STATUS AND NEEDS FOR SEISMIC INSTRUMENTATION OF STRUCTURES ALONG THE HAYWARD FAULT

M. Çelebi[1] and E. Kalkan[1]

ABSTRACT

The inventory of structures in heavily urbanized communities within the greater San Francisco (SF) Bay area that will experience strong ground motions from the rupture of the Hayward Fault includes a variety of types of recent and older structures built with a variety of materials and to different code standards. Those who remember the effects of the 1989 Loma Prieta earthquake on structures in the San Francisco Bay area also remember the collapse of one upper-deck segment of the Bay Bridge that halted transportation for approximately five weeks. In order to understand how these structures respond to earthquake motions and to improve building practices to resist these strong motions it is imperative that owners of these structures as well as governmental organizations acquire shaking response data from instrumented (or yet to be instrumented structures) during the forecast events. Within California, such data are acquired mainly by California Geological Survey and the United States Geological Survey. A small number of private owners contribute to this effort.

The inventory of existing instrumented structures is much less than 0.1% of the total, and thus statistically it is not sufficient. For example, some of the existing important regular or lifeline structures are not instrumented (e.g. Bart Trans-Bay Tunnel, many segments of the Bart elevated structures in the proximity of the Hayward Fault, the yet to be completed eastern part of San Francisco Bay Bridge, Hetch-Hetchy pipeline system crossing the Hayward Fault) even though attempts and proposals have been developed to do so in the past.

This paper presents a critical assessment of the status quo and the future needs for instrumentation of structures in the greater SF Bay area that includes the Hayward Fault. There are many new attempts and successes in instrumentation of structures in this region. Two successful examples are provided here, but more needs to be done. The paper does not present new research results; hence, it should be considered to be a “tutorial” paper.

INTRODUCTION

The purpose of this review paper is to present a summary of the current status and some samples from the present inventory of structures that have been instrumented to capture seismic response and performance data in the greater San Francisco Bay Area. The most likely sources of future earthquakes that will affect these structures are the Hayward, San Andreas, Calaveras and Rodgers Creek Faults. The paper also identifies structural types not covered by the current inventory that, if instrumented, would provide additional valuable new data for these types of structures.

Seismic monitoring of structural systems constitutes an integral part of the National Earthquake Hazard Reduction Program in the United States and similar hazard reduction strategies in seismically active regions of the world. An instrumented structure should provide enough information to (a) reconstruct the response of the structure in sufficient detail to compare it with the response predicted by mathematical models and with those observed in laboratories, the goal being to improve the models, (b) make it possible to explain the reasons for any damage to the structure, and (c) to facilitate decisions to retrofit/strengthen the structural systems when warranted. In addition, a structural array should include, if feasible, an associated free-field tri-axial accelerograph so that the interaction between the soil and the structure can be quantified.

Recent trends in the development of performance-based earthquake resistant design methods, related needs of the engineering community, and advances in computation, communication and data transmission capabilities have prompted development of new goals and approaches for structural monitoring. In particular, (a) verification of performance-based design methods and (b) needs of owners to rapidly, informedly and accurately assess the damage condition, and therefore the functionality of a building during and soon after an event. Such assessment of the damage condition or performance of a building is of paramount importance to stakeholders, which include owners, leasers, permanent and/or temporary occupants, city officials and rescue teams that are concerned with safety of those in the building, and those that may be affected in nearby buildings and infrastructures. These stakeholders will require answers to key questions such as: (a) Is there visible or hidden damage? (b) If damage occurred, what is its extent? (c) Does the damage threaten other neighboring structures? (d)

[1] USGS(MS977), 345 Middlefield Rd., Menlo Park, Ca. 94025

Can the structure be occupied immediately without compromising life safety or is life safety questionable? As a result, property damage and economic loss due to lack of permit to enter and/or re-occupy a building may be significant.

Rapid and accurate assessments of the performance of a structure require measurements of displacement rather than or in addition to accelerations, as is more commonly done. However, the development of new monitoring tools is being driven not only by the evolving needs of engineers but also by the advent of data acquisition systems with specific software that can record, digitize and process accelerations to obtain displacements in near real-time, and transmit both accelerations and displacements in real-time or near real-time.

This review paper presents a summary of the seismic monitoring issues as practiced in the past, as well as current applications and new developments to meet the needs of the engineering and user community. In addition, statistical information on the inventory of structures instrumented for seismic monitoring is presented. Because many specific types of structures currently are not instrumented there are numerous potential lost opportunities to obtain critical new data for these structures during forecasted future earthquakes. Two examples of successes in seismic instrumentation projects from the greater San Francisco Bay Area are demonstrated. These examples exhibit the most recent applications that can be used for verification of design and construction practices, real-time applications for the functionality of built environment and assessment of damage conditions of structures. To meet the needs of the engineering and user community in seismic monitoring and assessing the functionality and damage condition of structures, recent developments and approaches (e.g. double-integration of recorded accelerations) are being used to obtain displacements and, in turn, drift ratios, in real-time or near real-time to help assess damage condition and therefore functionality of a structure.

HISTORICAL PERSPECTIVE

In the United States, the California Strong Motion Instrumentation Program (CSMIP) of the California Geologic Survey and the United States Geological Survey (USGS) manage the largest two structural instrumentation programs. In addition, in California, there are private enterprises that have supported instrumentation programs aimed at their own properties and infrastructures (e.g., the Pacific Gas and Electric Company and a few financial institutions). Until recently, these programs have aimed to facilitate response studies in order to improve our understanding of the behavior and potential for damage to structures under the dynamic loads caused by earthquakes. The principal objective has been the quantitative measurement of structural response to strong and possibly damaging ground motions for purposes of improving seismic design codes and construction practices. However, to date, it has not been the objective of either instrumentation program to create a health monitoring environment for structures. Data from both programs are readily available through the internet[2].

To date, the USGS has conducted a cooperative strong-ground-motion and structural instrumentation program with other federal and state agencies and private owners. Table 1 summarizes the current inventory and cooperative affiliations of the USGS Cooperative National Strong-Motion Program (NSMP). Detailed procedures and overall description used by the USGS structural instrumentation program are described by Çelebi (2000, 2001). On the other hand, the California Strong Motion Instrumentation Program (CSMIP), which now has over 200 buildings instrumented in accordance to a predefined matrix of type of structures, aims to cover a wide variety of structural systems (Huang and Shakal, 2001; Shakal, Huang, Rojahn and Poland, 2001). Tables 1 and 2 provide summary inventory for Northern California as well as within all of California. As presented, the total numbers for Northern California is a very small percentage of the total inventory and represents limited number of types of structures instrumented. As exhibited in Figure 1, the distribution of instrumented structures within California and for Northern California (mainly the greater San Francisco Bay Area) is very sparse.

