NONLINEAR SEISMIC PERFORMANCE OF THREE CASE STUDY BUILDINGS AND THE ADOPTED RETROFITTING STRATEGIES

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Abstract

Limitations associated with the force-based seismic design approach have led to extensive research to develop alternative design philosophies, to reduce the seismic design uncertainties and construction costs. Performance based design is recognized as a reliable method for realistic seismic assessment of structural systems. Performance based design requires nonlinear time-history analysis which includes the nonlinear response of structural elements under cyclic loads. While nonlinear time history analysis is more time consuming and demands more technical expertise, it provides a more transparent assessment of the seismic response of the structures and leads to a more economical design.

With increasing efficiency of analysis tools and computational power, performance based seismic design can be incorporated as an efficient tool in practice for the seismic design and retrofit of structures. This paper reviews the limitations of the force-based methods of analysis that are common in current practice and the advantages of nonlinear time history analysis. The proposed guidelines for performing a nonlinear time history analysis as well as the guidelines for the selection and scaling of ground motion records are reviewed. Finally, the nonlinear response of 3 case study buildings as well as the adopted retrofitting strategies, using the reference design codes such as the National Building Code of Canada (NBCC 2010) and the American Society of Civil Engineers (ASCE 7-10) are presented and discussed. Recommendations for the use of nonlinear time history analysis, as a design tool, are made to assist practitioners in the application of these methods.

Keywords: Performance based design; Nonlinear Time-History Analysis; Case Studies; Force Based Design;

1. Introduction

Current design standards in North America such as NBCC 2015 [1] and ASCE 7-10 [2] provide seismic design guidelines based on a force-based seismic design philosophy, such as the equivalent static force procedure (ESFP). These guidelines ensure that structural collapse is prevented and have been widely practiced by structural engineers. However, they are accompanied by several limitations and fundamental flaws. These limitations include the estimation of the fundamental period of vibration in the linear range, the change in period as the structure starts to respond nonlinearly, the multi-directionality aspect of earthquakes, the equal displacement rule, uncertainty in determining the force modification factors and failing to provide any means for measuring response quantities such as residual deformations, acceleration time histories, velocity time histories, etc.

Linear modal response spectrum analysis and linear time history analysis provide improvements on some of the limitations of the ESFP such as the estimation of the elastic period and the contribution of higher modes. In addition, linear time history analysis accounts for the multi-directionality aspect of earthquakes and provides response quantities as functions of time. These improvements ultimately refine the seismic load spatial distribution and its distribution in time. Modal response spectrum can be used to refine the seismic force distributions by including the contribution of higher modes. In regular structures, at times, higher modes will act to decrease the seismic demands. Linear time history analysis, other than providing this refinement, will account for the sequence of response quantities during design seismic events, instead of simply assuming that the maximum response quantities occur concurrently. For instance, in a beam-column element, the maximum axial force will not always be accompanied by the maximum bending moment. Linear time history analysis accounts for this behavior and therefore makes the design much more economical.

Despite these improvements, linear methods of analysis still evaluate structural systems based on their dynamic characteristics in the elastic range and will not overcome many of the limitations associated with the forced-based design philosophy discussed above. In addition, the response quantities obtained from linear dynamic analysis methods must eventually be scaled up to the earthquake response obtained from the code-based equivalent static force procedure (ESFP). Therefore, these methods will mainly help to refine the distribution of earthquake induced forces in the building based on direction and sequence in time, without changing the design approach. This paper studies the benefits of incorporating nonlinear time history analysis for design and retrofit of structural systems in practice. The limitations of linear methods of analysis and the need for incorporating performance based design are reviewed and discussed. The guidelines for doing a nonlinear time-history analysis by the codes of practice are reviewed. In addition, the nonlinear seismic response of three case studies and the retrofitting strategies are discussed. The buildings are retrofitted to meet the requirements of the NBCC 2010 [3].

2. Limitations Associated with the Force-Based Seismic Design Approach

Linear dynamic methods of analysis, although improvements to the code-based seismic design, do not account for many of the shortcomings associated with force-based seismic design philosophy. Some of these limitations include:

2.1 Elastic period of vibration

The code-based equivalent static force procedure is based on an estimation of the elastic fundamental period of the structure. Modal response spectrum and linear time history analysis can capture the accurate elastic periods of the structure in all modes of vibration. However, the elastic period is not a proper representation of the structural behavior since almost the entire response of most structures, under lateral loads, is through inelastic actions [4]. Yielding of steel elements and cracking of reinforced concrete members lengthen the structure's periods and change its mode shapes and dynamic characteristics. Each of these can significantly alter the response of the structure. Performing a nonlinear time history analysis will adjust the buildings dynamic characteristics at any point in time during the response.

2.2 Force modification factors and ductility

In force based design approach, the elastic base shear is obtained by the ESFP or methods of linear dynamic analysis. Afterwards the elastic base shear is divided by a factor, meant to account for the over-strength and inherent ductility capacity in the structural system. The determination of these factors is based on judgement with very limited knowledge of the structure [4]. Design guidelines such as ASCE 7-10 [2] and NBCC 2015 [1], provide recommendations for force modification factors, solely based on the type of the lateral load resisting system. Important structural characteristics such as height, natural period and other dynamic characteristics are not used for this determination. In addition, the equal displacement rule which assumes that the deformations under the elastic base shear are equal to the nonlinear deformations is not valid for short period and long period structures [5]. In nonlinear time history analysis, this assumption is no longer required since the nonlinear response of the system is measured accurately.

