Leadership in Sustainable Infrastructure
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Vancouver, Canada
May 31 – June 3, 2017/ *Mai 31 – Juin 3, 2017*

SEISMIC ASSESSMENT OF STONE MASONRY BUILDING USING THE EQUIVALENT FRAME MODEL

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1 ABSTRACT

In Montreal there are several low-rise stone masonry buildings that were built in the nineteenth century and many of them are vulnerable to seismic excitations. To preserve their structural integrity and architectural value, these buildings need to be investigated before any retrofit action is proposed. The main objective of this study is to present analysis methods and to assess the seismic lateral load capacities of a low-rise stone masonry building located in Montreal. To conduct this study on heritage stone masonry buildings, relevant parameters contributing to the seismic response need to be discussed. These parameters include the mechanical properties of stone and mortar materials, their compressive and shear strength, the size and location of openings and the site class at the building location. Further, to study the impact of these parameters on the building response, a simplified numerical method that suits the engineering practice is recommended to be applied. With the availability of advanced technology, simulation software using experimental test data plays a major role in understanding the behavior of these structures. Hence, the macro-modeling technique using the equivalent frame model (EFM) provides a suitable tool for analyzing this structural system. In the present work, the EFM model is implemented in SAP2000 by using data from experimental tests results. This method is applied to assess the 2-storey stone masonry school building, known as Villa Maria School, located on site Class C in Montreal. This study aims to present a modeling procedure for engineering practice that endorses sustainability of stone masonry structures built in the 19th century.

2 INTRODUCTION

Stone masonry buildings form the aesthetic of cities and mark the city heritage. Many of these building structures are located in active seismic zones and are prone to damage due to their high vulnerability to lateral loads and the lack of design criteria at the time of construction (Elmenshawi et al. 2015). In order to rise the safety requirements, building codes are updated regularly and the seismic hazard level in many regions of Canada has been increased. For example, only in the last decades, the seismic risk was increased from 10% in 50 years probability of exceedance (NBCC 1995) to 2% in 50 years probability of exceedance (NBCC 2005). Hence, a significant number of existing stone masonry structures are deficient and at risk under potential seismic loading.

Among Canadian cities, Montreal has a great number of stone masonry buildings which are typically low-rise structures built in the 19th and the 20th century. According to a study conducted in the Old Montreal area, it was shown that more than 40% of buildings are masonry structures and represent the dominant category among steel and concrete materials (Nollet 2004). Even though Montreal is located on the stable North American Plate, it has witnessed several earthquakes in the past and will potentially experience them again in the future.

The study of stone masonry structures is a complex procedure especially in the domain of lateral response caused by ground shaking events. Structural modeling using advanced software has been utilized in order to understand the behavior of masonry elements during earthquakes. A wide variety of modeling approaches have been used. However, the diversity of factors that contribute to the response criteria such as the material quality, the construction type, the site location, and the ground motion characteristics put practitioners in difficulty when they need to consider all these aspects to produce a simplified and practical technique for analysis and design (Binda et al. 2005). Observing the behavior of masonry structures during earthquakes and experimental testing it provides information about the seismic response. It is noted that performing experimental studies it allows investigators to identify specific parameters of behaviour that help in the preparation of computational models for masonry structures. In such studies it is important to develop simple models that lead to high accuracy results. In that respect, in addition to simple approaches that schematize the structure as an equivalent frame, the macro-modeling techniques is quite attractive compared to other complex procedures such as the finite element analysis or micro-modeling. It is noted that piers are the principal vertical resistant elements that are linked by spandrels. In general, there are three types of spandrels defined in function of their behaviour: i) weak spandrels with no tension resistant elements, ii) strut spandrels with at least one tensile resistant element and iii) beam spandrel with two tensile resistant elements (Bucchi et al. 2013). The response of adjacent piers is affected by the type of spandrel element. Thus, in the case of weak spandrels, piers behave as cantilever members, in the case of strut spandrels piers are partially coupled and in the case of beam spandrels piers behave as shear type. The first equivalent frame model (EFM) was proposed by Tomazevic in 1978 and the method was labelled POR method. This method is applicable to investigate only low-rise stone masonry buildings because the model considers piers connected by rigid spandrels which leads to shear type behaviour. In the POR method, the rotation at the top of the pier is ignored, while the displacement of the building is overestimated. A more developed method is the Simplified Analysis Method (SAM) proposed by Magenes and Calvi (1997). This method was refined by Magenes and Della Fontana (1998). In the SAM method, the spandrel is considered deformable which means that it can translate and rotate. Meanwhile, in the SAM method, piers are modeled as column elements, spandrels as beam elements and rigid offsets replicate the joint panel. In this 2D equivalent frame model, piers and spandrels behave elastically until the threshold of failure criteria is reached. The failure criteria considered for piers are: rocking, diagonal shear and sliding shear and for spandrels are: rocking and shear. Further, Magenes (1999) proposed a 3D model to analyse the stone masonry structures.

