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A CROSS-PLATFORM APPROACH FOR THE SEISMIC PERFORMANCE ASSESSMENT OF A SHEAR CRITICAL RC FRAME

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ABSTRACT

Improvements in computer-based structural analysis have made advanced nonlinear dynamic analysis of structures possible. However, the approach used should consider the fundamental behaviour of the structure under investigation, instead of arbitrarily trusting the results from any advanced software. To illustrate this, a seismic performance assessment was conducted on a shear-critical reinforced concrete frame tested by Duong et al. in 2006. A nonlinear finite element analysis using VecTor2, a program specializing in reinforced concrete structures loaded in shear, was used to inform a fiber model built using OpenSees, which is unable to consider shear behaviour. Nonlinear static and dynamic analyses were performed using the two models respectively. It was found that while both models arrived at similar results for the given seismic demand, the VecTor2 analysis demonstrated that not considering shear behaviour could lead to dangerous results for higher demand.

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ABSTRACT

Improvements in computer-based structural analysis have made advanced nonlinear dynamic analysis of structures possible. However, the approach used should consider the fundamental behaviour of the structure under investigation, instead of arbitrarily trusting the results from any advanced software. To illustrate this, a seismic performance assessment was conducted on a shear-critical reinforced concrete frame tested by Duong et al. in 2006. A nonlinear finite element analysis using VecTor2, a program specializing in reinforced concrete structures loaded in shear, was used to inform a fiber model built using OpenSees, which is unable to consider shear behaviour. Nonlinear static and dynamic analyses were performed using the two models respectively. It was found that while both models arrived at similar results for the given seismic demand, the VecTor2 analysis demonstrated that not considering shear behaviour could lead to dangerous results for higher demand.

Introduction

The field of seismic engineering relies heavily on computer-based structural analysis tools to assess and safely design complex structures. However, as the capabilities and ubiquity of structural analysis software continues to grow, it is anticipated that prescriptive and solution-based approaches will become more prevalent within the engineering community. While this may be appropriate for the design of simple structures, blind trust without afterthought is particularly dangerous when conducting a performance assessment of a structure. Despite advances in our fundamental understanding of dynamics and structures, there is still no silver bullet for easily predicting the true behaviour of any structure during a seismic event. This is of particular importance when performing nonlinear analysis, where depending on the tools used and the skill of the engineer, many different estimates of strength can be obtained. Thus, as the profession moves towards a wider use of nonlinear seismic performance assessment of structures, engineers need to be able to not only use these powerful tools, but should also assess the validity of their modelling approach and assumptions with the fundamental behaviour of the structure in mind.

A reasonable approach to this dilemma is to use multiple analysis procedures to predict the behaviour of a structure in question, with the governing structural behaviour a key factor when selecting which software tools to use. To illustrate this modelling philosophy, a case study was performed on a two-storey reinforced concrete frame tested by Duong et al. at the University of

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Toronto in 2006. A nonlinear seismic performance assessment was conducted by using VecTor2, a nonlinear finite element analysis program developed at the University of Toronto which excels at describing the behaviour of reinforced concrete structures, and OpenSees. Throughout the analysis, the different results from the two programs were consistently validated against the behaviour displayed by published experimental results.

The seismic performance of the frame was assessed using both nonlinear static and nonlinear time history analysis procedures. The 2%/50-year hazard for Vancouver, obtained from the National Building Code of Canada (NBCC - 2010), was used as an appropriate seismic demand. The modified capacity spectrum method from FEMA-440 was used to perform a static analysis, followed by a nonlinear time history analysis using seven ground motions scaled to the hazard spectrum. Acceptance criteria from FEMA-356 were used to draw conclusions about the frame's performance.

Review of Structural Drawings and Modelling Methodology

The specimen tested by Duong was a two-storey reinforced concrete frame which is a scaled model of a 7-storey cement plant constructed in 1999 [1]. A detailed assessment of the cement plant identified key deficiencies in the design, namely insufficient anchorage of reinforcement in the joints, and the shear-critical nature of the beams. The test specimen constructed by Duong focused on the shear-critical nature of the structure, and its anchorage deficiencies were corrected. The complete details of Duong's frame can be found in Fig. 1.

