SEISMIC ANALYSIS OF THE WEST BLOCK PARLIAMENT BUILDING IN OTTAWA – CHALLENGES AND APPROACH

J. Krisanova[1], N. Ojdrovic[2] and S. Brzev[3]

[1] P. Eng., Senior Structural Engineer, Ojdrovic Engineering Inc., Toronto, ON, Canada, Nebojsa.ojdrovic@ojdrovic.com

[2] Ph.D., P. Eng., Principal, Ojdrovic Engineering Inc., Toronto, ON, Canada, jane.krisanova@ojdrovic.com

[3] Ph.D., P. Eng., Faculty, Dept. of Civil Engineering, British Columbia Institute of Technology, Burnaby, BC, Canada, Svetlana_Brzev@bcit.ca

ABSTRACT

The West Block Parliament Building in Ottawa is currently undergoing major rehabilitation and expansion. The building was built in three construction phases from 1860 to 1909, and has subsequently undergone a series of changes and partial rebuilding. The last significant renovation was completed in the 1960s.

Each of the first three phases of building construction used similar wall composition and structural assembly. Heavy cut stone-faced rubble core masonry walls form the main vertical load bearing system. The original floors consist of brick or terracotta arches on iron beams and Fox and Barret system. The roof framing consists of wrought iron and steel trusses, with terracotta, wood, or precast concrete plank infill. A number of contemporary concrete slab floors on steel beams were introduced as a part of the renovation performed in the 1960s.

The Public Works and Government Services Canada (PWGSC) Policy "Seismic Resistance of PWGSC Buildings" and the National Building Code of Canada 2010 form the basis for the seismic evaluation and upgrading of the West Block building.

This paper describes the key building features and discusses challenges associated with the seismic analysis of this complex heritage building. It describes as found condition, materials and construction practices of the construction period, and subsequent modifications that impacted the design and structural features. The methods used in seismic analysis and approach to various modelling issues are discussed.

KEYWORDS: heritage buildings, stone masonry, brick masonry, seismic analysis, retrofit, rehabilitation

INTRODUCTION

The Parliament Building complex is located at Parliament Hill, Ottawa, and it is a home to Canada's federal legislature and a significant national historic landmark. The West Block is one of the three Parliament Hill buildings that form a national historic site, along with the East Block and Centre Block [1]. Before it was vacated in 2011, the West Block housed offices for the ministers, members of Parliament and employees; committee rooms; and an important ceremonial feature – the Confederation Room. Once the rehabilitation is completed in 2017, the building will serve as a seat of Canadian Government, temporarily hosting the House of

Commons, during the rehabilitation of the Centre Block. It will also regain some of its core functions such as housing offices and committee rooms.

The first phase of the West Block construction took place from 1859 to 1865 in traditional Gothic Revival style. It consisted of South Wing, East Wing and a short return portion on the west. In 1875, the West Wing was added, together with monumental MacKenzie tower. In 1906 a smaller north wing was added to connect the end of East Wing with the building on the west, thus forming a closed rectangular courtyard. The last major renovation of the building performed in the 1960s resulted in substantial changes to the North-West wing, removal of a number of interior masonry shear walls, and replacement of some of the original floors.

Maintenance of the building envelope over the last fifty years was inconsistent. At the end of the 20th century it was evident that the deterioration had reached the level when a major overhaul is required. An ambitious architectural rehabilitation program for the building is based on the Long Term Vision and Plan for the entire Parliament Hill. According to the program, the West Block courtyard will provide a space for the temporary House of Commons, which will be covered with structural steel and glass enclosure. The existing West Block is currently undergoing a major rehabilitation that will include seismic strengthening and retrofit of the entire building. The rehabilitation of the South East Tower performed in 2005 to 2008 was the first pilot project for the interventions expected to be used on the remainder of the West Block Project. The interventions implemented on the South East Tower were reviewed and modified as needed for the second pilot project, the North Towers Renovation performed in 2008 to 2011. The work on these two pilot projects provided a wealth of information on original construction methods and practices, and allowed for refinement of the design and proposed construction procedures which will be implemented in the West Block rehabilitation. The design team Jokinen + Ojdrovic Engineers in Joint Venture (including the first and second author of this paper) received Preservation of a Heritage Building Award from Canadian Association of Heritage Professionals in 2010.

