were considered according to the requirements of a LDP following ASCE 41 by combining 100% of the earthquake in one direction with 30% in the perpendicular direction acting simultaneously. Additionally, given than accidental torsion of the building was demonstrated to be less than 25% of actual torsion, accidental torsion was ignored, as permitted by ASCE 41.

The results obtained from the analysis were post-processed using in-house developed workflow, allowing for an efficient comparison against all the failure modes described above for the large number of piers and spandrels evaluated.

ANALYSIS RESULTS

Following the analysis, retrofit strategies were designed for the failing wall piers only, as specified by ASCE 41. Failure mechanisms of each pier can either be in shear, bending or compression. The latter was shown not to govern: given the thickness of the existing walls, utilization ratios for axial compression were low with maximum values of approximately 60%. Shear failures include sliding shear or diagonal tension. Bending failures include rocking and toe crushing. From the analyses conducted, the wall piers in this case study requiring retrofit all failed in bending (rocking and toe crushing). No shear failures were observed (sliding shear of diagonal tension). Note that out-of-plane failures, as mentioned before, were deemed not governing the design, based on limiting slenderness rations proposed by ASCE 41. Figure 8 displays a plan view identifying the failing piers, where the numbers represent the number of storeys over which the indicated wall pier is failing (note this is from the top down, given that bending failures were closely linked with the lack of gravitational axial load to counteract tension forces in the wall at higher levels).

It is noted that the final sliding shear check was disregarded in our analysis. FEMA 306 [25], precursor of ASCE 41 and containing more details about the approach, states that significant strength degradation has been observed at storey drifts of 0.3-0.4% which are likely to correspond to complete erosion of bond capacity. Resistance is then based on friction only. Furthermore, FEMA 306 specifies that the final sliding failure is only applicable for drifts in the 0.4-0.8% range. It follows that if the drifts are very small (under 0.3%), this failure mechanism is not expected. The above discussion is also in line with drift limits recommended by ASCE 41, which are of 0.75% for bending-controlled failure modes and 0.4% for shear-controlled failure modes. The building in presented in this paper demonstrated drifts well below this limit, and it was deemed appropriate to ignore the sliding final action in the analysis.

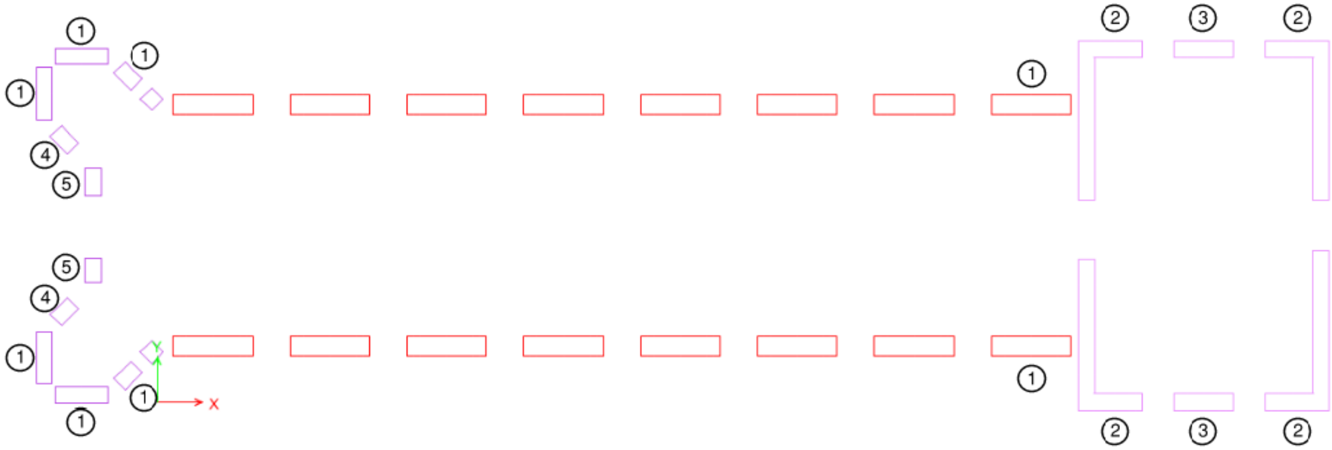

Figure 8. Map of the failing piers of the pavilion analyzed (note that the beams and floors are hidden for clarity).