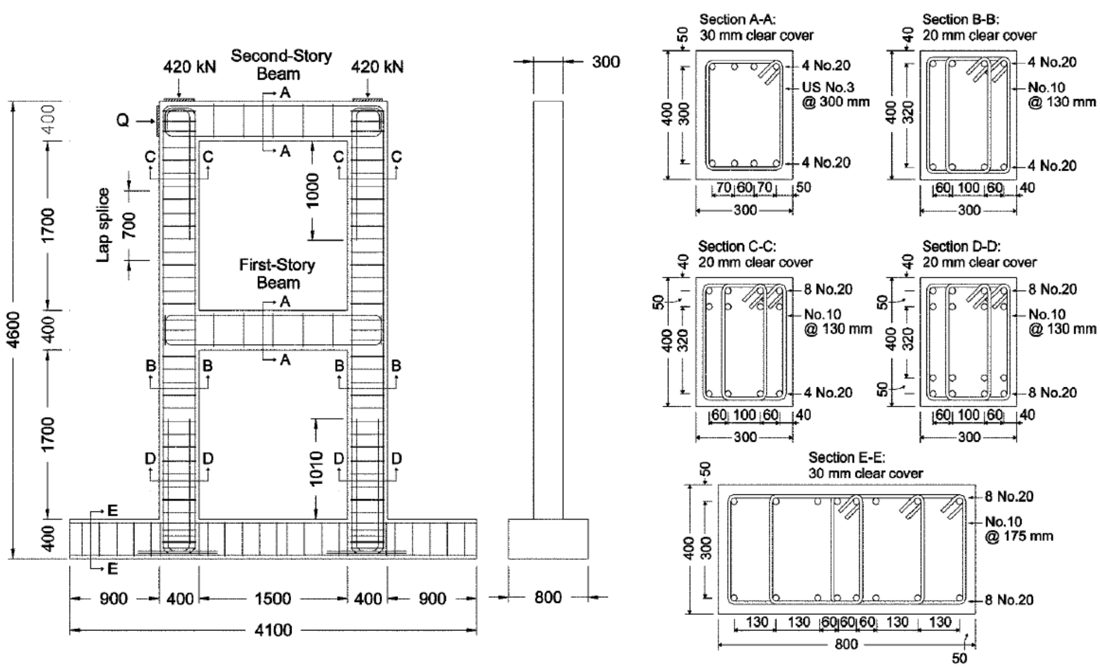

Figure 1. Reinforced concrete frame details - elevation and sections [1].

As shown in Fig.1, two vertical point loads of 420 kN each were applied to the top of the frame to simulate the weight of the storeys above. A horizontal load Q was then applied to the frame at the elevation of the second storey beams. The frame was attached to the strong floor below using prestressed anchor bolts positioned between the columns. During testing, a single reverse cyclic load cycle was applied at the top storey until the frame was at the onset of failure (stage 1), and then the frame was tested to failure following a repair of the damaged beams using an FRP wrap (stage 2). As intended in the specimen design, a shear failure in the first-floor beam governed the frame's behaviour. At the conclusion of the loading (prior to repairing the structure), flexural cracks were observed in the columns, particularly at the connection between the base and columns. Large shear cracks were also observed in the beams.

Table 1. Duong frame reinforcing steel properties

Type	(mm) Diameter	(mm^2) Area	(MPa) f_y	(MPa) f_u	(10^{-3}) ε_y	(10^{-3}) ε_{sh}	(MPa) E_s
10M	10	100	455	583	2.38	22.8	192 400
20M	20	300	447	603	2.25	17.1	198 400
US no. 3	9.5	71	506	615	2.41	28.3	210 000

Table 1 contains a summary of the material properties of the reinforcing steel. At the time of testing, the concrete strength was measured to be 43MPa. The maximum specified aggregate size of the mix was 10 mm.

Modelling Methodology

The numerical modelling was performed using both VecTor2 and OpenSees for both cross-validation and to overcome limitations in the respective software. OpenSees is a widely-used open-source platform which can perform both static and dynamic analyses of large structures. However, the distributed plasticity fiber-section elements used in OpenSees are unable to capture the nuances of shear behaviour in reinforced concrete. As the Duong frames was governed by its shear-critical beams, simply using OpenSees to model its response would potentially omit a crucial part of the structure's behaviour.

