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SEISMIC EVALUATION AND RETROFIT OF REINFORCED CONCRETE BUILDINGS

Denis MITCHELL
James McGill Professor, McGill University
denis.mitchell@mcgill.ca

Perry ADEBAR
Professor of Structural Engineering, The University of British Columbia
adebar@civil.ubc.ca

Patrick PAULTRE
Professor, Université de Sherbrooke
patrick.paultre@usherbrooke.ca

Murat SAATCIOGLU
Professor, University of Ottawa
murat@eng.uottawa.ca

ABSTRACT:

Some aspects of the ASCE 41-13 evaluation and retrofit guidelines arediscussed. Experimental studies on thin walls containing lap splices at their bases are summarized, emphasizing the poor seismic performance of this type of construction. The effects of wall thickness and reinforcement detailing of deficient reinforced concrete walls are also presented. The improved performance of deficient shear walls due to different retrofit techniques are illustrated. The results of an experimental study indicate that the concrete may crush in thin walls at low values of uniform compression strain, and unlike a tied column, a thin concrete wall may suddenly lose the complete axial load-carrying capacity. Experimental results are presented, demonstrating the effectiveness of transverse prestressing applied to poorly detailed columns. The response of poorly detailed concrete frames, retrofitted by different types of lateral bracing is presented. The results of the seismic evaluation of a 12-storey concrete building that was damaged in the 2010 Haiti earthquake are described.

1. Introduction

This paper summarizes some of the research carried out in the Canadian Seismic Research Network (CSRN) on the evaluation and retrofit of concrete building structures.

2. Evaluation and Retrofit Guidelines

ASCE 41-13 (ASCE, 2013) provides a comprehensive set of provisions for the evaluation and retrofit of existing buildings. This document provides the methodology and clear performance objectives for detailed evaluation and the assessment of retrofit measures. Included in this document are means of evaluating concrete structures with different framing systems including moment-resisting frames and shear walls.

Requirements are given for expected performance levels of these structures, considering a variety of deficiencies in both design and detailing.

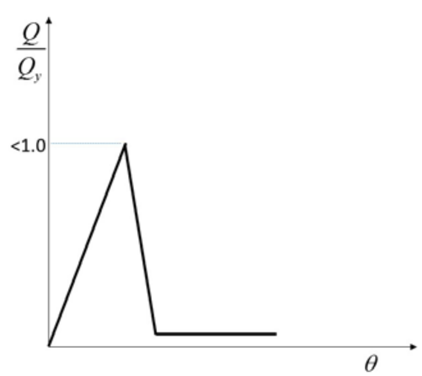
2.1. Determining Member Resistances

In carrying out an evaluation of a structure, ASCE 41-13 uses the nominal resistance of a member determined assuming expected material strengths. In the Canadian context, the nominal resistance of a member is determined assuming that both ϕ_c and ϕ_s are equal to 1.0.

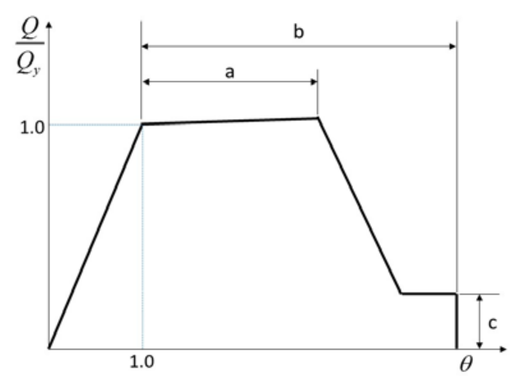
For cases where the specified concrete compressive strength is not known, ASCE 41-13 provides ranges for default lower bound values depending on the year of construction. In the absence of data on the specified concrete strength, the Canadian Highway Bridge Design Code (CHBDC) (CSA 2014) recommends a value of 20 MPa for non-prestressed concrete superstructures. If the reinforcing steel properties are not known then the default values given in ASCE-41 are helpful in determining appropriate yield and ultimate strengths for structural, intermediate and hard grade steel for the construction in the period 1914 to 1978. In addition, values of reinforcing steel yield strengths are given in the Canadian Highway Bridge Design Code (CHBDC) (CSA 2014) for the evaluation of existing bridges for various steel grades and years of construction. The CHBDC also gives default values if the reinforcing steel grade in unknown.

2.2. Load-Deformation Responses

ASCE 41-13 provides generalized force-deformation relationships for component modelling as shown in Fig. 1. Fig. 1(a) shows the response of a brittle element that does not reach flexural yielding due to an undesirable failure mode. Fig. 1(b) shows the response of a more ductile element with the deformation parameters, a b and c. The values of these parameters are given by ASCE 41-13 for different structural components. For a shear wall, the parameters a and b for the plastic hinge rotation as well as the residual strength ratio c are given depending on the reinforcement amounts, axial load level, shear stress level and whether or not the boundaries of the wall are confined.



(a) Brittle response

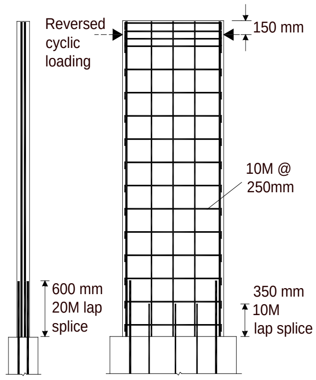


(b) Ductile response

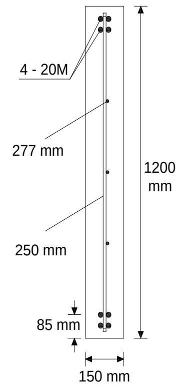
Fig. 1 – Component force-deformation relationships for modelling

3. Splice Failures in Thin Walls

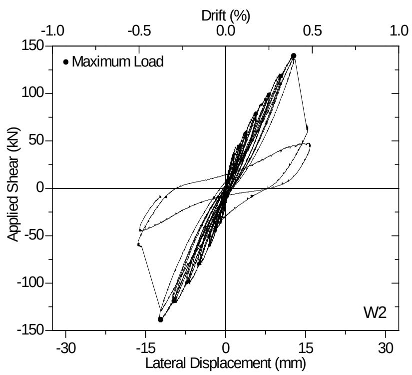
Figure 2 shows a 150 mm thick by 1200 mm long wall test specimen, with details typical of construction for low-rise buildings in Canada in the 1960's. The main flexural reinforcement consists of 4-20M bars at each end of the wall, with a 600 mm tension lap splice ($30d_b$) in the critical potential plastic hinge region at the base of the wall. The transverse reinforcing bars were inadequately anchored with vertical hooks at the wall ends due to the lack of space for proper anchorage in the thin wall. The wall was tested under reversed cyclic loading (Layssi et al. 2012). The concrete strength was 30.4 MPa and the yield strengths of the 20M and 10M bars were 460 and 470 MPa, respectively. Figure 2(c) shows the applied lateral load versus tip deflection response of Wall W2. The brittle failure of this wall was due to longitudinal splitting failure of the lap splices (see Fig. 2(d)) at a load corresponding to 80% of the predicted flexural capacity, assuming yielding of the longitudinal reinforcement. In order to evaluate a structure with thin walls and containing lap splices it is necessary to account for this brittle failure mode.



