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# Case Study of the Canadian Parliament Building's Unreinforced Masonry Tolerance to Excavation-Induced Vertical Movement

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## Abstract

Centre Block, Canada's iconic Parliament building, is currently undergoing a significant rehabilitation. Part of the planned work includes excavation of three additional partial basement levels. Throughout the excavation, the building will be partially supported on temporary steel shoring. This will be a system of drilled steel piles that will be exhumed and braced as the excavation proceeds. Vertical movements of the temporary steel shoring as well as the adjacent rock mass are expected. These vertical movements have the potential to damage Centre Block's heritage unreinforced masonry walls. A project specific vertical movement limit has been established to define an acceptable level of control. Development of the movement limit has accounted for the specific geometry and materials of Centre Block's masonry walls as well as the building's structural interaction with the shoring system. This paper discusses the non-linear analysis performed to establish the vertical movement limit. It is observed that many of Centre Block's walls have significant capacity to tolerate localized vertical movements. However, the associated load redistribution caused by the vertical movements needs to be considered in the shoring design. A summary of the vertical movement limits and corresponding shoring specification requirements is provided.

## Keywords

Canadian Parliament, Centre Block, unreinforced masonry, excavation induced movements, non-linear analysis

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## INTRODUCTION

The current multi-year rehabilitation of Canada's heritage masonry Parliament building, Centre Block (see Figure 1), includes excavation of three additional basement levels.

**Figure 1: Centre Block, Parliament Hill Ottawa**

The depth of excavation below Centre Block will be approximately 23 m (see Figure 2). The excavation plan extent will cover approximately 60% of the existing building's footprint (see Figure 3). Consequently, throughout the excavation, Centre Block will be partially supported on temporary steel shoring and partially on unexcavated rock. The excavation is expected to trigger vertical movements in the adjacent unexcavated rock mass as its pre-existing field stress is released. In addition, the temporary steel shoring, a system of drilled piles that will be exhumed and braced as the excavation progresses, will also subject Centre Block to vertical movements as it responds to thermal change. These vertical movements have the potential to cause damage to Centre Block's heritage unreinforced masonry walls. To ensure that this damage is kept within acceptable limits, it has been necessary to set an acceptable vertical movement threshold.

In practice, excavation induced structural movement limits are often based on generalized 'best practice' guidelines [1]. The limitation of this approach is that a building's structural interaction with varying stiffness support conditions is not specifically accounted for. It is possible to both underestimate and overestimate a building's tolerance to vertical movements. For Centre Block's rehabilitation, due to the high value of its heritage masonry and its assumed sensitivity to vertical support movements, an analytical approach was adopted to establish a vertical movement limit. This limit was used as the basis for the geotechnical rock support design and to define the temporary shoring system performance objectives. A description of the analytical methods, discussion and a summary of recommendations are provided in the following sections.

