Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 164

LIMITATIONS OF ESTIMATING BASE SHEAR DEMAND IN EXISTING BUILDING FROM RECORDED MOTIONS

Rakesh K. Goel[1]

Abstract

This investigation examined if the inertial base shear, defined as summation of floor inertial forces above the building's base where the floor inertial forces are 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. For this purpose, floor accelerations and true base shears of two building – 20-Story Reinforced-Concrete Hotel in North Hollywood and 19-Story Steel Office Building in Los Angeles – computed from response history analysis (RHA) for a suite of 30 ground motions recorded during past earthquakes were computed using *Perform3D*. The inertial base shear was then compared with the true base shear. It was found that the inertial base shear exceeds the true base shear in the median by 10% to 20%. For individual earthquakes, however, the inertial base shear may exceed the true base shear by as much as 75%. Therefore, inertial base shear should be used with caution to estimate 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 accelerations. The processed floor accelerations may be used to estimate base shear by adding all floor inertial forces above the base (Figure 1a); the floor inertial forces are computed as the product of floor acceleration and floor mass. It is tacitly assumed that this base shear is sufficiently close to the true base shear, which is sum of shear forces in all columns at the building's base (Figure 1b), and is compared against its capacity to check if the building suffered damage during an earthquake and may need detailed inspection/evaluation.

A recent investigation by Goel and Chadwell (2007) compared the inertial base shear with the base shear capacity estimated from nonlinear pushover analysis of buildings. It was found that the inertial base shear significantly exceeded the base shear capacity for several buildings. However, post earthquake inspection of these buildings did not indicate significant

[1]Professor, Dept. of Civil and Environmental Engineering, California Polytechnic State University, San Luis Obispo, CA 93407

damage. Therefore, there is a need to re-examine if the inertial base shear provides an accurate estimate of the "true" base shear.

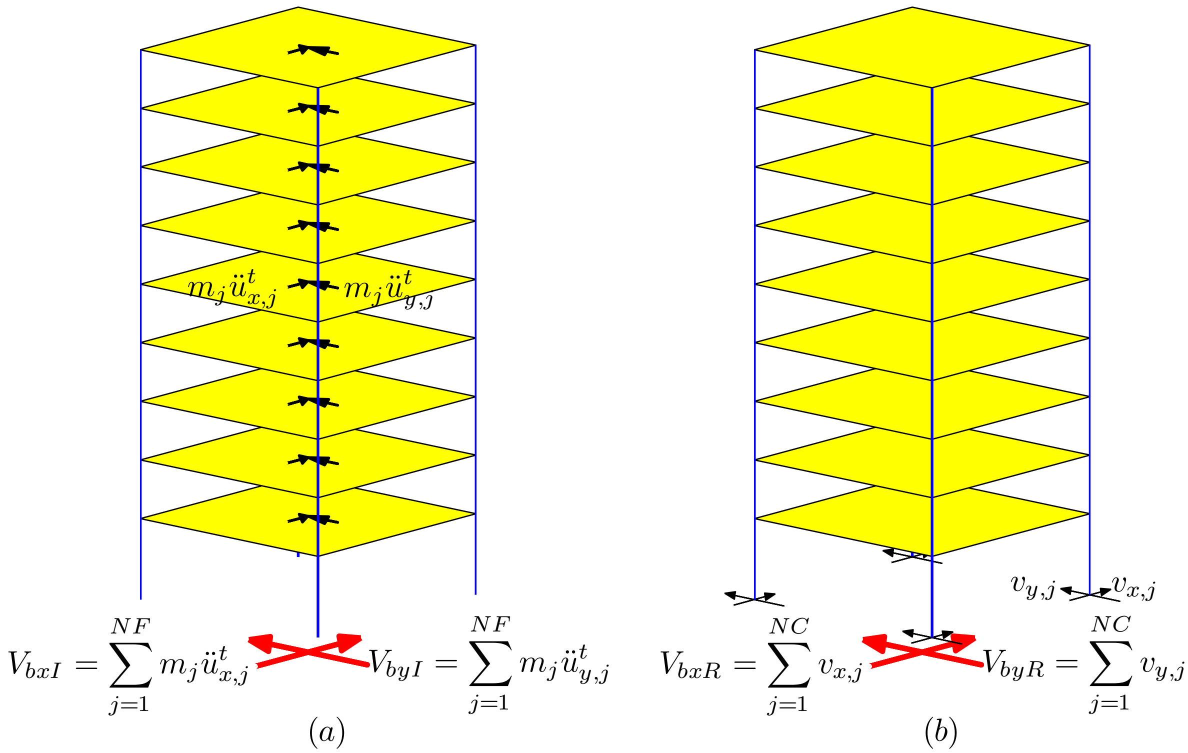

Figure 1. Computation of base shear: (a) Inertial base shear computed from summation of inertial floor forces; and (b) True base shear computed from summation of column base shears.