A nonlinear finite element analysis program, VecTor2, was used to verify and interpret the results from OpenSees. VecTor2, developed at the University of Toronto, makes use of the Disturbed Stress Field Model (DSFM) as its theoretical foundation [2]. Based on the Modified Compression Field Theory (MCFT) [3], the DSFM is specifically formulated to capture the behaviour of reinforced concrete structures in shear, and considers tension stiffening, compression softening, and crack slip. Because the DSFM can predict the full load-deformation of a reinforced concrete element in shear, information such as stresses, strains, displacements and crack widths can be predicted at any load stage. The VecTor2 model was useful for detailed analysis of the frame under monotonic and quasi-static reversed cyclic loading. However, the high computational demands associated with nonlinear time history analysis made VecTor2 impractical for performing dynamic analysis.

Because neither software could perform a full series of static and dynamic analyses while capturing the shear-critical nature of the structure, the results from both were used in tandem to obtain a clear understanding of the fundamental behaviour. Leading up to the nonlinear time history analysis performed in OpenSees, cross-verification with VecTor2 was performed on the basis of a pushover analysis to examine the effect of shear on the global structural response. In using this approach, the authors gained valuable information on the inelastic behaviour of the structure, and were therefore able to better explain and justify the final seismic performance assessment as produced by the OpenSees analysis.

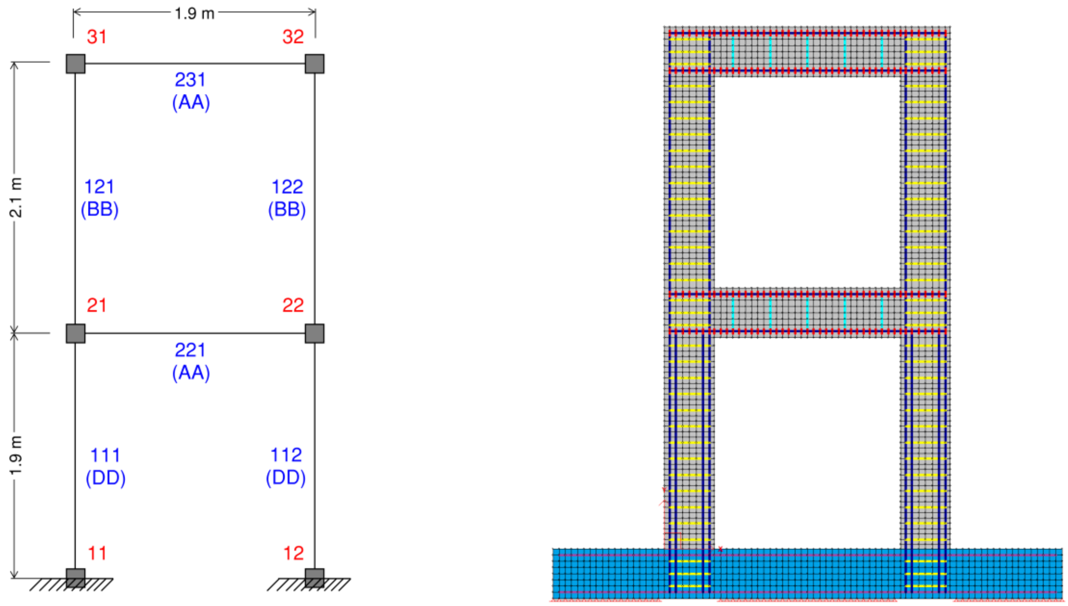

Figure 2. OpenSees frame model (left) and VecTor2 finite element model (right).