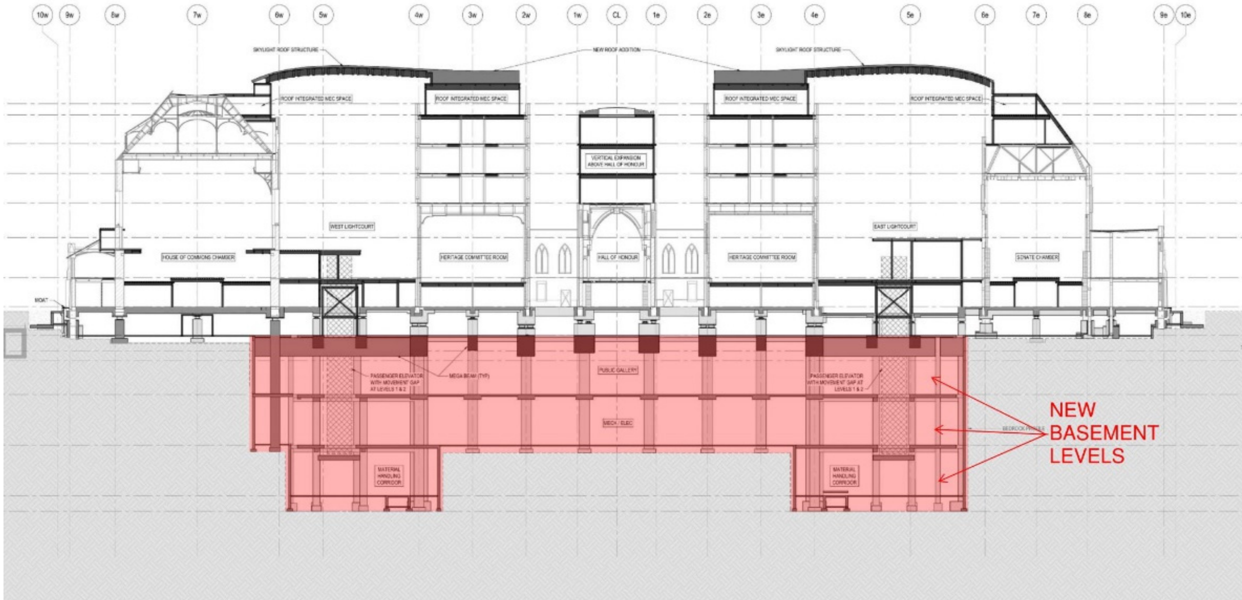


**Figure 2: East-west section through Centre Block showing new basement levels**

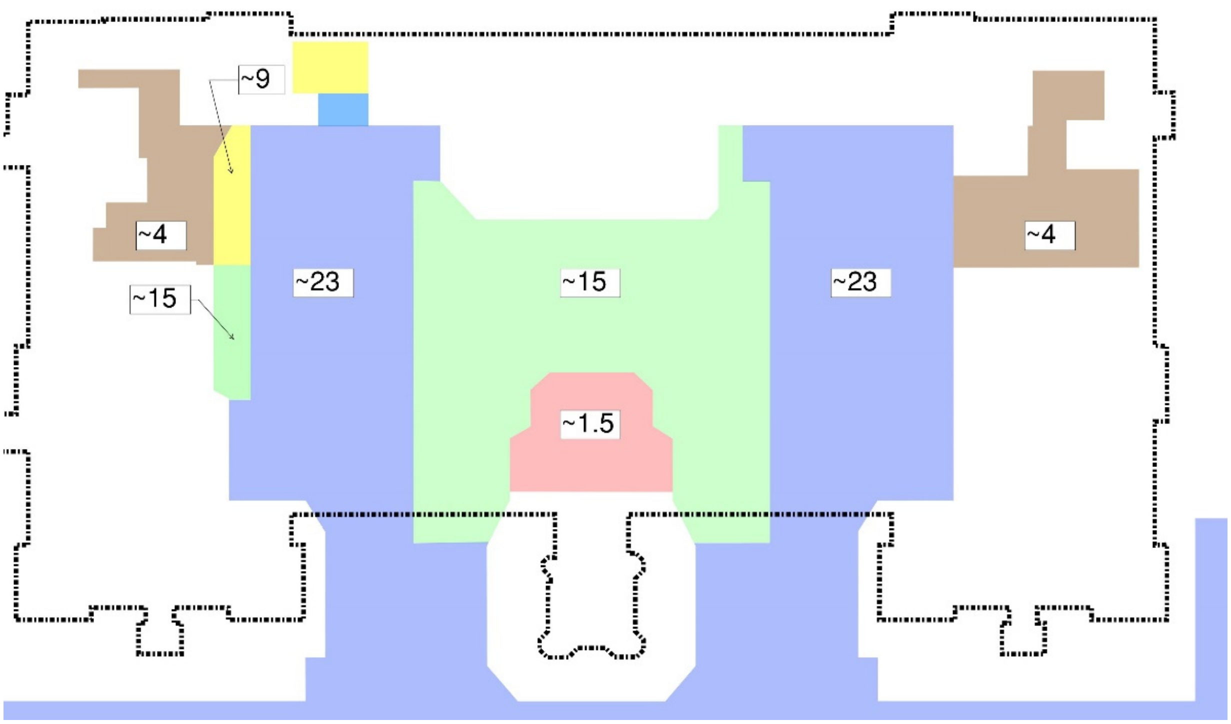


**Figure 3: Excavation extent and depth of new basement levels (m)**

## METHODOLOGY

A representative sample (16) of Centre Block's masonry walls were modelled using the non-linear finite element software VecTor2, that uses continuum material types, and a smeared rotating-crack formulation based on Modified Compression Field Theory and the Disturbed Stress Field Model [2].

Centre Block's walls typically comprise clay brick masonry constructed with a hard cement-based mortar. Its exterior walls have an exterior wythe of sandstone masonry built integrally with the clay brick wythes behind. A typical exterior wall cross-section is shown in Figure 4. The compression strength and compression modulus of Centre Block's masonry was determined through laboratory testing of extracted prisms and in-situ testing [3][4][5]. A summary of the average compressive strength and stiffness is provided in Table 1. A plot of the non-linear compression stress-strain