

SEISMIC ANALYSIS OF BUILDINGS TO RECORDED MOTIONS: MODELING AND SOFTWARE CHALLENGES

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Abstract

This investigation focused on developing an improved understanding of challenges associated with computation of nonlinear response of three-dimensional building to recorded ground motions, and if the inertial base shear is an accurate indicator of the true base shear. For this purpose, three-dimensional models of two buildings – one reinforced-concrete building and one steel building – are developed in *OpenSees* and *Perform3D*. The analysis of these models included pushover analysis for lateral force distribution proportional to the first mode in each of the two principle directions, and RHA to compute response for 30 ground motions recorded during past earthquakes. It was found that modeling assumptions as well as different software may lead to significantly different pushover curves: concentrated plasticity model leads to lower strength, early initiation of yielding, and post yielding strength loss in pushover curves compared to spread plasticity model, strength loss model for beams/columns leads to significant post yielding strength loss in the pushover curve, and differences in solution schemes and convergence criteria available in different software programs also affect the pushover curves. It was also found that there prediction of median peak response from different software can differ from 10% to 40%. Finally, the median inertial base shear exceeds the true base shear by 10% to 20% with the value exceeding by as much as 50% for individual earthquake and even a small time delay between different recording channels may lead to significant error in the inertial base shear. Therefore, inertial base shear should be used with caution as an estimate of the true base shear.

Introduction

Buildings are typically instrumented with accelerometers at selected number of floors: low-rise buildings (1 to 3 stories) at every floor; and mid- and high-rise buildings at base, roof, and a few intermediate floors. The raw (or uncorrected) acceleration recorded during earthquakes from these accelerometers are processed using well-established procedures to obtain base-line corrected (or processed) accelerations, velocity, and displacements. The processed floor accelerations and displacements may be used to estimate additional engineering demand parameters such as inter-story drift ratio defined as the differential displacement between two adjacent floors divided by the story height, and base shear defined as the summation of floor inertial forces above the base; the floor inertial forces are computed as the product of floor acceleration and floor mass. The engineering demand parameters thus estimated from recorded

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motions of buildings may be compared to those computed from various analytical procedures, such as nonlinear static pushover and nonlinear response history analysis, to evaluate the accuracy of these analytical procedures. These parameters may also be compared with limiting values to check if the building suffered damage during an earthquake and may need detailed inspection/evaluation.

For buildings with limited number of instrumented floors, estimation of various engineering demand parameters requires that the motions at non-instrumented floors be interpolated from those available at the instrumented floors. Typically, a piece-wise cubic polynomial interpolation procedure is used for conventional buildings (Naeim, 1997; De la Llera and Chopra, 1998; Goel, 2005, 2007; Limongelli, 2003) and a combination of cubic-linear interpolation is recommended for base-isolated buildings (Naeim, et al., 2004). It is generally believed that such interpolation procedures provide reasonable estimates of motions at non-instrumented floors (Naeim, 1997; Naeim et al., 2004; De la Llera and Chopra, 1998).

A recent study by Goel (2008) re-examined the adequacy of the traditionally used cubic polynomial interpolation procedure. It was found that results from the cubic polynomial interpolation procedure are sensitive to location of instrumented floors. While the cubic polynomial interpolation procedure may provide good estimate of floor displacements with proper selection of instrumented floors, this procedure may not accurately predict inter-story drifts and floor accelerations. This finding was also confirmed by Bernal (2007).

Another investigation by Goel and Chadwell (2007) compared the base shear estimated from motions interpolated using the traditional cubic polynomial interpolation procedure with the base shear capacity estimated from nonlinear pushover analysis of buildings. It was found that the base shear estimated from interpolated motions significantly exceeded the base shear capacity for several buildings. However, post earthquake inspection of these buildings did not reveal significant damage. This indicates that such base shear estimates may be questionable and possibly unreliable.

The preceding discussion clearly indicates that there is a need to comprehensively re-evaluate existing interpolation procedures. In particular, it is desirable to establish the level of accuracy that can be achieved in estimates of floor displacements and floor accelerations. Furthermore, it is necessary to examine if the base shear estimated from inertial floor forces is an accurate estimate of the “true” base shear which is defined as summation of shear forces in all columns at the base.

The aforementioned evaluation of interpolation procedures requires that “true” motions of buildings be available at each floor level. Since buildings are rarely instrumented at all floors and thus complete set of recorded responses that is needed for evaluating interpolation procedures is not readily available, response of buildings due to recorded ground motions computed from response history analysis (RHA) offers a viable alternative to recorded motions. However, there are several modeling and software challenges in implementing the RHA for buildings that are expected to be deformed beyond the linear elastic limit during strong ground shaking.

The primary objective of this investigation is to develop an improved understanding of challenges associated with computation of nonlinear response of three-dimensional building to recorded ground motions. Another objective is to evaluate the accuracy of the inertial base shear as an indicator of the true base shear using the results from the RHA. For this purpose, nonlinear response – floor displacements, floor accelerations, and base shear – of two buildings – 20-Story Reinforced Concrete Hotel in North Hollywood, and 19-Story Steel Office Building in Los Angeles – were computed from RHA for 30 ground motions recorded during past earthquakes using two different computer program – *OpenSees* and *Perform3D*. Also computed were the pushover curves of these buildings included pushover analysis for lateral force distribution proportional to the first mode in each of the two principal directions. First, challenges associated in computation of nonlinear response from the two computer programs are documented. Next differences in peak responses from the two programs are examined for effects of modeling and software. Finally, peak values of inertial and true base shears are compared to understand if the inertial base shear can provide accurate estimate of true base shear.

Selected Buildings and Ground Motions

Two buildings – 20-Story Hotel in North Hollywood and 19 Story Office Building in Los Angeles – are selected in this investigation (Table 1). This buildings are selected as representative of instrumented mid- to high-rise reinforced-concrete and steel buildings in California.

Table 1. Five concrete buildings selected.

Buildings name	CSMIP Station	Number of Stories	Structural System
Los Angeles – 19-Story Office Building	24643	19/4	Steel Concentric Brace Frame (Transverse) and Moment Frames (Longitudinal)
North Hollywood – 20-Story Hotel	24464	20/1	Concrete Moment Frames

A suite of 30 ground motions have been selected in this investigation (Table 2). Each ground motion consists of a pair of two horizontal components of ground motion recorded during indicated earthquake. These earthquakes are selected for a wide range of parameters: proximity to the fault, magnitude, peak ground accelerations and velocities. These ground motions were not selected to match any design spectrum but to ensure that they will induce different levels of inelastic behavior in the selected buildings: selected buildings are expected to remain within the linear elastic range for a few earthquakes where as these buildings are expected to be deformed well into the nonlinear range, and possibly collapse, during other earthquakes.

Analytical Models

The three-dimensional analytical models of the selected buildings were developed using the structural analysis software Open System for Earthquakes Engineering Simulation (*OpenSees*) (McKenna and Fenves, 2001) and *Perform3D* (CSI, 2006). Following is a description of the modeling procedures.

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Table 2. Selected ground motions.