OpenSees Modelling Assumptions

Four models of varying complexity were made in OpenSees, which were used to evaluate the effect of various modelling assumptions. The simplest model was based on the "Shear Building" assumption where only lateral deformations are allowed - these results were verified by hand calculations. Once this base model had been validated, more complexity was introduced. In the "Elastic Frame" model all deformation constraints were removed, and the effect of the restraint conditions in the model was checked; as expected, the fixed condition was found to be much stiffer. The design intent was for the frame to remain fixed at the base, but the real behaviour would lie somewhere between the idealized fixed and pinned conditions. The "Nonlinear Frame" model was the most structurally complex, as all the deformations constraints were removed, the base was kept fixed, and all the beam and column members were modelled with a distributed plasticity approach using nonlinear fiber sections based on the details shown in Fig. 1. The nodes, elements and sections are illustrated schematically in Fig. 2; this model was used in both the pushover and nonlinear time history analyses. It should be noted that for all models, the reinforcement layout was simplified so reinforcing bars which did not run the full height of each column were ignored.

VecTor2 Modelling Assumptions

Fig 2. also shows the finite element model created in VecTor2 to model the Duong frame. Reinforcing steel was modelled using discrete bars throughout the frame, and link elements were used to describe the deterioration of bond during cyclic loading. The Palermo model in the software was used to model the hysteretic response of the concrete. To replicate the actual connection of the frame to the floor, the full base was included in the model, and the base was pinned along its length to simulate the anchor bolts in the experiment. For comparison purposes, the reinforcement layout was simplified to be the same as the OpenSees model. It was found that this simplification did not cause any significant changes in the model behaviour.

Understanding the Inelastic Behaviour of the Duong Frame

Experimental Validation of Analysis Software

Prior to conducting the seismic performance assessment of the Duong frame, the specimen was modelled using the two software to assess how well they were able to describe the structure's behaviour. The results from the reverse cyclic analyses are shown in Fig. 3, which also contains the observed experimental load-displacement behaviour. Both analyses matched the experimental results quite well with regards to peak load and displacement. While the peak load was overestimated during the forward cycle, both software showed excellent agreement with each other, and the peak load in the backwards direction was well-predicted. The analysis results from VecTor2 capture the load-reversal behaviour quite well. However, the OpenSees analysis predicts a relatively generous amount of energy dissipation, and fails to capture the noticeable pinching effect - a ductile response that will be further discussed.

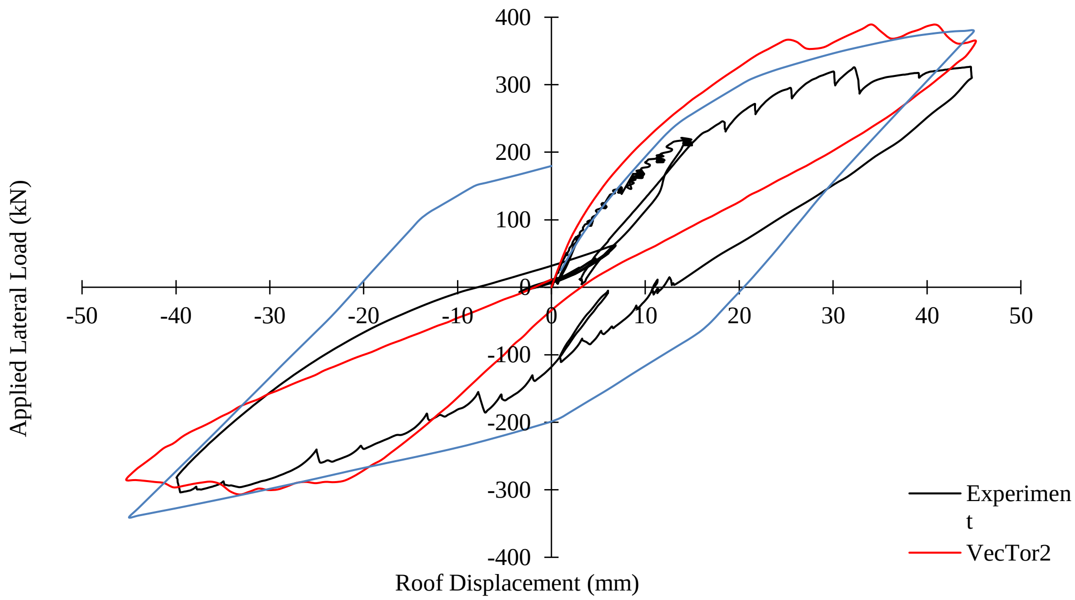

Figure 3. Predicted and actual behaviour of the Duong frame.

