ELSEVIER

Soil Dynamics and Earthquake Engineering 21 (2001) 537–555

SOIL DYNAMICS AND EARTHQUAKE ENGINEERING

www.elsevier.com/locate/soildyn

Evolution of accelerographs, data processing, strong motion arrays and amplitude and spatial resolution in recording strong earthquake motion ☆

M.D. Trifunac*, M.I. Todorovska

University of Southern California, Department of Civil and Environmental Engineering, Los Angeles, CA 90089-2531, USA

Accepted 11 February 2001

Abstract

This paper presents a review of the advances in strong motion recording since the early 1930s, based mostly on the experiences in the United States. A particular emphasis is placed on the amplitude and spatial resolution of recording, which both must be 'adequate' to capture the nature of strong earthquake ground motion and response of structures. The first strong motion accelerographs had optical recording system, dynamic range of about 50 dB and useful life longer than 30 years. Digital strong motion accelerographs started to become available in the late 1970s. Their dynamic range has been increasing progressively, and at present is about 135 dB. Most models have had useful life shorter than 5–10 years. One benefit from a high dynamic range is early trigger and anticipated ability to compute permanent displacements. Another benefit is higher sensitivity and hence a possibility to record smaller amplitude motions (aftershocks, smaller local earthquakes and distant large earthquakes), which would augment significantly the strong motion databases. The present trend of upgrading existing and adding new stations with high dynamic range accelerographs has lead to deployment of relatively small number of new stations (the new high dynamic range digital instruments are 2–3 times more expensive than the old analog instruments or new digital instruments with dynamic range of 60 dB or less). Consequently, the spatial resolution of recording, both of ground motion and structural response, has increased only slowly during the past 20 years, by at most a factor of two. A major (and necessary) future increase in the spatial resolution of recording will require orders of magnitude larger funding, for purchase of new instruments, their maintenance, and for data retrieval, processing, management and dissemination. This will become possible only with an order of magnitude cheaper and 'maintenance-free' strong motion accelerographs. In view of the rapid growth of computer technology this does not seem to be (and should not be) out of our reach.

Keywords: Strong motion recording; Strong motion arrays; Accelerographs; Data processing; Strong motion databases; Damage detection; Structural health monitoring; Permanent displacement

1. Introduction

1.1. Recording strong motion—the early beginnings

In the chapter on 'Ground Motion Measurements' of the first book on Earthquake Engineering in the United States, Hudson [1] introduced the subject by stating that any full-scale experimental study of 'earthquake engineering that is to have a sound scientific foundation must be based on accurate knowledge of the motions of the ground during destructive earthquakes. Such knowledge can be obtained only by actual measurements in the epicentral regions of strong earthquakes'. He continued by stating 'typical seismological observations with their sensitive seismographs are not intended to make measurements in the epicentral regions of strong earthquakes and cannot be adapted to do so effectively. Fundamentally different objectives of the engineer will require a basically different instrumentation than that needed for seismological studies. Such instrumentation must be designed, developed, installed and operated by earthquake engineers, who will be thoroughly familiar with the ultimate practical objectives of earthquake resistant design'. Today these statements are still timely and relevant.

In the epicentral regions of strong and moderate earthquakes, damage to structures is caused by the permanent fault displacement, as well as by strong ground shaking triggered landslides, large scale soil settling, liquefaction,

* Corresponding author. Tel.: +1-213-740-0570; fax: +1-213-744-1426.
E-mail address: trifunac@usc.edu (M.D. Trifunac).

☆ We dedicate this paper to Donald E. Hudson (1916–1999), a pioneer in the field of Earthquake Engineering, and our teacher and mentor. His contributions to academic research and development of earthquake instrumentation are without parallel. With a rare ability to attract, motivate and support young scientists, he created a long and impressive list of PhD graduates who are now professors, researchers and leaders in Earthquake Engineering.

0267-7261/01/$ - see front matter © 2001 Elsevier Science Ltd. All rights reserved.
PII: S0267-7261(01)00013-6

and lateral spreading, but the most significant and wide spread damage is caused by the strong shaking itself. To record the earthquake shaking of the ground and of structures, the US Congress provided funds in 1932 that made it possible to undertake observations of strong motion in California. The instruments were developed by the National Bureau of Standards, the Massachusetts Institute of Technology and the University of Virginia, and the first strong ground motion recorded by these instruments was that of the 10 March, 1933, Long Beach, California, earthquake. The first trigger and recording by an accelerograph was on 20 December, 1932, also in Long Beach, California, of weak motion from a distant earthquake in Western Nevada. The first strong motion accelerogram in a building was registered on 2 October, 1933, in the Hollywood Storage Building, in Los Angeles, California. By 1934–35, all the important elements of a modern experimental Earthquake Engineering observation programs were in place: strong motion observation [2], analysis of records [3], vibration observation in buildings [4], building and ground forced vibration testing [5]; and analysis of earthquake damage [6].

By 1935, some two dozens sites, in California were equipped with strong motion accelerographs (Fig. 1). Eight additional sites were instrumented with a Weed strong motion seismograph [2]. By 1956, there were 61 strong motion stations in the western US [7]. It is estimated that by 1963 about 100 strong motion accelerographs were manufactured by the US Coast and Geodetic Survey and later by the US Oceanographic Survey. By 1970, following the introduction of the first commercially manufactured strong motion accelerograph (AR-240, appeared in 1963), there were about 400 strong motion instruments deployed in the western US. Strong motion observation in Japan began in 1951 [8] and by 1970 there were 500 SMAC and DC-2 accelerographs in Japan [9]. As of the end of 1980 there were about 1700 accelerographs in the United States (1350 of those in California), and by January of 1982 over 1400 accelerographs in Japan.

1.2. Objectives and organization of this paper

This paper presents a review of selected aspects in the evolution of strong motion programs in California, examining some trends in instrumentation development, and data processing, dissemination and interpretation. The extraordinary recent advances in amplitude resolution of recording are contrasted with the neglect of the need to increase the spatial resolution of the recording networks. The examples are drawn mainly from the experience of the authors at the University of Southern California (USC). A comprehensive review on the subject is out of the scope of this paper. This paper is a continuation of an earlier paper by the authors on modeling of structures, on the role of full-scale versus laboratory experiments, and on the priorities in experimental research in Earthquake Engineering [10]. The emphasis of this paper will be on contrasting amplitude versus spatial resolution in recording strong motion, and seismological versus earthquake engineering needs and priorities for strong motion data recording.

The topics covered are strong motion instrumentation (Section 2), data processing, database growth and empirical scaling studies (Section 3), strong ground motion arrays and their adequacy (Section 4), and recording strong motion in buildings, its use for damage detection and monitoring changes in the building periods, understanding the causes of these changes and their significance for the building design codes, structural health monitoring, and recording permanent displacements (Section 5).

2. Strong motion instrumentation

This section reviews the developments in strong motion accelerographs and their performance (threshold levels and dynamic range).

2.1. Threshold recording levels of strong motion accelerographs

Fig. 2 shows a comparison of amplitudes of strong earthquake ground motion with the threshold recording levels of several models of strong motion accelerographs. The weak continuous lines illustrate Fourier amplitude spectra of acceleration at 10 km epicentral distance, for magnitudes $M = 1–7$. The light gray zone highlights the amplitude range of recorded strong motion, and the dark gray zone highlights the subset corresponding to destructive strong motion. The heavier solid curve corresponds to typical destructive motions recorded in San Fernando Valley of metropolitan Los Angeles during the 1994 Northridge, California, earthquake. The lower gray zone corresponds to a typical range of seismological aftershock studies. The three continuous lines (labeled 'quiet', 'noisy' and 'very noisy') show spectra of microtemor and microseism noise, some five orders of magnitude smaller than those of destructive strong motion. The zone outlined by a dotted line shows typical amplitudes of digitization and processing noise. It also serves as a lower bound of triggering levels for most analog accelerographs. The shaded horizontal dashes represent the threshold recording levels for several accelerographs (SMA-1, QDR, Etna, Mt. Whitney and Everest) and transducers (FBA and EpiSensor).

In the late 1920s and the early 1930s, the amplitudes and frequencies associated with strong earthquake ground motion were not known. Considering this fact, the first strong motion accelerographs were remarkably well designed [2,9,11]. During the first 50 years of the strong motion program in the western US, all recordings were analog (on light sensitive paper, or on 70 or 35 mm film). From the early 1930s to the early 1960s, most accelerograms were recorded by the USGS Standard Strong Motion Accelerograph, including the famous 1933 Long Beach [2] and 1940 El Centro [12] accelerograms. In 1963, the first

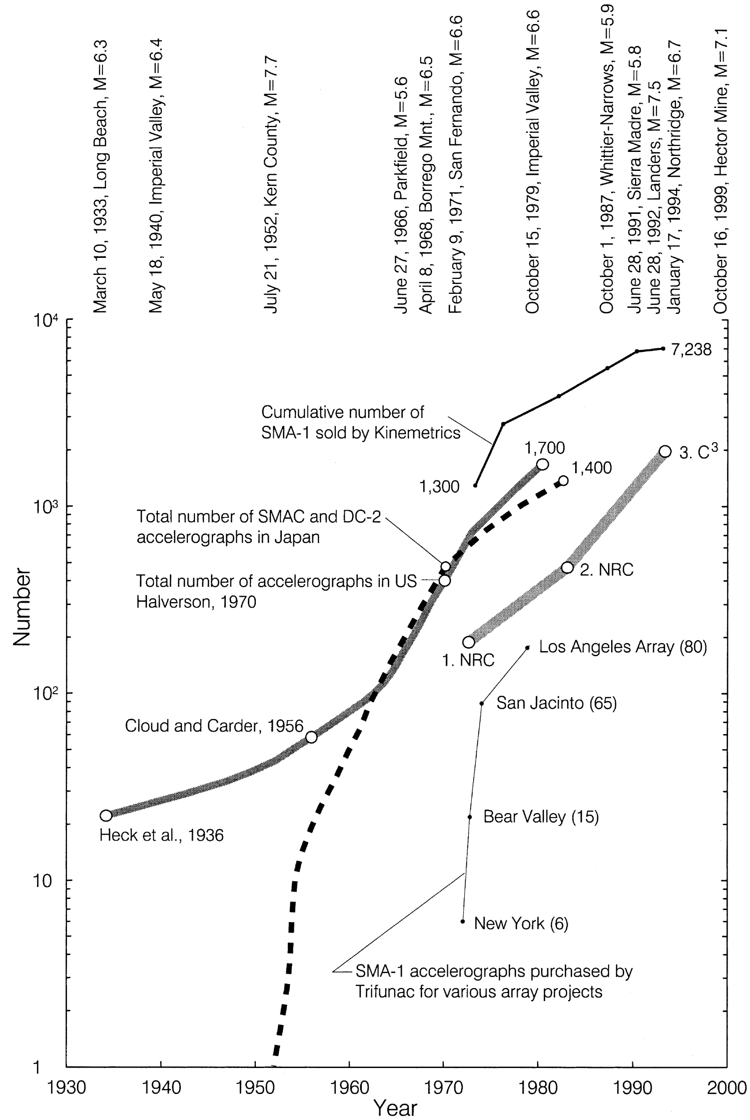

Fig. 1. Cumulative number of strong motion accelerographs in California and in Japan up to 1980, of SMA-1 accelerographs sold worldwide, of SMA-1 accelerographs installed by Trifunac and co-workers for four projects, and of uniformly processed three-component strong motion records used in our three generations of comprehensive empirical scaling studies, developed for Nuclear Regulatory Commission (NRC, 1 and 2) and California Department of Transportation, County of Los Angeles and City of Los Angeles (C^3, 3) Selected California earthquakes contributing to the strong motion database for southern California are also shown on the same time graph.

commercial accelerograph, AR-240, was introduced, and recorded the 1966 Parkfield [13] and the 1971 Pacoima Dam [14] accelerograms. In the late 1960s, the SMA-1 accelerograph was introduced by Kinematrics Inc., (then) of San Gabriel, California [15–17], and recorded so far most of the significant strong motion data in the western US. By the early 1990s, when its production was discontinued, more than 7200 units were sold worldwide (Fig. 1).