2.3 Limited response quantities

Current force based design methods do not provide response quantities including acceleration time histories, velocity time histories and residual deformations. Acceleration time histories can be useful for the assessment of uninterrupted service of sensitive equipment. Residual deformations are the deformations observed at the end of the structures response to earthquake excitations and are caused by the inelastic response of the elements of the lateral load resting system. McCormick et al. [6] have reported that residual drifts of 0.5% are perceivable by the occupants and when close to 1.0%, the occupants will experience dizziness. The study by Chio et al. [7] showed that buildings with buckling restrained braces as the lateral load resisting system, when designed to ASCE 7-05 will experience more than 0.5% residual deformations at the end of their response for 79% of earthquakes matching the design hazard. In addition, 60% of earthquakes caused the special moment resisting frames to exhibit residual deformations higher than 0.5% at the end of their seismic response. Given these results and that for residual drifts higher than 0.5% the structure can be considered a total loss, the consideration of residual deformations is of great importance. When using the force-based design approach, the designer would have no knowledge of the state of the structure at the end of its seismic response. By adopting a performance based design approach, the designer will be able to measure residual displacements and limit them to acceptable levels.

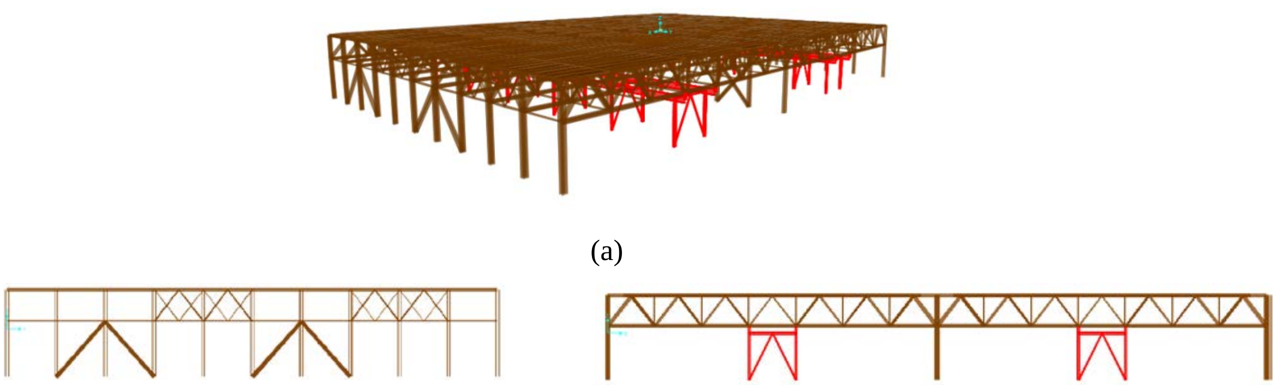
The shortcomings of the force-based design philosophy clearly indicate the advantages and the increasing need for performance based design in practice. In the current paper, the retrofit of three case study buildings, carried out by performing nonlinear time history analysis are presented and discussed.

3. Seismic Retrofit of Case Study Building I

3.1 Background

Building I is an industrial one-story building. The building is 9.2 meters in height and 48.8m by 68.3m in plan and is located in eastern Canada. The structure was originally constructed in the 1940s and had undergone structural upgrades in 1985. The existing conditions were assessed through studying the existing structural drawings and several site visits.

The existing structure consists of wood framing. The primary lateral load resisting system consists of diagonal wood braces and concentric chevron steel braces in the North-South and East-West directions, respectively. The roofing system consists of light wood panels and wood trusses in both orthogonal directions of the building. The roof wood trusses also act as a secondary LLRS. Fig. 1 shows an illustration of the structural system, taken from CSI-SAP 2000.



(a)

(b) (c)

Fig. 1 – Existing Building; (a) 3D illustration from SAP 2000, (b) section along the N-S direction, and (c) section along the E-W direction

3.2 Practical Challenges

The purpose of the current seismic assessment is to ensure that the structure will exhibit satisfactory seismic performance. The evaluation indicated that replacing the wood braces with steel braces and incorporating new braces in the other direction can ensure satisfactory seismic performance. The following challenges are faced in the execution of this retrofitting strategy:

1. The existing foundation system, particularly the foundation system below the existing wood braces does not have adequate uplift resistance. This is expected as the existing foundation system was built to accommodate uplift forces associated with the capacity of wood braces. The results of the analysis indicate that in order for the building not to experience uplift, the lateral load in each braced bay must not exceed 100.0 kN. The new braces are designed so that the lateral load associated with the buckling of the braces,

in compression, and the yielding of the braces, in tension, do not exceed 100.0 kN. In other words, the braces are designed to yield and buckle prior to foundation uplift, limiting the lateral load.

2. The operation of the building dictates that structural interventions in the building to be minimal. While many locations could accommodate adding braces from a structural point of view, the occupancy of the building limits the number of additional brace locations.

3.3 Code-based equivalent static analysis

As the preliminary analysis, an equivalent static analysis, in accordance with NBCC 2010 [3] is carried out to determine the seismic lateral demands on the structure. The building is on site class D. The short period spectral acceleration for the site, S_a (0.2), is 0.61g. With the importance factor, I_e, taken as 1.0, and R_dR_o assumed as 1.95, the static base shear is determined to be 1295.0 kN, in both principal directions. As mentioned earlier, the maximum lateral load in each braced bay, to avoid uplift, must be 100.0 kN. This means that at least 13 new braces must be added in each direction to avoid foundation uplift, if one ignores the contribution of wood trusses to the lateral capacity. A modal response spectrum analysis was performed. The base shear was reduced to 80% of that of the ESFP, and the number of braces was decreased to 10. However, the cost of adding 10 braces and location of braces is not feasible due to building operation. There are two possible solutions: (1) design and construction of new foundations locally, or (2) conducting a more advanced analysis and removing some of the uncertainties associated with the code-based seismic design approach.