Modal Analysis

To obtain the period, mode shapes and equivalent static force distribution for the Duong frame, modal analysis was performed using the four models in OpenSees. It should be noted that because of the small size of the frame, an initial analysis using only the self-weight of the structure resulted in a first mode period of 0.0385 s. This small period makes using the procedures to evaluate its seismic performance difficult. It was decided that a reasonable simplification to make this study purposeful was to consider the 420 kN point loads as additional lumped mass, and assign them to the frame nodes using a tributary height concept. The new storey masses (52 741 kg and 36 086 kg of the first and second storeys respectively) were used to obtain the results shown in Table 2, which contains values of the first mode period, mode shape and equivalent static force distribution. Since the fiber section model can both capture nonlinearity and showed reasonable agreement with the other simple elastic models, it was deemed acceptable to form the basis of further analysis.

Table 2. Summary of first mode results from the different OpenSees models.

Model	Description	(Sec) Period	(Roof Norm.) Eigenvector	(Mass Norm.) Force Vector
Shear Building	Lateral DOFs	0.144	{0.549, 1.000}	{0.442, 0.558}
Elastic Frame	Fixed Base	0.157	{0.492, 1.000}	{0.414, 0.585}
Elastic Frame	Pinned Base	0.271	{0.804, 1.000}	{0.537, 0.463}
Nonlinear Frame	**Fiber Sections**	**0.201**	**{0.436, 1.000}**	**{0.386, 0.614}**

Pushover Analysis

In preparation for the nonlinear static analysis of the frame, a pushover analysis was performed using both OpenSees and VecTor2 to better understand the characteristic behaviour of the frame. The mass–normalized force distribution of 0.386 and 0.614 at the 1st floor and roof level respectively, obtained from the nonlinear frame analysis, were used. It is important to note that the pushover analysis was run in displacement control – by scaling the force vector so that its components sum to 1.0, the pseudo–time scale factor at each load step is equivalent to the base shear. The results of the pushover analyses are shown in Fig. 5.

OpenSees Static Pushover Results

A static pushover analysis was run on the OpenSees model which utilized fiber sections, primarily because it has the highest level of material complexity and potential for calibration, and secondly since the validity of the model had been established in the modal exercise. Key values throughout the analysis are summarized in Table 3.

Table 3. OpenSees pushover behaviour.

Point	(mm) Roof Disp.	(kN) Base Shear	(%) Roof Drift
Nonlinear Start	5.8	190	0.15
Yield Plateau	34.4	477	0.86
Failure	213.5	510	5.33

During loading, hinging took place at the base of both columns on the first floor (evidenced by the moment-curvature relationship shown in Fig. 4), and structural failure occurred when a third plastic hinge formed at the top left corner of the frame. The beams behave in a ductile manner, with the maximum moment sustained following yield. Yet during the actual experiment, it was noted that the beams failed in a brittle manner - this behaviour is not captured due to the limited ability of fiber sections to consider shear behaviour. Due to the significant ductility of the frame, and lack of strength degradation, this behaviour predicted by the OpenSees model is displacement-controlled according to FEMA-356 [4].

