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Seismic Design of a 27 Storey Transit-Oriented Community in Toronto

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ABSTRACT

The principle of transit-oriented communities (TOC) is to integrate transit, residential, and commercial spaces, enhancing city neighbourhoods and creating more complete places to live, work and play. The city of Toronto is expecting development of several TOCs in the next 10 years alongside the delivery of new rapid transit projects. This paper evaluates the seismic response for one prototype 27-storey reinforced concrete TOC structure and its seismic design requirements following NBCC code provisions. Results of linear dynamic analysis, showing interstorey drifts and shear forces, are used to illustrate the TOC's seismic response. This paper shows that seismic behavior can be affected by several key decisions made during the conceptual design, such as choice of Seismic Force Resisting System (SFRS), coordination constraints structural elements and residence layouts, connections to adjacent buildings. In addition, this paper also evaluates how seismic design of TOCs in Toronto can be impacted by the increase in spectral acceleration values presented in the 6th generation seismic hazard model for Canada. This model, currently implemented in NBCC 2020, is expected to be mandatory in Ontario in 2024 after the next update to the Ontario Building Code.

Keywords: transit-oriented community, reinforced concrete, building code

INTRODUCTION

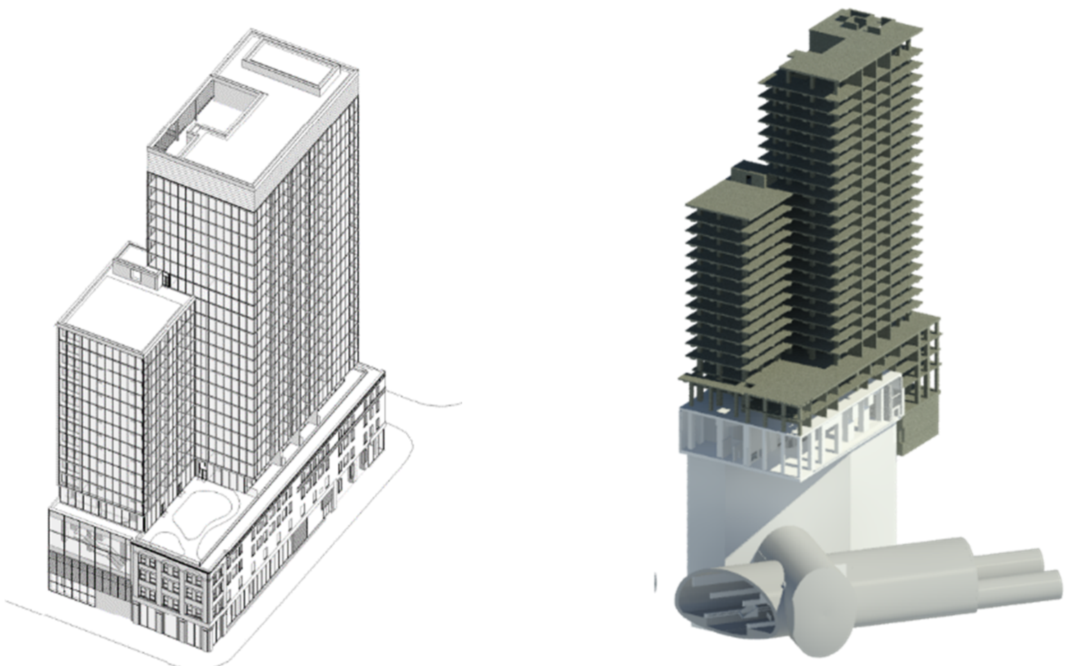
Transit-Oriented Communities (TOCs) comprise of high density, mixed-use developments. These structures are particular in that they integrate residential, commercial and transit infrastructure through a coordinated design approach.

Transit stations have long served to anchor commerce, recreation and retail, and small communities often emerge, centered about such hubs. Given the ever-increasing demand for additional housing within cities, TOCs offer increased density in these areas, providing transit-focused residences in prime locations while enhancing the communities in which they are built. Cities, including New York and London, have successfully demonstrated that with innovative design and a coordinated urban planning policy, TOCs are a viable way of creating integrated and vibrant communities. Toronto, Ontario is well positioned to adopt TOCs into its planning scheme. With the introduction of new transit lines through the city and its surrounding municipalities, opportunities exist to incorporate such structures along upcoming transportation networks. The low seismicity of the region also allows flexibility in the integration of these structures allowing certain structural irregularities which are not permitted in zones with higher seismic activity.

TOC structures are constructed directly adjacent to or directly above rail infrastructure (RI). While this is beneficial for the end-user, structurally, additional constraints are often generated through this arrangement. The degrees of interaction between the TOC and rail infrastructure largely dictate the complexity in the seismic design. While maintaining structural independence between the two structures is sometimes feasible, the following case study will examine a condition where the structures are integrated and use transfer structures at the connecting interface.

STRUCTURAL LAYOUT & COORDINATION

The site selected for the case study TOC is located within a congested city block. The property will house both the station headhouse - the aboveground subway building - and the TOC overbuild - the mixed-use development. Within the site boundary, only a limited area is unoccupied by the station headhouse. To maximize the TOC footprint, it is determined advantageous to fully overlap the structures. As such, a fully integrated solution, with the TOC overbuild constructed above the transit infrastructure, as shown in Figure 1, is advanced. Effectively, this results in a single structure built in two phases: The first being the construction of the transit infrastructure, and the second being the addition of the TOC overbuild.



(a) *(b)*

Figure 1. 3D view of 27 storey TOC overbuild and 3 storey transit station: (a) Architectural rendering, (b) Structural rendering with TOC structure (in gray) and station elements (in white).

Supporting Rail Infrastructure

The rail infrastructure consists of the above grade headhouse and the below grade entrance shaft and tunnel. The station headhouse is anticipated to be a double height single storey structure. The headhouse is supported on an entrance shaft, which descends 40 m to provide passengers access to the concourse and platform levels in the tunnel through a horizontal adit.

