16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 31
Registration Code: S-C1459881440

RESPONSES OF A 64-STORY UNIQUE SAN FRANCISCO, CA. BUILDING TO FOUR EARTHQUAKES AND AMBIENT MOTIONS

M. Çelebi[(1)], J. Hooper[(2)], R. Klemencic[(3)]

[(1)] *Senior Research Civil Engineer, U.S. Geological Survey, Menlo Park, Calif., USA, celebi@usgs.gov*
[(2)] *Magnusson Klemencic Associates, Seattle, Wash., USA, Director of EQ. Eng., jhooper@mka.com,*
[(3)] *Magnusson Klemencic Associates, Seattle, Wash., USA, Chairman and CEO., rklemencic@mka.com*

Abstract

We analyze the ambient and earthquake responses of a 64-story, instrumented, concrete core shear wall building in San Francisco, Calif. equipped with tuned sloshing liquid dampers (TSDs) and buckling restraining braces (BRBs). In an earlier paper, only ambient data were used to identify dynamic characteristics. Recently, the 72-channel instrumental array of the building recorded the 24 August 2014 Mw6.0 South Napa and three other earthquakes – allowing comparison of the dynamic characteristics using ambient and earthquake data. Peak accelerations of ambient and the larger South Napa EQ responses at the basement are 0.12 and 5.2 cm/s/s, respectively – a factor of ~ 42 and, at the 61st level, are 0.30 and 16.8 cm/s/s, respectively –a factor of ~56. Fundamental frequencies from spectral ratios for the NS (~0.3Hz), EW (0.27Hz) and torsional accelerations for the earthquake response vary within an insignificant frequency band of ~ 0.02-0.03 Hz as compared to those determined from ambient data. At the level of shaking, BRBs or TSDs are not effective enough to alter dynamic characteristics (frequency or damping). Under future stronger (e.g. design level) shaking of the building, the nonlinearities caused by actions of TSDs and BRBs can substantially shift the dynamic characteristics of the building.

Keywords: concrete core shear wall; ambient vibration; earthquake response; sloshing damper; buckling restrained braces

1. Introduction

The new 64-story unique landmark building in San Francisco, Calif, subject matter of this paper, equipped with a 72-channel seismic monitoring system, recorded its first four earthquake-related ground motions during 2014 (the Mw6.0 24 August 2014 South Napa earthquake, hereafter called South Napa EQ) and three other smaller magnitude earthquakes in 2015. Prior to, in between, and following these earthquakes, several sets of ambient response data from the array of accelerometers deployed throughout the building were acquired on demand.

The building was cooperatively instrumented by the California Strong Motion Instrumentation Program (CSMIP) of California Geological Survey (CGS) and the Strong Motion Project of the U.S Geological Survey (USGS). Figure 1a shows a photo of the building and core-shear wall, with outrigger columns in one direction linked to the core by buckling restrained braces (BRBs). Figure 1b shows NS and EW vertical sections of the building displaying the levels where accelerometers are deployed. Both Figures 1a and 1b are adopted from www.strongmotioncenter.org (last visited 4 February 2016). For ease in following the rest of the paper, Figure 1c displays the numbering of accelerometer channels according to orientation and elevation (floor level) – essential to performing the analyses.

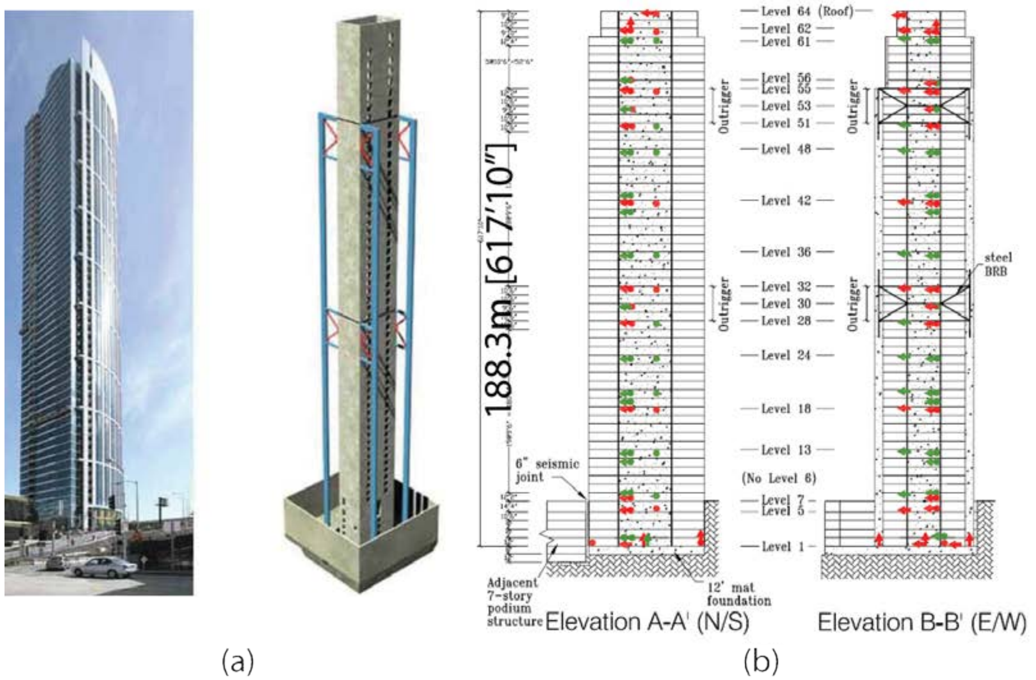
A comprehensive detailed description of the building, performance based seismic design (PBSD) information, acceleration response recording array and analyses of acquired ambient response data, before any earthquake recordings were available, have been presented by Çelebi et al. [1]. Summarizing only relevant information from that paper:

1) The 188.31 m (617.83 ft) tall building is a concrete core shear wall structure with an outrigger frame system and unique dynamic response modification features (such as two tuned sloshing liquid dampers [TSDs] and BRBs in EW direction that extend between 28th -32nd and 51st -55th floors). The thickness of the core shear walls are 32” (81.3cm) between the 1st (P4) level and 32nd level, 28” (71.12cm) up to the 55th level and 24”(61.0cm) above the 55th level. The wall-to-floor area percentages change from 2.4 to 3.9 %. Details of typical plans of several levels are shown in Figure 2 (also adopted from Çelebi et al. [1] and www.strongmotioncenter.org, last visited 27 January 2016).
2) It is the tallest building (188.3 m [617.83 ft]) in the United States designed that uses PBSD procedures.
3) The BRBs qualify the building to be the tallest PBSD in the world that uses BRBs. Also, it is the first building in California to have two TSDs.
4) The building sits on a 3.66 m (12 ft) mat foundation on San Francisco’s Rincon Hill– in very close proximity to the west (San Francisco-side) anchorage of the two suspension bridges of the Bay Bridge system.
5) The state-of-the-art, real-time recorder is capable of continuous streaming and has a buffer from which it is possible to retrieve select lengths of ambient and/or seismic response data.

Some of the significant findings reported in the 2013 study [1] are:

1. At low-amplitude ambient excitations, no effects of the BRBs or TSDs were observed in the responses.
2. At low-amplitude ambient shaking:
    a. The first modal frequencies computed using spectral analyses and system identification methods (*e.g.* NS [0.29Hz and 0.30 Hz], EW [0.28Hz and 0.27Hz] and Torsion [0.70Hz and 0.70 Hz], respectively) compare well.
    b. The first modal damping percentages obtained by system identification method are only: 0.9% (NS), 0.3-0.9%(EW) and 0.4% (Torsional). These are considered to be very low damping percentages but not abnormal for ambient data.

The recorded responses of the building to the South Napa EQ, at 48.1 km epicentral distance from the building, recorded by all 72 channels of its seismic monitoring array, are the largest to date. The largest peak accelerations [a(g)] at ground level and within the building were .005g and .021g (for CH36 at the 64th level), respectively. The largest displacement was 1.69 cm (CH36) [www.strongmotioncenter.org, last visited January 27, 2016]. The South Napa EQ acceleration amplitudes are approximately an order of magnitude larger than those of the 2015 Fremont earthquake (one of the other three earthquakes following South Napa EQ. - – see event 4 in Table 1), which in turn are another order larger than the 2012 ambient data used in this study.



Level	H(m)	H(ft)	Channel Numbering (used in analyses) NS	EW1	EW2
1	0	0	37	38	6
5	12.34	40.5	7	8	9
7	16.71	54.83	10	11	
8	20.41	67	39	40	41
12	32	105	42	43	
13	34.9	114.5	50	51	52
18	49.38	162	12	13	14
19	52.27	171.5	44	45	
20	55.17	181	53	54	55
24	66.75	219.66	56	57	58
28	78.33	257	15	16	65
30	84.73	278	66	17	
32	91.13	299	18	19	20
36	103.72	337	59	60	61
41	117.7	384.5	46	47	
42	120.1	394	21	22	23
43	122.99	403.6	48	49	
48	137.46	451	62	63	64
51	146.46	480.5	24	25	67
53	152.55	500.5	68	26	
55	159.46	523.17	27	28	29
56	162.67	533.67	69	30	
61	179.22	588	70	71	72
62	185.21	607.83	31	32	33
64	188.31	617.83	35		36

(c)

Fig.1- (a) Picture of the building and a model of its core skeleton with outrigger columns and attachment of BRBs to the core. (b) Vertical sections of the building showing locations of the accelerometers along the height of the building (www.stongmotioncenter.org, last visited January 28, 2016). Red and Green colors refer to channels installed by the CSMIP and USGS NSMP, respectively. (c) Table showing accelerometer channel information with H (m) above basement level.

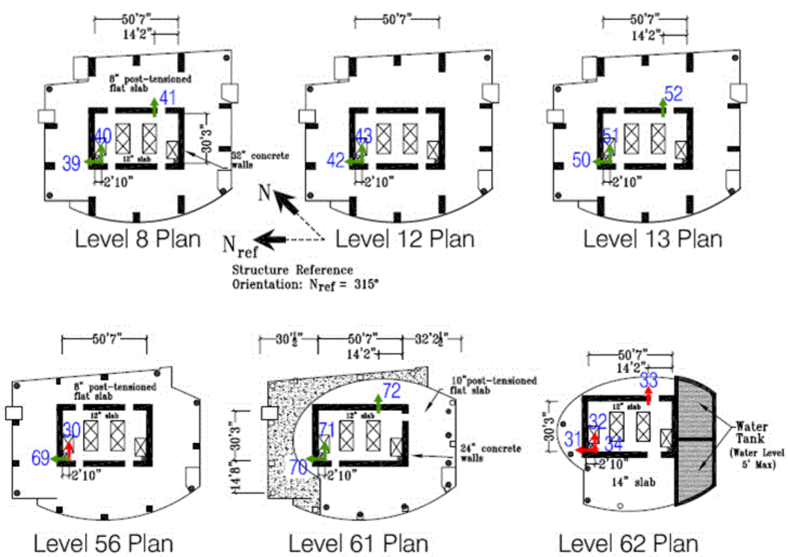


Fig. 2 - Typical plan views exhibiting sensor locations (green and red arrows), general dimensions and the core shear wall and outrigger columns (www.stongmotioncenter.org, last visited July 29, 2012). Note the building north reference direction (N_{ref}), which is termed NS in this paper. The thickness of core shear wall is 32" between Levels 1 and 32, 28" between Level 32 and Level 55 and 24" between Level 55 and Level 64 (Roof).