Comprehensive reviews of recorded strong motion and of distribution of accelerographs world-wide are beyond the scope of this paper. The reader may peruse example papers on these subjects for Argentina [18], Bulgaria [19], Canada [20], Chile [21], El Salvador [22], Greece [23], India [24–26], Italy [27], Japan [28–30], Mexico [31], New Zealand [32–35], Switzerland [36], Taiwan [37], Venezuela [38], and former Yugoslavia [39]. A useful older review of the world-wide distribution of accelerographs will be found in the paper by Kundson [40].

2.2. Dynamic range of strong motion accelerographs

The dynamic range of an analog strong motion accelerograph ($= 20 \log (A_{max}/A_{min})$, where A_{max} and A_{min} are the largest and smallest amplitudes that can be recorded) equals 40–55 dB and is limited by the width of the recording paper or film, thickness of the trace [41] and resolution of the digitizing system (Figs. 2 and 3). If the digitizing system can resolve more than 5–6 intervals (pixels) per trace width,

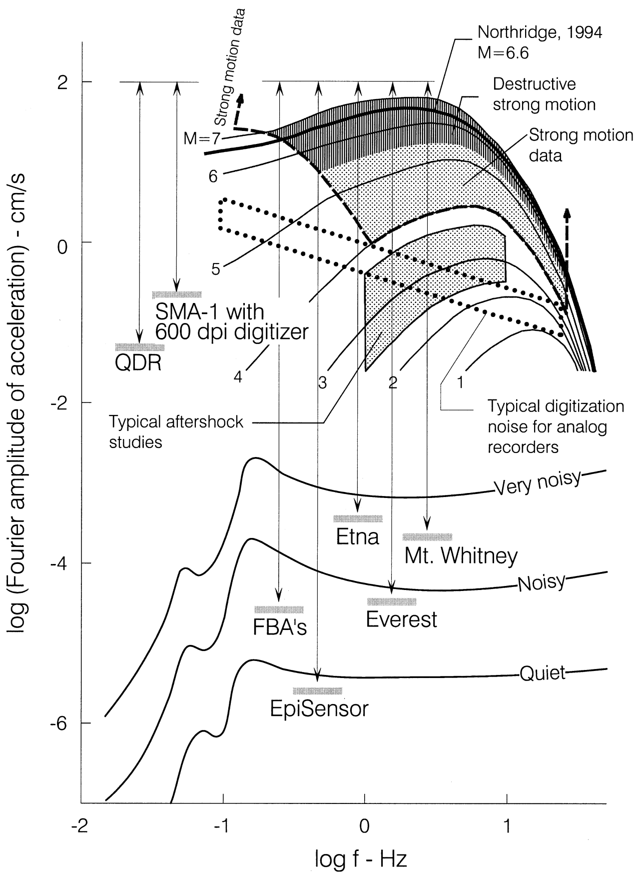

Fig. 2. Comparison of Fourier spectrum amplitudes of strong earthquake ground motion with those of typical aftershock studies and microtremor and microseism noise, threshold recording amplitudes of selected accelerographs, and typical digitization noise for analogue recorders.

then the limit is imposed only by the thickness of the trace [42]. Between 1969 and 1971, with semi-automatic hand operated digitizers, the dynamic range of the processed data was around 40 dB. In 1978, automatic digitizers, based on Optronics rotating drum and pixel sizes 50×50 microns were introduced [43]. For these systems, the amplitudes of digitization noise were smaller [44,45], but the overall dynamic range representative of the final digitized data increased only to 50–55 dB (Fig. 3). By the late 1970s and the early 1980s, there was a rapid development of digital accelerographs due to several factors, including: (1) the advances in solid state technology and commercial availability of many digital components for assembly of transducers and recording systems, (2) the increasing participation of seismologists in strong motion observation, influenced by the successful use of digital instruments in local and global seismological networks, (3) the desire to eliminate the digitization step from data processing, because of its complexity and requirement of specialized operator skills [46], and (4) the expectation that by lowering the overall recording noise it will be possible to compute permanent ground displacements in the near-field. At present, the modern digital recorders have dynamic range near 135 dB. To avoid clutter, in Fig. 2 the growth of dynamic range with time is illustrated only for instruments manufactured by Kinemetrics Inc., of Pasadena, California. A summary of the instrument characteristics prior to 1970 can be found in the book chapters by Hudson [1,47], prior to 1979 in the monograph by Hudson [11], and prior to 1992 in the paper by Diehl and Iwan [48].

The transducer natural frequencies, f_n, and the useable bandwidths of the recorded data increased respectively from about 10 and 0–20 Hz in the 1930s and 1940s to

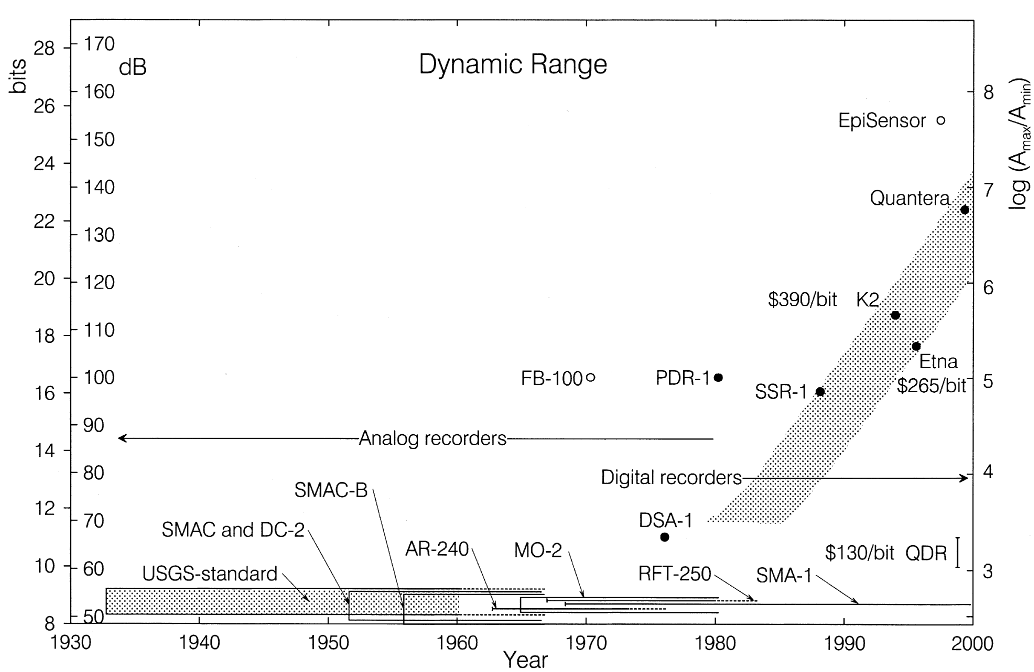

Fig. 3. Comparison of selected strong motion accelerographs and digital recorders in terms of resolution (in bits), dynamic range in dB ($= 20 \log (A_{max}/A_{min})$), and the ratio A_{max}/A_{min}, between 1930 and 2000.

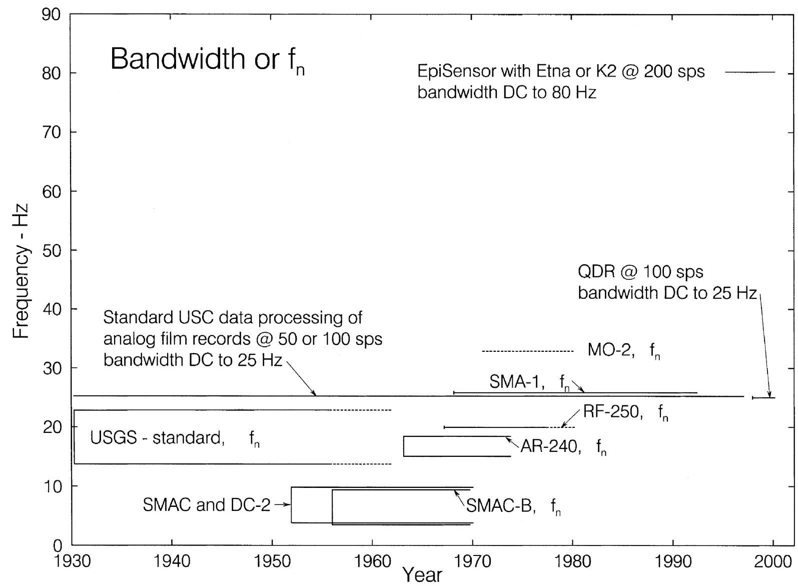

Fig. 4. Natural frequency, f_n, and useable bandwidth of commonly used strong motion accelerographs between 1930 and 2000.

about 50 and 0–80 Hz at present (Fig. 4). For the modern digital instruments, the bandwidth is limited less by hardware, and is chosen so that the useful information in the data, the sampling rate and the volume of digital data to be stored are optimized.

In Fig. 3, the rate of growth in resolution and dynamic range is illustrated via six recorder-transducer systems manufactured by Kinematics Inc. The DSA-1 accelerograph, introduced in late 1970s, had a 66 dB digital cassette recorder with 22 min recording capacity and 2.56 or 5.12 s pre-event memory. In 1980, the PDR-1 digital event recorder was introduced, with 12 bits resolution and 100 dB dynamic range using automatic gain ranging. The SSR-1, introduced in 1991, is a 16 bit recorder with 90 dB dynamic range, that can be used with FBA-23, (force balance accelerometer, 50 Hz natural frequency and damping 70% of critical), and with 200 Hz sampling rate. The Etna recorder (accelerograph) has 18 bits resolution and 108 dB dynamic range, and the K2 recorder has 19 bits resolution and 110 dB dynamic range. They were both introduced in 1990s, and can accommodate force balance type acceleration sensors (FBA or EpiSensor). Finally Quanterra Q330 is a broad-band 24 bit digitizer with dynamic range of 135 dB and sampling rate up to 200 Hz. It can be used with real time telemetry or can be linked to a local computer or recorder. The trend of rapidly increasing dynamic range begins in 1980s.

3. Data processing and databases

This section reviews the progress in software for automatic digitization and processing of accelerograms, the growth of the database of strong motion records available for regression analyses, the increasing detail in these analyses afforded by the increased size of the database, and the adequacy of the database for various analyses.

3.1. Software

The concept of Response Spectrum, in its rudimentary form, was first formulated by Maurice A. Biot in the second chapter of his doctoral dissertation [49,50]. Later, Biot extended this concept to multi-degree of freedom systems [51,52] and presented it as a tool for earthquake resistant design. Following the first recordings of strong motion (1933 Long Beach, 1935 Helena, 1937 Ferndale, and 1940 El Centro earthquakes), a torsional pendulum spectrum analyzer (TPSA) was used to compute response spectra [51]. It took 'an average of 8 h to plot one spectrum curve' [52]. TPSA did not require digitization of the recorded accelerograms. A mechanical follower was used to convert the acceleration to rotation of the end of the torsional wire supporting the pendulum. Fig. 5 shows the characteristics of digitization and data processing as they changed from 1940 to 2000. The heavy dashed line shows the time (in minutes) required for computing standard response spectrum curves (Fourier spectrum and response spectra for five values of damping) for one component of recorded motion. It is seen that the speed of calculating response spectra increased tenfold every 15 years between 1940 and 2000.

