WORKSHOP PROCEEDINGS:
EFFECTS OF EARTHQUAKE-INDUCED TRANSIENT GROUND SURFACE DEFORMATIONS ON AT-GRADE IMPROVEMENTS

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April 2004
CUREE, the Consortium of Universities for Research in Earthquake Engineering, is a non-profit organization incorporated in 1988 whose purpose is the advancement of earthquake engineering research, education, and implementation. There are 28 University Members of CUREE located in 18 states and approximately 340 individual professor members.

As its name states, CUREE is focused on research, earthquakes, and engineering. A basic criterion for all CUREE projects is the objectivity of the methodological phases of work as well as objectivity in the dissemination or implementation of the project results. CUREE’s Website Integrity Policy provides a succinct statement of this principle:

CUREE values its reputation as an objective source of information on earthquake engineering research and is also obligated to reflect the high standards of the universities that constitute CUREE’s institutional membership. The following Website Integrity Policy is designed to assure those who use the CUREE website that we adhere to criteria appropriate to our non-profit purpose, rather than conforming to minimal prevailing commercial standards.

CUREE provides a means to organize and conduct a large research project that mobilizes the capabilities of numerous universities, consulting engineering firms, and other sources of expertise. Examples of such projects include:

- Organization of the large, multidisciplinary conferences on the Northridge Earthquake for the National Earthquake Hazard Reduction Program federal agencies to bring together researchers and users of research;
- Participation in the SAC Joint Venture (CUREE being the “C”), which conducted a $12 million project for the Federal Emergency Management Agency to resolve the vulnerabilities of welded steel frame earthquake-resistant buildings that surfaced in the 1994 Northridge Earthquake;
- Management of the CUREE-Caltech Woodframe Project, a $7 million project funded by a grant administered by the California Office of Emergency Services, which included testing and analysis at over a dozen universities, compilation of earthquake damage statistics, development of building code recommendations, economic analyses of costs and benefits, and education and outreach to professionals and the general public;
- Establishment for the National Science Foundation of the consortium that will manage the Network for Earthquake Engineering Simulation;
- Conducting research investigations in the USA jointly with Kajima Corporation researchers in Japan since the 1980s;
- Conducting the Assessment and Repair of Earthquake Damage Project, aimed at defining objective standards for application to buildings inspected in the post-earthquake context;
- Participation as a sub-awardee to the Southern California Earthquake Center in the Electronic Encyclopedia of Earthquakes project funded by the National Science Foundation.
The goal of the Assessment and Repair of Earthquake Damage Project is to develop guidelines that provide a sound technical basis for use by engineers, contractors, owners, the insurance industry, building officials, and others in the post-earthquake context. Based on experimental and analytical research and a broad discussion of the issues involved, the guidelines produced by the project will reduce disparities in the evaluation of building damage and the associated need for repairs.
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April 2004

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Preface

This document was prepared as part of the ongoing research project, *Assessment and Repair of Earthquake Damage in Woodframe Construction* administered by the Consortium of Universities for Research in Earthquake Engineering (CUREE) with major funding from the California Earthquake Authority (CEA).\(^1\) The primary objective of the project is to bring sound science and engineering to the important but infrequent undertaking of earthquake damage assessment and repair of typical single- and multi-family woodframe residential buildings in California.

Work on the project is performed by academic institutions, commercial research laboratories, and practicing professionals under contract to CUREE. CUREE’s project manager is Dr. John Osteraas. Technical guidance and oversight for the project is provided by the Advisory Group, membership of which consists of:

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\(^1\) The CEA has provided funding for the project, but is not an author of this document, and is not responsible for its content.
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Dr. Akshay Gupta, of Exponent Failure Analysis Associates in Menlo Park, California, organized the workshop and edited these *Proceedings*.

An electronic version of this document may be found at the CUREE web site, [www.curee.org](http://www.curee.org). Comments, suggestions, and questions are welcomed and should be addressed to John Osteraas (osteraas@exponent.com).
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EXECUTIVE SUMMARY

A workshop was held on May 28, 2003, in Oakland, California, to review and summarize the state-of-science in the area of transient ground surface deformations and their effect on at-grade improvements. The primary objective of the workshop was to identify if and under what conditions, transient ground surface deformations may have an adverse effect on at-grade improvements. Of special interest are the effects these surface deformations may have on residential slabs and foundations.

One of the more controversial issues that arose from the January 17, 1994, Northridge Earthquake was the extent to which shallow residential building foundations and at-grade improvements (concrete floor slabs, driveways, sidewalks, patios, pool decks) can be damaged (i.e., cracked) by an earthquake. There are a number of recognized mechanisms that can cause such damage: earthquake-induced ground failure, pre-existing soil conditions that have undermined or stressed improvements, inertial forces generated by the superstructure, and various combinations of these factors. Following the Northridge Earthquake, cracking of pavement and foundations at sites with stable soil (and often little damage to the superstructure) was commonly attributed to ground surface deformation during the earthquake. The ultimate question to be addressed in this workshop was “Under what circumstances, if any, should transient ground surface deformations be considered as a potential cause of damage to at-grade improvements?” More specifically, the questions to be addressed during the workshop were:

- Can the magnitude of earthquake-induced transient ground surface strains at an arbitrary site be reasonably estimated given the current state-of-science?
- If so, what is necessary to develop an efficient methodology to relate common measures of ground motion to the magnitude of transient ground surface strains?
- If not, what research is needed to develop such a capability?
- What is the nature of earthquake-induced transient ground surface strains experienced at a given site with stable soil (i.e., no earthquake-induced ground failure)?
- What is the nature of demands (force and deformation) due to earthquake-induced transient ground surface strains on a concrete plate on the ground surface that is the size of a typical residential slab (less than 100 ft/30 m in any dimension)?
- Is it possible, given the current state-of-science, to identify any correlation between intensity of ground shaking (MMI or instrumental intensity) and potential for damaging earthquake-induced ground surface deformations?

Four distinguished panelists were invited to present and discuss the state-of-science addressing the foregoing questions. The panelists included Professor Bruce Bolt (University of California at Berkeley, Workshop Chair), Dr. Paul Somerville (URS Corporation), Dr. Norman Abrahamson (Pacific Gas & Electric Company), and Professor Aspasia Zerva (Drexel University). Short biographies of the panelists are included as Appendix A. Presentations by the panelists were
followed by a panel discussion that included audience questions and answers, and discussions on future research and implementation needs.

The primary conclusions reached during the workshop include the following:

- The reported observations of visible waves on the ground surface during strong ground shaking cannot be explained from a seismological perspective, as fundamentally the wave speeds for both body and surface waves are too high for the waves to be visible to the human eye. Under special circumstances where the ground is extremely soft, it may be possible to observe the surface waves, however these waves would likely not be damaging to at-grade improvements, as the wavelength of these waves would be expected to be much larger than the dimensions of the at-grade improvements.

- Conceptually, the general consensus was that the effect of surface strains on at-grade improvements would be inconsequential, except perhaps in the near fault region where large transient peak ground displacements (and consequently large surface strains) may occur.

- The magnitude of earthquake-induced transient ground surface strains at an arbitrary site can be reasonably estimated given the current state-of-science. Surface strains result from wave passage effects, spatial incoherency of ground motion, and variation of site amplification of the ground motion.

- An empirical formulation that relates the transient peak ground displacement at a site to the magnitude of transient ground surface strains at the site was proposed during the workshop. The formulation proposed takes the following form:

\[ \varepsilon = 5 \times 10^{-5} \times PGD(cm) \]

wherein \( \varepsilon \) is the transient ground surface strain, and PGD is the transient peak ground displacement in cm. The formulation was considered to be valid for both in-plane (horizontal) and out-of-plane (vertical) strains and for separation distances as large as 100m.

- The discussion at the workshop focused on ground surface strains and stopped short of transferring ground surface strains into at-grade improvements. Determination of the force and deformation demands on at-grade improvements from the ground surface strains is a kinematic soil-structure interaction problem that requires further research.

- While suggestions were made regarding identifying correlations between intensity of ground shaking or peak ground accelerations and potential for damaging earthquake-induced ground surface deformations, it was felt that the transient peak ground displacement or velocity provide more appropriate and easily obtainable scientific measures that should be used to estimate the ground surface strains.
Based on the presentations and discussions certain topics were identified that require additional research and development efforts in order to better understand, implement, and disseminate the information related to the effect of transient ground surface strains on at-grade improvements. The identified topics include the following:

- **Analysis of recorded peak ground displacement data from seismic arrays to estimate the ground surface strains (horizontal and vertical), thereby assessing and improving the accuracy of the formulation proposed during the workshop.** The analysis could be extended to cover different site soil conditions depending on the nature and extent of data available.

- **Translation of the ground surface strain estimates to force and deformation demands on at-grade improvements.** This involves addressing the kinematic soil-structure interaction problem through numerical modeling.

- **Development of a methodology, and its implementation to produce transient peak ground displacement maps similar to the rapid post-event instrumental intensity, peak ground acceleration and velocity maps produced by the U.S. Geological Survey.**

- **A study to assess the damage potential of transient surface strains to at-grade improvements located at cut/fill intersections.** Instrumentation of cut/fill intersections to assess the ground motion variability across these intersections.

- **Instrumentation of typical at-grade improvements located over stable soil, which would provide a direct measure of the demands induced in these improvements during strong ground shaking.**

- **Correlation of reports of observed ground surface waves with nearby ground motion recordings for well-instrumented earthquakes such as Northridge and Taiwan.**
INTRODUCTION

The lack of clear engineering guidelines for assessing residential earthquake damage, dissociating earthquake damage from non-earthquake damage, and identifying a reasonable scope of repairs for identified earthquake damage was a major contributing factor for public dissatisfaction with the response to the Northridge Earthquake. For example, one of the controversial issues was related to the adequacy of use of epoxy repair for stem wall foundations and slabs-on-grade. That specific issue has since been addressed through research conducted by the NAHB Research Center under contract to CUREE as part of the Assessment and Repair of Earthquake Damage in Woodframe Construction project (Assessment Project).²

The objective of this workshop was to address the related issue: the extent to which shallow residential building foundations and at-grade improvements (concrete floor slabs, driveways, sidewalks, patios, pool decks) can actually be damaged by an earthquake. There are a number of widely recognized mechanisms for such damage: earthquake-induced ground failure (landslide, liquefaction, lateral spreading, dynamic densification), pre-existing soil conditions that have undermined or stressed improvements (differential settlement and/or heave), inertial forces generated by the superstructure, and various combinations of these factors. However, following the Northridge Earthquake, cracking of pavement and foundations at sites with stable soil (and often little damage to the superstructure) was commonly attributed to transient ground surface deformations during the earthquake.