The main objective of this paper is to study the South Napa EQ response records and those from three other earthquakes as well as pre-South Napa and post-South Napa ambient data sets of the building to compare the major dynamic characteristics identified from the earthquake records with ambient response data. The pre-South Napa ambient data were analysed to identify modes and associated frequencies and damping [1]. Not unexpectedly, the low-amplitude dynamic characteristics are considerably different than those used during design analyses of the building. Thus, the different acceleration levels of the earthquake and ambient responses provide an opportunity to more fully evaluate the dynamic characteristics of the building.

The analyses results serve as a baseline against which to compare future stronger shaking responses. It is documented that, in the next 30 years, there are 18% and 26% probabilities for occurrence of a Mw 6.7 or larger earthquake on the northern and southern sections of the Hayward faults, respectively (Field et al. [2] and , *pers. comm*. D. Schwartz , 2015), which are located within 50 miles of the building. In addition, studies such as this one help to improve our understanding of the effectiveness of the response modification features at various levels of shaking, to evaluate the predictive capabilities of the design analysis tools, and to help improve similar designs in the future.

In this study, we use spectral analysis techniques (amplitude spectra and spectral ratios) and system identification methods to extract mode shapes and associated frequencies and damping, as described in Bendat and Piersol (1980) [3] and Ljung [4] and coded in the software, MATLAB [5]. Finite element model (FEM) analyses were not performed. Results from FEM analyses performed by the designers were reviewed in the previous study [1].

2.0 Data Description and Analyses

2.1 Data organization and significant characteristics

Since there were no nearby earthquake records available until the 24 August 2014 South Napa event, the analyses in the 2013 paper [1] were based only on ambient data acquired on demand from the buffer of the continuous streaming-capable recorder. A summary of the particulars of the eight sets of data used in this study is provided in Table 1.

Table 1 - Data sets used in this study (Building Coordinates: 37.7858N, 122.3921W), [d=epicentral distance (km), L=record length (s), sps= samples per second, D=hypocentral depth (km)] (Data from www.strongmotioncenter.org, last visited February 5, 2016 and NSMP Data Center at www.earthquake.usgs.gov).

Identifier	Data	Date	Coordinates	d (km)	L (s)	sps	D (km)
e1	Pre-South Napa Ambient	6/04/, 2012	-	-	120	100	-
e2	South Napa EQ (Mw6.6)	8/ 24/ 2014	38.22N, 122.31W	48.7	300	200	11.0
e3	Post-South Napa Ambient	12/12/2014	-	-	300	200	-
e4	Fremont EQ. (Mw4.0)	7/21/ 2015	37.58N, 121.97W	35.2	300	300	8.0
e5(2AM) e6(2PM)	Post-Fremont Ambient	8/15/2015	-	-	300	200	-
e7	Piedmont EQ (Mw4.0)	8/17/2015	37.84N, 122.23W	15.1	300	200	4.7
e8	San Ramon EQ (Mw3.6)	10/19/2015	37.79N, 121.96W	7.4	120	200	8.5

2.2 Time-history amplitude comparison of South Napa EQ with smaller response data

Sample relative amplitude time histories of one ambient (June 4, 2012) and two earthquake data sets (South Napa EQ and Fremont EQ) are shown in Figure 3. Each time history is plotted with a different vertical scale to compare the relative amplitudes of shaking. Clearly the recorded South Napa EQ response is the strongest to date of this building. Additionally, the time-histories for the South Napa EQ display beating effects (addressed later in the paper).

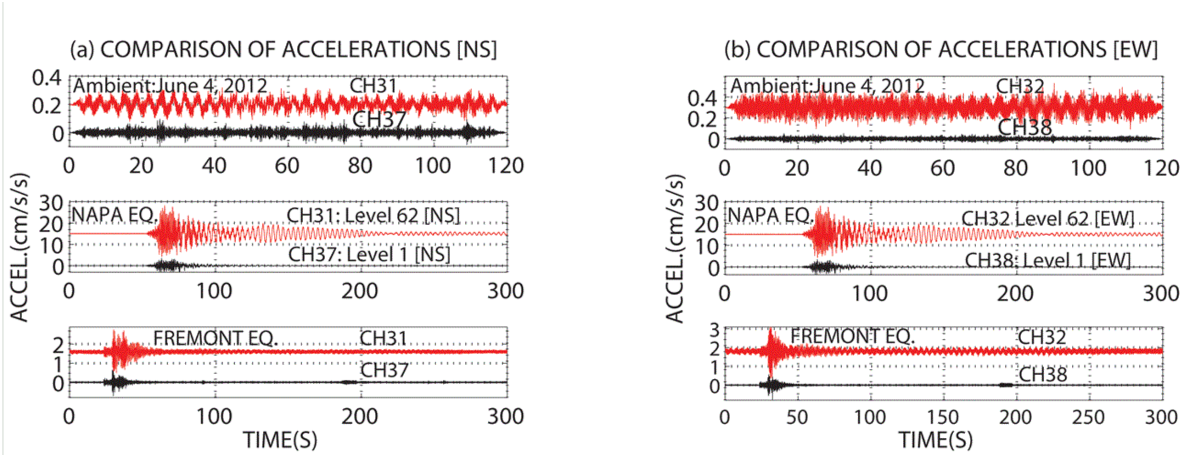


Figure 3. Equally scaled for each data set, time-history of accelerations are compared for pre-South Napa ambient (June 4, 2012), South Napa EQ and Fremont EQ (July 21, 2015) for (a) NS [CH31 at 62 level and CH37 at 1st level] and (b) EW [CH32 at 62nd level and CH38 at 1st level]. The figures indicate that, although small in amplitude, South Napa EQ accelerations are ~ 150 times the ambient accelerations.

