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SEISMIC PERFORMANCE EVALUATION OF MODERATELY DUCTILE RC FRAME STRUCTURES USING PERFORM-3D

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Abstract

The principles for seismic assessment of buildings have shifted from strength-based to performance-based evaluation. Performance-based assessment involves evaluation of structures for pre-determined performance objectives. The structures are analyzed under different earthquake records with different intensities through response history analysis. Inelastic response history analysis requires analytical modelling of buildings and computer software that captures structural performance in the non-linear range of deformations. The objective of this paper is to present analytical modelling techniques for reinforced concrete (RC) frame structures, suitable for use with PERFORM-3D software. A detailed discussion of the effects of modelling parameters on structural response is presented. The parameters of moment-rotation envelope used for hysteretic modelling of members are described. The features of hysteretic modelling, including stiffness degradation as controlled by the "energy degradation factor" and strength decay are presented. An analytical model of a 5-storey moderately ductile reinforced concrete frame building, designed based on the current Canadian practice, was generated for use with PERFORM-3D software. The structure was analyzed to assess global and component level responses by monitoring inter-storey drift ratios, formation and sequence of plastic hinging, and load-displacement hysteretic relationships. The suitability of the analytical model developed for PERFORM-3D was verified against SAP2000 and SeismoStruct software. The paper presents the construction of the analytical model, model validation, hysteretic modelling, and the results of dynamic analyses. Structural performance is presented at different limit states employed as performance objectives.

Keywords: RC Frame, PERFORM-3D, Analytical model, Moderately Ductile

1. Introduction

The emphasis in seismic vulnerability assessment of buildings in recent years has shifted from force-based to performance-based evaluation, where structural members are assessed for pre-determined performance objectives. The performance objectives are defined on the basis of selected hazard levels and target structural and non-structural performance levels. Building elements show different levels of performance under different levels of hazard, covering the entire spectrum of elastic and inelastic structural response. Performance-based seismic assessment of buildings requires dynamic inelastic response history analysis under different levels of seismic hazard.

The objective of this paper is to illustrate the development of an analytical model for a 5-storey reinforced concrete frame building for dynamic analysis. The objective also includes the investigation of modelling parameters on structural response, especially those that define the primary moment-rotation relationships of members, as well as those that affect stiffness and strength degradation of members using a moderately ductile building. The latter parameter is especially important as the global performance of a structure depends on performance of individual structural elements which may deform beyond their inelastic capacities, experiencing strength decay prior to developing global collapse. Therefore, it is essential to model strength degradation to account for gradual reduction of element contribution to overall structural resistance, as the members progressively experience failure.

The strength decay was modelled using three parameters; i) the deformation at onset of strength decay, ii) the slope of the descending branch of force-deformation envelop curve up to the residual deformation, iii) the residual deformation beyond which the resistance sharply drops to zero at a constant rotation. The onset of strength decay was defined following two approaches. The first approach involved the use of the recommendations of ASCE 41-13[1]. ASCE 41-13 provides rotational capacities for reinforced concrete structural components of existing buildings, as well as those for new components added to existing buildings. These rotational limits were adopted to model the onset of strength decay. The second approach involved the use of the National Building Code of Canada (NBCC)[2] and the associated material design standard for design of reinforced concrete elements, CSA A23.3-04[3]. For the moderately ductile building used in this investigation, ductility related force modification factor of R_d = 2.5 was used to define the strength decay onset point. This was done by multiplying the yield rotation computed on the basis of CSA A23.3-04 by the R_d factor. The slope of the strength decay branch (descending branch) and the residual deformation were modelled the same as those used in the first approach following the ASCE 41-13 recommendations. A moderately ductile frame building designed to fulfill the requirements of CSA A23.3-04 for the city of Ottawa in Canada was used for analytical modelling and response history analysis using computer software PERFORM 3-D [4].

The stiffness degradation in the hysteretic model used was implemented by means of the energy degradation factor (EDF), which is a parameter that affects the overall shape of hysteresis loops in PERFORM-3D. The parameter EDF is applied to the perfectly elasto-plastic hysteretic model to adjust the slopes of unloading and reloading branches to adapt it to reinforced concrete structures, resulting in a stiffness degrading hysteretic model. The shape of the resulting hysteresis loops may potentially affect inter-storey drift, which is used in the current investigation as a damage parameter. Since the main goal of the analysis was to find seismic damage state of structures, the effect of EDF on structural drift was studied by employing 20 seismic records that are compatible with the uniform hazard spectrum (UHS) specified in the 2010 NBCC for Ottawa.

2. Description of Selected Structure

A reinforced concrete frame building with a 5-storey height, having a regular floor plan was selected for analysis. The building had a square floor plan with 5 bays in each direction with a 7.0 m span length. The storey height was 4.0 m. The elevation view of the building is shown in Fig. 1. It was designed and detailed according to the provisions of CSA A23.3-04 as a moderately ductile structure located in the city of Ottawa in eastern Canada. The design live load was 2.4 kPa for all the floors, including the roof, and the superimposed dead load was 1.33 kPa over the self-weight. Uniform hazard spectrum for Ottawa was selected from the 2010 NBCC to calculate the seismic base shear. The peak ground acceleration (PGA) for the design earthquake was 0.32g. The building was selected for normal importance (I = 1.0) with a Site Class of C. The first 12 modes were considered to participate in seismic response. The elastic base shear, V_e, was reduced by the product R_dR_o, where R_d = 2.5 is the ductility related force modification factor and R_o = 1.4 is the over-strength related force modification factor for the city of Ottawa. The building was analyzed under the equivalent static seismic load, as well as the accompanying gravity loads as per 2010 NBCC using software ETABS [5] and the appropriate load combinations. The concrete used was 30 MPa with elastic modulus computed as $E_c = 4500\sqrt{f_c'}$. Cracked section properties with effective moment of inertia I_e as specified below, were used in the ETABS analysis:

Beam $I_e = 0.4\, I_g$

Column $I_e = \alpha_c I_g$ Where $\alpha_c = 0.5 + 0.6\, P_s / (A_g f_c') \leq 1.0$

Where I_g = Gross moment of inertia, P_s = Axial force on members resulting from the earthquake load combination, f_c' = specified compressive strength of concrete and A_g = gross area of section. Reinforcement yield strength was taken as f_y = 400 MPa. The analysis results provided design quantities for proportioning members. For the building modelled using ASCE 41-13, the effective moment of inertia was computed from the following equations:

Beam $I_e = 0.3\, I_g$

Column $I_e = 0.7\, I_g$ for columns with compression due to design gravity loads $\geq 0.5\, f_c' A_g$

$I_e = 0.3\, I_g$ for columns with compression due to design gravity loads $\leq 0.1\, f_c'\, A_g$

Linear interpolation was made for columns with axial compression between the above limits.

The difference in effective rigidities resulted in differences in the fundamental period. The period computed based on the CSA A23.3-04 approach resulted in a shorter value than that computed on the basis of ASCE 41-13. For the 5-storey building, the fundamental period was 2.04 sec when the CSA A23.3-04 rigidities were used, and it was equal to 2.23 when the ASCE 41-13 rigidities were used. The building was designed using ETABS Software for the appropriate load and material resistance factors specified in 2010 NBCC and CSA A23.3-04.

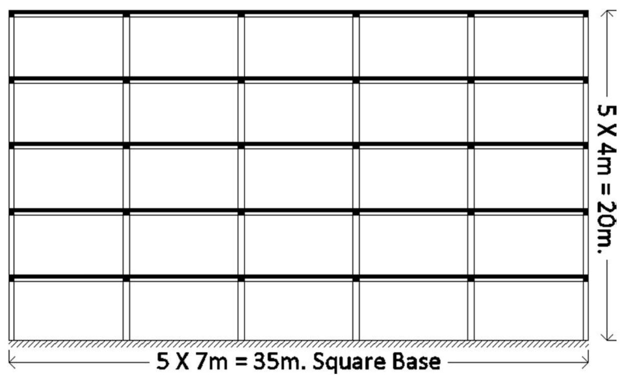


Fig. 1 – Sectional elevation of 5-storey RC frame structure

3. Development of Analytical Model

The analytical model for the 5-storey building selected was developed for computer software PERFORM-3D. PERFORM-3D is specialized software for damage assessment, specifically intended for performance-based seismic assessment of structures. The software permits monitoring of inelastic behavior of structural components with different levels of deformability. It was used by previous researchers to perform nonlinear dynamic analysis [6,7,8,9,10].

The building was modelled as a bare frame building, neglecting possible contributions from non-structural elements. The beams were modeled as concrete type FEMA beam-frame elements, consisting of chord-rotation based models as defined in FEMA 356 [11]. The FEMA beam with symmetrical sections at the ends was selected. The beam element had equal and opposite end moments with a point of inflection in the middle of the span. No member load was permitted along the beam length. Consequently, the beams rotated in double curvature consisting of two segments between the beam ends with the point of inflection in the middle. Each segment was modelled with an elastic beam element and a plastic hinge, as illustrated in Fig. 2. The effective slab width in the beam model was included as beam flange width, as defined in CSA A23.3-04. The slab concrete and reinforcement were taken into account when the beam flange was in compression, and the longitudinal slab reinforcement was taken into account when the beam flange was in tension. The finite width of beam, integral with the attached column was modelled as a rigid segment having 10 times the rigidity of the beam element. This implies that the beam-column joints were assumed to be rigid. The columns were modeled as FEMA column elements with axial force-flexure interaction accounted for in the two orthogonal directions. Similar to the beam element, FEMA column comprised of two elastic segments and two plastic hinges with rigid end zones.

The plastic hinges in PERFORM-3D were assigned hysteretic models that reflected the flexural stiffness of each member during loading, unloading and reloading under seismic excitations. The user specified data included flexural yield strength, ultimate strength prior to strength degradation and percentage of post-yield stiffness relative to the elastic stiffness. The stiffness degradation that occur during unloading and reloading, typically observed in reinforced concrete response, was modelled through the use of cyclic energy degradation factor (EDF). EDF is the ratio of an area under a degraded hysteresis loop to the area under a non-degraded loop (fully plastic hysteresis loop). In this study, the EDF was calculated from the column tests performed by Ozcebec and Saatcioglu (1987) [12]. Accordingly, the hysteretic force-displacement behavior of specimen U6 was adopted for hysteretic modelling as representative of column behaviour. Because the axial load applied on this specimen was only 12% of the nominal capacity, EDF derived from the same specimen was also used for the beam elements. Fig. 3 shows the stiffness degraded hysteresis loops and a non-degraded elastic-perfectly-plastic loop (with a parallelogram shape shown as reference). Loading and unloading braches of non-degraded loop was parallel to the effective elastic stiffness of the backbone curve. The effective elastic stiffness was taken as 40% of the initial stiffness based on uncracked sectional properties. The slope of the post-yield branch (strain hardening slope) of the non-degraded parallelogram was considered to be equal to 3% of the effective elastic stiffness. The same procedure was followed to calculate the EDF under increasing levels of inelastic deformation. The resulting factors were found to be 0.62 up to the yield point and 0.56 thereafter. A sensitivity analysis was conducted on the effect of EDF on structural response, as presented later in the paper.

