THE BEHAVIOR OF THE SALESFORCE TOWER, THE TALLEST SAN FRANCISCO BUILDING, INFERRED FROM EARTHQUAKE DATA

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Abstract

The newly constructed Salesforce Tower building is the tallest building in San Francisco, California that is designed in conformance with performance-based design procedures. The Salesforce Tower has 61 stories and is equipped with an accelerometric array that recorded the January 4, 2018 M4.4 Berkeley earthquake. The building is designed with concrete core shear walls and perimeter gravity steel columns. The earthquake records, as well as on-demand recorded ambient responses of the building, are studied to determine its dynamic characteristics and building-specific behavior. At the level of shaking of either the earthquake or the ambient excitation, the frequencies and low modal damping ratios (<2.6%) are similar. During the Berkeley earthquake, the building exhibited torsional behavior, most likely due to abrupt asymmetrical changes in the size of the core shear wall. The translational and torsional modes experienced during the earthquake are closely coupled, which leads to a beating effect, the period of which is calculable. Due to the relatively low-amplitude shaking during the earthquake, the drift ratios were small and did not result in damage. It is expected that during stronger shaking levels, these characteristics may change.

Keywords: earthquake response, mode shape, frequency, damping, drift ratio

1. Introduction

The number of tall buildings in the United States and around the world is increasing with an awe-inspiring race for height. While displaying novel architectural features and structural design innovations, tall buildings pose challenges to innovative structural engineering design, analyses, construction materials, and construction techniques. Their designs are also expected and scrutinized to provide requisite dynamic behavior and performances during extreme events, such as strong winds, as well as shaking caused by earthquakes that originate from both near and far seismic sources. The designs must also comply with updated and redefined seismic hazards, associated risks to the built-environment, and acceptable performance criteria. The introduction of new construction materials, high-strength concrete and steel, as well as structural response modification features, generate new opportunities for enhanced performance-based design. While the behavior and performances of tall buildings can be assessed by computations and visual inspections, data-based evaluations are finding utilization by owners and related stakeholders. However, it is fair to state that monitoring the responses of tall buildings during strong-shaking events is very limited in number (e.g. less than 1 % of tall buildings are being monitored). It is therefore an important attribute that one of the new additions to the panorama of San Francisco, California – the Salesforce Tower- is not only the current tallest (~319 m [~1070 ft]) structure in the City by the Bay (as San Francisco is also known) but it is also instrumented by the California Strong Motion Instrumentation Program (CSMIP) of the State of California Geological Survey (CGS) and is known as Station #58680. In addition, this new building is designed to have a concrete shear wall core and perimeter steel gravity frames and columns. This type of design and construction is not new, but recently it has been used more often with taller buildings; hence, the motivation for this paper. This 61-story building offers the opportunity to study the response of tall buildings to seismic (and, when needed, ambient) shaking. In particular, the response of the Salesforce Tower was recorded during the M4.4 January 4, 2018 Berkeley earthquake [02:39:37 PST, 37.8552N, 122.2568W, depth 12.3 km]. A Photo of the building is provided in Fig. 1a.

The purpose of this paper is to present the study of the responses of the building to the M4.4 Berkeley earthquake of January 4, 2018. The motivation for the study is borne by the recent ever-increasing shear wall core and perimeter framed type design and construction in major cities of the world (e.g., San Francisco, New York City, and Santiago). Availability of recorded response data of such a building makes it imperative to study and to understand its behavior and performance. The scope of this earthquake response study is limited by the incomplete data set, due to a few missing channels at top levels of the building, which were not installed at the time of the earthquake. A vertical channel at the P3 level did not record properly, which unfortunately, prevents computation of rocking using that key vertical channel data.

1.1 The building, seismic monitoring and site transfer function

The most visible new landmark in San Francisco, California, is a 319-m (1070-ft) tall, 61-story[1] building that includes an additional three parking levels. The building is a reinforced concrete core structure with perimeter steel columns and concrete slabs, supported by steel beams, walls, and columns. Base dimensions are 82.9 x 56.1 m (272 x 184 ft). A typical floor is ~50.9 m x 50.9 m (~167 ft x 167 ft), an assumed square in plan from Levels 1 to 26, that tapers to 48.5 m x 48.5 m (159 ft x 159 ft) at the 50th level, to 46.0 m x 46.0 m (151 ft x 151 ft) at the 56th level, to 41.1 m x 41.1 m (135 ft x 135 ft) at the 62nd level, to 38.4 m x 38.4 m (126 ft x 126 ft) at 64th level (roof), and finally to 27.7 m x 27.7 m (91 ft x 91 ft) at the top of the building.