ORIGINAL BUILDING CONSTRUCTION - KEY FEATURES

After the completion of all three construction phases, the final building shape resembles upside down letter “P”. An aerial view of the building and a typical floor plan are shown in Figure 1, showing South-East Tower enclosed in white tarp. Overall dimensions in plan are approximately 80 m and 115 m in East-West and North-South directions respectively. The courtyard dimensions are 43 by 40 meters. The north extension of the building is 43 m long. The building is approximately 20 m high with four storeys and a relatively low attic space. The MacKenzie Tower is approximately 83 m high, with the 35 m high steel roof.

The building is founded on bedrock. The strip footings consist of larger, relatively squared stones laid in mortar, and rubble core. Footings for the MacKenzie Tower consist of large squared and dressed stone with depths typically equal to the footing width. The footings typically extend by 200 mm from the wall face, except for the MacKenzie Tower, where several 200 mm stepped projections exist.

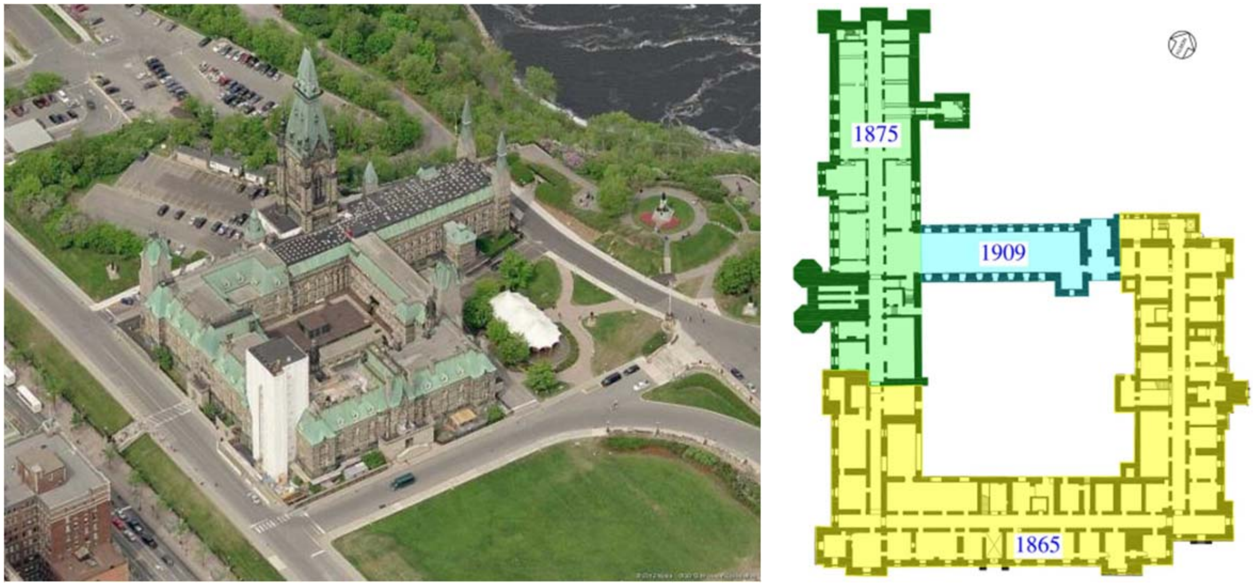

Figure 1: Aerial View of the West Block and a Typical Floor Plan

The structural frame consists of loadbearing stone and clay brick masonry walls that support variety of floor systems, including Fox and Barett system, jack arch system, and reinforced concrete slabs. Fox and Barret floor system consists of rolled iron joists which carry wooden boards resting on the lower flanges with poured concrete on top, such that the iron beams are deeply encased in it. This system was mainly used for corridors, and due to its state of deterioration and inferior load carrying capacity, it will be largely removed and replaced with composite concrete on steel deck floor. Jack arch floor system consists of hollow or solid clay block or brick arches resting on rolled iron beams which are supported by masonry walls. The floor system built in the 1960s consists of reinforced concrete slabs supported on rolled steel beams. This is the predominant floor system today. The slabs are typically poured against existing walls, sometimes with pockets for load bearing or shear transfer. The original wood floor framing cannot be found anywhere in the building.