In order to fill this need, this investigation compared the inertial and true base shears of two building – 20-Story Reinforced-Concrete Hotel in North Hollywood and 19-Story Steel Office Building in Los Angeles. Since recorded motions of these buildings are not available at all floor and true base shear is never measured, floor accelerations needed to compute the inertial base shear and the true base shear were computed from response history analysis (RHA) for 30 ground motions recorded during past earthquakes (Table 1) using the computer program *Perform3D*.

Analytical Models

The three-dimensional analytical models of the selected buildings were developed using the structural analysis software *Perform3D* (CSI, 2006). Following is a description of modeling procedure for each of the two selected buildings.

North Hollywood Hotel

The 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).

Table 1. 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

The plastic rotation values and the residual strength needed for the FEMA Concrete Beam model in *Perform3D* 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*.

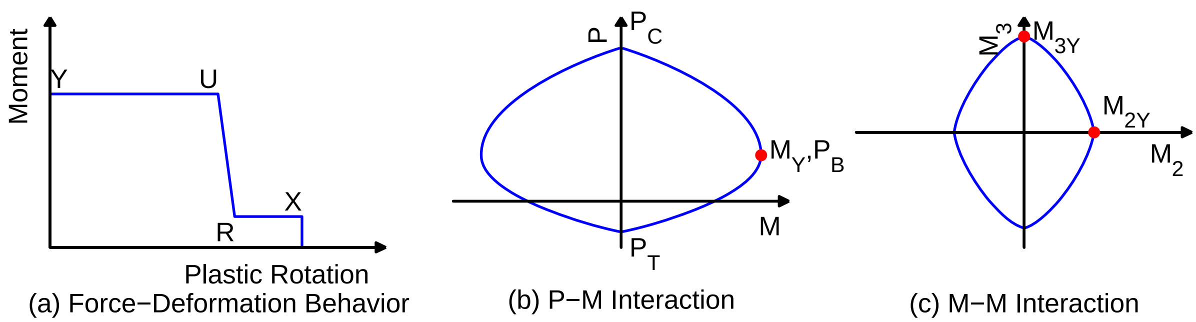

Figure 2. FEMA concrete beam/column element in *Perform3D*: (a) Force-deformation behavior of beam or column, (b) P-M interaction diagram for column; and (c) M-M interaction diagram for column.

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.

Los Angeles Office Building

The 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 *Perform3D*. The FEMA Steel Beam element requires moment-plastic-rotation relationship of Figure 3a. The yield moment of the steel beam section was computed automatically by *Perform3D* using section properties and steel strength. The plastic rotation values and the residual strength needed for the FEMA Steel Beam 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 3a). The plastic rotation value for point R is selected as $9.5\theta_y$ to model gradual strength loss between points U and R.

The FEMA Steel Column with strength loss element requires moment-plastic-rotation behavior of Figure (3a), P-M interaction diagram for bending about axis-2 and axis-3 (Figure 3b), and M-M interaction diagram between moments about axis-2 and axis-3 (Figure 3c). The yield moment needed to define the force-deformation behavior (Figure 3a) 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 3a). The shapes of the P-M interaction diagrams (Figure 3b) and M-M interaction diagram (Figure 3c) were also automatically generated in *Perform3D* based on the specified section properties and material strength.