The lateral force-resisting system is described as “special concrete core shear walls”, with thickness varying from 1.07 m (42") at the basement (or level P3) to level 15, 0.91 m (36”) between levels 15 and 39, 0.76 m (30”) between levels 39 and 50, and 0.61 m (24”) between levels 50 and 64 (roof). There is no core shear wall between level 64 (roof) and the top of the building (which is considered as the 30.52 m (100 ft 1inch) high “cap of the building.” It is important to note that north half of the core shear wall stops at level 50. The remaining south half of the core shear walls stop at the roof level (level 64) at ~293 m (961 ft) above grade. The tower is crowned with 46.5 m (152.5 ft) tall ordinary steel concentrically braced frames, supported at the top of concrete walls at level 64 (roof) and perimeter steel columns at level 62. Above the roof level, there are no shear walls, but the ~30.5-m- (~100 ft) high truss “cap” structure is stiff compared to the ~300 m structure system below it, and the cap structure acts as a stiffer appendage on the less stiff 64-story- (~300 m) building [1-3].

According to Klemencic et al. [1,2], the lateral force design of the building was governed by seismic loading and not wind loading. Essentially, three levels of ground shaking have been considered. Structural design was based on the 2010 San Francisco Building Code and performance-based seismic design procedures: (a) Elastic performance targeted for service-level shaking [SLE] (with a mean recurrence interval of 43 years), (b) Moderate structural damage expected for design-level shaking [DBE] (taken as ~ 2/3 of code-defined Maximum Considered Earthquake [MCE] shaking), and (c) Collapse prevention, with reduced probability of collapse consistent with Occupancy Category III targeted for MCE shaking [1,2].

[1] The building is designated as 61 stories. However, the designated roof is at the 64th level (top of concrete core). On top of the roof, there is a ~30.5m (100ft) high “cap” of the building designed and constructed with steel braced structural system.

Fundamental frequencies computed during SLE and DBE design/analysis processes, combined with those determined from earthquake data, are compared later in this paper.

Core shear walls are the most important skeletal part of this and similar building systems. Using the dimensions from the plan views in Figs.1b,c,d, as well as those at www.strongmotioncenter.org, conservative average ratios of the core shear wall area of a typical floor to the total floor area of the same floor of the building have been computed to be between 1.4-3.95%, smaller percentages at the upper and larger percentages at the lower levels [4]. Thus, with the smallest core shear wall ratio (~1-4 %), this building has comparable or higher shear wall area percentages when compared with the common practice in Chile, where core shear walls are widely applied in design and construction [5,6,7].

It is important to note that the discontinuity that half of the core shear wall imposes an asymmetry of the lateral stiffness, and this discontinuity makes the building more susceptible to torsional behavior and modes, in addition to the translational modes. In Fig.2b, the variation in size and thickness that are in concert with the decreasing square in-plan dimensions from base to roof (64th level) and discontinuity of one-half of the core shear wall above the 50th level are exhibited.

Fig.1b shows a schematic of a vertical cross section with dimensions and the distribution of deployed accelerometers throughout the building. Fig.1c shows another vertical cross section, depicting the abrupt change in the core along the height of the building, with a discontinuity of half of the core shear wall above the 50th level and a total discontinuity above the 64th level. Fig. 1d shows typical plan views with information about the changes in thicknesses of the core shear walls along the height of the building. The changes of the thicknesses along the elevation of the south wall of the core are also shown Figure 1b.

Figures 1b,c and d describe the structural monitoring array and how they are deployed (both locations near the core shear wall and orientations, as well as sample numbering system of accelerometers at various levels). These numbers are key to understanding the recorded data and analyses to follow. The accelerometric array is comprised of accelerometers at 10 instrumented levels. A complete set of plan views is available at www.strongmotioncenter.org (last accessed April 30, 2019).

Details of site effects and site transfer function computations using the site's depth-*Vs* profile (to 84 m, ~275 ft) from borehole data yields a site frequency of ~0.8 Hz. When depth-*Vs* profile from the U.S. Geological Survey (USGS) Bay Area 3-D Seismic Velocity Model [4] is considered, a site frequency range of 0.20-0.25 Hz is obtained – which is well in the range of the structural fundamental frequency of ~50- to 60-story-tall buildings. Details of the site transfer function computations are in [3].