The exterior walls are made of rubble core unreinforced stone masonry. The wall thickness is largest at the base (1200 mm). From the water table at the second floor level and up, the construction changes to a stone and brick masonry cavity wall. Exterior wall wythes are made of 530 mm thick rubble core stone masonry, and interior wall wythes consist of approximately 230 mm thick clay brick wall. Exterior and interior wall wythes are separated by a 100 mm air cavity, as shown in Figure 2. Different stone types and dressing practices were used for wall construction. The façade was usually built using random coursed Nepean sandstone. The interior wythes of stone masonry walls were built using irregular limestone backing, and the void between the two stone wall wythes was filled with stone rubble and lime mortar. Decorative stone arches were built using reddish Potsdam sandstone, and the quoins and window surrounds were made of cut Berea sandstone, as shown in Figure 3. While the original specifications called for massive “bond stones the whole thickness of the walls”, none of the many exploratory openings, and subsequent two pilot restoration projects revealed a single through-stone (bond stone) connecting the wall wythes.

Figure 2: Typical Cavity Wall **Figure 3: Typical Exterior Walls**

All towers, except for the MacKenzie Tower, were built using the same wall construction practices as the rest of the building. Walls in the MacKenzie Tower were built using large blocks of squared, hammer-dressed, and flat-bedded limestone. The veneer walls at the MacKenzie Tower follow the same principles of other exterior walls, except for a relatively thin rubble core.

The interior loadbearing walls consist of a mixture of rubble core stone masonry and solid clay brick walls. Many alterations in the past resulted in a heterogeneous structure, where brick and stone were often built up together in an irregular fashion (see Figure 4).

The building had an elaborate system of heating and ventilation, built into the clay brick walls. The presence of shafts and chimney flues caused a reduction in wall loadbearing capacity. Ground penetration radar, thermal imaging, and exploratory test openings were used to detect the shafts and other voids in seemingly homogeneous brick masonry walls. It was decided that as a part of the rehabilitation, all shafts, fireplace openings, and voids in walls will be filled using comparable clay brick masonry in lime mortar (see Figure 5).

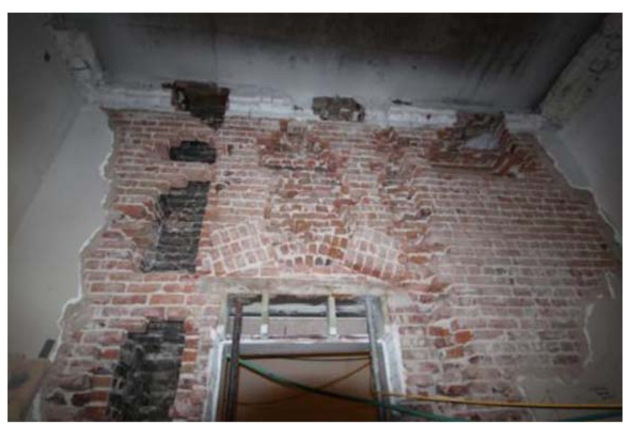

Figure 4: Brick and Stone Masonry Wall **Figure 5: Wall with Chimney Flues**

The original wood roof framing in the south wing was lost in the 1897 fire. Most of the roof framing consists of trusses made of flat and angle rolled iron elements riveted together to form various sections.

The building was heavily polluted with sprayed-on asbestos, which was used as a fire retardant during the 1960s renovations.