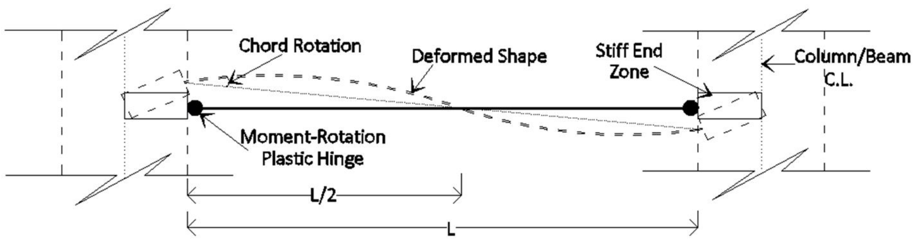


Fig. 2 – Analytical model of beam/column element used to develop frame structure in PERFORM-3D

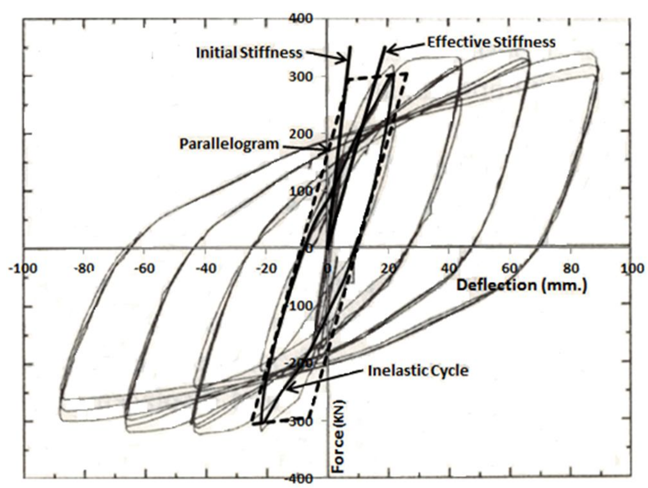


Fig. 3 – Evaluation of Energy Degradation Factor from tests performed by Ozcebe and Saatcioglu (1987)[12]

FEMA beam and column elements have rigid-plastic rotational hinges until member yielding. When a beam or a column exceeds yield capacity, additional rotations are developed in plastic hinges, which are then assigned to the hinges provided at the ends. The moment-rotation backbone curve of a beam or a column element was developed according to the guidelines provided in ASCE 41-13. The yield moment of the beam was calculated from sectional analysis using computer software SAP2000 [13] and the yield chord-rotation was calculated by the software as $(M_yL)/(6E_cI_e)$, where M_y is the yield moment and L is the clear span/height. Similarly, for the columns, SAP2000 was used to conduct sectional analysis in the presence of constant axial load. Hardening stiffness (post-yield slope) of moment-rotation curve was 3% and 4% of the effective elastic stiffness for beams and columns, respectively. For a moderately ductile structural element based on the ductility limit of CSA A23.3-04, the onset of strength degradation started at 2.5 times the yield rotation and the corresponding moment was considered as ultimate moment capacity. This approach was used when the structure was modelled following the requirements of NBCC and CSA. However, when the moment-rotation relationships of structural elements were modelled according to ASCE 41-13, the plastic chord rotation values provided in the document were used. In order to define the strength decay, the moment capacity degraded linearly from the ultimate capacity to a point (M_R) beyond which the element capacity dropped to zero at constant rotation. Strength degradation slope of the elements that modeled according to CSA A23.3-04 was considered as parallel to that modelled using ASCE 41-13. Both structures were designed and detailed as moderately ductile buildings for Ottawa, which is in a medium seismic zone. Fig. 4 shows the moment-rotation envelops with rotational values (Θ_a, Θ_b and 2.5 Θ_y) based on different standards.

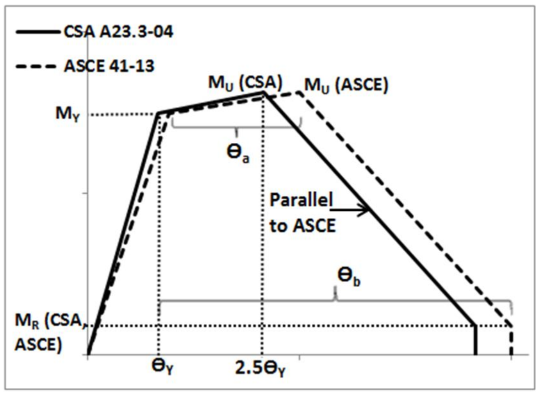


Fig. 4 – Typical moment rotation envelopes used to define structural elements of 5-storey buildings in PERFORM-3D.