Until the mid 1960s, digitization of analog accelerograms was difficult, time consuming, and a highly specialized task performed for a small number of records [11]. The modern digitization of paper and film records of strong motion accelergrams begin in the late 1960s, when digital computers were introduced to university computing centers (Fig. 5). Since the appearance of personal computers in the mid 1980s, the cost of the hardware for digitization systems has become so small that for all practical purposes it can be neglected in planning and organizing strong motion recording and processing laboratories. The present 'limitations' appear to be the lack of highly trained operators for

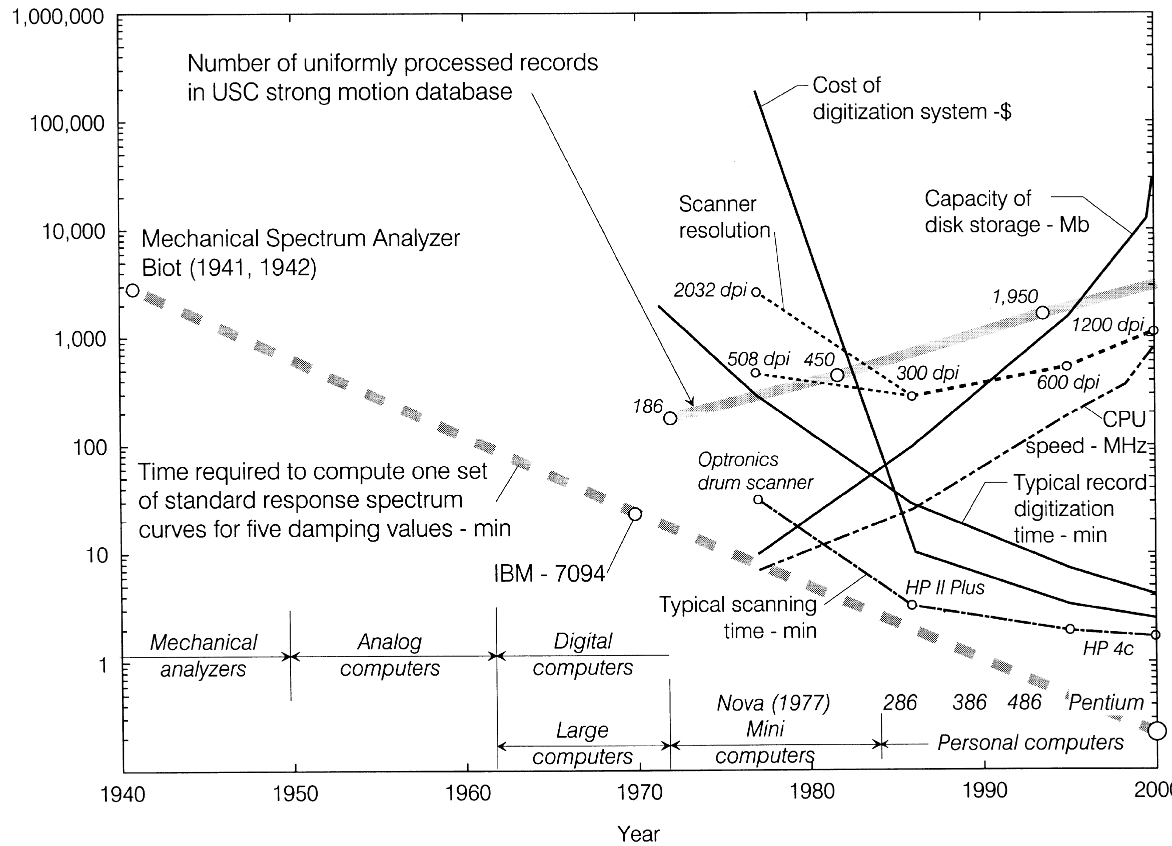

Fig. 5. Trends in capabilities and cost of accelerogram digitization and data processing systems.

the specialized digitization and data processing software [46].

3.2. Databases and empirical scaling laws

The total number of recorded strong motion accelerograms so far in California is not known, but is expected to be several tens of thousands. The number of uniformly processed accelerograms (with complete site information, ready for use in empirical scaling studies) is much smaller (Figs. 1 and 5). As an example, in the development of our three generations of scaling laws, respectively in 1973 [53–55] (the first empirical laws to scale spectra of strong motion directly from the computed spectra of records in the database, instead of scaling peak acceleration, velocity or displacement and fitting a spectral shape), 1983 [56–58] (the first empirical laws to use frequency-source size-magnitude dependent attenuation law), and 1994 [59] (the first empirical scaling laws with path-dependent attenuation), we had available 186, 450 and 1950 three-component records, all from the western US (Fig. 5).

As the number of records grew, the scaling laws became more detailed and included dependence on more source, path and site parameters, and the standard deviation of the observed data from the empirical laws (determined from the distribution of residuals) gradually (but systematically) decreased from more than 0.30 (on logarithmic scale) in the early 1970s to less than 0.25 in the early 1990s. An example of this scatter is shown in Fig. 6, which compares vertical peak accelerations from the 1994 Northridge, California, earthquake ($M_L = 6.4$, $H = 18$ km), which shook metropolitan Los Angeles, with a

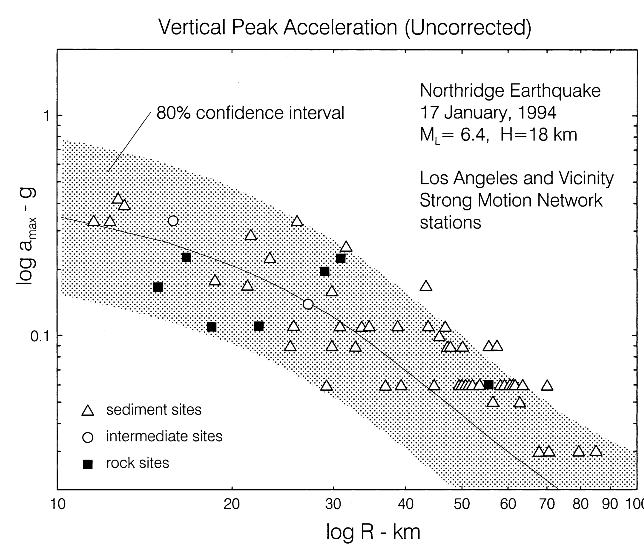

Fig. 6. Uncorrected peak ground accelerations (vertical component) recorded by 63 stations of the Los Angeles Strong Motion Network (the triangular, square and circular symbols) during the Northridge, California, earthquake of 17 January, 1994, and the average trend (the solid line) and 10–90% probability of exceedance interval predicted by the regression model of Trifunac [54] (redrawn from Trifunac et al. [80]).

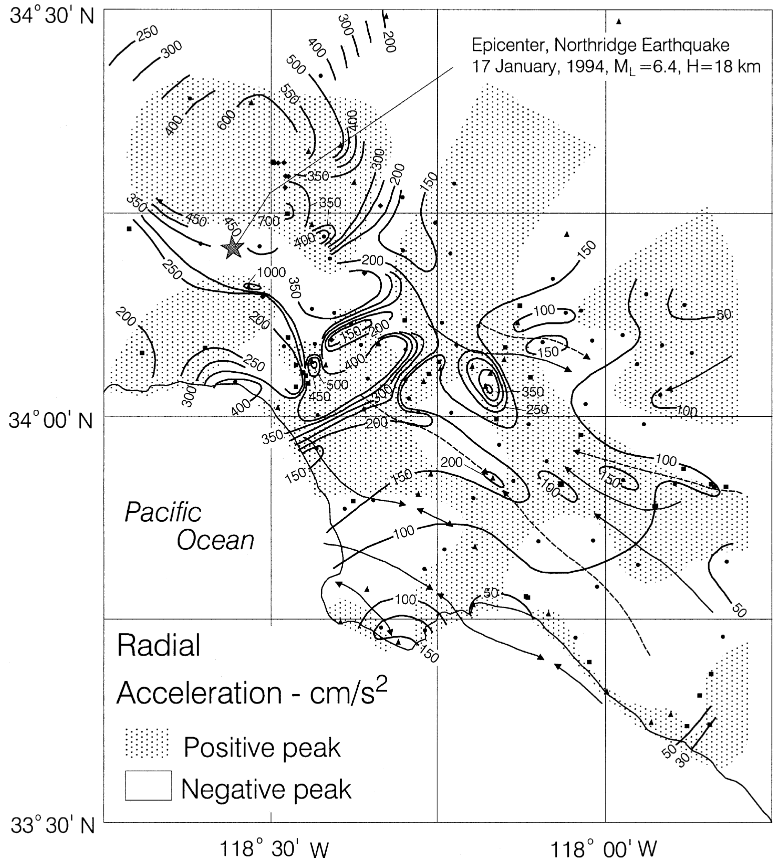

Fig. 7. Contour plot of peak (corrected) ground acceleration (in cm/s^2) for the radial component of motion recorded in metropolitan Los Angeles during the 1994 Northridge earthquake. The shaded region indicates areas where the largest peak has positive sign (redrawn from Todorovska and Trifunac [60]).

prediction by one of our oldest empirical scaling laws for peak amplitudes [54]. The shaded zone represents the 90–10% probability of exceedance interval, and has width consistent with standard deviation of about 0.35 on the logarithmic scale, or factor of five on linear scale.

In Fig. 6, the fluctuations of the data points about the average predicted by the empirical law are large. However, smooth contour maps of peak amplitudes of strong motion for the same earthquake show that the strong motion amplitudes are not so 'random', and that considering only source to station distance R (Fig. 6) is one of the key sources of uncertainty. Fig. 7 shows contours of peak radial acceleration for the 1994 Northridge earthquake; the shaded areas are regions where the peak amplitude has positive sign (i.e. away from the epicenter). The small full symbols show the location of the strong motion stations that recorded this earthquake. The lines with arrows show synclines (if solid) or anticlines (if dashed) in the geological basement. These contours show complex but deterministic variations of the peak amplitudes, principally caused by variations in the regional geology [60]. It is seen that the high frequency strong motion waves tend to 'flow' through low velocity sedimentary layers (e.g. Los Angeles Basin, stretching from the central part of the region in the map towards south-east), sometimes without much attenuation, and with slowly but regularly changing amplitudes. Another remarkable property of strong motion amplitudes that could be seen for the first time in these plots is a highly coherent and systematic variation of the sign of the peak amplitude (see the gray zones in Fig. 7). It is remarkable how slow and deterministic are the fluctuations of peak amplitudes and of their polarity, in particular when we recall that the peak accelerations are associated with short wave lengths. We found equally smooth and gradual variations of spectral amplitudes over large distances [61], which is not surprising because the peak amplitudes of the largest pulses in strong motion govern the amplitudes of response spectra.

Fig. 8 shows the distribution of damaged (red-tagged) buildings (small triangles) following the Northridge earthquake in two regions of metropolitan Los Angeles close to the earthquake source: San Fernando Valley and the Los Angeles–Santa Monica regions. There was a high concentration of damaged buildings in Sherman Oaks, Santa Monica, Culver City and Hollywood. The larger symbols show the locations of strong motion stations, that either recorded this earthquake [62] or were installed (or are planned to be installed) after the earthquake as part of the TriNet project (TriNet is a cooperative network of USGS, CDMG and Caltech). The open circles show stations of the Los Angeles Strong Motion Network (USC), the triangles show stations of the California

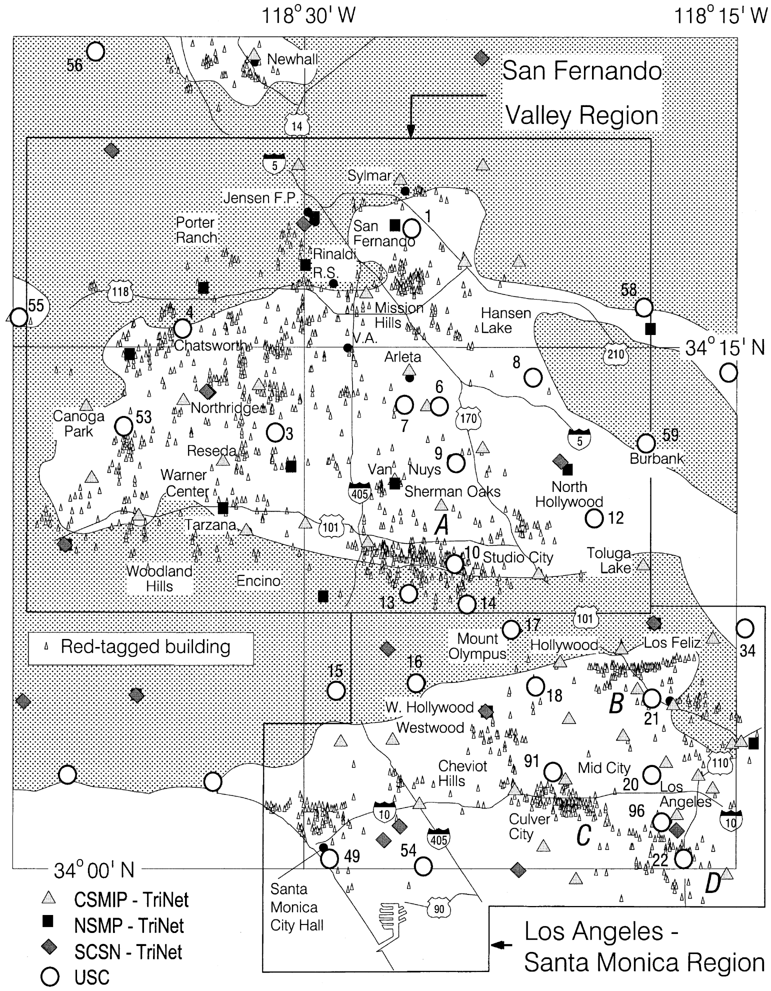

Fig. 8. Distribution of damaged (red-tagged) buildings in San Fernando Valley and Los Angeles–Santa Monica Area following the 1994 Northridge earthquake (small triangles) and of USC and TriNet (current and future) strong motion stations (different larger symbols).