Under contract to CUREE, in conformance with the objectives of the larger Assessment Project, Exponent Failure Analysis Associates (FaAA) organized a workshop on May 28, 2003, to focus on the state-of-science in the area of transient ground surface deformations and their effect on at-grade improvements. The primary objective of the workshop was to identify if, and under what conditions, transient ground surface deformations may have an adverse effect on at-grade improvements, specifically residential slabs and foundations. The ultimate question to be addressed as one aspect of this CUREE project was “Under what circumstances, if any, should transient ground surface deformations be considered as a potential cause of damage to improvements?” More specifically, the questions to be addressed during the workshop were:

- Can the magnitude of earthquake-induced transient ground surface strains at an arbitrary site be reasonably estimated given the current state-of-science?
- If so, what is necessary to develop an efficient methodology to relate common measures of ground motion to the magnitude of transient ground surface strains?
- If not, what research is needed to develop such a capability?
- What is the nature of earthquake-induced transient ground surface strains experienced at a given site with stable soil (i.e., no earthquake-induced ground failure)?

What is the nature of demands (force and deformation) due to earthquake-induced transient ground surface strains on a concrete plate on the ground surface that is the size of a typical residential slab (less than 100 ft/30 m in any dimension)?

Is it possible, given the current state-of-science, to identify any correlation between intensity of ground shaking (MMI or instrumental intensity) and potential for damaging earthquake-induced ground surface deformations?

Four distinguished panelists were invited to present and discuss the state-of-science addressing the foregoing questions. The panelists included Professor Bruce Bolt (University of California at Berkeley, Session Chair), Dr. Paul Somerville (URS Corporation), Dr. Norman Abrahamson (Pacific Gas & Electric Company), and Professor Aspasia Zerva (Drexel University). Short biographies of the panelists are included as Appendix A. Presentations by the panelists were followed by a panel discussion that included audience questions and answers, and discussions on future research and implementation needs. A list of Workshop attendees is included as Appendix B.

These Proceedings represent a consensus document on the topic of the effect of transient ground surface strains on at-grade improvements and are based on the presentations and discussions during the workshop. These Proceedings have been reviewed and approved by the panelists. Materials (presentations, technical papers, summary documents, notes) provided by the panelists for the workshop are included as Appendices C through F.
SUMMARY OF PRESENTATIONS

The following sections provide summaries of the panelists’ presentations and other workshop materials, as well as a summary of the discussions that took place subsequent to the presentations.

Bruce Bolt

A summary statement prepared by Professor Bolt on the issue of seismic ground surface deformation of residential slabs and foundations is included as Appendix C. Salient features of Professor Bolt’s remarks and discussion following his presentation are summarized below:

- Professor Bolt has personally not observed visible waves during earthquakes and has come across only limited reports in the literature on observed ground undulations during earthquakes. The length of the observed surface waves based on historical reports is considered to be of the order of 200 ft.

- Shear (transverse) waves are considered the predominant cause of strains in at-grade improvements. An approximate estimate of the strain in the slab, as proposed by Newmark and tested against finite element modeling, is the ratio of particle velocity in the slab to the apparent shear wave velocity during wave passage as given in Equation 1 below. For cases where no allowance for slip between the slab and soil is considered, the strain in the concrete is of the order of $10^{-4}$. Slip between slab and soil will result in a lower strain value.

A common formulation for the surface strain due to wave passage considered by the panelists is as follows

$$\varepsilon = \frac{PGV}{V_{app}} \quad (1)$$

wherein PGV is the peak ground particle velocity and $V_{app}$ is the apparent horizontal wave propagation velocity.

- Large vertical motion strong enough to break the bond between the soil and the slab is not expected to occur in Rayleigh waves, where the horizontal motion is dominant.

- Ratio of wavelength to dimension of the slab is a key factor. Wavelength is computed as the product of the apparent shear wave velocity of wave passage and the period of the wave. Thus an apparent shear wave velocity value of the order of 2-5 km/s and a wave period of 0.5 seconds results in a wavelength of 1.0-2.5 km, which is significantly larger than the dimension of the slab being considered.

- The problem being addressed is of a single layer over a half space, except in the particular case of slabs on soil. Then the upper layer has a finite dimension (smaller than typical wavelengths), and boundary effects become much more important.
Furthermore, the shear wave velocity in the slab is much higher than the underlying medium, which may result in waves reflecting back completely into the underlying medium depending on the incident angles.

- Rotational strains in soil of the order of $10^{-4}$ radians have been reported, and analysis based on data from the inner ring of the SMART 1 array for a $M=6.5$ event at a distance of about 20 km resulted in strains of the order of $10^{-5}$. These estimates of strain are considered to be too low to result in cracking of a concrete slab. No documented instances of earthquake damage to previously undamaged slabs exist, in the absence of earthquake-induced geotechnical effects (lurching, settlement, liquefaction, etc.).

- While boundary effects and the slip between slab and soil have not been explicitly considered, these effects are highly significant for computation of strain in the slab.

- While it is considered that use of ground motion parameters (such as peak ground velocity and displacement) is more appropriate for assessment of the damage potential of ground motions as opposed to intensity, MMI intensity of VIII and IX tends to emphasize damage to at-grade improvements in their descriptions. Caution is advised in using empirical relationships between peak ground acceleration/velocity and MMI, which may be inaccurate especially at high and low intensity levels.

- A peak ground acceleration of 0.6g (or ground velocity of 60cm/s) is considered a minimum level to result in displacements in slabs at which damage may occur. Use of peak ground velocity (as used with the Newmark method) is considered more appropriate for evaluating the damage potential of ground motions to at-grade improvements as the problem is a deformation problem and velocity provides a measure of the strain introduced in the slab.

**Paul Somerville**

Dr. Somerville’s paper titled “Near Fault Ground Motions and Strains,” is included as Appendix D along with a supporting paper titled “Characterizing Near Fault Ground Motion for the Design and Evaluation of Bridges.” Salient features of Dr. Somerville’s presentation and discussion following his presentation are summarized below:

- The focus of the presentation was characteristics of near fault ground motions; the magnitude of ground motions and of surface strains caused by the passage of seismic waves in firm ground. At-grade improvements are affected by inertial effects from translational and rotational ground motions and also by kinematic effects resulting from incoherency and spatial variation of ground motions.

- Near fault ground motions emanating from strike-slip earthquakes are associated with large dynamic acceleration and velocity pulses in the fault normal component, and a large displacement pulse in the fault parallel component. For dip-slip earthquakes, both the dynamic pulse and displacement pulse are on the fault-normal component.
The large values of acceleration, velocity, and displacement recorded in the near fault region indicate that inertial effects may be important for the behavior of at-grade foundation slabs. The characteristics of the pulse (one-sided or with reversal) may also influence the inertial effects. The period associated with these large pulses was of the order of 1-6 seconds. Translational ground motions recorded in the near fault region may have a significant impact on the inertial interaction between the soil and foundation.

- A large variability is observed in the recorded measurements of rotational ground motions, with the largest rotational velocities obtained being of the order of 1 degree/second.

- Body waves have very high phase velocities, with surface waves having lower (but still very high) horizontal phase velocities. The horizontal phase velocity of surface waves in stiff alluvium is typically observed to be about 1km/sec.

- For body waves, the strain based on wave passage effect, can be estimated using Equation 1. This can also be used as an approximation for surface waves. A summary of strains computed for extreme values of coherent ground motions (peak ground velocity of the order of 2 m/s) was reported as of the order of $10^{-4}$ to $10^{-3}$, with the higher value of strain being estimated for near fault basin edge waves, a condition applicable to the 1995 Kobe Earthquake but not the 1994 Northridge Earthquake.

- Ground strains during the Northridge Earthquake are expected to have been well below the levels given above (of the order of $10^{-4}$ to $10^{-3}$) in most locations. Highest values from near fault shear waves would be expected in the northern San Fernando Basin and the Santa Clarita Basin, and from basin surface waves just south of the Santa Monica fault in the northwestern Los Angeles Basin.

- Reports of visible waves moving across the ground surface during large earthquakes include larger amplitudes, shorter wavelengths, and slower phase velocities than those of any recorded ground motions. Recorded horizontal phase velocities (of the order of 1km/sec) would likely be too high for visual observation. Currently, there is no seismological explanation for visible waves. No such waves have been detected even in well-recorded earthquakes such as the Taiwan and Northridge earthquakes.

- Existence of gravity waves in soft saturated sediments has been proposed in the literature, though none have been observed. The discussion by Dr. Somerville in the workshop was limited to waves in firm ground.

- Spatial incoherency of the ground motion is the non-uniformity of the ground motions at two locations; it induces torsion and strain in the foundation through kinematic interaction. Dr. Somerville referred to the degree of incoherency as a function of frequency based on a model by Dr. Norman Abrahamson. A discussion on the effect of spatial incoherency of ground motion is contained in the next two sections.
While estimates of ground strains from coherently propagating waves for a separation distance of 30m are low, spatial incoherency may result in differential motions, strains, and affect the kinematic interaction between the soil and foundation.

Norman Abrahamson

The presentation given by Dr. Abrahamson during the workshop is included as Appendix E, as are additional support materials he has provided. Salient features of Dr. Abrahamson’s presentation and discussion following his presentation are summarized below:

- A preliminary model for surface strain on stiff soil sites, based on (currently) unpublished reports, was proposed as the following:

\[
\varepsilon = 5 \times 10^{-5} \times PGD(cm) \tag{2}
\]

wherein \(\varepsilon\) is the transient ground surface strain, and PGD is the transient peak ground displacement in cm. The formulation was considered valid for both in-plane (horizontal) and out-of-plane (vertical) strains; the use of the proposed formulation for assessment of out-of-plane strains was thought to be conservative. The formulation was considered valid for separation distances as large as 100m. It was recognized that there is uncertainty associated with this formulation as it is based on a limited evaluation of a limited data set. Further analysis of recorded data would be required to assess and improve the accuracy of the proposed formulation as well as to explicitly extend the formulation to different soil types and for vertical strain estimation.