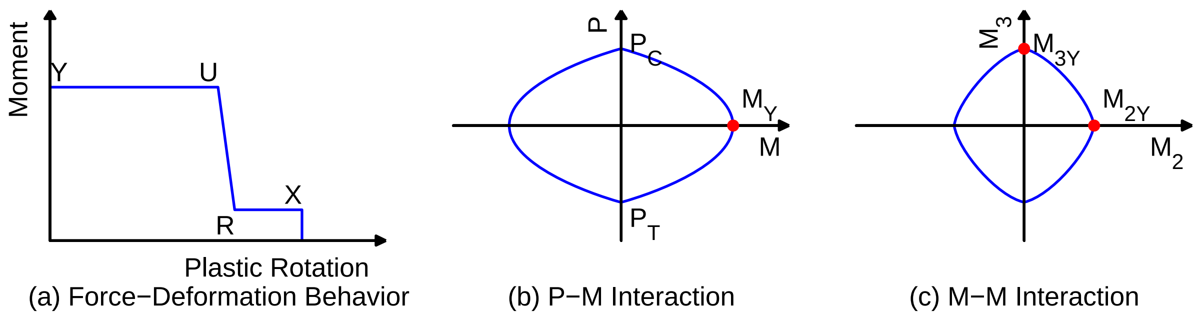

Figure 3. FEMA steel beam/column element in *Perform3D*: (a) Force-deformation behavior of beam or column, (b) P-M interaction diagram for column; and (c) M-M interaction diagram for column.

Estimation of Base Shear from Floor Accelerations

Compared in this section are the inertial and true base shears for the selected ground motions. It is useful to note that the ground motions in Table 1 were not selected to match any design spectrum but to ensure that they will induce different levels of inelastic behavior in the selected buildings. It was found during RHA that the selected buildings experienced excessive deformation due to several of the ground motions and collapsed. For example, the North Hollywood Hotel collapsed for ground motions number 7 to 11, 13, 17, 18, 21, and 26, 29, and the Los Angeles Buildings collapsed due to ground motions number 5 to 11, 13, 17, 18, and 26 to 29.

Examined first were the time-variations of inertial and true base shears for selected ground motions. This examination showed that the inertial base shear matched the true base shear quite well for some earthquakes but the difference was very large for others. Since the length limitation of this paper prohibit presentation of all results, selected results are presented for each of the two buildings in Figures 4 to 7 to demonstrate cases where the two base shears matched quite well and where they differed significantly.

The results for the North Hollywood Hotel indicate that the inertial base shear tracks the true base shear quite well for earthquake no. 14. Furthermore, the peak value of inertial base shear is essential equal to the true base shear in the longitudinal direction (Figure 4a) and exceeds the true base shear by no more than 4% in the transverse direction (Figure 4b). While the inertial base shear tracks the true base shear quite well for earthquake no. 9, the peak value may differ by about 10% in the longitudinal direction (Figure 5a) and by about 20% in the transverse direction (Figure 5b).

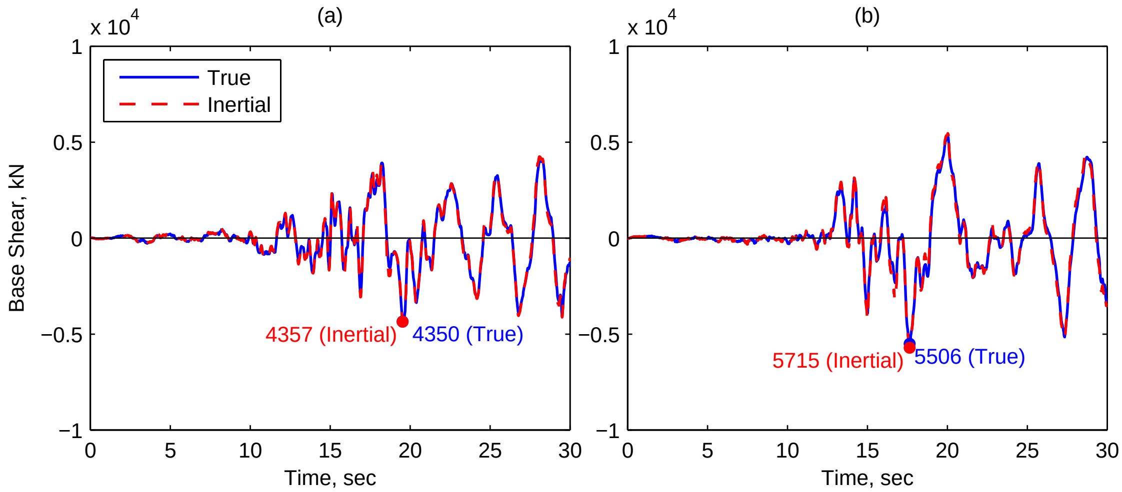

Figure 4. Comparison of inertial and true base shears for North Hollywood Hotel for Earthquake No. 14: (a) Longitudinal direction, and (b) Transverse direction.