Division of Mines and Geology (CSMIP), the squares show stations of the National Strong Motion Program (USGS), and the diamonds show stations of the Southern California Seismic Network (SCSN). It can be seen that even with TriNet the spatial density of strong motion stations is still at least one order of magnitude too small to detect spatial variations in the nature of strong motion, which could explain the observed variations in the level of damage. We discuss this problem in the next sections.

4. Recording strong ground motion

This section discusses the developments in monitoring strong ground motion by arrays, guided by the needs of theoretical analyses and by the observations of its damaging effects, and discusses the adequacy of the spatial resolution of these arrays.

4.1. Deployment of strong motion arrays

In the early stages of most strong motion programs, the accelerographs are distributed in small numbers over vast areas, with density so low that often only one or two stations are placed in large cities or on important structures (for California prior to 1955, see Cloud and Carder [7] and for Japan prior to 1952 see Takahashi [8]). Also, in the beginning of most strong motion programs (before the early 1970s) it was believed that 'an absolute time scale is not needed for strong-motion work'. However, for instrumentation in structures, 'several accelerographs in the basement and upper floors of a building were connected together for common time marks… so that the starting pendulum that first starts will simultaneously start all instruments' [1,47].

Haskell's pioneering work [63] on near-field displacements around a kinematic earthquake source provided a theoretical framework for solving the inverse problem, i.e. computing the distribution of slip on the fault surface from recorded strong motion. The first papers dealing with this problem [64,65] showed the need for absolute trigger time in strong motion accelerographs [66–68], and for good azimuthal coverage, essentially surrounding the source with strong motion instruments. The first true array of strong motion accelerographs

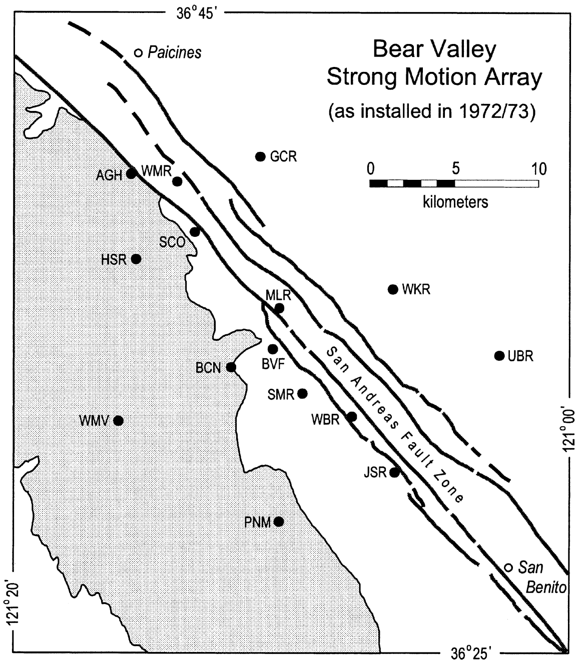

Fig. 9. Bear Valley Strong Motion Array (the first strong motion array with absolute radio time) as installed in 1972/73.

designed in this manner was deployed in 1972 in Bear Valley, central California [69]. It had 15 stations, eight along the San Andreas fault and three on each side, between Paicines and San Benito (Fig. 9). The purpose of this array was to measure near-field strong motion by a small aperture array (20 × 30 km). Since 1973, the Bear Valley array has recorded many earthquakes.

The source inversion studies of Trifunac and Udwadia [64,65] showed the uncertainties associated with inverting the dislocation velocity as function of dislocation rise time and the assumed dislocation amplitudes. To reduce these uncertainties by direct measurement, it was decided that active faults in Southern California should be instrumented with strong motion accelerographs. To this end, and to provide adequate linear resolution that would allow one to follow the dislocation spreading along the surface expression of the fault, we installed the San Jacinto Strong Motion Array in 1973/74 (Fig. 10). This was the first linear (along the fault) strong motion array. So far, it has not recorded a propagating dislocation, because there has not been such an earthquake on the San Jacinto fault since 1974. This array was very successful nevertheless. It recorded strong motion from numerous earthquakes in the highly seismically active area surrounding the array.

In 1979/80, we installed the Los Angeles and Vicinity Strong Motion Network (Fig. 11) to link the San Jacinto Array with the strong motion stations in many tall buildings in central Los Angeles (Figs. 12 and 13). This also constituted our first attempt to find out what can be learned from a two-dimensional surface array with spatial resolution of 5–10 km. Between 1987 (Whittier-Narrows earthquake) and 1999 (Hector Mine earthquake), this network

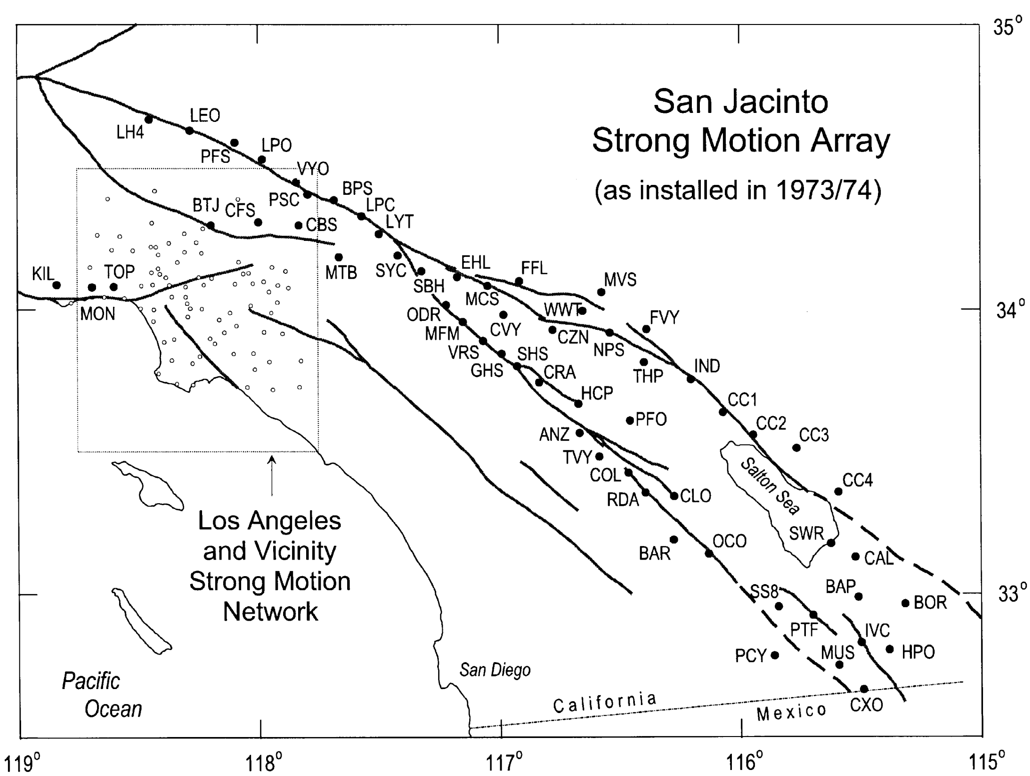

Fig. 10. Stations of the San Jacinto Array (solid dots; currently operated by US Geological Survey) and the Los Angeles and Vicinity Strong Motion Array (open circles, operated by USC) as installed respectively in 1973/74 and in 1979/80.

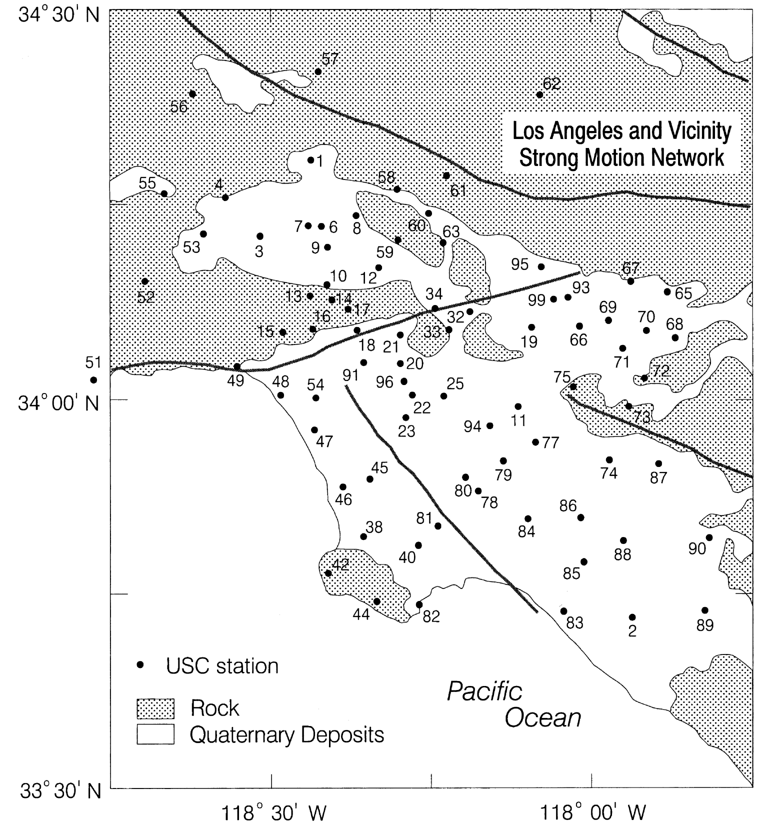

Fig. 11. Los Angeles and Vicinity Strong Motion Network (operated by USC). All stations are in small or one-story buildings (i.e. approximately in 'free field').

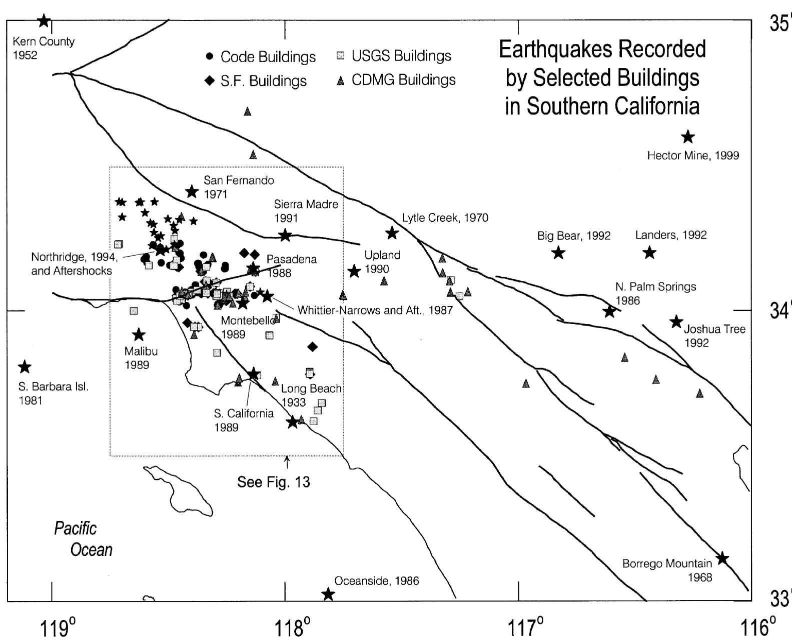

Fig. 12. Larger earthquakes recorded by accelerographs in buildings in Southern California.

contributed invaluable strong motion data (about 1500 three component records), which will be studied by earthquake engineering researchers for many years to come.