(a) Splice details



(b) Reinforcement details



(c) Load-deflection response

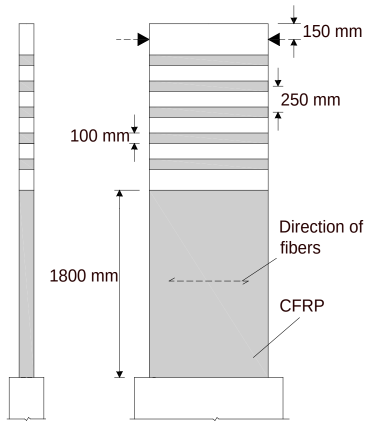
(d) Splice failures

Fig. 2 – Tension splice failures in thin walls (Layssi et al. 2012).

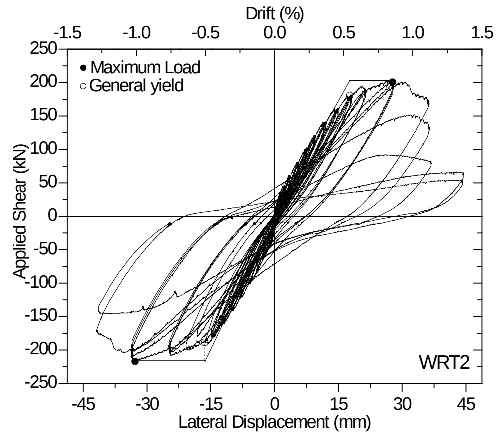
Figure 3 illustrates two different retrofit techniques used to improve the performance of these walls:

<u>Retrofit with carbon-fibre wrap (CFRP)</u> – CFRP wrap was applied with epoxy over a height of 1.5 times the length of the wall region from the base of the wall to provide some confinement to the lap splice region (Fig. 3(a)). Above this region, CFRP strips were epoxied to the wall to provide additional shear reinforcement. The applied lateral load versus tip deflection response of retrofitted Wall WRT2 is shown in Fig. 3(b). The CFRP retrofit improved the splice strength, enabling the development of yielding such that a displacement ductility of 2.0 was achieved. This retrofit technique provides some improvement and minimizes the disturbance to the operations of the structure during retrofit.

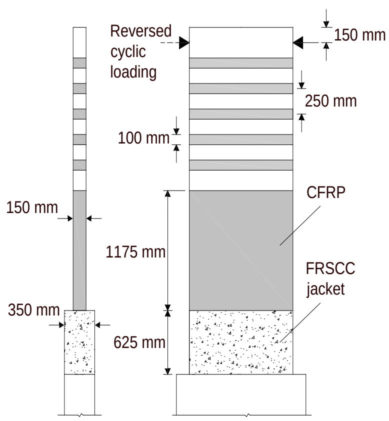
<u>Retrofit with fibre-reinforced self-consolidating concrete (FRSCC) and CFRP</u> – FRSCC sleeving was used over the height of the lap splices as shown in Fig. 3(c). Additional vertical bars were drilled into the foundation block and anchored with epoxy. The other ends of these bars were anchored with headed bars. The FRSCC contained additional confinement reinforcement and was attached to the existing wall by threaded bars passing through the wall. The CFRP strips outside of the lap splice region provided additional shear reinforcement. The flexural reinforcement exhibited yielding and the confined lap splice performed well. A more ductile response was achieved than with the CFRP retrofit. This retrofit resuted in improved response and the reinforced concrete sleeving was achieved by replacing some concrete at the wall ends so that the length of the wall was not increased.



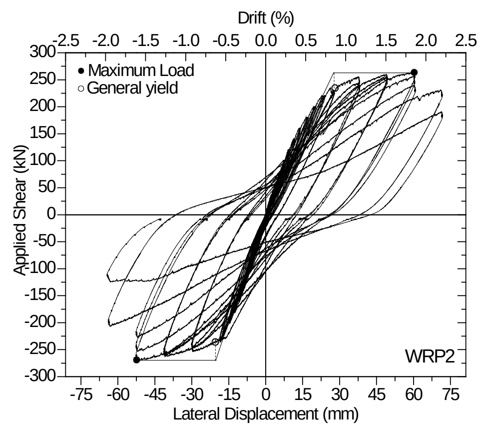
(a) CFRP retrofit



(b) Response with CFRP retrofit



(c) Splice retrofit with fibre-reinforced self-consolidating concrete (FRSCC) plus CFRP