3.4 Selection of ground motions

Both linear and nonlinear time-history analysis require selection and scaling of ground motion records to match the target design uniform hazard spectrum (UHS). NBCC 2010 commentary [8] adopts the provisions provided by ASCE 7-10 [2]. According to ASCE 7-10 [2], if seven seismic events are used in the time history analysis, mean response quantities can be used in design. In regions without a database of ground motion records, simulated time history records have been developed. Atkinson (2009) [9] has developed simulated ground motion records which match the UHS in eastern and western Canada. Seven ground motions were selected from this database available at www.seismotoolbox.ca which are representative of the seismological characteristics of the region.

The selected records are scaled so that the SRSS PSA of each individual pair of ground motions (seismic event) would match the target UHS over the period range of interest, with the mean PSA of all seismic events equal to or exceeding the target UHS throughout the period range. The period range is selected as per the recommendations provided by ASCE 7-10 [2]. The building fundamental period is 1.25 seconds. Therefore, the period range of interest is 0.25s to 1.88s. Fig. 2 illustrates the PSA of the scaled records versus the target UHS. The shaded area shows the period range of interest. Filiatrault et al. [10] provide descriptions and examples of the scaling method used.

3.5 Time history analyses

To remove some of the uncertainties associated with the code-based approach, first a linear time history analysis, using the records shown in Fig. 2, is carried out. Afterwards, using the same records a nonlinear time history analysis is carried out.

The advantages of doing a linear time history analysis include accurately determining the structures elastic period, accounting for the contribution of higher modes, and removing the conservative assumption that the direction of the maximum component of the ground motion coincides with the principal axes of the building. While performing a modal analysis will offer the first two advantages, it does not account for the multi-directionality of ground motions. Most codes and design guidelines introduce a lower limit on the base shear obtained from a linear time history analysis, similar to that imposed on the base shear obtained from a modal response spectrum analysis. This lower limit is given as $0.8V_{st}$ in NBCC 2010 [3], where V_{st} is the base shear obtained from an equivalent static analysis.

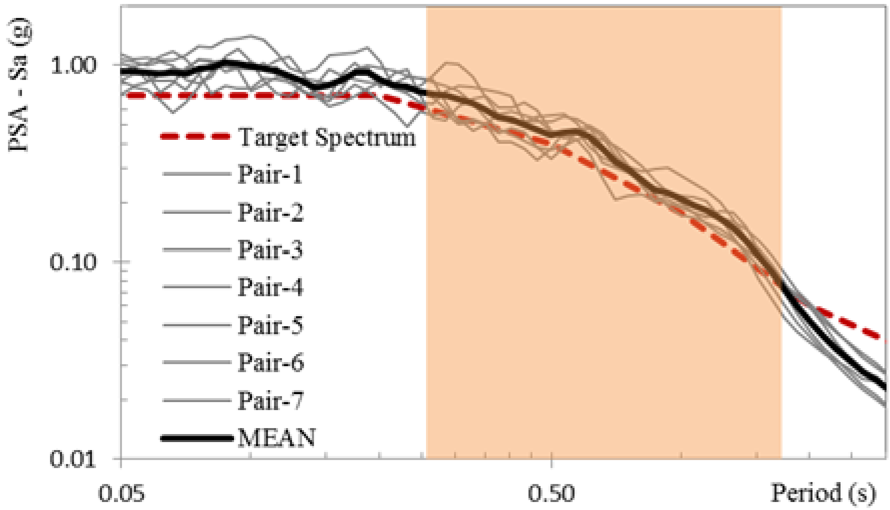


Fig. 2 – SRSS of the pseudo acceleration response spectrum (PSA) of the scaled records versus the target uniform hazard spectrum (UHS) for Building I

Nonlinear time history analysis accurately accounts for changes in buildings dynamic characteristics throughout the response of the structure and removes the uncertainties associated with equal displacement rule. Both NBCC 2010 [3] and ASCE 7-10 [2] do not impose a lower limit on the response quantities obtained from a nonlinear time history analysis. In this structure, the nonlinearity is confined to the steel braces. All other structural elements, including the wood trusses, are capacity protected. The size of the braces is chosen such that they would buckle/yield prior to foundation uplift. A Clough hysteretic response is assigned to the braces so that their response would display the pinching behavior associated with the response of concentric braces as shown by Tremblay et al. (2003) [11]. Throughout the analyses, direct time-step integration of the equations of motion is carried out and a damping ratio of 5% is assumed for the first and the second modes.

3.6 Results of the analyses

The results of the analyses are presented in Table 1. The retrofit of the structure was optimized at every stage of the analysis and the number of braces was reduced. By performing a linear time history analysis, the base shear was reduced to lower than 80% of the static base shear. Due to the code limit, the design base shear is 80% of the equivalent static base shear as per NBCC 2010 [3]. The number of braces in the X direction was reduced to 10, where the braces in the Y direction were reduced to 8. In the final nonlinear time-history analysis, the base shear was reduced to 67% of the equivalent static base shear. It is further of interest to note that the drop in the base shear in each principal direction was higher due to the simultaneous action of the earthquake. In one principal direction alone, the base shear was reduced to 57% of the equivalent static base shear. The number of braces was reduced to 6 braces in each direction. The lateral displacements in the X and Y directions were 50% and 30% of the corresponding values from the modal response spectrum analysis and were within the post-disaster limits. By performing a nonlinear time history analysis, the roof wood trusses and all the other structural members are capacity protected, foundation uplift was avoided, the residual displacements were evaluated and other response quantities such as acceleration and velocity time histories are available. More importantly, the required number of braced bays and the structure’s down time during construction is significantly reduced saving significant construction costs.