Detailed studies and new research and interpretation of strong shaking from the 1994 Northridge earthquake are yet to be carried out and published. So far, our effort has been devoted to data preservation and only general and elementary description of the observed earthquake effects

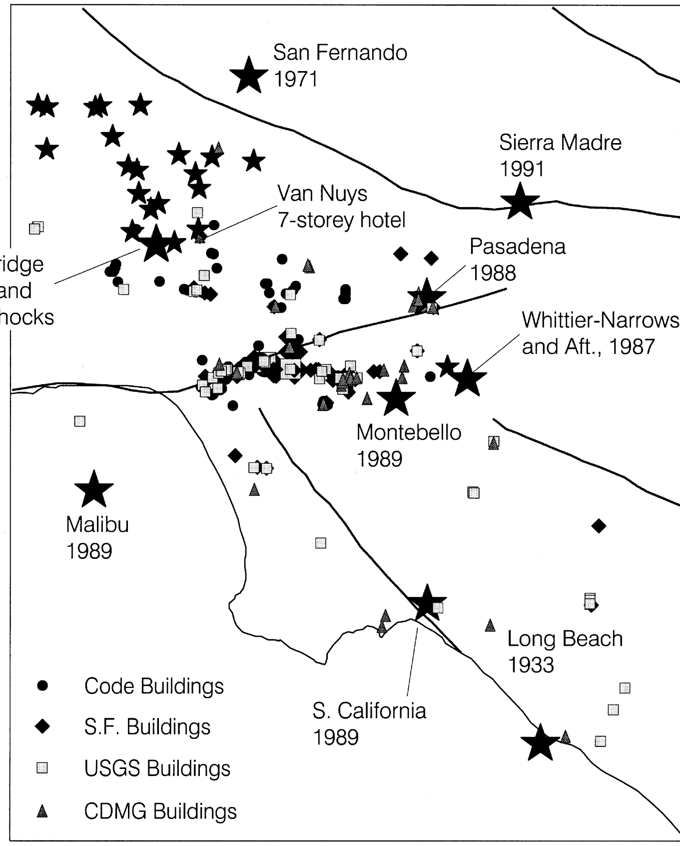

Fig. 13. An enlargement of the rectangular window in Fig. 12 (Los Angeles metropolitan area).

[60–62,70–86]. Nevertheless several important observations have already emerged from the above studies. The first one is that the density of the existing strong motion stations is not adequate to properly describe the spatial variations of the damaging nature of strong motion, and the second one is that the spatial variation of spectral amplitudes, and of peak motion amplitudes and their polarity indicate ‘coherent’ motions (i.e. slowly varying peak amplitudes and the polarity of the largest peak) over distances of the order of 2-km, even for ‘short’ waves, associated with peak accelerations. This suggests that the large scatter in the empirical scaling equations of peak amplitudes (Fig. 6) or of spectral amplitudes of strong motion may be associated in part with the sparse sampling over different azimuths. It was further found that the non-linear response of soils, for peak velocities larger than 5–10 cm/s begins to interfere with linear amplification patterns and that for peak velocities in excess of 30–40 cm/s it completely alters and masks the linear transfer functions determined from small and linear motions at the same stations [78].

4.2. Adequacy of the spatial resolution of strong motion arrays

By comparing the spatial variability of observed damage, with the density of strong motion stations, during the Northridge, 1994, earthquake, in Fig. 8 we illustrated the need for higher than the current density of the observation stations. The spatial variability of amplitudes of strong ground motion results from: (1) the differences along the paths traveled by the strong motion waves and (2) variations in the local site conditions. By recording the motion with dense arrays this variability can be mapped for each contributing earthquake. Then, by some generalized inverse approach and assuming a physical model, the results can be inverted to determine the causes of the observed differences, and to further test and improve the assumed models.

In analyses of strong motion recordings of different earthquakes at a same station, it is sometimes assumed that the local site conditions are ‘common’ to all the recorded events and that only the variations in propagation paths contribute to the observed differences in the recorded spectra. It can be shown, however, that the transfer functions of site response for two- and three-dimensional site models depend on the azimuth and incident angle of strong motion waves [87], so that theoretically calculated or empirically determined spectral peaks in the site specific response are not always excited [84]. In view of the fact that the analyses of reoccurring characteristics of site response can be carried out at any strong motion station where multiple records are available, analyses of such recordings should be carried out prior to the design and deployment of dense strong motion arrays, so that the findings can be used in the design of future dense strong motion arrays. Unfortunately only isolated studies of this type can be carried out at present [84,88], because the agencies archiving and processing strong motion data usually do not digitize and process strong motion data from aftershocks. Also, it should be clear from the above discussion that strong motion stations (in free-field or in the structures) should not be 'abandoned' when the instruments become obsolete, or due to changes in the code or in the organization responsible for maintenance and data collection and archiving. Stations that have already recorded numerous earthquakes are particularly valuable, and their continued operation and maintenance should be of high priority.

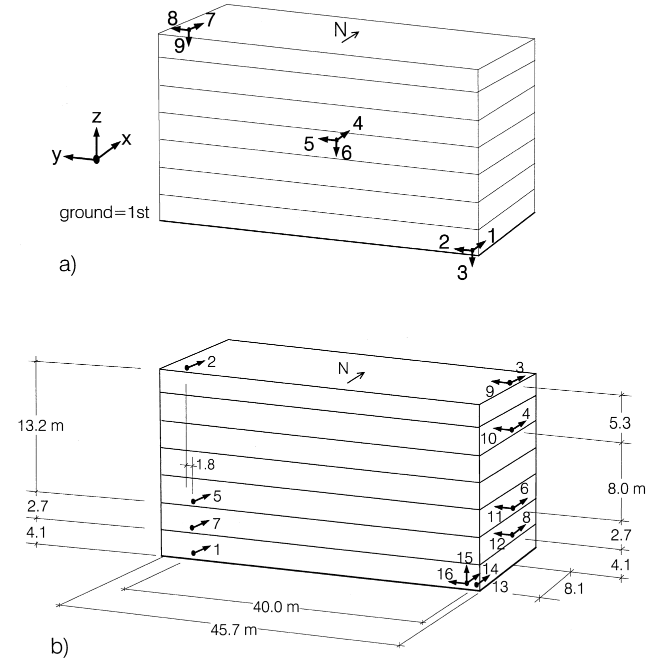

Fig. 14. Location of strong motion recorders in the Van Nuys seven-storey Hotel (VN7SH) before (part a) and after (part b) 1975.

5. Recording strong motion in buildings

This section reviews recording earthquake response of buildings, and the use of these data for identification of soil–structure systems and for damage detection. It also discusses the variability of building periods, determined from strong motion data, its significance for the building codes and for structural health monitoring. At the end, it addresses measurement of permanent displacements in structures and future challenges in recording and interpreting strong motion in buildings.

5.1. Instrumentation

For many years, typical building instrumentation consisted of two (basement and roof) or three (basement, roof and an intermediate level, Fig. 14(a)) self-contained triaxial accelerographs interconnected for simultaneous triggering [47]. The early studies of recorded motions noted that such instrumentation cannot provide information on rocking of building foundations, and that this information is essential for identification of the degree to which soil–structure interaction contributes to the total response [89]. Beginning in the late 1970s, new instrumentation was introduced with a central recording system and individual, one-component transducers (usually force–balance accelerometers; Fig. 14(b)). This instrumentation provided greater flexibility to adapt the recording systems to the needs of different structures, but budget limitations and the lack of understanding of how different structures would deform during earthquake response often resulted in recording incomplete information [10,90–92]. The outcome is that the recorded data is used rarely in advanced engineering research, and usually only to provide general reference for the analyses.

5.2. Damage detection from recorded structural response

One of the reasons for testing full-scale structures, before during and after earthquakes, has been to detect damage caused by severe earthquake shaking [85,86,93–95]. In an ideal setting, the measurements should identify the location, evolution and the extent of damage. For example, the recorded data would show the time history of reduction of stiffness in the damaged member(s), and would identify the damaged member(s). Minor damage, that weakens some structural members but does not alter the form of their participation in the overall stiffness matrix, is expected to modify only those terms of the system stiffness matrix that correspond to those members. This will result in changes of the corresponding mode shapes and natural periods of vibration [96]. Hence, a partially damaged member would reduce the overall stiffness of the system, and would cause the natural periods of vibration to lengthen. A simple approach to structural health monitoring has been to measure these changes in the natural periods (usually the first period, T_1) before and after strong shaking [89]. However, there are at least two problems with this approach. The first problem is that such period changes are usually small, and therefore are difficult to measure accurately [97]. The second problem is that the apparent system period, T, which is the quantity usually measured, depends also on the properties of the foundation soil, that is:

$$T^2 = T_1^2 + T_r^2 + T_h^2 \qquad (1)$$

where T_1-first fixed-base building period, T_r period of the building rocking as a rigid body on flexible soil, and T_h period of the building translating horizontally as a rigid body on flexible soil. The apparent system period, T, can and often does change appreciably during strong shaking, by factors which can approach two [85,86,98]. These changes are caused mainly by non-linear response of the foundation soils, and appear to be self-healing, probably due to dynamic settlement and compaction of soil during aftershocks and small earthquakes. To detect changes in T_l only, special purpose instrumentation must be installed in structures. With the currently available instrumentation in various buildings in California, one can evaluate changes in T, but separate contributions from T_r, T_h and T_l cannot be detected [10,89,99] accurately.

For periods shorter than T_l (this corresponds to short wavelengths and to higher modes of building vibration), the soil–structure interaction effects become more complex and must be analyzed by wave propagation methods [100]. In principle, this higher complexity may offer improved resolution for the purposes of identification of the soil–structure parameters, and depends on our ability to model the system realistically [10], but calls for far more detailed full-scale tests and more dense strong motion instrumentation in buildings. Therefore most studies consider measured data only in the vicinity of T.

To illustrate the order of magnitude of the changes in T_l, consider the model shown in Fig. 15. Assume that this model deforms in shear only, and let the period of the first mode of vibration be equal to T_l. Since the modeshapes represent interference patterns of waves propagating up and down the structure [101–103], T_l is proportional to the travel time H/β. Before any damage has occurred:

$$T_l = 4H/\beta \tag{2}$$

where β is the shear-wave velocity in this structure and H is the height of the building. After strong shaking, some columns may have been damaged at a particular floor. Let h_d be the 'length' of this damaged zone, and β_d be the reduced velocity of shear waves within this damaged zone. Then, the period of the first mode:

$$T_d \sim (H - h_d)/\beta + h_d/\beta_d \tag{3}$$

and the percentage increase in T_d, relative to T_l will be:

$$p = \frac{100h_d}{H}\left(\frac{\beta}{\beta_d} - 1\right) \tag{4}$$

For example, for $H = 20$ m, $h_d = 0.5$ m, $\beta = 100$ m/s and $\beta_d = 50$ m/s, $p = 2.5\%$.

We explored whether simple measurements of wave velocity in structures during strong shaking can be carried out, and whether the location of the observed changes (reduction in apparent wave velocity) will coincide with the areas of observed damage. For this purpose, we analyzed strong motion recordings in a seven-storey reinforced concrete hotel building in Van Nuys, California, (Fig. 14(a),(b)) severely damaged by the 1994 Northridge earthquake [83]. We showed that this task appears to be feasible, and suggested that accurate digitization of accelerograms recorded in buildings is essential, before this type of analysis can be developed and refined further [91–95].