RETROFIT

Based on the identified bending failure modes, two retrofit strategies were used to improve the bending capacity of the retrofitted piers: single-sided steel wall plates (or steel angles for corner walls), and grouted vertical steel reinforcement and anchorage bars, centred on the wall's thickness [26], installed at both extremities of the failing piers providing tension reinforcement under bending actions. Note that the use of a single-sided approach was necessary, since the exterior, exposed façade face of the walls must remain unaltered. Both these retrofit strategies were selected because they are minimally invasive from an architectural point of view and lend themselves well to the heritage aspirations of the building. Given that they are relatively novel in the local construction market, their technical and construction merits were evaluated in detail.

A literature review of previous research demonstrated that the use of one-sided reinforcement of different types improves the capacity of URM walls [27-39]. Steel strips in particular showed an improvement in tested shear and lateral strength in the order of 30-80% when compared to unreinforced masonry walls. The disadvantage of the method is that the eccentricity introduced by the placement of the reinforcement will induce out-of-plane torsional moments, which leads to out-of-plane bending in some diagonal compression tests. Post-installed vertical rebar or anchor systems have been successfully applied to the retrofit of several heritage URM buildings in Canada and internationally, notably the retrofit of Ottawa's Parliament Hill Central Block and Library buildings, also stone buildings. While post-installed vertical anchors do not induce out-of-plane effects because the retrofit is centred on the cross-section, as well as having the advantage of being invisible after retrofit is completed and thus even less invasive from a heritage perspective, their cost was demonstrated to be significantly higher than for the plate retrofit. This cost is mainly associated to the careful operation needed to drill vertically into the wall over several metres.

Figure 9a shows a schematic detail for a steel plate retrofit at one end of the wall pier, viewed in plan. The use of steel plates is the preferred approach for most of the walls because it is more cost-effective than the anchor system, and the plates are simpler to install, as discussed above. For the case studied, it was demonstrated that the moment generated because of the eccentric retrofit is relatively small and can be resisted by out-of-plane bending in the walls, resolved into axial tension and compression forces in the diaphragms.

Figure 9b shows a 3D view of Pavilion L, with vertical anchors represented as dotted lines. Vertical anchors were used only in the southern towers of the studied pavilion, because of their curved shape, that would make the installation of the steel plates more complex, as well as the limited connectivity between the main diaphragms and the towers.

It is to note that, given the quality of the rock on site, it was found that no retrofit was needed for the foundations of the building.