CURRENT CONDITION AND IN-SITU INVESTIGATIONS

In general, the West Block Building and its components are characterized by a notable structural integrity. However, the building has deteriorated over the last 150 years, mostly due to environmental effects. Substantial deterioration of exterior masonry walls occurred due to irregular and inconsistent maintenance. The results of the two in-situ surveys performed in 1994 and 2001 confirmed progressive wall deterioration. Bulging of the outer stone wythe of random coursed Nepean stone was observed at a few locations. The bulging was restrained by a steel frame enclosure used for the restoration of South East Tower (Figure 6). The investigative openings and subsequent pilot restoration projects indicated delamination of the exterior wall wythe from the rubble core (see Figure 7).

Figure 6: Emergency Stabilization of Wall Bulging

Figure 7: Evidence of Wall Delamination

Cracks caused by past earthquakes were observed at several locations. The crack width was likely increased due to water infiltration and freeze-thaw cycles which occurred over many years (Figure 8).

Structural modifications performed in the 1960s altered the original load paths. For example, it appears that the original floor structure was supported by the exterior stone wythe of the cavity walls. The original iron beams, bearing either on iron or stone plates, were embedded into the exterior stone wythes (Figure 9). However, the steel beams installed in the 1960s bear on interior brick wythes of cavity walls. As a result, the load on the exterior stone walls has been reduced; but there is an increase in the wall slenderness due to the absence of wall-to-diaphragm connections. Interior brick wythes of cavity walls, which were not originally intended to carry vertical loading, support the floors now.

The attic floors are mostly made of hollow terracotta tiles, which are either individually suspended on steel joists or form a flat arch structure, as shown in Figure 10. This floor system has a limited gravity load capacity and is not able to act as a diaphragm for seismic effects; for those reasons, it will be replaced as a part of the current rehabilitation project. Demolition of finishes has revealed many areas with discontinuities in seismic force-resisting system due to the modifications made in 1960s, such as an absence of connections between the reinforced concrete floors and existing walls (see Figure 11).

Figure 8: A Diagonal Crack in the Tower

Figure 9: An Original Beam Bearing Detail

Figure 10: Terracotta Attic Floor

Figure 11: Floor-to-Wall Connection

PREVIOUS INVESTIGATIONS

The current analysis is based on the findings of previous studies, including in-situ testing and seismic evaluation. Several close-up surveys of the walls and towers of the West Block were performed since 1994. The walls were thoroughly surveyed from a stage suspended from a boom truck. A number of test openings in walls and test pits were made. A limited number of available existing construction drawings and specifications were studied in detail.

In 1997, UMA Engineering Ltd. performed a flat jack testing of the West Block Walls as a part of the larger testing program [2]. The modulus of elasticity (E) of stone and brick walls varied from 1,650 to 3,950 MPa. The E values varied more in rubble core stone walls than in clay brick masonry because of inherent heterogeneous composition matrix of such walls. The testing also covered investigation of shear resistance of brick walls and a determination of in-situ wall stresses. Ground penetrating radar testing was performed initially on MacKenzie Tower walls, and later on a number of interior walls to determine the layout of shafts, flues and voids in walls.

In 2007, Jokinen + Ojdrovic Engineers in Joint Venture conducted a seismic study of the West Block building [3]. Both an equivalent static and a dynamic seismic analyses were performed. The study identified seismic deficiencies in the building, such as shear resistance of existing masonry walls, as illustrated in Figure 12. Other structural deficiencies included shear transfer between walls and floors, diaphragm capacity of existing floors and roof, slenderness of exterior walls and uneven vertical load distribution, etc. Other areas of concern were identified based on visual investigation and in-situ testing, such as cracks in the masonry walls.