Two direct interfaces, the station roof, and the station south wall, are instrumental in allowing the integration of the TOC. The station roof provides a two-way transfer structure on which level 03 of the TOC is supported. This diaphragm distributes TOC loads to the station walls and columns below. The south wall provides a support for the floor slabs from level 03 down and supplies a continuous lateral load path from level 27 to the station foundation. The seismic requirements of the TOC-RI interface and its connections are presented in a separate study [1].

Description of TOC Structure

The case study TOC structure is developed as a residential building. As is common practice for high-rise residential buildings in Toronto, the structure uses cast-in-place reinforced concrete construction. As illustrated in Figure 1a, the structure is comprised of a 27 storey tower, with a change in floor area at storey 17, and a 5 storey podium structure. Two basement levels abut the south face of the station shaft and fill the area of the property not occupied by the station headhouse. A raft foundation, tied into the underground station shaft walls, supports the TOC structure which overhangs the station footprint. Above grade, levels 01 and 02 are equally constrained within the available site boundary while levels 03 to 05 extend over and above the station entrance roof, utilizing the length of the site. The footprint is then set back to form the tower. The upper tower, climbing to 89m, is located at the south end of the property with approximately 70% of its length descending to the station roof and the remainder falling within the off-station area. At the 17th storey, the tower extent is expanded, adding floorspace at the NE corner of the site and descending to the station structure roof.

The reference design attempts to align TOC column lines with the station walls below, minimizing the number of transfer structures where possible. TOC walls and columns are located above the rail infrastructure in an effort to provide equalized and useful grid divisions for a residential building, avoid interference with a retained heritage façade and accommodate the station architectural concept below.

SEISMIC DESIGN CODE REQUIREMENTS

At present, the seismic design of a TOC SFRS must follow provisions under the current OBC 2012 and as a result must meet NBCC 2015 requirements with seismic demands based on the 5th generation seismic hazard values [2,3,4]. However, because TOC buildings in Toronto are expected to be designed and constructed after the year 2025, the concept design presented herein follows expected design provisions in OBC 2024 [5]. These will align with NBCC 2020 provisions and demands based on the 6th generation seismic hazard values [6,7].

Site Conditions and Design Spectrum

Prior to the structural design, a geotechnical study of the site classifies the ground conditions as site class C. Further geotechnical investigation on the site, requested during structural design, reported the average shear wave velocity, V_{S30}, in the upper 30m of overburden to be 477 m/s.

The site design spectrum is first determined for a site class C condition and the site's coordinates, using the seismic hazard calculator for NBCC 2020 [7]. The resulting spectrum, $S_a(T, X_C)$, is the envelope of the spectral values corresponding to shear wave velocities, V_{S30}, ranging from 360 m/s to 760 m/s – which are shear wave velocities characteristic of site class C sites.

A comparison with the corresponding NBCC 2015 design spectrum, $S_a(T)$, shows an increase in the seismic hazard for all periods, as illustrated in Figure 2. The increase in spectral accelerations is measured as 1.45 times for short periods (T<0.5 sec), and 1.75 times for intermediate to long periods (0.5sec<T<5 sec). The increase in spectral values for the city of Toronto are mainly due to new Ground Motion Models (GMM) used in the 6th generation seismic hazard, which incorporate modern GMMs combined with a classical-weighted-GMM approach [8].

The final design spectrum is determined based on the site's measured V_{S30}, resulting in spectral accelerations, $S_a(T, X_{V=477m/s})$. This update provides a more precise measure of the seismic hazard for the site, as compared to using the envelope $S_a(T, X_C)$. The spectral accelerations for intermediate to longer periods (0.5sec<T<5 sec) reduce to 0.82 times the values based on site class, $S_a(T, X_C)$.