Serial No.	Station Name	Earthquake	Mag.	Epic. Dist. (km)	PGA (H1, H2, V) - g	PGV (H1, H2, V) - cm/s
1	Parkfield-Fault Zone 1	Parkfield, September 28, 2004	6.0	9	0.59, 0.82, 0.26	63, 81, 10
2	Parkfield-Fault Zone 14	Parkfield, September 28, 2004	6.0	12	1.31, 0.54, 0.56	83, 42, 23
3	Templeton-1-story Hospital GF	San Simeon, December 22, 2003	6.5	38	0.42, 0.46, 0.26	33, 27, 16
4	Amboy	Hector Mine, October 16, 1999	7.1	48	0.15, 0.18, 0.13	20, 27, 12
5	Taiwan-CHY028	Chi-Chi, September 21, 1999	7.6	7 to fault	0.82, 0.65, 0.34	67, 72, 36
6	Taiwan-TCU129	Chi-Chi, September 21, 1999	7.6	1 to fault	0.63, 1.01, 0.34	36, 60, 35
7	Taiwan-TCU068	Chi-Chi, September 21, 1999	7.6	1 to fault	0.46, 0.56, 0.49	176, 263, 187
8	Taiwan-CHY028	Chi-Chi, September 21, 1999	7.6	10 to fault	0.42, 1.16, 0.34	46, 115, 25
9	Sylmar-County Hospital Lot	Northridge, January 17, 1994	6.7	16	0.59, 0.83, 0.53	77, 129, 19
10	Newhall-LA County Fire Station	Northridge, January 17, 1994	6.7	20	0.57, 0.58, 0.54	75, 95, 31
11	Los Angeles-Rinaldi Rec. Station FF	Northridge, January 17, 1994	6.7	9	0.47, 0.83, 0.83	166, 73, 51
12	Santa Monica-City Hall Grounds	Northridge, January 17, 1994	6.7	23	0.88, 0.37, 0.23	42, 25, 14
13	Lucerne Valley	Landers, June 28, 1992	7.4	1 to fault	0.72, 0.78, 0.82	98, 32, 46
14	Yermo-Fire Station	Landers, June 28, 1992	7.4	84	0.15, 0.24, 0.13	29, 51, 13
15	Big Bear Lake-Civic Center Grounds	Big Bear, June 28, 1992	6.5	11	0.48, 0.55, 0.19	28, 34, 11
16	Petrolia-Fire Station	Cape Mendocino, April 26, 1992	6.6	35	0.59, 0.43, 0.15	61, 30, 13
17	Petrolia-Fire Station	Petrolia, April 25, 1992	7.1	8	0.65, 0.58, 0.16	90, 48, 21
18	Cape Medocino	Petrolia, April 25, 1992	7.1	11	1.04, 1.50, 0.75	41, 126, 60
19	Rio Dell-Hwy101/Painter Street Overpass FF	Petrolia, April 25, 1992	7.1	18	0.39, 0.55, 0.20	45, 43, 10
20	Corralitos-Eureka Canyon Road	Loma Prieta, October 17, 1989	7.0	7	0.48, 0.63, 0.44	48, 55, 19
21	Los Gatos-Linahan Dam Left Abutment	Loma Prieta, October 17, 1989	7.0	19	0.40, 0.44, 0.13	95, 84, 26
22	Saratoga-Aloha Ave.	Loma Prieta, October 17, 1989	7.0	4	0.32, 0.49, 0.35	44, 41, 26
23	El Centro-Imperial County Center Grounds	Superstition Hills, November 24, 1987	6.6	36	0.26, 0.34, 0.12	41, 47, 8
24	Los Angeles-Obregon Park	Whittier, October 1, 1987	6.1	10	0.43, 0.41, 0.13	22, 13, 5
25	Chalfant-Zack Ranch	Chafant Valley, July 21, 1986	6.4	14	0.40, 0.44, 0.30	43, 36, 12
26	El Centro-Array #6	Imperial Valley, October 15, 1979	6.6	1 to fault	0.43, 0.37, 0.17	109, 63, 56
27	El Centro-Array #7	Imperial Valley, October 15, 1979	6.6	1 to fault	0.45, 0.33, 0.50	108, 45, 26
28	El Centro-Imperial County Center Grounds	Imperial Valley, October 15, 1979	6.6	28	0.24, 0.21, 0.24	64, 36, 17
29	El Centro-Hwy8/Meloland Overpass FF	Imperial Valley, October 15, 1979	6.6	19	0.31, 0.29, 0.23	72, 91, 29
30	El Centro-Irrigation District	El Centro, May 18, 1940	6.9	17	0.34, 0.21, 0.21	33, 37, 11

20-Story Hotel in North Hollywood

OpenSees Model

The beams and columns for the North Hollywood Hotel were modeled with *beamWithHinges* element in *OpenSees*. This element used fiber section containing confined concrete, unconfined concrete, and steel reinforcing bars. The stress-strain behavior of concrete, both confined and unconfined, was modeled with several different available concrete materials in *OpenSees*. The first concrete material model used in this investigation is *Concrete01* (Figure 1a) which has residual strength after crushing strain. The second model is a modified version of the *Concrete01* model (Figure 1b) which has no residual strength after reaching crushing strain. Further details of these two material models are available in Mander et al. (1988) and Karson and Jirsa (1969). The third concrete model is *Concrete04* which is similar to the modified *Concrete01* model but uses slightly different parameters (see Popovics, 1973 for details). The crushing strain of the unconfined concrete was selected to be equal to 0.004 and that for confined concrete was selected to be that corresponding to the rupture of confining steel using the well established Mander model (Mander et al., 1988). The stress-strain behavior of steel was modeled with *ReinforcingSteel* material in *OpenSees* (Figure 1c). The strength of concrete and steel was selected based on the values specified in the structural drawings. The P-Delta effects were included in the pushover analysis by applying the gravity loads prior to pushover analysis or RHA.

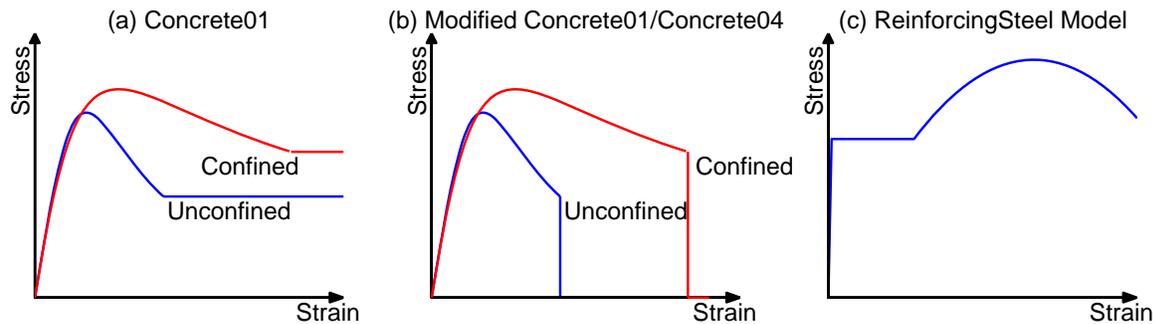


Figure 1. *OpenSees* material models: (a) *Concrete01* model; (b) Modified *Concrete01* and *Concrete04* models; and (c) *ReinforcingSteel* model.

Perfrom3D Model

In the *Perfrom3D* model, beams were modeled with FEMA Concrete Beam with strength loss and unsymmetrical section strength, columns were modeled with FEMA Concrete Column with strength loss and symmetrical section strength, and shear walls were modeled with linear elastic column elements. The FEMA Beam element requires moment-plastic-rotation relationship of Figure 2a. The yield moment of the beam section needed to define the FEMA force-deformation behavior is computed from section moment-curvature analysis using computer program *XTRACT* (TRC, 2008).

The plastic rotation values and the residual strength needed for the FEMA Concrete Beam model in *Perfrom3D* are selected as per FEMA-356 (ASCE, 2000) recommendations: plastic rotations are selected as 0.02 for point U and 0.03 for point X, and the residual strength

for points R and X are selected as 20% of the yield moment (Figure 2a). The plastic rotation value for point R is selected as 0.022 to model gradual strength loss between points U and R.

The FEMA Concrete Column with strength loss element requires moment-plastic-rotation behavior of Figure (2a), P-M interaction diagram for bending about axis-2 and axis-3 (Figure 2b), and M-M interaction diagram between moments about axis-2 and axis-3 (Figure 2c). The yield moment needed to define the force-deformation behavior (Figure 2a) was obtained from *XTRACT* moment-curvature analyses of column sections about axis-2 and axis-3. Similarly, the parameters needed to define P-M interaction diagrams about axis-2 and axis-3 (Figure 2b) were estimated from *XTRACT* P-M interaction analyses of columns sections. The shapes of the P-M interaction diagrams (Figure 2b) and M-M interaction diagram (Figure 2c) were defined using default values of various exponents in *Perform3D*. The material models used for columns in *XTRACT* analysis were the same as for beams.