Except for a few cases that can be described as special array deployments with real-time data transmittal capabilities that serve special purpose (an example of which is provided later in the paper), the overwhelming majority of the deployments follow the non-telemetered (hereinafter called “classical”) method whereby the data is not transmitted in real-time. The general descriptions of such classical deployments are introduced in Appendix A of this paper.

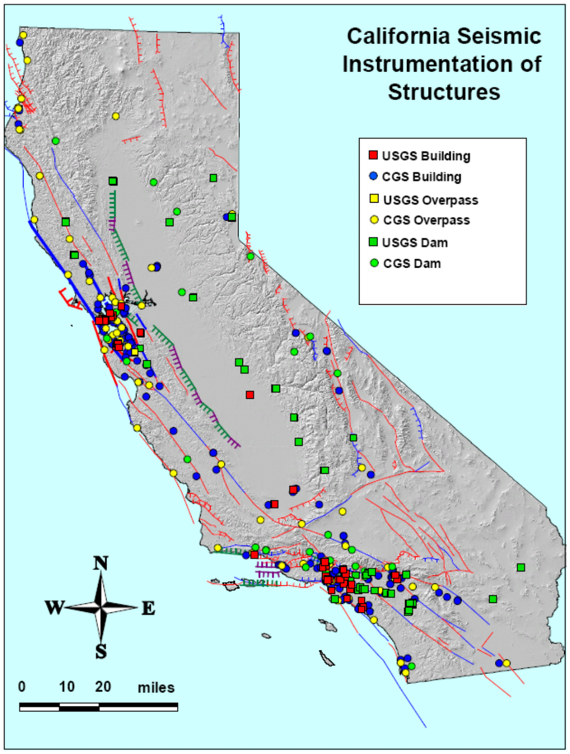
[2] USGS data are available via www.nsmp.wr.usgs.gov and CSMIP data are available via www.consrv.ca.gov/cgs/smip. Both are accessible via www.cosmos-eq.org and www.strongmotioncenter.org

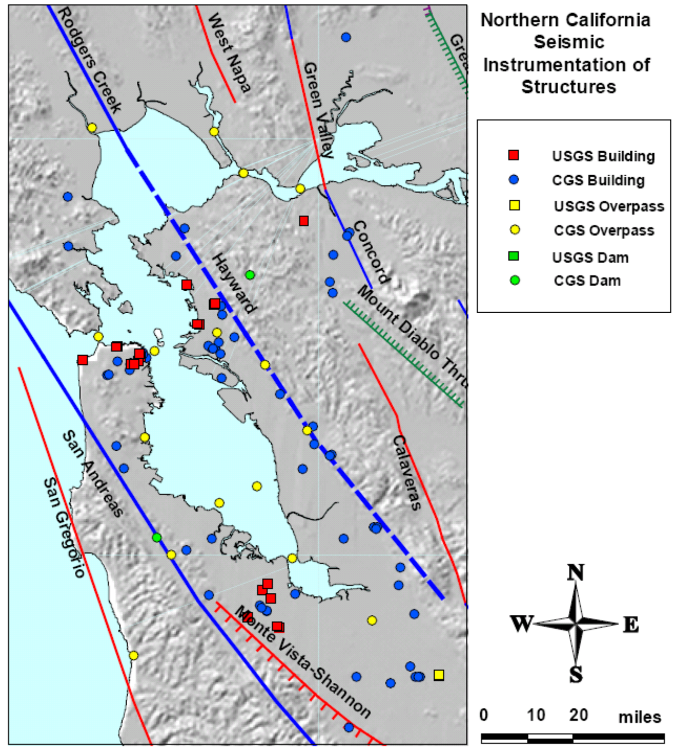
TABLE 1. Number of structures instrumented by USGS Cooperative Program (ACOE, VA, GSA, UPR, CITY of SF, ODOT, UDOT, MWD, JPL) [Note: 3 channels or more within a structure]

Structure Type	US (including HI, AK, PR)	California	Southern California	Northern California	San Francisco Bay Area
Building	115	47	29	18	16 (4 East Bay, 7 San Francisco 5 Peninsula)
Dam	60	25	13	12	0
Bridge	16	2	1	1	1
Special (e.g., Filtration plant array that includes structures)	1	1	1	0	0

TABLE 2. Number of structures instrumented by CSMIP of the California Geological Survey [Note: 3 channels or more within a structure; data compiled from internet and other data bases]

Structure Type	Total California	Southern California	Northern California	San Francisco Bay Area
Building	200	98	102	61 (12 in San Francisco, 35 Peninsula, 9 East Bay, 2 Walnut Creek, 3 Santa Rosa)
Dam	27	15	12	
Bridge	66	23	43	14 (4 in San Francisco, 3 Hayward-Oakland, 3 Peninsula, 4 Vallejo)
Geotechnical Array	30	12	18	9 (1 in San Rafael, 2 San Francisco, 2 Peninsula, 3 Benicia, 1 Hayward)
Special (e.g. tunnel, water tank, wharf, power plant, airport control tower)	12	6	6	





(a) (b)

Figure 1. (a) Distribution of Instrumented Structures in California; (b) Distribution of Instrumented Structures in Northern California (covering San Francisco Bay Area)

EXAMPLE OF AN EXTENSIVELY AND CLASSICALLY INSTRUMENTED STRUCTURE - RECORDED DATA AND ANALYSES

Pacific Park Plaza, Emeryville, Ca.

The Building, Design Spectra and Instrumentation

The Pacific Park Plaza (PPP) Building is an equally-spaced three-winged, cast in place, thirty-story, 312 ft. (95.1 m) tall, ductile reinforced concrete moment-resisting frame building. The three wings of the building are constructed monolithically and are equally spaced at angles of 120 degrees around a central core. Shear walls in the center core and wings extend to the second floor level only, but column lines are continuous from the foundation to the roof. The foundation is a 5-foot-thick concrete mat supported by 828 (14-inch-square) pre-stressed concrete friction piles, each 20-25 m in length, in a primarily soft-soil environment that has an average shear-wave velocity between 250 and 300 m/s and a depth of approximately 150 ft (~50 m) to harder soil. A three-dimensional schematic of the building and its seismic instrumentation is shown in Figure 2. The instrumentation integrates arrays for the structure, surface, and downhole, and comprises a 30-channel accelerometer deployment uniquely designed to capture (a) the translational motions of the wings of the building relative to its core, (b) the vertical motions of the mat foundation slab at the ground floor level, and (c) free-field motions at the surface and at a downhole depth of 200 ft (61 m). The South Free-field (SFF) station is often referred to as the Emeryville (EMV) ground site. The building instrumentation is considered as extensive. The recorder systems are digital and record data at 200 samples per second. Data is transmitted via phone lines. This building is selected for this study because there is a variety of old and new data and because there is no evidence that it experienced any damage during the various levels of shaking described in this paper.