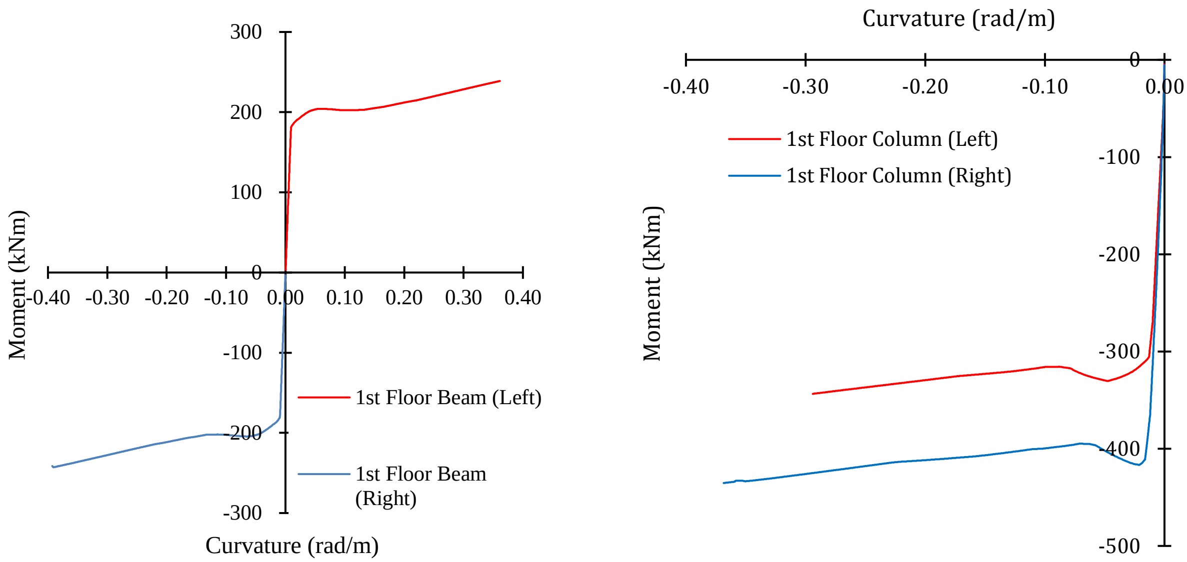

Figure 4. First floor beam (left) and column base (right) hinging in OpenSees pushover analysis.

VecTor 2

To provide an alternative interpretation to the results from OpenSees, a pushover analysis was also performed in VecTor2. The displaced shape and crack pattern obtained from the analysis are shown in Fig. 5. Failure occurred due to a shear failure in the first storey beam, followed by yielding of the longitudinal steel in the base of first storey column and subsequent crushing of the concrete there. This progression of events was a more realistic description of the actual frame. The loss in strength following shear failure would classify the frame as being forced-controlled according to FEMA-356 [4].

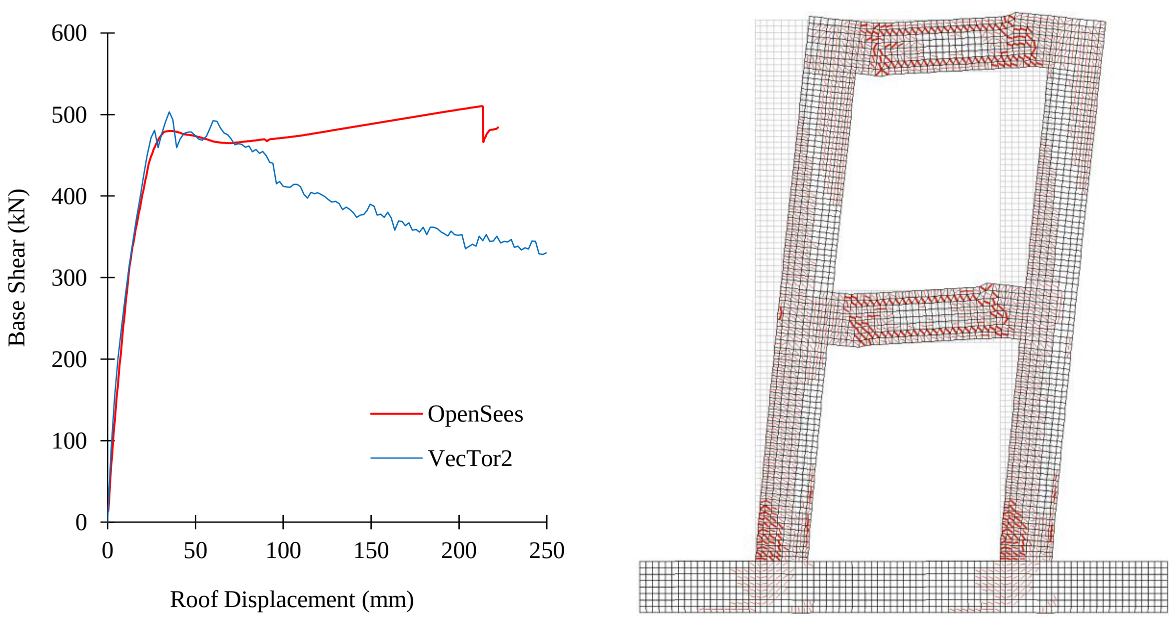

Figure 5. Pushover results (left) and displaced shape and crack pattern in VecTor2 (right).