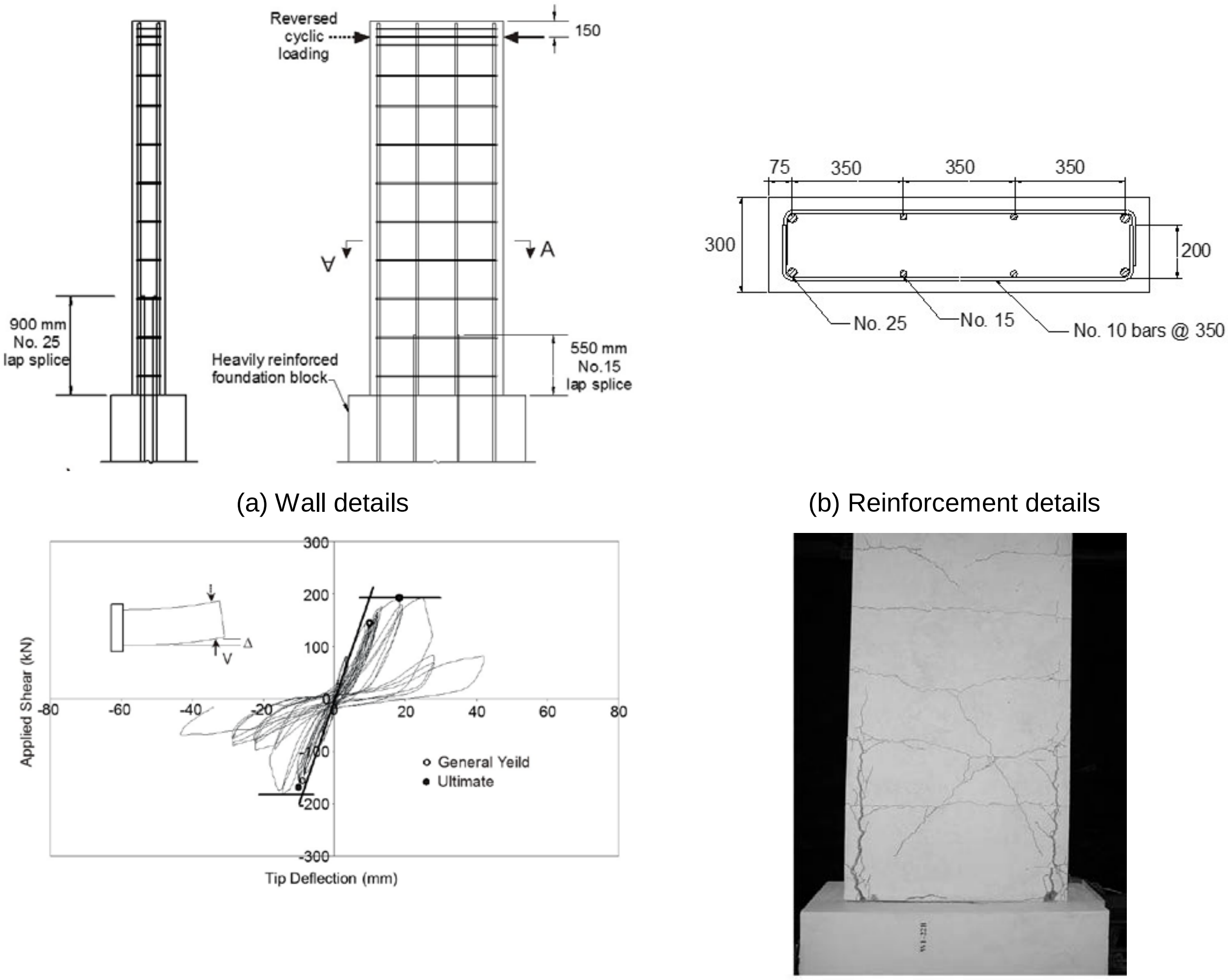


(d) Response with FRSCC splice retrofit plus CFRP wall retrofit

Fig. 3 – Retrofit of thin walls with lap splices at base (Layssi et al. 2012).

4. Effect of Wall Thickness and reinforcement Details

Figure 4 shows the details of a wall that was tested to investigate a proposed retrofit of an existing building, built in the 1960's, in Berkeley, California (Mar et al. 2000). Wall specimen W1 was 300 mm thick and contained lap splices at the base of the wall (see Fig. 4(a)). The concentrated reinforcement at the ends of the wall consisted of 2-25M bars with a 900 mm ($36d_b$) lap splice at the base of the wall. The 10M transverse reinforcing bars had a spacing of 350 mm and were anchored by 90^o hooks (see Fig. 4(b)). The concrete compressive strength was 25.9 MPa and the yield strengths of the 25M and 15M bars were 423 and 453 MPa, respectively. The wall was tested under reversed cyclic loading (Paterson and Mitchell 2003). Wall W1 achieved a displacement ductility of 1.5 and the shear versus displacement response was pinched (see Fig. 4(c)). Although yielding of the reinforcement occurred, the yielding was concentrated in the dowel bars at the base of the wall resulting in a very short plastic hinge length. The failure that limited the ductility was due to bond splitting along the lap splices and some significant shear cracking was also observed. Tests were also carried out with the lap splices located above the base to simulate the situation where a lap splice is present at the second or third floor level of the building. These specimens behaved much better with a displacement ductility of 4.0 and with little signs of pinching in the response.



(a) Wall details

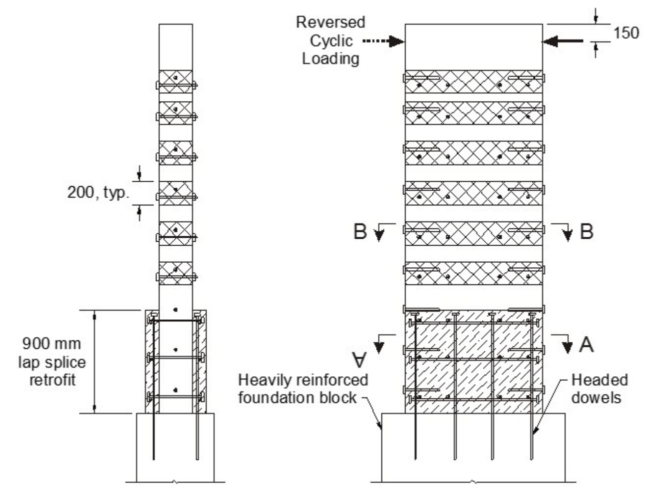
(b) Reinforcement details

(c) Shear-deflection response

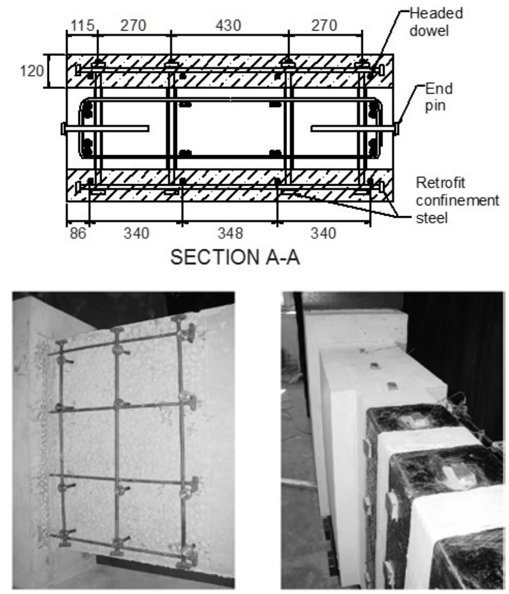
(d) Splice distress and shear cracks

Fig. 4- Performance of 300 mm thick wall with lap splices at base (Paterson and Mitchell 2003).