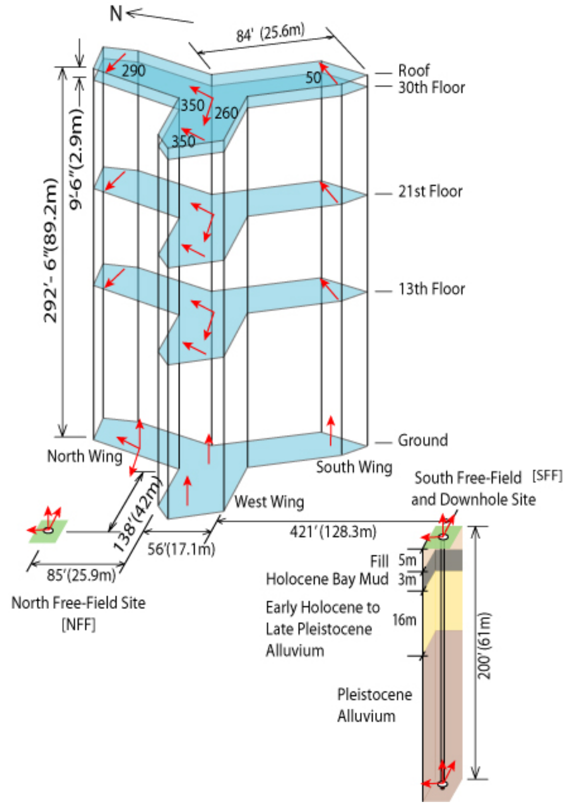


Figure 2. A three-dimensional schematic of the building array with integrated surface and downhole array. Red arrows indicate sensor locations and orientations. The tri-axial downhole accelerograph was added after the 1989 Loma Prieta earthquake.

Design Spectra and Significant Shaking Experienced

To date, the most significant shaking recorded by the building arrays was during the 1989 Loma Prieta (LPE), CA earthquake (M_s=7.1). The data set from LPE is extensively used in several studies as well as in this investigation that specifically dwells upon the variation of fundamental period with level of shaking. As previously mentioned, the building was not damaged.

Responses of the building and the surface free-field recorded during the strong shaking caused by the LPE earthquake exhibit distinct amplification of motions (Fig. 3) at the site of the building as compared to the motions at Yerba Buena Island, both approximately 100 km (and at similar azimuths) from the epicenter of the LPE. The east-west components of acceleration recorded at the roof and the ground floor of the structure and at the associated free-field station (SFF in Fig. 2) are shown in Figure 3a. The motion at Yerba Buena Island (YBI), the closest rock site, had a peak acceleration of 0.06 *g*, and is also shown for comparison. The response spectra (Fig. 3b) clearly demonstrate that the motions at Emeryville (SFF) were amplified by as much as five times when compared with YBI. This is also inferred by the amplitude of the peak accelerations (0.26 *g* for SFF and 0.06 *g* for YBI). Furthermore, the differences in peak acceleration at SFF (0.26 *g*) and at the ground floor of the building (0.21 *g*) (Fig. 3a) suggest the possibility of significant soil-structure interaction. Fig. 3c shows a comparison of actual response spectra with site-specific design response spectra (based on the probabilistic earthquakes related to levels of performance) used in the design of the building: (a) the maximum probable earthquake (50 % probability of being exceeded in 50 years with 5 % damping) anchored at zero period acceleration (ZPA) of 0.32g. [curve A in Fig. 3c], and two maximum credible earthquakes both with 10 % damping but 10 % probability of being exceeded in (b) 100 years (ZPA of 0.63 g) [Curve B in Fig. 3c] and (c) 50 years [ZPA of 0.53 g][3]. The spectra of the EW components of recorded motions at the ground

[3] Not shown in the figure.

floor and SFF are also shown in Fig. 3c. At 100 km from the epicenter, even though the recorded EW peak acceleration at SFF (0.26 g) is smaller than the ZPA of the postulated maximum probable earthquake (0.32 g), the spectral accelerations of the EW component of SFF is considerably higher than the maximum probable earthquake for periods >0.6 seconds – that is, practically the first three modes of the building. This implies that, when large earthquakes occur closer to the structure, the level of shaking and the response spectra of motions are likely to be higher (for some period bands) than the design response spectra, and, in many cases, the code design response spectrum (*e.g.* the 1979 Uniform Building Code).

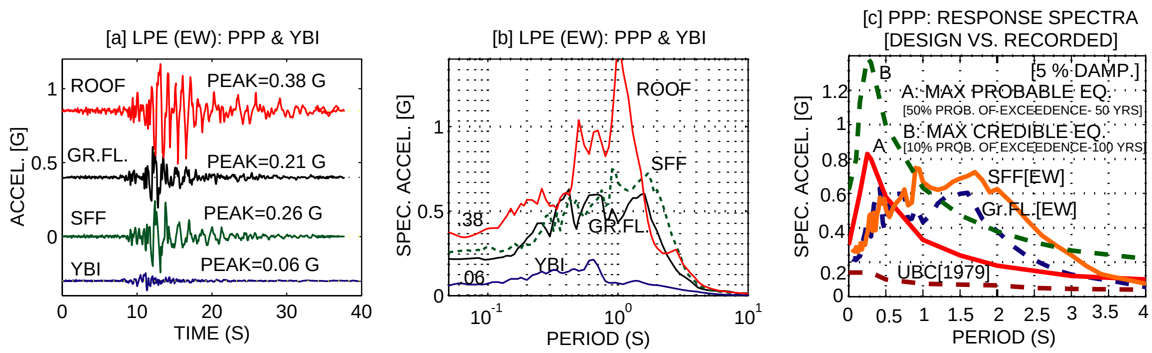


Figure 3. (a,b) Amplified (EW) motions and their corresponding response spectra (5% damped) at the South Free-Field (SFF), ground floor and roof of the Pacific Park Plaza array as compared to the motions at Yerba Buena Island (YBI) at approximately the same epicentral distance as PPP. (c) Design response spectra and response spectra of recorded motions at the ground floor and SFF of Pacific Park Plaza. Also shown is the 1979 UBC response spectrum for comparison. [Note: Curve B is for 10% damping].

Summary of Studies Related to the Building

Extensive data sets from this building include not only the Loma Prieta earthquake response data but also those from smaller earthquakes and from forced and ambient vibration tests (Stephen et al, 1985, Çelebi et al, 1993). Table 3 summarizes the events (including LPE) that have been recorded by the building array. Sample analyses of data related to LPE and test data are summarized in Table 4.