The behaviour in shear was considered to be elastic. This was assumed to be true even after flexural yielding. In this study E_c remained unchanged and poisson's ratio was taken as 0.2, which resulted in a constant shear rigidity of $0.4E_cA_w$, where A_w is the gross area of web as specified by ASCE 41-13.

The damping ratio was taken as 5% of critical damping for all modes of vibration. A small amount (0.2%) of Rayleigh damping was applied to ensure that higher modes would not dominate the response. Fig. 5 shows the effect of Rayleigh damping, which increases exponentially after the period (T) reduces to 5% of the fundamental period (T_1). Rayleigh damping becomes 0.05% when T increases to a value greater than 50% of T_1. The mass associated with self-weight of the structure, superimposed dead load and live load were applied at each node.

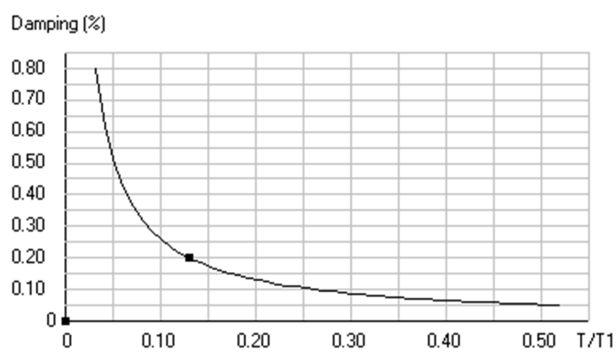


Fig. 5 – Rayleigh damping assigned to the structures in PERFORM-3D

4. Non-linear Model Validation

The analytical model and the dynamic analysis results obtained by PERFORM-3D for the five-storey building discussed in the preceding section were verified against those obtained by the use of two general purpose dynamic analysis software; SAP2000 and SeismoStruct [14]. The validation and comparisons of results are presented in the following sub-section.

4.1 SAP 2000

SAP 2000 is structural analysis software that is commonly used for static and dynamic analysis of structures, developed by Computers and Structures Inc. The same sectional analyses conducted earlier for columns and T-beams were used for modelling the structure in SAP2000. The same seismic masses were assigned to each node. Beam/column elements were modeled as consisting of elastic elements and member-end hinges. Non-linear moment-rotation properties were assigned to the hinges at the ends of beam/column elements. For the purpose of verification of the models in both PERFORM-3D and SAP 2000 models, the beam effective flexural rigidity (EI_e) was taken as constant and was equal to 50% of the gross (uncracked) flexural rigidity. For the columns, the effective flexural rigidity was taken as 70% of the gross column rigidity. The same rotational properties, as recommended by ACI 369R-11 were used in plastic hinges of both PERFORM-3D and SAP 2000 models. The descending branch of the moment-rotation envelop curve was also specified based on the ACI 369R-11 recommendations. Appropriate hysteretic models were used with degrading stiffness characteristics to model the nonlinear hysteretic behaviour of structural elements. Rayleigh damping was applied as 5% of critical damping.

4.2 SeismoStruct

SeismoStruct is software which was developed to perform non-linear dynamic analysis of structures. This software was also used to validate the PERFORM-3D model and the analysis results. In SeismoStruct, beam/column elements were defined as inelastic frame elements (infrmFB), which consisted of fibres with non-linear stress-strain relationships specified for concrete and reinforcing steel. The integration of individual fibre behavior resulted in the overall element performance. InfrmFB provided chord rotations which were compared with those computed by PERFORM-3D. Simplified uniaxial trilinear concrete model “con_tl” was used to define non-linear concrete properties. In con_tl the initial modulus of concrete is linear and can be changed to incorporate the effects of member cracking. Hence, the initial elastic modulus of concrete was reduced to 50% when used to model beam elements, and 70% to model column elements. This resulted in the same E_cI_e properties of elements as defined in PERFORM-3D. The degradation of element stiffness was modelled by reducing the slope of the stress-strain relationship of concrete beyond its peak strength. This was done by reducing the slope of the descending branch of the concrete model such that it resulted in the descending branch of elements to 10% of cracked element stiffnesses for both the beams and columns. The material strengths used to compute the flexural capacities of elements in PERFORM-3D were kept the same to attain the same element strength values in the SeismoStruct model. Damping in SeismoStruct is computed through hysteretic damping, which is included in the nonlinear fibre model formulation of infrmFB. To achieve similar damping as that used in PERFORM-3D analysis, 4% Rayleigh damping was applied to simulate the friction mechanism along the concrete cracks and friction between structural and non-structural members. Seismic masses were assigned to the nodes, as before.

4.3 Comparison of Analysis Results

All three analytical models had the same fundamental period and were subjected to the same earthquake record. The results shown in Fig. 6 indicate that the structural response obtained from SAP 2000 and PERFORM-3D was similar. This observation was expected because the member properties were defined using the same envelop curve in both the PERFORM-3D and SAP 2000 models. On the other hand, member properties in SeismoStruct were defined using concrete and steel constitutive models. Hence the envelope curve was computed using the material constitutive models rather than being specified by the user. Although the structural response within the elastic range showed very good correlation with PERFORM-3D response, the non-linear range of deformations obtained by SeismoStruct did not show as good correlation as that obtained by SAP 2000. However, the onset of plastification and the response wave form showed reasonably good agreement even within the inelastic range of deformations. This is shown in Fig. 6. Hysteretic responses of elements are also compared. Fig. 7 illustrates the hysteretic behaviour of an exterior beam at the first floor level. Figure 7(a) and (c) show the total chord rotation responses, consisting of elastic plus plastic rotations obtained from PERFORM-3D and SeismoStruct analyses, respectively. The comparison shows very good correlation. The same figure (Figure 7(b)) also includes the hysteretic relationship of exterior hinge of the same beam for plastic rotations only, obtained from SAP2000 analysis, showing similar hysteresis loops. It can be concluded from the foregoing discussion that PERFORM-3D modelling and analysis techniques are verified against two commonly accepted, industry standard software, and hence can be used in further developments.