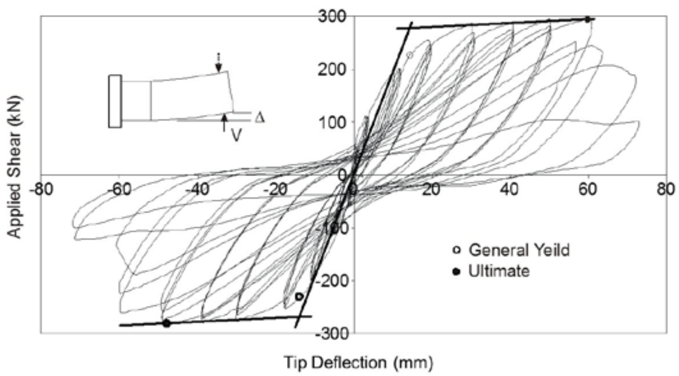
For the retrofit, a reinforced collar was constructed on the sides of the wall over the full length of the lap splice (see Fig. 5(a)). Headed reinforcing bars were used to provide adequate anchorage of the added longitudinal and transverse reinforcement bars in the limited space (see Fig. 5(b)). Headed bars were also installed through the thickness to ensure composite action between the new and existing reinforced concrete. Headed reinforcing pins were grouted into the ends of the wall to improve the confinement of the 90^0 hooks which anchored the transverse reinforcement (see Fig. 5(a) and (b)). The use of the pins at the ends of the wall enabled additional confinement without the need to increase the length of the wall. Strips of carbon fibre wrap were epoxied to the wall and anchored with headed bars drilled through the wall thickness to increase the shear strength.



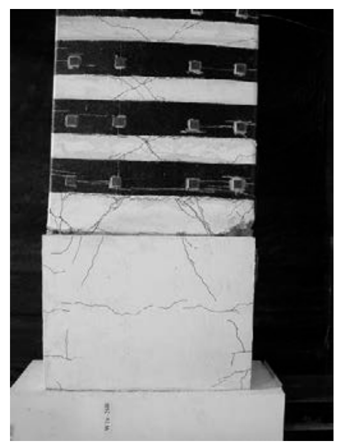
(a) Retrofit details



(b) Retrofit details



(c) Shear-deflection response



(d) Flexural hinging above the lap splices and control of shear cracking

Fig. 5 - Performance of retrofitted 300 mm thick wall with lap splices at base and inadequate transverse reinforcement (Paterson and Mitchell 2003).

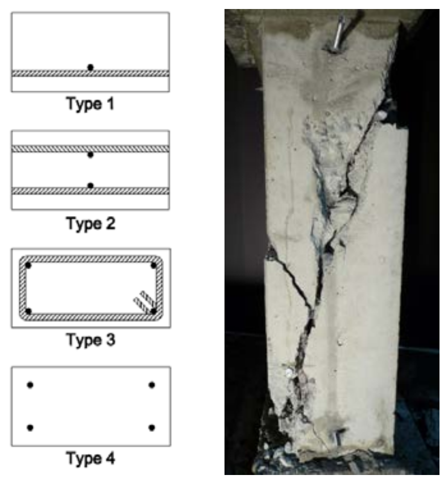
5. Compression Failure of Thin Walls

Many concrete shear wall buildings were badly damaged during the 2010 Maule (Chile) Earthquake and subsequently had to be demolished. A common type of damage was compression failure of thin shear walls (less than or equal to 200 mm thick). Many older shear wall buildings in Canada have thin concrete walls, as do some new buildings in regions of lower seismicity such as Toronto. Most of these thin concrete walls do not have tied vertical reinforcement at the ends of the wall. An experimental study, briefly described here, was undertaken to better understand compression failures of thin concrete walls and based on the results, a number of significant changes were made to the 2014 edition of CSA A23.3

5.1. Phase I: Wall Element Subjected to Uniform Compression

In the first phase (Adebar and Lorzadeh, 2012; Adebar, 2013), many small wall elements were subjected to cyclic axial compression. The main parameters were wall thickness, which varied from 140 to 250 mm, number of layers of horizontal wall reinforcement (no horizontal reinforcement, one layer or two layers), clear cover to horizontal reinforcement, whether the wall had any cross ties (out-of-plane reinforcement), and the height (slenderness) of the wall elements. A schematic diagram of four basic types of specimen cross sections is shown in Fig. 6(a). The standard protocol involved five cycles to each compressive strain level; the first was 0.0005 and the subsequent levels were 0.00025 larger than the previous.

While all specimens reached an average maximum compression stress equal to about the cylinder compression strength of concrete, the maximum compression strain that was achieved depended on whether the element was subject to uniform strain or a significant strain variation. When the elements were subjected to a significant variation of compression strain, the maximum compression strain was much larger than when the compression strain was uniform over the element. As an example, one specimen failed when the compression strain was 0.006 at one end and 0.0015 at the other end (average of 0.0037), while an identical specimen failed at a uniform compression strain of only 0.0023. The end subjected to higher compression strains had visible damage prior to failure, while the end subjected to lower compression strains had no visible damage prior to failure and was able to stabilize the damaged concrete. Uniform strain over a small element best represents the conditions at the end of a long thin wall.



(a) (b)

Fig. 6 – Phase I Wall Element Tests: (a) types of specimens, (b) influence of horizontal reinforcement on observed failure pattern (from Adebar 2013).

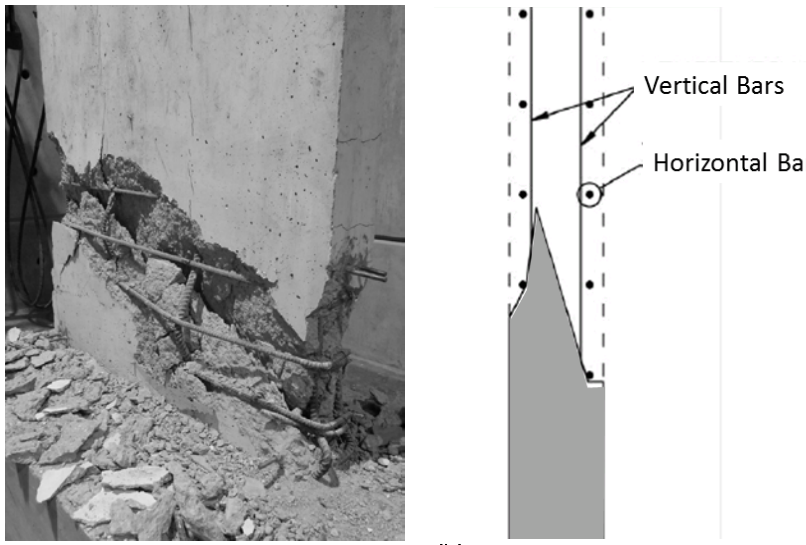
After cycling one element to a compression strain demand of 0.0010 with no visible damage, the thin wall "exploded" when pushed to a compression strain of 0.0013. Figure 6(b) shows how the specimen looked after failure – the damage pattern clearly was influenced by the horizontal reinforcement in the wall. The measured (uniform) compression strain capacities generally increased with wall thickness, because the thicker layers of undamaged concrete were more stable. The 140, 200 and 250 mm thick walls had minimum compression strain capacities of: 0.0010, 0.0015 and 0.0018, respectively.

