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Seismic Upgrade Using External Buckling Restrained Bracing and Performance-Based Design

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ABSTRACT

An existing four storey steel moment frame office building was upgraded to 100% of NBC 2015 code requirements, a first of its kind in Canada employing the supplemental energy dissipation provisions of NBC 2015. The innovative design allowed the upgrade work to be limited to the ground and two basement levels of the structure with no work in the upper three floors such that all levels remained operational and accessible during construction. The upgrade included addition of Buckling Restraint Braces (BRBs) externally at the ground level for added stiffness and strength to eliminate the soft-storey mechanism. The upgrades in the basement included addition of concrete shear walls and Fibre Reinforced Polymers (FRP) diaphragm upgrades. The structural analyses included performing a combination of non-linear pushover and non-linear time-history analyses through an iterative approach to optimize the properties of the BRBs. In this novel approach, non-linear backbone curves for each level of the structure were obtained from pushover analyses of each individual storey and implemented in a simplified 2-dimensional stick model of the structure along with preliminary BRB properties for performing non-linear time-history analyses. Peak displacement and drift profiles along the height of the structure were obtained and were treated as acceleration profiles to perform non-linear pushover analyses on the detailed model. Performance point of the structure was obtained per Equivalent Linearization Method of FEMA 440 and peak BRB displacement demands and moment frame hinge distribution along the height of the structure were checked at the performance point. By utilizing the strengths and capabilities of each analysis and taking a performance-based approach, it was possible to efficiently analyze and design an upgrade solution to 100% of code requirements, while significantly reducing the disruption to the normal operation of the building and reducing the cost compared to a conventional upgrade concept.

Keywords: Performance-based design, seismic upgrade, buckling restrained bracing, non-linear analysis, steel moment frame

INTRODUCTION

The City of Vancouver's 515 West 10th Avenue building (also called the West Annex building) was built in 1975 and houses four floors of office space with extensive public use. The building is very unconventional in that it is a four storey steel moment frame in both orthogonal directions with two storeys of parking in the basement made with precast concrete columns, beams and floor planks. This is in contrary to the typical all cast-in-place concrete or braced steel frame structures for buildings of this scale and era, especially in British Columbia. Other unique characteristics of the building include floor overhangs in the top two levels increasing the floor space in the upper floors relative to the lower ones and the presence of heavy precast concrete cladding making this a top heavy building. The main structural deficiency of the building was the soft-storey at the ground level of the building.

The seismic upgrade was achieved by making use of buckling restrained bracings (BRBs) external to the building, a first in Canada. Ausenco developed this solution from preliminary design, through analysis and detailed design, and provided site services during construction. In taking a performance-based approach in the design and detailing of the seismic upgrade solution, the design was modified from the City's conceptual upgrade design such that the upgrade work was limited only to the ground floor and basement levels and mostly external as opposed to substantial internal upgrades in all six levels in the original design concept. Two precast concrete plank floor diaphragms were upgraded and drag struts installed in two basement levels by extensive use of fibre reinforced polymers (FRP), along with unique first-in-Canada FRP-to-steel connection details.

Buckling restrained bracing, a technology developed in Japan in the 1980s, has been used sparingly in North America in the past 20 years, and there are only four other seismic upgrade projects in Canada utilizing BRBs (all internal to the building). A BRB consists of a light steel section surrounded by grout not bonded to the steel, enclosed in an outer steel tube. The inner steel section can yield in both tension and compression (freely move within the grout), thus dissipating much more energy than conventional bracing, in a stable and predictable manner.

In this seismic upgrade, the intent was to achieve a performance level equal to that of a new building designed to 100% of the seismic demands as per the National Building Code of Canada 2015 (NBC 2015). To achieve this performance level and optimize the properties of the BRBs, a performance-based analysis and design approach was required involving non-linear analysis of both the BRBs and the original structure. The concept was to limit the location of the BRBs to only the ground floor and to locate them externally to the office spaces to minimize disturbance to the public areas. A unique feature of the design included inclination of the plane of the BRBs to optimize resistance in each orthogonal direction of the building. Only 16 new BRB members were required to achieve the very cost effective upgrade solution.

DESIGN OBJECTIVES

The original seismic upgrade concept involved a conventional forced-based upgrade scheme with work required on every floor level of the building and basement. As per the prevailing City bylaw, the building was to be upgraded to at least 75% of the new building code force level requirements. Furthermore, the client wanted an upgrade concept that was cost effective with minimal impact and disruption on tenants, building staff and the public users for the duration of the construction.