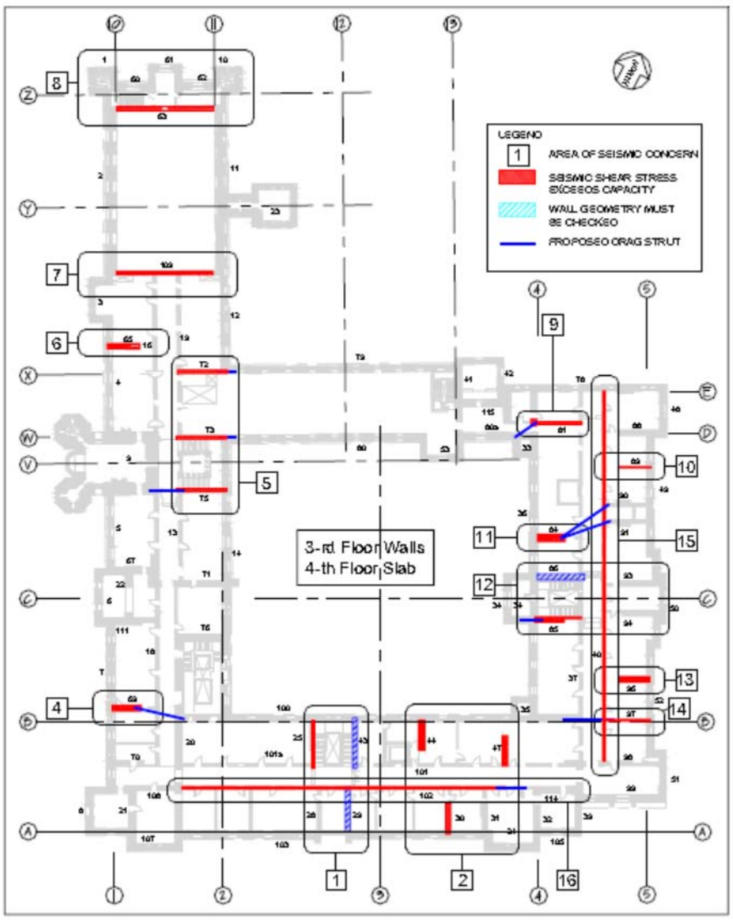

Figure 12: 2007 Seismic study - preliminary concerns - fourth floor level [3]

SEISMIC ANALYSIS: CRITERIA AND METHODS

The seismic analysis of the West Block building was performed in compliance with the Public Works and Government Services Canada (PWGSC) Policy “Seismic Resistance of PWGSC Buildings” and the National Building Code of Canada 2010 (NBCC 2010) [4]. The selection of seismic analysis procedure was made based on the seismic hazard of the building site, building importance, and structural characteristics of the building. The building is located in an area of moderate seismic hazard (Ottawa), and the corresponding seismic hazard index $I_E F_a S_a(0.2)$ is 0.484, based on the spectral response acceleration at 0.2 sec, $S_a(0.2)$, of 0.64 and the acceleration-based seismic coefficient, F_a, of 0.764 (corresponding to Site Class A - rock). The owner decided to use importance factor, I_E, of 1.0. Since this is an unreinforced masonry building, it was deemed appropriate to use force reduction factors $R_d R_o$ of 1.0.

The building is characterized by multiple structural irregularities according to the NBCC 2010 Cl.4.1.8.6.: vertical stiffness irregularity, weight (mass) irregularity, vertical geometry irregularity, re-entrant corners and diaphragm discontinuity. The special feature of this building is MacKenzie Tower. The weight of the tower accounts for approximately 10% of the weight of the whole building. The tower is slender and it was expected that this tower would affect the behaviour of the whole building or the part of it during the seismic event. It was also expected that MacKenzie Tower would increase the fundamental period of the building and reduce the seismic force. The influence of the tower could be investigated only by means of the dynamic analysis.

Based on the above, the two main reasons to perform a dynamic analysis were NBC requirements for irregular buildings and great uncertainties with the fundamental period of the building.

An uncertainty associated with modelling rubble stone masonry wall structures is much higher than for otherwise similar wall structures made of homogeneous materials, such as reinforced concrete. Linear elastic modal dynamic analysis (response spectrum analysis) was selected as the simplest and most practical analysis method for this project.

ANALYSIS MODEL AND CHALLENGES

SAP2000 structural analysis software package [5] was chosen for the detailed dynamic analysis over ETABS because it provided more tools for the modelling and interpretation of the result. It was decided to consider the distribution of heavy masonry walls along the height of the building as opposed to concentrating the weight at floor diaphragm levels.