Despite the uncertainties due to the assumptions involved, the macro-modelling approach is able to provide good results if calibrated appropriately (Lourenco 2002). In this domain several macro-models have been suggested such as: non-linear springs or strut-and-tie model that are intended to represent the behaviour of the pier and its interactions with the adjacent structural elements especially the spandrels (Marques and Lourenco 2011).

3 EQUIVALENT FRAME MODEL

The concept of equivalent frame model (EFM) is explained in Fig. 1. Herein, it is illustrated a typical stone masonry wall simulated by piers and spandrel beams. Hence, the vertical elements represent the piers which are the main structural components and are proportioned to carry the vertical loads. The horizontal elements are the spandrel beams which form the coupling effect between two adjacent piers. Finally, the connections between the spandrel and piers are considered as rigid and no failure is expected in this zone

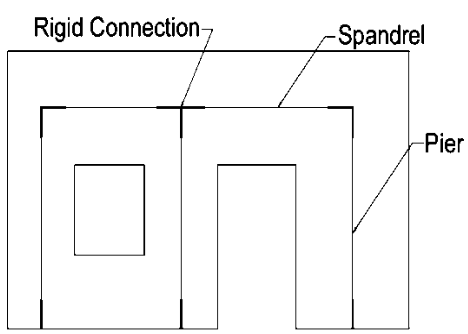

Figure 1: Equivalent frame discretization

(Bucchi et al. 2013). The EFM method is applicable to masonry walls with regular openings which allow extracting distinctive structural elements required to develop the equivalent frame model. In this paper, some modifications are applied to the shape of the facade wall in order to regularize the distribution of openings without disturbing the actual distribution of stresses and further the assessment of structure's capacity. The discretization of masonry wall in effective zones follows the original dimensions of structural elements centered within the masonry panels. The presence of openings in the wall is used to define the geometrical properties of each element considered in the equivalent frame model.

As expected, the plastic zones are allowed to occur in piers and spandrel beams under the gravity load in combination with lateral forces from potential magnitude earthquakes. The three types of failure modes that are generally developed in piers are: diagonal shear failure, sliding shear failure and flexural failure (Pasticier et al. 2008), as are illustrated in Fig. 2. These failure modes are affected by several factors including the aspect ratio, vertical stresses, and the strength of the element in flexure, shear, or tension. Thus, the cracks are usually concentrated in piers which are the stiff part of the masonry wall attracting the most of loads.

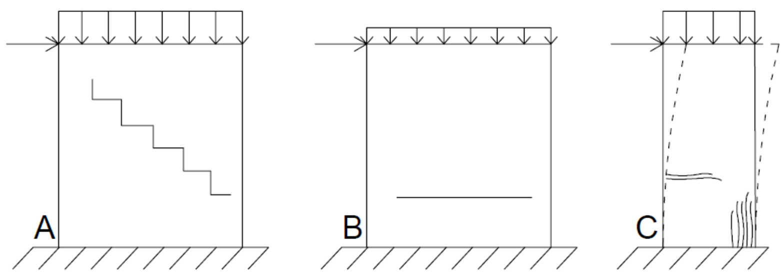

Figure 2: Failure mechanisms that occur in masonry piers due to lateral and vertical loads. Diagonal shear failure (A), Sliding shear failure (B), Flexural failure (C).