The ground surface strains are attributed to the following three sources:

1. **Wave passage effect**: This effect is described as the same signal moved along in space. Strain normalized to the peak ground displacement (in cm) is given as:

\[
\frac{\varepsilon}{PGD(cm)} = e^{(5.8-0.69M)} \frac{1}{V_{app}} \tag{3}
\]

wherein PGD is the transient peak ground displacement in cm, M is the magnitude of the earthquake event, and \(V_{app}\) is given in km/s units. For magnitude 6-7, the relationship results in a strain/PGD of about \(2 \times 10^{-5}/\text{cm}\) (using an apparent velocity of 2 km/s), i.e., for a transient peak ground displacement of 1 cm, the strain due to the wave passage effect is \(2 \times 10^{-5}\).

2. **Spatial incoherency effect**: This effect is described as the change in phase angles (does not involve amplitude). Based on empirical incoherency functions and forward modeling of spatially incoherent ground motions, the following value was proposed:

\[
\frac{\varepsilon}{PGD(cm)} = 3.0 \times 10^{-5} / \text{cm} \tag{4}
\]
3. **Variation of site amplification**: This effect covers the change in amplitude across the separation distance of interest. Currently an empirical model exists for quantifying the variability of ground motion in terms of spectral acceleration over short distances. A similar model for peak ground displacement does not exist. Using the model for spectral acceleration at a frequency of 0.7 Hz, a standard deviation of about 0.03 is noted at a separation distance of 10 m, resulting in the following:

\[
\varepsilon = 3.0 \times 10^{-5} / \text{cm}
\]  

(5)

Summation of the wave passage effect, incoherence effect, and site amplification effect results in a strain/PGD value of \(8 \times 10^{-5}/\text{cm}\). Considering the conservative nature of the preliminary model, a value of \(5 \times 10^{-5}/\text{cm}\) was considered appropriate for estimation of ground surface strains normalized to PGD. Thus, for a site having a transient peak ground displacement of 1 cm, the estimated strain is \(5 \times 10^{-5}\). The model is limited to stiff soil sites, and only provides free ground surface strains.

- Surface waves do not appear to be significant for surface strains.

- In response to a question, previously stated information was clarified. The peak ground displacement is only the transient peak ground displacement value and does not include the component of permanent ground displacement. The fault normal component should be used in the near fault region for obtaining the peak ground displacement value.

- The proposed formulation is applicable for a free field problem only, i.e., the strains estimated are at the surface of the soil.

- The preliminary model is considered applicable over a separation distance range of 5-100 m.

- In response to a question related to the use of a frequency of 0.7 Hz for estimation of the site amplification effect, it was clarified that the reduction in the final estimate from \(8 \times 10^{-5}/\text{cm}\) to \(5 \times 10^{-5}/\text{cm}\) accounts for any overestimation.

- In response to a question, it was clarified that coherency was insensitive to site conditions in terms of being rock or soil.

- In response to a question, previous information was clarified. There would be about 30-40% difference in site amplification at 10 m separation between rock and soil sites. The effect would be more pronounced for small magnitude earthquakes, but would reduce for larger magnitude earthquakes.

- In comparing the proposed preliminary model to the Newmark Rule, it was pointed out that for a transient peak ground displacement of 10 cm the two methods resulted in the same strain value of \(5 \times 10^{-4}\). The differences between the two models can be observed in the near fault region where the transient peak ground displacements could
exceed 10 cm. In general, the strain estimates reported in the literature for particular earthquake events and those obtained from the empirical models are mutually consistent, and represent low values of strain except in the near fault region.

- The spacing of recording stations for certain earthquakes is within the range of distances that are relevant to the problem discussed during the workshop. Thus, analysis of recorded peak ground displacement data from seismic arrays to estimate the ground surface strains can be carried out to assess and improve the accuracy of the formulation proposed during the workshop.

Aspasia Zerva

Dr. Zerva’s paper titled “Transient Seismic Ground Strains: Estimation, Modeling and Simulation,” is included as Appendix F. The paper contains a summary of the state-of-science in empirical and analytical evaluation of transient seismic ground strains. Salient features of Dr. Zerva’s presentation and discussion following her presentation are summarized next:

- Dr. Zerva focused on the advantages and limitations of obtaining strain estimates using empirical formulations, numerical codes, and recorded seismic data from either a single station or an array of stations. Effects of spatial variation of ground motion (phase and amplitude variation), as well as wave propagation on seismic ground strains were discussed. Background information on causes of spatial variability, and analytical and empirical coherency models were presented.

- An empirical estimate of maximum strain is obtained as the ratio of the maximum particle velocity to the apparent propagation velocity of motion for both body waves and surface waves. The nuances are in the identification of the dominant contributing waves in the motions (body or surface waves), and the estimation of their apparent propagation velocity.

- Methods (dating from the early 1980s to recently) for obtaining estimates for the apparent wave propagation velocity of body and surface waves were highlighted. Examples of strain evaluation based on the ratio of particle velocity over the apparent propagation velocity of the motions (the peak value of this ratio is Eq. 1) included:
  
  o For the 1971 San Fernando Earthquake the median apparent propagation velocity of shear waves was estimated to be 2.12 km/sec, and, for near fault sites, strains of the order of $10^{-4}$ to $10^{-3}$ were reported.

  o For surface waves, the phase velocity is a function of the thickness of the layer and the frequency of the waves. An example for the evaluation of critical differential displacements and strains at distances compatible to foundation dimensions caused by dispersive surface waves resulted in strains of the order of $10^{-3}$.

  o Parenthetically, it was noted that, for the evaluation of vertical strains caused by a Rayleigh wave, the ratio between its horizontal and vertical components needs to
be considered. For a homogenous half-space with Poisson’s ratio of 0.25 the ratio is –0.68.

- Results obtained by different investigators for transient ground strains resulting from the Northridge Earthquake were presented. Maximum strains differed by an order of magnitude, $10^{-3}$ and $10^{-4}$, when surface and shear waves, respectively, were used in the evaluations at the same location (Rinaldi Receiving Station). Additionally, the analyses that utilized shear waves reported vertical strains less than half the horizontal ones, whereas vertical strains deduced from analyses using surface waves were significantly higher. The discrepancies in the estimates were attributed to a large degree to the different contributing waves with their associated propagation velocities.

- Analytical evaluations of seismic ground strains based on results reported in the literature were also presented:
  - Longitudinal strains and tilts obtained analytically in the vicinity of a thrust fault buried in a layered medium were compared with strain estimates. The approximation of strain as being the ratio of ground velocity to apparent propagation velocity was valid from the tip of the fault forward. The apparent propagation velocity was close to the rupture velocity at the source, indicating that the apparent wave propagation velocity needs to be the velocity in bedrock as opposed to the velocity in the uppermost layer.
  - Attenuation relationships for radial and shear ground strains, indicating their distance and magnitude dependence, have also been reported in the literature. The results were obtained from numerical codes for the evaluation of the peak ground velocity. The relation between peak velocity, $v_{\text{max}}$, and strain was:
    $$\varepsilon = \frac{A v_{\text{max}}}{\beta_1}$$
    For a specific site with a 50m layer under slip faulting conditions, $A$ was reported as approximately 0.2 (for shear strains), $\beta_1$ about 300 m/s, resulting in an effective apparent propagation wave velocity of 1.5 km/s.
  - A normalized peak strain microzonation analysis of the Los Angeles metropolitan area has also been reported in the literature. Seismic strains would vary from $10^{-4}$ to $10^{-3}$ (for high probability, $p = 0.9$, of exceedance in 50 years) to $10^{-2}$ (for very low probability, $p = 0.01$, of exceedance in 50 years).

- Seismic strains can be estimated from recorded data based on two general methods:
  - Single station estimates, which require seismometer recordings at a single station and an estimate for the apparent propagation of the motions, assume that the seismic energy travels as plane waves, the medium is laterally homogenous, and azimuth and horizontal velocity of the motions is known. (The strain estimates
for the Northridge Earthquake highlighted above are based on this type of evaluation. Comparison of such strain estimates with strainmeter data (i.e., recordings of actual strains) revealed discrepancies that were attributed to the lack of consideration of scattering of the waves in the estimates and the consideration that the entire signal propagates with a constant velocity on the ground surface.

- **Seismo-geodetic analyses**, which are based on strain evaluation from spatially recorded (array) data, consider that deformation is spatially uniform and displacements vary linearly. Comparisons of single station and seismo-geodetic results were also reported in the literature. It was noted that, whereas the results from the two approaches were similar in the low frequency range (0.5-1.0Hz), the differences were significant in a frequency range of 4.0-8.0Hz. The researchers reporting the evaluation concluded that in the higher frequency range the assumption of plane wave propagation was no longer valid and that displacements vary nonlinearly over distances of the order of 50m. This conclusion points directly to the significance and contribution of the spatial variability of the motions for the estimation of strains.

- The causes for the spatial variability of seismic ground motions were described. These include, in addition to the propagation of the motions, their amplitude variability (denoted as variation of site amplification in other sections of this summary), and their loss of coherency (denoted as spatial incoherency in other sections of this summary) over extended areas.

- Methods to obtain the average apparent wave propagation velocity of the motions from recordings at an array of sensors, such as the CV and MUSIC methods, were discussed. The resulting velocities can then be used with the single station technique to obtain estimates of surface strains.

- The presentation included a discussion on coherency, which is a measure of the phase variability of the ground motions. Lagged coherency (or similarity in seismic motions) decreases with increasing frequency and separation distance, with this observation being validated from analyses of recorded data. Two commonly used coherency models, specifically the ones developed by Harichandran and Vanmarcke (1986) and Luco and Wong (1986), were discussed.

