Intensity-based performance assessment of middle-rise steel office buildings

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ABSTRACT

This paper presents the assessment of earthquake-induced economic losses of middle-rise steel braced frame buildings located in urban Eastern Canada. The prototype office building is braced in both orthogonal directions by moderately ductile concentrically braced frames (CBF). According to FEMA P-58 methodology, the building probable performance assessment method is intensity-based conducted for a user-selected location knowing the elastic acceleration response spectrum. To evaluate the seismic response, firstly, three intensity hazard levels representing the design-level (2% probability of exceedance over 50 years of building life expectancy), the medium probability of earthquake occurrence (5%/50 years) and frequent probability (50% /50 years) are considered and analysed using a suite of artificial ground motions. Secondly, the demand is incrementally increased until collapse and three sets of fragility curves highlighting the history of storey drift, residual drift, and floor acceleration are developed using a detailed numerical model and time history nonlinear dynamic analyses. Damage levels are maped on IDA curves as a function of earthquake intensity and the structure seismic response. From this study it was found that residual drift is the key engineering demand parameter used to define the severe damage state. The global dynamic instability occurred at a larger demand (6 x design-level intensity on average) and one type of collapse mechanism involving the bottom three floors was identified. Then, considering the structure collapse fragility and the fragility functions of nonstructural components populating the fragility database of PACT, consequeces of each damage level are expressed in terms of economic loss value. Using PACT, hundreds realizations are performed and each realization represents one possible performance outcome. The suite of each performance outcome corresponding to a suite of earthquake intensity form the loss vulnerability curves: structural repair loss, nonstructural repair loss, demolition loss, collapse loss and total loss.

Keywords: intensity-based assessment, steel structure, nonlinear dynamic analysis, performance, economic loss value.

INTRODUCTION

All buildings of a given importance category designed according to the current code provide a minimum safety-related seismic performance. In general, the building's performance is deemed suitable once the seismic response shows an acceptable probability of collapse when subjected to the 2% in 50 years design earthquake. Meanwhile, a code-designed building could achieve the objective of preventing loss of life-threatening but may exhibit extensive nonstructural damage and substantial economic losses. Although the first generation of performance based seismic design (PBSD) concept was proposed two decades ago, it did not reach the consensus of the committee of National Building Code of Canada (NBCC). In the PBSD process, the design of buildings is adjusted to reach the target performance levels. Within this concept, the probability of collapse is the main parameter considered for building performance assessment. To communicate the performance of a building in ways that better relate to the decision-making needs, a next-generation PBSD was proposed in the frame of FEMA P-58-1 [1]. This framework is open to other potential quantitative criteria that could vary as a function of building occupancy type and stakeholders' decision. These criteria can be group to respond to: safety-related measures expressed in terms of the risk of causalities, damage-control to functionality-related equipment, potential-control for loss of beneficial use and others. Once the criteria is defined, the intensity-based assessment procedure can be used to quantify the probable building performance for code design earthquake and/ or for other target intensity levels. The shaking intensity is defined by the elastic response spectrum.

According to [1], the flowchart of the performance methodology assessment consists of five steps: i) develop the building performance model, ii) define earthquake hazard, iii) analyze the building response, iv) develop the collapse fragility curve and v) calculate performance. In this article, the case study is a middle-height moderately ductile concentrically braced frame building of normal importance category located in eastern Canada. The structural and nonstructural components of selected building are categorized into fragility groups and performance groups. It is noted that fragility groups are made of similar components that have the same potential damage characteristics and performance groups are parts of a fragility group that experience the same earthquake demands in response to earthquake loading. The earthquake hazard is intensity-based and the selected analysis method is the nonlinear response history analysis. The selected engineering demand parameters are storey drift, residual storey drift and floor acceleration. To assess the building performance, the modes of structural collapse are required in order to develop the adjusted collapse fragility functions. To calculate losses for a given earthquake intensity, the PACT [1c] was employed and hundreds realizations were considered. Each realization represents one possible performance outcome. The suite of each performance outcome corresponding to a suite of earthquake intensity form the loss vulnerability curves such as: structural repair loss, nonstructural repair loss, demolition loss, collapse loss and total loss.

CASE STUDY

Basic Building Data, Fragility and Performance Groups

A fictitious 8-storey prototype office building located in Montreal on site Class C is designed and analyzed according to NBC 2015 [2] and steel design standard CSA/S16-14 [3]. The floor area is 2299 m^2 and the plan is illustrated in Figure 1a. The typical storey height is 3.6 m and that of ground floor is 4.0 m. The slab of each floor is made of composite steel deck and curtain walls were selected for the building enclosure. According to RSMeans [4] the building replacement cost is $ 27 million and the replacement time is 23 months. The building has one basement level that is neglected in this study. In each orthogonal direction, the building is braced by four identical CBFs with multi-storey X-bracing configuration. The CBFs are designed as moderately-ductile (MD) with $R_d R_0 = 3.9$, where R_d and R_0 are the ductility- and overstrength-related force modification factor, respectively. All CBFs braces are designed as tension-compression members. The CBF elevation and member cross sections, as well as the design loads are given in Figure 1b. In the elevation, *N* is the total number of vulnerable storeys. As shown, floor level *N* comprises the slab at the level and all structural and nonstructural components up to the storey immediately above; the roof is allowyes (*N*+1). This is the typical storey designation number considered in the building performance model.