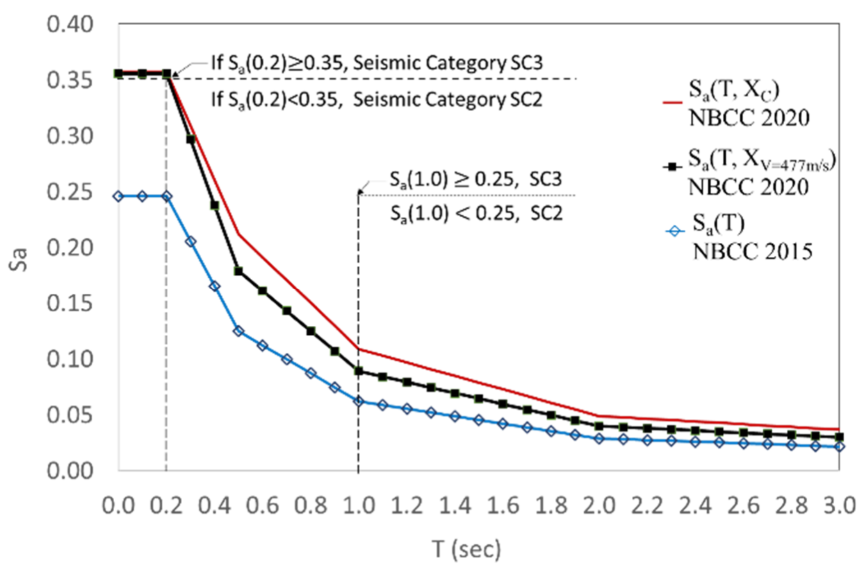


Figure 2. Design spectrum for TOC site based on NBCC 2015 and NBCC 2020

Seismic Category

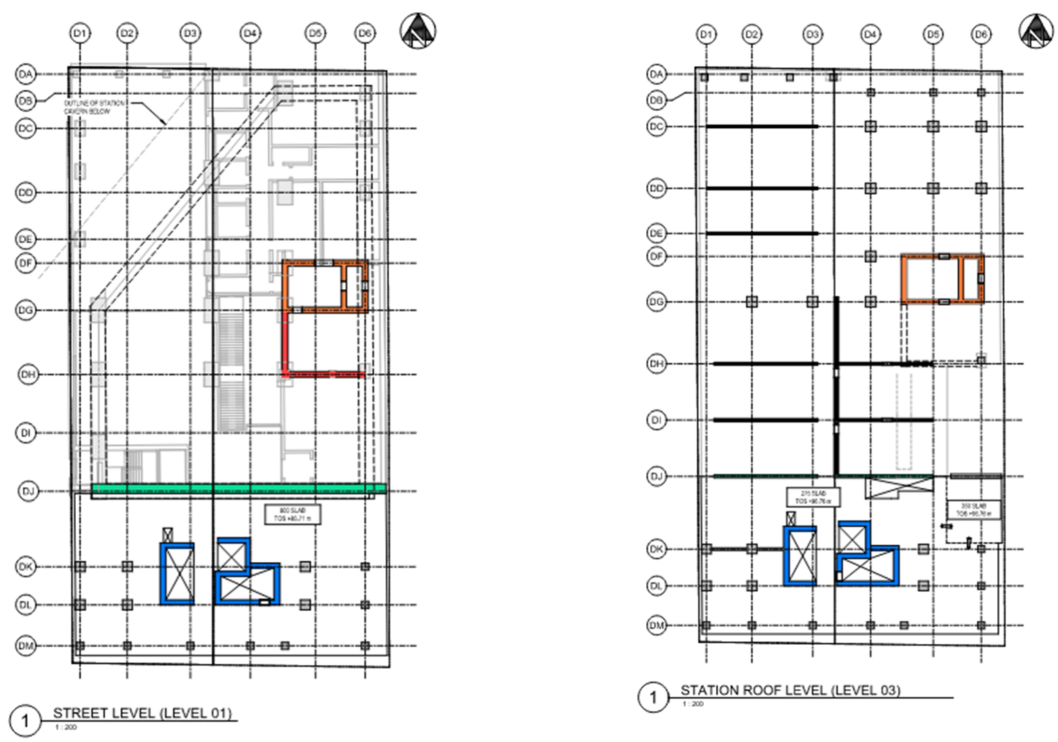
The TOC's importance factor and seismic category designation are determined in accordance with NBCC 2020 Tables 4.18.5A and 4.18.5B, respectively [6]. The TOC's importance factor, I_E, is equal to 1.0 (normal importance). The TOC's seismic category is SC3, given that $I_E S_a(0.2)$ is equal to 0.356 which is within the range 0.35 to 0.75.

Seismic Force Resisting System

The SFRS in the TOC overbuild consists of RC shear walls in the east-west direction and a combination of RC coupled walls and shear walls in the north-south direction. As the building exceeds the 40 m height restriction for conventional construction under seismic category SC3, moderately ductile requirements were used in design. In contrast, the station headhouse can be designed as conventional construction. Seismic design of RC shear walls for moderately ductile and conventional construction properties are designed following requirements in CSA A23.3:19 sections 21.5 and 21.6, respectively [9].

The placement of the SFRS elements within the overbuild take advantage of the TOC area overhanging the station structure, that is, the area south of gridline DJ. Elevator cores and stairwells were concentrated within this space to provide centered, continuous shear walls over the full height of the structure. This is beneficial in that neither a transfer through a seismic diaphragm, nor an interface with the station are required within this load path. In essence, the loads, both gravity and lateral, are directly carried through the TOC. These shear walls are highlighted in blue in Figure 3.

Within the station, the location of the SFRS elements were limited by the operational and aesthetic constraints imposed within the transit infrastructure. The structure aimed to protect the majority of its floor area to be open space, and as such, eliminated walls within its public spaces. This limited the opportunity to align walls in the TOC with those in the rail infrastructure. Functional locations for continuous RC shear walls were determined to be at the station's center-east section. In the station these walls are enclosed within the back of house area and serve to partition the space. Within the TOC, these form a set of stairs. These walls, between gridlines DF and DG, are shown in orange in Figure 3. Another continuous wall is located along the station's south end, on gridline DJ. This wall, shown in green in Figure 3, serves as the exterior south wall for the station, and divided into two in the TOC, partitions the residential units. Two additional shear walls in the station, shown in red in Figure 3a, form an L-shape between grids DG and DH and tie into the rectangular core. These, however, do not continue within the TOC levels. These wall elements will serve as SFRS for the station and for the TOC overbuild, after the latter is constructed.



(a) *(b)*

Figure 3. Structural wall layout: (a) Street Level , (b) Level 03 (Station Roof)

Several additional TOC SFRS elements, located above the station consist of discontinuous RC shear walls. These serve to partition the building units and act as primary gravity support elements over the height of the 27-storey tower. Within the station, these walls are supported vertically by station columns which carry the tower gravity loads while lateral forces are transferred through the seismic diaphragm onto adjacent RC shear walls. In the east-west direction, these walls are located on gridlines DI, DH, DE, DD and DC. In the north-south direction, the discontinuous wall is located east of grid D3. Of note, this final wall is a coupled shear wall, owing to the addition of door openings along its length. The use of coupling beams to join these walls segments affords the structure additional ductility in the north-south direction.

Plastic Hinge Region

The structural design is aimed for the plastic hinge region to develop at the base of the TOC overbuild and away from the station headhouse. This approach confines the inelastic behavior within the moderately ductile SFRS and can be used to limit the lateral forces transferred to the station headhouse. Accordingly, different types of SFRS systems will be incorporated into the design of the structure; expressly the station conventional construction shear walls and the TOC moderately ductile shear walls. To address this change in ductility, the seismic design of the station headhouse must consider the larger of the following forces [10]:

- The forces determined through analysis of the entire structure using the R_dR_o values for conventionally constructed shear walls, or;
- The design forces related to the lateral capacity of the overbuild.

Generally, this design approach permits capping the design forces to the seismic shear demand amplified such that R_dR_o equals 1.3. However, the use of this allowance is precluded when it does not prevent a weak storey.