Discussion

Despite the conflicting failure modes reported by the two analyses, both pushover curves achieve similar peak loads, with extensive roof displacements predicted to occur following yielding of the steel at the base of the first storey columns. A key difference is the loss in strength described in the VecTor2 due to the shear failure in the beams. Since VecTor2 correctly considered the shear-critical nature of the structure, it was decided that the pushover curve from VecTor2 would be used when performing the nonlinear static analysis of the structure.

Evaluation of Seismic Performance

To assess the seismic performance of the Duong frame, nonlinear static and nonlinear dynamic procedures were used. The modified capacity spectrum method from FEMA-440 was used in conjunction with the VecTor2 pushover curve [5]. Nonlinear time history analysis was performed using seven ground motions applied to the frame model in OpenSees, which despite its inability to capture shear failure, was still considered to provide a reasonable representation of the structure. For both analyses, the design response spectrum corresponding to the 2%/50-year hazard in Vancouver was used, which is an upper bound for the demand expected to be applied to a Canadian structure. Rayleigh damping was considered in the dynamic analysis, assuming 5% damping for both modes 1 and 2.

Modified Capacity Spectrum Method

The pushover curve obtained from VecTor2 was converted to into ADRS form using the base shear effective modal mass and influence factors corresponding to the first mode. Here, M_1* was found to be 75 690 kg (85.2% of the total mass), and the influence factor, Γ_1, was found to be 1.28. The iteration process, shown visually in Fig. 6 and tabulated in Table 4, required three iterations to converge at the spectral displacement of 16 mm, and spectral acceleration of 0.535 g.

Table 4. Modified capacity spectrum iteration information.

	(g)	(m)	g	(m)	(sec)			(%)	(sec)		(%)	(%)
Iteration	a_{pi}	d_{pi}	a_y	d_y	T_0	α	μ	β_{eff}	T_{eff}	B	Δ_d	Δ_a
Start	0.434	0.011	0.250	0.0030	0.688	0.276	3.667	18.99	1.171	1.506	-	-
1	0.590	0.019	0.265	0.0033	0.701	0.253	5.848	20.55	1.339	1.552	75.5	35.9
2	0.550	0.017	0.255	0.0035	0.736	0.304	4.800	20.22	1.306	1.542	12.95	6.78
3	**0.535**	**0.016**	-	-	-	-	-	-	-	-	4.76	2.73

The corresponding base shear and roof drift for the computed spectral displacement are 400 kN and 20.5 mm respectively. As inter-storey drifts and beam rotations are useful metrics for quantifying the performance of the structure, the displaced shape corresponding to the base shear was obtained from the VecTor2 analysis. Fig. 6 shows the displaced shape, idealized as a frame with plastic hinges. The beam rotations were found to be 0.0012 radians, and inter-storey drifts of 0.643% and 0.657% were obtained for the first and second storeys respectively.