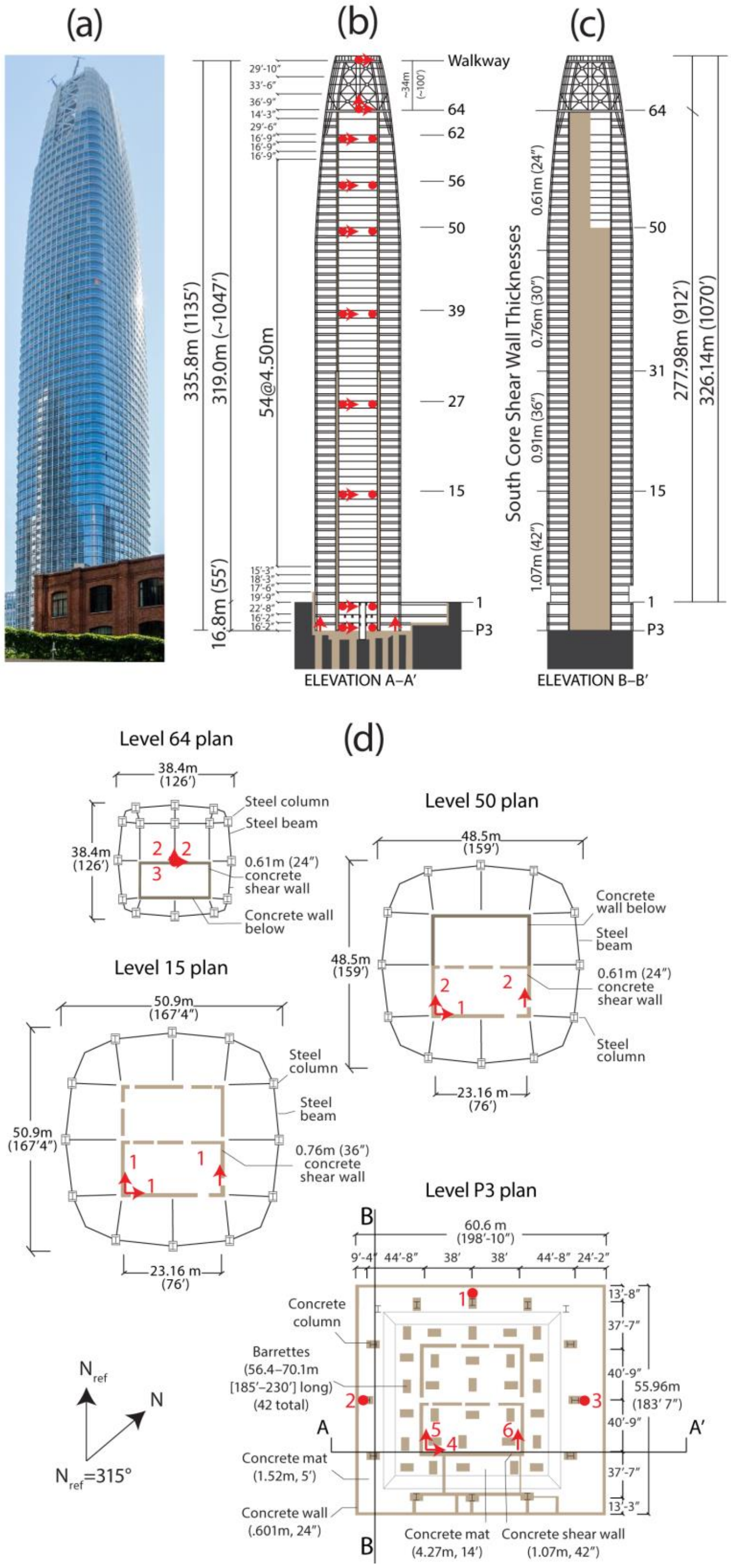

Fig. - 1. (a) Picture of Salesforce Tower in San Francisco, California, (b) East-west vertical cross section depicting general dimensions and levels at which accelerometers are deployed, (c) North-south vertical cross section depicting discontinuity of the core-shear wall, and (d) typical plan views (Figure modified from [3]).

Table 1 - Distribution and number labeling of channels along the height (and floor level) of the building

Level	Height, H (m)/[ft]	NS1	NS2	EW	UP	Center	
						NS	EW
P3	0	5	6	4	1,2,3(+)		
1	16.764 [55'0"]	8	9	7			
15	83.294[273'3"]	11	12	10			
27	137.244[450'3"]	14	15	13			
39	186.698[627'3"]	17	18	16			
50	236.152[789'6"]	20	21	19			
56	263.127[878'0"]	23	24	22(*)			
62	291.927[972'6"]	26	27	25			
64	305.262[1016'3"]				30(**)	29 (**)	28(**)
Top	335.780[1161'4"]					32(**)	31(**)
At the time of the earthquake, (*) channel 22 did not record, (+) channel 3 did not record properly, (**) channels not installed							

2. Earthquake Response Data Analyses

At the time of the M4.4 January 4, 2018 Berkeley, California earthquake, most accelerometer channels (27 of 32) were installed – covering levels P3 to level 62 (Figures 2a and 3). This data set is the first earthquake response recorded from this unique building. The largest peak accelerations recorded during the earthquake are summarized in Table 2.

Table 2. Largest peak accelerations recorded during the earthquake and ambient vibration.

	Largest peak accelerations (cm/s/s) [M4.4 January 4, 2018 Berkeley Earthquake]							
	Top		**Level 62**		**Level 1**		**Level P3**	
	NS	EW	NS	EW	NS	EW	NS	EW
Channel	32	31	26	25	8	7	5	4
Acceleration	-	-	29.63	14.29	11.59	9.21	11.69	8.96

Fig. 2 shows acceleration time histories of the horizontal (NS and EW), as well as torsional, channels represented by the difference between two NS channels on one floor. Similarly, Fig. 3 shows the corresponding displacement time-histories. The original records are 61 seconds long. However, to better visualize them, the time-history plots in both Figures 6 and 7 show the 28-61 second windows of the records. This helps to visualize, at least in the NS and EW displacement time histories, that there are about 4 cycles of longer periods that occur in a 20-second window (indicating a first mode at ~ 5 seconds, superimposed by ~4 cycles of approximately 1.25 second period, indicating a second period on each one of the single 5-second cycles). Similarly, from the displacement time history for torsional response, we can say that there is ~1 cycle within a ~15-second response (possibly a beating period of 15 s, as will be addressed later in the paper).