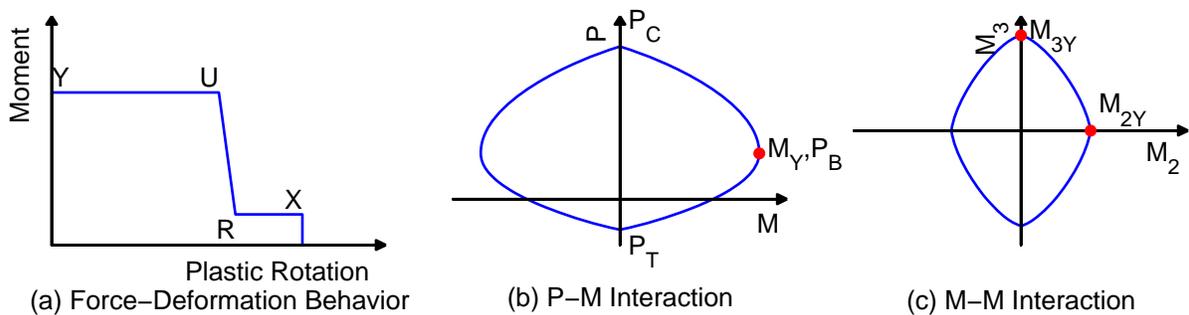


Figure 2. FEMA concrete beam/column element in *Perform3D*: (a) Force-deformation behavior of beam or column, (b) P-M interaction diagram for column; and (c) M-M interaction diagram for column.

Similar to the beams, the plastic rotation values and the residual strength needed for the FEMA Concrete Column model in *Perform3D* are selected as per FEMA-356 recommendations: plastic rotations are selected as 0.02 for point U and 0.03 for point X, and the residual strength for points R and X are selected as 20% of the yield moment (Figure 2a). The plastic rotation value for point R is selected as 0.022 to model gradual strength loss between points U and R.

19-Story Office Building in Los Angeles

OpenSees Model

In *OpenSees* model of the Los Angeles building, the beams and columns were modeled with *nonlinearBeamColumn* elements, and braces were modeled with nonlinear *truss* elements between 1st floor and roof; and beams, columns, and shear walls were modeled with linear *elasticBeamColumn* elements and braces were modeled with linear *truss* elements in the basement. The *nonlinearBeamColumn* element used fiber steel sections with stress-strain behavior of steel fibers modeled with *ReinforcingSteel* material in *OpenSees* (Figure 1c). The nonlinear *truss* elements were modeled with *Hysteretic* material in *OpenSees* (Figure 3). This material model assumed that the stress linearly reduces to zero from the buckling stress at strain value equal to twice the buckling strain; the buckling stress was computed from Euler's Buckling stress formulation. It is useful to note that *OpenSees* does not have an explicit buckling model for steel braces; the buckling options in the *ReinforcingSteel* material are designed only for

reinforcing bars in reinforced-concrete beams and columns and can not be conveniently used for steel braces.

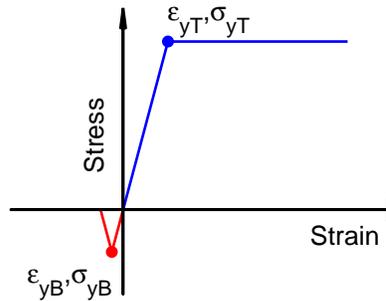


Figure 3. *OpenSees* material model for nonlinear *truss* element.

Perfrom3D Model

In the *Perfrom3D* model of the Los Angeles building, beams were modeled with FEMA Steel Beam with strength loss and symmetrical section strength, columns were modeled with FEMA Steel Column with strength loss and symmetrical section strength, shear walls were modeled with linear elastic column elements, and braces were modeled with Simple Bar element. The material properties for braces were specified by Inelastic Steel Buckling material in *Perfrom3D*. The FEMA Steel Beam element requires moment-plastic-rotation relationship of Figure 4a. The yield moment of the steel beam section was computed automatically by *Perfrom3D* using section properties and steel strength. The plastic rotation values and the residual strength needed for the FEMA Steel Beam model in *Perfrom3D* are selected as per FEMA-356 recommendations: plastic rotations are selected as $9\theta_y$ for point U and $11\theta_y$ for point X in which θ_y is the yield rotation, and the residual strength for points R and X are selected as 60% of the yield moment (Figure 4a). The plastic rotation value for point R is selected as $9.5\theta_y$ to model gradual strength loss between points U and R.

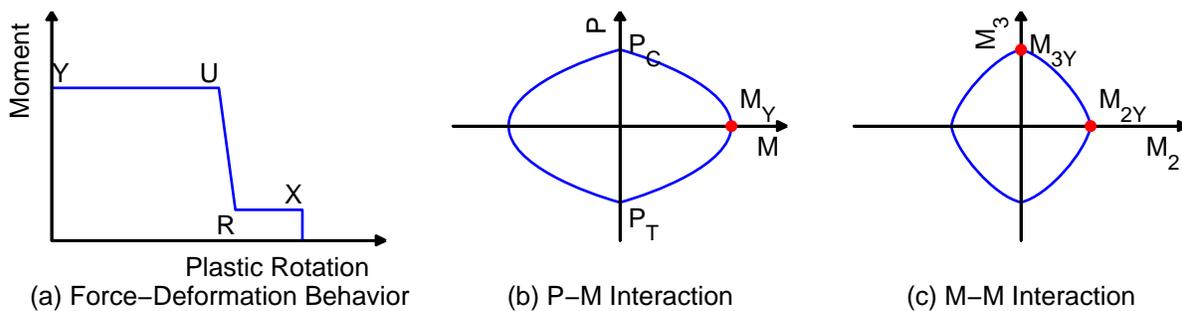


Figure 4. FEMA steel beam/column element in *Perfrom3D*: (a) Force-deformation behavior of beam or column, (b) P-M interaction diagram for column; and (c) M-M interaction diagram for column.

The FEMA Steel Column with strength loss element requires moment-plastic-rotation behavior of Figure (4a), P-M interaction diagram for bending about axis-2 and axis-3 (Figure 4b), and M-M interaction diagram between moments about axis-2 and axis-3 (Figure 4c). The yield moment needed to define the force-deformation behavior (Figure 4a) was automatically

computed by *Perform3D* based on section properties and material strength. Similar to the beams, the plastic rotation values and the residual strength needed for the FEMA Steel Column model in *Perform3D* are selected as per FEMA-356 recommendations: plastic rotations are selected as $9\theta_y$ for point U and $11\theta_y$ for point X in which θ_y is the yield rotation, and the residual strength for points R and X are selected as 60% of the yield moment (Figure 4a). The shapes of the P-M interaction diagrams (Figure 4b) and M-M interaction diagram (Figure 4c) were also automatically generated in *Perform3D* based on the specified section properties and material strength.

Pushover Curves

Pushover curves for the selected buildings were developed for transverse and longitudinal direction using height-wise distribution of lateral loads proportional to the first mode in each direction.

20-Story Hotel in North Hollywood

Figure 5 compares the pushover curves for the North Hollywood Hotel computed from *OpenSees* and *Perform3D*. This comparison indicates that the two programs lead to pushover curves that may differ significantly. The two programs provide essentially identical pushover curves for first transverse and longitudinal modes in the initial elastic region (Figures 5a and 5b). Thereafter, the pushover curves from the two programs differ significantly. The pushover curves from *Perform3D* exhibit early initiation of nonlinear action, much lower yield strength, and significant post yielding strength loss compared to the pushover curves from *OpenSees* (Figures 5a and 5b). This is the case because *Perform3D* used FEMA-356 models for force-deformation behavior of beams and columns with strength loss (see Figure 2a) whereas *OpenSees* used fiber section models for beams and columns with concrete and steel material properties defined by Figure 1a and 1c, respectively. As elements begin to yield and lose strength during pushover analysis, pushover curves from *Perform3D* would begin to yield earlier, would have lower strength, and would show strength loss as more and more elements are deformed beyond point U on the force-deformation relationship (see Figure 2a). On the other hand, the elements in the *OpenSees* model continue to support the load because of gradual spread of plasticity over the member fiber section.

One major concern with the original *OpenSees* model is that the concrete model (*Concrete01*) did not adequately represent concrete crushing, i.e., the concrete fibers continue to support stresses even after crushing strain (see Figure 1a). Therefore, two other material models were considered that adequately address this issue: a modified version of *Concrete01* and *Concrete04* material model with no residual strength after crushing strain (see Figure 1b). The pushover curves were generated from *OpenSees* with these two additional material models and are compared in Figure 6 with those from the original model. These results indicate that the concrete material model has minimal effect on the pushover curves as the pushover curves for all material models are essentially identical.