Next assume that recordings of strong motion are available at two adjacent floors (Figs. 14(b) and 15), and that it is possible to measure the velocity of shear waves propagating in the structure [91,92]. Before damage has occurred, the

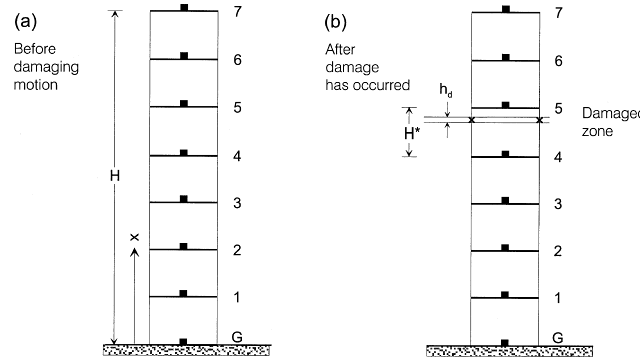

Fig. 15. A multi degree-of-freedom system (a) before and (b) after localized damage has occurred (e.g. in the columns below the 5th floor). The solid squares indicate locations of the strong motion instruments.

travel time between two adjacent floors, i and j, would be:

$$t_{i,j} = H^*/\beta \tag{5}$$

and after damage has occurred

$$t^d_{i,j} = (H^* - h_d)/\beta + h_d/\beta_d, \tag{6}$$

where $H^* = H/N$ is the story height and N is the number of stories (in our example, $N = 7$). The percent change from t_{ij} to $t^d_{i,j}$ is then

$$p = \frac{100h_d}{H^*}\left(\frac{\beta}{\beta_d} - 1\right) \tag{7}$$

For $h_d = 0.5$ m, $\beta_d = 0.5\beta$, and H* = 20/7 m, $p = 17.5\%$. This is N times larger than the percent change in T_l, (because the observation 'length' has been reduced N times).

For typical values $H^* = 3$ m and $\beta = 100$ m/s, $t_{ij} \sim 0.03$ s. The old data processing of strong motion acceleration provided equally spaced data at 50 points/s. Since the early 1990s, most data is processed with time step $\Delta t = 0.01$ s or 100 points/s [43,104,105]. Clearly, to detect time delays of the order of 0.03 s, the accuracy of origin time and the accuracy of the time co-ordinates in digitized and processed data must be better than 0.03 s [91,92,94].

There is one obvious limitation of the above method. It has to do with its ability to resolve 'small' and concentrated damage zones. The method can offer only an order of magnitude ($\sim N$) improvement over measurements of changes in natural frequencies. It is, of course, possible in principle to saturate buildings with transducers, densely distributed, on all structural members, but this is obviously not a practical alternative. The best we can expect, at present, is to have one instrument recording translation per principal direction per floor. In the near future we might have two additional instruments per floor, each recording three components of rotation (two components of rocking about transverse and longitudinal axes, and one component recording torsion). This will correspond to approximately three times better spatial resolution then in the above example.

5.3. Limitations and suggestions for improvement

At present the state-of-the-art of modeling structural responses during strong earthquake shaking is limited by the simplicity and non-uniqueness in specifying the structural models [10]. Our lack of knowledge and absence of constraints on how to define better these models comes mainly from the lack of detailed measurement of response in different structures during very strong earthquake shaking. Thus, until a quantum jump is made in the quality, detail and completeness of full-scale recording of earthquake response, little change will be possible in the modeling techniques. Conceptually and practically, the earthquake resistant design is governed by the procedures and sophistication of the dynamic response analyses that are feasible within the framework of the response spectrum technique. This is essentially a discrete vibrational formulation of the problem. This formulation usually ends up being simplified further to some equivalent single degree-of-freedom system deformed by an equivalent pseudo static analysis, assuming peak deflections (stains), which then determine the design forces. Over the years, the attempts to extend the applicability of this approach to non-linear levels of response have resulted in so many and so complex and overlapping 'correction' factors that the further refinement of the procedures has reached the point of diminishing return. The only way out is to start from the beginning, and use a wave propagation approach in place of the vibrational approach [91,92]. However, again, this requires verification through observation of response, but using far more dense networks of recording stations than what is available today (Fig. 14(b)). This does not mean that nothing new can be learned from the currently available strong motion data. To the contrary, a lot of invaluable new information can be extracted from the recorded but never digitized data, and the methods currently in use can be further refined. At the same time, to prepare sound experimental basis for future developments, far more detailed observational networks in structures must be deployed.

5.4. Variability of the building periods

Our ongoing analyses of building response to earthquake shaking [85,86] show that the time and amplitude dependent changes of the apparent system frequencies are significant. For example, during twelve earthquake excitations of a seven-story reinforced concrete building, between 1971 and 1994, the peak ground velocities, v_{max}, were in the range from 0.94 to 50.93 cm/s. For average shear wave velocity in the top 30 m of soil $\bar{v}_{S,30} = 300$ m/s, the surface strain factors in the free-field [73,106] were in the range from $10^{-4.7}$ to $10^{-2.8}$. During the Northridge earthquake excitation, the largest vertical shear strain associated with rocking of the building was of the order of 10^{-2}. Within the above strain range, the apparent frequencies of the soil–structure system, f_p, varied from 0.4 to 1.5 Hz (factor of 3.8). The corresponding range of rocking accelerations was 10^{-4} to 2×10^{-1} rad/s^2, while the range of rocking angles was 10^{-6} to 2×10^{-2} rad.

From the nature of the changes in f_p ($= 1/T$) versus the excitation amplitudes, it appears that these changes were associated with non-linear response of the soil surrounding the foundation, including both material and geometric non-linearities. Future research will have to show how much the observed range of changes is due to the fact that the building is supported by friction piles. There is no doubt that f_p changes during strong motion for buildings with other types of foundations [98]. What future research must find is how broad these variations are for different types of structures and foundations and how common is the property of different sites that the effective soil stiffness essentially regenerates itself after a sequence of intermediate and small earthquakes. To carry out all this research it will be necessary to deploy more dense instrumentation (in the structures and in the surrounding soil).

5.4.1. Implications for the building codes

Most code provisions approach the earthquake resistant design, by evaluating the base-shear factor $C(T)$ in terms of the 'building period' T. Older analyses of T erroneously assumed that the effects of soil structure interaction are of 'second order' [107], while some more recent studies either do not consider it explicitly [108–110] or approximate T by fitting the functional form of analytical representations of inertial interaction to the observed data on T [110]. All these studies encounter large scatter in the data about the trend predicted by assumed formulae for T, but, with few exceptions, most studies ignore dependence of T on non-linear response of the soil.

Using linear identification techniques, it is common to estimate T for a linear soil–structure system (with or without explicit attempt to identify T_r, T_h and T_1). This means that for most studies that use actual earthquake data [108], the estimates of f_p (that is $1/T$) depend on the average amplitude of the response in the data set included in the analyses, and, because in most cases there are only one or two analyzed earthquake excitations per building, often these estimates are used regardless of the level of excitation. There are other related simplifications in the code provisions, which should be re-evaluated in the light of the fact that f_p experiences the described fluctuations. An obvious (and in part compensating) effect is associated with the relationship between the dependence of the shape of $C(T)$ on magnitude (usually ignored at present) and the dependence of T on the strong motion amplitudes [109,110]. To identify the source and spatial extent of the material undergoing non-linear deformation (soil, structure or both) and contributing to the observed changes in f_p, denser arrays of recording accelerographs will have to be installed.

5.4.2. Implications for structural health monitoring

Most algorithms for structural health monitoring and for control of structural response depend on prior or real-time identification of the structural 'system' in terms of a set of model parameters [10]. When and if the changes of T can be incorporated into advanced non-linear models, in such a way that only a manageable number of identified parameters can describe all the relevant changes (including the ability of the soil to settle and densify during strong shaking, thereby restoring the original system stiffness), will the methods of structural health monitoring and response control be able to function. An interesting and challenging situation will occur when the system responds with frequency higher than expected (on the basis of previous observations). To function, both structural health monitoring and control algorithms must be based on realistic models of structures, and must be able to adapt in ways that can be modeled by other than 'simple', 'equivalent' models with reduced number of degrees of freedom. The soil–structure interaction must be modeled realistically and must be included in the differential equations of the system response.

During the period from 1987 to 1994 (preceding the Northridge earthquake) the apparent frequencies of the seven-story reinforced concrete structure moved within the range from about 0.7 to about 1.8 Hz [85,86]. During this time, the building displayed no visible signs of distress or damage. During the Northridge earthquake, the longitudinal apparent frequencies were between about 0.43 and 0.91 Hz (factor of 2.1), while the transverse frequencies ranged from about 0.47 to 0.92 Hz (factor of 2.0). Thus, to be useful in real life applications, structural health monitoring algorithms should be based not only on monitoring the changes in the system frequency, but must consider the proximity of the observed frequencies to those corresponding to levels of response leading to structural damage [85,86]. This will require: (1) modeling of soil–structure systems where both the soil and the structure can enter the non-linear range of response and (2) development of advanced identification algorithms to detect concurrently the levels of non-linear response in the soil and in the structure. To determine the spatial concentration and distribution of the non-linear response, dense arrays of recording instruments will be required.

5.5. Measurement of permanent displacement

The amplitude resolution of modern digital accelerographs, recording translational components of strong motion at present is approaching 24 bits (about 140 dB). For earthquake engineering applications, this high resolution is not necessary unless rotational components of ground motion are recorded simultaneously, and results in expensive instrumentation. For calculation of permanent displacements of damaged structures and of permanent displacement of the ground in the vicinity of shallow and surface faults, it is essential to record all six degrees of freedom (three translations and three rotations) [111]. Otherwise, the rotational components of strong ground motion begin to appear above the recording noise, starting with resolution of 11–12 bits (~70 dB) [112]. Recording only three translations with resolution higher than 12 bits does not improve the recording accuracy, because the contributions of the rotations become part of the translational record and cannot be eliminated. Thus, to compute permanent deformation in buildings (following damage) and in soil, it will be necessary to develop commercially available accelerometers that measure all six components of motion.

5.6. Future challenges

Studies of the spatial distribution of damaged buildings and breaks in the water distribution pipes following the 1994 Northridge earthquake showed that the damage to one-story residential wood-frame buildings is significantly reduced in the areas where the soil experienced non-linear response [71,72]. This has been interpreted to mean that non-linear soil absorbs part of the incident wave energy, thus reducing the power and the total energy available to damage structures [73,77]. The presence of piles beneath a building foundation increases the scattering of the incident wave energy from the volume of soil and piles, which can be stiffer than the surrounding soil. Then the forced vibration of the entire pile-foundation system creates a volume of anisotropic 'soil' capable of absorbing considerable amount of incident wave energy, and also with a natural ability to recover some or all of its pre earthquake stiffness, via shaking by aftershocks and small earthquakes. It is beyond the scope of this paper to analyze this energy absorption mechanism. It should be clear, however, that it represents a powerful, convenient and inexpensive 'base isolation and energy absorbing system', which in many ways is superior to the conventional base-isolation methods (it does not introduce discontinuities into the soil–structure system, it can be designed as an extension of common foundation systems on piles, and it deflects and absorbs the incident seismic energy before this energy enters the structure). The challenge for future work is to quantify these phenomena, by dense strong motion measuring networks, verify their repeated occurrence and predictability during future full-scale studies, and finally implement this approach into future design of similar pile supported reinforced concrete structures.

6. Conclusions

As of this writing, the cost of a basic 12 bit digital accelerograph (e.g. QDR) is 2–3 times smaller than the cost of 18 and 19 bit recorders (e.g. Etna and K2). Assuming constant total funding of various strong motion programs, this results in 2–3 times fewer instruments and in 2–3 times lower spatial densities in the recording strong motion networks.

The priorities in the current strong motion instrumentation programs appear to be influenced by the seismological point of view. This is reflected in the popular demand for the highest possible dynamic range in recording, the efforts to have as much as possible real-time data transmission to central laboratories, and the new deployment of fewer but more advanced recording systems. Obviously, this strategy has not been optimized for the earthquake engineering needs. The engineers need to have larger densities of recording instruments, which can be achieved if the cost per instrument is reduced by about one order of magnitude. If the experience with the 1994 Northridge, California, earthquake is used as an example and guide, it appears that the current spatial resolution of recording should be increased by at least two orders of magnitude. Such an increase will require major changes in the organization of deployment of instruments, their maintenance, and management of large volumes of recorded data.