The design objective was to improve on the original upgrade concept in all aspects; this included designing a seismic upgrade solution to 100% of the new code requirement (hence exceeding on the original objective) while reducing the cost and further minimizing the impact on the building users such that all public and office spaces remained accessible for the entire duration of the construction. Through a performance-based approach following the latest provisions in clause 4.1.8.21 of NBC 2015 for supplemental energy dissipation devices [1], combined with the novel analysis algorithm developed for this upgrade, the solution eliminated the need for any upgrade work on the 2nd to 4th levels with most of the work at ground level performed external to the building.

STRUCTURAL UPGRADE LAYOUT

The layout of the structural upgrade included a set of eight BRBs on each side of the building oriented in an inclined plane. On each side of the building a pair of V-configuration BRBs resisted load only along the building length while four other inclined BRBs resisted load in the other orthogonal direction. The upgrade required only a total of 16 BRBs at the ground level in order to eliminate the soft-storey mechanism, as shown in Figure 1. In addition to the BRBs, steel drag struts connected to the top of the BRBs were designed and implemented into the ground floor ceiling. Concrete shear walls were added in the two parking (basement) levels in order to provide a continuous load path for the seismic loads and effectively transfer the seismic demands from the BRBs to the foundation. Furthermore, the precast concrete floor planks at the basement level were "stitched" together using fibre reinforced polymers (FRP) and FRP drag struts were also added which transferred the seismic demands to the new concrete shear walls.

Figure 1. Two pairs of V-configuration BRBs and four inclined BRBs on each side of the building, external to the building (seven of eight on one side in left photo; BRBs visible in lowest floor, left portion in right photo).

STRUCTURAL ANALYSIS

The main structural lateral load resisting deficiency which resulted in this building being categorized as high seismic risk was the presence of a soft-storey mechanism at the ground level of the building. This was due to the first storey height exceeding the storey height of the floors above by about 350 mm and having a 'near-pinned' connection at the base of the steel columns. For the purpose of modeling and analyzing the structure, a detailed 3-dimensional model of the steel moment frame was created in SAP2000, as shown in Figure 2, with non-linear hinges assigned to all steel beam and column elements throughout the model. The detailed 3-dimensional model was used in parallel with a simplified stick model representation of the structure in a series of non-linear pushover and non-linear time history analyses conducted in an iterative loop to fine tune and optimize the properties of the BRBs and the overall performance of the structure such as drift and hinge formation along the height of the structure.

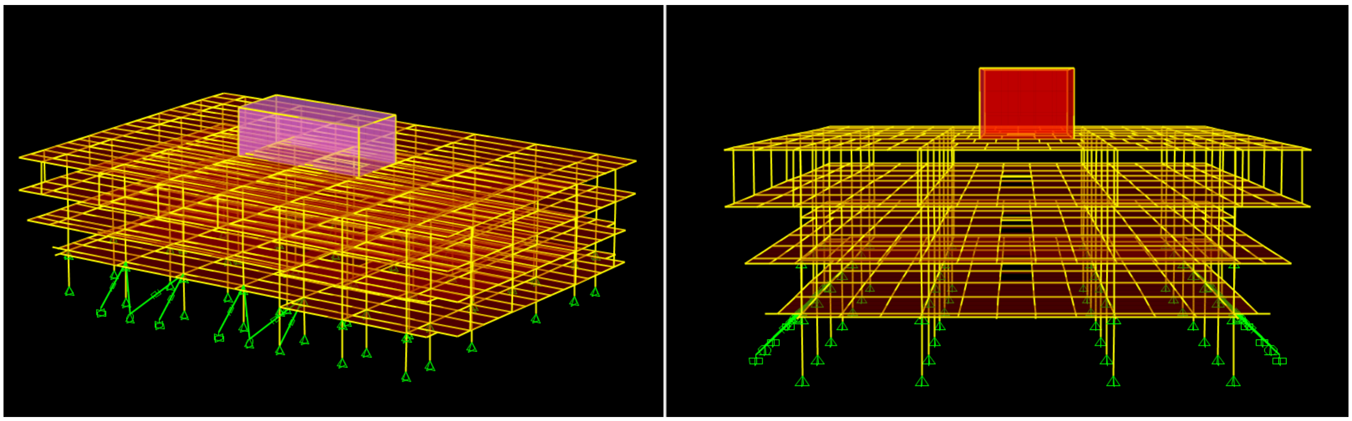

Figure 2. Views of the 3-dimensional model created in SAP2000, four storey steel moment frame portion of building.