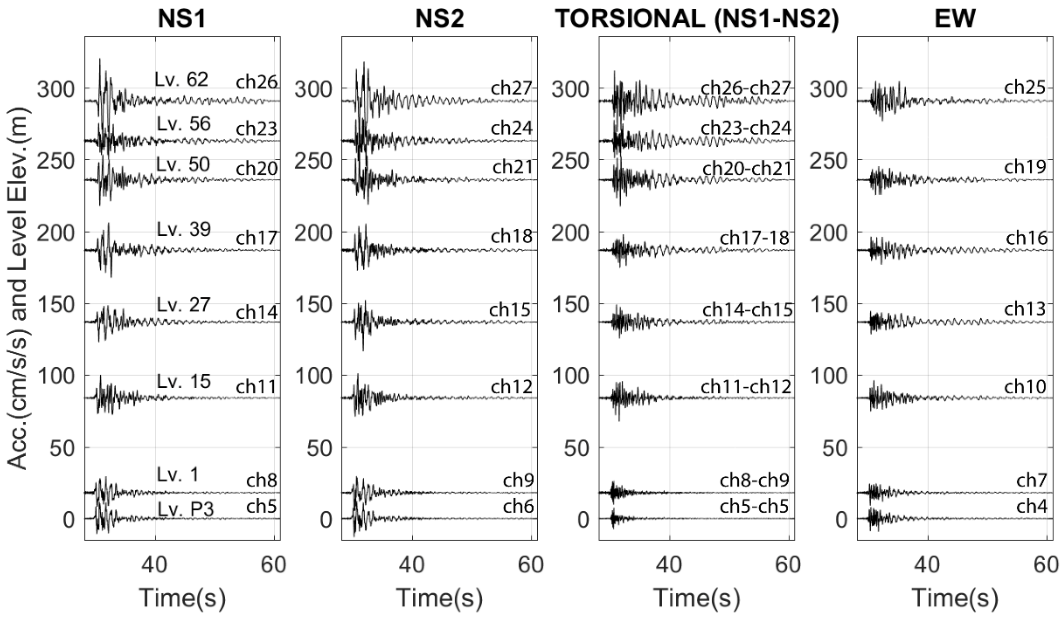

Fig. – 2. Acceleration time-histories of horizontal channels plotted against the level, elevation, and corresponding height of the building and according to the line-up summarized in Table 1. Plots are truncated to a 28- to 61-s window of the total 61-s record to better visualize the signals. Note the vertical axis is used for both amplitude of acceleration and elevation.

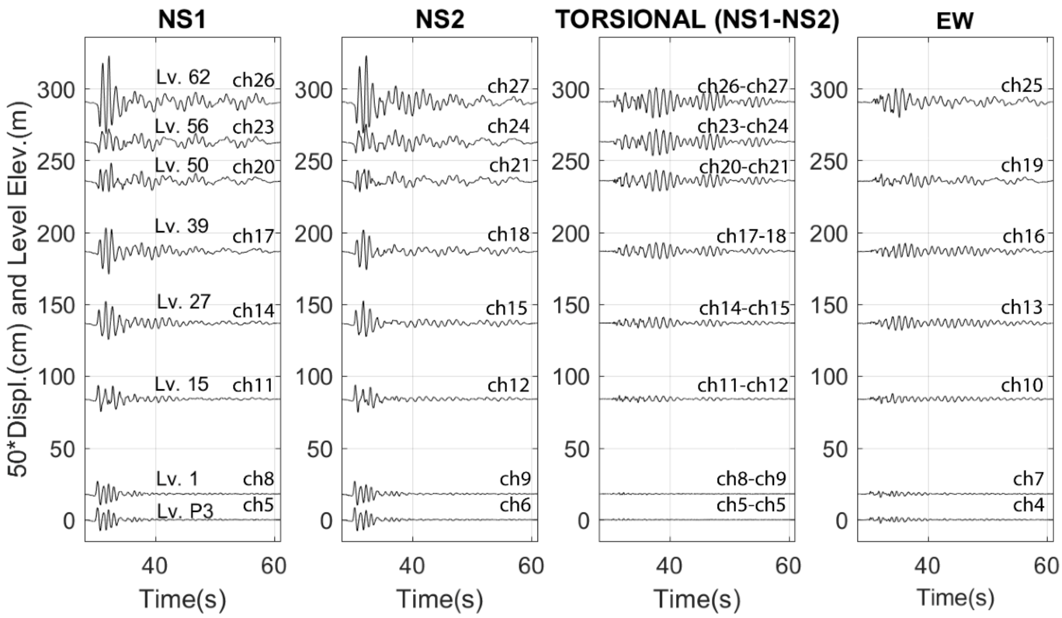

Fig. -3. Displacement time-histories of horizontal channels plotted against the level, elevation, and corresponding height of the building and according to the line-up summarized in Table 1. Plots are truncated to a 28- to 61-s window of the total 61-s record to better visualize the signals. Note the vertical axis is used for both amplitude of displacement (amplified by 50) and elevation.