- Examples in the literature related to the evaluation of the effect of wave propagation and loss of coherency on the response of rigid foundations were presented. Conclusions from the studies were that loss of coherency and wave propagation effects are significant on foundation response, though the studies differed on the relative contribution of the two effects on the response. The differences between the results were attributed to different solution methodologies, and specific conditions considered for the example studies. The effect of the difference in coherency models was illustrated through an example that indicated that depending on the coherency model used, the differential displacement across a separation distance of 100m could range from less than 0.05 cm (strain of the order of $10^{-6}$) to 0.3 cm (strain of the order of $10^{-5}$).
• Examples comparing seismic ground strain simulations to strain estimates obtained using Equation 1 were presented. The Luco and Wong Coherency Model was used with two values for the coherency drop parameter: $2.5 \times 10^{-4}$ sec/m, an average from the ones suggested by Luco and Wong so that the exponential decay of the model agrees with the decay of recorded data, and $1 \times 10^{-3}$ sec/m, which represents significant loss of coherency in the motions. It was shown that Equation 1 (wave propagation effects only) underestimates the ground strains when motions exhibit significant loss of coherency, for reasonable values of the apparent wave propagation velocity. The example underscored the importance of coherency effects in ground surface strain estimation.

• In response to the discussion regarding the parameter controlling the rate of decay in the Luco and Wong Coherency Model, it was noted by an audience member that they had obtained a value of about $6-8 \times 10^{-4}$ sec/m for alluvial soils. The value is higher than but of the same order of magnitude as suggested by Luco and Wong ($2-3 \times 10^{-4}$ sec/m), which would indicate a lower coherency for alluvial soils.

• The characteristics of coherency are observed to be different at shorter separation distances as opposed to longer separation distances. For example, studies have noted the coherency to decay faster with frequency than separation distance at shorter separation distances (< 100m), while other studies have noted the decay with frequency and distance to be similar at larger separation distances (> 100m). This suggests that different factors control the loss of coherency in the data at shorter and longer separation distances.

• Equation 1 may be adequate to estimate ground surface strains from surface waves (which have a lower wave propagation velocity as opposed to body waves), as wave propagation effects overshadow effects of loss of coherency.

• Local coherency models were discussed, with the conclusion that site stochasticity may contribute more to the total values of seismic ground strains as opposed to incident motion variability. However, additional issues need to be considered in order to properly evaluate local coherency effects.

• Comparative evaluations between recorded motions and reconstructed coherent, propagating motions were presented. Arrival time perturbations, i.e., differences in the estimated arrival times at various recording stations based on a constant apparent wave propagation velocity and actual arrival times, were presented, as was the variability in phases and amplitudes of the recorded data about the coherent motions’ phase and amplitude. All of these characteristics induce differential motions and strains.

• In conclusion, while Equation 1 is generally used for ground surface strain estimation, the estimation is influenced by the choice of the apparent wave propagation velocity and needs to consider the variation in amplitude and phase of the motion across short separation distances, as well as perturbations in the arrival times of the waves.
Discussion Following Presentations

While most of the questions and answers that followed the individual presentations are included in the summaries above, the following notes summarize the discussion subsequent to all four presentations.

- In response to questions regarding the estimation of surface strains using Equation 2, the following considerations were presented:
  - It was believed that Equation 2 presented a reasonable engineering model for estimation of surface strains.
  - Equation 2 was applicable for estimation of free ground surface strains, i.e., it did not include any soil-structure interaction effects.
  - Local coherency effects were implicitly included in the formulation through consideration of amplitude variation.

- While no seismological explanation was available for the reported visible ground surface waves, it was felt that these waves may be possible on extremely soft soils where the apparent wave propagation speed slows down. These waves were related to hydrodynamic waves, which would have no effect of at-grade slabs. A potential future study was suggested wherein observations of waves on the ground would be correlated to ground motion recordings for the Taiwan earthquakes.

- Incoherency of ground motions was considered an important contributor to surface strains.

- The observation was made that the strain estimates reported in the literature, and those obtained from Equation 2 were of the same order of magnitude, and in general, small. Other than the near fault region, where the transient peak ground displacement may be high, the strain estimates were considered to be too low to result in any damage to at-grade slabs and foundations. Furthermore, it was pointed out that any differential motion across the slab would first be accommodated within control joints, or previously existing cracks in the slab.

- In response to a question regarding vertical (out-of-plane) strains, it was suggested that vertical strains be conservatively taken as equal to the horizontal (in-plane) strain estimates. The rationale was that while the amplitude of vertical velocity is lower, it is associated with larger spatial incoherence. More specific studies are required to develop better estimates for the vertical strains.

- An observation was made that surface strains would result in in-plane shear, which should manifest as diagonal shear cracking of the slabs, which is inconsistent with field observations. Furthermore, cracking due to shear strains should be pervasive across the slab, which also is inconsistent with field observations.
• Shrinkage and pre-stressed nature of the slab needs to be considered.

• A discussion took place regarding the effect of cut/fill transitions on wave propagation and whether cracks that are observed at cut/fill intersections could be related to some peculiar motions. Further investigation of this issue may be warranted, including instrumentation of cut/fill sites.
CONCLUSIONS AND IDENTIFIED RESEARCH NEEDS

The primary conclusions developed during the workshop and additional research issues of further interest are summarized below.

Conclusions

The primary conclusions reached during the workshop are as follows:

- The reported observations of visible waves on the ground surface during strong ground shaking cannot be explained from a seismological perspective, as fundamentally the wave speeds for both body and surface waves are too high for the waves to be visible to the human eye. Under special circumstances where the ground is extremely soft, it may be possible to observe the surface waves, however these waves would likely not be damaging to at-grade improvements.

- Conceptually, the general consensus was that the effect of surface strains on at-grade improvements would be inconsequential, except perhaps in the near fault region where large transient peak ground displacements (and consequently large surface strains) may occur.

- The magnitude of earthquake-induced transient ground surface strains at an arbitrary site can be reasonably estimated given the current state-of-science. Surface strains result from wave passage effects, spatial incoherency of ground motion, and variation of site amplification of the ground motion.

- An empirical formulation that relates the transient peak ground displacement at a site to the magnitude of transient ground surface strains at the site was proposed during the workshop. The formulation proposed takes the following form:

\[
\varepsilon = 5 \times 10^{-5} \times PGD(cm)
\]

wherein \(\varepsilon\) is the transient ground surface strain, and \(PGD\) is the transient peak ground displacement in cm. The formulation was considered to be valid for both in-plane (horizontal) and out-of-plane (vertical) strains and for separation distances as large as 100m.

- The discussion at the workshop focused on ground surface strains and stopped short of translating the ground surface strains onto at-grade improvements. Determination of the force and deformation demands on at-grade improvements from the ground surface strains was considered a soil-structure interaction problem that requires further research.

- While suggestions were made regarding identifying correlations between intensity of ground shaking or peak ground accelerations and potential for damaging earthquake-induced ground surface deformations, it was felt that the transient peak ground
displacement or velocity provide more appropriate and easily obtainable scientific measures that should be used to estimate the ground surface strains.

Identified Research Needs

Based on the presentations and discussions certain topics were identified that require additional research and development efforts in order to better understand, implement, and disseminate the information related to the effect of transient ground surface strains on at-grade improvements. The identified topics include the following:

- Analysis of recorded peak ground displacement data from seismic arrays to estimate the ground surface strains (horizontal and vertical), thereby assessing and improving the accuracy of the formulation proposed during the workshop. The analysis could be extended to cover different site soil conditions depending on the nature and extent of data available.

- Translation of the ground surface strain estimates to force and deformation demands on at-grade improvements. This involves addressing the kinematic soil-structure interaction problem through numerical modeling.

- Development and implementation of a methodology to produce transient peak ground displacement maps similar to the rapid post-event instrumental intensity, peak ground acceleration and velocity maps produced by the U.S. Geological Survey.

- A study to assess the damage potential of transient surface strains to at-grade improvements located at cut/fill intersections. Instrumentation of cut/fill intersections to assess the ground motion variability across these intersections.

- Instrumentation of typical at-grade improvements located over stable soil, which would provide a direct measure of the demands induced in these improvements during strong ground shaking.

- Correlation of reports of observed ground surface waves with nearby ground motion recordings for well-instrumented earthquakes such as Northridge and Taiwan.
APPENDIX A – BRIEF BIOGRAPHIES OF WORKSHOP PANELISTS

Professor Bruce Bolt
Dr. Paul Sommerville
Dr. Norm Abrahamson  (biography not available at time of printing)
Professor Aspasia Zerva
Professor Bruce Bolt

BORN: February 15, 1930

Degrees and Diplomas: B.Sc. (with honors), New England University College of the University of Sydney, 1952; M.Sc., University of Sydney, 1955; Ph.D., University of Sydney, 1959; D.Sc., University of Sydney, 1972; Diploma of Education, Sydney Teachers College, 1953.

Academic and Research Career (partial list):

Director, Seismographic Stations, University of California, Berkeley, 1963-89.
Chairman, Graduate Council, University of California, Berkeley, 1980-82.
Member, Coordinating Committee for Graduate Affairs, University of California, 1980-81.
Vice-chairman, Academic Senate, University of California, Berkeley, 1992-93.
Chair, Academic Senate, University of California, Berkeley, 1993-96.
Visiting Professor, American University of Armenia, Yerevan, Summer, 1992.
Board of Trustees, California Academy of Sciences, 1980-92 (President, 1982-85).
Commissioner, Seismic Safety Commission, California, 1978-93 (Vice Chairman, 1983-84; Chairman, 1984-86).

Elected or Appointed Positions (partial list):

Advisory Committee on the International Decade for Natural Hazard Reduction, National Research Council 1987-91
Secretary, Board of Directors, California Universities for Research in Earthquake Engineering (CUREE) 1988-90
President, The Faculty Club, University of California, Berkeley 1996-

Honors (partial list):

Fulbright Research Scholar 1960
Fellow, American Geophysical Union 1967
Fellow, California Academy of Sciences 1972
Member, U.S. National Academy of Engineering 1978
Overseas Fellow, Churchill College, Cambridge 1980
Associate (Foreign Member), Royal Astronomical Society 1987
Fellows Medal, California Academy of Sciences 1989
University of California, Berkeley Citation 1993
Dr. Paul Somerville

Dr. Somerville is Principal Engineering Seismologist at URS Corporation and manager of its Pasadena office. He develops and applies seismological methods for establishing seismic design ground motions in earthquake engineering practice. He is involved in the development of building codes through participation in the Seismic Hazards Mapping Technical Subcommittee of the Provisions Update Committee of the Building Seismic Safety Council, which revises the NEHRP Recommended Provisions for Seismic Regulations for New Buildings. He is an affiliate member of the Structural Engineers' Association of California, and has been involved in code development work as a member of the Base Isolation Subcommittee. He has lead developments in the engineering characterization of near-fault ground motions, and his response spectral models of near-fault ground motions formed the basis for quantifying the near-source factor in the 1996 SEAOC Blue Book and the 1997 Uniform Building Code.