Since there are considerable data acquired within the approximately four year period between 2012 and 2015, we organized our analyses as follows: (1) Only for the South Napa EQ data set, we (a) compare the amplitudes of NS, EW and torsional motions at the top floors and basement, (b) perform system identification analyses, (c) extract and plot mode shapes, (d) compute average drift ratios and (f) comment on beating effects. Such comparison is not repeated for other earthquake or ambient data sets. (2) For all eight data sets, we compute spectral ratios from amplitude spectra of accelerations at the roof with respect to that in the basement. (3) We compare and discuss the frequencies and damping percentages identified from earthquake and ambient data sets.

3.0 South Napa Earthquake Data and Analyses

Both of the ambient data sets as well as the South Napa event data set used in this study were retrieved from the buffer of the continuous streaming system. In this paper, we concentrate mainly on detailed study of the South Napa EQ. data. Following that, we compare the significant dynamic characteristics with the pre and post-South Napa ambient data. In comparisons, we use only selected spectral ratios of the three excitations.

3.1 Comparison of South Napa EQ time-histories of accelerations at top levels with those at the basement

Figure 4 shows equiscaled (a) NS, (b) EW and (c) torsional acceleration time-history plots each for the 61st, 62nd and 1st levels. Thus, relative amplitudes in accelerations are displayed. Note the beating effect at the 61st and 62nd level accelerations in the NS and EW directions – mostly due to structural factors such as low damping, as identified later in the paper. There are significant differences between the 1st level and the 61st and 62nd level accelerations for the NS, EW and torsional directions.

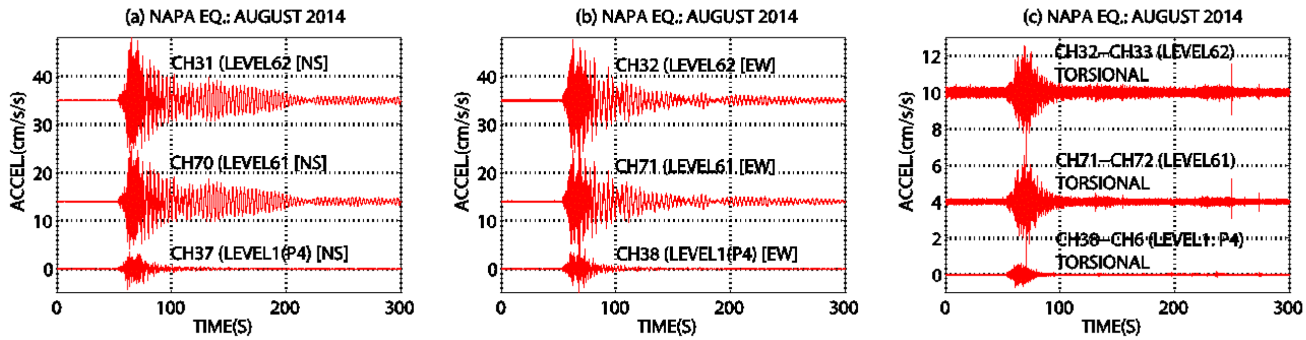


Fig. 4 - Equiscaled (a) NS, (b) EW and (c) torsional acceleration time-history plots for the 61st, 62nd and 1st levels. Note the beating effect at the 61st and 62nd floors in the NS and EW directions.

3.2 System identification and mode extraction using South Napa earthquake data

System identification method N4Sid within MATLAB [5] is used to extract modal frequencies, modal critical damping percentages (ξ) and mode shapes for the South Napa EQ. Details of this method are not repeated herein as they are provided in many other publications, including Ljung [4], Van Overschee and De Moor [6] and Juang [7]. For the first 3 modes the extracted frequencies and damping are tabulated in Table 2. It is noted that critical damping percentages (ξ) for the largest shaking (South Napa earthquake) data set are consistently lower than 2.2% for the first NS, EW1 and torsional modes. When the EW2 direction is included, ξ exceeds 3% for two of the EW2 modes. EW1 and EW2 refer to line-up of EW oriented accelerometers as shown in Figures 1c and 2. The cases below 3% of critical damping are consistent with findings of analyses of data from other tall buildings (e.g. a tall building in Osaka from the M9 2011 Tohoku earthquake shaking [8]). This observation is important because during the design process and development of design response spectra, generally the smallest critical damping percentage used is 5%. Lowering the damping from 5% to 3 % can result in more conservative design.

Table 2 - Modal frequencies and modal damping percentages of the building extracted by system identification from South Napa EQ data.

	Modal Frequencies (Hz)				Modal Damping ξ (%)			
	NS	EW1	EW2	TORSION	NS	EW1	EW2	TORSION
Mode 1	0.29	0.26	0.26	0.68	1.2	2.2	4.1	0.8
Mode 2	1.27	1.11	1.25	1.98	1.4	1.2	6.02	2.8
Mode 3	2.62	2.45	2.65	3.67	.46	1.9	1.8	1.2

The mode shapes extracted are shown in Figure 5. Frequencies and damping for each mode are shown within each frame. The mode shapes are as can be expected and do not indicate abrupt changes due to the presence of the BRBs or slosh dampers. This means that these dynamic response modification features (BRBs and TSDs) did not alter the response to the point where we observe significant changes at these levels in the recorded input and output responses. This observation is reinforced by average drift ratios and mode shapes that do not indicate distinct variations, as shown below.

3.3 Drift Ratios

For the South Napa EQ response record, average drift ratios are computed from displacements and relative displacements as displayed in Figure 6: (i) displacements at basement, outriggers and the roof for NS and EW directions respectively in Figures 6a and 6b, (ii) relative displacements between roof and basement, higher and lower ends of each outrigger in Figures 6c and 6d,and (iii) average drift between roof and basement, higher and lower ends of each outrigger in Figures 6e and 6f. The largest average drift ratio is ~ 0.01% - a level expected not to cause damage to the building.