2.1 Relative displacements and average drift ratios

Even though shaking during the Berkeley earthquake did not occur at a damage-causing level, as a routine check, it is useful to quantify average drift ratios as one of the parameters that can be extracted from the records to informally state that there was no damage during the event.

For the Salesforce Tower, the maximum relative displacements occur between the 50th and 62nd levels – understandably, since the core shear wall area is reduced to approximately half, and the thickness of the shear wall is reduced to ~61.0 cm (24”) from (91.4 cm (30”) below 50th level. Fig. 4a shows a plot of relative displacements between the 62nd and 50th levels, with a peak of ~0.75 cm in the NS direction. Fig. 4b shows the computed drift ratio between the 62nd and 50th levels. Fig. 4c shows a plot of relative displacements between the 62nd and 56th levels that results in the larger drift ratio between 62nd and 56th levels (Fig. 4d), which occurs because of the smaller difference in level elevations. Thus, this translates into a peak drift ratio of ~0.015%, which is too small to cause damage.

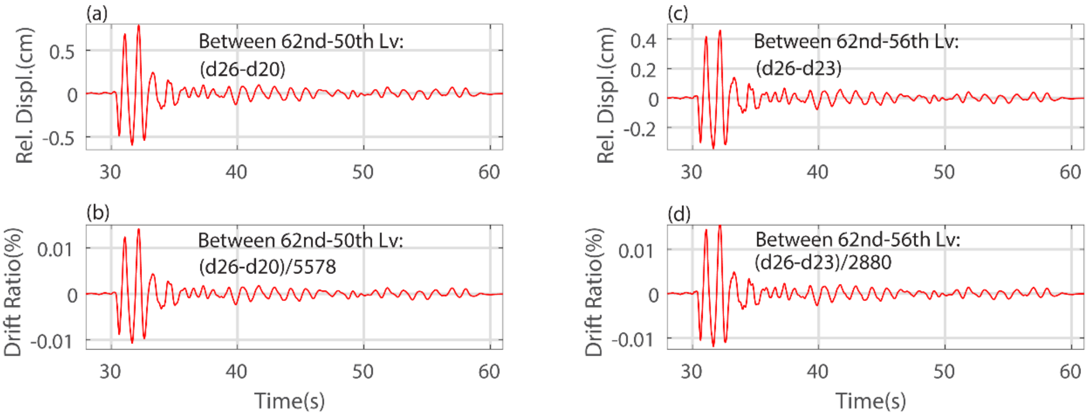

Fig. - 4 (a) Relative displacements and (b) drift ratios experienced between the 62nd and 50th levels of the Salesforce Tower (with the caveat that there are no earthquake response data above level 62). (c) Relative displacements between 62nd and 56th levels. (d) Drift ratios between levels 62nd and 56th.

2.2 Amplitude spectra and spectral ratios

In Fig. 5, amplitude spectra of NS, EW, and torsional displacements are shown for all available data. For each direction, each spectrum is shifted in the vertical axis to display the stacked peaks. The translational NS and EW spectra clearly display the low-frequency peaks at ~0.2 Hz, as well as a second-mode frequency around ~0.80-0.90 Hz. This second-modal frequency can also be seen in the torsional spectra.

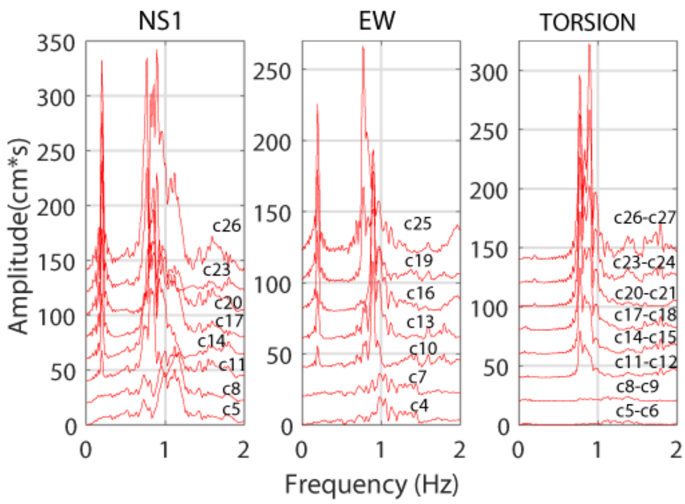

Fig. – 5. Amplitude spectra of displacements in the NS, EW, and torsional directions. Using displacements, lower fundamental NS and EW frequency peaks are better identified. The peaks around 0.8-0.9 Hz in the NS, EW, and torsional directions are similar and infer coupling of second NS and EW modes with the first torsional mode.