Original Structure

The first step in the analysis was to identify the backbone curves of the individual floor levels from the detailed 3-dimensional model. In doing so, non-linear pushover analysis of each floor relative to the floor below with P-delta effects included was conducted for each orthogonal direction of the building. The results from the storey pushover analysis are shown in Figure 3 for every floor of the structure with the BRBs included, for each orthogonal direction. The presence of a soft-storey mechanism with negative post-yield stiffness in the ground level of the structure can be seen in the storey backbones for the 1st floor.

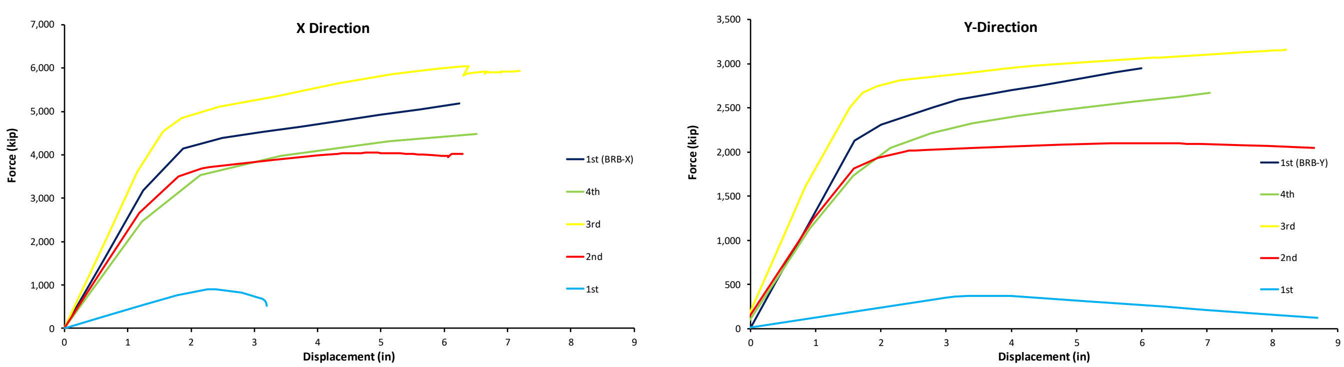

Figure 3. Non-linear storey backbone curves obtained from non-linear pushover analysis on each level, each orthogonal direction, including new BRBs on 1st level.

Based on the storey pushover plots obtained, a simplified stick model of the structure was created in SAP2000 whereby each level of the structure was represented by a non-linear link element with the corresponding non-linear backbone curve shown in Figure 3 assigned to the link elements and the storey masses lumped at the corresponding floor levels. The backbone curves for each orthogonal direction were assigned independently and hence the simplified model shown in Figure 4 represented a 2-dimensional representation of the structure used primarily for conducting time-history analysis in each direction. Preliminary BRB properties for initiating the design and property optimization process were estimated by assigning

BRB properties to obtain a desired backbone for the first floor as shown in Figure 3. Note that the eight diagonal BRBs were represented by two elements and the four pairs of V-configuration BRBs were represented by four elements.

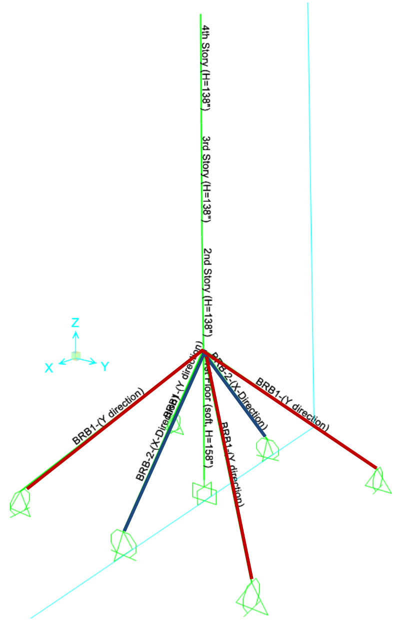

Figure 4. Simplified stick model of the structure for use in non-linear time-history analysis.