The modelling was a complex process, and it was challenging to determine an appropriate degree of accuracy for modelling the geometry and other building characteristics. The model consisted mainly of shell elements representing both walls and floors. An attempt was made to capture all openings in walls and floors, and account for variations in wall and floor thicknesses. The size of the model as shown in Figure 13 was significant - it consisted of 52,600 shell elements and 54,000 nodes.

In general, different mechanical properties were assigned to walls and floors. However, it was decided to use the same mechanical properties for stone and clay brick masonry walls. Each cavity wall was modelled as a single wall wythe with the thickness equivalent to the sum of thicknesses of stone and clay brick wythes. In-situ investigations revealed that, after the plaster was stripped from the walls, many walls were built as composite stone and brick masonry walls in an irregular fashion (see Figure 4).

The original flat terracotta arches on iron beams, the Fox and Barret floors, and the contemporary reinforced concrete floors on steel beams, were all modelled as supported directly on walls. Most of these systems act as one-way slabs, and the direction of load transfer was accounted for in the analysis. The contemporary rolled steel beams installed in the 1960s were included in the model, however the original iron beams of flat arch terracotta and Fox and Barret floors were smeared within the floor elements.

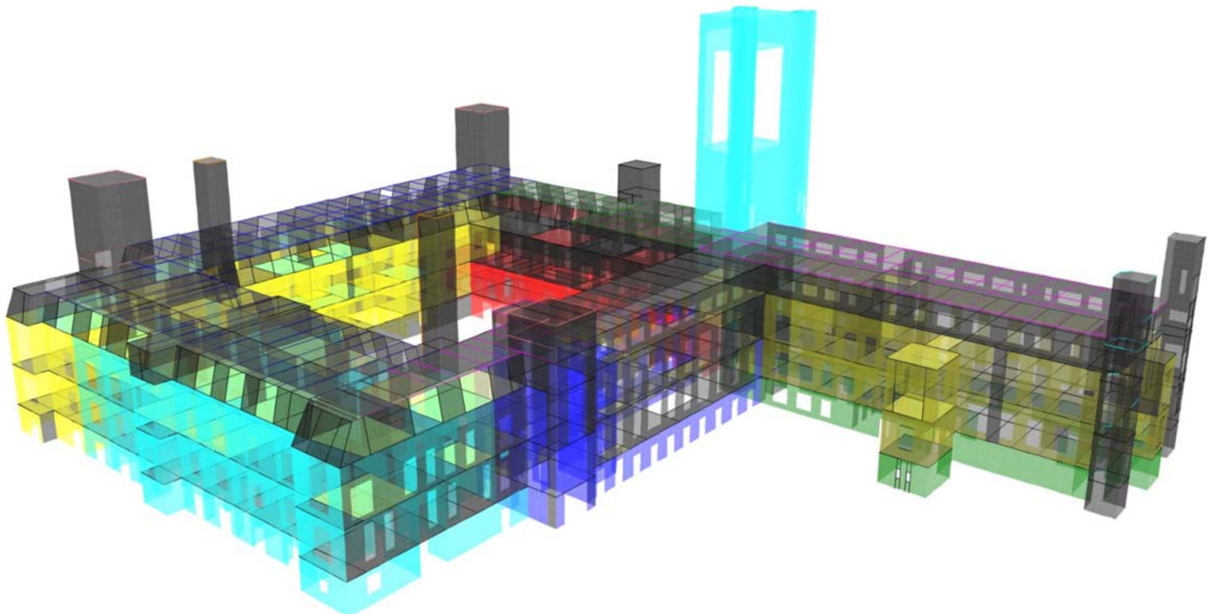

Figure 13: SAP2000 Analysis Model of the West Block Building

An additional challenge was related to modelling the connections between intersecting walls and the wall-floor connections. Several smaller analysis models were created to evaluate the effect of existing connections and boundary conditions upon the dynamic properties of the structure (such as fundamental period). For example, floor slabs in the building are not continuous - they usually span between adjacent parallel walls within a room. These slabs are not monolithically connected to the walls. The effect of discontinuity in the floor slabs was investigated using a smaller model. The slab discontinuity was modelled using LINK elements in SAP2000. The study showed that slab discontinuity does not have a significant influence on the fundamental period of the structure.