Fig. 6 shows ratios of amplitude spectra of displacements on all instrumented levels and in the NS, EW, and torsional directions. Again, it is observed that the frequencies for translational NS and EW fundamental modes are identifiable as ~0.2 Hz, for the second modes as ~ 0.8-0.9 Hz, and the torsional modes as ~ 0.8-0.9 Hz. Thus, we observe close coupling between the second translational mode and first torsional mode.

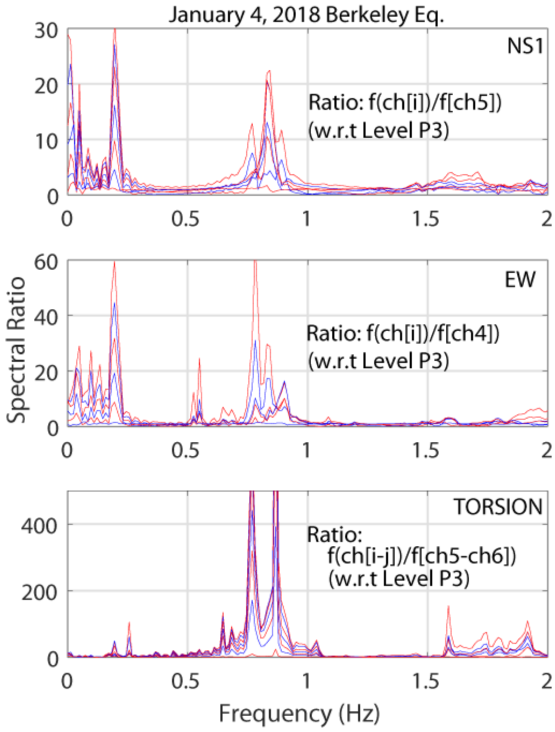

Fig. - 6. NS, EW, and torsional spectral ratios computed from amplitude spectra of displacement of channel *i* in direction x at any instrumented level from top to bottom with respect to amplitude spectrum at level P3 in direction x.

2.3 Modes and mode shapes extracted by system identification

As noted earlier with spectral ratios, success in obtaining fundamental modal frequencies or damping at ~ 0.2 Hz was not possible using acceleration data. Therefore, we also used displacements for system identification.

The system identification method, known as Numerical Algorithms for Subspace State Space System Identification (N4SID) within MATLAB [8], is used to extract modal frequencies, modal critical damping percentages (ξ), and mode shapes. Further details of the background of this method are not repeated herein, as they are provided elsewhere, including Juang [9], Van Overschee and De Moor [10], and Ljung [11]. Essentially all data, including those at the basement or ground floor of a building, are used as output. For the first three modes, the extracted mode shapes, corresponding frequencies, and damping (using the N4SID Method) are shown in Fig. 7 and are also tabulated in Table 3 later in the paper. It is noted that the critical damping percentages (ξ) [shown in Fig. 7 as d1, d2 and d3] for the earthquake shaking data set are consistently lower than ~2.6% for NS, EW, and torsional fundamental modes. This is consistent with several other studies of recorded responses of tall buildings, including but not limited to [12,13,14. It is also consistent with the recent Recommendations of Los Angeles Tall Buildings Structural Design Council (LATBSDC) [15], as well as the Tall Building Initiative (TBI) of the Pacific Earthquake Engineering Research Center (PEER) [16]. In addition, the 2nd and 3rd mode shapes above the 56th level have smaller slopes compared to the rest of the shapes. This is attributable to the decreasing stiffness of the building above the 50th level, due to decreased core shear wall area.