Time History Analysis

For conducting the time-history analyses, three suites of ground motion time history records were considered; 11 subduction (off Vancouver Island) records, 5 crustal (shallow, local) records, and 5 inslab (deep, local) records for a total of 21 records. The selection and scaling of the suite of ground motions was consistent with that outlined in Commentary J of NBC 2015.

Fast non-linear time-history analysis (FNA) available in SAP2000 was carried out on the simplified model for the selected suite of ground motions and the 90° rotated motions. For each suite of motions ran, the maximum drift and displacement profiles along the height of the structure were output and the median, mean and mean plus/minus one standard deviation for each suite of motions were plotted (results for one suite shown in Figure 5) and the results were reviewed. The properties of the BRBs such as yield strength and effective stiffness were fine tuned and adjusted accordingly and the set of time-history analyses was repeated until the desired system performance was achieved. By utilizing the pushover outputs from a detailed 3-dimensional model and then implementing it in a simplified representation of the structure, combined with the efficient algorithm of FNA tool available in SAP2000, it was possible to analyze the structure for 21 records of ground motions in 42 sets of analyses in an efficient and timely manner.

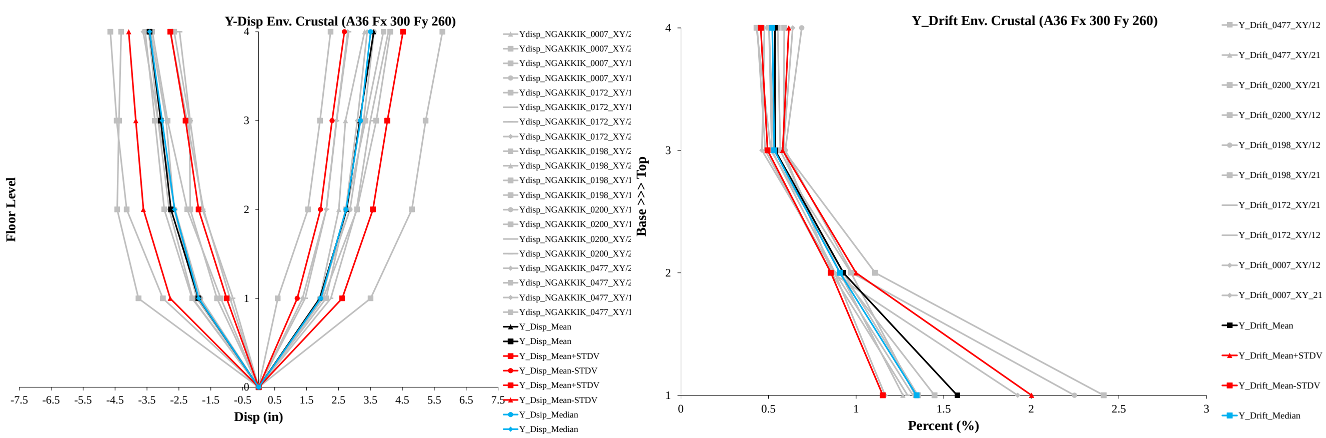

Figure 5. Displacement (left side of figure) and drift (right side of figure) profiles obtained from 2-dimensional time-history analysis for the suite of crustal ground motions.

Pushover Analysis

After the desired properties of the BRBs were obtained from the simplified time-history analyses, the detailed 3-dimensional model was updated with the optimized BRB properties for the purpose of performing non-linear pushover analyses on the structure. The load profile applied in the pushover analyses was obtained by treating the mean displacement profiles output from the time-history analyses (with the optimized BRBs) as acceleration profiles acting along the height of the structure and the corresponding seismic mass at each level.

Non-linear pushover analysis was conducted on the structure for each orthogonal direction of the building and the output of the analysis was used in the equivalent linearization method of FEMA 440 in order to obtain the performance point of the structure on the pushover curve for the design spectrum. In the equivalent linearization method available in SAP2000, the pushover curve of structure as well as the design spectrum are converted into equivalent spectral acceleration versus spectral displacement plots and are positioned on the same chart. The performance point along the pushover curve is obtained through an iterative approach by finding the intersection between the pushover curve and the modified (damped) design spectrum. The modified design spectrum is computed for an effective structural damping associated with an effective structural period for a given S_a, S_d value along the pushover curve. The procedure is illustrated in Figure 6; further details on the specifics of the procedure can be found in FEMA 440 [2].