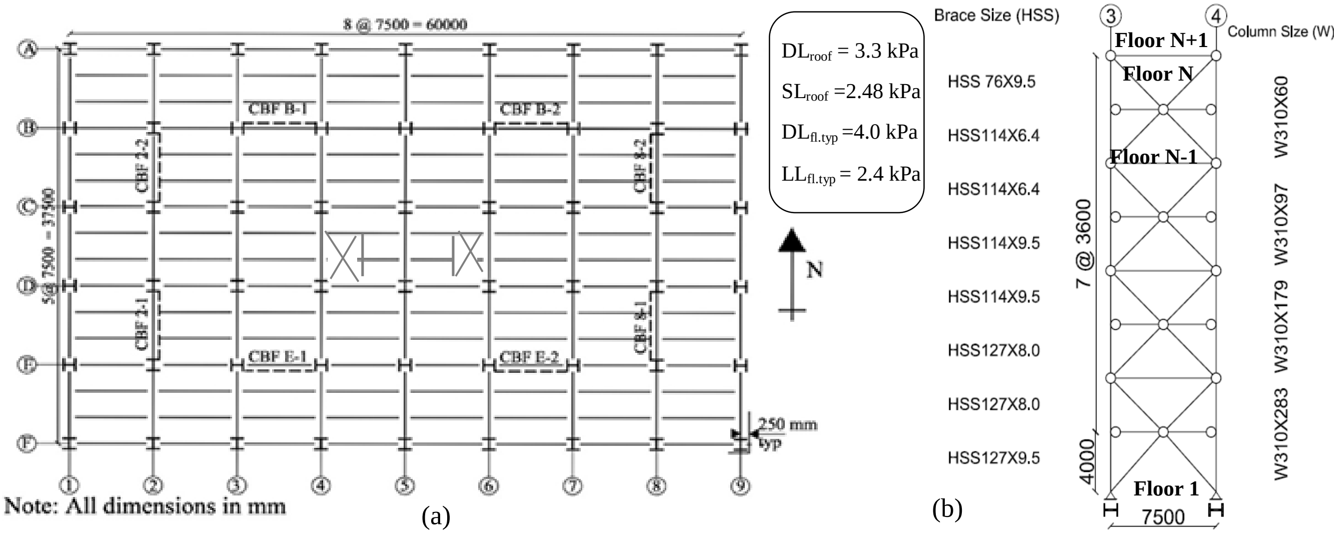

Figure 1. Building studied: a) floor plan and b) CBF1 elevation

According to NBC 2015 [2], the equation used for the fundamental period is $T_a = 0.05h_n$ which leads to $T_a = 1.46$ s for h_n=29.2 m, where h_n is the building height. The seismic weight including the building exterior enclosure and 25% snow load at roof is W = 71768 kN. Using the equivalent static force procedure, the design base shear V is computed as follows: $V = I_E S(T_a) M_v W / R_d R_0$ where I_E in the earthquake importance factor of structure, $S(T_a)$ is the spectral acceleration ordinate corresponding to period T_a and M_v is the higher mode effects on base shear. In the above equation, $I_E = 1.0$, $M_w = 1.0$ and $S(T_a) = 0.11$ g which leads to V = 2046 kN. The building is regular in plan and elevation. Using a three-dimensional structural model developed in ETABS [5], the first mode period computed in N-S direction is $T_1 = 1.76$ s and the associate base shear V_{dyn} =1656 kN resulted slightly greater than 0.8V which is deemed acceptable in [2]. The torsion caused by accidental eccentricity was neglected and P-delta effect was considered. The member sizes for CBF1 are given in Fig. 1b.

In PACT [1c], the replacement cost is used when the building exhibits a damage level that renders it irreparable. This occurs when the residual interstorey drifts exceed the level considered practicable to repair or when collapse occurs. The replacement cost includes replacement of building structure, exterior enclosure, the mechanical, electrical, and plumbing system and other nonstructural components such as partition walls, celling, etc. Although FEMA [1b] considers the building irreparable when 50% of replacement cost is required, past studies suggest that 40% of the replacement cost is a practical total loss threshold. To account for demolition and site clearance, the building replacement costs may increase by 20% according to FEMA [1b].

In PACT, structural and nonstructural components are categorized into fragility groups and performance groups. Each fragility group is identified by a typical fragility classification number, description of the fragility group (e.g. structural CBF system, curtain walls, partitions, composite steel decks, HVAC system, etc.) and the associated demand parameter (e.g. storey drift for drift-sensitive nonstructural components and floor acceleration for the acceleration-sensitive nonstructural components). Fragility information of nonstructural components are embedded in *Fragility Database* of PACT. A performance group is a subset of fragility groups defined for each orthogonal direction and each floor level in function of the associated demand parameter as storey drift or floor acceleration. Thus, the performance groups are organized by each storey level and loading direction (e.g. N-S, E-W). When the earthquake demand increases, the level of building damage increases as well. In PACT, a series of discrete damage states is assigned to each fragility group. The repair cost, repair time, and casualties are the potential performance measures. However, in this article, the performance measure based on repair cost is considered.