Discontinuous Shear Walls

The presence of discontinuous shear walls in the TOC adds various considerations into the seismic design, as per NBCC 2020 provisions. A review of these provisions determines the following:

- Discontinuous shear walls are acceptable for this seismic design given that the design spectra's value Sa(1.0) is less than 0.25; as per per NBCC 2020 clause 4.1.8.10.3.
- The structure is classified as irregular, as per NBCC 2020 clause 4.1.8.6, which requires linear dynamic analysis and base shear scaling to 100% of V calculated based on NBCC 2020 clause 4.1.8.11.2.
- Given the TOC is designated as SC3, the supporting elements shall be designed for the lateral load capacity of the SFRS (NBCC 2020 clause 4.1.8.10.5) but need not exceed the design forces associated with a base shear, V_{ed}, with R_dR_o taken equal to 1.3 (NBCC 2020 clause 4.1.8.15.8).
- A weak storey formation is not permitted for the conditions of the TOC site, as per NBCC 2020 clause 4.1.8.10.1. The structural analysis will need to evaluate the discontinuity in lateral capacity by determining the story shear strength at the base of the TOC overbuild and designing the station below to have a greater storey shear strength.

ANALYTICAL MODEL

A 3D structural model of the TOC structure is developed using the commercial software ETABS [11]. The model includes overbuild, station headhouse and station substructure elements. Shear wall elements and transfer beams are modeled using shell elements, supporting columns and coupling beams using frame elements and floor slabs modeled using shell elements with semi-rigid diaphragm constraints.

Shear walls and coupling beams are assigned stiffness modifiers in accordance with CSA A23.3:19 Table 21.1[9]. Stiffness modifiers for shear walls, α_w, are dependent on the SFRS's R_dR_o value and the wall overstrength factor, γ_w. The initial analysis uses α_w equal to 0.65 based on an assumed value of overstrength equal to 1.4. Final analysis use α_w equal to 0.9 based on the calculated value of γ_w equal to 2.15 obtained for the y direction.

Station columns supporting discontinuous walls are assigned moment releases at top and bottom ends. This approach is taken to ensure lateral forces from discontinuous walls are not taken by the supporting columns but instead transferred through diaphragm action to adjacent shear walls.

The model, and the associated results, assign the x-axis to the east-west direction and the y-axis to the north-south direction.

LINEAR DYNAMIC ANALYSIS

Analysis Approach

The analysis in ETABS consists of running the response spectrum analysis (RSA) method using the site's design spectrum for a damping ratio, ξ, of 5%. The analysis is performed in accordance with NBCC 2020 clause 4.1.8.12. Modal analysis is checked to have a minimum 90% mass participation for both directions. The resulting base shear is scaled to 100% of the specified lateral earthquake force, V, calculated in accordance with NBCC 2020 clause 4.1.8.11.2. Seismic loads are also enveloped to consider accidental torsion due to a 10% eccentricity.

Interstorey Drifts and Shear Forces

From modal analysis, the first modes of vibration are presented in Table 1. Design base shear forces, V_{ed_x} and V_{ed_y} are equal to 12,900 kN and 12,100 kN, respectively.

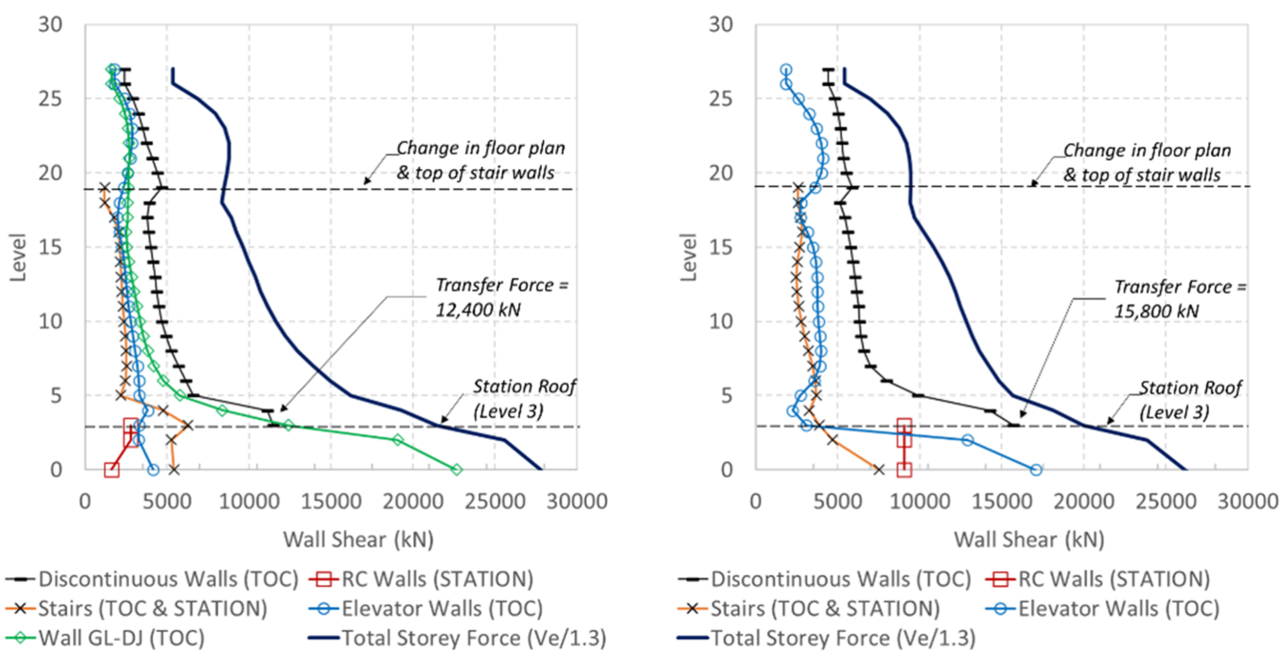
Table 1. First Modes of Vibration

Direction	Period, T (s)
X	1.94
Y	2.22
XY	1.77

For the TOC overbuild, the maximum interstorey drifts, Δ_{i_x} and Δ_{i_y} are equal to 0.13% and 0.20%, respectively; with maximum roof drifts, Δ_x and Δ_y, both equal to 0.10%. For the station headhouse, the maximum interstorey drifts, Δ_{i_x} and Δ_{i_y} are equal to 0.17% and 0.09%, respectively.