The observation of strong motion in buildings and in the free-field for earthquake engineering purposes also requires long observation and continuity. The Hollywood Storage Building in Los Angeles has been instrumented since 1933, and many modern buildings in the city were first instrumented in the late 1960s. For a relatively small number of buildings in California (less than 100), recorded strong motion data has been processed and distributed to researchers for periods longer than 25 years, and there are multiple digitized and processed records (of large and of small motions) available for study. Analyses of multiple recordings at a site [84,88] and in a building [85,86] can be informative and valuable for development of new methods of analysis and design. Multiple earthquake recordings, with amplitudes ranging from ambient noise to destructive strong motion are essential for understanding the sources of non-linearity and the threshold amplitudes beyond which non-linear response analysis must be used [85,86]. To properly understand and quantify all these effects, many already recorded accelerograms from multiple recordings in buildings should be digitized and processed. This data should be distributed to the earthquake engineering researchers for interpretation and analysis. With the modern digital instruments, the recording threshold could be lowered to increase the frequency of triggers. This will produce additional data for analysis of response of full-scale structures.

References

[1] Hudson HE. Ground motion measurements, Chapter 6. In: Wiegel R, editor. Earthquake engineering. Englewood Cliffs, NJ: Prentice-Hall, 1970. p. 107–25.

[2] Heck NH, McComb HE, Ulrich FP. Strong-motion program and tiltmeters, Chapter 2. Earthquake investigations in California 1934–35. Special Publication no. 201, US Department of Commerce, Coast and Geodetic Survey, 1936. p. 4–30.

[3] Neumann F. The analysis of records, Chapter 5. Earthquake investigations in California 1934–35. Special Publication no. 201, US Department of Commerce, Coast and Geodetic Survey, 1936. p. 31–42.

[4] Carder DS. Vibration observations, Chapter 5. Earthquake investigations in California 1934–35. Special Publication no. 201, US Department of Commerce, Coast and Geodetic Survey, 1936. p. 49–106.

[5] Blume JA. The building and ground vibrator, Chapter 7. Earthquake investigations in California 1934–35. Special Publication no. 201, US Department of Commerce, Coast and Geodetic Survey, 1936. p. 117–42.

[6] Martel RR. A report on earthquake damage to Type III buildings in Long Beach, Chapter 8. Earthquake investigation in California 1934–35. Special Publication no. 201, US Department of Commerce, Coast and Geodetic Survey, 1936. p. 143–62.

[7] Cloud WK, Carder DS. The strong motion program of the coast and geodetic survey. Proceedings of the First World Conference on Earthquake Engineering, Berkeley, California 1956;2:1–10.

[8] Takahashi R, The SMAC. Strong motion accelerograph and other latest instruments for measuring earthquakes and building vibrations. Proceedings of the First World Conference in Earthquake Engineering, Berkeley, California 1956;3:1–11.

[9] Halverson HT. Modern trends in strong movement (strong motion) instrumentation, dynamic waves in civil engineering. In: Howells DA, Haigh IP, Taylor C, editors. Proceedings of the Conference Organized by the Society for Earthquake and Civil Engineering Dynamics, Swansea. London: Wiley-Interscience, 1970. p. 341–63.

[10] Trifunac MD, Todorovska MI. NATO advanced research workshop on strong motion instrumentation for civil engineering structures, June 2–5. Istanbul: Kluwer, 1999 (in press).

[11] Hudson DE. Reading and interpreting strong motion accelerograms. Earthquake Engineering Res Inst Monograph. California: Berkeley, 1979.

[12] Trifunac MD, Brune JN. Complexity of energy release during the Imperial Valley, California, earthquake of 1940. Bull Seism Soc Amer 1970;60:137–60.

[13] Housner GW, Trifunac MD. Analysis of accelerograms—Parkfield earthquake. Bull Seism Soc Amer 1967;57:1193–220.

[15] Trifunac MD, Hudson DE. Analysis of Pacoima Dam accelerogram, San Fernando, California earthquake of 1971. Bull Seism Soc Amer 1971;61:1393–411.

[15] Trifunac MD, Hudson DE. Laboratory evaluation and instrument corrections of strong motion accelerographs. Earthquake Eng Res Lab, EERL 70-04. Pasadena, California: California Institute of Technology, 1970.

[16] Todorovska MI. Cross-axis sensitivity of accelerographs with pendulum-like transducers—mathematical model and the inverse problem. Earthquake Engrg and Struct Dynam 1998;27(10):1031–51.

[17] Todorovska MI, Novikova EI, Trifunac MD, Ivanović SS. Advanced accelerograph calibration of the Los Angeles strong motion array. Earthquake Eng and Structural Dynamics 1998;27(10): 1053–68.

[18] Rosas JS. Strong motion accelerograms processing at the National Institute of Seismic Hazard Prevention in Argentina. Bull International Institute of Seismology and Earthquake Engineering 1983;20:21–33.

[19] Nenov D, Georgiev G, Paskaleva I. Strong earthquake ground motion data in EQINFOS: accelerograms recorded in Bulgaria. In: Lee VW, Trifunac MD, editors. Department of Civil Eng. Report no. 90-02. Los Angeles, California: University of Southern California, 1990;1–55.

[20] Milne WG, Rogers GC. Development of strong motion seismographs in Western Canada. Proceedings of the Fifth World Conference on Earthquake Engineering, Rome Italy 1973;1:1042–5.

[21] Midorikawa S, Riddell C, Cruz EF. Strong motion array observation in Santiago, Chile (SMASCH Array). DIE 90-2, Depto. De Ingenieria Estructural, Pontifica Universidad Catalica de Chile, Santiago, Chile, 1999.

[22] Bommer JJ, Udias A, Cepeda JM, Hasbun JC, Salazar WM, Suarez A, Ambraseys NN, Buform E, Cortina J, Madariaga R, Mendez P,

Mezena J, Papastamatiou D. A new digital accelerograph network for El Salvador. Seismol Res Let 1997;68(3):426–37.
[23] Makropoulos KC, Diagourtas D. The Corinthian Gulf (Greece) strong-motion data bank. European seismic design practice, research and application. Proceedings of the Fifth SECED Conference, Balkema, Rotterdam 1995:317–22.
[24] Gupta ID, Rambabu V, Joshi RG. Strong earthquake ground motion data in EQINFOS for India: Part 1A. In: Trifunac MD, Todorovska MI, Lee VW, editors. Department of Civil Engineering Report no. 93-03. Los Angeles, California: University of Southern California, 1993;1–60.
[25] Chandrasekaran AR, Das JD. Strong earthquake ground motion data in EQINFOS for India: Part 1B. In: Trifunac MD, Todorovska MI, Lee VW, editors. Department of Civil Engineering Report no. 93-03. Los Angeles, California: University of Southern California, 1993;1–90.
[26] Chandrasekaran AR. Strong motion arrays in India. Proceedings of the Ninth World Conference on Earthquake Engineering 1988;VIII:131–6.
[27] Iaccarino E, Zaffiro C. Studies of a network of strong motion accelerographs to be installed in Italy. Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, Italy 1973;1:1038–41.
[28] Ohtani K, Takahashi H. The history and status of strong motion earthquake in Japan, NIST SP776, wind and seismic effects. Proceedings of the 21st Joint Meeting of US–Japan Co-operative Program in Natural Resources Panel on Wind and Seismic Effects, Gaithersburg, Maryland: National Institute of Standards and Technology, 1989. p. 172–87.
[29] Tanaka T, Kudo K, Sakane M, Shima E, Osawa Y. An observation network of earthquake strong motions in Suruga Bay region of the Izu Peniusula. Proceedings of the Eighth World Conference on Earthquake Engineering, Vol. II. 1984. p. 15–22.
[30] Okawa I, Kashima T, Koyama S. Dense instrumentation in BRI building and surrounding ground. Proceedings of the UJNR Workshop on Soil–Structure Interaction, USGS Open File Report 1999;99-142:22/1–9.
[31] Anderson JG, Quaas R. Guerrero strong motion array. Proceedings of the Ninth World Conference on Earthquake Engineering, Vol. III. 1988. p. 143–8 Paper SB-5.
[32] Berrill JB. Suggested extensions of the New Zealand strong motion accelerograph network. Bull New Zealand National Society for Earthquake Eng 1979;12(3):264–8.
[33] Cousins WJ. Ground condition at strong-motion recording sites in New Zealand, seismic report. Lower Hutt, N. Zealand: Institute of Geological and Nuclear Sciences, 1996.
[34] Hefford RT, Randel PM. Procedures for operating the New Zealand strong-motion accelerograph network. Bull New Zealand Society for Earthquake Eng 1983;16(1):45–58.
[35] Skinner RI, Stephenson WR, Hefford RT. Strong motion earthquake recording in New Zealand. Bull New Zealand Society for Earthquake Engineering 1971;4(1):31–42.
[36] Smit P. Swiss national strong motion network, strong motion bulletin October 1993–September 1994, Publ. Series of the Swiss Seismological Service, Federal Institute of Technology, no. 107, Zurich, Switzerland, 1994.
[37] Liu K-S, Shin T-C, Tsai Y-B. A free field strong motion network in Taiwan. Terrestrial, Atmospheric and Oceanic Sciences 1999;10(2):377–96.
[38] Malaver A, Chacon C. Venezuelan strong motion accelerograph network. Proceedings of the Eighth World Conference on Earthquake Engineering, Vol. II. 1984. p. 7–13.
[39] Jordanovski LR, Lee VW, Manić MI, Olumčeva T, Sinadinovski C, Todorovska MI, Trifunac MD. Strong earthquake ground motion data in EQINFOS: Yugoslavia, Part 1, Department of Civil Eng. Rep. no. 87-05, University of Southern California, Los Angeles, California, 1987.
[40] Knudson C. Present day status of world wide strong-motion instruments. Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, Italy, Vol. 1. 1973. p. 1028–37.
[41] Halverson H. The strong motion accelerograph. Proceedings of the Third World Conference on Earthquake Engineering, Vols. I-III. 1965. p. 75–93.
[42] Trifunac MD, Udwadia FE, Brady AG. High frequency errors and instrument corrections of strong-motion accelerogram. Earthq Engrg Res Lab, EERL 71-05, Calif. Inst. of Tech., Pasadena, California, 1971.
[43] Trifunac MD, Lee VW. Automatic digitization and processing of strong motion accelerograms, Parts I and II. Department of Civil Eng. Rep. no. 79-15, University of Southern California, Los Angeles, California, 1979.
[44] Lee VW, Trifunac MD, Amini A. Noise in earthquake accelerograms. ASCE, EMD 1982;108:1121–9.
[45] Amini A, Trifunac MD, Nigbor RL. A note on the noise amplitudes in some strong motion accelerographs. Soil Dynamics and Earthquake Eng 1987;6(3):180–5.
[46] Trifunac MD, Todorovska MI, Lee VW. Common problems in automatic digitization of strong motion accelerograms. Soil Dynamics and Earthquake Engrg 1999;18(7):519–30.
[47] Hudson DE. Dynamic tests of full-scale structures, Chapter 7. In: Wiegel R, editor. Earthquake engineering. Englewood Cliff, NJ: Prentice-Hall, 1970. p. 127–49.
[48] Diehl JG, Iwan WD. A technical review of modern strong motion accelerographs. Tenth World Conference on Earthquake Engineering, Balkema, Rotterdam 1992;1:169–75.
[49] Biot MA. Theory of elastic systems vibrating under transient impulse with an application to earthquake-Proof buildings. Proc Nat Acad Sc 1932;19(2):262–8.
[50] Biot MA. Theory of vibration of buildings during earthquakes. Z.A.M.M. Bd 14, Heft 1934;4:213–23.
[51] Biot M. A mechanical analyzer for the prediction of earthquake stresses. Bull Seism Soc Amer 1941;31(2):151–71.
[52] Biot M. Analytical and experimental methods in engineering seismology. Transactions American Society of Civil Engineers 1942:365–408 Paper no. 2183.
[53] Trifunac MD. Preliminary empirical model for scaling Fourier amplitude spectra of strong ground acceleration in terms of earthquake magnitude, source of station distance and recording site conditions. Bull Seism Soc Amer 1976;66:1343–73.
[54] Trifunac MD. Preliminary analysis of the peaks of strong earthquake ground motion—dependence of peaks on earthquake magnitude, epicentral distance and the recording site conditions. Bull Seism Soc Amer 1976;66:189–219.
[55] Trifunac MD, Brady AG. A study of the duration of strong earthquake ground motion. Bull Seism Soc Amer 1975;65:581–626.
[56] Trifunac MD, Lee VW. Frequency-dependent attenuation of strong earthquake ground motion. Department of Civil Eng., Report no. 85-02, University of Southern California, Los Angeles, California, 1985.
[57] Trifunac MD, Lee VW. Preliminary empirical model for scaling pseudo relative velocity spectra of strong earthquake acceleration in terms of magnitude, distance, site intensity, and recording site conditions. Department of Civil Eng., Report no. 85-04, University of Southern California, Los Angeles, California, 1985.
[58] Trifunac MD. Dependence of Fourier spectrum amplitudes of recorded strong earthquake accelerations on magnitude, local soil conditions and on depth of sediments. Earthquake Eng and Structural Dynamics 1989;18(7):999–1016.
[59] Lee VW, Trifunac MD, Todorovska MI, Novikova EI. Empirical equations describing attenuation of peaks of strong ground motion, in terms of magnitude, distance, path effects and site conditions. Department of Civil Eng., Report no. 95-02, University of Southern California, Los Angeles, California, 1995.
[60] Todorovska MI, Trifunac MD. Amplitudes, polarity and time of peaks

of strong motion during the 1994 Northridge, California. Earthquake. Soil Dynamics and Earthquake Eng 1997;16(4):235–58.