The diagonal shear failure mode is the dominant type of failure in masonry elements and especially in piers. Two types of diagonal shear may occur. In the first case, the crack propagates through the bed and head joints at the interface between the masonry units and the mortar because the latter is much weaker than the masonry itself. In the second case which is very rare, the crack propagates through the masonry units. It is noted that shear failure occurs in squat piers with low aspect ratios. The sliding shear failure along the bed joint is due to low vertical stresses and it can be represented by Coulomb's friction law accounting for cohesion of the material and the friction. Finally, the flexural failure occurs in slender piers with high aspect ratios and characterized by vertical cracking at the opposite toe from the load application and flexural cracks due to bending.

Experimental investigations were conducted on full scale piers with different aspect rations at University of Pavia, Italy. These piers were subjected to in-plane cyclic loads and it was concluded that flexural behaviour was experienced by slender piers which exhibited large displacements, low maximum strength, and most importantly less energy dissipation during the cyclic loading (Magenes et al. 2010a).A similar conclusion is formulated by (Yi et al. 2006) who conducted an experimental test at Georgia Tech, US, on a full scale two-story building, whereas the cyclic loads were applied on walls with large and small openings. It is noted that shear mechanism experienced by squat piers is characterized by brittle behavior with high maximum strength, less displacements than the slender piers, and high energy dissipation. In this paper only the strength criteria leading to diagonal and sliding shear failures of piers are considered.

4 IMPLIMENTATION OF EFM IN SOFTEWARE

The equivalent frame method was implemented in the SAM Italian commercial software (Marques and Lourenco 1998) and Tremuri (3Muri) software developed in Italy (Lagomarsino et al. 2014). The Tremuri software launched in 2001 at the University of Genoa, is a specialized program for modeling masonry structures with flexible diaphragms. Using the concept of equivalent frame model implemented in different seismic codes such as Eurocode 8, ASCE/SEI and NTC 2008, a clear definition of spandrels and piers which form the 2D wall is given. When several wall elements are assembled together with special nodal connections it forms the 3D model of the building. A versatile modeling of the floor diaphragm allows the simulation of stress and strain distribution of connected walls. The Tremuri model is based on kinematic implementation where each masonry panel is formed of eight degrees of freedom that simulates the bending-rocking and the shear failures in the masonry walls. The panel is divided into three sub-structures, where the exterior parts are of infinitesimal thicknesses that takes the bending and the axial deformations, while the middle part takes the shear deformations that controls the strength deterioration and stiffness degradation of the wall (Penna et al. 2004).

A study aiming to verify the SAM method for modeling masonry buildings was carried out at the University of Pavia, Italy (Magenes 2010). Piers and spandrels are modeled as beam-column elements connected through rigid offsets with more brittle post-peak behavior of spandrels. Both elements have almost the same formulation for the flexural, sliding shear, and diagonal shear failure mechanisms; however for spandrels the shear is only due to the cohesion excluding the vertical stresses effect. The ultimate strength value is computed according to current proceedings and is assigned to hinges provided in the model. For three dimensional models the elements reproduce independent response to forces applied on orthogonal walls. However, due to proper simulation of connections, in some cases, rigid offsets are assigned between the elements to account for the continuity of displacements at the floor level. A comparison between the SAM and FEM models on two and three story buildings showed good correlation of results even with the consideration of elastic-brittle behavior of spandrels assumed in the SAM model.

To study the response of stone masonry buildings in Lisbon, Portugal a comparison between 3Muri and SAM software was performed (Marques and Lourenço 1998). The macro-element modeling approach was considered to reduce the computational burden defined through proper discretization technique. Further, the selection of pushover analysis allowed to analyze the system in the plastic range while activating the box behavior due to the modeled connections between the horizontal rigid slabs and the walls. Two types of building prototypes were considered. The first prototype is a common two story building with openings in the Y-direction. By analysing the results obtained in 3Muri and SAM it resulted similar base shear force. However, using the 3Muri it resulted higher stiffness due to the flexural failure compared to the brittle shear failure in SAM, as well as larger displacement. The second prototype is two half-way connected two story building used to illustrate the interaction between wall planes. Although an agreement in the deformation modes and the location of the collapse is obtained, there was a difference in the quantitative results in base shear and displacements due to the different collapse mechanisms generated by the software. The results might be different between the two software, this confirms the need for further experimental tests to lessen these variations and reduce the percentage of error present in the assumptions used to define the initial conditions. In this paper, an implementation of SAM method in the user friendly software SAP2000 V-19 is presented in order to study the seismic behavior of an old low-rise stone masonry building located in Montreal.