relationships used in the VecTor2 masonry material definitions is shown in Figure 5. A mortar tensile strength, determined from bond-wrench tests on extracted masonry prisms of 0.65 MPa was used (not shown). To assess the sensitivity of Centre Block's walls to localized vertical movements, supports at the base of individual wall piers were progressively 'softened' in the analytical models relative to adjacent supports until cracking in the walls exceeded 2 mm in width. This crack width was selected to correspond to an acceptable level of damage and considered readily repairable. The vertical displacement of the piers at the 'softened' supports corresponding to the crack width limit was recorded for each analysis run.

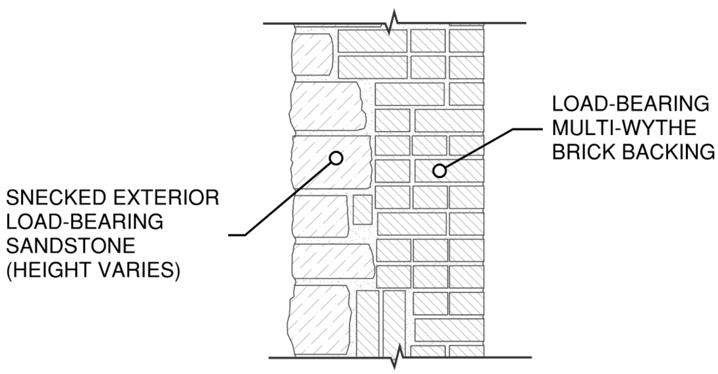


**Figure 4: Typical Centre Block exterior masonry wall section**

**Table 1: Average compression strength and stiffness of Centre Blocks' masonry**

| Material | Compressive strength (MPa) | Compression modulus, E (MPa) |
|---|---|---|
| Brick masonry | 10.3 | 5 800 |
| Stone masonry | 25.7 | 13 300 |
| Brick-stone masonry* | 15.4 | 8 300 |