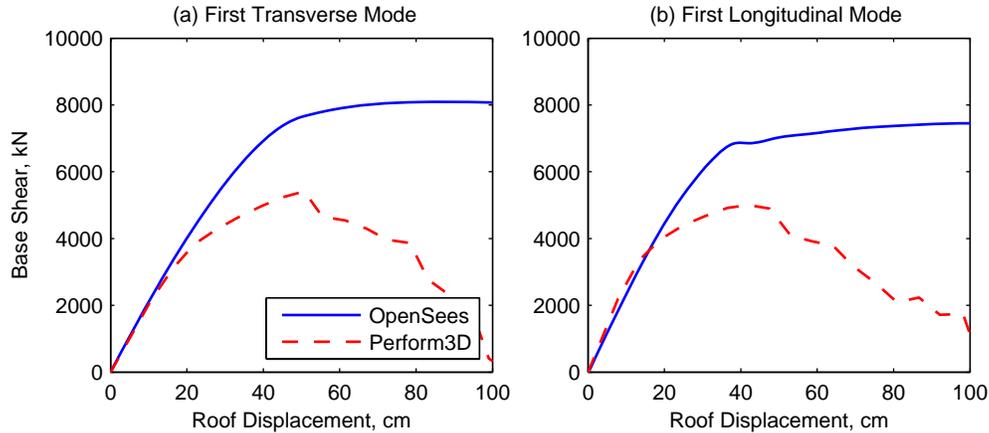


Figure 5. Comparison of pushover curves from *OpenSees* and *Perform3D* for 20-Story Hotel in North Hollywood.

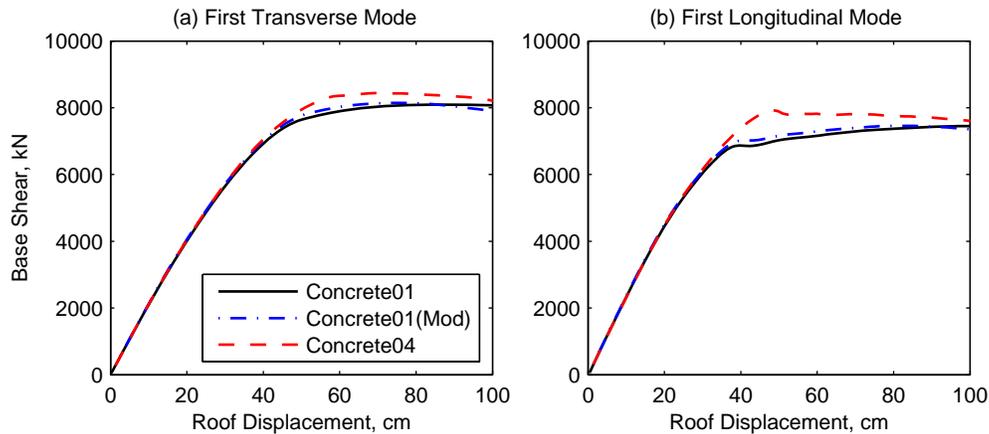


Figure 6. Comparison of “modal” pushover curves from *OpenSees* for three different concrete material models – *Concrete01*, modified *Concrete01*, and *Concrete04* – for 20-Story Hotel in North Hollywood.

Since the original *Perform3D* model modeled beams and columns with strength loss, which is quite different from the *OpenSees* modeling, a second *Perform3D* model was developed in which no strength loss was considered, i.e., the moment in the force-deformation behavior of beams and columns retained the yield moment value even after point *U* (Figure 2a). The pushover curves from the revised *Perform3D* model are compared in Figure 7 with those from the *OpenSees* model. These results indicate that pushover curves from *Perform3D* exhibit lower strength in several modes but no strength loss when compared to pushover curves from *OpenSees* (Figures 7a and 7b).

The results of Figure 7 also lead to another important observation: the models based on concentrated plasticity may lead to lower estimate of building strength than models based on spread plasticity. It is useful to recall that *Perform3D* model is based on concentrated plasticity as it uses a concentrated hinge at the beam-column ends and elastic behavior in-between. The *OpenSees* model on the other hand is a spread plasticity model as the nonlinear action spreads

gradually across the beam-column section as material fibers undergo increasing stresses and strains.

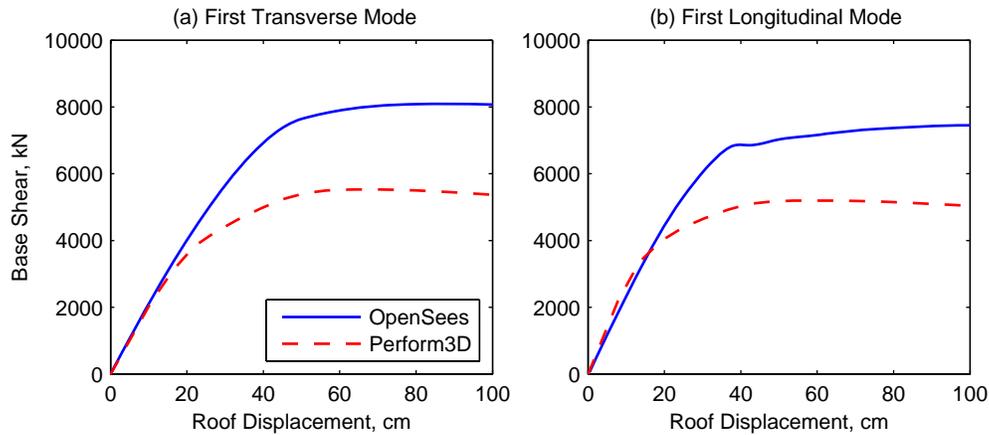


Figure 7. Comparison of pushover curves from *OpenSees* and *Perform3D* for 20-Story Hotel in North Hollywood; *Perform3D* results are using no strength loss model for beams and columns.

19-Story Office Building in Los Angeles

Figure 8 compares the pushover curves for the Los Angeles Office Building computed from *OpenSees* and *Perform3D*. It is useful to recall that the lateral load resisting system in this building consists of steel moment resisting frames in the longitudinal direction and concentric braced frames in the transverse direction. This comparison also indicates that the two programs lead to pushover curves that differ significantly. In the transverse direction, the direction in which lateral load resisting system consists of concentric braced frames, *Perform3D* provides pushover curves that has slightly lower strength and much earlier initiation of nonlinear action compared to the curves from *OpenSees* (Figures 8a). The pushover curve from *Perform3D* also exhibits post yield strength loss whereas that from *OpenSees* does not show strength loss (Figure 8a).

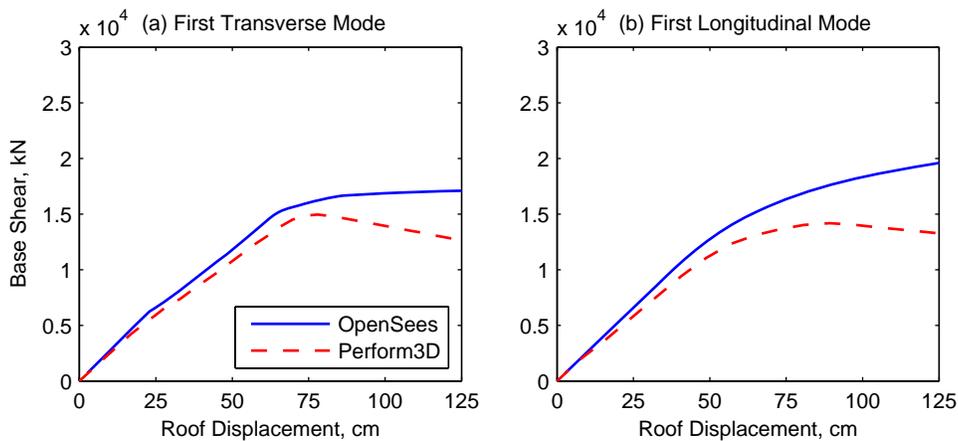


Figure 8. Comparison of “modal” pushover curves from *OpenSees* and *Perform3D* for 19-Story Office Building in Los Angeles.