Table 3. Events that have been recorded by the PPP arrays

Event/ Date	UTC	Lat. (N)/ Long. (E)	Dist. (km)	Azim. (deg)	Depth (km)	Mag.
Loma Prieta 10/18/1989	04:15	37.036 -121.883	96	157	18.0	M_s 6.9
El Cerrito 12/04/1998	12:16	37.920 -122.290	9	4	6.8	M_w 4.0
Yountville 09/03/2000	08:36	38.379 -122.413	61	350	10.1	M_w 5.0
Piedmont 09/05/2003	01:39	37.845 -122.222	7	85	12.4	M_w 3.9
Berkeley 03/02/2006	06:08	37.863 -122.245	5	96	11.4	M_d 2.8

The building has been studied in detail or as part of a larger investigation by several researchers (Çelebi and Safak, 1992, Safak and Çelebi, 1992, Anderson et al, 1991, Bertero et al, 1992, Kagawa et al, 1993a, b, Aktan et al, 1992, Kambhatla et al, 1992, Çelebi, 1992, 1998). Using different methods, including spectral analyses, system identification techniques (Çelebi, 1998), and mathematical models, the majority of the investigators are in agreement that, for the 1989 Loma Prieta earthquake data, the predominant three response modes of the building and the associated frequencies (periods) are 0.38 Hz (2.63 s), 0.95 Hz (1.05 s), and 1.95 Hz (0.51 s). These three modes of the building are torsionally-translationally coupled (Çelebi, 1998) and are depicted in the cross-spectra (S_{xy}) of the orthogonal records obtained from the roof, ground floor and SFF (the south free-field site) and the normalized cross-spectra of the orthogonal records (Fig. 4). The site frequency at 0.7 Hz (1.43 s) observed in the cross-spectrum of the roof (Fig. 4a) appears as the dominant peak in the cross-spectra of the ground floor and the south free-field (SFF) (Fig. 4b and 4c). This site frequency has been also confirmed by the wave propagation method using site borehole data by Gibbs and others (1994) and as determined from this set of records are reported in detail in Çelebi (2003).

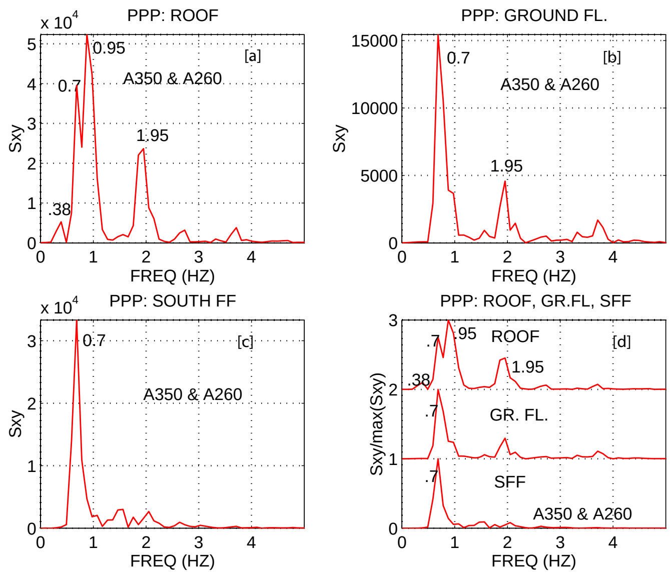


Figure 4. Cross-spectra of orthogonal motions at the [a] roof, [b] ground floor, [c] free-field of PPP, and [d] the normalized cross-spectra depicting structural and site frequency peaks.

Dynamic characteristics of the building extracted from the data sets are summarized in Table 4, and show considerable differences in the fundamental frequency determined from strong shaking versus low-amplitude shaking and analyses. The differences are attributed to SSI effects during strong shaking (Çelebi, 1998, Kagawa et al, 1993a,b, Aktan et al, 1992, Kambhatla et al, 1992), and frequencies from recorded motions can be matched when SSI is incorporated into the mathematical models (Kagawa et al, 1993a,b). Furthermore, a study of the building for dynamic-pile-group interaction (Aktan et al, 1992, Kambhatla et al, 1992) indicates that there is significant interaction. The study shows that computed responses of the building using state-of-the-art techniques for dynamic-pile-group interaction compares well with the recorded responses. Clearly, the mathematical models developed at that time needed improvements (Stephen et al, 1985). This conclusion could only be reached because we have recorded on-scale motions.

In addition, system identification techniques, when applied to the records of this building, yielded very large damping ratios corresponding to the 0.38-Hz first-mode frequency. These are 11.6 percent (north-south) and 15.5 percent (east-west) [Table 4] (Çelebi, 1996, 1998). Such unusually high damping ratios have been attributed to radiation damping that commonly occurs for buildings with large mat foundations in relatively soft geotechnical environment (Çelebi, 1996).

Table 4. Peak Accelerations and System Identification Results for (Pre-1991) PPP Data

Peak Accelerations (A[g]))				
	Loma Prieta Eq. (1989) [Çelebi, 1998]		Low-Amp. Tests & Analyses [Stephen et al 1985, Çelebi et al 1993]	
	NS	EW	NS	EW
Roof	0.24	0.38	<0.01	<0.01
Gr. Fl.	0.17	0.21	<0.01	<0.01
FF	0.21	0.26	-	-
Dynamic Characteristics (System Identification)				
f_o (Hz)	0.38	0.38	0.48-0.59	
T_o (s)	2.63	2.63	1.69-2.08	
ξ(%)	11.6	15.5	0.6-3.4	

Anderson and others (1991) compared the design criteria, code requirements, and the elastic and nonlinear dynamic response of this building due to the earthquake. They also found the fundamental frequency of the building to be ~ 0.37-0.39 Hz. However, contrary to others, but based only on comparison of ground level motions with those at the free-field, they concluded that soil-structure interaction was insignificant for this building during this earthquake.

SPECIAL ARRAYS – LOOKING TO THE FUTURE

Displacement Measurement Needs and Arrays

Two important motivations are driving development of new technologies for measuring displacements in real-time or near real-time: (a) the evolution of performance-based design methods and procedures which rely on displacement as the main parameter, and (b) the needs of local and state officials and prudent property owners to establish procedures for assessing the functionality of buildings and other important structures, such as lifelines, following a significant seismic event. As a result, structural engineers increasingly demand measurement of displacements during strong shaking events in order to readily compute drift ratios that in turn are related to performance of the structure.