Design Forces and Distribution

The shear force diagrams of the RC shear walls are compared to the total storey shear in Figure 4. The lateral forces used in the comparison are amplified to the upper bound lateral force limit in accordance with NBCC 2020 clause 4.1.8.15.(8), which corresponds to multiplying RSA results by a factor of 2.15. The upper bound limit is used because it is expected RC shear walls will present some shear amplification due to flexural overstrength and higher mode effects [12].



(6) (b)

Figure 4. Upperbound shear demands : (a) X direction, (b) Y direction

The comparison of lateral forces shows that discontinuous shear walls resist between 40% to 57% of the total storey shear in the east-west (X) direction and between 48% to 82% in the north-south (Y) direction. In contrast, the TOC elevator walls, resist an average of 26% of the storey shear in the east-west direction and between 31% in the north-south direction. This difference in forces is likely due to the discontinuous walls having a lateral stiffness greater than that of the elevator walls.

The forces in the discontinuous walls consistently increase from top to bottom with only a minor reduction noted where the stair core is introduced at level 19. A large increase in shear occurs at level 04, due to greater floor mass resulting from deep slab supporting a green roof.

At the base of the discontinuous walls, the shear is transferred through the station roof. In the X direction, the majority of the 12,400 kN shear force is displaced to the shear wall on gridline DJ. As a result, the shear force in this wall increases from 12,500 kN to 19,100 kN. In the Y direction, of the 15,800 kN load in the coupled wall, 62% is transferred to the TOC elevator walls and 38% is transferred to the RC station walls. The shear in TOC elevator walls increases by a factor of 4, from 3080 kN to 12900 kN. At grade, TOC elevator walls resist 65% of the storey shear.

STOREY STRENGTH ANALYSIS

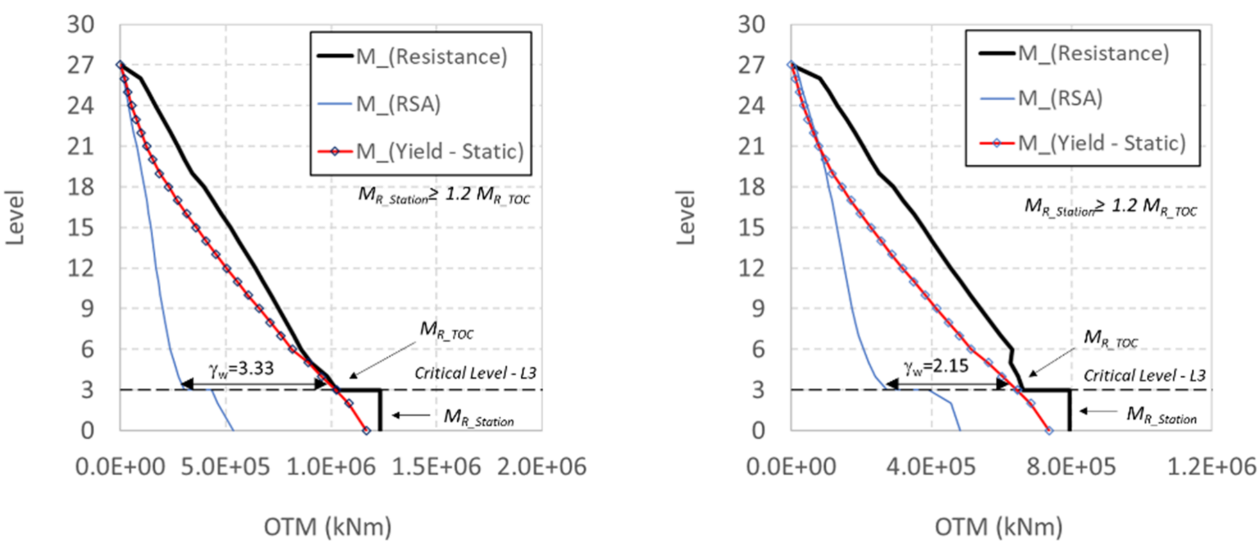
Analysis Approach

A linear static analysis procedure is used to determine the storey shear strengths for the TOC overbuild and the station below. The procedure consists of three steps. First, the SFRS's $M_{Resistance}$ is determined as the sum of moment resistance per storey of all elements considered part of the SFRS. Second, the static load pattern from NBCC 2020 clause 4.1.8.11. (7) is applied to the SFRS, shown in Eq. (1) and Eq. (2); and is increased until its total shear force, V_{TOC}, produces the moment demand, M_{yield_static}, equal to the moment resistance, $M_{Resistance}$, at the base of the TOC overbuild. This represents a plastic hinge formation at level 03. Third, a separate analysis is done, applying the load pattern onto the SFRS where the total shear force, $V_{Station}$, is increased until moment demands indicate a plastic hinge formation at the base of the station. And fourth, the storey shear strength, V_{TOC}, is checked to be less than the storey shear strength, $V_{Station}$, proving the prevention of a weak storey mechanism.

$$F_x = (V - F_t)\, W_x h_x / \left(\sum_{i=1}^{n} W_i h_i\right) \quad (1)$$

$$F_t = (0.07)\, T_a V \leq 0.25V \quad (2)$$

Storey Strengths and Moment Resistance



(a) *(b)*

Figure 5. Moment resistance, moment demand and moment at yield plots: (a) X direction, (b) Y direction

The results of the storey strength analysis consist of the shear storey strengths for the TOC overbuild, V_{TOC}, and for the station headhouse, $V_{Station}$. The moment diagram at yield, $M_{Yield\ Static}$, and the nominal moment resistance diagram, $M_{Resistance}$ are also produced from this analysis. These results are used to test strength discontinuity, determine the plastic hinge region, and calculate the SFRS moment overstrength. Resulting moment diagrams are compared to those obtained from linear dynamic analysis, M_{RSA}, as shown in Figure 5.