A simplified model was also used to simulate and study seismic behaviour of the building with flexible diaphragms. It was not possible to achieve the flexible behaviour of the floor diaphragms in the model of the West Block building because the existing shear walls are closely spaced.

RESULTS

Several different analyses were performed for various combinations of wall and floor rigidities. Results of a linear dynamic analysis depend on the building's rigidity, which is a function of the modulus of the elasticity of masonry walls. Based on the 1997 study [2], review of international professional literature, and consultation with experts in the field of earthquake engineering and stone masonry, it was decided to use the modulus of elasticity of stone and brick masonry, E_{st}, of

1,500 MPa. The modulus of elasticity for the few new reinforced concrete walls, E_c, is 15,750 MPa, according to CSA A23.3-04 (assuming cracked concrete section properties and characteristic compressive strength of 25 MPa).

A sensitivity study was performed to evaluate the effect of variation in modulus of elasticity (E_{st}) on fundamental period of the building. Figure 14 illustrates variation in fundamental period depending on the modulus of elasticity value for dominant vibration modes (and the corresponding mass participation factors). Note that the fundamental period of 0.44 sec was obtained from the NBCC 2010 empirical formula. The authors felt comfortable with the period of around 0.4 s.

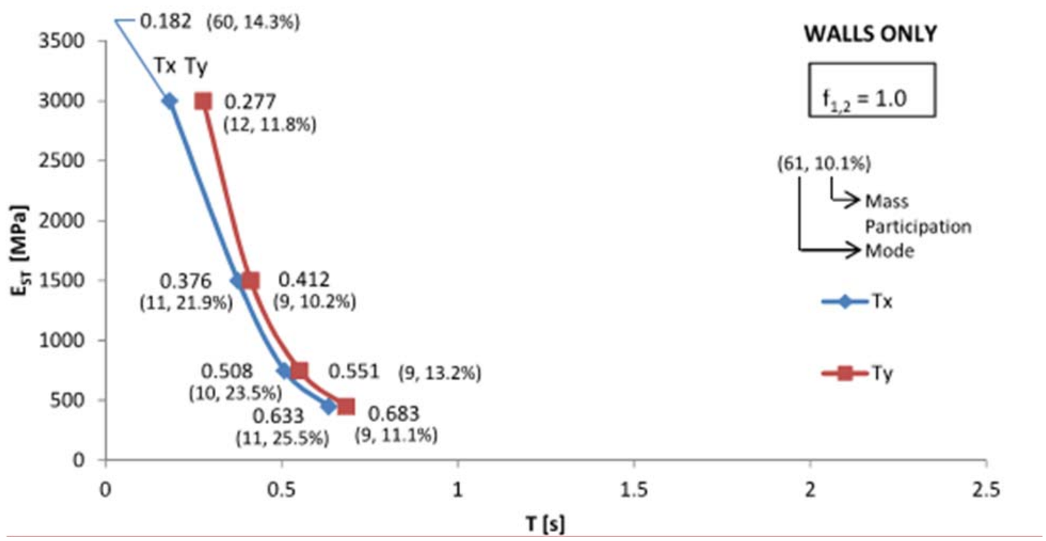

Figure 14: Fundamental Period as a Function of the Modulus of Elasticity

A large number of different analyses were performed for various combinations of wall and floor rigidities. These analyses show that the rigidity of the building is greater than that assumed by the code empirical formula. The analyses have also shown that the presence of MacKenzie Tower increases fundamental period of the building. The impact of MacKenzie Tower on the remainder of the building was assessed by comparing the results of analysis with and without the part of the tower above the uppermost floor level. It was determined that the influence does not propagate far away from the tower, and that the damage to the surrounding walls would isolate the tower and reduce its impact on other parts of the building during an earthquake event.