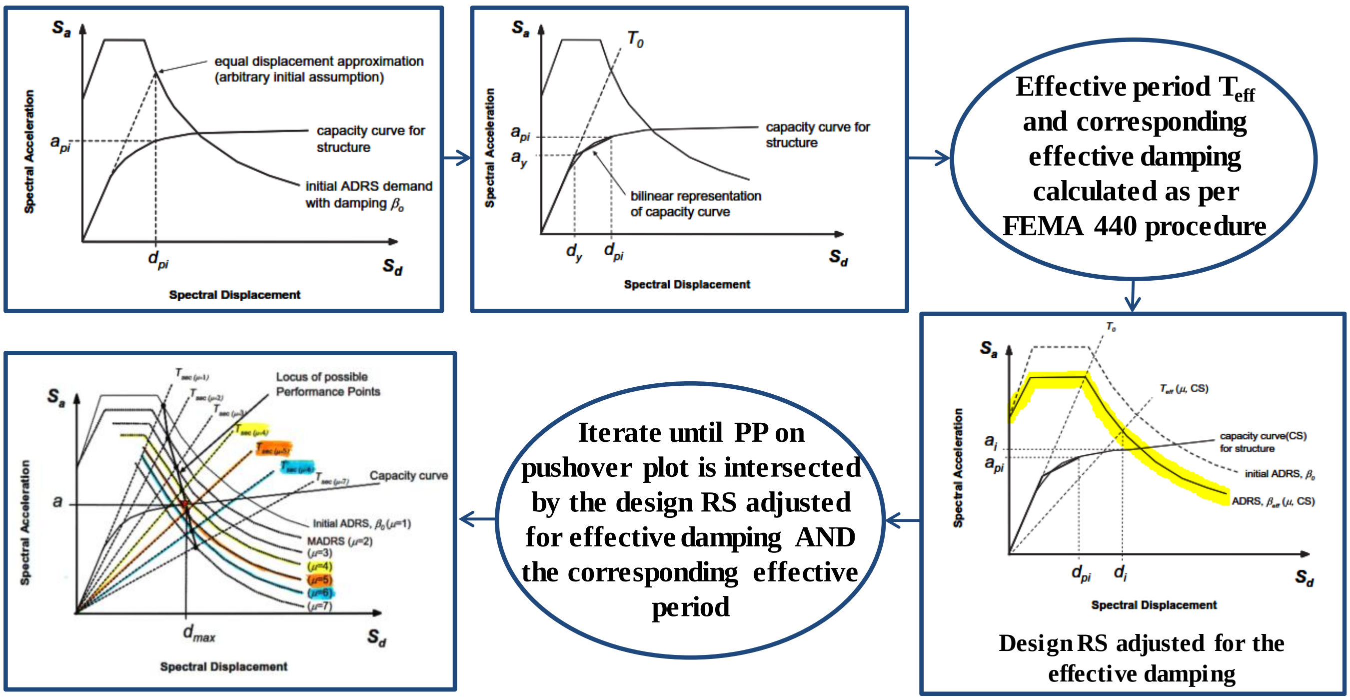

Figure 6. Equivalent linearization method of FEMA 440.

The performance parameters of the BRBs such as ductility demands, hinge formation along the height of the structure and rotation demands of the steel moment frame hinges were reviewed at the performance point obtained from the 3-dimensional pushover study using the equivalent linearization method. The performance point and hinge formation along the height of the structure output from the SAP analysis are shown in Figure 7. The properties of the BRBs were further refined and finalized during this phase of the modeling until a system was achieved where the ductility demands on the BRBs remained within the design limit and a uniform hinge distribution within the inelastic rotation tolerances along the height of the structure was obtained. As a final check of the analyses, the optimized BRB properties from the pushover analyses were put back into the simplified model of the structure shown in Figure 4 and the full set of ground motions were reanalyzed in order to verify that the drift and displacement profiles remained within the code specified limits.