A challenge to meeting these objectives is that directly measuring relative displacements between floors in real-time is very difficult and, except for tests conducted in a laboratory (e.g., using displacement transducers), has yet to be readily and feasibly achieved for a variety of real-life structures. However, recent technological developments have already made it possible to successfully develop and implement two approaches: (a) near-real time displacement computation from streaming acceleration data, and (b) direct displacement measurement using GPS receivers deployed at rooftops of buildings. Real-time displacements can be directly translated to inter-story drift ratios to be used for performance evaluation and health-monitoring of structures. How drift ratios can be related to damage is qualitatively schematized in Figure 5. In the figure, lateral deformation is a generic definition which can be absolute or relative displacement, inter-story drift, or any other appropriate deformation. Thus, once drift ratios can be readily computed in near real-time, technical assessment of the damage condition of a building can be made since several threshold stages at which damage condition of a story as defined by drift ratios are pre-defined using relevant structural parameters such as the type of connections and story geometry (e.g. story height).

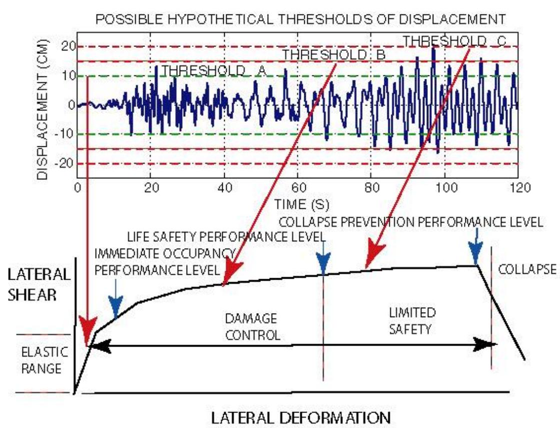


Figure 5. Hypothetical displacement time-history as related to FEMA- 274 (ATC, 1997)

Displacement via real-time double integration

GPS applications are currently limited to sampling at ≤ 20 Hz, and for building monitoring, displacements measurements (with GPS) are possible only on at the roof. This limits the application to long period structures rather than wide variety of structural systems. Therefore, the challenge is to compute displacements from recorded acceleration responses in real-time or near real-time (Çelebi and Sanli, 2002).

A new approach for obtaining displacements in real-time is depicted in Figure 6 which schematically shows the distribution of accelerometers in an existing building in San Francisco, CA. The building is array designed to provide data from several pairs of neighboring floors to facilitate drift computations. The system has a server that (a) digitizes continuous analog acceleration data, (b) pre-processes the 1000 sps digitized data with low-pass filters (herein called as the preliminarily filtered uncorrected data), (c) decimates the data to 200 sps and streams it locally, (d) monitors and applies server triggering threshold criteria and locally records (with a pre-event memory) when prescribed thresholds are exceeded, and (e) broadcasts the data continuously to remote users by high-speed internet (Çelebi and others, 2004).

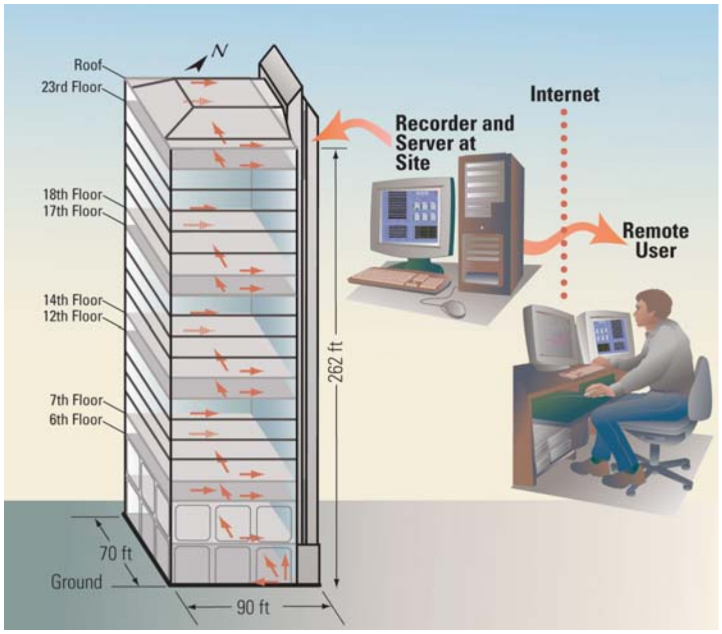


Figure 6. General schematic of data acquisition and transmittal for seismic monitoring of an existing building in San Francisco, CA.

The streamed real-time acceleration data are acquired remotely using client software configured to compute velocity, displacement, and a selected number of drift ratios. Figure 7 shows two PC screen snapshots of the client software display configured for 12 channels of streaming acceleration or velocity or displacement or drift ratio time series. Figure 7a shows each paired set of acceleration response streams is displayed with a different color. The upper right shows amplitude spectra of acceleration for one of the channels. Note that several frequency peaks are clearly identifiable. In the lower left, time series of drift ratios are shown for 6 locations, each color corresponding to the same pair of data from the window above. The drift ratios are derived using displacements computed from double-integration of filtered acceleration data; specific filter options are built into the client software for processing of the streaming acceleration data. To compute drift ratios, story heights need to be manually entered. Figure 7b shows the computed pairs of displacements that are used to compute the drift ratios. Corresponding to each drift ratio, there are 4 stages of colored indicators. The "green" color indicates that the computed drift ratio is below the first of three specific thresholds. The thresholds of drift ratios for selected pairs of data must also be manually entered in the boxes. As drift ratios exceed the designated three thresholds, additional indicators are activated with a different color (e.g. Figure 7b hypothetically shows that the first level of threshold is exceeded, and the client software is recording data as indicated by the illuminated red button [in the upper right]). The drift ratios are calculated using data from pairs of accelerometer channels oriented in the same direction. The threshold drift ratios are computed and defined by structural engineers using structural information and are compatible with the performance-based theme, as illustrated in Figure 5 (Fig. C2-3 of *FEMA-274* [ATC 1997]) and summarized in Table 5 for this particular building.