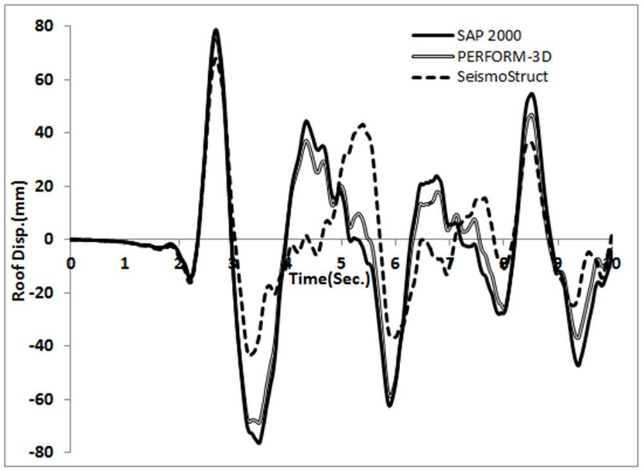
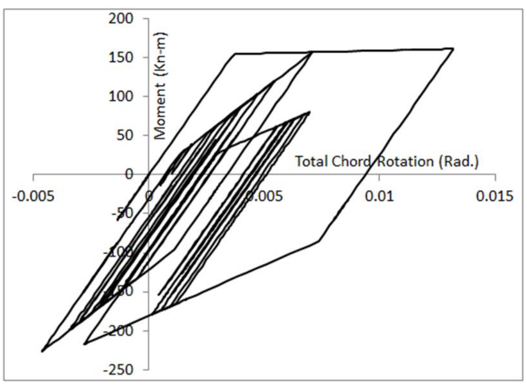
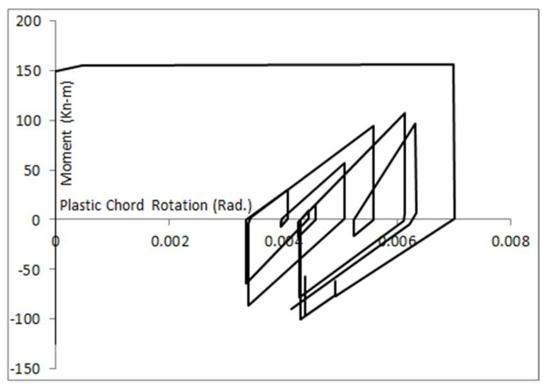


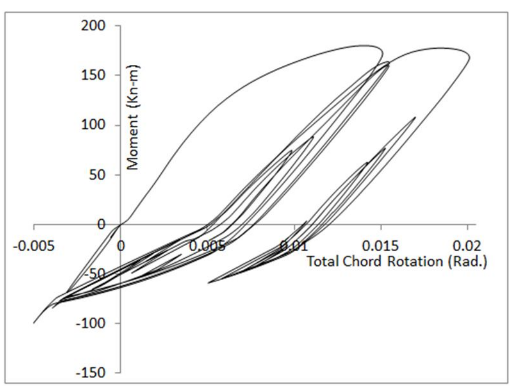
Fig. 6 – Comparison of time-history response of 5-storey analytical models.



(a)



(b)



(c)

Fig. 7 – Exterior beam at first floor level (a) Moment vs Total Chord Rotation in PERFORM-3D, (b) Moment vs Plastic Chord Rotation in SAP2000 and (c) Moment vs Total Chord Rotation in SeismoStruct.

5. Sensitivity Analysis of Energy Degradation Factor (EDF) Used in PERFORM-3D

EDF is a parameter that affects the overall shape of hysteresis loops in PERFORM-3D; in particular the slopes of unloading and reloading branches of the hysteretic model. The shape of hysteresis loops may potentially affect inter-storey drift, which is used in the current investigation as a damage parameter. Since the main goal of analyses was to find seismic damage state of structures, the effect of EDF on structural drift was studied. Zeynep Tuna (2012) [8] suggested a value of EDF for shear critical coupling beam elements to be between 0.5 and 0.35 in the post-yield region, up to the development of residual capacity. Wen-Cheng Liao (2010) [9] used pinching material model to calculate EDF and suggested a value between 0.25 and 0.15 depending on the section properties. Ghodsi and Ruiz (2010) [7] used 0.24 to 0.2 as EDF for frame beam elements. Higher values were found to be suitable for confined ductile flexure-dominant elements, and lower values were found to be suitable for shear controlled elements.