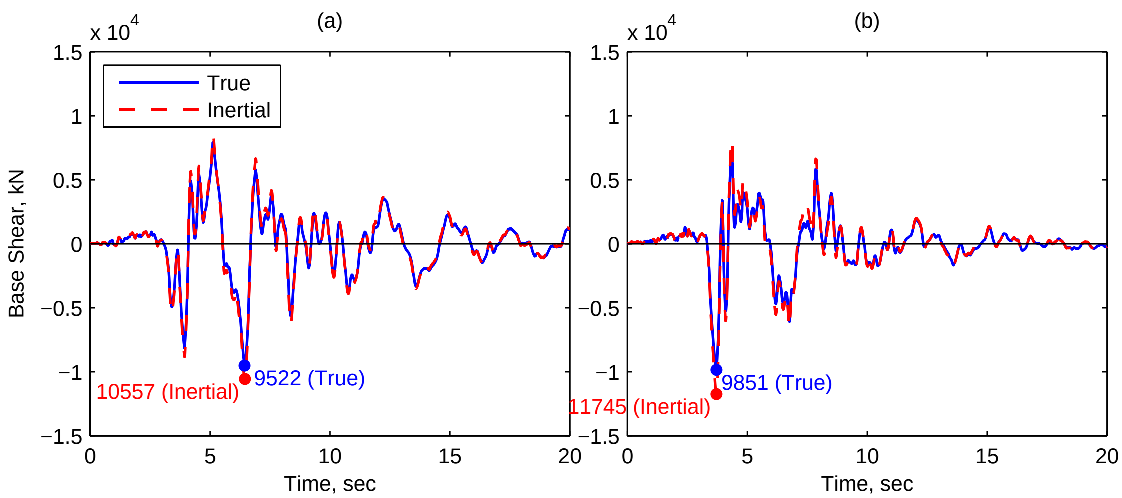

Figure 5. Comparison of inertial and true base shears for North Hollywood Hotel for Earthquake No. 9: (a) Longitudinal direction, and (b) Transverse direction.

The results presented for the Los Angeles Building indicates a very good match between inertial and true base shears for earthquake no. 4 (Figure 6). For earthquake no. 15, however, the inertial base shear differs significantly from the true base shear not only in the peak value but frequency content as well (Figure 7). The peak value of inertial base shear exceed the true base shear by about 70% in the longitudinal direction (Figure 7a) and by about 35% in the transverse direction (Figure 7b). The results of Figure 7 also show that the inertial base shear has significantly larger high-frequency content compared to the true base shear. Therefore, it appears that the inertial base shear may significantly exceed the true base shear for ground motions with very large high-frequency content.

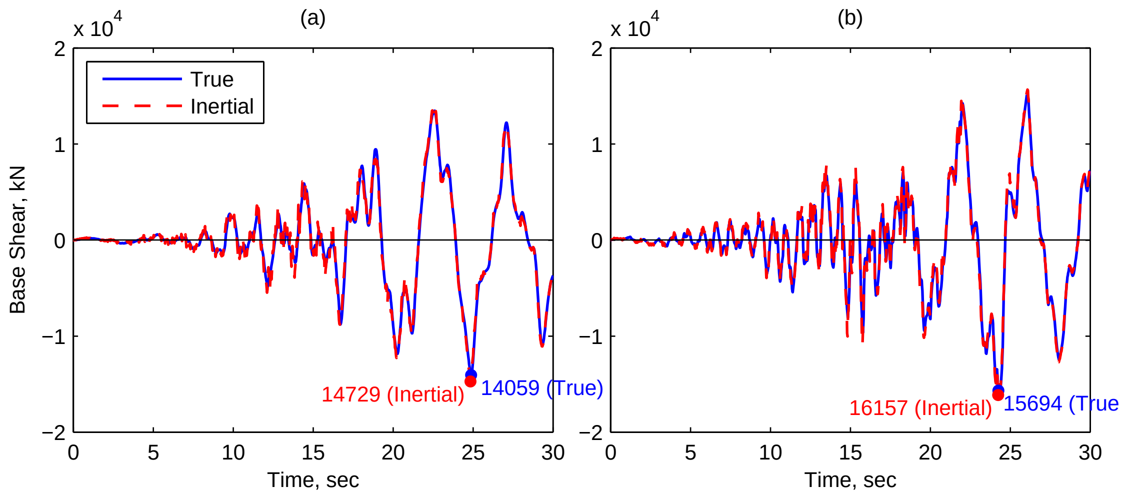

Figure 6. Comparison of inertial and true base shears for Los Angeles Building for Earthquake No. 4: (a) Longitudinal direction, and (b) Transverse direction.