The base shears obtained from the nonlinear time history analysis are smaller than the corresponding base shears from the linear time history analysis. The primary reason for this is that performing a nonlinear time history analysis accounts for the softening of the structure. In addition, the design is optimized and the number of braces is reduced, and the structure is more flexible in the elastic region. In the current analysis, it is observed that under the design earthquake, the braces in compression will buckle but the braces in tension do not reach their tensile capacity. The buildings hysteretic response under the first six seismic events, in the X direction, is illustrated in Fig. 3. The vertical axis, *V*, shows the base shear in KN where the horizontal axis, *Δ*, shows the lateral displacement in millimeters.

Table 1 – Results of the time-history analyses in terms of seismic base shears for Building I

Case	**Linear Time-History Analysis**			**Nonlinear Time-History Analysis**		
	X (kN)	**Y (kN)**	**SRSS (kN)***	**X (kN)**	**Y (kN)**	**SRSS (kN)****
Event 1	1012.4	819.2	1036.0	618.8	726.9	821.7
Event 2	978.8	493.1	1036.0	777.6	656.2	934.9
Event 3	1031.9	585.9	1036.0	868.3	495.6	880.5
Event 4	979.1	434.6	1036.0	731.7	533.0	749.5
Event 5	1014.7	706.8	1036.0	810.2	536.2	811.5
Event 6	1035.7	561.5	1036.0	912.7	580.9	913.5
Event 7	768.4	999.5	1036.0	477.9	863.7	928.0
Mean	974.5	657.2	1036.0	742.5	627.5	862.8

* *Base shears scaled to 80% of the ESFP*
** *No scaling of base shear is required*

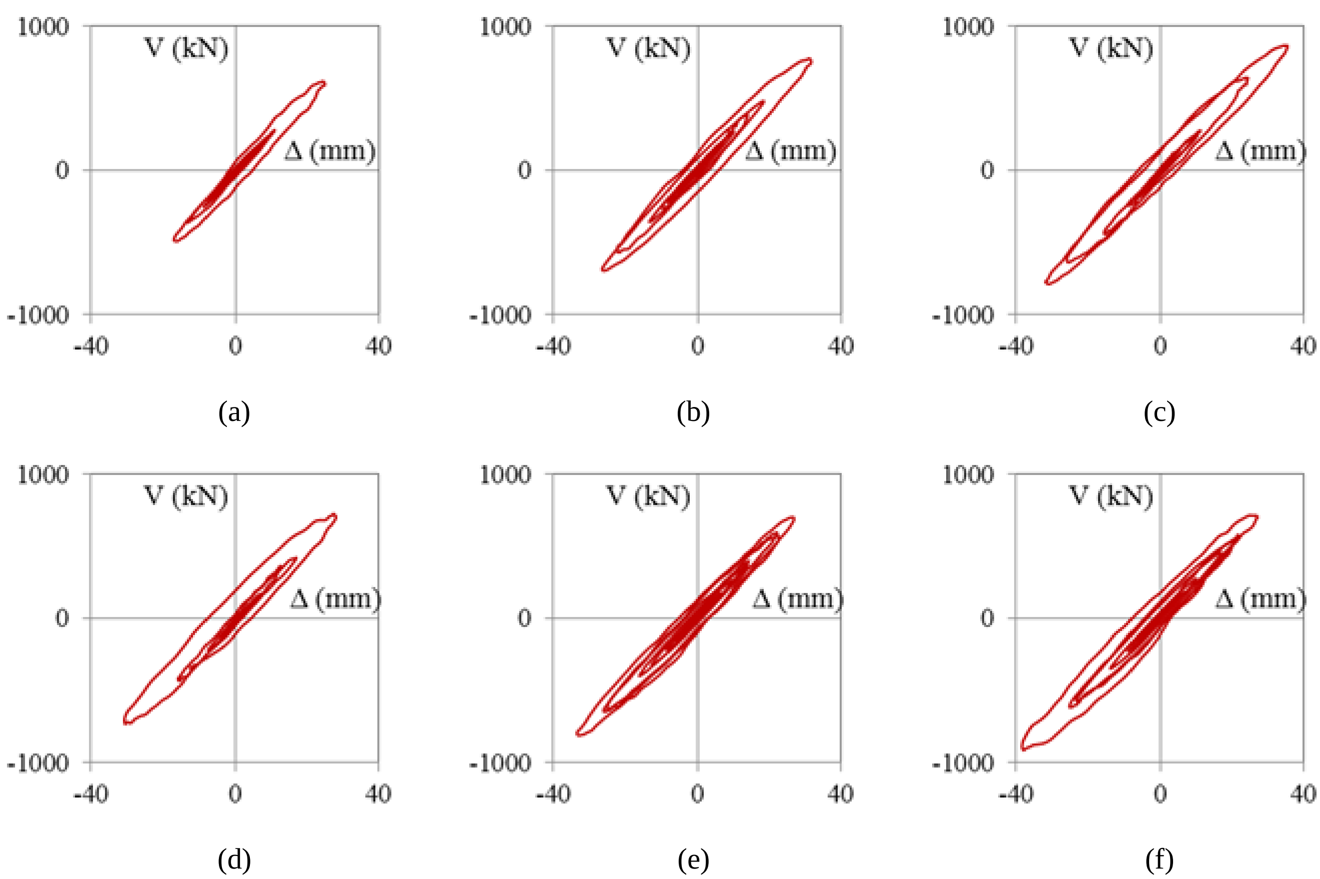
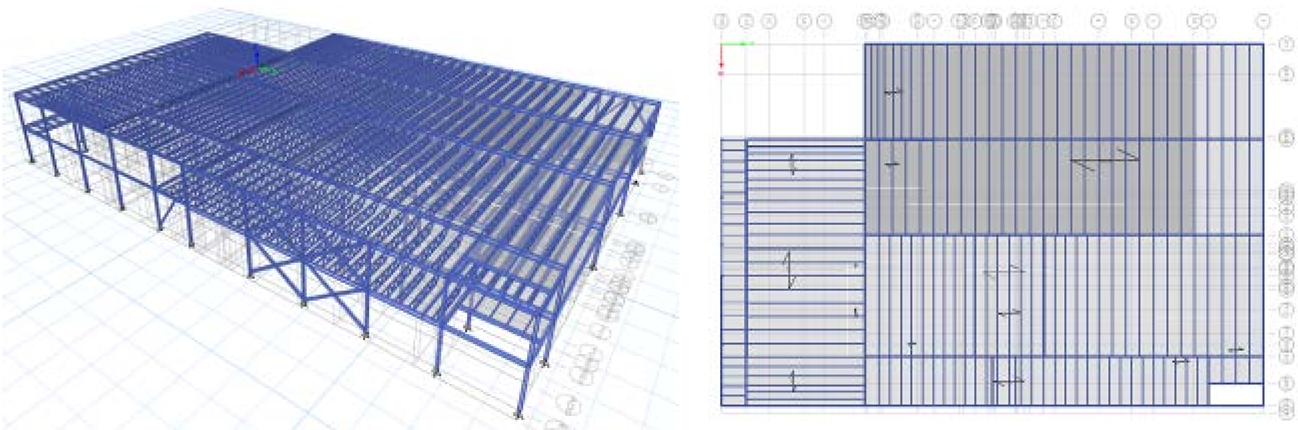