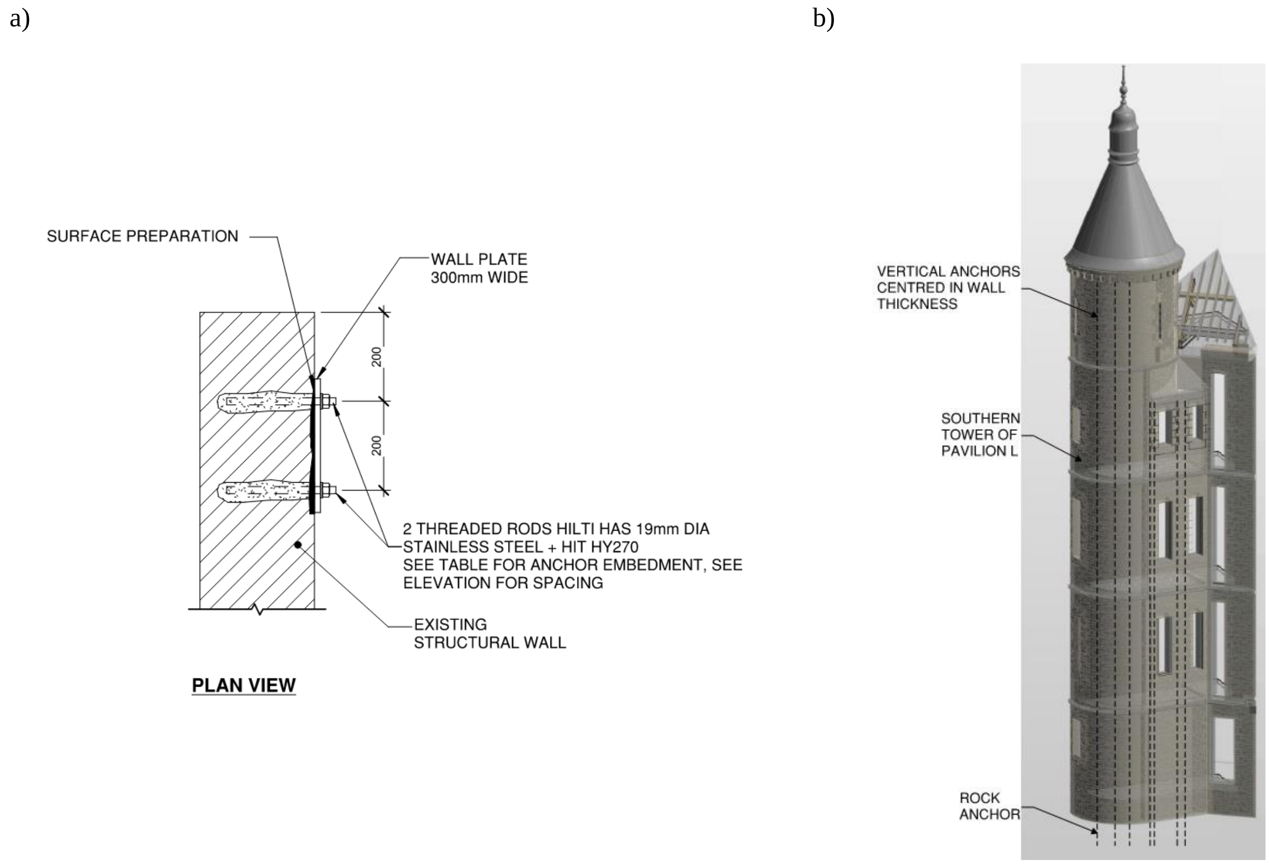

Figure 9. Rubble wall retrofit strategies considered: a) steel wall plate (typical plan detail); and b) schematic view of vertical anchors.

Analysis of the spandrels showed that most of them do not have sufficient capacity to resist seismic loads. Further analysis demonstrated that, even with the stiffness of the spandrels removed entirely from the models, drifts remained below the acceptable limits and forces on the wall piers decreased. It was therefore deemed conservative to analyze the piers with forces determined from the analysis including spandrels, while reinforcing them only to remain stable even after significant cracking under seismic deformations would occur. Steel lintel reinforcement details were developed to resist the seismic movements of the structure along with gravity loads imposed by the spandrels (self-weight of the spandrel and superimposed loads, including those from floor beams supported by the spandrels). It is noted that while ASCE 41 permits the evaluation of existing lintels, these are primarily composed of wood sections with insufficient capacity or ductility, that also require replacement for fire safety. For this reason, the reinforcement relies on new steel lintels, as can be seen in Figure 10 showing one of the typical retrofit details developed.

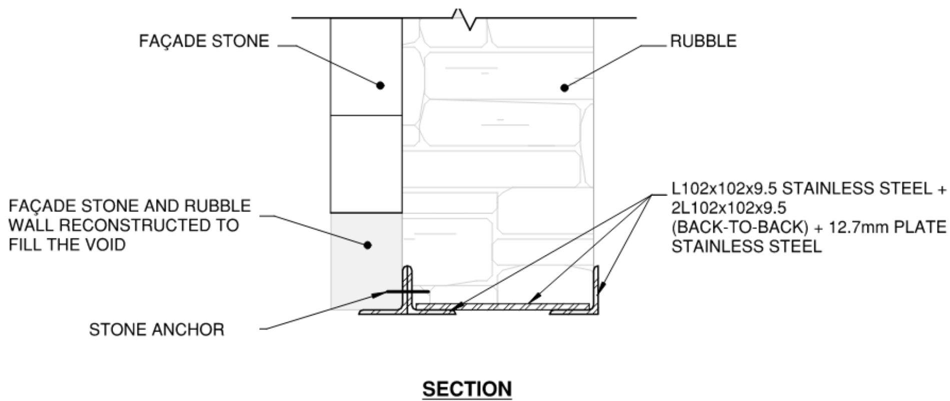

Figure 10. Spandrel retrofit strategy: post-installed steel lintels (typical detail).

CONCLUSIONS

This paper presents a case study of the seismic evaluation and retrofit of rubble wall buildings in a Canadian context, following the ASCE 41 methodology for unreinforced masonry. Regarding the application of the ASCE 41 procedure for URM buildings, the following conclusions can be drawn:

- ASCE 41 proved to be an appropriate and valid method for seismic evaluation of URM buildings in Canada with a more comprehensive framework compared to the use of the NBCC, intended primarily for new buildings.
- For the studied case, the use of ASCE 41 allowed for a retrofit solution compatible with the heritage aspirations of the buildings by avoiding a façade retention approach, common in Montreal, and by allowing the conservation of the building's characteristic interior open plan. Despite the fact that the results obtained are not necessarily transferable to high-seismicity zones, historic heritage buildings in Canada are mainly located in low to moderate seismic zones, typical of Eastern Canada and as such the findings of this case study might be applicable to a significant building stock of similar characteristics.
- Material properties for rubble walls similar to those found on site are difficult to determine. Available literature and site-specific characterization tests must be performed to obtain reliable values. Given the limitations of testing rubble stone walls, it is suggested to use a parametric approach to determine the impact of variation of the material properties.
- A majority of the walls of Pavilion L passed the acceptance criteria defined in ASCE 41 procedure, and hence meet the collapse prevention performance level under the considered earthquake without retrofit. About 20% of the piers did not meet the acceptance criteria, failure being governed by bending behaviour, that is rocking and toe-crushing for all of them. Minimally invasive retrofit of walls, adding steel plates and anchors to increase the tension capacity of the piers, allowed to assure a retrofit that is code compliant and in line with the heritage constraints of the evaluated building.
- Due to the difficulty in establishing some of the required input values, the retained approach for the evaluation of spandrels considered boundary cases representing outcomes in which the spandrels either resisted seismic loads or deteriorated to the point they would not contribute to the lateral stiffness of the building. The final retrofit strategy ensures life safety by providing vertical support for the rubble wall spandrels even if extensively cracked, and it was demonstrated that the most conservative scenario was retained for the determination of wall pier forces.

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REFERENCES

[1] Mitchell, D., Paultre, P., Tinawi, R., Saatcioglu, M., Tremblay, R., Elwood, K., and DeVall, R. (2010). Evolution of seismic design provisions in the National building code of Canada. *Canadian Journal of Civil Engineering*, 37(9), 1157-1170.

[2] McGill University. https://newvic.mcgill.ca/the-new-vic/, consulted on March 15, 2023

[3] Terry, N. (1994). *Royal Vic: The Story of Montreal's Royal Victoria Hospital, 1894-1994*. McGill-Queen's University Press, Montreal, Canada.

[4] American Society of Civil Engineers - ASCE. (2017). *ASCE 41-17, Seismic Evaluation and Retrofit of Existing Buildings*. Prepared by the ASCE, Reston, VA, USA.

[5] National Research Council of Canada – NRCC. (2010). *National Building Code of Canada (NBCC), 12th ed*. Prepared by the NRCC, Ottawa, ON, Canada.

[6] National Research Council of Canada – NRCC. (2023). *Code de construction du Québec, Chapitre I : Bâtiment, et Code national du bâtiment : Canada 2015 (modifié)*. Prepared by the NRCC, Ottawa, ON, Canada.

[7] National Research Council of Canada – NRCC. (2015). *National Building Code of Canada (NBCC), 13th ed*. Prepared by the NRCC, Ottawa, ON, Canada.

[8] American Society of Civil Engineers - ASCE. (2016). *ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Prepared by the ASCE, Reston, VA, USA.

[9] Howe, J. A. (Ed.). (2016). *Geology of building stones*. Routledge.

[10] ASTM. (2015). ASTM Standard C568/C568M. *Standard Specification for Limestone Dimension Stone*. ASTM International, West Conshohocken, PA, 2003. doi:10.1520/C0568_C0568M-15.

[11] Goudge, M. F. (1933). *Canadian Limestone for Building Purposes* (No. 733). FA Acland, printer to the king.

[12] Department of Transport, 2001. BD21/01 The Assessment of Highway Bridges and Structures, Design Manual for Roads and Bridges, Volume 3, Section 4, Part 3,London: HMSO.

[13] British Standard Institute-BSI (1992). *BS 5628-1:1992 Code of practice for the use of masonry*. Prepared by the BSI, London, UK.

[14] Canadian Standard Association - CSA (2014). *CAN/CSA S304-14 Design of masonry structures*. Prepared by the CSA, Toronto, ON, Canada.

[15] Vasconcelos, G. and Lourenço, P. B. (2006). Assessment of the in-plane shear strength of stone masonry walls by simplified models. *Proceedings of the 5th International Conference Structural Analysis of Historical Construction*. New Delhi, India. Edited by: Lourenço, P., Roca, P., Modena, C. and Agrawal, S. November. pp.6–8. New Delhi, India: Macmillan India, Ltd

[16] Milosevic, J., Gago, A. S., Lopes, M., & Bento, R. (2013). Experimental assessment of shear strength parameters on rubble stone masonry specimens. *Construction and Building Materials*, 47, 1372-1380.

[17] Angelillo, M., Lourenço, P. B., & Milani, G. (2014). Masonry behaviour and modelling. *Mechanics of masonry structures*, 1-26.

[18] British Standard Institute-BSI (2005). *NA to BS EN 1996-1-1 : 2005 UK National Annex to Eurocode 6: Design of masonry structures — General rules for reinforced and unreinforced masonry structures.* Prepared by the BSI, London, UK.