Professional Committees:

Earthquake Engineering Research Institute
   Past Chairman, Research Policy Committee
   Past Member, Board of Directors

Building Seismic Safety Council
   Member, Technical Subcommittee 1, Provisions Update Committee

Structural Engineer's Association of Southern California, Seismology Committee
   Base Isolation Subcommittee

Seismological Society of America
   Board of Directors

Education

University of British Columbia, Vancouver, Canada: Ph.D., Geophysics, 1976
University of British Columbia, Vancouver, Canada: M.Sc., Geophysics, 1972
University of New England, Armidale, Aust.: B.Sc.(Honors), Geophysics, 1969
University of Sydney, Australia: Diploma in Education, 1966
University of New England, Armidale, Australia: B.Sc., Physics, 1965

Professional History
URS Corporation, Principal Seismologist, 1976-date
Earthquake Research Institute, Tokyo University, Research Fellow, 1977-1979
York University, Ontario, Canada, Lecturer in Physics, 1976
Papua-New Guinea University of Technology, Lae, Senior Tutor in Physics, 1969
Brandi High School, Wewak, Papua-New Guinea, Education Officer, 1966-67
Dr. Norm Abrahamson

Biography not available at the time of printing.
Professor Aspasia Zerva

Current Position: Department of Civil and Architectural Engineering
Drexel University, Philadelphia, PA 19104
Phone: (215) 895-2340 (Office); (215) 895-1363 (Fax)
E-mail: aspa@drexel.edu
URL: http://www.pages.drexel.edu/~zervaa

Education:
University of Illinois at Urbana-Champaign,
Department of Civil Engineering, Ph.D., 1985
Department of Theoretical and Applied Mechanics, M.Sc., 1982
Aristoteleion University of Thessaloniki, Greece
Department of Civil Engineering, Diploma (Hons), 1980

Experience:

2000- Present Professor, Department of Civil, Architectural & Environmental Engineering
Drexel University

2001- Visiting Professor, Department of Civil and Environmental Engineering
Drexel University

2003-2004 Visiting Fellow, Department of Civil and Environmental Engineering
Princeton University

2002-2003 Program Director, Earthquake Engineering Research Centers
Division of Engineering Education and Centers, Directorate for Engineering
National Science Foundation

1998-1999 Visiting Associate, Division of Engineering and Applied Science
California Institute of Technology

1995-1996 Visiting Professor, Department of Civil Engineering
Aristoteleion University of Thessaloniki, Greece

1992-2000 Associate Professor, Department of Civil and Architectural Engineering
Drexel University

1989-1992 Assistant Professor, Department of Civil and Architectural Engineering
Drexel University

1985-1989 Assistant Professor, Department of Civil Engineering
City University of New York


Publications: Over 80 publications (in journals, books, conference proceedings and reports).


## APPENDIX B – WORKSHOP ATTENDEES

<table>
<thead>
<tr>
<th>Name</th>
<th>Role/Panel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Norman Abrahamson</td>
<td>Presenter</td>
</tr>
<tr>
<td>Nesrin Basoz</td>
<td></td>
</tr>
<tr>
<td>Bruce Bolt</td>
<td>Workshop Chair, Presenter</td>
</tr>
<tr>
<td>David Breiholz</td>
<td>CUREE Project Advisory Panel</td>
</tr>
<tr>
<td>Debanik Chaudhuri</td>
<td></td>
</tr>
<tr>
<td>Kelly Cobeen</td>
<td></td>
</tr>
<tr>
<td>Pendo Duku</td>
<td></td>
</tr>
<tr>
<td>Dan Dyce</td>
<td>CUREE Project Advisory Panel</td>
</tr>
<tr>
<td>Andrew Gillespie</td>
<td>CUREE Project Advisory Panel</td>
</tr>
<tr>
<td>Akshay Gupta</td>
<td>Workshop Project Manager</td>
</tr>
<tr>
<td>Ed Kavazanjian</td>
<td>CUREE Project Advisory Panel</td>
</tr>
<tr>
<td>Kearson Malmgren-Strong</td>
<td></td>
</tr>
<tr>
<td>John Osteraas</td>
<td>CUREE Project Manager</td>
</tr>
<tr>
<td>Daniel Pradel</td>
<td></td>
</tr>
<tr>
<td>Balakrishna Rao</td>
<td></td>
</tr>
<tr>
<td>Bob Reitherman</td>
<td>CUREE Project Advisory Panel</td>
</tr>
<tr>
<td>James Russel</td>
<td></td>
</tr>
<tr>
<td>Paul Somerville</td>
<td>Presenter</td>
</tr>
<tr>
<td>Jon Stewart</td>
<td>CUREE Project Advisory Panel</td>
</tr>
<tr>
<td>Joel Wolf</td>
<td></td>
</tr>
<tr>
<td>Aspasia Zerva</td>
<td>Presenter</td>
</tr>
</tbody>
</table>
Seismic Ground Surface Deformations of Residential Slabs and Foundations

by
Bruce A. Bolt
Professor of Seismology Emeritus
University of California, Berkeley

Damages to residential slabs and at-grade improvements are often found to be caused by earthquake-induced lurching, subsidence and other movement of the sub-soil. When these causes are put aside the question addressed here relates to feasible damage mechanisms from the response of the slabs, etc. to the passage of the seismic waves. There are a few field reports of observers seeing ground undulations. Unequivocal observations of damage to previously undamaged slabs, etc are rare. The engineering problem is a special case of soil-structure interaction. For given site properties, the case of an infinite slab is similar to the often finite-element modeled response of a partly buried pipeline. The case is unusual geophysically because the slab layer has a higher shear wave velocity (6000 ft/sec) than the subsoil (1000 ft/sec). Because a residential foundation has small dimensions slipping on the subsoil and edge effects (zero stress and nonlinearity) are highly significant.

From a seismology viewpoint, the bonded slab-soil system corresponds to an elastic layer of finite extent bonded to a half space. No doubt shear (transverse) waves dominate with particle velocities in the slab that produce the strain. The ratio of wavelength to slab dimension is the decisive factor. In practice, there will be a wide range of impedance conditions between the slab and its foundation. An approximate estimate of concrete strain (suggested by Newmark) is the particle velocity in the slab divided by the apparent shear wave velocity during wave passage. In the cases considered here (with no allowance for slip) the transient strain in the concrete is less than 0.0001. Peak rotational soil motion in the 1979 Imperial Valley Earthquake was probably less than 10-4 rad. The strain for cracking of concrete is calculated to be in excess of 0.00015.

Experience with calculations like those above indicates that input seismic motions would need to exceed 0.6g (or ground velocities 60 cm/sec) for cracking strengths to be approached for slabs etc. with no deterioration, MMI = 8 to 9.
APPENDIX D – PAUL SOMERVILLE WORKSHOP MATERIALS

Workshop on the Effect of Earthquake-Induced Transient Ground Surface Deformations on At-Grade Improvements

Oakland, California, May 28, 2003

NEAR FAULT GROUND MOTIONS AND STRAINS

Paul Somerville, URS Corporation, Pasadena

ABSTRACT

This paper addresses the issue of the magnitude of ground motions and of surface ground strains caused by the passage of seismic waves in firm ground. We first address the translational and rotational characteristics of ground motions, which may affect the inertial interaction between the soil and the foundation. We then address ground strains and differential ground motions, which may affect the kinematic interaction between the soil and the foundation.

1. TRANSLATIONAL GROUND MOTIONS

The characteristics of near fault ground motions are described by Somerville (2002, 2003). Briefly, near-fault ground motions consist of large dynamic pulses of ground motions in the fault normal direction, and large permanent displacements of the ground in the direction of fault movement. In strike-slip earthquakes, these two types of motions occur on orthogonal components (fault normal and fault parallel), while in dip-slip earthquakes, they both occur on the fault-normal component.

Approximate values of the largest recorded near fault ground motions in recent large earthquakes are summarized in Table 1. The large values suggest that inertial effects may be important for the behavior of at-grade foundation slabs. Whether the ground displacement reverses itself, as in the directivity pulse, or does not, as in the fling step, may have some influence on inertial effects.

Table 1. Amplitude and Period of Largest Recorded Translational Near Fault Ground Motions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Amplitude</th>
<th>Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak acceleration</td>
<td>1,000 cm/sec</td>
<td>1 second</td>
</tr>
<tr>
<td>Peak velocity – 2 sided pulse (directivity)</td>
<td>200 cm/sec</td>
<td>2 seconds</td>
</tr>
<tr>
<td>Peak displacement – 1 sided pulse (directivity)</td>
<td>150 cm</td>
<td>4 seconds</td>
</tr>
<tr>
<td>Peak velocity – 1 sided pulse (fling)</td>
<td>500 cm/sec</td>
<td>6 seconds</td>
</tr>
<tr>
<td>Peak displacement – ramp (fling)</td>
<td>1,000 cm</td>
<td>6 seconds</td>
</tr>
</tbody>
</table>
2. ROTATIONAL GROUND MOTIONS

Nearly all seismic instruments record the translational components of ground motion but not the rotational components. Estimates of the rotational ground motions have mostly been derived from dense array measurements of translational measurements (Oliveira and Bolt, 1989), although instruments have recently been developed to record the rotational components (Nigbor, 1994). Near fault measurements of rotational motions of near-fault ground motions by Takeo, et al. (1998), using rotational measurements, and Niazi (1986) and Huang (2003) using translational array measurements show very diverse results, summarized in Table 2. The largest rotational velocities obtained are on the order of one degree per second. The values measured by Niazi (1986) on a rigid foundation slab presumably incorporate the effect of spatial averaging.