**Brick-stone properties based on a weighted average of 1/3 stone and 2/3 brick properties*

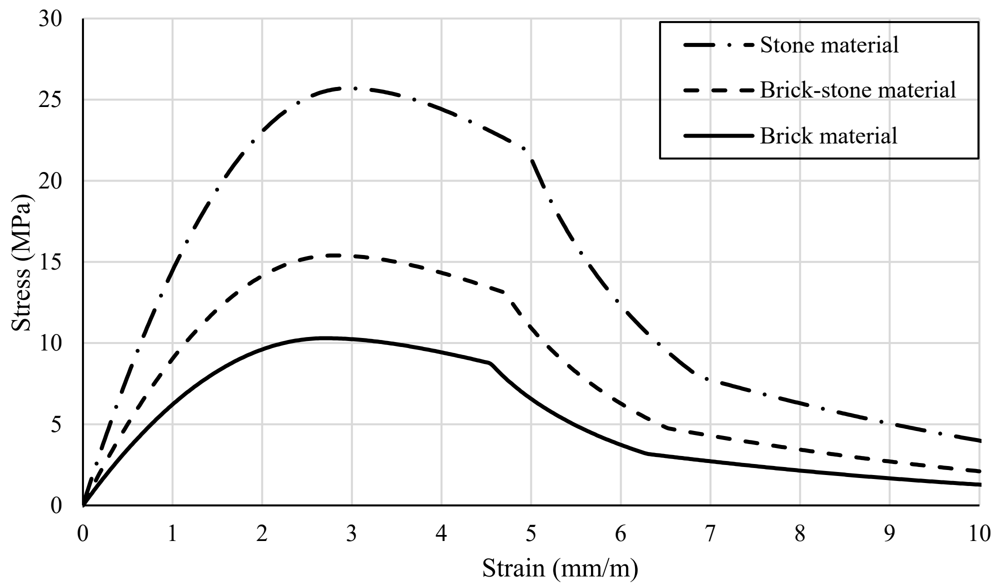


**Figure 5: Masonry compressive stress-strain definition used in VecTor2**

## RESULTS AND DISCUSSION

The analysis results demonstrated that the response of Centre Block's walls to a softened support could be characterized as either a) walls that are 'self-supporting', or b) walls that are 'non-self supporting'. 'Self-supporting' walls typically had the capacity to span over a localized soft support condition whilst 'non-self supporting' walls could not. Examples of Centre Block's 'self-supporting' and 'non-self supporting' walls are shown in Figure 6 and Figure 7 respectively. The 'self-supporting' walls typically have smaller, regularly spaced openings whilst the 'non-self-supporting' wall have larger, more irregular openings.

A summary of the recorded vertical movements at the crack width limits for the 'self-supporting' and 'non-self supporting' walls is provided in Table 2 and Table 3 respectively. Centre Block's 'non-self supporting' walls were found capable of tolerating at least 4-5 mm of vertical movement at a softened support before reaching the wall crack width limit of 2 mm. The 'self-supporting walls' could typically tolerate complete removal of a support whilst still meeting the crack width limit. However, this response also resulted in a significant load redistribution to supports adjacent to the 'softened' support. This had implications for the design of temporary steel shoring to be used within the excavation. A summary of the observed support load increase is provided in Table 4. The average support load increase was approximately 42%, and the maximum was 77%.