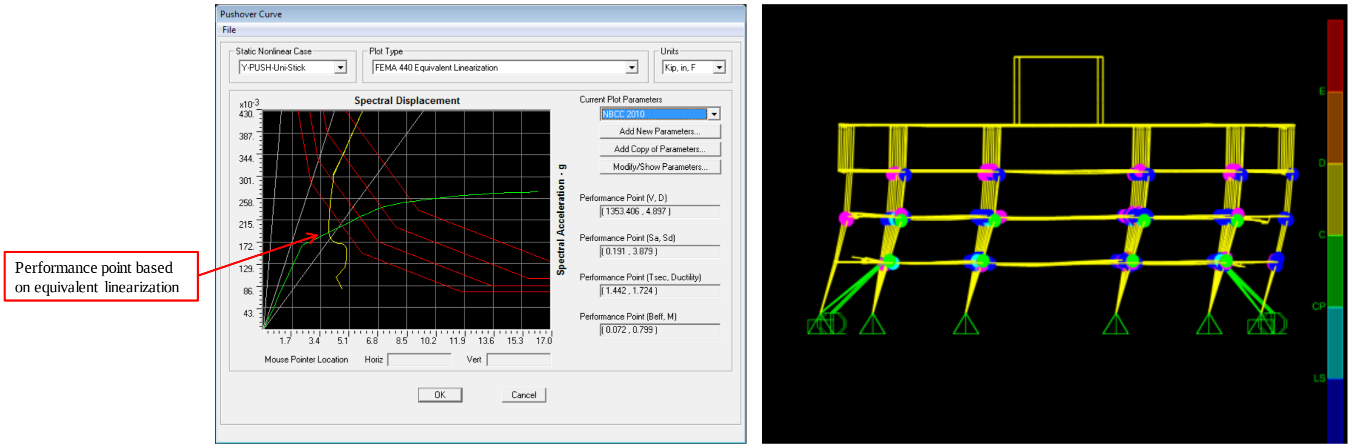

Figure 7. Equivalent linearization method in SAP2000 and hinge formation throughout the structure at the performance point.

The flowchart for the analyses utilized in this seismic upgrade project is summarized in Figure 8. The approach allowed for the design and optimization of the upgrade solution to be performed in an efficient and timely manner by combing the benefits and strength of non-linear pushover analyses with a simplified representation of the structure and the algorithm efficiency of FNA approach for performing non-linear time-history analyses. The efficient analyses algorithm enabled investigation of many iterations of the model in order to effectively eliminate the soft-storey mechanism in the ground level and utilize the capacity of the existing steel moment frame structure by controlling the drifts and hinge rotation demands.

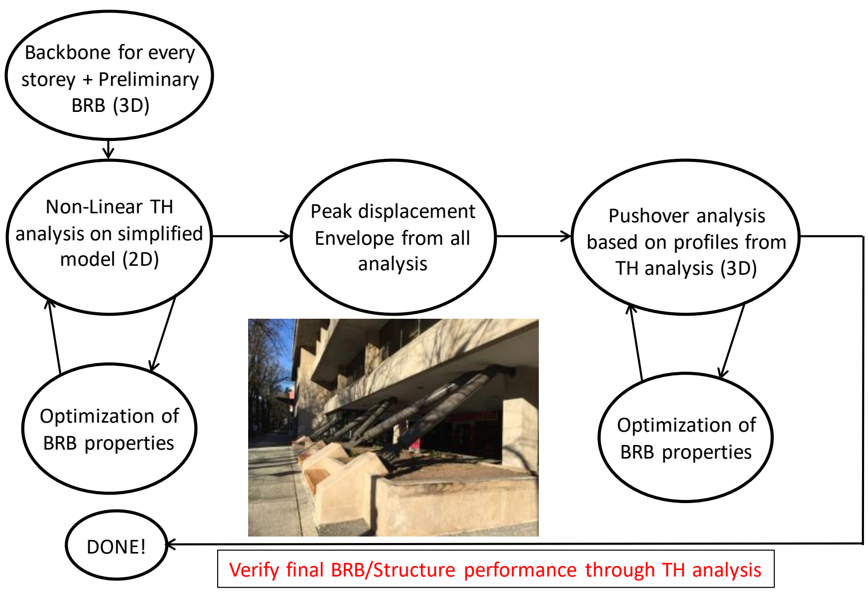

Figure 8. Analyses flowchart in the design and optimization.

SEISMIC UPGRADE DETAILS

Two sizes of BRBs were designed and optimized for each orthogonal direction of the building. The inclined and the V-configured BRBs as shown in Figure 1 are decoupled and designed independently for loading in each orthogonal direction of the building. In order to ensure the response of the two sets of BRBs will be decoupled, a slotted connection detail was used at the base of V-shaped set as shown in Figure 9. In addition to the design and implementation of the BRBs, other critical components along the seismic load path were upgraded. These components included six major steel 'drag struts' in the ceiling of the first floor to transfer lateral loads to the new BRBs, FRP diaphragm and 'drag struts' upgrades on the ceiling of the two parking levels in the basement and new concrete shear walls in the two basement levels for transferring the seismic demands from the BRB's, first floor and parking levels to the shear walls and subsequently to the foundation.