Table 2. Measurements of Near Fault Rotational Ground Velocity

<table>
<thead>
<tr>
<th>Earthquake Magnitude</th>
<th>Closest Distance (km)</th>
<th>Siting Conditions</th>
<th>Rotational Velocity (deg/sec)</th>
<th>Predominant Period (sec)</th>
<th>Author</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.2</td>
<td>5</td>
<td>Point measurement</td>
<td>1.5</td>
<td>~0.1</td>
<td>Takeo (1998)</td>
</tr>
<tr>
<td>6.5</td>
<td>5</td>
<td>Array on rigid foundation slab</td>
<td>0.06</td>
<td></td>
<td>Niazi (1986)</td>
</tr>
<tr>
<td>7.6</td>
<td>6</td>
<td>Array on earth dam</td>
<td>0.02</td>
<td>5</td>
<td>Huang (2003)</td>
</tr>
</tbody>
</table>

3. GROUND STRAINS FROM COHERENT SEISMIC WAVES

Ground strains are not widely recorded, but the ground strains caused by coherently propagating seismic waves can be estimated from their amplitudes and horizontal phase velocities. The horizontal phase velocity measures the apparent speed of wavefronts crossing the ground surface. For shear body waves, the horizontal phase velocity is given by the seismic ray parameter \( p = v / \sin(i) \), where \( v \) is the shear wave velocity at the earthquake source and \( i \) is the angle of incident of the wave at the ground surface. Snell’s law states that the ray parameter i.e. the horizontal phase velocity is the same along the entire length of the ray path, even though the incidence angle is changing.

For a point source earthquake occurring in rock with a shear wave velocity of 3 km/sec, the horizontal phase velocity will range from infinity (for waves traveling directly upward, i.e. with and angle \( i \) of zero, such that the waves arrive at the same time at neighboring locations on the ground surface) to 3 km/sec (for waves traveling horizontally from the fault and then bending upward to the surface, creating a wave passage effect). For the case of rupture propagation of a fault toward a site, which typically occurs at about 80% of the shear wave velocity, the horizontal phase velocity may be close to the rupture velocity (Bouchon and Aki, 1982). In any case, the minimum phase velocity for body waves is very high. Seismic surface waves can have lower horizontal phase velocities, which in stiff alluvium are typically observed to be about 1 km/sec, which is still quite high.
For body waves, the strain is given by the ratio of the peak velocity to the horizontal phase velocity. For surface waves, this relation does not hold but we have used it as an approximation for the present purposes. Table 3 estimates the strains for extreme values of ground motions. Even for these extreme ground motion values, the strains are not very large. Ground strains during the Northridge Earthquake should have been well below these levels in most locations, with the highest values from near fault shear waves in the northern San Fernando basin and the Santa Clarita basin, and from basin surface waves just south of the Santa Monica fault in the northwestern Los Angeles basin. The near fault basin edge wave condition applies to the 1995 Kobe Earthquake, not to the 1994 Northridge Earthquake.

Table 3. Approximate Estimates of Ground Strains for Extreme Values of Coherent Ground Motions of Various Type

<table>
<thead>
<tr>
<th>Seismic Wave Type</th>
<th>Peak Velocity</th>
<th>Phase Velocity</th>
<th>Strain x 10^-4</th>
<th>Strain x 30 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near fault shear waves</td>
<td>200 cm/sec</td>
<td>&gt; 3 km/sec</td>
<td>&lt; 6</td>
<td>&lt; 2 cm</td>
</tr>
<tr>
<td>Basin surface waves</td>
<td>60 cm/sec</td>
<td>1 km/sec</td>
<td>~ 6</td>
<td>2 cm</td>
</tr>
<tr>
<td>Near fault basin edge wave</td>
<td>200 cm/sec</td>
<td>1 km/sec</td>
<td>~ 20</td>
<td>6 cm</td>
</tr>
</tbody>
</table>

Observations of Ground Waves during Earthquakes

Observers’ accounts of large earthquakes often include descriptions of visible waves that are seen moving across the ground surface. The reported amplitudes of these waves are typically much larger, their wavelengths are much shorter (on the order of tens of meters), and their phase velocities are much slower than those of coherent seismic waves. The high phase velocities in Table 3 represent wave motion that is probably too fast for the human eye to perceive. There currently exists no seismological explanation of observers’ accounts of visible waves that are seen moving across the ground surface during large earthquakes.

If the observers’ accounts are accurate, then they must pertain to a type of wave that has so far evaded the detection of seismologists using seismic instruments. If such waves were a pervasive feature of earthquake ground motions, it is reasonable to expect that they would have been detected in well-recorded earthquakes such as the 1994 Northridge and 1999 Chi-Chi, Taiwan earthquakes. The existence of gravity waves (like water waves) in soft saturated sediments has been proposed by Lomnitz (1999), but not observed. Waves in such soils are beyond the scope of this paper, whose focus is on waves in firm ground.

4. SPATIAL COHERENCY OF GROUND MOTIONS

Empirical models of the spatial variation of ground motions have been derived from dense array measurements (Abrahamson et al., 1990, Zerva and Zervas, 2002). Recorded seismic waves have a lower degree of spatial coherency than would be predicted based on the coherent
propagation of seismic waves. The reduction in coherency with increasing frequency for points separated by 30 meters based on the model of Abrahamson et al. (1990), is portrayed in Table 4. The total coherency values describe how much of the ground motion at two locations is explained by vertical shear wave propagation; this part of the motion is available to induce inertial interaction effects with the foundation. The remainder represents the degree of non-uniformity in the ground motions at the two locations, and is available to induce torsion and strain in the foundation through kinematic interaction.

Table 4 also lists the contribution to the overall coherency that is due to the inclined incidence of coherent waves, i.e. the wave passage effect. At low frequencies, the wave passage effect has high coherency and contributes little to the overall spatial incoherence of ground motions. However, the wave passage contribution to the incoherence becomes much larger at high frequencies (above 10 Hz).

**Table 4. Spatial Coherency for 30 Meter Separation as a Function of Frequency**

<table>
<thead>
<tr>
<th>Component</th>
<th>1 Hz</th>
<th>5 Hz</th>
<th>10 Hz</th>
<th>20 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave Passage</td>
<td>0.999</td>
<td>0.99</td>
<td>0.925</td>
<td>0.55</td>
</tr>
<tr>
<td>Total Coherency</td>
<td>0.99</td>
<td>0.92</td>
<td>0.75</td>
<td>0.50</td>
</tr>
</tbody>
</table>

5. SUMMARY

This paper addresses the issue of the magnitude of ground motions and of surface ground strains caused by the passage of seismic waves in firm ground. The best understood and most commonly recorded ground motions are translational motions, which in the near fault region may have a significant effect on the inertial interaction between the soil and the foundation. Data on the rotational components of ground motions, which may also affect the inertial interaction between the soil and the foundation, are quite sparse. Estimates of ground strains across a 30 m foundation from coherently propagating seismic waves are quite low. However, recorded ground motions have a high degree of spatial incoherence, causing differential motions that may induce strains and affect the kinematic interaction between the soil and the foundation. There currently exists no seismological explanation of observers’ accounts of visible waves that are seen moving across the surface of firm ground during large earthquakes.

6. REFERENCES


Characterizing Near Fault Ground Motion For The Design And Evaluation Of Bridges

Paul Somerville

ABSTRACT

Near-fault ground motions are different from ordinary ground motions in that they often contain strong coherent dynamic long period pulses and permanent ground displacements. The dynamic motions are dominated by a large long period pulse of motion that occurs on the horizontal component perpendicular to the strike of the fault, caused by rupture directivity effects. Near fault recordings from recent earthquakes indicate that this pulse is a narrow band pulse whose period increases with magnitude, as expected from theory. This magnitude dependence of the pulse period causes the response spectrum to have a peak whose period increases with magnitude, such that the near-fault ground motions from moderate magnitude earthquakes may exceed those of larger earthquakes at intermediate periods (around 1 second). The static ground displacements in near-fault ground motions are caused by the relative movement of the two sides of the fault on which the earthquake occurs. These displacements are discontinuous across a fault having surface rupture, and can subject a bridge crossing a fault to significant differential displacements. The static ground displacements occur at about the same time as the large dynamic motions, indicating that the static and dynamic displacements need to be treated as coincident loads.

At the FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities held in San Francisco on May 29-30, 1997, a consensus was reached that the response spectrum alone is not an adequate representation of near-fault ground motion characteristics, because it does not adequately represent the demand for a high rate of energy absorption presented by near-fault pulses. This is especially true for high ground motion levels that drive structures into the non-linear range, invalidating the linear elastic assumption on which the elastic response spectrum is based. To fully portray the response of structures to near-fault ground motions, nonlinear time history analysis may be required. Fortunately, near fault ground motions containing forward rupture directivity may be simple enough to be represented by simple time domain pulses, thus simplifying the specification of ground motion time histories for use in structural response analyses. Preliminary equations relating the period of the pulse to the earthquake magnitude, and the effective velocity of the pulse to the earthquake magnitude and distance, have been developed. The directivity pulse can be combined with the static fault displacement to provide a complete description on near-fault ground motions. The effect of the simultaneous dynamic and static ground motions on the response of a bridge should be analyzed using time histories that include both types of motion.

The probabilistic approach to seismic hazard analysis has an important advantage over the deterministic approach in that it takes into account the degree of activity of the faults that contribute to the hazard, providing explicit estimates of the likelihood of occurrence (or return period) of the hazard level that is specified in the design ground motions.
INTRODUCTION

An earthquake occurs when elastic strain that has gradually accumulated across a fault is suddenly released in the process of elastic rebound. The elastic energy stored on either side of the fault drives the motion on the fault. The elastic rebound generates dynamic strong ground motions that last for a few seconds to a few minutes, constituting a primary seismic hazard. The elastic rebound also generates static deformation of the ground. The static deformation of the ground consists of a discontinuity in displacement on the fault itself, and a gradual decrease in this displacement away from the fault on either side of the fault. If there is surface faulting, the static displacements are discontinuous across the fault at the ground surface, constituting a primary seismic hazard. Even if the fault does not break the surface, there is static deformation of the ground surface due to subsurface faulting.