Thin wall elements with a single layer of reinforcement failed by suddenly splitting into two pieces along the reinforcement layer; but at larger compression strains than the walls with two layers of reinforcement because the total thickness of undamaged concrete was larger. There are many thin walls with a single layer of reinforcement in Canada. The collapse of the Pyne Gould Building in Christchurch, New Zealand was in part due to the compression failure of such a wall (Beca, 2011).

Wall elements with some minimum cross ties similar to what is required in gravity-load columns, e.g., Type 3, were able to tolerate much larger levels of uniform compression strain. Some Type 3 elements were subjected to a large number of cycles to determine if the many cycles of strong motion during the 2010 Maule subduction earthquake was a major contributing factor in the compression failure of thin walls. The tests indicated very little influence of the repeated cycles to strains as large as 0.003.

5.2. Phase II: Thin Wall Subjected to Reverse Cyclic Bending

In the second phase of the study (Chin, 2012; Adebar, 2013), a 140 mm thick shear wall with Type 3 reinforcement was subjected to axial compression and reversed cyclic lateral load. Because the wall was subjected to a very significant strain gradient (due to short wall length) and the horizontal reinforcement acted as cross ties at the end of the wall, concrete crushing occurred only after reaching a large maximum compression strain. However, when the end of the wall began to crush in compression, the wall lost all load capacity and completely collapsed. Figure 7(a) shows how the wall looked after the failure and Fig. 7(b) illustrates how the failure pattern was again influenced by the horizontal reinforcement. Tied columns with square cross sections that were tested as part of the same study maintained a significant core of undamaged concrete within the tied vertical reinforcement and therefore were able to support the applied axial load. The thin concrete wall lost all axial load carrying capacity because the undamaged concrete between the two layers of horizontal reinforcement was very thin.



(a) (b)

Fig. 7 – Phase II Wall Test: (a) complete collapse after concrete crushing at end of wall, (b) influence of horizontal reinforcement on observed failure pattern (from Adebar, 2013).

5.3. Changes to CSA A23.3 – 2014

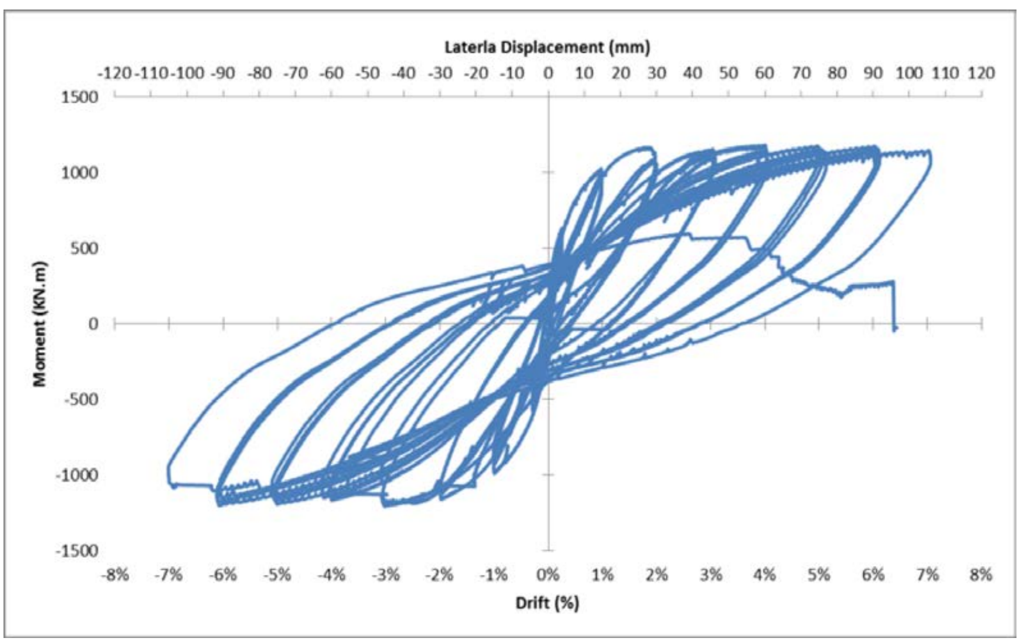
A number of significant changes were made to the 2014 edition of CSA A23.3 in order to avoid compression failures of thin concrete walls. These changes include: (1) Analogous to the way spirally-reinforced columns have a larger $P_{r,max}$ as a portion of P_{ro} than tied columns because they have increased toughness, thin concrete walls now have a significantly lower $P_{r,max}$ than tied columns to account for the reduced toughness. (2) Unless a small zone of tied vertical reinforcement is provided at the end of the wall, a check must be done to ensure the end of all shear walls, including walls designed for wind loading, will not crush prematurely anywhere over the height of the wall. (3) A statement has been added about unintended strong-axis bending of long bearing walls causing compression failure at the end of the wall.

6. Transverse Prestressing of Concrete Columns for Improved Ductility:

A new seismic retrofit methodology was developed to improve the lack of concrete confinement and shear resistance in older building and bridge columns through transverse prestressing and the associated active lateral pressure. Full-size column specimens with circular, square and rectangular cross-sections were tested under simulated seismic loading. The transverse prestressing improved the confinement pressure, which improved column deformability while also improving shear resistance provided by concrete and transverse steel. Active lateral pressure delayed the formation of diagonal tension cracks significantly, potentially eliminating the need for post-earthquake repair work (Saatcioglu and Yalcin 2003). The same technique was extended to include columns reinforced externally by high-strength packaging straps, with some prestressing. This method proved to be less labor intensive especially for smaller size columns, where the straps can be placed with smaller bend diameters. Fig. 8 illustrates the performance of a test column (Sabri 2013).