In the longitudinal direction, the direction in which lateral load resisting system consists of moment resisting frames, *Perform3D* led to pushover curves with lower strength and earlier initiation of nonlinear action compared to *OpenSees* (Figures 8b). The pushover curve from *Perform3D* also exhibits post yielding strength loss (Figure 8b). As noted previously for the North Hollywood building, these differences in the longitudinal pushover curves are due to different modeling assumptions in the two programs: *Perform3D* used FEMA-356 models for force-deformation behavior of beams and columns with strength loss (see Figures 3a) whereas *OpenSees* used fiber section models for beams and columns with concrete and steel material properties defined by Figure 1c, respectively.

Effects of Modeling Assumptions and Software on Pushover Curves

The results presented so far indicate that modeling assumptions may significantly affect the pushover curves: (1) The concentrated plasticity model leads to lower strength, early initiation of yielding, and post yielding strength loss in pushover curves compared to spread plasticity model; and (2) Consideration of strength loss in beam/column model leads to significant strength loss in the pushover curve. The concrete material model, on the other hand, appears to have minimal effect on the pushover curves of reinforced concrete buildings.

The pushover curves may also depend on the software that is used for analysis. While most of the differences may be attributed to differences in modeling options available in different programs (e.g., *OpenSees* does not have an option for modeling FEMA-356 force-deformation behavior with strength loss whereas *Perform3D* does), some differences may also occur due to differences in solution schemes and convergence criteria.

Comparison of Peaks Responses from RHA

Responses – floor displacements, floor accelerations, and base shear – of the two selected buildings were computed for selected ground motions using *OpenSees* and *Perform3D*. It is useful to note that selected buildings experienced excessive deformation due to several of the ground motions in this suite and collapsed for these motions. For example, the North Hollywood Hotel collapsed for ground motions number 7 to 11, 13, 17, 18, 21, and 26, 29. The Los Angeles Buildings collapsed due to ground motions number 5 to 11, 13, 17, 18, and 26 to 29.

The following difficulties were encountered during RHA of the two selected buildings. First, *OpenSees* experienced significantly more convergence problem compared to *Perform3D*. While *Perform3D* failed to converge only for cases where the building collapsed, *OpenSees* failed to converge even for some cases where the building is not expected to collapse. For these cases, *OpenSees* failed to converge even when different solution strategies were used. It is useful to note that *OpenSees* models of the two selected buildings are much more complex compared to *Perform3D* models; *OpenSees* models used fiber section modeling whereas *Perform3D* used concentrated plasticity modeling for beams and columns.

The differences in the peak floor displacements, accelerations, and base shear from the two programs are investigated next by examining height-wise variation of the ratio of peak floor displacements, u_{OS}/u_{P3D} , peak floor accelerations, a_{OS}/a_{P3D} , and peak base shear, V_{OS}/V_{P3D} , from *OpenSees* and *Perform3D*. The results are presented in Figures 12 to 17 for earthquakes for

which the building did not collapse. The results for the North Hollywood Hotel are for the *OpenSees* model which used *Concrete01* material model with residual strength and *Perform3D* model with beams and columns modeled with FEMA-356 force-deformation behavior with strength loss. The other two concrete models – *Concrete01* without residual strength and *Concrete04* model – led to responses similar to the *Concrete01* material model with residual strength when the solution converged. However, *OpenSees* model with these two concrete materials experienced much more convergence problem compared to the model with *Concrete01* material with residual strength. The results for the Los Angeles Building are for the *OpenSees* Model with *Hysteretic* steel material for braces to capture post-buckling strength loss and for the *Perform3D* model with beams and columns modeled with FEMA-356 force-deformation behavior with strength loss.

The presented results include variation of the ratios for individual earthquakes along with the median values and median plus/minus one standard deviation. Median is an indicator of *OpenSees* over or under predicting response compared to *Perform3D* whereas the band formed by median plus/minus one standard deviation is an indicator of the dispersion in the response prediction.

20-Story Hotel in North Hollywood

The median of displacement ratios in Figure 9 for the North Hollywood Hotel indicates that *OpenSees* tends to provide larger estimate of displacements in lower and upper floors and about the same estimates of displacements in middle floors compared to the *Perform3D*. The difference in median value of the displacement ratio in upper and lower floors varies from 1.1 to 1.4 indicating that the response from the two programs can differ by 10% to 40%. The width of the median+ σ or median- σ band is about 0.15 implying that there is about 15% dispersion in the response prediction from the two programs.

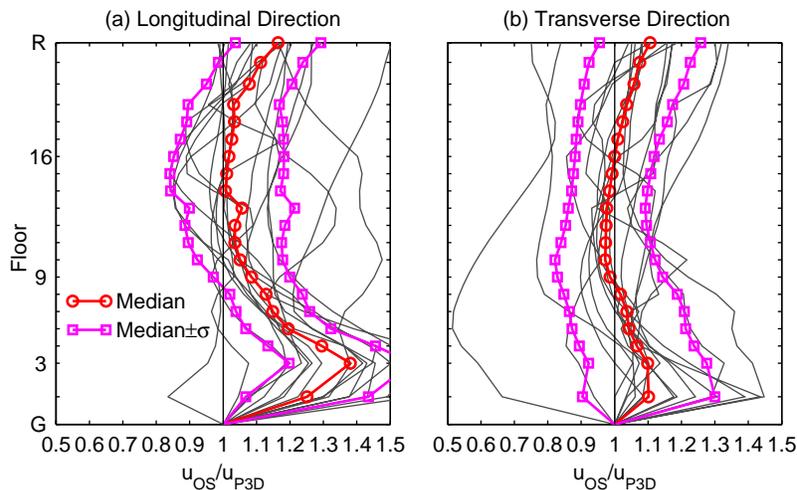


Figure 9. Height wise variation of ratio of peak floor displacements from *OpenSees* and *Perform3D* for North Hollywood Hotel.

The median of acceleration ratios in Figure 10 show that the *OpenSees* generally provides comparable estimates of floor accelerations throughout the building height as those from

Perform3D: the median of the floor accelerations ratio is very close to one. The exception may occur at a few floors where the ratio may differ from one by 0.05 to 0.15, e.g., 2nd floor and roof in the longitudinal direction (Figure 10a). The width of the median+ σ or median- σ band for floor accelerations varies from 0.05 (Figure 10b) to 0.1 (Figure 10a) implying that there is 5% to 10% dispersion in the response prediction from the two programs.

The median of ratio in Figure 11 shows that the *OpenSees* generally provides comparable estimates of base shear to that from *Perform3D*: the median of the base shear ratio is very close to one. The width of the median+ σ or median- σ band is about 0.1 implying that there is 10% dispersion in the response prediction from the two programs.

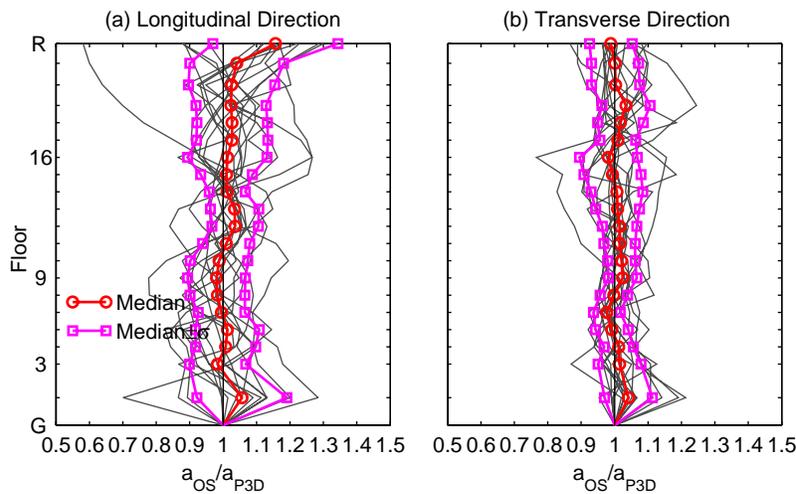


Figure 10. Height wise variation of ratio of peak floor accelerations from *OpenSees* and *Perform3D* for North Hollywood Hotel.