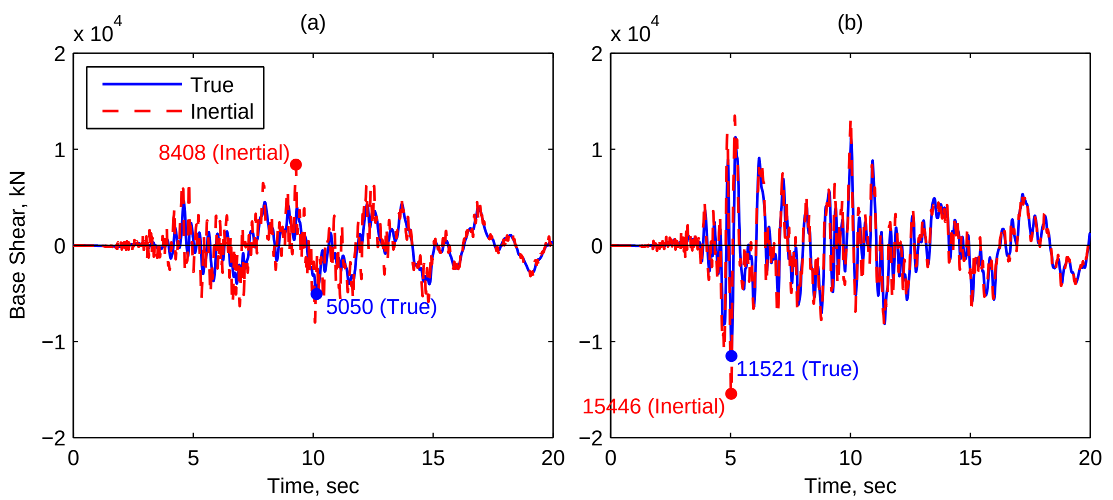

Figure 7. Comparison of inertial and true base shears for Los Angeles Building for Earthquake No. 15: (a) Longitudinal direction, and (b) Transverse direction.

Examined next are the ratios, V_{bI}/V_{bR}, of the inertial and true base shears for the two buildings. The results are presented in Figures 8 and 9 for earthquakes for which the building did not to collapse. The presented results include ratio, V_{bI}/V_{bR}, for individual earthquakes along with the median values and median$\pm\sigma$ values.

The results presented in Figure 8 for the North Hollywood Hotel show that the ratio V_{bI}/V_{bR} for some earthquakes can be as high as 1.2. This indicates that inertial base shear may exceed the true base shear by up to 20%. The median value of the ratio is, however, much smaller: the median ratio is from 1.05 (Figure 8a) to 1.1 (Figure 8b). Therefore, it may be expected that the inertial force will exceed the true base shear in the median by about 5 % to 10%. The width of the band of median+σ or median-σ is about 0.05 implying that there is additional 5% uncertainty for 84% confidence in the base shear prediction.

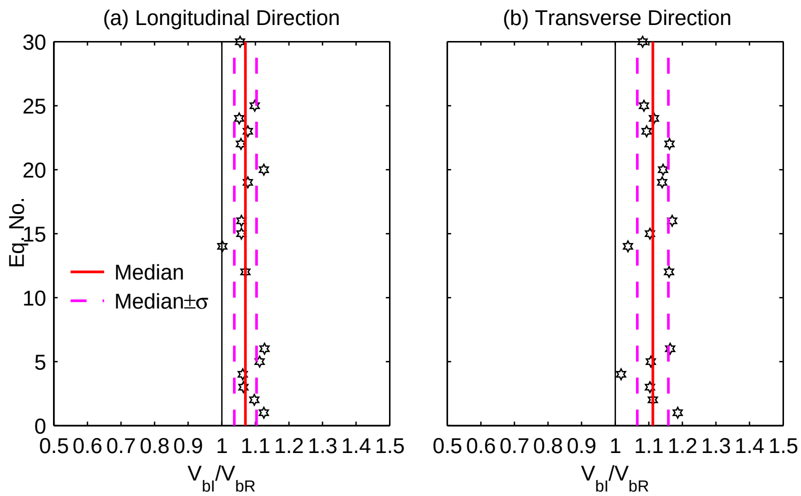

Figure 8. Ratio of peak inertial and true base shears for North Hollywood Hotel.