Ground motions and design spectrum

Due to lack of historical ground motions in Easter Canada, a suit of artificial records corresponding to Site Class C (360 m/s < $\boldsymbol{V_{s30}}$ <760 m/s) in Montreal, developed by Atkinson [6] is considered and selected from www.seismotoolbox.ca. These artificial records correspond to earthquakes of magnitude 7 at an epicentral distance varying from 13.8 km to 50.3 km. According to NBC 2015, the minimum number of records used in analysis should be not less than 11. However, for a defined scenario-specific target spectrum, using fewer than 11 records per suite is permitted but the number should not be less than 5. In this study, 7 records of about 20 s duration are considered and their seismic characteristics are provided in Table 1. Herein, the PGA and PGV are the peak ground acceleration and peak ground velocity, respectively, t_D is the Trifunac duration, T_p is the main period of ground motion record and T_m is the mean period. As resulted, these records are reach in high frequency content.

To perform nonlinear time history analysis, these ground motions are scaled with respect to NBC 2015 procedure such that the mean spectrum of a suite of minimum records discussed above is not less than 90% of the design spectrum in the period range of $0.2T_1$ to $2.0T_1$. For Montreal, the design spectrum is associated to 2%, probability of exceedance in 50 years which corresponds to a rare earthquake. However, to investigate the potential effect of a medium or frequent earthquake the 5% and 10% probability of exceedance in 50 years are also considered. The 2%, 5% and 10% in 50 years spectra for Site Class C in Montreal are plotted in Figure 2a and the scaled records with respect to 2% in 50 years design spectra is depicted in Figure 2b.

Table 1. Seismic characteristics of artificial ground motions selected

Event	$\boldsymbol{M_w}$	Station	PGA $(\boldsymbol{g})$	PGV $(\boldsymbol{m/s})$	PGV /PGA	$\boldsymbol{t_D}$ $(\boldsymbol{s})$	$\boldsymbol{T_p}$ $(\boldsymbol{s})$	$\boldsymbol{T_m}$ $(\boldsymbol{s})$
M7C1-13.8	7.0	Simulated	0.727	0.370	0.052	7.180	0.120	0.244
M7C1-20.1	7.0	Simulated	0.653	0.396	0.062	6.012	0.140	0.296
M7C1-25.2	7.0	Simulated	0.386	0.187	0.049	7.320	0.060	0.243
M7C1-25.6	7.0	Simulated	0.339	0.194	0.058	7.846	0.160	0.266
M7C1-25.8	7.0	Simulated	0.293	0.178	0.062	7.308	0.080	0.282
M7C2-41.6	7.0	Simulated	0.229	0.144	0.064	7.614	0.140	0.306
M7C2-50.3	7.0	Simulated	0.151	0.075	0.051	8.744	0.160	0.277

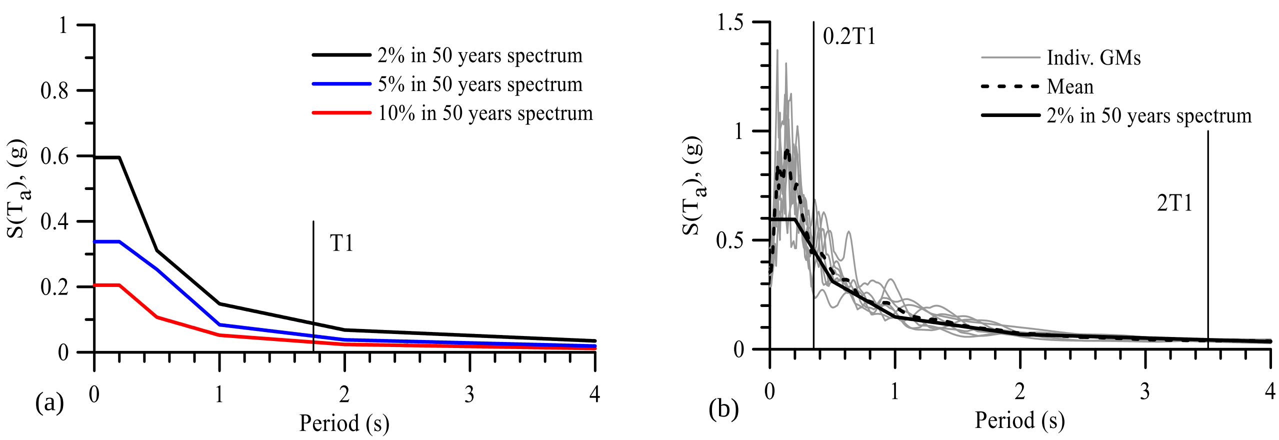

Figure 2. Response spectra: a) design spectrum and b) scaled response spectrum of selected records in the range $0.2T_1 – 2T_1$

Response history analyses through collapse

All braces are considered pin-ended, as well as, the beams of MD-CBFs. Columns are pinned at their base and are continuous with constant cross-section over two storeys. Beams and columns are made of W-shape sections and braces of square hallow structural section, HSS. All structural members are made of steel with yield strength F_y = 350 MPa. In capacity design, the probable yield stress is taken as R_yF_y, where R_y = 1.1 for W-shape sections and the product R_yF_y = 460 MPa for tubular HSS sections. The linear dynamic analysis by means of modal response spectrum was employed using ETABS to obtain the building storey drifts needed for P-delta consideration. The period of the first three modes in the N-S direction resulted from ETABS are given in Table 2.