It is seen in Figures 6e and 6f that, when compared with overall average drift ratios, the average drift ratios for the floor levels between the top and lower outriggers where BRBs are installed are not significantly different. As in the mode shapes, this is an indication that, at the level of motions caused by South Napa EQ, the BRBs did not alter the dynamic characteristics of the building.

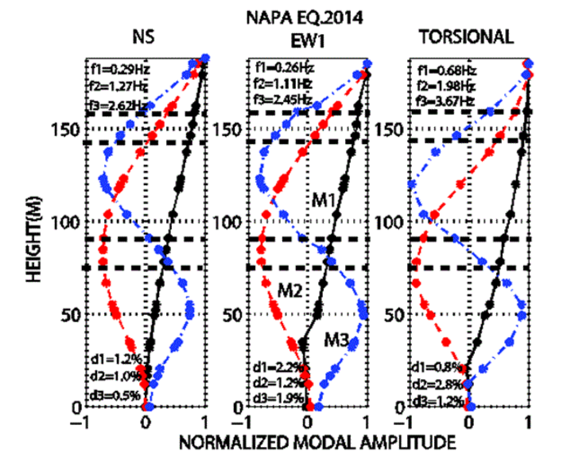


Fig.5 - Three mode shapes and associated modal frequencies and damping percentages extracted from NS, EW and torsional accelerations. Heavy black dashed lines show levels where BRBs are connected to the outriggers and the core. The solid symbols on the mode shapes indicate that at the corresponding elevation there are accelerometers and therefore data. M1 (black), M2 (red) and M3 (blue) refer to modes 1, 2 and 3, respectively.

3.4 A Note on Beating Effects

Beating effects, observed in several building response records in the past, occur when translational and torsional frequencies are close to one another and the structural system has low damping [9 -13]. Also, beating effects may be one of the reasons for elongated durations of “replenished” shaking when repetitively stored potential energy during coupled translational and torsional deformations turns into repetitive vibrational energy. Thus periodic, repeating and resonating motions ensue. The beating becomes severe if the system is lightly damped. The beating effect period (Tb) is computed using the relationship: Tb=2T1Tt/(T1-Tt) given by Boroschek and Mahin [10]. In this relationship, T1 and Tt are fundamental translational and torsional periods, respectively. In this case, if T1=3.45s (f1=0.29Hz, T1=1/f1=3.45s) and Tt=1.47s (f1=0.68Hz, Tt=1/.68), Tb is computed to be around 5 seconds, but visual observation (Figure 6) indicate much larger beating periods (~30-40s). It is possible that ~5s beating may occur during stronger shaking. Thus, we conclude that computed beating periods are not consistent with visually observed ones. Such differences are observed in other cases [13]. Nonetheless, the main point is that beating occurs in this building as evidenced by the South Napa records from continuous data. This is important to note as such beating effects prolong the responses and therefore increase the number of large and small cycles of responses. Even the increased number of smaller amplitude cycles could become important due to possible low-cycle fatigue that can result in nonlinear behaviour at joints.

4.0 Normalized Spectral Ratios (for all 8 events)

As summarized earlier in Table 1, the eight data sets from events e1-e8 include four earthquake and four ambient acceleration sets acquired over three years. All eight data sets are used to explore whether there are significant variations in frequencies of the building due to wide variations in amplitude of shaking between earthquake and ambient data sets. Large variations of frequencies may imply nonlinear behavior and/or also may indicate the effect of dynamic response modification features (BRBs and TSDs) in shifting the frequencies. Two of the four ambient data sets were acquired in a 12-hour time frame on August 15, 2015 to further explore any presence of temporal variations of frequencies within one day. Normalized spectral ratios for all data sets are presented in Figure 7. Modal frequencies are extracted from the spectral ratios by peak picking.