[19] National House Building Council Foundation – NHBC Foundation. (2008). *The use of lime-based mortars in new build*. NF12. NHBC Foundation, Amersham. UK. ISBN 978-1-84806-066-1

[20] British Standard Institute-BSI. (2005). *BS 5628-1: Code of practice for use of masonry-Part 1: Structural use of unreinforced masonry*. Prepared by the BSI, London, UK.

[21] Network Rail. (2006). *The Structural Assessment of Underbridges*. Network Rail, London. NR/GN/CIV/025.

[22] Moon, F. L., Yi, T., Leon, R. T., & Kahn, L. F. (2006). Recommendations for seismic evaluation and retrofit of low-rise URM structures. *Journal of Structural Engineering*, 132(5), 663-672.

[23] Beyer, K. (2012). Peak and residual strengths of brick masonry spandrels. *Engineering Structures*, 41, 533-547.

[24] Computers and Structures, Inc - CSI. (2019). ETABS Ultimate, Version 19. Berkeley, California, USA

[25] Applied Technology Council - ATC. FEMA 306 (1998) *Evaluation of earthquake damaged concrete and masonry wall buildings*, ATC-43 Project for The Partnership for Response and Recovery. Prepared by Applied Technology Council Washington, D.C., USA.

[26] Bilson, T., & Kirkpatrick, S. W. (1990). Cintec anchors for historic biodam bridge. *International Journal of Construction Maintenance and Repair*, 4(3).

[27] Darbhanzi, A. et al. (2020). Improving in-plane seismic response of unreinforced masonry walls using steel strips. *Advances in Structural Engineering*, 23(12), 2709–2723.

[28] Farooq, S. H., Shahid, I., & Ilyas, M. (2014). Seismic performance of masonry strengthened with steel strips. *KSCE Journal of Civil Engineering*, 18, 2170-2180.

[29] Jafari, A., Oskouei, A. V., Bazli, M., & Ghahri, R. (2018). Effect of the FRP sheet's arrays and NSM FRP bars on in-plane behavior of URM walls. *Journal of Building Engineering*, 20, 679-695.

[30] Arifuzzaman, S., & Saatcioglu, M. (2012). Seismic Retrofit of Load Bearing Masonry Walls by FRP sheets and Anchors Sheets and Anchors. In *Proceedings of the 15th World Conference on Earthquake Engineering*.

[31] Banerjee, S., Nayak, S., & Das, S. (2020). Shear and flexural behaviour of unreinforced masonry wallets with steel wire mesh. *Journal of Building Engineering*, 30, 101254.

[32] Guerreiro, J., Proença, J., Ferreira, J. G., & Gago, A. (2018). Experimental characterization of in-plane behaviour of old masonry walls strengthened through the addition of CFRP reinforced render. *Composites Part B: Engineering*, 148, 14-26.

[33] Moeini, M. E., Razavi, S. A., Yekrangnia, M., Pourasgari, P., & Abbasian, N. (2022). Cyclic performance assessment of damaged unreinforced masonry walls repaired with steel mesh reinforced shotcrete. *Engineering Structures*, 253, 113747.

[34] Shabdin, M., Attari, N. K., & Zargaran, M. (2018). Experimental study on seismic behavior of Un-Reinforced Masonry (URM) brick walls strengthened with shotcrete. *Bulletin of Earthquake Engineering*, 16(9), 3931-3956.

[35] Türkmen, Ö. S., De Vries, B. T., Wijte, S. N. M., & Vermeltfoort, A. T. (2020). In-plane behaviour of clay brick masonry wallettes retrofitted with single-sided fabric-reinforced cementitious matrix and deep mounted carbon fibre strips. *Bulletin of Earthquake Engineering,* 18(2), 725-765.

[36] Borri, A. et al. (2008) “Consolidation and reinforcement of stone walls using a reinforced repointing grid” Structural Analysis of Historic Construction – D’Ayala & Fodde.

[37] Borri, A., Corradi, M., Speranzini, E., & Giannantoni, A. (2008). Consolidation and reinforcement of stone walls using a reinforced repointing grid. In *Structural Analysis of Historic Construction: Preserving Safety and Significance-Proceedings of the 6th International Conference on Structural Analysis of Historic Construction*, SAHC08 (Vol. 2, pp. 981-989). Taylor & Francis Group, London, UK

[38] Borri, A., Castori, G., Corradi, M., & Sisti, R. (2014). Masonry wall panels with GFRP and steel-cord strengthening subjected to cyclic shear: An experimental study. *Construction and Building Materials*, 56, 63-73.

[39] Corradi, M., Di Schino, A., Borri, A., & Rufini, R. (2018). A review of the use of stainless steel for masonry repair and reinforcement. *Construction and Building Materials*, 181, 335-346.