Table 2. Vibration periods of MD-CBF building in ETABS and OpenSees (N-S direction)

Period (s)	Linear dynamic analysis (ETABS)	Nonlinear dynamic analysis (OpenSees)
T_1	1.752	1.753
T_2	0.596	0.585
T_3	0.320	0.307

To obtain the nonlinear response of the building, the suite of ground motions given in Table 1 was considered. Nonlinear time history analysis was performed using OpenSees [7]. To analyze the response of building to free vibrations, additional 10 seconds of zero amplitude were added to the total duration of each ground motion. In the nonlinear building model, braces were simulated using the force-based nonlinear beam-column element with distributed plasticity and fiber cross-section discretization. Each HSS brace was made of 16 nonlinear beam-column elements with three integration points per element. An out-of-straightness of 1/500 of effective brace length was assigned out-of-plane at brace mid-span to allow buckling. Steel02 material, as well as a low-cycle fatigue model to replicate the brace fracture explained in [8] were assigned to braces. The brace to frame gusset plate connection was simulated by two rotational springs and one torsional spring installed in the zero-length element connecting the brace to a rigid link. Beams and columns of CBFs are simulated using the same force-based nonlinear beam-column element with distributed plasticity and fiber cross-section discretization and Steel02 material. A 2% mass and stiffness proportional damping was assigned to the first and third vibration mode to members expected to respond elastic. The elastic period of the first three modes resulted in the N-S direction from OpenSees are also given in Table 2.

To investigate the building response, the selected engineering demand parameters are: the storey drift, storey residual drift, and floor acceleration. To capture the potential damage of nonstructural components, the building was subjected to selected ground motions scaled to 10%/ 50years, 5%/50 years and 2%/50 years probability of exceedance. In Figure 3 is depicted the distribution of engineering demand parameters along the building height, their mean values, as well as the mean plus standard deviation (Mean +SD). As resulted from Figure 3c, the nonlinear response to records scaled to code design spectrum (2% probability of exceedance in 50 years) shows a peak of Mean+SD storey drift of 0.5% h_s at the upper top floors, a peak of mean+SD residual storey drift of about 0.05% h_s at the 3rd and 6th floor and a uniformly distributed floor acceleration slightly less than 0.4g. When records are scaled to 5%/50 years, the peak of Mean+SD storey drift decreases to 0.4% h_s at top floor and the floor acceleration of 0.3g is still uniformly distributed as depicted in Figure 3b. It is noted that the residual storey drift is negligible. When the building response is investigated for lower demand (10%/50 years) the peak of Mean+SD storey drift decreases to 0.2% h_s and the floor acceleration to 0.15g (Figure 3a). As depicted, the building responded in the first vibration mode.

To assess the distribution of nonlinear seismic response at every level of structural behavior from yielding to global dynamic instability (failure) the incremental dynamic analysis (IDA) [9] is employed using the same suite of ground motions scaled to a series of incremented intensity levels considering a multiple of 0.05g. The selected intensity measure (IM) is the 5% damped spectral acceleration at the first-mode period of the building, $S_a(T_{1,}5\%)$ and the selected engineering demand parameter (EDP) is the storey drift. The response of structure to each incrementally increased ground motion until the global dynamic instability is reached is represented by an IDA curve depicted in Figure 4a. Analyzing the shape of each IDA curve it results that the building nonlinear response is sensitive to the frequency content of each ground motion. Thus, under some ground motions, IDA curves show a softening behaviour which means that damage accumulates at higher rates. In general, this type of behaviour occurs when the ground motion accelerogram presents a suite of sustained amplitudes in both oscillation directions. Conversely, if the accelerogram presents a few peaks, the IDA curve shows a consolidated behaviour and the structure is able to sustain larger intensity demand while exhibiting moderate storey drifts. Thus, for an IM ~ 0.03g on average, the first flexural buckling of braces occurs (the black solid line in Figure 4a). The median IDA curve (red line) of a suite of seven is also depicted in Figure 4a. This shows that the studied building could withstand a median intensity of 0.65g while undergoing $5.05\%h_s$ storey drift. It can be summarized that the median intensity associated to global dynamic instability is about six time greater than the code design spectrum intensity marked with a black dashed line in Figure 4a. To investigate the level of damage of acceleration-sensitive nonstructural components the suited EDP is the floor acceleration. Figure 4b presents the seven IDA curves (black lines) and their median IDA curve (red line) which shows that the building could withstand an IM = 0.65g while undergoing a peak floor acceleration of 1.5 g. It is noted that around 1.0 g floor acceleration which, in this case, is associated with IM ~ 0.4g, several acceleration-sensitive nonstructural components undergo damage. Furthermore, to investigate the level of damage exhibited by the drift-sensitive nonstructural components, the residual storey drift was also selected as an EDP. The seven IDA curves, as well as, the median curve are depicted in Figure 4c. From this figure it is clearly shown that the median residual drift starts accumulated at higher rates when the demand is larger than IM~ 0.4g, which is about four times the design-level.