A parametric study was performed by varying EDF for both columns and beams, with EDF values ranging between 0.1 and 0.7. Fig. 8 shows the change of hysteresis loops and the resulting changes in hysteresis areas (energy dissipations) due to the variation in EDF. A total 80 analyses were conducted using 20 earthquake records and two sets of EDF values for columns and beams. The records were synthetic seismic records compatible with the UHS of Ottawa given in the 2010 NBCC. They were adjusted as suggested by Gail Atkinson (2009) [15]. The adjusted records were then amplified and applied in such a way to form hinges in beams and columns, while the structure was approaching collapse. This would amplify the effect of beam/column EDF in the non-linear state of structure. Results are shown in Fig. 9 for the variation of EDF in beams under a single earthquake record, for both a predominantly elastic response with limited yielding, and a non-linear response with extensive yielding. It is evident in this figure that for constant column EDF, structural drift was not sensitive to changes in beam EDF when the structure behaved essentially elastic with inelastic hinges forming in only few structural members without the collapse of any element. On the other hand, when inelastic hinges formed in most structural members, with collapse experienced in some members, the maximum variation of inter-storey (ISD) and full height drift (FHD) was found to be 19% and 17%, respectively.

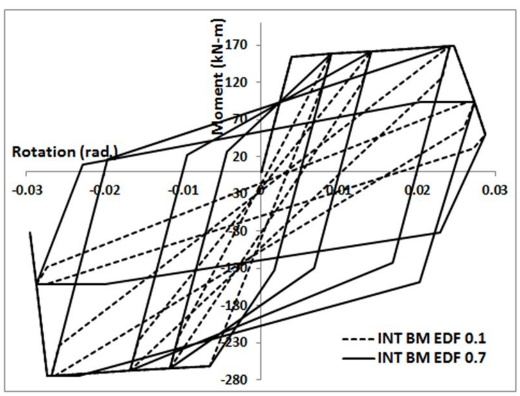
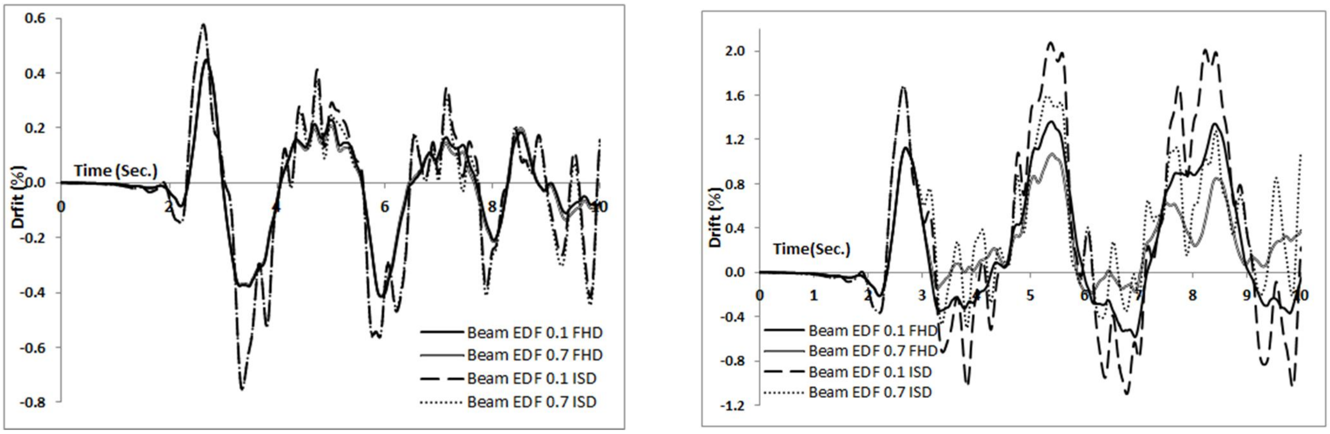


Fig. 8 – Effect of EDF on Interior beam Moment vs Total Chord Rotation hysteresis loop area in PERFORM-3D

When the variation in EDF was introduced to the column, as shown in Fig. 10, the effect was negligible even for high levels of inelasticity. Table 1 lists the details of all the records and the beam EDF variation, along with the results obtained. It is evident that beam EDF had a minor effect on structural drift. For a wide range of Beam EDF, changing between 0.1 and 0.7, resulted in an average change in FHD and ISD of 6.7% and 6.8%, respectively when the column EDF was kept constant. The maximum variation was found to be 29% for the FHD and 25% for ISD. Dissipated inelastic energy of the structure increased on average 25% with a maximum variation of 51% in beam EDF. At the end of the sensitivity analysis it was decided to adopt the experimentally observed values of EDF for beams and columns, as 0.62 up to the full yield of element (including the curved portion beyond the initial yield point) and 0.56 thereafter.



(a) (b)

Fig. 9 –Effect of beam EDF on full-height (FHD) and inter-storey drift (ISD) at second floor level in (a) predominantly linear and (b) non-linear stage of structure.