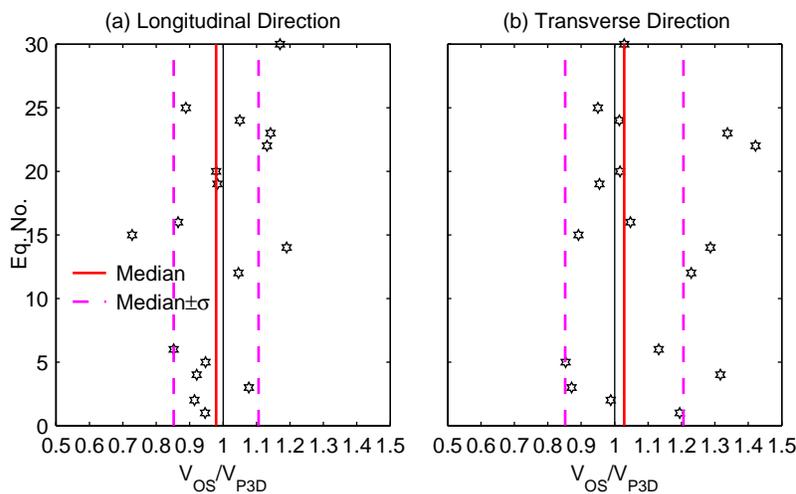


Figure 11. Ratio of base shear from *OpenSees* and *Perform3D* for North Hollywood Hotel.

19-Story Office Building in Los Angeles

The median of displacement ratios in Figure 12 for the Los Angeles Building indicates that *OpenSees* tends to provide larger estimate of displacements throughout the building height

in the longitudinal direction (Figure 12a) and in upper floors in the transverse direction (Figure 12b) compared to the *Perform3D*. The difference in median value of the displacement ratio in upper and lower floors varies from 1.05 to 1.1 indicating that the response from the two programs can differ by 5% to 10%. The width of the median+ σ or median- σ band is about 0.1 implying that there is 10% dispersion in the response prediction from the two programs.

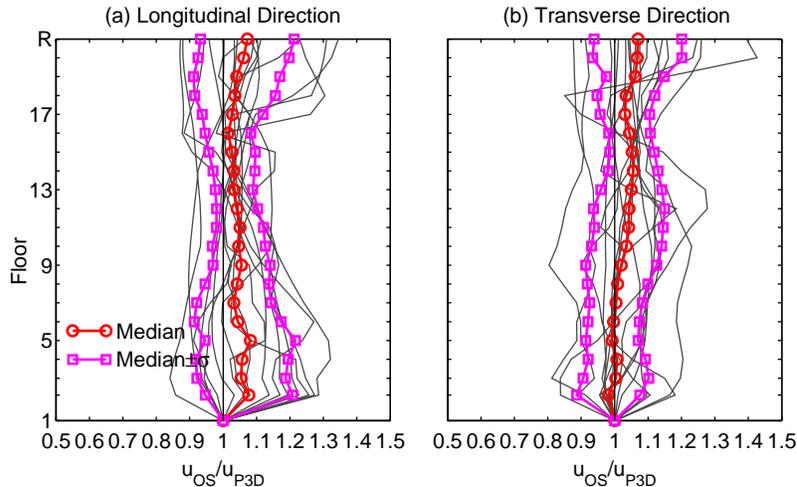


Figure 12. Height wise variation of ratio of peak floor displacements from *OpenSees* and *Perform3D* for Los Angeles Building.

The median of acceleration ratios in Figure 13 show that the *OpenSees* generally provides comparable estimates of upper floor accelerations as those from *Perform3D*: the median of the floor accelerations ratio is very close to one. For lower floors, where a soft story condition occurs in the Los Angeles building due to taller story height, the ratio may differ from one by 0.1 to 0.25, e.g., 2nd floor (Figures 13a and 13b). The width of the median+ σ or median- σ band for floor accelerations varies from 0.05 (Figure 13a) to 0.1 (Figure 13b) implying that there is 5% to 10% dispersion in the response prediction from the two programs.

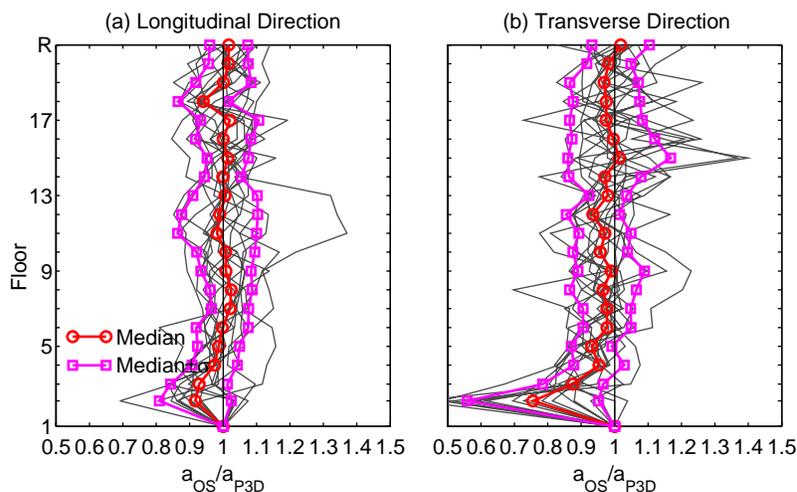


Figure 13. Height wise variation of ratio of peak floor accelerations from *OpenSees* and *Perform3D* for Los Angeles Building.

The median of ratio in Figure 14 show that the *OpenSees* provides lower base shear in the longitudinal direction and comparable base shear in transverse direction than that from *Perform3D*. The width of the median+ σ or median- σ band varies from 0.05 to 0.1 implying that there is 5% to 10% dispersion in the response prediction from the two programs.

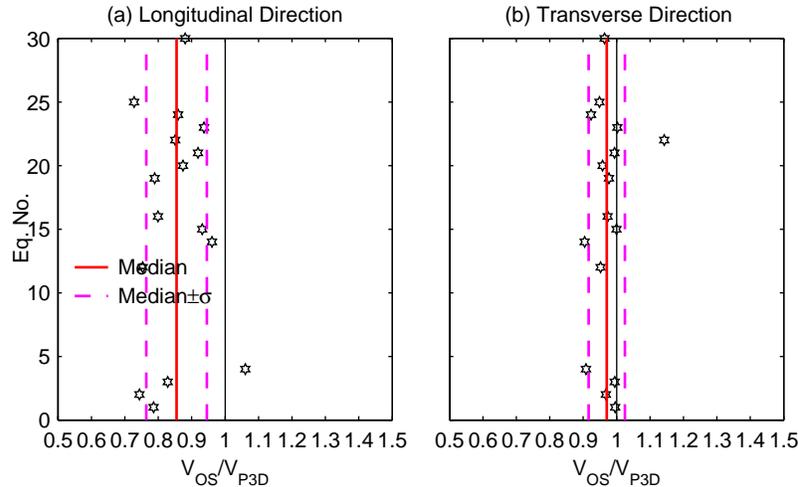


Figure 14. Ratio of peak base shear from *OpenSees* and *Perform3D* for Los Angeles Building.

Effects of Software on Peak Response from RHA

The results presented so far indicate that there can be 10% to 40% difference in prediction of median peak response from different software. The difference is much higher for reinforced concrete building compared to the steel building, and in median prediction of floor displacements compared to prediction of floor accelerations and base shear. It is useful to recall that *OpenSees* used a fiber section model which captured spread of inelastic action over the member section whereas *Perform3D* used a FEMA-356 type concentrated plastic hinge with strength loss. Therefore, larger variability in the response prediction for the reinforced concrete building appears to be due to significant behavior differences in reinforced-concrete beam/columns models available in the two selected computer programs. On the other hand, smaller variability in the response of the steel building appear to be due to less significant differences in the steel beam-column models available in the two programs. Furthermore, there is 10% to 15% dispersion as apparent from median $\pm\sigma$ band. It is useful to emphasize that above observations are for median response ratios only. Response ratio for individual ground motions from the two programs may vary by as much as 50%.

Comparison of Inertial and True Base Shears

As mentioned previously, base shear in buildings with recorded motions is typically estimated from summation of floor inertial forces above the building’s base (Figure 15a). For this purpose, the floor inertial forces are computed by multiplying the floor masses with the total floor accelerations. The base shear thus calculated is designated as the “inertial base shear” in this investigation. This base shear is generally accepted to provide a good estimate of the “true base shear” which is equal to sum of shears in all columns at the building’s base (Figure 15b).