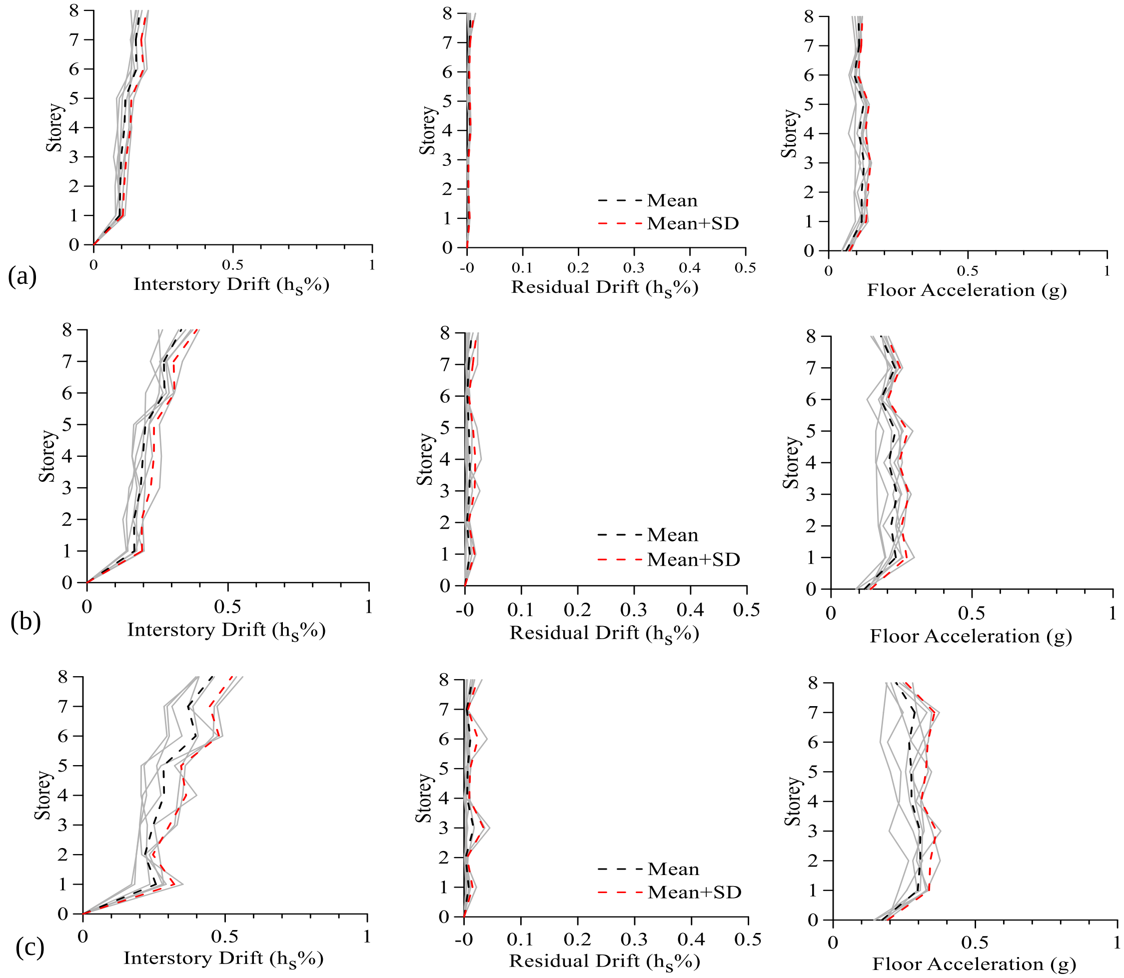

Figure 3. Nonlinear response of MD-CBF building with GMs scaled to: a) 10% probability of exceedance in 50 years; b) 5% probability of exceedance in 50 years; c) 2% probability of exceedance in 50 years (design spectrum)