The base shear of 90,000 kN was determined based on the period calculated by NBC 2010 formulae. The base shear of 120,000 kN was obtained from the static analysis based on the period calculated by SAP 2000. The base shear of approximately 60,000 kN was obtained based on the dynamic analysis. It was decided to scale the seismic force in load combinations to 100,000 kN.

The walls attached to the MacKenzie Tower are subjected to stress levels that exceed the permitted values, as shown in Figure 15. Note that the wall stresses are within 60% of their capacity. The stress capacity was calculated as a sum of 40% of wall weight plus 0.2 MPa.

Note that space between the two walls shown in the figure is occupied by MacKenzie Tower, which is attached to the building. In the contemporary construction, an expansion joint would prevent situation like this, however the creation of an expansion joint in an existing heritage masonry building would not be acceptable. In this case, an independent "catchment" structural system will be designed to provide the support to the floor slabs and catch the debris around the MacKenzie Tower in case of a catastrophic earthquake.

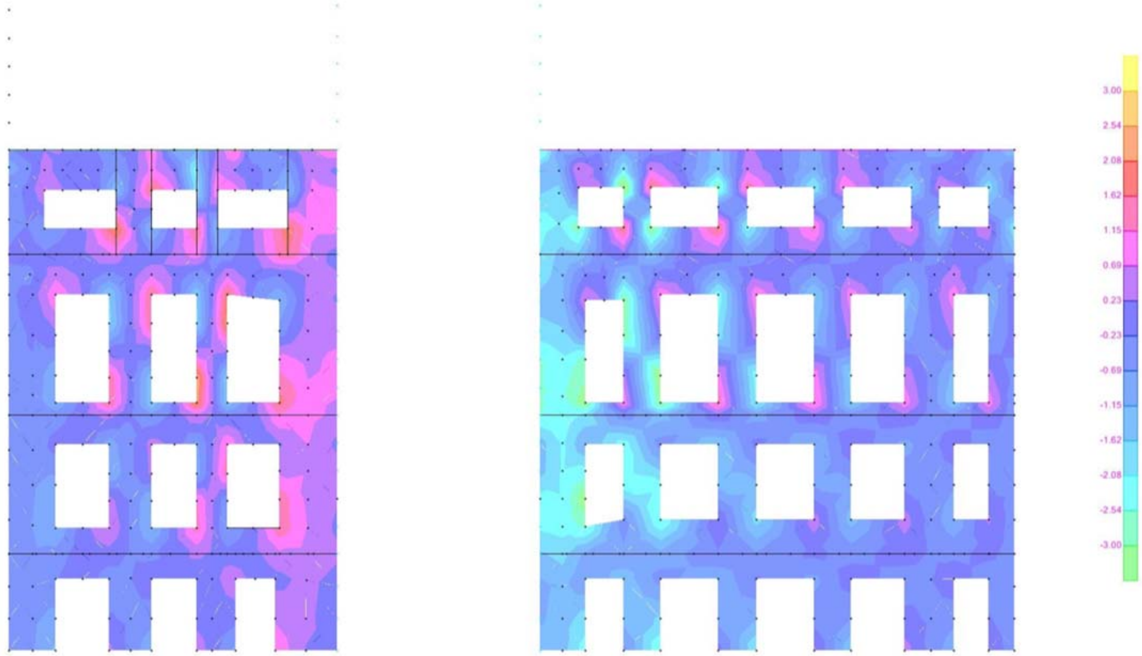

Figure 15: Stress Contours in Walls Attached to the MacKenzie Tower for Earthquake in North-South Direction

CONCLUSIONS

The paper discusses seismic analysis of a complex heritage building made of a combination of unreinforced rubble stone masonry walls, clay brick walls, and a few different types of heritage and contemporary floor structures. A number of design parameters which could influence the fundamental period of the building were varied in the analysis and modelled using both simplified and the detailed building model. The authors have discussed challenges associated with the selection of input parameters for the model, and the interpretation of the modelled seismic response.

The results of the study allowed for better understanding of possible behaviour of the building during the earthquake. The results also indicated a potential effect of the earthquake on the different parts of the building. Some of the results confirmed expectations based on the experience and simplified analysis, while other results brought new challenges.

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