Fig. 3 – Hysteretic response of building I in the X direction subjected to six of the seismic events: (a) event 1, (b) event 2, (c) event 3, (d) event 4, (e) event 5, (f) event 6

4. Seismic Retrofit of Case Study Building II

4.1 Background

Building II is a two story steel building, constructed in the 1990s. The roofing system is composite steel decks with concrete topping supported by open web steel joists. The primary lateral load resisting system in both orthogonal directions consists of concentric steel chevron braces. In addition, the building has a long reinforced concrete shear wall along its west side on the first floor. The height of the structure is 8.5 meters. The building has a total foot print of 2222 m^2 and is located on site class 'D'. The PGA and the short period spectral acceleration for the site are 0.32g and 0.64g. Fig. 4 shows an illustration of the structural system, taken from CSI-ETABS 2015.



(a) (b)

Fig. 4 – Existing Building; (a) 3D illustration from ETABS 2015, (b) plan of the top storey

4.2 Practical Challenges

The purpose of this analysis is to evaluate the structural seismic performance and determine whether any retrofitting action is required. In addition, the extent of the structural intervention, if necessary, must be determined.

The presence of the reinforced concrete shear wall along one of the edges of the building in the first story makes the building irregular in both plan and height. Given that the structure is sensitive to torsion along with the irregularity in height, performing a dynamic analysis is required as per the requirements of the NBCC 2010 [3]. Other than making the system sensitive to torsion and irregular in height, the shear wall increases the building stiffness, seismic mass and, therefore, the seismic base shear.

The shear walls are not gravity load bearing members and are not continuous in height. These two important conditions of the reinforced concrete shear walls make it possible to structurally separate them from the system to reduce the mass, stiffness and seismic forces. The nonlinear behavior of the walls is not easily determined. The walls have small aspect ratio and are not likely to show notable ductility. Therefore, in the first set of analyses the structure's performance was evaluated in the linear elastic range. After structurally separating the shear walls from the structure, nonlinear time history analyses were performed.

4.3 Retrofitting Strategy

As the preliminary analysis, the equivalent static force procedure, as outlined by NBCC 2010 [3] was carried out. The calculated β factor which is a measure of building's sensitivity to torsion was higher than 1.7, indicating that the building is sensitive to torsion. Since a dynamic analysis is required, a modal response spectrum analysis was carried out. The results of the analysis indicated that the braces do not have adequate capacity to provide satisfactory performance for the design earthquake. The first retrofit task was to structurally separate the existing shear walls. As a result of the decreased mass and stiffness, the force on the existing braces is reduced. The second task was to increase the size of the existing braces and use a performance based approach for the final design.

4.4 Selection of ground motions

For ground motion selection and scaling, the provisions provided by ASCE 7-10 [2] and NBCC 2010 Commentary [8] are adopted. Seven pairs of ground motion, compatible with the seismological characteristics of the site, were selected from the database provided by Atkinson [9]. Records pairs E6D1-9 and E6D1-10, E6D1-11 and E6D1-12, E6D1-17 and E6D1-23, E6D1-25 and E6D1-26, E6D1-31 and E6D1-32, E6D1-33 and E6D1-34, and, E6D1-43 and E6D1-44, with scale factors of 0.76, 0.79, 0.72, 0.72, 0.74, 0.85 and 1.00, respectively, are selected for the analysis. The structure fundamental period is 0.49 seconds. Therefore, the period range of interest is 0.10 to 0.73 seconds. Other assumptions for the selection and scaling of ground motion records are the same as those described in section 3.4.