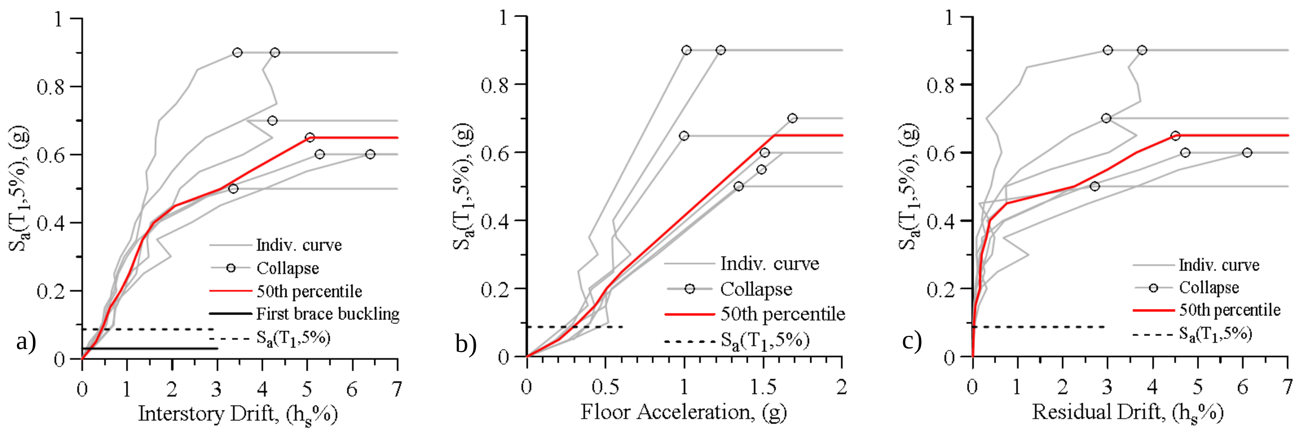

Figure 4. IDA curves of the 8-storey MD-CBF building computed in terms of: a) Storey drift; b) Floor acceleration, c) Residual storey drift.

To follow the performance methodology assessment, the number of failure mechanism types and the percentage of each floor undergoing failure under the suite of ground motions are required. Analyzing the permanent deflected shape of CBF1 triggered by each ground motion scaled to IM associated to dynamic instability limit state (collapse point), a similar collapse mechanism type involving the bottom three floors was observed. The response of studied CBF structure to three selected ground motions (e.g. M7C1-3.8, M7C1-25.6 and M7C2-50.3) scaled to IM corresponding to the collapse point is depicted in Figure 5 together with the peak storey drift experienced by each floor. Fracture of left and right brace located at the 3rd floor, as well as, that of the tensile brace located at ground and second floor was observed. A plastic hinge was also formed at the mid-span of bottom beam to which the braces experienced fracture were attached. The type of failure mode resulted under seven ground motions is similar. Hence, as depicted, the damage is concentrated at bottom three floors, while the upper floors experience buckling and yielding of braces. No damage of CBF columns was observed. Furthermore, the response of structure indicates that most of gravity columns remain elastic. However, at this stage, the building is considered total loss. For CBF buildings located on high-seismic zone on the Pacific Coast different collapse mechanism types were observed [10-11].

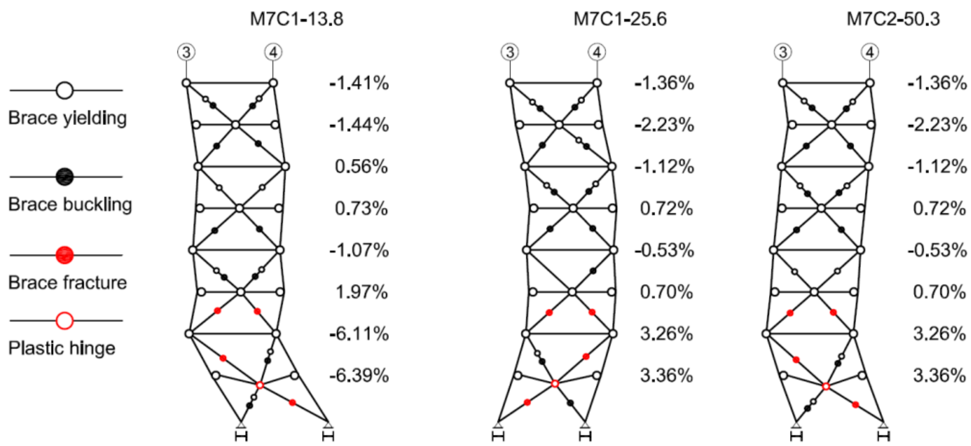

Figure 5. Failure mode of the studied 8-storey MD-CBF building

Fragility curves

In this article, the resulting collapse capacity of studied building and the record-to-record variability is adjusted to take into account the spectral shape factor, SSF, as described in FEMA P-695 [12]. Accordingly, the adjusted collapse margin ratio, ACMR is calculated as the product of CMR and SSF, where CMR is the collapse margin ratio computed as $\hat{S}_{CT}$ / S_{MT}, where $\hat{S}_{CT}$ is the median collapse capacity defined at the intensity of ground motion at which half of the records in the selected suite cause collapse and S_{MT} is the design spectral acceleration intensity at T_1. Herein, for T_1 = 1.75s it results S_{MT} = 0.088g and from Figure 4a it results $\hat{S}_{CT}$= 0.65g which leads to CMR = 7.39. From tables provided in [10] the SSF = 1.32 which leads to ACMR = 9.75. This large ACMR value shows that the studied building possesses a larger margin safety to collapse. The lognormal standard deviation parameter β_{TOT}, describing total collapse uncertainty is computed as follows: β_{TOT} =(β_{RTR}^2 +β_{DR}^2 +β_{TD}^2 +β_{MDL}^2)$^{0.5}$ where β_{RTR} = record-to-record collapse uncertainty, β_{DR} = design requirements-related collapse uncertainty, β_{TD} = test data-related collapse uncertainty and β_{MDL}= modeling-related collapse uncertainty. It is noted that the acceptance collapse criteria is based on the composite uncertainty, β_{TOT}. To quantify β_{TOT} according to [12] the following assumptions are made: (1) the quality of design requirements were considered as "(A) Superior" with corresponding $\beta_{\mathrm{DR}} = 0.1$; (2) the quality of test data was considered as "(B) Good" with corresponding $\beta_{\mathrm{TD}} = 0.2$; (3) the model quality was considered as "(B) Good" with corresponding $\beta_{MDL} = 0.2$. From calculation it results β_{TOT} = 0.5. Acceptable values of adjusted collapse margin ratio are based on β_{TOT} and on the established values of acceptable probability of collapse based on the assumption that the distribution of spectral intensity at collapse is lognormal with a median value $\hat{S}_{CT}$ and a lognormal standard deviation equal to β_{TOT}. Figure 6a shows the demand associated with severe damage (residual drift 0.5%h_s) and collapse. Figure 6b shows the adjusted collapse fragility curve which describes the probability of collapse P(C|*IM*) as a function of $S_a(T_1, 5\%)$. Similarly, considering the IM level of $S_a(T_1,5\%)$ ~ 0.4g associated to 0.5%h_s residual storey drift, the severe damage (SD) fragility curve is also plotted.