5 CASE STUDY

The case study is conducted on the central building of Villa Marie high school shown in Fig. 3. This stone masonry building was originally a convent and was built in 1804. Recognized by its Neo-Palladian style, this building was the official residence for the General Governor of Canada in the year 1844. To accommodate the change of building occupancy type, the building was renovated in the same year by the architect George Browne.

Hence, the case study is a two story stone masonry building with basement. The stone masonry is classified as limestone originated from sedimentary rocks. A density test was performed in Concordia lab in order to identify its category. Weight measurements in the field conditions of two stone samples with different sizes were made in water (w_{water}) and in air (w_{air}) and are illustrated in Figs. 5 and 6. The density was calculated based on the following formula:

[1] $\rho_{stone} = w_{air}/(w_{air} - w_{water})$

The measurement yielded a density of 26.9 KN/m^3 which is classified into compact limestone with unit compressive strength ranges between 78 – 186 MPa and unit elastic modulus ranges between 39000 – 68000 MPa (Como 2012). Another study on the physical properties of the Lindsay-Cobourg Limestone conducted at McGill University shows similar values (saturated density of 26.9 KN/m^3 and a dry density of 26.8 KN/m^3) to those obtained in Concordia lab. The stone material is classified as argillaceous limestone with visible heterogeneity ranging between light to dark grey color that are formed as a sub-division of the

Middle Ordovician Limestone in Southern Ontario bedrock. The uniaxial compressive strength of the unit stone ranges between 22 – 140 MPa (Hekimi 2012). Scanning the literature to find experimental tests conducted on stone masonry assemblages in order to extract the initial shear and compressive strength of small wallets, it was found that a diagonal shear test was performed using Credaro sandstone originating from sedimentary rocks with density of 25.79 KN/m^3 and a unit compressive strength ranging between 165 – 172 MPa. Thus, the values extracted from these experimental tests were used to develop the shear strength formulations in this paper. The preliminary mechanical properties considered for the stone masonry assemblage are: f_m= 3.28 MPa compressive strength; E = 2550 MPa; f_v = 0.197 MPa diagonal shear strength; and G = 840 MPa (Magenes et al. 2010b).

Figure 3: The central building of Villa Maria high school (main facade).

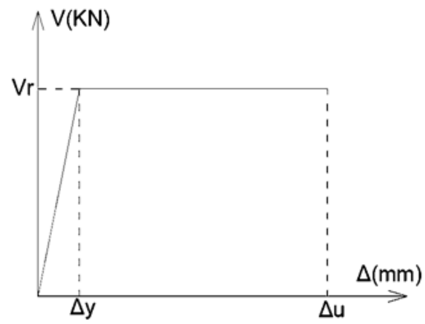

Figure 4: A typical shear-displacement bilinear curve.

Figure 5: Limestone masonry samples collected from the field near the building prepared for density test.

Figure 6: Measurement of the weight in water using a chain hooked to the weight balance.

As mentioned before, the model developed herein simulates only the facade wall of the central building of Villa Maria School. The discretization of the wall is shown in Fig. 7. In this model, there is a slight modification of the facade wall in order to ensure the symmetry feature required in the case of SAM method application. As depicted in Fig. 7, only the part of the basement above the ground is considered. Because the middle two piers at the basement level have smaller height/length ratio than that of adjacent piers, no hinges are assigned in these two elements. In addition, these two piers possess high rigidity implied by their large cross sections; hence no failure is expected to take place in that area. All plans of the building were obtained from the owner and the Congregation de Notre-Dame. From the building plans it results the following wall thicknesses: 1037mm for the basement, 1000mm for the first floor, and 864mm for the second floor. The vertical stresses are due to the own weight of the wall when the tributary mass of the diaphragm was negligible. The absence of internal lateral walls indicates the dominancy of the in-plane resistance. The equivalent frame model is shown in Fig. 8 with the assigned shear hinges in the middle of the piers.

Based on the formulations used in this model, the diagonal shear capacity was more critical than the sliding shear capacity. Thus, all hinges replicate the diagonal shear failure in piers. The shear formulations used are presented according to clauses 7.10.1.1. and 7.10.4.1 of CSA S304.1 standard for diagonal shear and sliding shear, respectively:

In Eq. 2 and Eq. 3 are given the expressions for shear calculation.