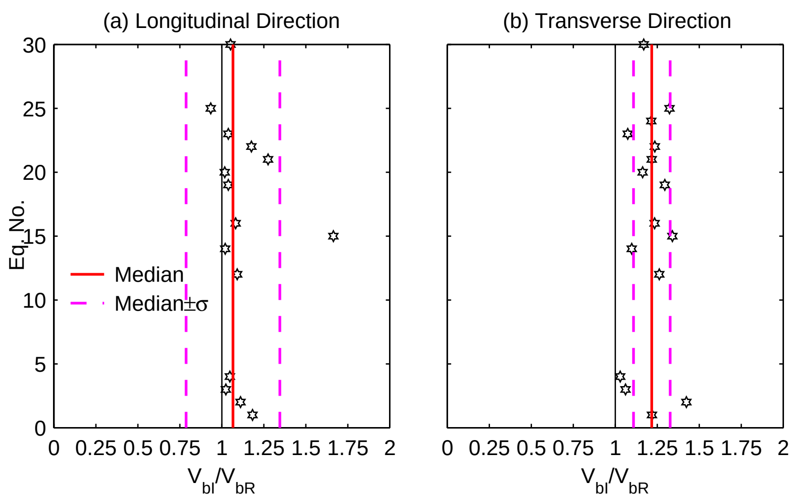

Figure 9. Ratio of peak inertial and true base shears for Los Angeles Building.

The results presented in Figure 9 for the Los Angeles building show that the median value of the ratio varies from 1.05 (Figure 9a) to 1.2 (Figure 9b) implying that the inertial base shear exceeds the true base shear in the median by 5% to 20%. Furthermore, the width of the median+σ or median-σ band varies from 0.1 (Figure 9b) to 0.2 (Figure 9a) indicating additional 10% to 20% uncertainty for 84% confidence in the base shear prediction. For individual earthquake, the ratio can be as high as 1.75 (Figure 9a).

The discussion so far indicates that the median inertial base shear exceeds the true base shear by 10 to 20%. For individual earthquakes, however, the inertial base shear may exceed the true base shear by as much as 75%. Furthermore, the large discrepancy between inertial and true base shears occurs for ground motions with very large high-frequency content. Therefore, inertial base shear should be used with caution as an estimate of the true base shear.

Conclusions

This investigation 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. It was found that the median inertial base shear exceeds the true base shear by 10 to 20%. For individual earthquakes, however, the inertial base shear may exceed the true base shear by as much as 75%. It was also found that the large discrepancy between inertial and true base shears occurs for ground motions with very large high-frequency content. Therefore, inertial base shear should be used with caution as an estimate of the true base shear.

Acknowledgment

This investigation is supported by the California Department of Conservation, California Geological Survey, Strong Motion Instrumentation Program, Contract No. 1007-907. This support is gratefully acknowledged. The author would also like to acknowledge the support provided by Prof. Graham Powell on implementation of *Perform3D* and by Dr. Charles Chadwell on use of *Xtract*.

References

ASCE, 2000. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. *Report No. FEMA-356*, Building Seismic Safety Council, Federal Emergency Management Agency, Washington, D.C.

CSI, 2006. Perform3D: Nonlinear Analysis and Performance Assessment for 3d Structures: Version 4. Computers and Structures, Inc., Berkeley, CA. <www.csiberkeley.com)>.

Goel, R.K. and Chadwell, C., 2007. Evaluation of Current Nonlinear Static Procedures for Concrete Buildings Using Recorded Strong-Motion Data, *CSMIP Data Interpretation Report*, Strong Motion Instrumentation Program, CDMG, Sacramento, CA.

TRC, 2008, Cross Sectional Analysis of Structural Components, v3.0.8, TRC/Imbsen Software Systems, Rancho Cordova, CA. <http://www.imbsen.com/>