Assessment of economic losses conditioned on seismic intensity

The building-specific loss estimation methodology discussed in Ramirez and Miranda [13] considers the following equation to compute the total loss in a building (L_T) subjected to an earthquake event with a ground motion intensity *IM*:

$$L_T = L_{NC\cap R} + L_{NC\cap D} + L_C \quad (1)$$

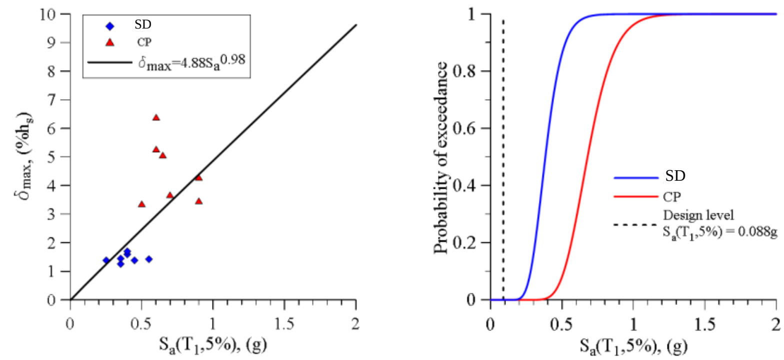

Figure 6. Seismic demand and the adjusted collapse fragility curve and severe damage fragility curve

In the above equation, $L_{NC\cap R}$ is the loss in the building given that the collapse does not occur (no collapse) and the structure is repaired, $L_{NC\cap D}$ is the loss in the building where no collapse occurs but the building is demolished because of the significant cost associated with repairing and straightening the permanent deformations exhibited by structure and rebuilding is required, and L_C is the loss in the building when collapse occurs and the building shall be rebuilt. Assuming that these consequences are mutually exclusive, the expected value of the loss in the building for a given seismic intensity *IM* is computed as follows:

$$E[L_T|IM] = E[L_T|\, NC \cap R, IM]\, P(R|\, NC, IM)\, P(NC|\, IM) + E[L_T|\, NC \cap D]\, P(D|\, NC, IM)\, P(NC|\, IM) + E[L_T|\, C]\, P(C|IM) \quad (2)$$

where $E[L_T|\, NC \cap R, IM]$ is the expected value of the total loss in the building under an earthquake of intensity *IM* when collapse do not occur and reparation is required, $E[L_T|\, NC \cap D]$ is the expected value of total loss in the building when there is no collapse but demolition is required. In the calculation, this loss is assumed to be equal to the replacement cost of the building plus additional 10% of replacement cost for demolition and site clearance. The $E[L_T|\, C]$ is the expected value of the total loss in the building when collapse occurs and this value corresponds to the replacement cost. Then, $P(R|\, NC, IM)$ is the probability that the building will be repaired given that no collapse occurs and $P(NC|\, IM)$ is the probability of no collapse for a given earthquake intensity *IM*. Furthermore, $P(D|\, NC, IM)$ and $P(C|IM)$ are the probability that the building will be demolished although no collapse occurs and the probability that collapse occurs, respectively, for the given earthquake intensity *IM*. Knowing the EDPs at each storey for a suite of intensity measure values and considering the fragility curves of nonstructural components provided in PACT library, a storey-based building specific loss estimation approach is employed. Using data from RS Means [4] for an office building with tinted plate glass panels enclosure, the distribution of building cost in percentage assigned to structural and nonstructural components are plotted in Figure 7a. As resulted, the building structure is 20% of total building cost and the neglected substructure is only 3%. In PACT which is correlated with RS Means, the Exterior enclosure: division B20 and the Interiors: division C (e.g. partitions, doors, wall finishes, etc.) are drift-sensitive nonstructural components dependent on the earthquake direction. All the other nonstructural components of division D are acceleration-sensitive. Figure 7b presents the loss-vulnerability curves, where the vertical line shows the economic loss in percentage of building replacement cost at various ground motion intensity levels (e.g. from $S_a(T_1, 5\%)_{design} = 0.088g$ to $S_a(T_1,5\%)/S_a(T_1,5\%)_{design} = 7$). As depicted from IDA curves based on residual drift, for IM = 0.46 g the residual drift increases significantly for 5 out of 7 ground motions