Strong ground motions recorded on digital accelerographs in recent earthquakes, including the 1985 Michoacan, Mexico, 1999 Chi-chi, Taiwan and 1999 Kocaeli, Turkey earthquakes, contain both dynamic ground motions and static ground displacements. Figure 1 shows the strong motion recording of the strike-slip Kocaeli Earthquake at Yarimca. The static displacement of the ground is about 2 meters in the east-west direction, parallel to the strike of the fault, consistent with geological and GPS data. The large dynamic ground velocity pulse is oriented north-south, in the fault normal direction. The static ground displacement is coincident in time with the largest dynamic ground velocities, as shown in Figure 1, and occurs over a time interval of several seconds. It is therefore necessary to treat the dynamic and static components of the seismic load as coincident loads.

In some earthquakes, faulting of the ground surface occurs on a distributed system of sub-parallel faults instead of occurring on a single fault trace. This distributed fault system may have a width of several tens to hundreds of meters. This does not have a significant impact on the estimation of dynamic ground motions, but can complicate the estimation of the static ground displacement field, including surface faulting. Ground shaking can cause secondary ground deformation by inducing soil liquefaction with concomitant lateral spreading, and landslides. Excluding these complications and secondary effects, the near fault ground motion hazards that influence an individual bridge support include:

- Dynamic displacements at a bridge support due to seismic waves.
- Permanent displacements at a bridge support due to the static displacement field.

These dynamic and static ground displacements need to be quantified in separate hazard analyses, because they are not strongly correlated, as shown below. Once they have been separately quantified, the two components of the hazard can then be combined into a single ground motion time history, like those shown in Figure 1, that contains both dynamic and static ground displacements at a bridge support.

Bridges have multiple supports and are thus also affected by differential displacements of the ground. These differential displacements may be caused by both dynamic and static ground displacements. The kinds of seismic hazards that cause differential ground displacements of bridge piers in the near fault environment include:
17 Aug 1999, Kocaeli, Mw7.4 - ypt, CD=5.03 km, Site=r

Figure 1. Strong motion recording of the south (180), east (090), and vertical (up) components of the 1999 Kocaeli, Turkey earthquake at Yarimea.
• Dynamic differential displacements between supports due to seismic waves. These include the effects of wave passage, and differences in the amplitude and phase of ground motions at multiple supports due to wave incoherence effects [1].

• Permanent differential displacements between supports due to the static displacement field. These are potentially large when surface faulting occurs between supports. Even without surface displacements between supports, differential displacements may occur due to spatial variations in the static displacement field, which may be significant near the fault.

These dynamic and static differential displacement hazards need to be quantified in separate hazard analyses. However, both components of the hazard can be specified together by suites of ground motion time histories at multiple supports that contain both dynamic and static ground displacements whose differential values between supports are consistent with the estimated differential displacement hazards.

NEAR FAULT RUPTURE DIRECTIVITY PULSE

An earthquake is a shear dislocation that begins at a point on a fault and spreads at a velocity that is almost as large as the shear wave velocity. The propagation of fault rupture toward a site at a velocity close to the shear wave velocity causes most of the seismic energy from the rupture to arrive in a single large pulse of motion that occurs at the beginning of the record [2], [3]. This pulse of motion represents the cumulative effect of almost all of the seismic radiation from the fault. The radiation pattern of the shear dislocation on the fault causes this large pulse of motion to be oriented in the direction perpendicular to the fault plane, causing the strike-normal component of ground motion to be larger than the strike-parallel component at periods longer than about 0.5 seconds. To accurately characterize near fault ground motions, it is therefore necessary to specify separate response spectra and time histories for the strike-normal and strike-parallel components of ground motion.

Forward rupture directivity effects occur when two conditions are met: the rupture front propagates toward the site, and the direction of slip on the fault is aligned with the site. The conditions for generating forward rupture directivity effects are readily met in strike-slip faulting, where the rupture propagates horizontally along strike either unilaterally or bilaterally, and the fault slip direction is oriented horizontally in the direction along the strike of the fault. However, not all near-fault locations experience forward rupture directivity effects in a given event. Backward directivity effects, which occur when the rupture propagates away from the site, give rise to the opposite effect: long duration motions having low amplitudes at long periods.

The conditions required for forward directivity are also met in dip slip faulting. The alignment of both the rupture direction and the slip direction up dip on the fault plane produces rupture directivity effects at sites located around the surface exposure of the fault (or its up dip projection if it does not break the surface). Unlike the case for strike-slip faulting, where forward rupture directivity effects occur at all locations along the fault away from the
hypocenter, dip slip faulting produces directivity effects on the ground surface that are most concentrated in a limited region up dip from the hypocenter.

**ORIENTATION OF DYNAMIC AND STATIC NEAR FAULT GROUND MOTIONS**

The top part of Figure 2 schematically illustrates the orientations of dynamic and static near fault ground motions. The strike-slip case is shown in map view, where the fault defines the strike direction. The rupture directivity pulse is oriented in the strike-normal direction and the static ground displacement ("fling step") is oriented parallel to the fault strike. The dip-slip case is shown in vertical cross section, where the fault defines the dip direction; the strike direction is orthogonal to the page. The rupture directivity pulse is oriented in the direction normal to the fault dip, and has components in both the vertical direction and the horizontal strike normal directions. The static ground displacement is oriented in the direction parallel to the fault dip, and has components in both the vertical direction and the horizontal strike normal direction.

The bottom part of Figure 2 schematically illustrates the partition of near fault ground motions into the dynamic ground motion, which is dominated by the rupture directivity pulse, and the static ground displacement. For a strike-slip earthquake, the rupture directivity pulse is partitioned mainly on the strike-normal component, and the static ground displacement is partitioned on the strike-parallel component. If the static ground displacement is removed from the strike-parallel component, very little dynamic motion remains. For a dip-slip earthquake, the dynamic and static displacements occur together on the strike-normal component, and there is little of either motion on the strike-parallel component. If the static ground displacement is removed from the strike-normal component, a large directivity pulse remains.

**PRESERVING ORIENTATION IN THE ARCHIVING, ANALYSIS, AND APPLICATION OF NEAR FAULT GROUND MOTIONS**

Figures 1 and 2 demonstrate that near-fault ground velocities and displacements have orientations that are controlled by the geometry of the fault, specifically by the strike, dip, and rake angle (direction of slip) on the fault. Consequently, it is necessary to treat them as vector, rather than scalar, quantities. The simplest method or treating them as vector quantities is to partition them into strike-normal and strike-parallel components. The dynamic and static motions are distinctly different on these two components at all near-fault locations.

Accordingly, near-fault ground motion recordings should be archived in the strike-normal and strike-parallel components, as shown on the left side of Figure 3. The rotation of the two-recorded components North (N) and East (E) into strike-parallel and strike-normal components SP and SN is accomplished using the following transformations:

\[
SP = N \cos \phi + E \sin \phi; \quad SN = -N \sin \phi + E \cos \phi
\]
where \( \phi \) is the strike of the fault measured clockwise from North. If the recording orientation is not North and East but rotated clockwise by the angle \( \psi \), then \( \phi \) would be reduced by \( \psi \).

Distinct models for the strike-normal and strike-parallel components of near-fault ground motions, derived from appropriately archived recordings and from simulations based on seismological models, are needed for seismic hazard analysis. In order to represent near-fault effects, ground motion simulations need to be based on the summation of complete Green’s functions that contain near-, intermediate-, and far-field terms. This is done using the elastodynamic representation theorem, which states that the ground motion \( U(t) \) can be calculated from the convolution of the slip time function \( D(t) \) on the fault with the Green's function \( G(t) \) for the appropriate distance and depth, integrated over the fault rupture surface [4]:

\[
U(t) = \sum D(t) * G(t)
\]

When a near-fault ground motion time history is used for the analysis of a structure at a site, the strike-normal and strike-parallel components need to be oriented with respect to the strike of the fault that dominates the seismic hazard at the site. The strike-normal and strike-parallel components may be transformed into longitudinal and transverse components, preserving the orientation of the motions with respect to the fault strike, as illustrated on the right side of Figure 3. If the axis of the structure is aligned at some angle \( \theta \) to the strike of the fault, then the longitudinal and transverse time histories can be derived from the strike-normal (SN) and strike-parallel (SP) time histories using the following transformation:

\[
\text{long} = \text{SP} \cos \theta + \text{SN} \sin \theta; \quad \text{trans} = \text{SP} \sin \theta - \text{SN} \cos \theta
\]
Figure 2. Top: Schematic orientation of the rupture directivity pulse and fault displacement ("fling step") for strike-slip (left) and dip-slip (right) faulting. Bottom: Schematic partition of the rupture directivity pulse and fault displacement between the strike normal and strike parallel components of ground displacement. Waveforms containing static ground displacement are shown as dashed lines; versions of these waveforms with the static displacement removed are shown as dotted lines.
Figure 3. Archiving (left) and application (right) of strike-normal and strike-parallel components of ground motion.

Figure 4. Fault-normal velocity pulses recorded near three moderate magnitude earthquakes (left column) and three large magnitude earthquakes (right column), shown on the same scales.
NEAR FAULT GROUND MOTION MODELS

The relationships between the dynamic and static components of near-fault ground displacements are quite complex, as illustrated in Figure 2. For example, the rupture directivity pulse and the static ground displacement occur on orthogonal components in strike-slip faulting, but on the same component in dip-slip faulting. The rupture directivity pulse can be very strong off the end of a strike-slip fault, where there is little or no static displacement. The 1989 Loma Prieta and 1994 Northridge earthquakes produced strong rupture directivity pulses even though they did not rupture the ground surface. This indicates that separate models are needed for predicting the dynamic and static components of near-fault ground displacements at a site. The separately estimated dynamic and static components of the ground motion can be combined to produce ground motion time histories representing both effects. In the following, we present models for predicting dynamic near fault ground motions. The static displacement field of earthquakes can be calculated using theoretical methods [4], and surface fault displacements can be estimated using empirical models [5].

Broadband Directivity Model

Somerville et al. [3] developed a model for near-fault ground motions that assumes monotonically increasing spectral amplitude at all periods with increasing magnitude. This model can be used to modify conventional ground motion attenuation relations to account for the amplitude and duration effects of rupture directivity. Abrahamson [6] demonstrated that incorporation of a modified version of this model in a probabilistic seismic hazard calculation results in an increase of about 30% in the spectral acceleration at a period of 3 seconds for an annual probability of 1/1,500 at a site near a large active fault.