Evaluation of Strength Discontinuity

This study found that to develop yielding at the station roof, the moment resistance below the station, $M_{R\_Station}$, needed to be equal to or greater than 1.2 times the moment resistance of the TOC overbuild, M_{R_TOC}. As a result, the station storey strength, $V_{Station}$, is equal to or greater than 1.05 times the TOC overbuild strength, V_{TOC}.

In the east-west direction, the calculated storey strengths for the TOC overbuild, V_{TOC}, and the station headhouse, $V_{Station}$, are equal to 13450 kN and 14250 kN, respectively. Similarly, in the north-south direction, the storey strength values, V_{TOC}, and $V_{Station}$, are equal to 8600 kN and 9000 kN, respectively. This confirms that in both directions, that the station storey strengths exceed the TOC storey strengths, and a weak storey mechanism is prevented. The plastic hinge region will form at the base of the overbuild.

SFRS Overstrength

The SFRS' flexural overstrength, γ_w, measured in the east-west and north-south directions, are 3.33 and 2.15, respectively. These values are determined as the ratio of nominal moment resistance, M_{R_TOC}, and M_{RSA} at the station roof. The high values are attributed to the use of structural walls as primary gravity support elements and the large wall-to-floor area ratio in the overbuild. Based on these results, by amplifying the SFRS base shear by the respective overstrength, the maximum base shear forces at yield, are 43,000 kN and 26,000 kN in the X and the Y directions, respectively.

The SFRS overstrength values are equal to or greater than the upper bound limit factor of 2.15 for moderately ductile walls. This indicates that the SFRS members, for the east-west direction will not dissipate energy through inelastic response for the design level earthquake. In the north-south direction, the SFRS overstrength value of 2.15 suggests that minor yielding may develop in the coupling beams, while hinging at the wall bases is not expected.

Plastic Hinge and Yield Mechanism

Given the findings above, at the design level seismic event the SFRS response is likely to remain approximately elastic and not develop a plastic hinge at the base of the TOC overbuild. For a greater seismic demand, the plastic hinge will occur at the base of the TOC overbuild while the station headhouse remains elastic, thus preventing a weak storey mechanism.

CAPACITY DESIGN & DESIGN RECOMMENDATIONS

Notable in this TOC design is the risk of discontinuity in storey strength caused by discontinuous walls, which is worsened by the walls' level of overstrength. Generally in NBCC 2020, provisions that require capacity design allow capping the design forces to the upper limit, corresponding to lateral forces assuming R_dR_o equal to 1.3. However, this limit is effectively restricted by code provisions to prevent weak storey mechanisms as well as vertical variations in R_dR_o (Commentary to NBC Sentence 4.1.9.8.9.(4)). The authors conclude that the restriction in the use of the upper limit for this TOC design is effective in preventing the risk of a weak storey mechanism. Additionally, the authors suggest that the design of similar structures should avoid an overstrength that is greater than the upper bound shear limit, where feasible.

In order to capacity protect the SFRS from a weak storey mechanism, capacity design must be implemented for structural elements at and below level 03. Below are recommended steps for columns supporting shear walls, seismic diaphragms and station shear walls.

The station columns supporting discontinuous walls must be designed to resist the axial loads that develop from the wall's vertical load and the seismic overturning moment. To ensure yielding occurs in the wall, the column must also be designed for an axial tension strength greater than the yielding strength of the walls concentrated vertical reinforcement. It is recommended that the moment couple developed be 1.2 times greater than the wall moment resistance, which is based on the results of the storey shear strength analyses.

To keep the level 03 diaphragm elastic during seismic response, as per NBCC clause 4.1.8.15.(1), it must be capacity designed for the inertial forces developed on the diaphragm and the transfer forces from the discontinuous shear walls. The transfer forces based on the lateral capacity of the discontinuous walls, are equal to 19,200 kN and 15800 kN for the east-west and north-south directions, respectively.

The capacity design of shear walls at level 3 requires the wall's design shear strength to correspond to either the lateral capacity of the TOC or to the linear dynamic results amplified by the SFRS overstrength. Furthermore, the design value may not be limited by the upper limit in NBCC 2020. This prevents a shear sway mechanism in the shear walls that resists the forces transferred by the discontinuous walls above.

COMPARISON OF TOC DESIGN BASED ON SEISMIC CATEGORY SC2 AND SC3

For the city of Toronto, a building's seismic category designation can vary between SC2 and SC3. Spectral acceleration Sa(0.2) values under NBCC 2020 can vary from 0.28 to 0.38 for site class C [8], crossing the short-period signal value. If Sa(0.2) is less than 0.35 the designation will be seismic category SC2 and if Sa(0.2) is equal to or greater than 0.35 then it will be SC3. The difference in designation of seismic category between SC2 and SC3 impacts the minimum seismic design requirements.

This study evaluated the TOC site's spectral values with varying shear wave velocities and found the cutoff point, i.e. the minimum value of V_{S30}, where Sa(0.2) is less than 0.35. There is variation of spectral values for all periods for the range of shear wave velocities representative of site class C ground conditions, as illustrated in Figure 6. The cutoff point occurs for a V_{S30} value equal to 520 m/s, with a greater V_{S30} resulting in a seismic category SC2 and a lower V_{S30} in SC3. It is noted that the TOC's measured shear wave velocity, V_{S30}, is equal to 477 m/s which is 8% below the cutoff point.

TOC sites in Toronto with small differences in shear wave velocities can be assigned different seismic categories and subsequently different seismic design requirements. To evaluate possible differences, the TOC design described in earlier sections, herein referenced as SC3 TOC design, is compared to an alternative TOC seismic design based on seismic hazard values for a V_{S30} equal to 520m/s and following SC2 design requirements. This alternate design will herein be referred to as SC2 TOC design. The design parameters compared are: SFRS restrictions, ductility design, elements supporting discontinuous walls, members not considered part of the SFRS and estimated SFRS strength and roof drift capacities.