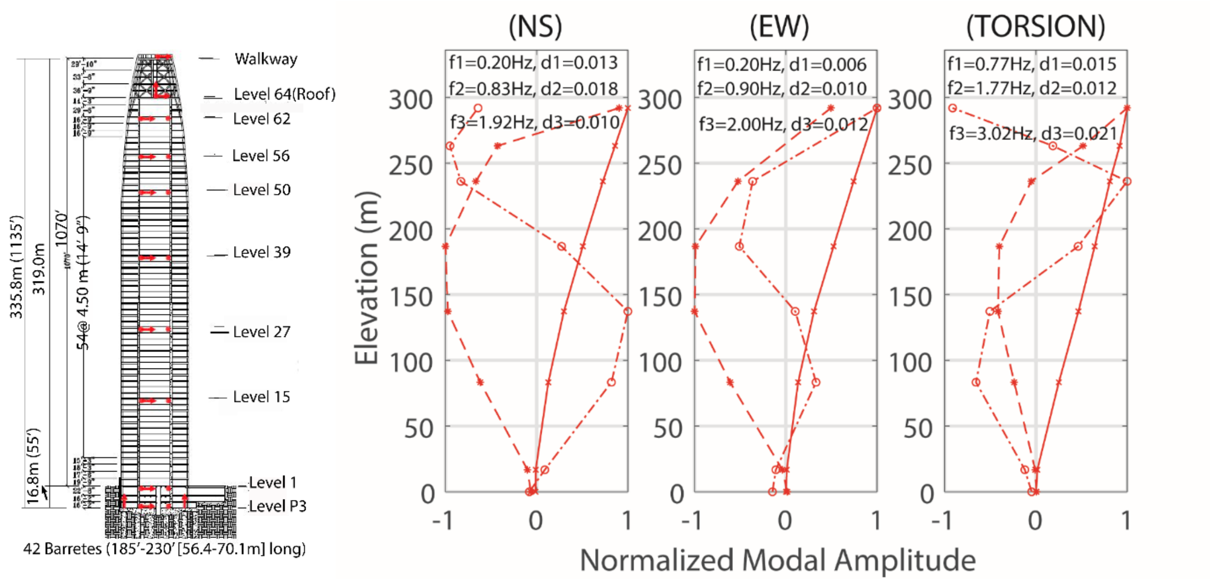

Fig. - 7. Normalized mode shapes for the first three of each of NS, EW, and torsional modes, with identified frequencies and critical damping ratios indicated for the Salesforce Tower. It is noted that these mode shapes are computed with available displacement data (during the January 4, 2018 Berkeley earthquake), which included channels 1-27 up to and including those on floor level 62. The mode shapes and frequency and damping ratios may vary slightly if all channel data were available. Note also the smaller slope of the 2nd and 3rd mode shapes above the 56th level.

2.4 A short note on beating and torsion

Beating is a periodic, resonating, and prolonged vibrational behavior caused by distinctive close coupling of translational and torsional modes of a lightly damped structure [17-19]. Thus, repetitively stored potential energy during the coupled translational and torsional deformations turns into repetitive vibrational energy, causing the ensuing prolonged motions. The energy periodically flows back and forth between closely coupled modes, generally with regular periodicity. The coupled motions reinforce and weaken each other.

The "beat frequency" (f_b) - as it is generally referred to in acoustical physics - is denoted by the absolute value of the differences in frequencies ($|f_1-f_2|$) that cause the phenomena. In some cases, the beating effect can be substantial. A recent study introduces a process to quantify the effect of beating [19]. Due to low damping and the beating effect, the period of shaking lengthens, and in the long run, a building can be subjected to low-cycle fatigue.

The "beating period" (T_b) is twice the inverse of beat frequency ($T_b=2/f_b$). The beating period will be computed by the following equation [17]:

$$T_b=1/F_b=2/f_b=2/\mathrm{abs}(f_1-f_2)=2T_1T_2/\mathrm{abs}(T_1-T_2) \qquad (1)$$

In the earthquake data set studied herein, beating effects are observed clearly in the acceleration and displacement plots (Figs. 2 and 3).

Of course, it is understood that the actual response of a structure is not simple. The amplitudes of reinforcing signals may vary drastically with time, and there may be contributions from other modes that further complicate the resulting response and visual identification of the beating period. Unfortunately, the record length is short, and for the 61-second available record, the useful signals are between ~28 and 61 seconds (~ 33 second signal). This is due to the fact that the recording of the data is based on trigger and de-trigger thresholds. In continuous recording, the useful record lengths are almost always longer. In any case, the beating period can be approximately computed for the first torsional mode frequency (at 0.77 Hz) and 2nd EW translational mode frequency (EW ~0.90 Hz). The result is ~15.4 seconds calculated as follows: $T_{b=}$ $2T_1T_2/\mathrm{abs}(T_1-T_2)= 2(1/0.77)(1/0.90)/[(1/0.77)- (1/0.90)]= 15.38$ seconds.

This is demonstrated in Fig. 8a. If we use the second torsional mode frequency (1.77 Hz) and 3rd translational mode frequency (1.92 Hz), then $T_{b}= 2(1/1.77)(1/1.92)/[(1/1.77)- (1/1.92)]= 13.33$ seconds.

Torsional behavior is expected to be different between locations on levels that have significantly smaller core shear walls due to the discontinuity of about half the core above the 50th level (Fig. 1c). The effect can be quantified and compared, as in Fig. 8b, which depicts torsion per meter of height between 62nd and 56th levels, between 39th and 15th levels, and an average value between 62nd and 15th levels. As seen in the figure, torsion between the levels with the half core shear wall area is between 2-3 times that of the torsion between levels with a full shear core wall. It is also noted that this depiction also displays a beating effect, seen in Figure 8a.