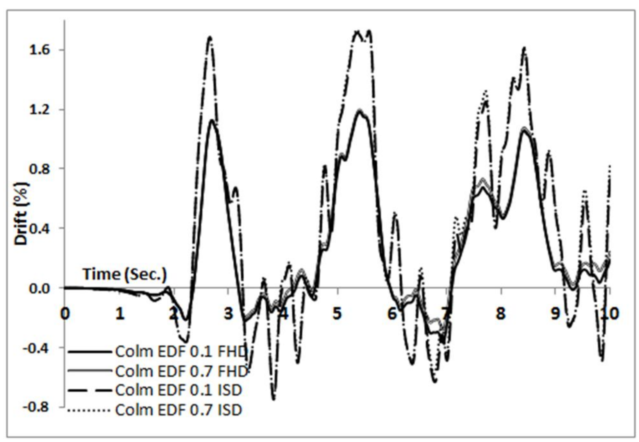


Fig. 10 –Effect of column EDF on full-height (FHD) and inter-storey drift (ISD) in non-linear stage of structure

6. Performance Evaluation of the Selected Structure

Incremental Dynamic Analysis (IDA) was employed using the analytical model described in the preceding sections to illustrate the performance of the 5-storey moderately ductile frame building selected. An NBCC compatible earthquake record was first selected. The record was then incrementally scaled to attain different earthquake intensity levels. Computer software PERFORM-3D was used to conduct nonlinear dynamic analysis. Repeated analysis under different intensity of earthquake records resulted in an IDA curve, providing a relationship between the earthquake intensity expressed in terms of spectral accelerations (S_a) and a structural deformation quantity. Inter-storey drift ratio was used as the structural deformation quantity, denoting performance levels as prescribed in ASCE 41-13. The inter-storey drift ratio of 1% indicated “Immediate Occupancy” performance level, whereas 2% inter-storey drift ratio indicated “Life Safety” performance level. The structural failure was defined either by side-sway collapse (structural instability) or when the rate of change in deformations (the slope of the IDA curve) reached 20% of the initial effective elastic slope. This point on the IDA curve identified “Collapse Prevention” performance level. Fig. 11 illustrates the IDA curve generated with earthquake record 6C1-3 for the 5-storey building selected. It was observed during the IDA analysis that flexural hinges initiated at the 3rd floor level of both the exterior and interior beams just before 1% drift ratio was attained. Beam hinging propagated towards the 4th, 5th and 2nd floors. The first column hinge formed at the first storey level between the life-safety and collapse prevention performance levels, at about 3% drift ratio. This was followed by the hinging of columns at the 3rd floor level. Since the columns were designed such that the 1st and the 2nd floor columns had higher capacities than those at the 3rd to 5th floor levels, column hinges formed at the 3rd to 4th floor columns before the 2nd floor columns. The structure reached collapse level by the failure of beams at the 3rd, 4th, 5th and 2nd floor levels, followed by the collapse of the columns at the 1st floor level. This resulted in dynamic instability at about 3.5% inter-storey drift level.

Table 1 –Effect of beam EDF on drift and energy dissipation in non-linear stage for various seismic records.

Seismic Record	**Duration (Sec.)**	**PGA (g)**	**Accln. Scale Factor**	**Full Height Drift**		**Inter-storey Drift**		**Dissipated Inelastic Energy (% of Total Energy)**	
				Beam EDF 0.1	Beam EDF 0.7	Beam EDF 0.1	Beam EDF 0.7	Beam EDF 0.1	Beam EDF 0.7
6C1-3	5	0.904	5	0.015	0.015	0.026	0.025	35	41
6C1-7	5	0.327	5	0.011	0.011	0.019	0.019	35	40
6C1-12	5	0.645	5	0.015	0.015	0.023	0.023	38	43
6C1-30	5	0.474	5	0.012	0.011	0.026	0.026	33	38
6C1-42	5	0.431	5	0.010	0.009	0.015	0.015	36	43
6C2-3	7	0.438	5	0.018	0.018	0.030	0.030	44	46
6C2-9	7	0.438	6	0.011	0.009	0.018	0.018	27	36
6C2-13	7	0.531	5	0.008	0.007	0.016	0.016	29	38
6C2-15	7	0.298	5	0.014	0.014	0.024	0.024	41	46
6C2-17	7	0.545	5	0.014	0.014	0.019	0.019	28	36
7C1-6	17	0.484	3	0.012	0.012	0.020	0.020	34	39
7C1-18	17	0.270	4	0.010	0.011	0.020	0.016	34	39
7C1-28	17	0.351	4	0.014	0.010	0.024	0.019	34	44
7C1-32	17	0.326	5	0.013	0.013	0.021	0.021	34	45
7C1-36	17	0.393	3	0.015	0.016	0.027	0.027	42	44
7C2-1	20	0.258	4	0.013	0.014	0.020	0.021	31	43
7C2-3	20	0.257	4	0.010	0.009	0.016	0.017	31	42
7C2-7	20	0.203	6	0.015	0.014	0.022	0.017	32	46
7C2-11	20	0.202	6	0.017	0.015	0.034	0.028	32	46
7C2-36	20	0.157	6	0.019	0.014	0.013	0.016	32	49

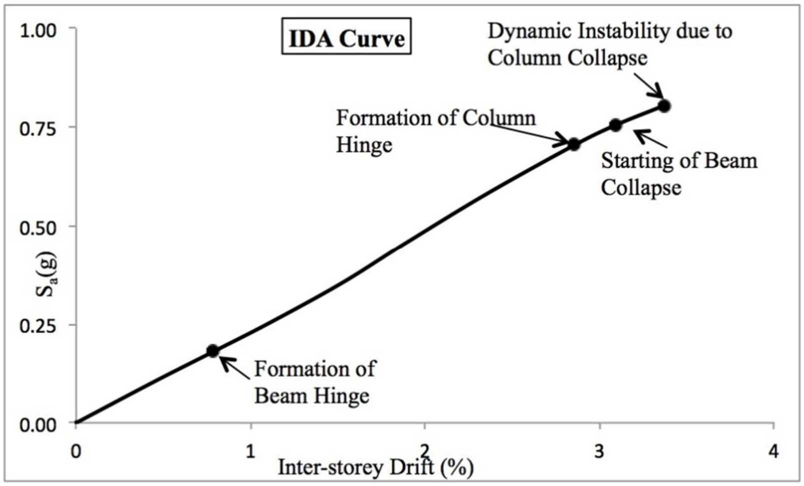


Fig 11 – IDA Curve and the performance of the 5-storey frame building with moderate ductility