(a) Typical retrofitted column

(b) Critical region at 6% lateral drift



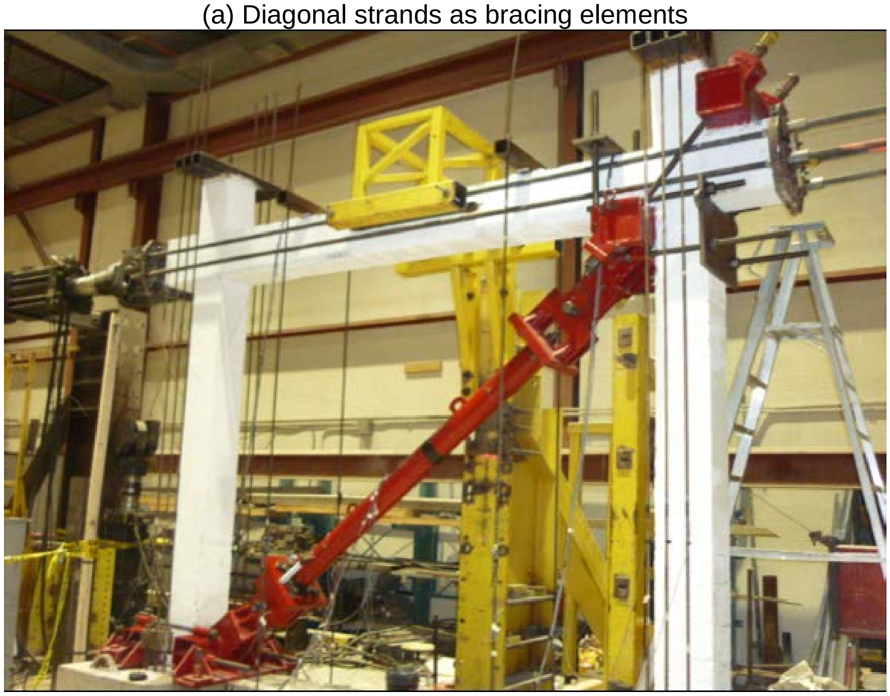
(c) Hysteretic response

Fig. 8 - Behaviour of a circular building column retrofitted with high-strength steel straps.

7. Lateral bracing of non-ductile RC Frames:

Experimental investigations of large-scale reinforced concrete frames were conducted at the University of Ottawa to develop economically viable, structurally sound new retrofit techniques that consist of lateral braces. Two systems have been developed, i) lateral bracing with high-strength prestressing strands, and ii) buckling restrained braces. Frames with a storey height of 3.5 m and a beam span of 3.8 m were tested under slowly applied lateral deformation reversals, with and without the implementation of the retrofit strategies. The frames were designed based on the 1965 NBCC without any seismic detailing, and with significantly lower seismic design forces than that required by the current code (NBCC 2010). Figure 9 shows typical test frames.

(a) Diagonal strands as bracing elements



(b) Buckling restrained brace

Fig. 9 - Retrofit techniques for non-ductile reinforced concrete frames

Both systems provided significant strength enhancement and drift control. In the case of bracing with strands, the required amount of steel strands depended on the seismicity of the region and the level of drift control required. Yielding of the strands resulted in energy dissipation, but increased pinching of the hysteresis loops upon load reversals, until the deformation in the opposite direction increased to offset the plastic deformation experienced in the strands. Increased area of steel resulted in increased lateral strength, and prestressing the strands generated increased rigidity and more effective drift control. Fig 10 shows the comparison of hysteretic relationships of three frames with different number of strands and level of prestressing (Molaei, A. 2013).

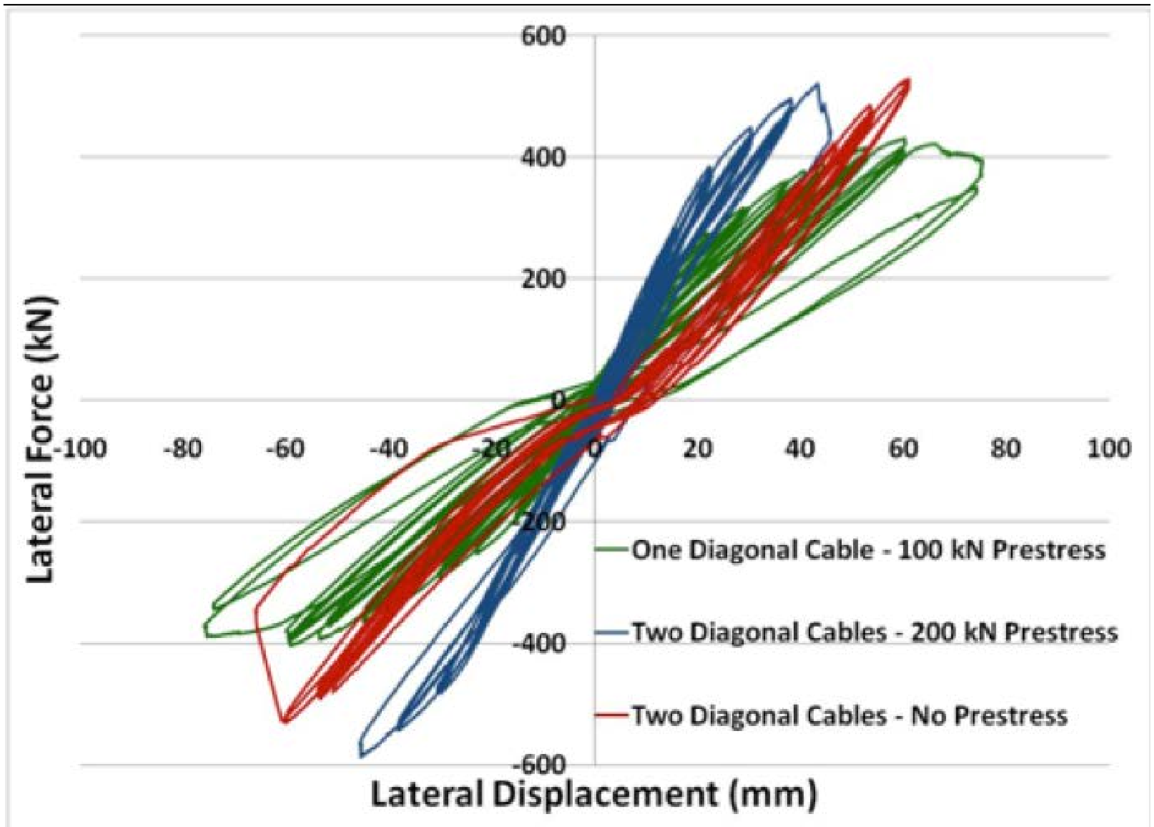


Fig. 10 - Hysteretic response of reinforced concrete frames with steel strands as tension braces.

The second retrofit system is a new buckling restrained brace (BRB) with a circular core and specially designed end attachments that allow inelastic deformations while continuously supporting the bar against buckling (see Fig. 9(b)). The new BRB was found to offset lack of sufficient seismic force resistance in older concrete frame buildings, with significant drift control to protect non-ductile frames. Different types of steel cores have been tested with different steel grades and elongation capacities. It was found that stainless steel core with higher strength and 40% elongation capacity provided the best alternative for both strength enhancement and lateral drift control while dissipating seismic induced energy (Al-Sadoon, 2015).