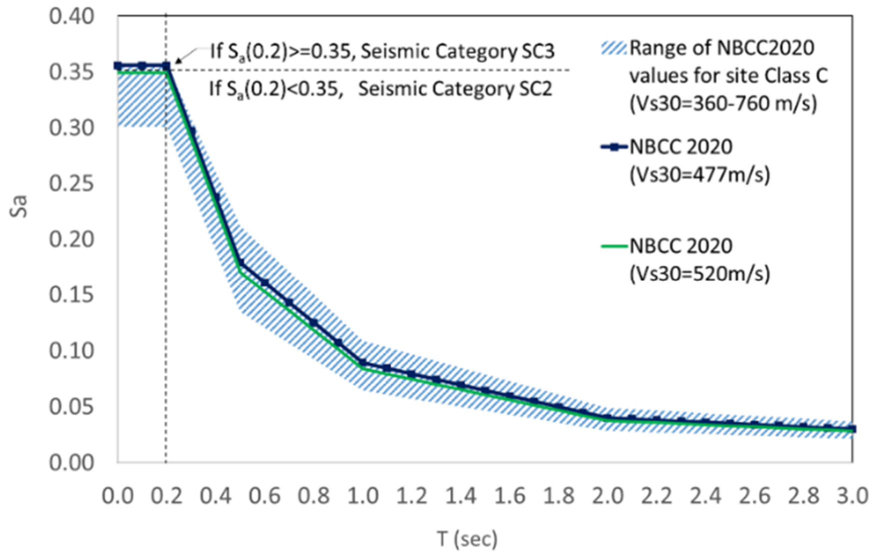


Figure 6. Variation of TOC design spectrum vs shear wave velocity, V_{S30}

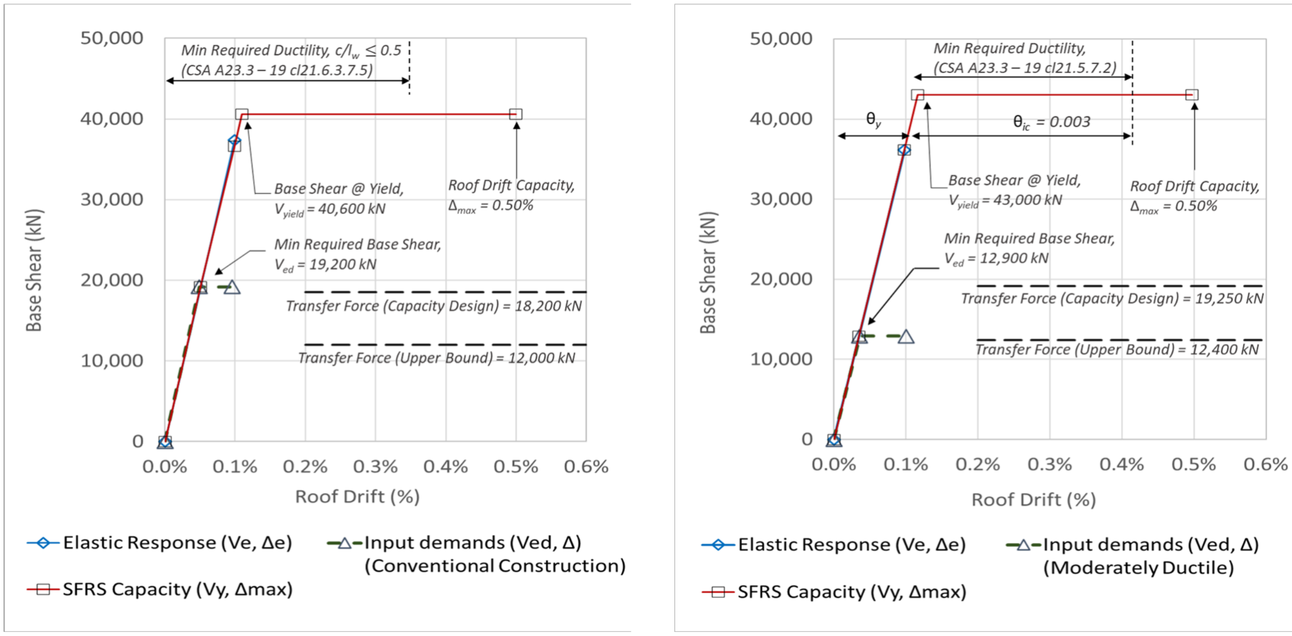
SFRS Restrictions and Ductility Design

The SC2 and SC3 designs have different SFRS types due to restrictions based on site category, as per NBCC 2020 clause 4.1.8.9. Under seismic category SC2, SFRS can be designed as conventional construction without limits to the building height. Under SC3, conventional construction cannot be implemented for the 89m high TOC building due to the 40m height limit. Therefore, the SFRS under SC3 must be designed at minimum as moderately ductile. Due to the difference in SFRS type, the two designs differ in seismic detailing and ductility checks, as illustrated in Figure 7.

In the SC2 TOC design, conventional construction shear walls are designed to have the length of neutral axis, c, limited to half the wall length, l_w, as per CSA A23.3:19 clause 21.6.3.7.5. This design requirement ensures, at minimum, a total rotation capacity, of 0.35%, as calculated by Equation (3). In the case of SC2 TOC design, the maximum c/l_w ratio is equal to 0.35 which corresponds to a wall rotation capacity, θ, equal to 0.50%.

$$\theta_{total} = \frac{\varepsilon_{cu}\, l_w}{2c} \tag{3}$$

For the SC3 TOC design, the moderately ductile shear walls must be designed for the minimum inelastic rotational demand, θ_{id}, of 0.003, in accordance with clause 21.5.7.2 of CSA A23.3:19. In the case of the SC3 TOC design, the maximum neutral axis length to wall length ratio, c/l_w, of 0.36 corresponds to an inelastic rotational capacity, θ_{ic}, of 0.003, as per equation 21.12 in CSA A23.3:19.