This section re-examines if the inertial base shear, V_{bl} , provides a good estimate of the true base shear, V_{bR} . For this purpose, ratios of the inertial and true base shears for the two buildings were computed from *OpenSees* and *Perform3D*. The accelerations used in computing the inertial base shear were those computed from RHA. The results are presented in Figures 16 to 19 for earthquakes for which the building was deemed not to collapse. The presented results include ratio for individual earthquakes along with the median values and median $\pm\sigma$ values.

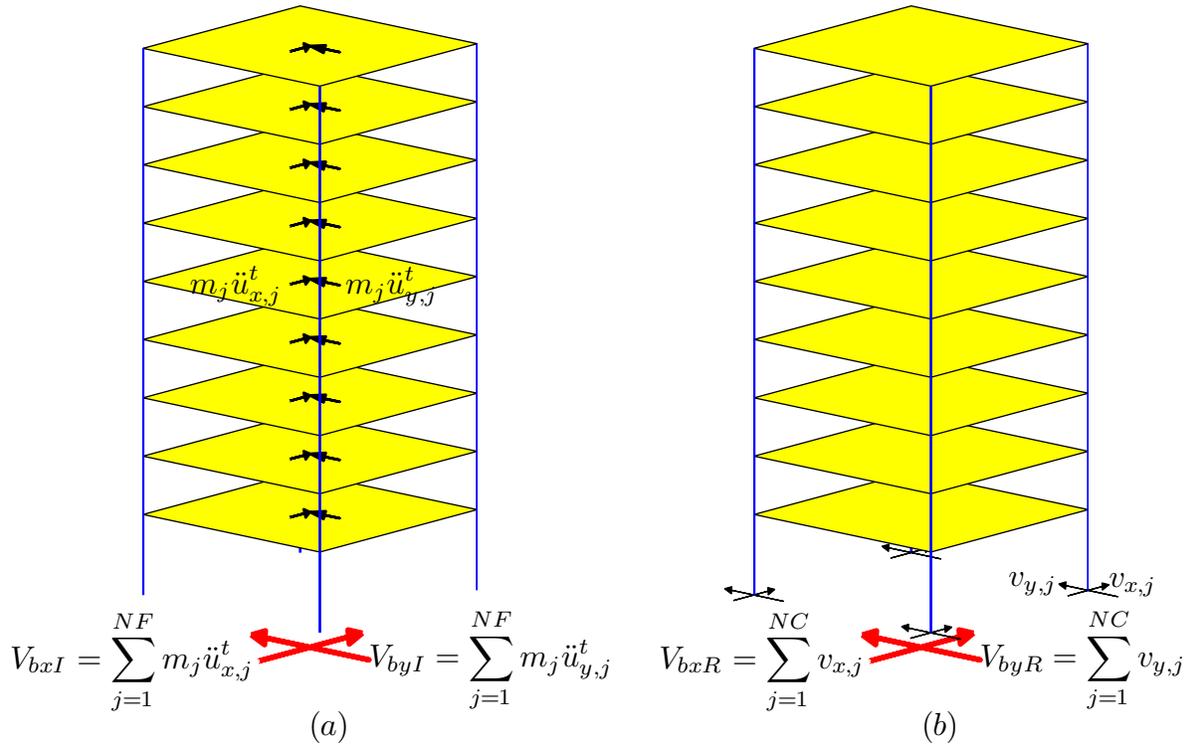


Figure 15. Computation of base shear: (a) Base shear computed from summation of inertial floor forces; and (b) Base shear computed from summation of column base shears.

20-Story Hotel in North Hollywood

The results presented in Figure 16 for the North Hollywood Hotel show that the ratio V_{bl}/V_{bR} from *OpenSees* for some earthquakes can be as high as 1.25. This indicates that inertial base shear may over predict the true base shear by up to 25%. The median value of the ratio is, however, much smaller: the median ratio is from 1.08 (Figure 16a) to 1.12 (Figure 16b). Therefore, it may be expected that the inertial force will over predict the true base shear in the median by about 10%. The width of the median $+\sigma$ or median $-\sigma$ band varies from 0.05 to 0.08 implying that there is 5% to 8% dispersion in the response prediction. The results presented in Figure 17 for *Perform3D* results show trends similar to those for *OpenSees* results in Figure 16 with the variations being slightly smaller for the former program.

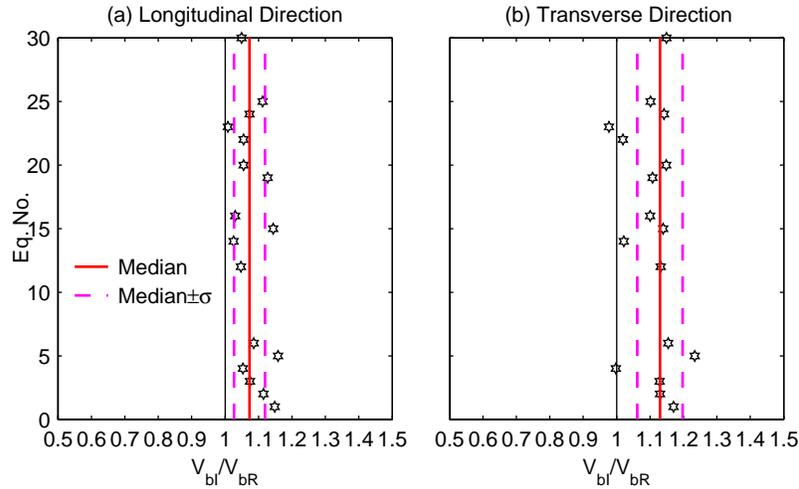


Figure 16. Ratio of peak inertial and true base shears from *OpenSees* for North Hollywood Hotel.

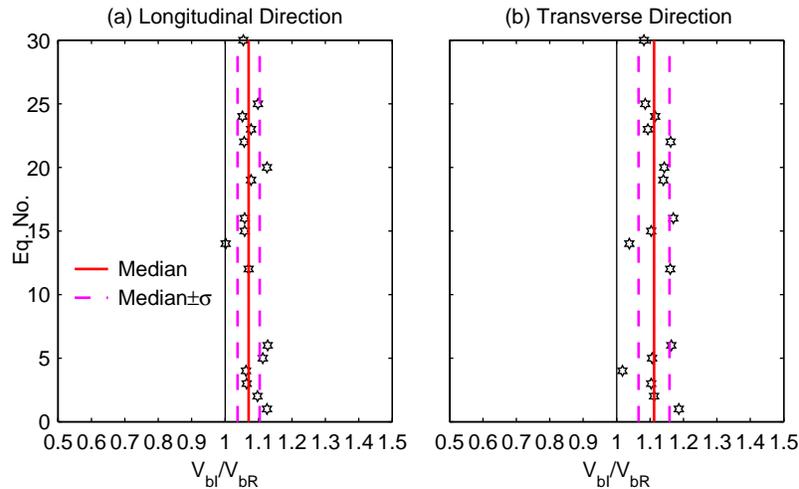


Figure 17. Ratio of peak inertial and true base shears from *Perform3D* for North Hollywood Hotel.

19-Story Office Building in Los Angeles

The results presented in Figure 18 for the Los Angeles building show very little variation in V_{bl}/V_{bR} from *OpenSees*: the median is very close to one (Figures 18a and 18b). The width of the median+ σ or median- σ band is about 0.2 in the longitudinal direction (Figure 18a) and very small in the transverse direction (Figure 18b). The results from *Perform3D* show median value of the ratio to range from 1.05 (Figure 19a) to 1.2 (Figure 19b) with the width of the median+ σ or median- σ band to range from 0.1 (Figure 19b) to 0.2 (Figure 19a). For individual earthquake, however, the inertial base shear may exceed the true base shear by as much as 75% (see Figures 18a and 19a).