[2] $V_r = \emptyset_m (V_m b_w d_v + 0.25 P_d)\gamma_g$

[3] $V_r = 0.16\emptyset_m \sqrt{f'_m} A_{uc} + \emptyset_m \mu P_1$

Herein, $Ø_m$ is the resistance factor for masonry which is 0.6, *bw* is the wall thickness, d_v is the effective depth of the piers but shouldn't be less than $0.8L_w$ where L_w is the length of the wall. In addition, $g_g = A_e/A_g$

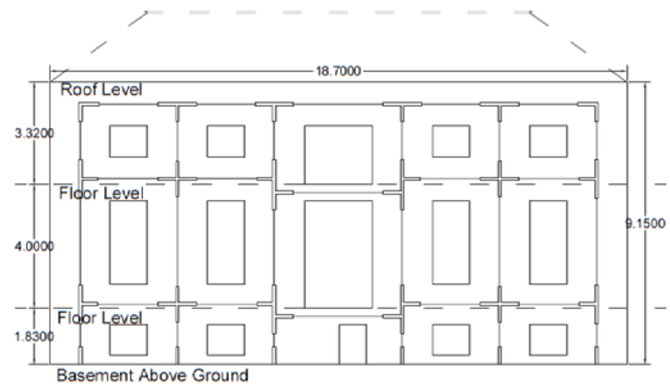

Figure 7: Discretization of walls into equivalent frames representing the piers, spandrels, and the rigid connection. Units are in meter.

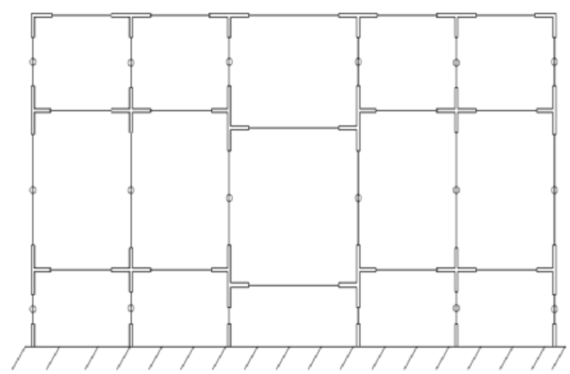

Figure 8: Equivalent frame model with the assigned shear hinges in the middle of piers. (o – shear hinges).

is a factor to account for partially grouted walls or non-solid units which was considered as 1.0 in this paper because we have solid masonry units with fully filled mortar joints where A_e and A_g are the effective and gross cross-sectional area of the pier respectively, P_d is the axial compression load on piers equal to 0.9x(dead load + axial load due to bending in the coupling beams). It is important to note here that the effect of bending in the beams was neglected in this paper as the additional axial load is minor in low-rise structures; however in must be considered in a more detailed analysis. Other parameters are: A_{uc} which is the uncracked effective cross-sectional area of the pier that contributes to shear bond, P_1 is the compressive force equal to P_d plus the compression resulting from the strut action in the infill walls which is not the case here, V_m and f_m' are the shear and compressive strength of wall assemblages, respectively, μ is the coefficient of friction taken as 1.0 for masonry-to-masonry. In this work, the effective cross-sectional areas were considered uncracked as repair work is taking place on the building walls. The minimum V_r value resulting from Eqs. 2 and 3 is the critical value and is chosen to represent the ultimate shear capacity of the pier.

Another requirement is the calculation of displacements at the top of the piers associated with the shear strength values at yielding and fracture of the elements. It is important to mention here that, for simplicity, no hinges were assigned in spandrels. However, the stiffness of the spandrels was reduced by 50% in order to account for the early flexural cracks due to bending which reduces the coupling effect. The formulations for calculating the displacements was obtained from the work of (Lang 2002) and modified to suit this work presented as follows:

[4] $\Delta_y = \delta_y . h_p$

where Δ_y is the yielding displacement at the top of the pier that we are interested in, δ_y is the drift at the top of the pier at yielding calculated with Eq. 5, and h_p is the total height of the pier up to the floor level. In order to account for the coupling effect of the spandrels on the piers the location of zero moment should be estimated. The spandrels in the building have a considerable size which creates a moment resisting fixity at the top of each pier. In light of this, these piers cannot be considered as free end or vertical cantilever. These effects are incorporated in the calculation of the drift.