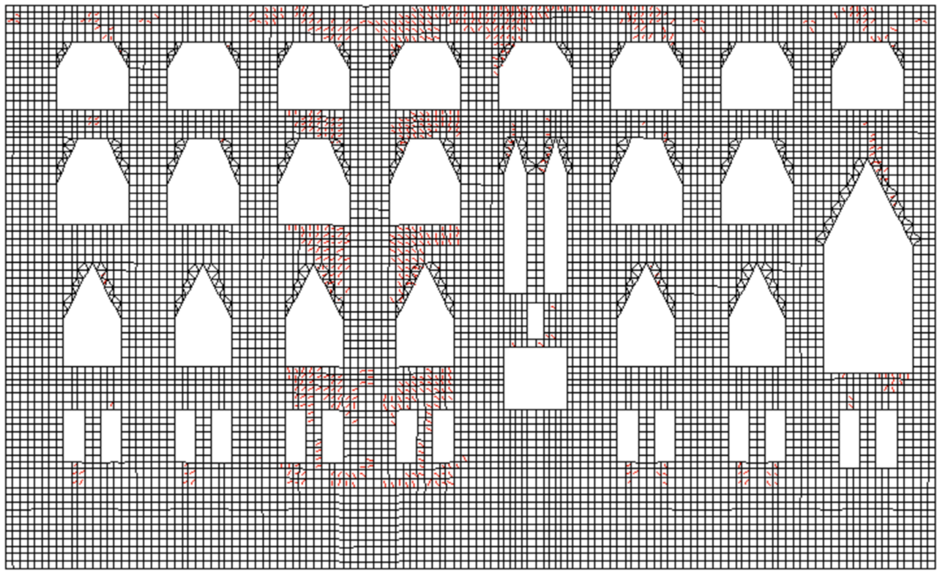

**Figure 6: 'Self-supporting' wall example**

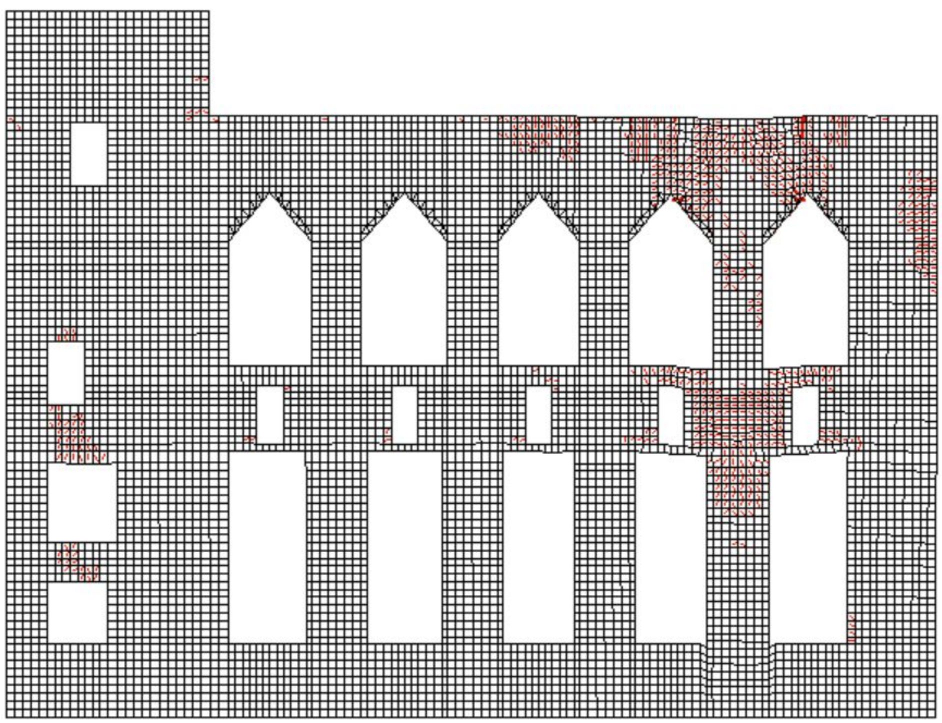

**Figure 7: 'Non-self-supporting' wall example**

**Table 2: Maximum vertical movements of 'self-supporting' walls**

| Wall identifier | Vertical displacement of governing pier* (mm) | Max. observed crack width (mm) |
|---|---|---|
| W8-P5 | 1.8 | 0.1 |
| E8-P4 | 0.6 | 0.1 |
| W20-P5 | 0.5 | 0.1 |
| E18-P5 | 1.0 | 0.1 |
| E4-P4 | 4.2 | 0.1 |
| J-a-P4 | 2.0 | 0.3 |
| W55-b-P3 | 4.2 | 0.9 |
| E34-P5 | 4.2 | 0.6 |
| E56-b-P3 | 5.0 | 0.6 |
| W47-P2 | 5.4 | 0.1 |
| E15-P2 | 0.3 | 0.1 |
| W7-a-P2 | 1.0 | 0.1 |