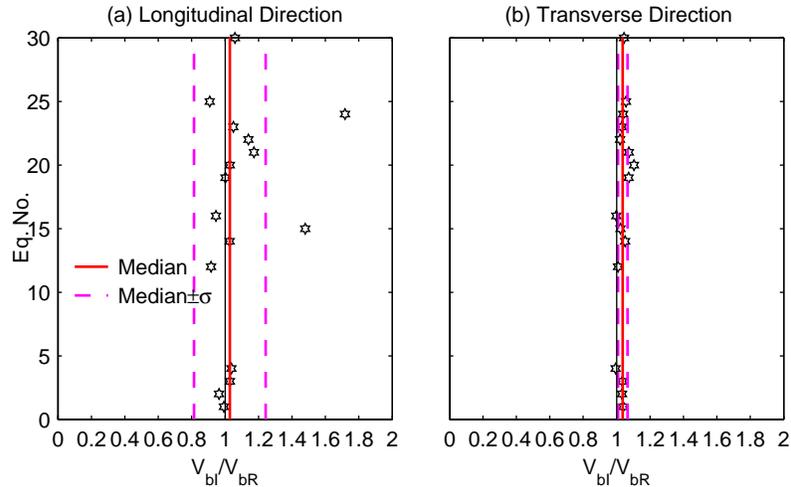


Figure 18. Ratio of peak inertial and true base shears from *OpenSees* for Los Angeles Building.

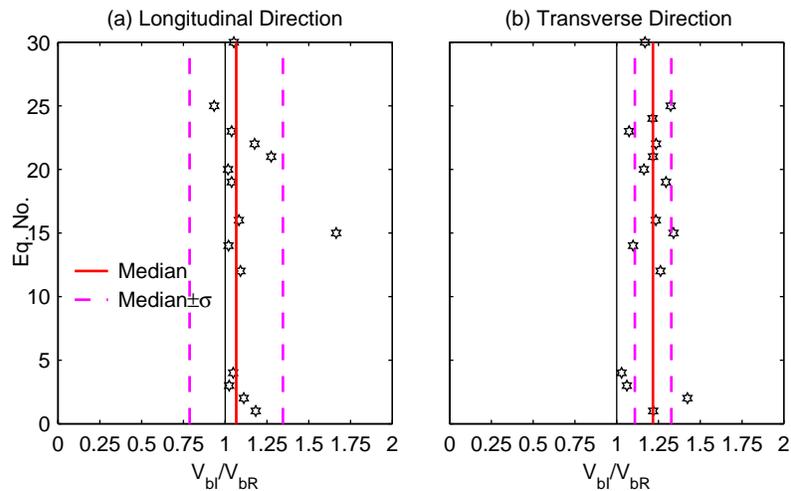


Figure 19. Ratio of peak inertial and true base shears from *Perform3D* for Los Angeles Building.

The presented so far indicate that the median inertial base shear exceeds the true base shear by 10 to 20%. For individual earthquakes, however, the inertial base shear may overestimate the true base shear by as much as 75%. Therefore, inertial base shear should be used with caution as an estimate of the true base shear.

Effects of Time Shift on Inertial Base Shear

The inertial base shear is typically used to estimate the true base shear in buildings with acceleration recorded during earthquakes. A mid- or high-rise building is instrumented with a large number of accelerometers and all of these accelerometers may not be connected to a single recorder. Although, the recorders are generally time synchronized, there is still a possibility of time shift between different recorders. Examined next is the effect of time shift on inertial base shear. For this purpose, time shift equal to one time step is introduced between the base acceleration and the relative acceleration computed from RHA of the two selected buildings using *OpenSees* and the inertial base shears are recomputed; recall the inertial base shear requires total floor accelerations which are defined as sum of the base acceleration and relative floor

acceleration. The results are presented in Figure 20 for the North Hollywood Hotel and Figure 21 for the Los Angeles Building. These results indicate a large discrepancy between the inertial and true base shears. The median of inertial base shear may exceed the true base shear by a factor of 2 to 4. Furthermore, the inertial base shear may be as large as 10 times the true base shear for individual earthquakes. Clearly, time shift, even a small one, can lead to erroneous estimate of base shear demand in buildings with accelerations recorded during earthquakes.

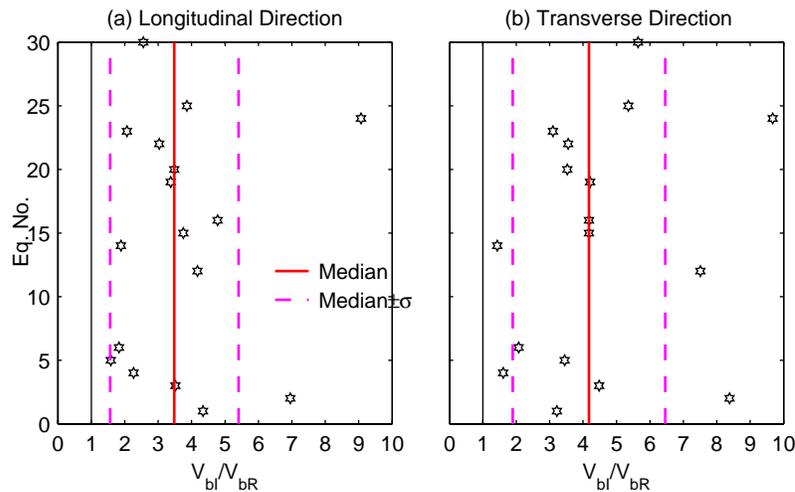


Figure 20. Ratio of peak inertial and true base shears from *OpenSees* for North Hollywood Hotel; results are computed by introducing a one time step delay between base acceleration and relative floor acceleration.

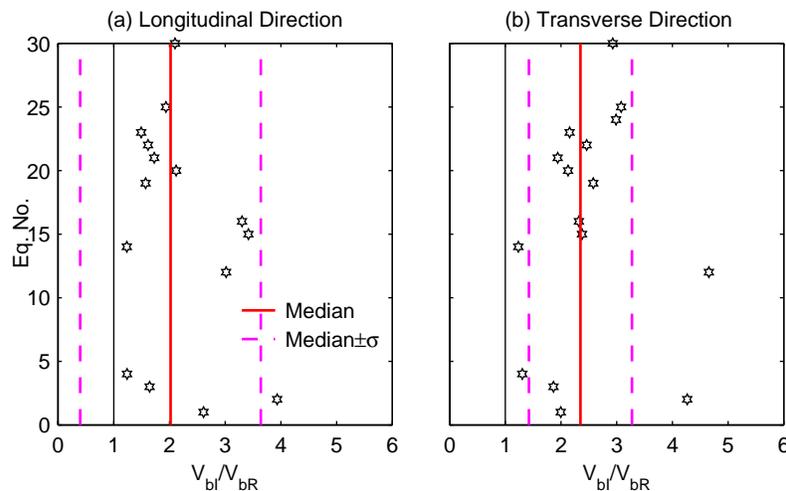


Figure 21. Ratio of peak inertial and true base shears from *OpenSees* for Los Angeles Office Building; results are computed by introducing a one time step delay between base acceleration and relative floor acceleration.

Conclusions

The investigation first examined the effects of modeling assumptions and two different computer programs on nonlinear response of three-dimensional buildings. It was found that modeling assumptions may significantly affect the pushover curves: concentrated plasticity

model leads to lower strength, early initiation of yielding, and post yielding strength loss in pushover curves compared to spread plasticity model; and strength loss model for beams/columns leads to significant post yielding strength loss in the pushover curve. The concrete material model, on the other hand, appears to have minimal effect on the pushover curves of reinforced concrete buildings. While most of the differences may be attributed to differences in modeling options available in different programs (e.g., *OpenSees* does not have an option for modeling FEMA-356 force-deformation behavior with strength loss whereas *Perform3D* does), some differences may also occur due to differences in solution schemes and convergence criteria available in different software programs.

It was also found that the prediction of median peak response from different software can differ from 10% to 40%. The difference tends to be much higher for reinforced concrete building compared to the steel building, and for floor displacements compared to floor accelerations and base shear. Furthermore, there is dispersion of about 10% to 15% in the median prediction as apparent from median $\pm\sigma$ band. For individual ground motions, the peak responses from different computer programs may vary by as much as 50%.

This investigation also examined if the “inertial base shear”, defined as summation of floor inertial forces above the building’s base with the floor inertial forces computed by multiplying the floor masses with the total floor accelerations, can provide an accurate estimate of the “true base shear” which is equal to sum of shears in all columns at the building’s base. This investigation indicated that the median inertial base shear exceeds the true base shear by 10 to 20%. For individual earthquakes, however, the inertial base shear may overestimate the true base shear by as much as 50%. It was also found that even a small recording time delay between different acceleration channels may lead to large errors if these accelerations are used to estimate base shear demand during an earthquake event. Therefore, inertial base shear should be used with caution as an estimate of the true base shear.

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