4.5 Time history analyses

Performing a linear or a nonlinear time history analysis offers the advantages described in section 3.5. In this building, the design was optimized through a number of iterative modal response spectrum analyses and linear time history analyses. For the final design, the structure is subjected to the seven selected and scaled ground motions and the performance of the structure is evaluated by performing nonlinear time history analyses. A Clough hysteretic model was assigned to the braces so that their response would display the pinching behavior associated with the response of concentric braces as shown by Tremblay et al. (2003) [12]. Throughout the analyses, direct time-step integration of the equations of motion is carried out and a damping ratio of 5% is assumed for the first and the second modes.

4.6 Results of the analyses

The base shears obtained from the nonlinear time history analyses are presented in Table 2. In the final design, the nonlinearity is confined to steel braces and all the other structural members including columns and diaphragms are capacity protected. Foundation uplift is controlled and avoided. By performing nonlinear time history analysis, the uncertainties associated with the ESFP and modal response spectrum analysis are reduced. The design base shear is reduced to 86% of the corresponding value obtained from an ESFP. The lateral displacements in the X and Y directions obtained from the nonlinear time history analyses are 56% and 70% of the corresponding values from the ESFP and are within the limits defined for post-disaster structures. Reducing the number of required braced bays, other than reducing the costs, reduces the downtime of the structure during retrofit. The residual displacements are evaluated and controlled by adjusting the layout of the braces. Fig. 5 shows the displacement time histories at both stories, under the 5th seismic event. The building hysteretic response in the X direction, to 6 of the selected seismic events, is shown in Fig. 6.

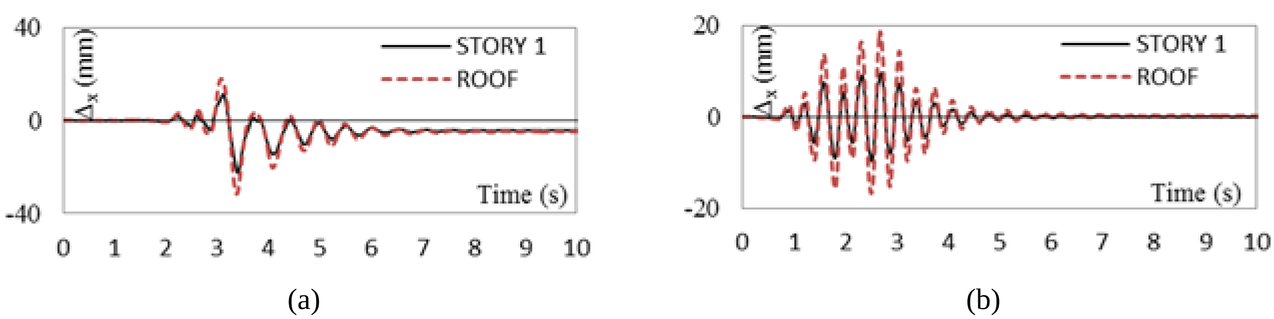


Fig. 5 – Buildings displacement time histories under seismic event 5, (a) displacement in the X direction, (b) displacement in the Y direction