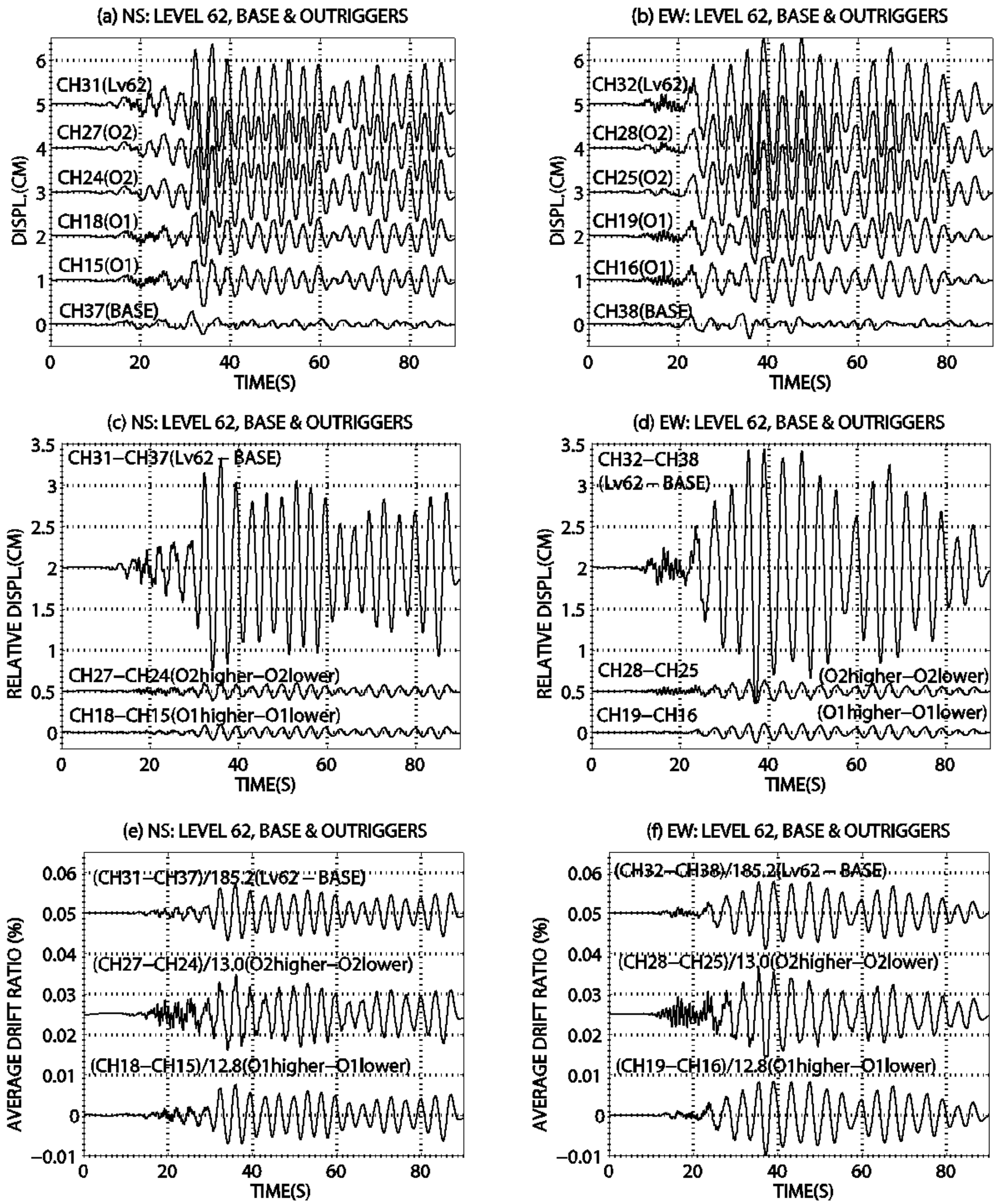


Fig. 6 - For NS and EW directions respectively in a and b, displacements at base, outriggers and the roof; in c and d, relative displacements between roof and basement, higher and lower ends of each outrigger, and in e and f, average drift between roof and basement, higher and lower ends of each outrigger.

Figure 7 shows spectral ratios of amplitude spectra (0-5Hz frequency band) computed using the 62nd and P4 (basement) level accelerations of the eight data sets and for each of the (a) NS [CH31/CH37], (c) EW [CH32/CH38] and (e) torsional [(CH32-CH33)/(CH38-CH6)] directions, respectively. Similarly, 0-2 Hz frequency band versions are provided in Figure 7 for (b) NS, (d) EW and (f) torsional directions, respectively.

These figures indicate that the frequencies identified from amplitude spectra and spectral ratios compare well, with no significant differences except those possibly caused by small errors in peak picking. Similarities without significant variations imply lack of nonlinearity due to (a) soil-structure interaction, (b) shaking not large enough to activate the BRBs and TSDs, and/or (c) other material nonlinearities. Naturally, it is expected that during shaking amplitudes larger than experienced to date the BRBs and TSDs will alter the frequencies and increase damping percentages. Since the observed differences are small, particularly for the fundamental NS, EW and torsional modes, any plot of variation of the frequencies versus shaking amplitudes would not be useful. On the other hand, large variations have been found from sets of data from other buildings (e.g. Loma Prieta data sets versus ambient data sets for Pacific Park Plaza in Emeryville, CA [14] and other buildings [15]).

5.0 Discussion of results of frequencies and damping for 8 events

All results determined from the spectral ratios and by system identification are summarized in Table 3. Even when small, the variations of frequencies and damping among different events are evident in the table. Insignificant variations of dynamic characteristics exist between those determined from earthquake and ambient motions. First modal damping percentages are all <1.5%, which is a significant result since normally a 5% damping percentage is used during design. The low damping is most likely the cause of beating effects described earlier. Note that the system identification results are only for the South Napa EQ.

Table 3. Identified dynamic characteristics for e1-e8 (this sudy) and comparison with previous study with only ambient excitation [1]. [T=Torsion, M1=mode 1, M2=mode2, M3=mode3]. (Note: * Mode 2 for e3 [South Napa EQ] ~1.15 Hz and those from other events are closer, **2.45 Hz for Mode 3 is from system ID for e3 [South Napa EQ].

(a) This study	Frequencies (Hz) (Period [s]): Obtained for all events (e1-e8) and by system identification for only e3 (South Napa EQ).			Damping (%): Obtained only for e3 (South Napa EQ) and only by system identification (from Table 4).		
	NS	EW	T	NS	EW	T
M1	.29 (3.45)	.26-.27 (3.7-3.85)	.68 (1.47)	1.2	1.4	.46
M2(*)	.7-.87 (1.15-1.30)	.89 (1.12)	2.01 (~.5)	2.2	1.2	1.9
M3(**)	2.86 (.35)	2.45-2.65 (.38-.41)	3.65-3.75 (.27)	4.1	6.0	1.8
(b) 2014 Study	Frequencies (Hz) (Period [s]): Ambient data only results by spectral analyses and system identification. [1]			Damping (%): Ambient data only results by spectral analyses and system identification. [1]		
M1	.29-.30 (3.33)	.27-.28 (3.57-3.7)	.7 (1.43)	0.9	0.3-0.9	0.4
M2	.77-.78 (1.28-1.30)	.86-.88 (1.14-1.16)	2 (.5)	0.5	2.1-4.4	0.8
M3	2.83 (.35)	2.59-2.60 (.39)	3.73 (.27)	1.9	0.3	1.4