[5] $\delta_y = V_r \left(\frac{h_p(3h_o - h_p)}{6EI_{eff}} + \frac{k}{GA_{eff}} \right)$

Herein, V_r is the ultimate shear strength resulted from Eqs. 2 or 3, h_o is the height from the bottom of the pier to the location of zero moment calculated with Eq. 9 mentioned later, E and G are the elastic and shear moduli of the pier respectively that shouldn't be confused with the mechanical properties of the units, I_{eff} is the effective second moment of inertia of the pier, A_{eff} is the effective cross-sectional area, and *k* is a factor depending on the shape of the section taken as 6/5 for rectangular sections.

Hence, the ultimate displacement at the top of the pier that we are interested in, Δ_u is given below:

[6] $\Delta_u = \mu_p . \Delta_y$

where μ_p is the ductility ratio given in Eq. 7 and Δ_y results from Eq. 4.

[7] $\mu_p = \delta_u / \delta_y$

Here, δ_u is the drift at the top of the pier at ultimate displacement which is calculated in Eq. 8.

[8] $\delta_u = \begin{cases} 0.8(0.8 - 0.25\sigma_n)\,, & {h_p}/{l_w} < 0.5 \\ 0.8 - 0.25\sigma_n\,, 0.5 < {h_p}/{l_w} < 1.5 \\ 1.2(0.8 - 0.25\sigma_n), & {h_p}/{l_w} > 1.5 \end{cases}$

The calculation of the zero moment location has to be done in two steps. Firstly, it is calculated the ratio of flexural stiffness of the spandrel divided by the flexural stiffness of the pier. Then, it is used the graphs given in Lang (2002) and is taking into consideration the outer and inner piers, the number of aligned walls, and the number of stories. The height from the bottom of the pier to the location of zero moment is:

[9] $h_0 = {(EI_{sp}/l_o)}/{(EI_p/h_p)}$

where l_o is the length of the spandrel measured from the center of the pier which has to be constant on both sides of the pier considered.

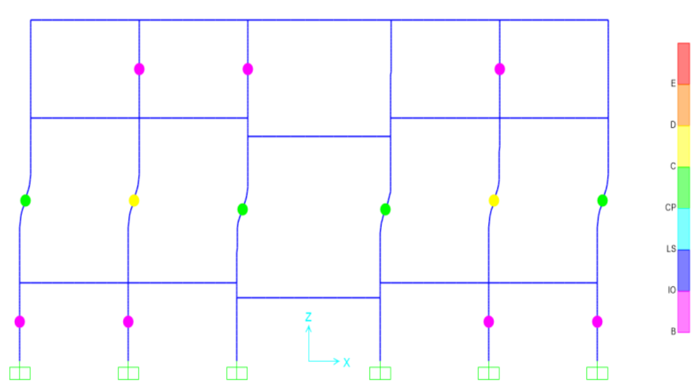

Figure 9: Collapse mechanism in the facade wall showing a diagonal shear failure in the first floor represented by the two yellow dots in the middle piers at the left and portion of the wall.

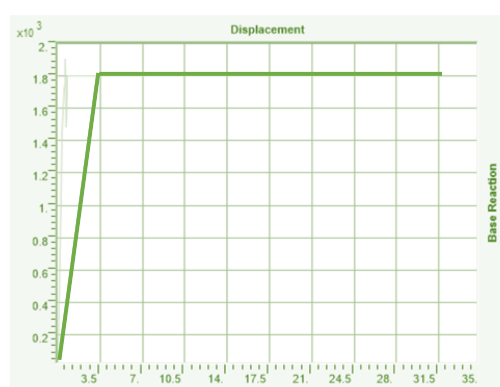

Figure 10: Pushover curve from SAP2000-V19 analysis shows max. base-shear 1780KN and max. displacement 31.67mm