[61] Todorovska MI, Trifunac MD. Distribution of pseudo spectral velocity during Northridge, California, earthquake of 17 January, 1994. Soil Dynamics and Earthquake Eng 1997;16(3):173–92.

[62] Todorovska MI, Trifunac MD, Lee VW, Stephens CD, Fogleman KA, Davis C, Tognazzini R. The ML = 6.4 Northridge, California, Earthquake and Five $M > 5$ aftershocks between 17 January and 20 March 1994—Summary of processed strong motion data. Department of Civil Eng. Report 99-01, University of Southern California, Los Angeles, California, 1999.

[63] Haskell NA. Elastic displacements in the near-field of a propagating fault. Bull Seism Soc Amer 1969;59:865–908.

[64] Trifunac MD. A three-dimensional dislocation model for the San Fernando, California, earthquake of February 9, 1971. Bull Seism Soc Amer 1974;64:149–72.

[65] Trifunac MD, Udwadia FE. Parkfield, California, earthquake of June 27, 1966: a three-dimensional moving dislocation. Bull Seism Soc Amer 1974;64:511–33.

[66] Jordanovski LR, Trifunac MD, Lee VW. Investigation of numerical methods in inversion of earthquake source mechanism. Department of Civil Eng., Report no. 86-01, University of Southern California, Los Angeles, California, 1986.

[67] Jordanovski LR, Trifunac MD. Least square model with spatial expansion: application to the inversion of earthquake source mechanism. Soil Dynamics and Earthquake Eng 1990;9(6):279–83.

[68] Jordanovski LR, Trifunac MD. Least square inversion with time shift optimization and an application to earthquake source mechanism. Soil Dynamics and Earthquake Eng 1990;9(5):243–54.

[69] Dielman RJ, Hanks TC, Trifunac MD. An array of strong motion accelerographs in Bear Valley, California. Bull Seism Soc Amer 1975;65:1–12.

[70] Trifunac MD, Todorovska MI. Non-linear soil reponses—1994 Northridge California earthquake. J Geotechnical Eng, ASCE 1996;122(9):725–35.

[71] Trifunac MD, Todorovska MI. Northridge, California, earthquake of January 17, 1994: density of red-tagged buildings versus peak horizontal velocity and site intensity of strong motion. Soil Dynamics and Earthquake Eng 1997;16(3):209–22.

[72] Trifunac MD, Todorovska MI. Northridge, California, earthquake of 17 January 1994: density of pipe breaks and surface strains. Soil Dynamics and Earthquake Eng 1997;16(3):193–207.

[73] Trifunac MD, Todorovska MI. Non-linear soil response as a natural passive isolation mechanism—the 1994 Northridge, California earthquake. Soil Dynamics and Earthquake Eng 1998;17(1):41–51.

[74] Trifunac MD, Todorovska MI. The Northridge, California, earthquake of 1994: fire ignition by strong shaking. Soil Dynamics and Earthquake Eng 1998;17(3):165–75.

[75] Trifunac MD, Todorovska MI. Damage distribution during the 1994 Northridge, California, earthquake, relative to generalized categories of surficial geology. Soil Dynamics and Earthquake Eng 1998;17(4):239–53.

[76] Trifunac MD, Todorovska MI. Amplification of strong ground motion and damage patterns during the 1994 Northridge, California, earthquake. Proceedings of the ASCE Speciality Conference on Geotechnical Earthquake Engineering and Soil Dynamics, Seattle, Washington, Geotechnical Special Publ. no. 75, ASCE, 1. 1998. p. 714–25.

[77] Trifunac MD, Todorovska MI. Reduction of structural damage by nonlinear soil response. J Structural Eng, ASCE 1999;125(1):89–97.

[78] Trifunac MD, Todorovska MI. Can aftershock studies predict site amplification factors—Northridge California earthquake of 17 January 1994. Soil Dynamics and Earthquake Eng 2000;19(4):233–51.

[79] Trifunac MD, Todorovska MI. Long period microtremors, microseisms and earthquake damage: Northridge, California earthquake of 1994. Soil Dynamics and Earthquake Eng 2000;19(4):253–67.

[80] Trifunac MD, Todorovska MI, Ivanović SS. A note on distribution of uncorrected peak ground accelerations during the Northridge, California, earthquake of 17 January 1994. Soil Dynamics and Earthquake Eng 1994;13(3):187–96.

[81] Trifunac MD, Todorovska MI, Ivanović SS. Peak velocities, and peak surface strains during Northridge, California, earthquake of 17 January 1994. Soil Dynamics and Earthquake Eng 1996;15(5):301–10.

[82] Trifunac MD, Todorovska MI, Lee VW. The Rinaldi strong motion accelerogram of Northridge, California, earthquake of 17 January 1994. Earthquake Spectra 1998;14(1):225–39.

[83] Trifunac MD, Todorovska MI, Ivanović SS. Seven-story reinforced concrete building in Van Nuys California: strong motion data recorded between 7 February 1971 and 9 December 1994, and description of damage following Northridge, 17 January 1994 earthquake. Department of Civil Eng., Report no. 99-02, University of Southern California, Los Angeles, California, 1999.

[84] Trifunac MD, Hao TY, Todorovska MI. On reoccurrence of site specific response. Soil Dynamics and Earthquake Eng 1999;18(8):569–92.

[85] Trifunac MD, Ivanović SS, Todorovska MI. Apparent periods of a building. Part I: Fourier analysis. J of Structural Eng, ASCE 2001 (in press).

[86] Trifunac MD, Ivanović SS, Todorovska MI. Apparent periods of a building. Part II: Time-frequency analysis. J of Structural Eng, ASCE 2001 (in press).

[87] Trifunac MD. Surface motion of a semi-cylindrical alluvial valley for incident plane SH Waves. Bull Seism Soc Amer 1971;61:1755–70.

[88] Trifunac MD, Ivanović SS, Todorovska MI. Reoccurrence of site specific response in Montenegro (submitted for publication), 2001.

[89] Luco JE, Wong HL, Trifunac MD. Soil–structure interaction effects on forced vibration tests. Department of Civil Eng., Rep. no. 86-05, University of Southern California, Los Angeles, California, 1986.

[90] Kojić S, Trifunac MD, Anderson JC. A post-earthquake response analysis of the Imperial County Services Building. Department of Civil Eng., Report no. 84-02, University of Southern California, Los Angeles, California, 1984.

[91] Todorovska MI, Ivanović SS, Trifunac MD. Wave propagation in a seven-story reinforced concrete building. Part I: theoretical models. Soil Dynamics and Earthquake Eng 2001 (in press).

[92] Todorovska MI, Ivanović SS, Trifunac MD. Wave propagation in a seven-story reinforced concrete building. Part II: observed wave numbers. Soil Dynamics and Earthquake Eng 2001;21:225–236.

[93] Ivanović SS, Trifunac MD. Ambient vibration survey of full scale structures using personal computers (with examples in Kaprielian Hall). Department of Civil Eng., Rep. no. 95-05, University of Southern California, Los Angeles, California, 1995.

[94] Ivanović SS, Trifunac MD, Todorovska MI. On identification of damage in structures via wave travel times. NATO Advanced Research Workshop on Strong Motion Instrumentation for Civil Engineering Structures, June 2–5 1999 (in press).

[95] Ivanović SS, Trifunac MD, Novikova EI, Gladkov AA, Todorovska MI. Two ambient vibration tests of a seven-story reinforced concrete building damaged by Northridge 1994 Earthquake. Department of Civil Eng. Rep. no. 99-03, University of Southern California, Los Angeles, California, 1999.

[96] Udwadia FE. Uniqueness problems in structural identification from strong motion records. Proceedings of the Sixth World Conference on Earthquake Engineering, India, II. 1977. p. 1010–11015.

[97] Safak E. Detection of seismic damage in multi-story buildings by using wave propagation analysis. Proceedings of the Sixth US National Conference on Earthquake Engineering, Seattle, Washington 1998.

[98] Udwadia FE, Trifunac MD. Time and amplitude dependent response of structures. Earthquake Engrg and Struct Dynam 1974;2:359–78.

[99] Moslem K, Trifunac MD. Effects of soil–structure interaction on the response of buildings during strong earthquake ground motions.

Department of Civil Eng., Report no. 86-04, University of Southern California, Los Angeles, California, 1986.

[100] Todorovska MI, Trifunac MD. Radiation damping during two-dimensional in-plane building–soil interaction. Department of Civil Eng., Report no. 91-01, University of Southern California, Los Angeles, California, 1991.

[101] Todorovska MI, Trifunac MD. Anti-plane earthquake waves in long structures. J Engrg Mechanics, ASCE 1989;115(2):2687–708.

[102] Todorovska MI, Trifunac MD. Propagation of earthquake waves in buildings with soft first floor. J Engrg Mechanics ASCE 1990;116(4):892–900.

[103] Todorovska MI, Trifunac MD. Analytical model for the in-plane building–foundation–soil interaction: incident P-, SV-, and Rayleigh waves. Department of Civil Eng., Report no. 90-01, University of Southern California, Los Angeles, California, 1990.

[104] Trifunac MD, Lee VW. Routine computer processing of strong-motion accelerograms. Earthquake Eng Res Lab, EERL 73-03, Calif. Inst. of Tech., Pasadena, California, 1973.

[105] Lee VW, Trifunac MD. Automatic digitization and processing of accelerograms using personal computers. Department of Civil Eng., Rep. no. 90-03, University of Southern California, Los Angeles, California, 1990.

[106] Trifunac MD, Lee VW. Peak surface strains during strong earthquake motion. Soil Dynamics and Earthquake Eng 1996;15(5):311–9.

[107] Housner GW, Brady AG. Natural periods of vibrations of buildings. J Eng Mech Div, ASCE 1963;89(4):31–65.

[108] Goel RK, Chopra AK. Period formulas for concrete shear wall buildings. J Struct Eng, ASCE 1998;124(4):426–33.

[109] Trifunac MD. Comments on period formulas for concrete shear wall buildings. Journal of Structural Engrg, ASCE 1999;125(7):797–8.

[110] Trifunac MD. Comments on seismic soil–structure interaction in buildings, Paper I: analytical methods (paper no. 16525), and Paper II: empirical findings (paper no. 16526). J Geotech and Geoenvir Eng, ASCE 2000;126(7):668–70.

[111] Graizer VM. On inertial seismometry. Izvestiya, Earth Physics, A.N. SSSR 1989;25(1):26–29.

[112] Trifunac MD, Todorovska MI. A note on the useable dynamic range in accelerographs recording translation. Soil Dynamics and Earthquake Eng 2001;21:275–286.