(a) *(b)*

Figure 7. Seismic design force vs roof drift: (a) SC2 TOC design, (b) SC3 TOC design

The two ductility designs differ in minimum rotation capacity requirements. Conventional construction indirectly provides a minimum total rotational capacity, θ_{Total}, through the c/l_w ratio. Moderately ductile design requires a post-yield rotational capacity, θ_{ic}, which must be greater than the estimated inelastic rotational demand, θ_{id}, but shall not be less than 0.003. In general, a design following moderately ductile design requirements is found to provide more detailing and calculation requirements than conventional construction. These provisions add ductility capacity to the SFRS and increases their ability to achieve a life-safety performance for intensity levels equal to or greater than the design level earthquake.

Elements Supporting Discontinuous Walls

A distinction between the two designs is provision 4.1.8.15.(5) in NBCC 2020. This clause indicates that elements supporting a discontinuous wall shall be designed to the wall's lateral capacity if the site is seismic category SC3 or SC4. In the case of seismic category SC1 or SC2, the same elements are designed based on loads from the lateral analysis. Despite this difference in requirements, both designs result in the RC columns supporting the TOC walls being designed for the corresponding wall strength. In order ensure plastic hinge formation at the TOC wall above the station, (NBCC 2020 clause 4.1.8.15.(6)), and to prevent weak storey formation (NBCC 2020 clause 4.1.8.15.(1)) the higher design standard must be met.

With respect to the wall elements, the design value for the shear force transferred to adjacent RC walls is capacity protected for SC2 TOC, in accordance with CSA A23.3:19 clause 21.6.3.4. It is expected that a similar approach to that used in the SC3 TOC design, neglecting the upper limit, would be utilized in the SC2 TOC design. This results in a similar design shear value for conventional construction as that established in the moderately ductile design.

In the case of the seismic diaphragm, both designs follow capacity design as it is required for all seismic categories, based on NBCC 2020 clause 4.1.8.15 (1). The resulting minimum design values for the discontinuous shear walls' transfer shear force for SC2 TOC and SC3 TOC are 18,200 kN and 19,250 kN, respectively. The lesser SC2 TOC design value is due to the lower minimum concentrated vertical reinforcement requirement in CSA A23.3:19 clause 21.6.3.7.4 compared to that for moderately ductile walls in CSA A23.3:19 clause 21.5.6.2.

Members not considered part of SFRS

Two conditions exist, as per CSA A23.3:19 clause 21.11.1.1, which eliminate the need to design for seismic demands in RC column, slab, and beam elements. The first is a low seismic category, and the second is a maximum interstorey drift at any level of less than 0.50%. For the SC2 TOC, this design step is not required given the seismic category SC2 meets the first condition. Comparatively, the SC3 TOC does not meet the low seismic category condition, but does fall under the second condition. The structure's maximum interstorey drift is 0.17%, and as such, seismic design of the non-SFRS members is not required.

Demand vs Capacity

Despite differences in SFRS types, both SC2 TOC and SC3 TOC designs result in lateral strengths, V_{yield}, with high overstrength compared to the design base shear, V_{ed}. For loading in the east-west direction, the SFRS strength, V_{yield}, exceeds the elastic seismic demand, V_e, which indicates the SFRS will remain approximately in the elastic range for the design level earthquake, as illustrated in Figure 7.

Similarly, roof drift capacities, Δ_{max}, for SC2 TOC and SC3 TOC are both equal to 0.50% which is greater than the design level roof drift demand, Δ, equal to 0.10%.

Findings of the Comparison

This comparison finds that, for the studied 27 storey TOC, the seismic design following NBCC 2020 provisions for seismic category SC3 is similar in strength, ductility and capacity design requirements than for a design following provisions for seismic category SC2.

CONCLUSIONS

The analysis and preliminary design were undertaken for a TOC structure located in Toronto, Ontario. The seismic design of the 27 storey structure, due to its location and V_{S30} values necessitated that it be designed following NBCC 2020 provisions. Increases in the hazard values between the 2015 and 2020 national codes, increased seismic demands for the TOC structure by a factor of approximately 1.45. Moreover, this increase subsequently led to design Sa(0.2) values in the range of 0.28g to 0.35g and triggered seismic category SC3 design requirements. Previous seismic hazard values in Toronto were below this limit. Investigating the impacts of this change, this study found the TOC case study designed following NBCC 2020 provisions for seismic category SC3 resulted in similar strength, maximum drift and capacity design properties as those designed for requirements under seismic category SC2.

The integration of the overbuild with the transit infrastructure resulted in several structural features which imposed additional seismic design requirements. Most notable of these were the inclusion of many discontinuous walls and the variation in the ductility of the SFRS between the station headhouse and the overbuild. These irregularities are both permitted in low to moderate seismic zones. It is expected that future TOC structures directly supported by rail infrastructure would equally include these structural irregularities. The design choices made in this study are intended as practical solutions in meeting the functional needs of the differing buildings.

This study determines that the overbuilds' lateral capacity is equal to or greater than the upper bound design limit in NBCC 2020 provisions. In order to protect the station from developing a weak storey formation it is recommended that the station structure be designed for the lateral force capacity of the future overbuild. The minimum code requirement to cap design forces to the upper bound limit, which in concrete, corresponds to linear dynamic analysis forces using R_dR_o equal to 1.3, is not applicable for this TOC's structural arrangement.

In similar structures, the authors recommend that overstrength values greater than the upper bound shear limit be avoided. The findings in this study indicate that, in these cases, prevention a weak storey mechanism requires capacities which exceed the upper bound shear limit. Consequently, very strong elements are designed. These will remain fully elastic beyond the design earthquake loads and their inherent overstrength and ductility go unused. In principle, these high overstrength values appear to suggest opportunities for added efficiencies in the design.

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