8. Evaluation of a 12-Storey Damaged in the 2010 Haiti Earthquake

The 2010 Haiti earthquake caused massive damage to buildings that left the capital city Port-au-Prince in a state of emergency resulting in an estimated death toll of more than 300 000. The 12-storey reinforced-concrete Digicel building, located in a neighbourhood that was almost completely destroyed, suffered repairable damage in the main building (see Fig. 11). This good behavior could be explained by the use of numerous shear walls as seismic force resisting system. However, on close inspection, it was found that beams and columns suffered significant damage only in the upper 6 storeys.

Fig. 11 - The Digicel Building in the background and the damaged "Hopital de Turgeau" in the foreground (both buildings completed just prior to the earthquake).

To understand the behaviour of shear wall buildings under significant seismic excitation and to explain the identified damage, a research program under the hospice of CSRN was undertaken that comprises (i) visual assessment to characterize the damage, (ii) ambient vibration tests (AVT) to identify the building's key dynamics properties (natural vibration frequencies, mode shapes, and damping ratios), (iii) updating of ETABS finite element model after the AVT to generate a model representing the building's actual dynamic behaviour in its damaged state and (iv) nonlinear analyses using the identified properties to reproduce the observed damage and estimate the likely seismic excitation at the site. The study showed that the FE method is reliable for predicting the dynamic behaviour of structures, but is very sensitive to the modelling assumptions. The models could predict the vibration frequencies precisely, but an accurate representation of the mode shapes required careful model updating. In particular, it was necessary to account for all the unreinforced masonry walls.

The building was constructed in four consecutive phases, identified as 1 to 4 in the plan view of the ground level shown in Fig. 12. Phase 3 represents the high-rise building that consists of one basement, a ground level, eleven floors, and the roof, with a total height of 49 m from the ground level to the roof. A square helicopter landing pad centered at axis G-4 spans two bays on the roof. The main floors are 24 m by 42 m, with a total area of 1 008 m², and are mostly open office space. The tower's structure (phase 3) is a dual system of moment frames and shear walls acting together to resist lateral loads (Fig. 13). L-shaped walls are present at the north and south corners. U-shaped walls enclosing a staircase and an elevator are found at the east corner, with a door opening on the ground level along axis L. More information can be found in Boulanger et al. (2013a).

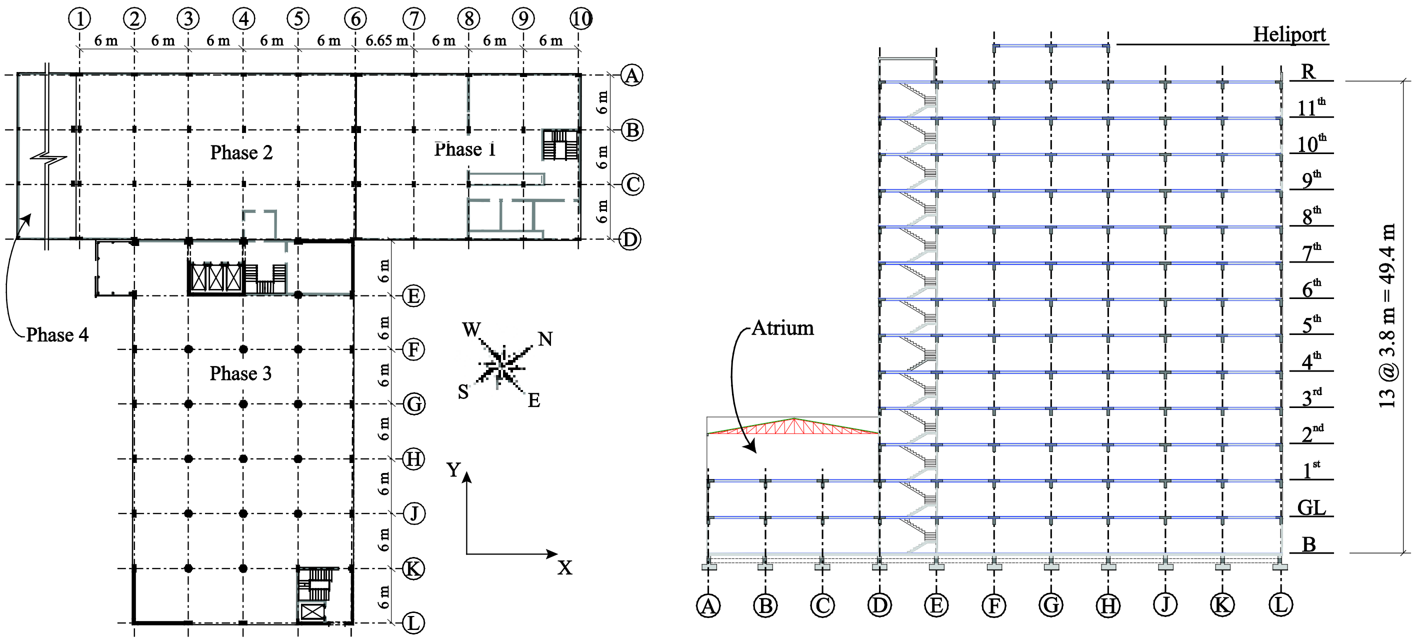


Fig. 12 - Plan view of the ground level (secondary beams are not shown for clarity) (top) and elevation view of the high-rise building (bottom).

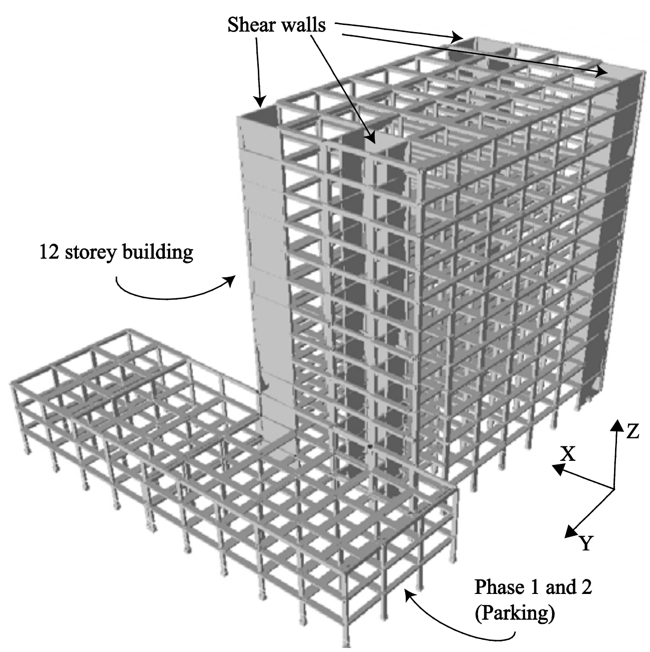


Fig. 13 – 3-D view of finite element model.