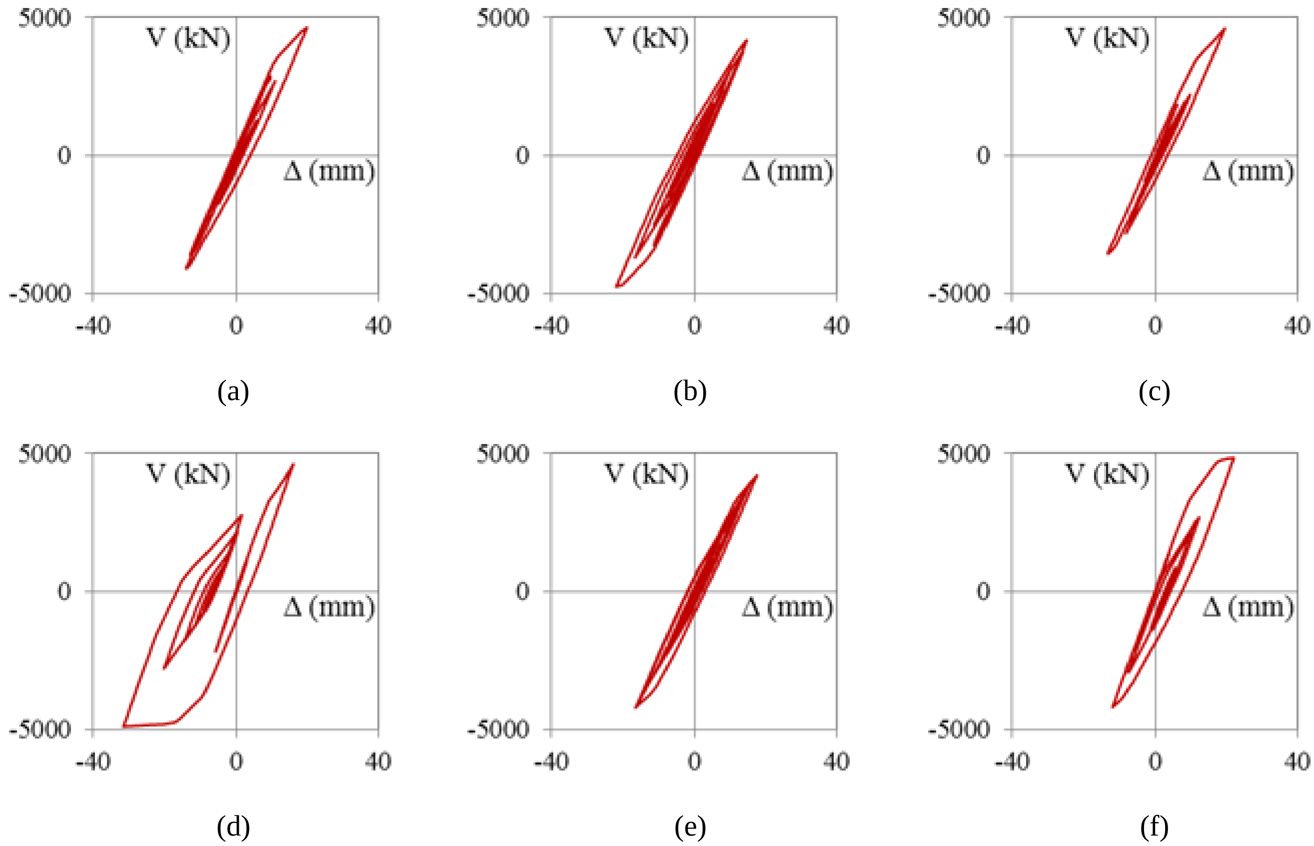


Fig. 6 – Hysteretic response of building II in the X direction subjected to six of the seismic events: (a) event 1, (b) event 2, (c) event 3, (d) event 4, (e) event 5, (f) event 6

Table 2 – Results of the nonlinear time-history analyses in terms of seismic base shears for Building II

Case	Event 1	Event 2	Event 3	Event 4	Event 5	Event 6	Event 7	Mean
X (kN)	4643.8	4845.9	4746.7	4609.9	4890.1	4219.6	4008.3	4566.3
Y (kN)	5533.4	7667.6	6863.7	6175.5	6668.9	8632.0	6811.9	6907.6
SRSS (kN)	6177.1	9069.9	6875.3	6363.4	7557.6	8665.8	6882.1	7370.2

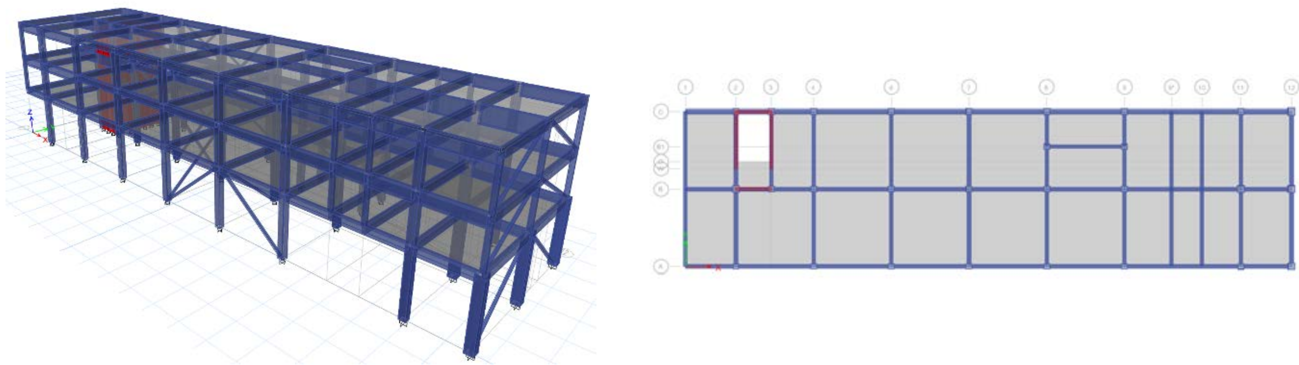
5. Seismic Retrofit of Case Study Building III

5.1 Background

Building III is a three-story reinforced concrete structure. Gravity loads are resisted by concrete two-way slabs spanning between concrete moment resisting frames. The primary lateral load resisting system is formed by friction dampers. The shear walls around the egress route act as a secondary lateral load resisting system. Concrete moment resisting frames provide a reserved lateral capacity for the structure. The building is 680 m^2 in plan and 11.5 meters in height. The PGA and short period spectral acceleration for the site are 0.5g and 0.92g, respectively. Fig. 7 shows an illustration of the structure, taken from CSI-ETABS 2015.

5.2 Practical Challenges

The friction dampers are not distributed evenly throughout the height of the structure and shears walls around the egress route shift the center of rigidity to the east of the structure, making the building sensitive to torsion. Therefore, for the existing conditions, the seismic load is distributed unevenly both in plan and height. The addition of new braces or friction dampers can adjust the location of the center of rigidity to make the building less sensitive to torsion. However, operation requirements limit the number of new braces which can be added. The location of the shear walls around the egress route, requires that cracking and inelastic behavior are limited to provide a safe exit path for the buildings occupants.



(a) (b)

Fig. 7 – Existing Building; (a) 3D illustration, (b) typical floor plan

5.3 Retrofitting Strategy

The primary evaluation of the seismic performance of the building indicated that the existing shear walls should be separated from the structure to provide adequate performance. The result of separating the shear walls is the distribution of the seismic base shear in plan more efficiently and engaging the friction dampers in the seismic response of the system more effectively. In addition, the structure is more flexible and the elastic seismic base shear drops. Adding new braces optimizes the response and minimizes the irregularity in height.

5.4 Selection of ground motions

The ground motion selection process and criteria are similar to those described for previous case studies. The ground motions used for this case study are historical records obtained from the PEER ground motion database [12].

5.5 Time history analyses

Given the irregularity in height, a dynamic analysis is required as per the provisions of the NBCC 2010 [3]. The analysis/design procedure, similar to that described for building II, is an iterative process of doing modal response spectrum analyses and linear time history analyses and optimizing the design in each stage. In the final optimization, nonlinear time history analysis is performed. An elastoplastic hysteretic response is assigned to the friction dampers. Damping ratio of 5% is assigned to the first and the second mode and direct time step integration of the equations of motions is carried out.