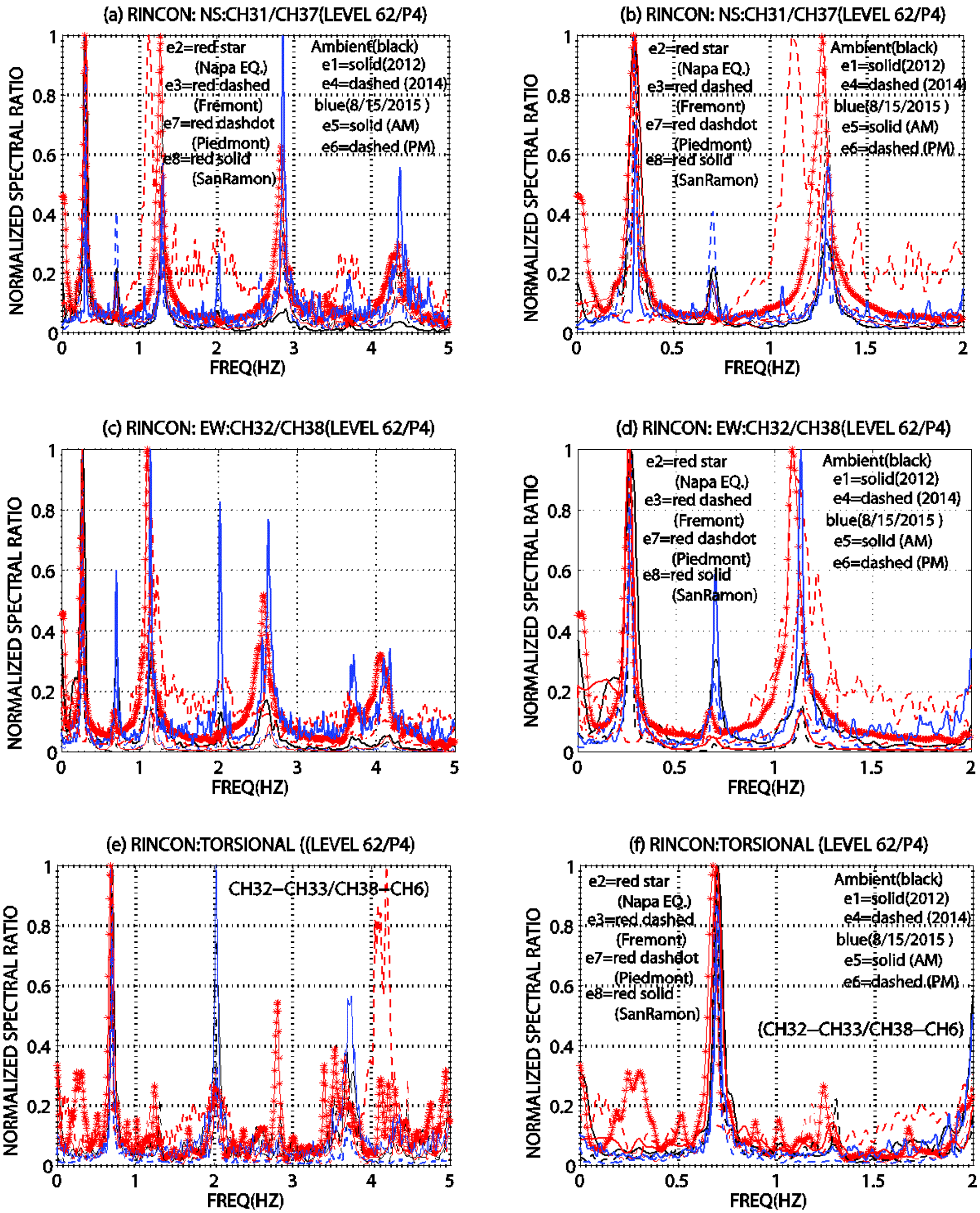


Fig. 7 - Comparison of 0-5Hz band normalized spectral ratios of amplitude spectra from 62nd level and 1st (P4) level accelerations for (a) NS[CH31/CH37], (c) EW[CH32/CH38] and (e) Torsional [(CH32-CH33)/(CH38-CH6)]. (b), (d) and (f) repeat the same for 0-2Hz frequency band.

6.0 Conclusions and Implications

Study of ambient and earthquake records from 72-channels of an array deployed in a 64-story core shear wall San Francisco building with response modification features indicate that:

1. There are small differences among the frequencies computed from ambient acceleration data compared with those from earthquake acceleration data - even though the earthquake data amplitudes are one or two orders of magnitude larger than the amplitudes of ambient data.
2. Critical damping percentages obtained by system identification using the South Napa EQ data with larger acceleration amplitudes are considerably lower than the 5% prescribed in most of the currently used codes (e.g. International Building Code [16]) and lower than even the procedures recently recommended by the Los Angeles Tall Buildings Design Council [LATBDC] [17] in which the maximum modal damping is recommended as 2.5%. However, it should be mentioned that the LATBDC document indicates that this maximum is an additional modal damping during nonlinear analyses and for primary modes of the tall buildings. The result from the South Napa EQ data set is consistent with results from studies of other tall buildings in California [18] and in Japan during the 2011 M9.0 Tohoku earthquake [19, 20].
3. At low amplitudes of excitation, the response modification features (BRBs and TSDs) do not appear to have altered the response characteristics (e.g. mode shapes, damping percentages and frequencies) of the building. The effectiveness of these modification features should be carefully evaluated from larger amplitude response data obtained during future earthquakes.
4. Average drift ratios computed for the total height of the building as well as at outrigger locations are similar, both in amplitude peaks and phases. They are small; hence no damage is inferred. There are no observable or computed nonlinearities in the behavior and performance of the building.
5. Beating effects are visually observed from the South Napa EQ records obtained by continuous recording. However, the beating periods do not agree with the estimation formula established by previous studies.