Flexural damage to beams and columns was observed in the top storeys in the longitudinal direction. From the 7th storey up, most columns exhibited apparent concrete crushing (Fig. 14) due to large member rotation demands. Light concrete crushing, bond splitting, and even rebar buckling were observed in many beams directly connected to shear walls. The most damaged beams were those connecting to the east U-shaped wall on axes 5 and 6 in the top four storeys.

Fig. 14 - a) Evidence of pounding between the high-rise and low-rise parts; b) Damaged parking-structure column; c) 10th-storey columns on axis 5; d) Damaged beam attached to a wall.

Several nonlinear analyses involving various ground motion intensities were conducted and the results were compared with the damage reported during the on-site survey. The numerical models reproduced well the observed damages and helped to explain them. The overall response of the mixed frame-wall structure was clearly dominated by the high stiffness of the shear walls that kept the inter-storey drifts to reasonable levels. Observed damage to the beams and columns can be explained by large rotational demands imposed on them by the shear walls displacements and rotations during the earthquake (see Fig. 15). More information can be found in Boulanger et al. (2013b). New requirements have been introduced in the Design of Concrete Structures Standard CSA A23.3 (CSA 2014) to ensure that members not considered part of the seismic-force-resisting system have sufficient ductility with a new design chart for the simplified analysis those members in shear wall buildings.

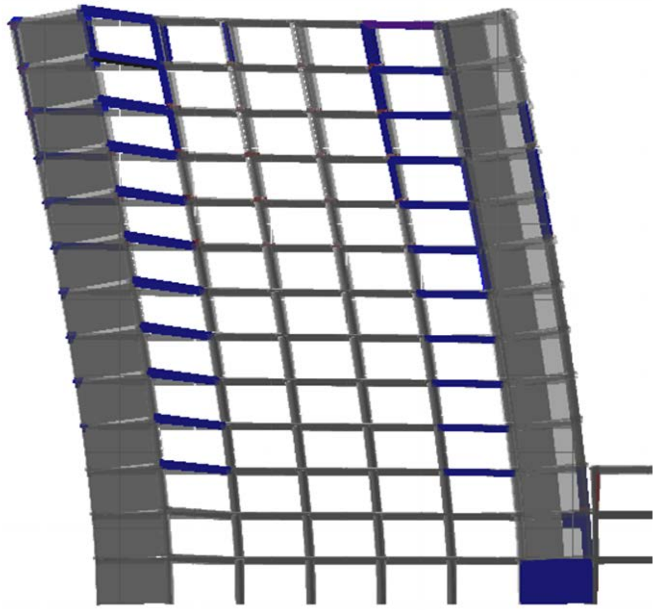


Fig. 15 - Deformed shape of the building, in the Y-direction, under a seismic input.

9. Acknowledgements

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10. References

ADEBAR, Perry, “Compression Failure of Thin Concrete Walls during 2010 Chile Earthquake: Lessons for Canadian Design Practice,” *Canadian Journal of Civil Engineering*, V. 40, No. 8, Aug. 2013, pp. 711-721.

ADEBAR, Perry, LORZADEH, Amir. “Compression Failure of Concrete Walls,” *15th World Conf. on Earthquake Eng.*, Lisbon, Portugal, 2012, 10 pp.

AL-SADOON, Zaid “Seismic Retrofitting of Conventional Reinforced Concrete Moment Resisting Frames Using Buckling Restrained Braces.” PhD Dissertation, Department of Civil Engineering, University of Ottawa, 2015.

ASCE, ASCE/SEI 41-13, “Seismic Evaluation and Retrofit of Existing Buildings”, American Society of Civil Engineers, 2013, 518 p.

BECA CARTER HOLLINGS & FERNER LTD., “Investigation into the Collapse of the Pyne Gould Corporation Building on 22nd February 2011,” *Report*, 2011, 46 pp.

BOULANGER, B., LAMARCHE, Charles-Philippe, PROULX, Jean and PAULTRE, Patrick, “Analysis of a damaged 12-storey frame-wall concrete building during the 2010 Haiti earthquake – Part I: Dynamic behaviour assessment”, *Canadian Journal of Civil Engineering*, 40(8), 791-802, 2013, DOI: 10.1139/cjce-2012-0098.

BOULANGER, B., PAULTRE, Patrick and LAMARCHE, Charles-Philippe, “Analysis of a damaged 12-storey frame-wall concrete building during the 2010 Haiti earthquake – Part II: Nonlinear numerical simulations”, *Canadian Journal of Civil Engineering*, 40(8), 803-814,2013 DOI: 10.1139/cjce-2012-0099.

CHIN, Helen, “Bending displacement capacity of elongated reinforced concrete columns, *M.A.Sc. thesis*, Dept. of Civil Eng., Univ. of British Columbia, Vancouver, BC, 2012, 441 pp.

CSA. 2014. “CSA Standard A23.3, Design of Concrete Structures”, Canadian Standards Association, Mississauga, ON.

CSA. 2014. “CSA Standard S6, Canadian Highway Bridge Design Code”, Canadian Standards Association, Mississauga, ON.

LAYSSI, Hamed, COOK, William and MITCHELL, Denis, "Seismic Response and CFRP Retrofit of Poorly Detailed Shear Walls", *ASCE J. of Composites for Construction*, V. 16, N. 3, 2012.

LAYSSI, Hamed, Seismic Retrofit of Deficient Reinforced Concrete Shear Walls, PHD Thesis, McGill University, 2013

MAR, David, PANIAN, Leo, DAMERON, Robert A., HANSEN, Brian E., VAHDANI, Shahriar, MITCHELL, Denis and PATERSON, James, “Performance-Based Seismic Upgrade of a 14-Story Suspended Slab Building Using State-of-the-Art Analysis and Construction Techniques*”, 69th Annual Structural Eng. Assoc. of California Convention*, Vancouver, BC, August 2000,16 p.

MOLAEI, Ali, “Seismic Retrofit of Reinforced Concrete Frames with Diagonal Prestressing Cables.” MASc Thesis, Department of Civil Engineering, University of Ottawa, 2013.

PATERSON, James and MITCHELL, Denis, Seismic Retrofit of Shear Walls with Headed Bars and Carbon Fiber Wrap”, *J. Structural Eng. ASCE*, Vol. 129, No. 5, 2003.

SAATCIOGLU, Murat and Yalcin, Cem “External Prestressing Concrete Columns for Improved Seismic Shear Resistance.” *J. Structural Eng. ASCE*, 129(8), 2003.

SABRI, Amirreza “Seismic Retrofit of Concrete Columns by Transverse Prestressing.” MASc Thesis, Department of Civil Engineering, University of Ottawa, 2013.