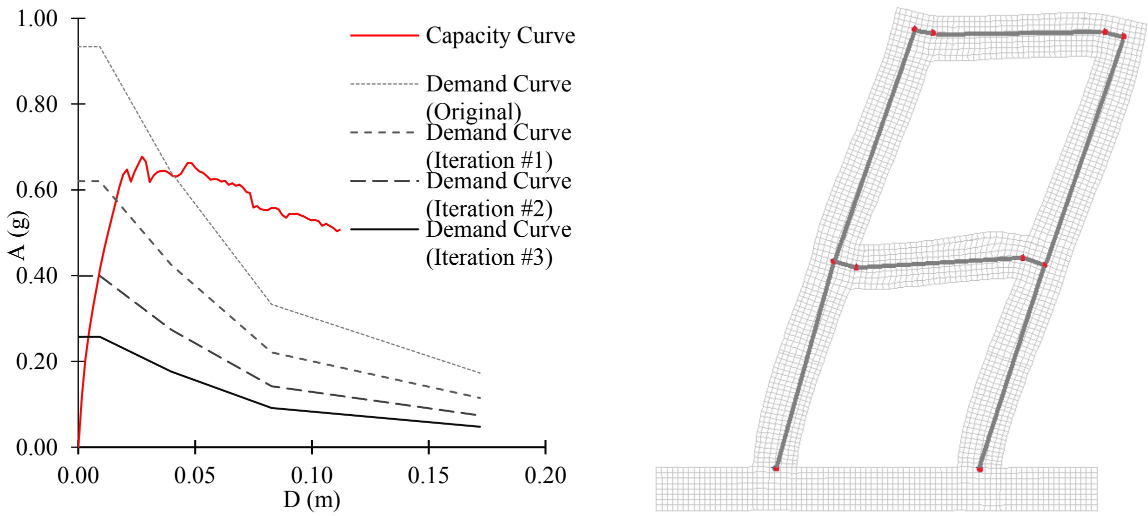

Figure 6. Results from modified capacity spectrum method (left) and displaced shape at performance point (right) (magnification factor of 50x).

Nonlinear Time History Analysis

VecTor2 is not capable of performing a nonlinear time history analysis, therefore the OpenSees fiber-section model was used. While this model does not capture the final shear failure in the first storey beam, its otherwise excellent agreement with the VecTor2 pushover results and experimental data is justification for its use in representing the structural behaviour in this exercise.

Selected Ground Motions

The structure was subjected to seven different ground motions (Table 5) that were scaled to the same hazard spectrum as was specified in the modified capacity spectrum section. The ground motions were taken from the PEER database, scaled up to match the Vancouver hazard spectrum for the period range of 0.2T-1.5T.

Table 5. Ground motions used for nonlinear time history analysis.

GM	Record #	Scale Factor	EQ Name	Year	Magnitude	dt	# pts
1	452	4.6	Morgan Hill	1984	6.19	0.005	8000
2	962	3.2	Northridge-01	1994	6.69	0.01	4000
3	964	3.2	Northridge-01	1994	6.69	0.01	3980
4	976	4.3	Northridge-01	1994	6.69	0.01	3500
5	1015	3.9	Northridge-01	1994	6.69	0.02	2000
6	1094	4.1	Northridge-01	1994	6.69	0.02	3650
7	3863	1.6	Chi-Chi	1999	6.30	0.004	10000

The maximum roof displacements, inter-storey drifts, base shears and first storey beam joint rotations were determined for each ground motion. These peaks were averaged using the standard square root of the sum of the squares (SRSS) approach.

Comparison and Discussion

The averaged peak values from the time history analysis are compared to the results from the modified capacity spectrum method in Table 6. Both methods show good agreement, but the nonlinear dynamic analysis predicts a more ductile response as evidenced by the higher beam rotations and roof displacements. This can be attributed to the deterioration of the concrete and yielding of the steel throughout the applied dynamic loading scenario. Both analyses report maximum displacements which correspond to the pre-peak of the pushover curve, indicating that global yielding of the structure is not anticipated given the applied demand.

Table 6. Comparison of results from nonlinear analysis methods.

Analysis Method	ISD (1st floor)	ISD (2nd floor)	First Floor Beam Rotation	Base Shear	Roof Displacement
Capacity Spectrum Method	0.643%	0.657%	0.0012 rad	400 kN	20.5 mm
Nonlinear Time History	0.668%	0.960%	0.0015 rad	349 kN	31.2 mm

Conclusion

To illustrate the need to select appropriate tools when performing a nonlinear seismic performance assessment of structures, a shear-critical reinforced concrete frame was modelled using both OpenSees and VecTor2. Both nonlinear static and nonlinear dynamic analyses were performed, with similar results obtained from both software. However, a comparison of the pushover curves showed discrepancies in the predicted behaviour of the frame, with OpenSees failing to capture the shear failure in the frame's beams. Only through the process of cross-validation was it established that these modelling short-comings were not critical; a higher seismic demand or structural configuration would reveal the dangers of neglecting the shear behaviour.

While a cross-platform approach allows the structural response to be understood with more confidence, comprehensive modelling software which can perform nonlinear dynamic analysis while considering key behaviour – such as shear failures – does not exist. Until analysis techniques are developed which can account for these effects in full, engineers should continue to exercise their judgement and avoid blindly trusting software outputs, relying instead on first principles and sound experience wherever possible.

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