Table 5. Summary of Sample Threshold Stages and Corresponding Drift Ratios

SampleThreshold Stage	1	2	3
Sample Drift Ratio	0.2%	0.8%	1.4-2.0%

(a) (b)

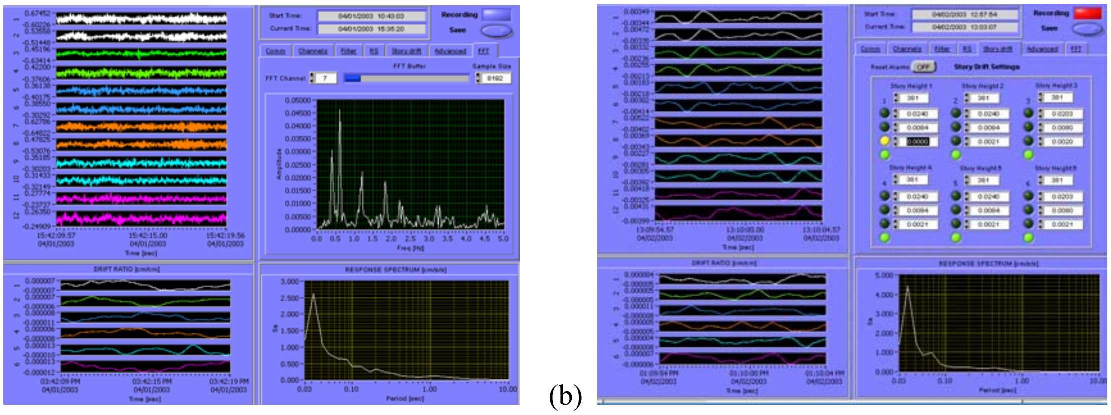


Figure 7. (a) Screen snapshot of client software display showing acceleration streams and computed amplitude and response spectra, and drift ratios in normal status when no drift threshold is exceeded. (b) Screen snapshot of client software display showing 12-channel (six pairs with each pair a different color) displacement and corresponding six-drift ratio (each corresponding to the same color displacement) streams. Also shown to the upper right are alarm systems corresponding to thresholds that must be manually input. The first threshold for the first drift ratio is hypothetically exceeded to indicate the starting of the recording and change in the color of the alarm from green to yellow.

Sample Recorded Low-amplitude Earthquake Response Data and Analyses

During the December 22, 2003 San Simeon, CA. earthquake (Mw=6.4), at an epicentral distance of 258 km, a complete set of low-amplitude earthquake response data was recorded in the (San Francisco, CA) building. The largest peak acceleration was approximately 1 % of g. Synchronously recorded accelerations were double-integrated to obtain the displacements exhibited in Figure 8 all in one direction on one side of the building. Figure 9 (left frame) further exhibits computed displacements 20-40 into the record, and reveals the propagation of waves from the ground floor to the roof. The travel time is extracted as about 0.5 seconds. Since the height of the building is known (80 m), the travel velocity is computed as 160 m/s. One approach for detection of possible damage to structures is by keeping track of significant changes in the travel time, since such travel of waves will be delayed if there are cracks in the structural system (Safak, 1999).

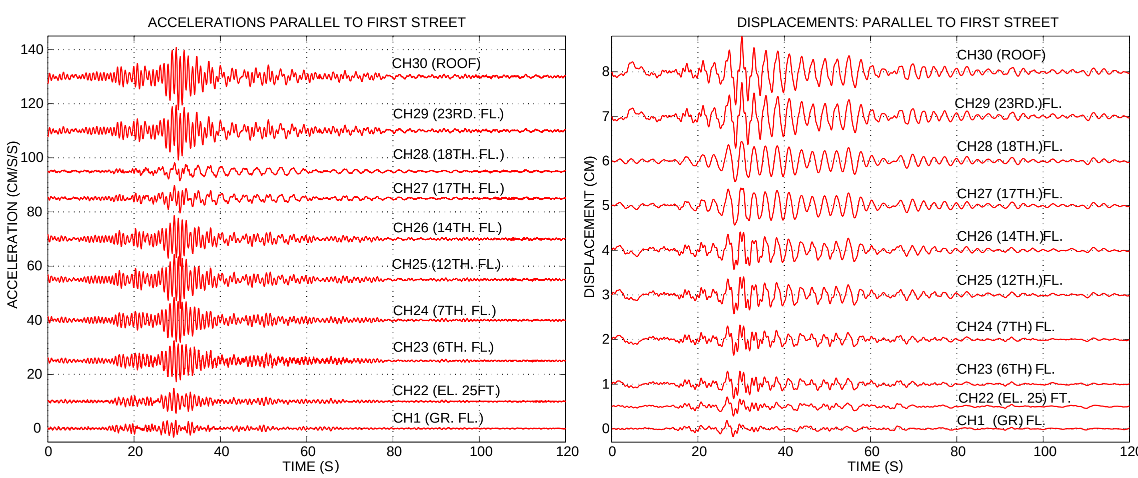


Figure 8. Accelerations (left) and displacements derived by double-integration (right) at each instrumented floor (from Ground floor to the roof) on one side of the building [San Simeon earthquake, December 22, 2003].

In Figure 9 (right frame), the two parallel and orthogonal motions recorded at the roof are used to identify translational and torsional frequencies as 0.38 Hz and 0.60Hz respectively.

Benefits of using such real-time systems for either direct measurement of displacements using GPS or real-time computation of displacements by double-integration of accelerations during very strong shaking caused by earthquakes or other extreme events are yet to be proven. However, analyses of data recorded during smaller events or low-amplitude shaking show promise.

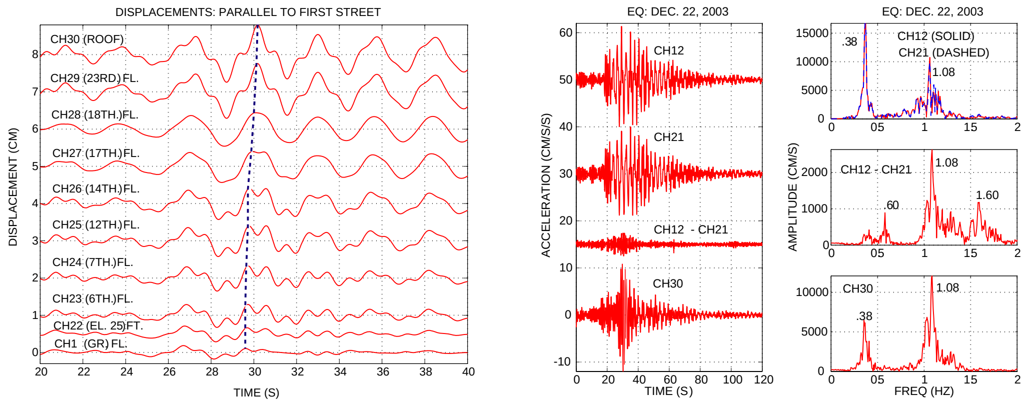


Figure 9. (Left) A twenty second window plotted from 20-40seconds into the record of computed displacements. Travel time of propagating vibrational waves from the ground floor to the roof of the 80 m tall building is approximately 0.5 second. (Right) Acceleration data (San Simeon, Ca. earthquake of December 22, 2003) obtained at the roof from parallel channels (CH12 & CH21), their difference (CH12-CH21), and from CH30, orthogonal to CH12 and CH21 (left) and corresponding amplitude spectra (right)

ADDITIONAL NEEDS FOR SEISMIC INSTRUMENTATION OF STRUCTURES AND SUMMARY

The probability for a damaging M≥6.7 earthquake in the whole San Francisco Bay Region in 30 years is 63%, for the Hayward/Rodgers Creek Fault is 31 %, and for the San Andreas Fault is 59% (WGCEP, 2007). It is clear that the total number of instrumented structures is much less than 0.1 % of the total inventory both within the greater San Francisco Bay Area and for the entire State. Since future earthquakes in the region will affect the behavior and performance of all structures in the region, it is incumbent on the earthquake community to take action to capture valuable response data from instrumented structures. As demonstrated with the examples provided, there are successes in enhancing both the quantity of instrumented structures and the quality of the instrumentation and data retrieved from past earthquakes. However, if thoughtful instrumentation of structures is not pursued, opportunities to capture critical new data from future earthquakes, especially for representative structures not currently instrumented, will be lost.