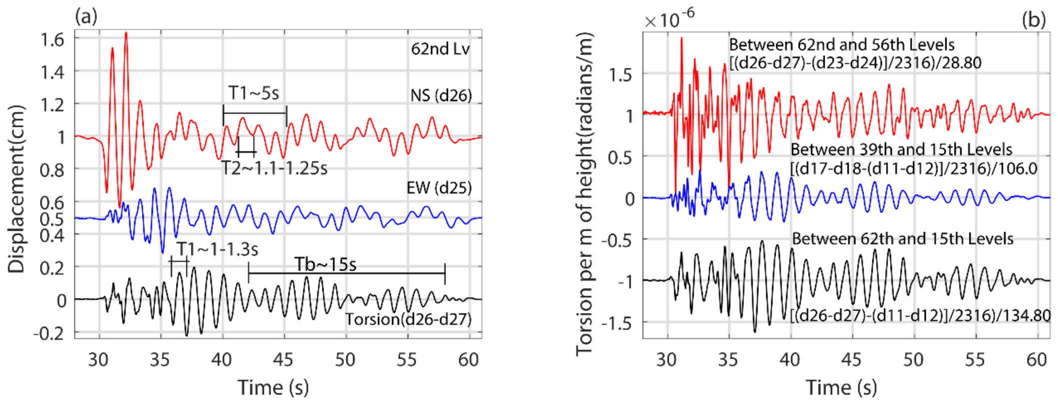

Fig. - 8. (a) NS (channel 26) and E-W (channel 25) translational displacements depict superimposed first and second modal periods. Torsional displacements (channel 26 - channel 27) depict first modal torsional period, as well as a beating period of ~15 seconds. (b) Torsion per meter of height is distinguishably quantified for the upper levels with a half core shear wall when compared with the lower levels with a full core shear wall.

3. Conclusions

In Table 3, for the first three modes and for each of the N-S, E-W, and torsional directions, identified frequencies (periods) and critical damping percentages are summarized for the earthquake dataset of the January 4, 2018, Berkeley event and for the ambient dataset obtained on demand from the seismic array in the Salesforce Tower. As noted, the frequencies for earthquake and ambient excitation, although of considerably large amplitude acceleration ratios, are quite similar. This implies that the earthquake shaking is not significantly large enough to force the building to nonlinearity nor to significantly shift its vibrational frequencies. It is expected that during future higher earthquake shaking levels, this may change but not to the point of strutural failure. Similarly, the damping percentages are low (e.g., 1.5% for fundamental translational and 2.6% for fundamental torsional modes). The fact that damping ratios are <2.5% is consistent with findings from analyses of earthquake response data acquired from seismic instrumentation arrays installed in buildings in the United States, Japan, Turkey, and other countries and is also consistent with recent recommendations published by LATBSDC (2017) and PEER-TBI (2017) - both of which recommend 2.5% damping based on analyses such as this.

The analyses showed that the second mode is dominant for the earthquake record but not in the ambient record.

Table 3. Summary of modal frequencies (periods) and damping percentages determined by system identification using data recorded during the Mw4.4 January 4, 2018 Berkeley earthquake and comparison with those from SLE and DBE analyses

Orientation/ Mode	Modal Frequency [Hz] (Period [s])			Modal damping (%)			Freq(Hz)(Period[s])	
	1	2	3	1	2	3	1	1
Earthquake							SLE	DBE
NS	0.20 (5.00)	0.83/(1.20)	1.92 (0.52)	1.30	1.80	1.00	0.17 (5.95)	0.13(7.78)
EW	0.20 (5.00)	0.90/(1.11)	2.00 (0.50)	0.60	1.00	1.20	0.16(6.11)	0.13(7.98)
TOR	0.77 (1.30)	1.77/(0.57)	3.02 (0.33)	1.50	1.20	2.10	0.53(1.90)	0.47(2.14)

The architectural visuals of the building **(**Fig. 1**)** give it the appearance of being perfectly symmetrical in plan design throughout its height. However, (i) abrupt changes in the core shear wall (e.g. halved above 50th level), (ii) differences in the thicknesses of the south and north walls of the core at multiple levels, and also (iii) the normal decreases in thicknesses at several levels in ascending from the P3 to 64th levels causing shifts in the EW (east-west) neutral axis in-plan, make the structural system vertically asymmetric and naturally prone to torsional behavior. Torsional rotation (radians per meter of height) is 2-3 times larger above the 50th level as compared to below the 50th level. This is significant because, due also to light damping (<2.6%) and closely coupled torsional and translational modes, the structural system exhibits a beating effect with an approximate beating period of ~15 s.

The 30.5-m (~100 ft) high "cap" of the building (above the 64th level) is stiffer when compared with the ~300-m (~1000 ft) tall main building below the 64th level, thus, exhibiting an identifiable frequency of 2.43 Hz in the ambient translational acceleration records at the 62nd and above levels.

Major structural dynamic characteristics from both a small magnitude earthquake and ambient response of this unique building are identified. It is necessary to repeat that these characteristics may change during larger shaking of the building. It is anticipated that the accelerometric array will capture future events.

It is recommended here to lower the de-triggering thresholds of the earthquake records so that longer shaking records can be acquired during earthquakes. This will be useful to better assess the behavioral features (e.g. beating effect) of this unique landmark of San Francisco.

Data Sources: www.strongmotioncenter.org (last accessed April 30, 2019)

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

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