7.0 References

[1] Çelebi, M., Huang, M., Shakal, A., Hooper, J., and Klemencic, R (2013): Ambient response of a unique performance-based design tall building with dynamic response modification features, Wiley Online Library *Journal of the Structural Design of Tall and Special Buildings*, pp. 816-829, doi:10.1002/tal.1093.

[2] Field, E. H., Biasi, G. P., Bird, P., Dawson, T. E., Felzer, K. R., Jackson, D. D., Johnson, K. M., Jordan, T. H., Madden, C., Michael, A. J., Milner, K. R., Page, M. T., Parsons, T., Powers, P. M., Shaw, B. E., Thatcher, W. R., Weldon II, R. J. and Zeng, Y.,(2015): Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast (UCERF3), Bulletin of the Seismological Society of America, Vol. 105, No. 2A, pp. 511–543, April 2015, doi:10.1785/0120140093.

[3] Bendat, J. S. and Piersol, A. G. (1980): Engineering Applications of Correlation and Spectral Analysis. John Wiley and So

[4] Ljung, L. (1987): *System Identification: Theory and User*. Prentice hall, Englewood Cliffs, NJ.

[5] Mathworks (2013 and previous versions): *Matlab and Toolboxes*, South Natick, Mass.

[6] Van Overschee, P. and De Moor, B. (1996): *Subspace identification for linear systems*. Kluwer Academic Publishers, Dordrecht, The Netherlands.

[7] Juang, J. N. (1994): *Applied System Identification*, Prentice Hall, Upper Saddle River, NJ. 394 pages.

[8] Çelebi, M., Okawa, I., and Kashima, T., S. Koyama, and M. Iiba, (2014); Response of a tall building far from the epicenter of the March 11, 2011 M=9.0 Great East Japan earthquake and its aftershocks, *The Wiley Journal of The Structural Design of Tall and Special Buildings*, 23, 427–441 (2014). doi:10.1002/tal.1047

[9] Boroschek, R. L., Mahin, S. A., and Zeris, C. A. (1990): Seismic Response and analytical modeling of three instrumented buildings, in PROC. 4th U.S. National Conference on Earthquake Engineering, Palm Springs, Calif., v.2, pp.219-228.

[10] Boroschek, R. L., and Mahin, S. A. (1991): Investigation of the Seismic Response of a Lightly Damped Torsionally Coupled Building, Report *UCB/EERC-91/18*, Earthquake Engineering Research Center, University of California, Berkeley, 291 p.

[11] Çelebi, M.,(2004):Responses of a 14-Story (Anchorage, Alaska) Building to Far-Distance (M_w=7.9) Denali Fault (2002). and Near Distance Earthquakes in 2002, *Earthquake Spectra*, Journal of EERI, *Earthquake Spectra*, vol.20, no.3, pp. 693-706, August 2004.

[12] Çelebi, M., (2006): Recorded Earthquake Responses from the Integrated Seismic Monitoring Network of the Atwood Building, Anchorage, Alaska, *Earthquake Spectra*, Vol. 22, No.4, pages 847–864, November 2006.

[13] Çelebi, M., 2007, Beating Effect Identified from Seismic Responses of Instrumented Buildings. In New Horizons and Better Practices (*ed*. By R. Lyons): ASCE Structures Congress 2007, Long Beach, California, May 16-19, pp. 1-12. doi:10.1061/40946(248)14

[14] Çelebi, M., (2007): On the Variation of fundamental Frequency (Period) of an Undamaged Building – A Continuing Discussion, Int'l. Conf. on Experimental Vibration analyses for Civil Engineering Structures, Porto, Portugal, Oct. 24-26, 2007.

[15] Çelebi, M., Phan, L., and Marshall, R. (1993): Dynamic characteristics of five tall buildings during strong and low-level amplitude motions, *International Journal of Design of Tall Buildings*, John Wiley and Sons, Ltd, March 1993.

[16] International Building Code, 2012, *International Building Code*, Falls Church, Va.

[17] LATBSDC (Los Angeles Tall Buildings Design Council), 2015, An alternative procedure for seismic analyses and design of tall buildings located in the Los Angeles region: A consensus document, 54pp., August 14, 2015. (http://www.tallbuildings.org, last accessed February 16, 2016).

[18] Çelebi, M., Ulusoy, H., and Nakata, N., (2016): Responses of a tall building in Los Angeles, California as inferred from local and distant earthquakes (in print, *Earthquake Spectra).*

[19] Çelebi, M. , Kashima, T. , Ghahari, F. , Abazarsa, F. and Taciroglu, E. , (2016): Responses of a tall building with US code-type instrumentation in Tokyo, Japan to events before, during and after the Tohokuearthquake of March 11, 2011. *Earthquake Spectra.* Vol. 32, No. 1, pp. 497-522. doi**:**10.1193/052114EQS071M

[20] Çelebi, M., Yoshiaki Hisada, Y., Omrani, R., Ghahari, F., and Taciroglu, (2016): Responses of two tall buildings in Tokyo, Japan, before, during, and after the M9.0 Tohoku Earthquake of March 11, 2011.,*Earthquake Spectra*. Volume 32, No. 1, pages 463–495, pp. 463-495, doi:10.1193/092713EQS260M.

8.0 Disclaimer: Any use of trade, firm, or product names is for descriptive purposes only and does not imply endorsement by the U.S. Government.

9.0 Data Source: South Napa earthquake data from www.strongmotioncenter.org (last visited March 10, 2016), continuous monitoring earthquake and ambient data from (http://earthquake.usgs.gov/monitoring/nsmp/) (contact: GS-G-WR_ESC_NSMP@usgs.gov or cdstephens@usgs.gov).

10.0 Acknowledgements

The authors thank reviewers Dr. Nicolas Luco (USGS) and Dr. S. Farid Ghahari (UCLA) for their review of the draft of this paper.

11. Copyrights