The authors believe that the following types of structures in the San Francisco Bay Area are well represented:

1. Base isolated structures (e.g. Oakland City Hall, SF City Hall, Hearst Mining Building on UC Berkeley Campus),
2. Bay Area Bridges (through an agreement between CSMIP and California Department of Transportation [Caltrans], most long-span bridges are or are being instrumented).

but that the following are the types of structures that are either under- or not at all represented:

1. Lifeline structures (Bart Tunnel, Hetch-Hetchy Pipelines crossing the faults, Bart elevated structures, existing eastern part of San Francisco Bay Bridge even though it will be replaced by a new one)
2. Tilt-up structures
3. Unreinforced masonry or retrofitted masonry structures
4. Single family wood buildings or multi-level wood buildings
5. Retrofitted (formerly non-ductile) reinforced concrete structures
6. Steel structures designed following revisions to connections (that did not perform well during the 1994 Northridge earthquake).
7. High-rise Reinforced Concrete Structures

8. Structures that have been retrofitted with new techniques such as (a) unbonded diagonal bracing, (b) viscous dampers, (c) external bracing, and (d) buildings/structures with columns jacketed with carbon fiber reinforcement.
9. Others of engineering importance and interest with future repeat applications.

Since each structure is unique, the number and configuration of sensors required to adequately understand the behavior and performance of a structure needs to be assessed on a case-by-case basis by a responsible structural engineer. Although cost issues are not discussed in this paper, the extent of instrumentation must be optimized with respect to cost.

SUMMARY

In this paper, future needs for instrumentation of structures in the greater San Francisco Bay Area are discussed. Presented are (a) general information on past and current practices and methods for seismic monitoring of structures, and (b) new applications and emerging technologies for meeting the needs of the engineering and user community. Both historical and current trends and methods are described in terms of utilization of data acquired by seismic monitoring. The extent to which a structure should be instrumented to meet the code recommendations versus special needs are discussed without consideration of costs. Two examples of instrumented buildings with limited data analyses are provided, one following a classical approach whereby the data are not retrieved in real-time, and the other designed to rapidly acquire and automatically evaluate response data during a strong shaking event in order to help in the process of making informed decisions regarding the structural health and occupancy.

APPENDIX I : SEISMIC INSTRUMENTATION – GENERAL ISSUES

A.1 Data Utilization

Ultimately, the design of instrumentation must be tailored considering multiple possible uses of data acquired during future earthquakes will be utilized. Table A1 summarizes some data utilization objectives with sample references.

A.2 Code-Recommended vs. Extensive Instrumentation

The most widely used code in the United States, the International Building Code (IBC-2006), recommends but does not always require, that for seismic zones 3 and 4 a minimum of three tri-axial accelerographs be placed in every building over six stories in height and with an aggregate floor area of 60,000 square feet or more, and in every building over ten stories regardless of the floor area. The purpose of this recommendation is to monitor a limited number of response characteristics of the building (such as peak accelerations at the roof and at the ground floor) rather than to analyze the complete structural response. Code-recommended instrumentation is illustrated in Figure A1a. In 1982, more that a decade after the 1971 San Fernando earthquake, , the code requirement in Los Angeles was reduced to one tri-axial accelerometer at the roof (or top floor) of a building (Darragh and others, 1994). In general, code instrumentation is being de-emphasized as a result of strong desire by the structural engineering community to gather more data from instrumented structures to perform more detailed structural response studies. Experiences from past earthquakes show that the minimum guidelines established by IBC (or UBC) for three tri-axial accelerographs in a building are not sufficient to perform meaningful model verifications. For example, three horizontal accelerometers are required to define the (two orthogonal translational and a torsional) horizontal motions of a floor. Rojahn and Mathiesen (1977) concluded that the predominant response of a high-rise building can be described by the participation of the first four modes of each of the three sets of modes (two translations and torsion); therefore, a minimum of 12 horizontal accelerometers would be necessary to record these modes. Instrumentation needed to provide acceptable documentation of the dominant response of a structure is addressed by Hart and Rojahn (1979) and Çelebi and others (1987). This type of instrumentation scheme is called the ideal extensive instrumentation scheme as illustrated in Figure A1b.

Specially designed instrumentation arrays are needed to understand and resolve specific response problems. For example, thorough measurements of in-plane diaphragm response require sensors in the center of the diaphragm (Fig. A1c) as well as at boundary locations. Performance of base-isolated systems and effectiveness of the isolators are best captured by measuring tri-axial motions at the top and bottom of the isolators as well as the rest of the superstructure (Fig. A1d). In case of base-isolated buildings, the main objective usually is to assess and quantify the effectiveness of isolators. Additional sensors can be deployed between the levels above the isolator and roof to capture the behavior of intermediate floors.

A.3 Associated Free-Field Instrumentation

More information is required to interpret the motion of the foundation substructure relative to the ground on which it rests. This requires free-field instrumentation associated with a structure (Fig. A1b). However, this is not always possible in an urban environment[A1]. Engineers use free-field motions as input at the foundation level, or they obtain the motion at foundation level by convolving the motion through assumed or determined layers of strata to base rock and deconvolving the motion back to foundation level. Confirmation of these processes requires downhole instrumentation near or directly beneath a structure. These downhole arrays will yield data on:

(1) the characteristics of ground motion at bedrock (or acceptably stiff media) at a defined distance from a source and
(2) the amplification of seismic waves in layered strata.

Downhole data from sites in the vicinity of instrumented building or other structures are especially scarce. Two new building monitoring arrays in the United States that include downhole sub-arrays are described later in the chapter.