**Vertical displacement at which the force in the pier above the softened support approached zero load*

**Table 3: Maximum tolerable 'non-self supporting' wall pier movements**

| Wall | Pier | Vertical displacement of governing pier (mm) | Max. wall crack width (mm) |
|---|---|---|---|
| W49 | P1 | 6.0 | 2.0 |
| | P2 | 4.6 | 1.9 |
| | P3 | 13.4 | 1.9 |
| | P4 | 12.9 | 1.9 |
| | P5 | 13.2 | 1.9 |
| | P6 | 8.9 | 2.3 |
| | P7 | 4.8 | 2.0 |
| Y-b | P1 | 7.0 | 2.0 |
| | P2 | 3.6* | 1.5 |
| | P3 | 4.6* | 1.1 |
| | P4 | 2.7* | 0.9 |
| | P5 | 7.1* | 1.5 |
| AX-a | P1 | 12.6* | 1.5 |
| | P2 | 15.4 | 2.2 |
| W33 | P1 | 7.8 | 2.0 |
| | P2 | 6.0 | 1.9 |
| | P3 | 9.5 | 2.0 |
| | P4 | 11.1 | 2.1 |
| | P5 | 11.1 | 2.1 |
| | P6 | 7.9 | 1.8 |
| | P7 | 5.0 | 2.1 |

**Vertical displacement at which the force in the pier above the softened support approached zero load*

**Table 4: Observed load redistribution adjacent to 'softened' support**

| Pier with softened support | Adjacent pier | Initial pier support reaction (KN) | Pier support reaction after load redistribution (KN) | % increase in support reaction |
|---|---|---|---|---|
| W8-P5 | W8-P4 | 694 | 923 | 33 |
| | W8-P6 | 309 | 326 | 6 |
| E8-P4 | E8-P3 | 485 | 649 | 34 |
| | E8-P5 | 256 | 452 | 77 |
| W20-P5 | W20-P4 | 540 | 713 | 32 |
| | W20-P6 | 693 | 899 | 30 |
| E18-P5 | E18-P4 | 411 | 601 | 46 |
| | E18-P6 | 640 | 901 | 41 |
| J-a-P4 | J-a-P3 | 1094 | 1643 | 50 |
| | J-a-P5 | 583 | 829 | 42 |
| W55-b-P3 | W55-b-P2 | 457 | 712 | 56 |
| | W55-b-P4 | 472 | 737 | 56 |
| E34-P5 | E34-P4 | 1549 | 2081 | 34 |
| | E34-P6 | 1037 | 1513 | 46 |
| E56-b-P3 | E56-b-P2 | 928 | 1451 | 56 |
| | E56-b-P4 | 926 | 1552 | 68 |
| E15-P2 | E15-P1 | 1014 | 1223 | 21 |
| | E15-P3 | 941 | 1148 | 22 |
| | | | Average | 42 |

## CONCLUSIONS AND RECOMMENDATIONS

- An analytical method for specific determination of an unreinforced masonry building's tolerance to vertical movements has been presented.
- For the specific case of the Canadian Parliament building, a vertical movement limit between adjacent piers in the range of 4-5 mm was established.
- At this level of movement, cracks widths in its masonry walls are expected to be constrained to 2 mm or less.
- A 3 mm relative vertical movement limit between support points was set as a performance objective for the geotechnical rock support design and the temporary structural steel shoring required to execute the planned basement excavation.
- An absolute vertical movement of 6 mm was also adopted.
- These limits were developed through specific non-linear analysis of Centre Block's masonry walls but compare well to other published guidelines for heritage masonry structures [1].
- It was observed that many of Centre Block's walls can 'self-support' over a localized soft support point, which will provide additional protection against movement induced damage. However, this behaviour also creates the potential for a significant load increase at adjacent supports.
- An additional 1.5 load factor to be used in conjunction with typical ULS load factors for the temporary shoring design was specified to account for this. This is less than the analytically observed maximum, however the jacking points on the steel shoring system will be monitored continuously with load cells through the excavation and adjusted as necessary.

## ACKNOWLEDGEMENTS

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