6. Summary and Conclusions

Modeling techniques for dynamic inelastic analysis of reinforced concrete frame structures for PERFORM-3D software are explained with sensitivity analyses conducted for selected modelling parameters. The effect of EDF, which defines the degree of stiffness degradation in the hysteretic model, is discussed. It was found that a wide range of variation in this parameter for beam hysteretic modelling between 0.1 and 0.7 resulted in an average change in inter-storey drift ratios of up to 7%. The same level of variation in the column hysteretic models showed negligible effects on drift ratios. Further investigation of the hysteretic model features was conducted by varying the strength decay properties of elements. The provisions of ASCE 41-13 were used to model the onset and rate of strength decay in members designed according to CSA A23.3-04. It was concluded that ASCE 41-13 rotational values can be successfully implemented to model strength decay of members designed according to CSA A23.3-04.

The PERFORM-3D model and the resulting dynamic inelastic time history analyses were verified against additional analytical results obtained by SAP2000 and SeismoStruct software. The same 5-storey frame building modelled and analyzed using PERFORM-3D was also modelled and analyzed under the same earthquake record using these two computer programs. The results indicate good correlations with some differences in maximum deformations computed. It is concluded that PERFORM-3D is reliable software to model RC structures for seismic damage assessment.

The structural model followed the expected path of performance under incrementally increasing earthquake intensity. Performance of component level responses, formation and sequence of plastic hinges and load-displacement hysteretic relationships was assessed throughout the dynamic analysis. It can be concluded that the analytical model developed can be used with PERFORM-3D satisfactorily to predict seismic performance of reinforced concrete frame structures.

7. References

[1] ASCE (2014). Seismic evaluation and retrofit of existing buildings. *American Society of Civil Engineers*. ASCE/SEI 41-13.

[2] NBCC (2010). National building code of Canada 2010. *National Research Council of Canada*.

[3] CSA (2004). Design of concrete structures. CSA standards update service. *Canadian Standards Association*. CSA A23.3-04.

[4] CSI (2013). PERFORM-3D. Version 5.0.1. *Computers and Structures*, Inc. Berkeley, CA.

[5] CSI (2008). ETABS. Nonlinear Version 9.5.0. *Computers and Structures*, Inc. Berkeley, CA.

[6] Reyes, JC., Chopra, AK. (2012). Modal pushover-based scaling of two components of ground motion records for nonlinear RHA of structures. *Earthquake Spectra.* August 2012, Vol. 28, No. 3, pp. 1243-1267

[7] Ghodsi, T., Ruiz, J.A.F. (2010). Pacific earthquake engineering research/seismic safety commission tall building design case study 2. *The Structural Design of Tall and Special Buildings.* Volume 19, Issue 1-2, pages 197–256, 2010

[8] Tuna, Z. (2012). Seismic Performance, Modeling, and Failure Assessment of Reinforced Concrete Shear Wall Buildings. *Ph.D. Thesis.* University of California. Los Angeles.

[9] Liao, W. C. (2010). Performance-based plastic design of earthquake resistant reinforced concrete moment frames. *Ph.D. Thesis.* University of Michigan.

[10] Hopper, M. W. (2009). Analytical models for the nonlinear seismic response of reinforced concrete frames. *M.Sc. Thesis.* Pennsylvania State University.

[11] FEMA (2000). Prestandard and commentary for the seismic rehabilitation of buildings. *Federal Emergency Management Agency*. FEMA 356.

[12] Ozcebe, G., Saatcioglu, M. (1987). Confinement of concrete columns for seismic loading. *ACI Structural Journal*. Volume 84, Issue 4, Pages 308-315.

[13] CSI (2013.a). SAP2000. Ultimate 16.0.0. *Computers and Structures, Inc*. Berkeley, CA.

[14] Seismosoft (2013). SeismoStruct. Version 6.5. *Seismosoft Ltd*.

[15] Atkinson, G.M. (2009). Earthquake time histories compatible with the 2005 National building code of Canada uniform hard spectrum. *Canadian Journal of Civil Engineering*. Volume 36, Number 6, June 2009.

[16] Erberik, M.A., Elnashai, A.S. (2004). Fragility analysis of flat-slab structures. *Engineering structures.* Volume 26, Issue 7, Pages 937-948, June 2004.

[17] Rajeev, P., Tesfamariam, S. (2012). Seismic fragilities for reinforced concrete buildings with consideration of irregularities. *Structural Safety*. Volume 39, Pages 1-13, November 2012.

[18] Kircil, M.S., Polat, Z. (2006). Fragility analysis of mid-rise R/C frame buildings. *Engineering Structures*. Volume 28, Issue 9, July 2006, Pages 1335-1345. July 2006

[19] FEMA (2000). Recommended seismic design criteria for new steel moment-frame buildings. *Federal Emergency Management Agency*. SAC Joint Venture. FEMA 350.