Load transfer in the basement includes a unique load path of FRP strips on the underside of the main floor connected to steel brackets, to Dywidag rods (installed through cores through the existing precast columns and beams) ultimately anchored in the shear walls as shown in Figure 9. Furthermore, the poorly connected precast planks of the main floor and first parking level were 'stitched together' with bidirectional FRP to create a diaphragm of adequate strength. Additionally, the FRP drag strut to steel brackets incorporated a Canada-first FRP to steel connection in order to provide continuity of the load path from the drag struts through the added concrete shear walls.

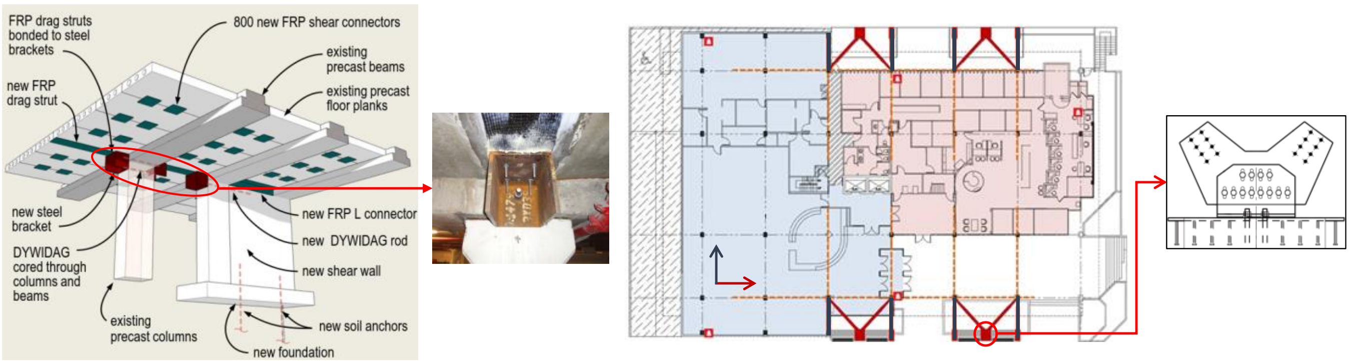

Figure 9. FRP drag strut to steel bracket connection to provide continuity of load path through columns and into the shear walls(left side of figure); layout of drag struts (red dotted lines) relative to BRBs and slotted connection at base of V-configured BRBs.

CONCLUSIONS

The fundamentals of the seismic upgrade solution were as follows: installation of BRBs at the ground floor level with very specific strength, stiffness and energy dissipation to eliminate the soft-storey at the ground floor level, and spread the energy dissipation over the height of the structure within critical inelastic rotation tolerances determined for each existing steel moment frame member. By taking a performance-based approach and utilizing the capacity of the existing moment frame in a controlled manner in the design of the seismic upgrade, the implemented solution was substantially less intrusive and far more cost effective than the conventional upgrade scheme originally conceived for this building while achieving the desired performance limits for 100% of the code seismic demands (as oppose to 75% in the original scheme). The non-linear analyses carried out were unique and followed all-new provisions in NBC 2015 for 'supplemental energy dissipating' structures requiring design to use the 'mean plus one standard deviation' results. The combination of time-history analyses on a simplified 2-dimensional model and pushover analyses on a detailed 3-dimensional model allowed for an efficient iterative method for the tuning of the BRBs and drift control for the structure.

The BRB system will reduce seismic demand by some 30% in the upper floors, engage some 600 beam or column locations to yield in the existing structure, and control the building drift to 1% per storey for the upper floors, and to less than 2% in the first storey (less than the 2.5% allowed by code for new buildings).

REFERENCES

[1] National Building Code of Canada - NBC 2015, Volume 1.*Part 4: Structural Design.* Prepared by the National Research Council of Canada, Ottawa, Canada.

[2] FEMA 440 (2005). *Improvement of Nonlinear Static Seismic Analysis Procedures.* Prepared by the Applied Technology Council (ATC-55 Project), Redwood City, CA, USA.