The values extracted from the strength and displacement formulations are used to build a bilinear capacity curve for each pier as is shown in Fig. 4. This curve represents a simplified capacity model (Abo-El-Ezz et al. 2013). A pushover analysis was performed using the NBCC 2010 codes to define the equivalent seismic forces on the building. A default inverse-triangular load distribution is applied which is equal to the total base-shear calculated by the software using the period, mass, and the respective response spectrum of Montreal computed for site Class C. A displacement controlled load case was used until the failure of the first pier is reached. Figure 9 presents the resulting failure mechanism which shows a diagonal shear failure in the first floor in the left and right side of the building. As built, the height of the 1st floor is the highest among the others. The fundamental period of the building is 0.045 sec. From pushover analysis, the wall experiences an ultimate displacement of 31.67 mm and ultimate base-shear of 1780 KN. The pushover curve is showed in Fig. 10. According to NBCC 2010, the interstorey drift limit for schools classified in the High Importance Category buildings is 2%h_s, where h_s is the storey height. From calculations, the resulted maximum interstorey drift is 0.8%h_s and corresponds to the computed ultimate displacement of 31.67 mm and story height of 4.0 m. The resulted interstorey drift is below 1%h_s showing that the failure is happening well below the specified limit. Thus the school building may be vulnerable when exposed to lateral seismic forces. It is noted that the code interstorey drift limit is not adequate for unreinforced stone masonry structures with shear wall resisting system.

6 VALIDATION OF MODELING TECHNIQUE AND RESULTS OF ANALYSIS

Another formulations for element strength criteria recommended in Eurocode 8 are used to validate the equivalent frame modeling technique using the SAM method. Hence, (Pasticier et al. 2008) conducted a study on a typical two story masonry building in Italy, which geometry is showed in Fig. 11.

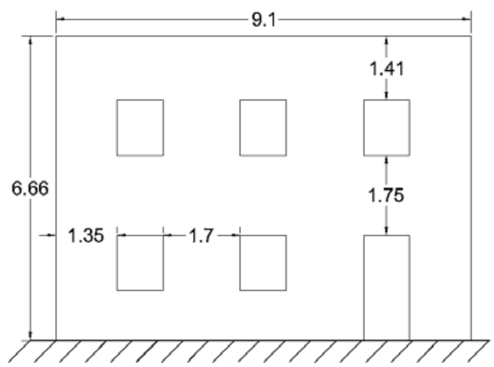

Figure 11: Typical facade of 2-story masonry building with spandrels and piers (measurements are in meter).

In the numerical model developed for SAP2000 V-10 software they included diagonal shear, sliding shear, and flexural hinge properties that were defined for piers and spandrel beams. By replicating their work using SAP2000 V-19, similar results are obtained in terms of collapse mechanism shown in Fig. 12 and the pushover curve plotted in terms of base shear against the interstorey drift which is showed in Fig. 13. The collapse mechanism occurs at the 2nd floor and is due to sliding shear. The results showed in Figs. 12 and 13 illustrate the efficiency of the EFM applied to simulate the response of the facade wall of Villa Maria building. However, different formulations used in this study do not provide a 100% validation of the results of the new model due to the variations in the criteria between the Italian and the Canadian provisions. To overcome this drawback, a building in Catania studied by the Research Group of Genoa University was selected because of its similarity with Villa Maria school building (Liberatore 2000). The Catania building is three floors height with symmetrical three piers about a centered large opening for the main entrance as is illustrated in Fig. 14. The pushover curve resulted for Catania building is plotted in black in Fig. 15 and the pushover curve replicated in this study for Villa Maria building is plotted in green. Both models show similar ultimate strength values referring to close shape of the wall. The large difference witnessed in the elastic behaviour and the initial stiffness of the walls might be due the difference in the walls' thicknesses and material properties used in the construction. However, the results are proven to be reasonable for the behavior of low-rise masonry structures.

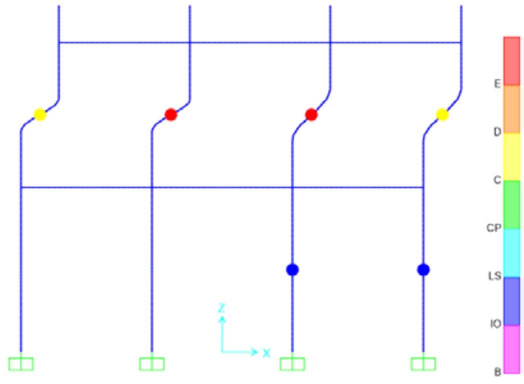

Figure 12: Collapse mechanism in the replicated facade wall using SAP2000 V-19.