5.6 Results of the analyses

The benefits of performing a nonlinear time history analysis are similar to those described for case study buildings I and II. The advantages include having a more economical design, reduction in the seismic loads by removing the uncertainties, optimizing the design by improved knowledge of the structure and evaluation of the residual deformations. In the final retrofitted condition, the design SRSS base shear is 80% of that obtained from the modal response spectrum analysis. Base shear components in the X and Y directions are 70% of their corresponding base shears from the ESFP. The lateral displacements in the X and Y directions obtained from the nonlinear time history analyses were 57% and 41% of the corresponding values from the modal response spectrum analysis and are within the limits specified for post-disaster structures. The slip values in the friction dampers are evaluated and lie within the acceptable range. Fig. 8 shows the hysteretic response of the structure to three of the seven design seismic events. The post-yield stiffness in the hysteretic response shows the lateral stiffness of the concrete moment resisting frames.

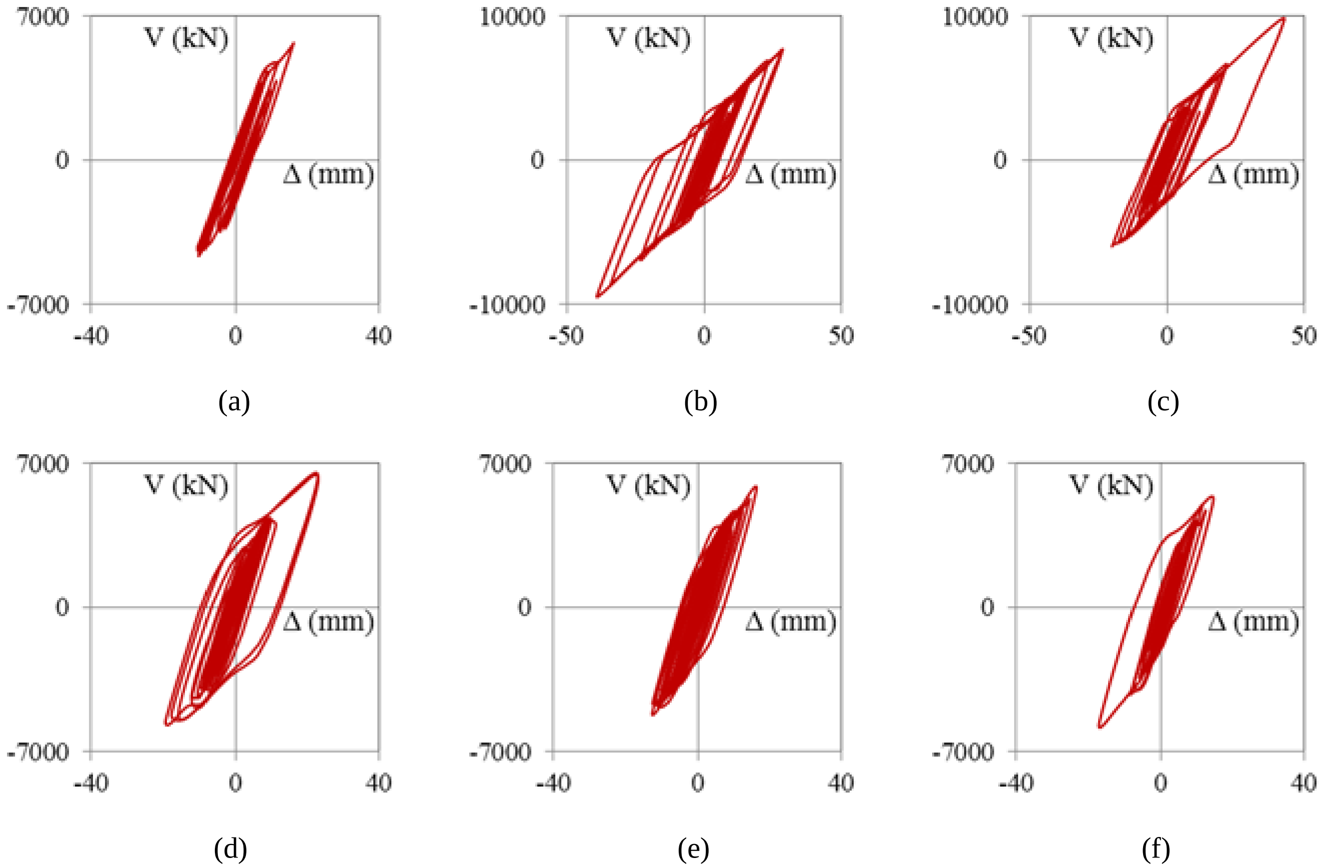


Fig. 8 – Hysteretic response of building III in orthogonal directions (a) displacement in the X direction for event 1, (b) displacement in the X direction for event 2, (c) displacement in the X direction for event 3, d) displacement in the X direction for event 1, (e) displacement in the X direction for event 2, f) displacement in the X direction for event 3

6. Conclusions

The study presented in this paper evaluated the seismic performance of three structural system by using performance based design and nonlinear time history analysis as the main method of analysis. The following conclusions are drawn from the current study.

1. Force-based seismic design philosophy has several limitations including the use of the fundamental elastic period to determine the seismic base shear, uncertainty in the force modification factors and adopting the equal displacement rule.

2. Performance based design can be used as a reliable method to reduce the limitations associated with the force-based design approach. Performing nonlinear time history analysis leads to a better understanding of the seismic response of the structure and provides the designer with means to measure response quantities that are not determined using the force-based seismic design procedure such as acceleration, velocity time histories and residual displacements.

3. The use of performance based design for regular structures, although not required, is strongly recommended. Adopting performance based design as the method of seismic design leads to more

economical design. Further, the designer will be able to better choose the performance level and the hazard.

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