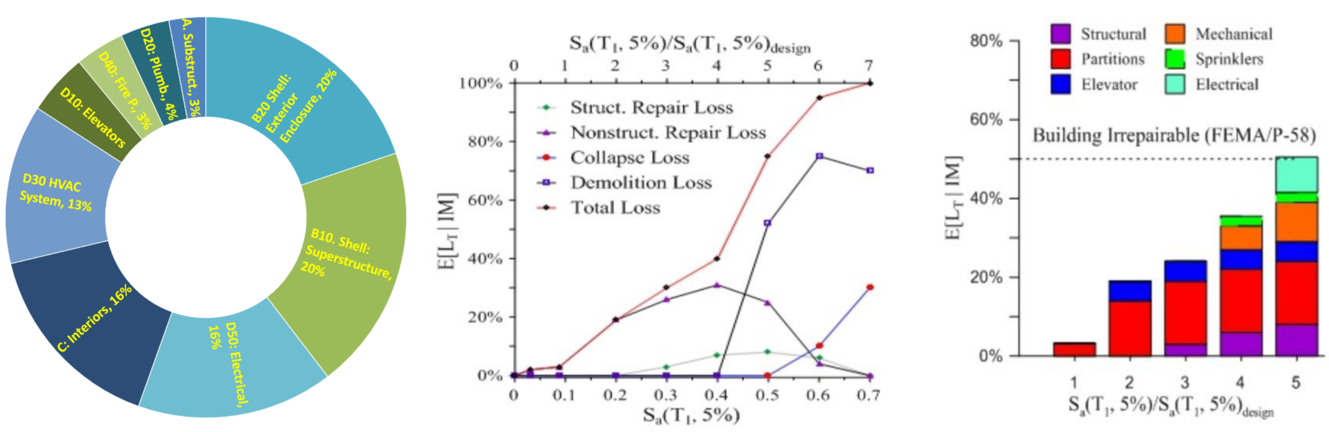

Figure 7. Earthquake loss estimation: a) distribution of building cost among its components, b) Loss-vulnerability curves for 8-storey MD-CBF building in Montreal, c) detailed distribution of % of building loss value at several IM levels

and demolition is required. At design-level intensity, minor damage of partition walls is expected. In Figure 7c is depicted a detailed distribution of damage exhibited by the drift-sensitive and acceleration-sensitive nonstructural components, respectively, and structure, resulted for incremented intensity levels. In terms of performance, from Figure 7, can be concluded that at design-level earthquake, the level of economic loss is about 3%, at 2 *x* design-level intensity the economic loss is 20% and at 4 *x* design-level intensity it increases to 40% which may be the threshold for building demolition and replacement.

CONCLUSIONS

This paper assessed the performance of an 8-storey MD-CBF building of normal importance category located on firm soil in Montreal using the intensity-based methodology linked to an earthquake-induced economic losses approach. The building's structure was designed to 2%/50 years design-level earthquake as per NBC 2015. Due to lack of historical ground motions in Easter Canada, a suit of seven artificial records corresponding to earthquakes of magnitude 7 of about 20 s duration were used for analyses. Because these records are rich in high frequency content, as expected, they cause more damage to stiff structures with high natural frequency than to flexible structures which are discussed herein. The main findings are summarized as:

1. The flexural buckling of steel braces occurred at small storey drift (e.g. 0.3%h_s on average). For 2%/50 years demand all braces in the upper half floors exhibited buckling. When the building was investigated to lower demand (e.g. 5%/ 50 years) only braces in the upper two floors experienced buckling. At design-level earthquake, the building response showed a peak storey drift of 0.5%h_s at upper floors and uniformly distributed floor acceleration of 0.3 g on average.
2. To investigate the building response at incremented earthquakes demand, the IDA was used. Three sets of IDA curves were computed to emphasize the variation of EDPs expressed in terms of storey drift, residual storey drift, and floor acceleration. It was found that up to a demand of 4 *x* design-level earthquake intensity, damage was slowly accumulated in the structure (e.g. 1.5%h_s drift, 0.5%h_s residual drift and 1.0g floor acceleration on average). After that, for a small increase in demand, the residual drift increased at faster rates reaching 2.5%h_s for 5 *x* design-level earthquake. Analyzing the type of collapse mechanism driven by each one of the seven ground motions it resulted a similar type targeting the bottom three floors where a two-storey failure mechanism leading to sudden collapse was formed.
3. Both aleatoric and epistemic uncertainties were considered to compute the adjusted collapse fragility curve and the severe damage fragility curve that are used to assess the structural and nonstructural building damage transposed in economic losses. Using the methodology proposed in FEMA P-58 linked to the building-specific loss estimation methodology discussed in Ramirez and Miranda the loss-vulnerability curves for the 8-storey MD-CBF building in Montreal were computed. It was found that at design-level earthquake, the level of economic loss is about 3%, at 2 *x* design-level intensity the economic loss is 20% and at 4 *x* design-level intensity is 40%. The latter is recommended to be the threshold for building demolition and replacement. It was also found that residual drift is a key engineering demand parameter.

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