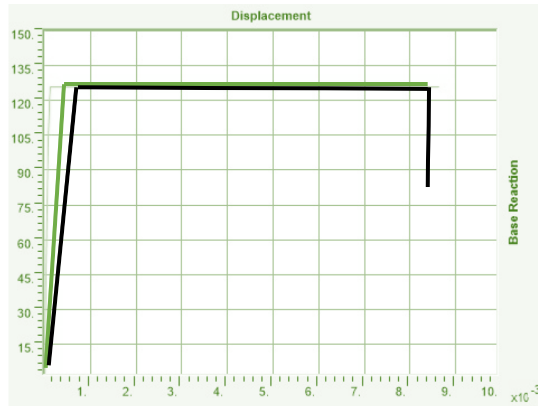

Figure 13: Comparison between Pasticier et al. 2008 (black) and SAP2000 V-19 (green).

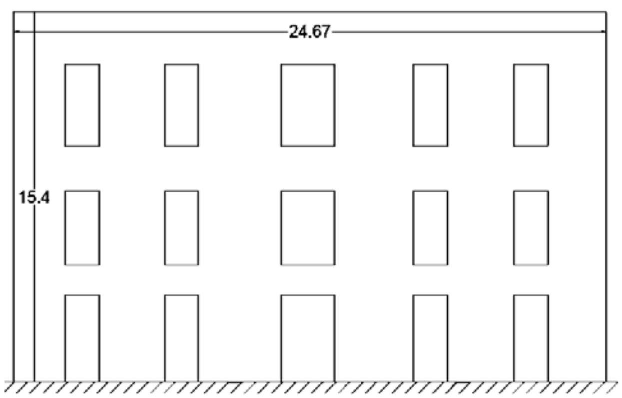

Figure 14: The facade of the wall from Catania Project that is close in shape to the facade of Villa Maria School.

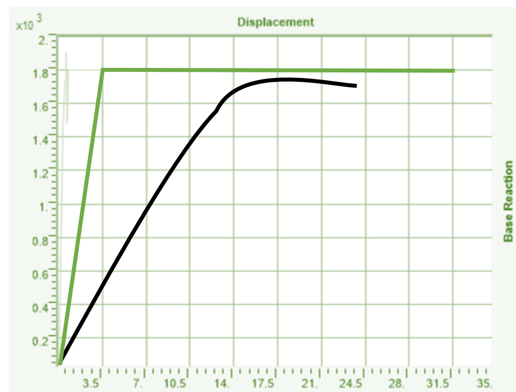

Figure 15: Comparison between pushover curve for Villa Maria facade wall (green) and the Genoa replicated model of Catania Project wall (black).

7 CONCLUSIONS

The equivalent frame model using SAM method is proven to be a simplified technique used to facilitate the modeling of unreinforced stone masonry structures. Implemented in specialized software in Italy, it was possible to apply this procedure using the SAP2000 V-19 software. Adapting shear strength formulations as given in Canadian standard provisions to numerical model was a major step of this work. The facade wall of Villa Maria School in Montreal was subjected to an equivalent seismic load computed according to NBCC 2010 requirements until the failure of the structure occurred. A typical shear story mechanism of the first floor was observed and the pushover curve expressed in terms of base shear versus the interstorey drift is plotted. These results are validated against similar studies selected from the literature. The interstorey drift limit in the NBCC 2010 code is much larger than the interstorey drift at failure resulted in the case study. Thus, the building code neglects the behavior of unreinforced masonry stone buildings and may raise the vulnerability of these buildings to lateral seismic loads. It is to be noted that the results of this paper are still preliminary requiring validation of exact formulations using experimental tests on a stone masonry wall. The stone masonry material in Canada is shown to have their special characteristics and mechanical properties in addition to the unique composition of the wall sections and thicknesses. An important conclusion resulted from this study is the necessity to conduct full scale testing on multi-story unreinforced stone masonry buildings in order to identify their dynamic characteristics, as well as their vulnerability to seismic loads. This is considered as an important step to ensure the sustainability of such buildings that are part of the Canadian patrimony.

8 ACKNOWLEDGMENTS

The authors would like to thank the Congregation of Notre Dame, the owner of Villa Maria Building, and architect Keven Blondin for their generous support in the project. The financial support provided by Concordia University and IC-Impacts Research Centre of Excellence Network Canada is gratefully acknowledged.

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