A.4 Record Synchronization Requirement

High-precision record synchronization must be available within a structure and at associated free-field installations if the response time histories are to be used to reconstruct the overall behavior of the structure. Traditionally such synchronization has been achieved through extensive cabling from each of the individual sensors to the recorder. Recent technological developments enable decreasing or minimizing, and in certain cases eliminating, the use of extensive cabling. For example, the global positioning systems (GPS) is now widely used to synchronize building instrumentation with that of a separate recorder for the free-field, thus eliminating cable connection between the free-field recorder and recorders within a structure.

[A1] For example, in San Francisco, California, it is not possible to find a suitable free-field location around the Transamerica building, which is extensively instrumented.

Table A1. Sample List of Data Utilization Objectives & Sample References

Data Utilization Objective	References and/or Comments
GENERIC UTILIZATION	
Verification of mathematical models (usually routinely performed)	Boroschek et al, 1990
Comparison of design criteria vs. actual response	usually routinely performed
Verification of new guidelines and code provisions	Hamburger, 1997
Identification of structural characteristics (Period, Damping, Mode Shapes)	usually routinely performed
Verification of maximum drift ratio	Astaneh, 1991, Çelebi, 1993
Torsional response/Accidental torsional response	Chopra, 1991, De La Llera, 1995
Identification of repair & retrofit needs & techniques	Crosby, 1994
SPECIFIC UTILIZATION	
Identification of damage and/or inelastic behavior	Rojahn & Mork, 1981
Soil-Structure Interaction Including Rocking and Radiation Damping	Çelebi, 1996, 1997
Response of Unsymmetric Structures to Directivity of Ground Motions	Porter, 1996
Responses of Structures with Emerging Technologies (base-isolation, visco-elastic dampers, and combination	Kelly and Aiken, 1991, Kelly, 1993, Çelebi, 1995
Structure specific behavior (*e.g.* diaphragm effects)	Boroschek and Mahin,1991, Çelebi, 1994
Development of new methods of instrumentation hardware (e.g. GPS, wireless)	Çelebi et. al., 1997, 1999, 2001; Straser, 1997
Improvement of site-specific design response spectra and attenuation curves	Boore, et. al. 1997, Campbell, 1997, Sadigh *et. al*., 1997, Abrahamson and Silva, 1997
Associated free-field records (if available) to assess site amplification, SSI and attenuation curves	Borcherdt, 1993, 1994, 200a, 2002b, Crouse and MacGuire, 1996
Verification of Repair/Retrofit Methods	Crosby et al, 1994, Çelebi and Liu, 1996
Identification of Site Frequency from Building Records	Çelebi, 2003
RECENT TRENDS TO ADVANCE UTILIZATION	
Studies of response of structures to long period motions	Hall et al, 1996
Need for new techniques to acquire/disseminate data	Straser, 1997, Çelebi, 1998, Çelebi and Sanli, 2002, Celebi and others, 2004
Verification of Performance Based Design Criteria	future essential instrumentation work
Near Fault Factor	more free-field stations associated with structures needed
Comparison of strong vs weak response	Marshall, Phan and Çelebi, 1992, Çelebi, 1998
Functionality	Çelebi, 2004 [Needs additional specific instrumentation planning]
Health Monitoring, Damage Detection, Wave Propagation and other Special Purpose Verification	Heo et al, 1997, Sohn et. al., 2003, Safak, 1999, Çelebi et. al. , 2004

A.5. Recording Systems, Constraints and New Developments

Until recently, commercially available recording systems have been limited to a maximum of 12-18 channels *(e.g.* analog recorder CRA-1[A2] , up to 13 channels; the digital K-2, 12 channels; digital Mt. Whitney, 18 channels). Although multiple numbers of recording units may be used to accommodate requisite multiple-channel instrumentation systems for a structure, cost restrictions usually limit the number of channels to 12 or 18 (or multiples thereof), unless more channels are needed or special financing is available. Recently, however, with the development of PC-based data acquisition systems that utilize multiple A/D converters, several dozen channels of data can be accommodated. In such systems, the only constraints are the cost of the sensors and data transmission media required. More modern versions of recording systems are available from various manufacturers.

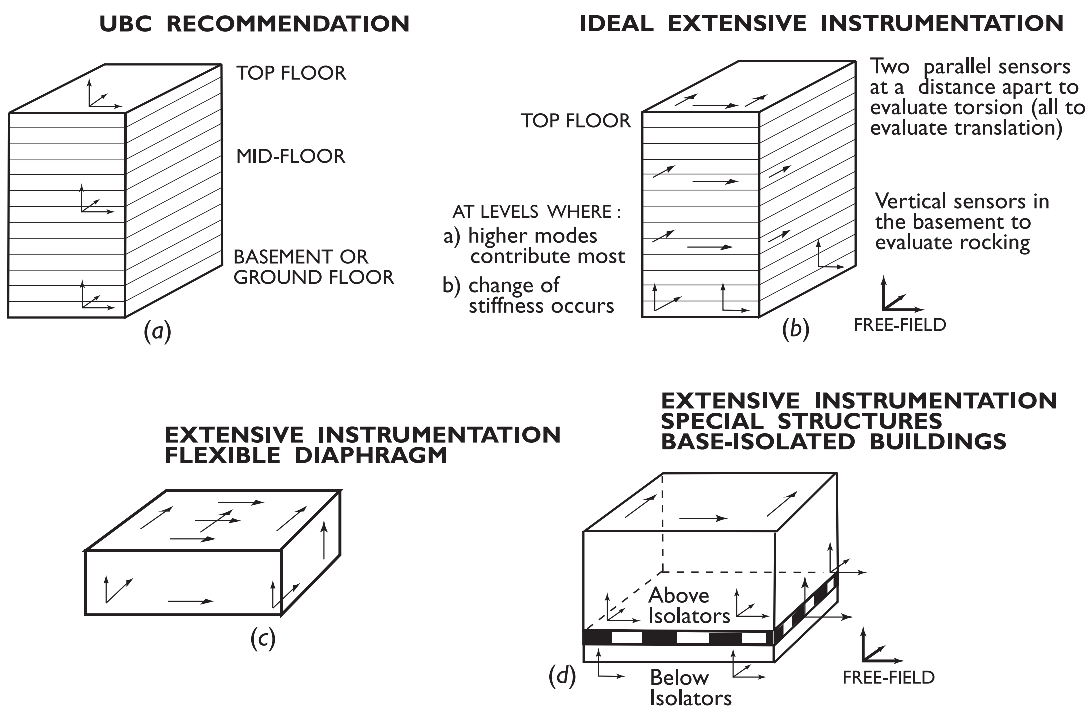


Figure A1. Typical instrumentation schemes

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[A2] Use of commercial names or trademarks cited herein does not imply endorsement of these products by the U.S. Geological Survey.

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