Narrow Band Directivity Model

Strong motion recordings of the recent large earthquakes in Turkey and Taiwan indicate that the near fault pulse is a narrow band pulse whose period increases with magnitude. In Figure 4, forward rupture directivity pulses of earthquakes in the magnitude range of 6.7 to 7 are compared with pulses from earthquakes in the magnitude range of 7.2 to 7.6. The narrow band nature of these pulses causes their elastic response spectra to have peaks, as shown in Figure 5. The fault normal components (which contain the directivity pulse) are shown as solid lines, and the fault parallel components, which are much smaller at long periods as expected, are shown by long dashed lines. The 1994 UBC spectrum for soil site conditions is used as a reference model for comparison. The spectra for the large earthquakes (right column) are compatible with the UBC code spectrum in the intermediate period range, between 0.5 and 2.5 seconds, but have a peak at a period of about 4 seconds where they significantly exceed the UBC code spectrum. The spectra of the smaller earthquakes (left column) are very different from those of the larger earthquakes. Their spectra are much larger than the UBC code spectrum in the intermediate period range of 0.5 - 2.5 sec, but are similar to the UBC spectrum at longer periods.

The recent large earthquakes in Turkey and Taiwan, which caused large surface ruptures, have surprisingly weak ground motions at short and intermediate periods. These new observations are consistent with our finding from previous earthquakes that the strong ground motions of
earthquakes that produce surface faulting are weaker than the ground motions of events whose rupture is confined to the subsurface. All of the earthquakes in the magnitude range of 6.7 – 7.0 shown in Figures 4 and 5 are characterized by subsurface faulting, while all of the earthquakes in the magnitude range of 7.2 to 7.6 are characterized by large surface displacements. Consequently, some of the differences seen in these figures may be attributable not only to magnitude effects, but to the effects of buried faulting [7].

The magnitude scaling exhibited in the data in Figures 4 and 5 is contrary to all current models of earthquake source spectral scaling and ground motion spectral scaling with magnitude, which assume that spectral amplitudes increase monotonically at all periods. However, these magnitude scaling features are the natural consequence of the narrow band character of the forward rupture directivity pulse. The period of the near fault pulse is related to source parameters such as the rise time (duration of slip at a point on the fault) and the fault dimensions, which generally increase with magnitude.

Preliminary response spectral models that include the magnitude dependence of the period of the rupture directivity pulse are shown in Figure 6. These models are derived from empirical relations between pulse period and magnitude [7]. Figure 6 compares the response spectra for rock and soil predicted by this model with the standard model of Abrahamson and Silva [8], which does not explicitly include directivity effects, and the broadband model of Somerville et al. [3], whose directivity effects are based on the monotonic increase of ground motion amplitudes with magnitude at all response spectral periods. The narrowband model produces larger response spectra in the period range of about 0.5 to 2 seconds for earthquakes smaller than $M_w 7.5$, and smaller response spectra at all periods for earthquakes larger than $M_w 7.5$, compared with the broadband model.

**TIME DOMAIN MODELS OF NEAR FAULT GROUND MOTIONS**

The response spectrum models shown in Figure 6 do not adequately represent the demand for a high rate of energy absorption presented by near-fault pulses. Near fault ground motions containing forward rupture directivity may be simple enough to be represented by simple time domain pulses, thus simplifying the specification of ground motion time histories for use in structural response analyses. Preliminary equations relating the period of the pulse to the earthquake magnitude, and the effective velocity of the pulse to the earthquake magnitude and distance, have been developed by Somerville [7], Somerville et al. [9], Alavi and Krawinkler [10], and Rodriguez-Marek [11]. The effect of the simultaneous dynamic and static ground motions on the response of a structure can be analyzed using time histories that include both the dynamic rupture directivity pulse and the static ground displacement.
Figure 5. Spectral velocity of fault-normal pulses of moderate (left) and large (right) earthquakes.
Figure 6. Near fault response spectral model, strike-slip, 5km for rock sites (left) and soil sites (right). Top: model without directivity (Abrahamson and Silva, 1997). Middle: Broadband directivity model (Somerville et al., 1997). Bottom: Narrow band directivity model (Somerville, 2001).
CONCLUSIONS

Near-fault ground motions differ from ordinary ground motions in that they often contain strong coherent dynamic long period pulses and permanent ground displacements. The dynamic motions are dominated by a large long period pulse of motion that occurs on the horizontal component perpendicular to the strike of the fault, caused by rupture directivity effects. Near fault recordings from recent earthquakes indicate that this pulse is a narrow band pulse whose period increases with magnitude, as expected from theory. This magnitude dependence of the pulse period causes the response spectrum to have a peak whose period increases with magnitude, such that the near-fault ground motions from moderate magnitude earthquakes may exceed those of larger earthquakes at intermediate periods (around 1 second). The static ground displacements in near-fault ground motions, caused by the relative movement of the two sides of the fault on which the earthquake occurs, are discontinuous across a fault having surface rupture, and can subject a bridge crossing a fault to significant differential displacements. The static ground displacements occur at about the same time as the large dynamic motions, indicating that the static and dynamic displacements need to be treated as coincident loads.

REFERENCES


Model for Strains from Transient Ground Motions

Norm Abrahamson
Pacific Gas & Electric Co
Causes of Strains

- Wave passage effect
- Spatial Incoherency
- Variation of site amplification
Role of Surface Waves

- Dense strong motion array recordings
  - S-coda (e.g. surface waves) have horizontal wave speeds of about 30% to 100% of the direct S waves (average about 50%)
  - Very slow surface waves are not seen

- Surface waves do not appear to be significant for strains
Wave-Passage Effect

- S-waves horizontal speeds
  - $V_{app} = 2.0$ to $4.0$ km/s

- Strain
  - Strain = PGV / $V_{app}$
  - Strain/PGD = (PGV/PGD) / $V_{app}$
  - Strain/PGD(cm) = $\exp(5.8-0.69M)/V_{app}$
Incoherence Effect

- Empirical incoherency functions
  - Based on strong motion array data
- Forward modeling of spatially incoherent ground motions
  - Strain / PGD(cm) = 3.0 x 10^{-5} /cm
Site Amplification Effect

- Empirical model of variability of ground motion over short distances
  - Currently have model of $Sa$ variability
  - Need PGD variability
  - For now, use $Sa(F=0.7 \text{ Hz})$ variability as estimate of PGD variability
Variability of Site Amplification

Separation Distance (m)

Standard Deviation (LN units)

F=0.7 Hz
### Preliminary Model for Strain on Stiff Soil Sites

<table>
<thead>
<tr>
<th>Effect</th>
<th>Strain/PGD (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave Passage</td>
<td>1.5 - 2.5 x 10^{-5} /cm (M6 - M7)</td>
</tr>
<tr>
<td>Incoherence</td>
<td>3 x 10^{-5} /cm</td>
</tr>
<tr>
<td>Site Amplification</td>
<td>3 x 10^{-5} /cm</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>8 x 10^{-5} /cm</td>
</tr>
</tbody>
</table>
Analysis of Wave Speeds from Strong Motion Array Data

Recordings of strong motion and weak motion from 2-dimensional arrays are used to estimate the apparent velocity of the waves during the strong shaking on the horizontal component.

In this analysis, recordings from 5 dense arrays are used: SMART 1, EPRI/Taipower LSST, EPRI Parkfield, Chiba, and Garni. The array configurations are shown in Figures 1a-1e. The greater the number of stations, the better the array can resolve the wave speed.

The events used in this study are listed in Table 1. These events were selected because they had good signal to noise ratio and were readily available. The horizontal wavespeed is estimated using frequency-wavenumber spectral analysis (conventional method). An example frequency-wavenumber spectrum is shown in Figure 2. The wave speed is estimated from the slowness (inverse of velocity) at which the frequency-wavenumber spectrum reaches its maximum.

The results are summarized in Figure 3a and 3b which show the average velocity for each array and the frequency band for which the average applies. The average wave speeds range from 2.1 to 4.2 km/s with an average of 2.7 km/s. The two arrays on rock (EPRI Parkfield and Garni) show lower wave speeds than the three arrays on soil; however, estimates of the uncertainties of the estimated waves speeds have not been computed as yet, so this difference may not be significant.

Form this data set, a reasonable estimate of the wave speed for the strong shaking is 2.5 km/sec. A reasonable lower bound wave speed for strong shaking is 2.0 km/sec.

Wave Speeds and Peak Velocities in the Coda

Using the Chiba array data, the wave speeds and peak velocities were estimated for three consecutive 5 second time windows. The first time window included the strong shaking and the subsequent time windows were in the S coda. In each time window, the wave speed (slowness) and peak velocity were estimated. The peak strain is often assumed to be proportional to the product of the slowness and the peak velocity (or ratio of the peak velocity to the wave speed).

The wave speeds in the later time windows are often less than for the strongest shaking, but the peak velocity is also reduced (Figure 4a,b,c). The ratio of the slowness * PGV for the later windows to window 1 is shown in Figure 5. The slowness*PGV for the second time window is similar to that
for the first time window. For the third time window, it is less than for the first time window.

For this data set, there is not a significant systematic increase in the product of slowness*PGV for the later time windows compared to the strong shaking time window.
Table 1. Events Analyzed

<table>
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<tr>
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<th>Event</th>
<th>Date</th>
<th>Mag</th>
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<td>5</td>
<td>1/29/81</td>
<td>5.8</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>39</td>
<td>1/16/86</td>
<td>6.0</td>
<td>22</td>
</tr>
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<td></td>
<td>40</td>
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<td>67</td>
</tr>
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<td></td>
<td>45</td>
<td>11/14/86</td>
<td>7.8</td>
<td>79</td>
</tr>
<tr>
<td>EPRI/Taipower LSST (soil)</td>
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<td>4.6</td>
<td>29</td>
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<tr>
<td></td>
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<td>81</td>
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<td></td>
<td>2</td>
<td>5/25/89</td>
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<td></td>
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<td>1/12/91</td>
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<td>94</td>
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</table>
Figure 15. Array configuration of the EPRI/Taipower LSST array.
Figure 1c. Array configuration of the EPRI Parkfield array.
Figure 3. Array configuration of the Chiba array.
Figure 16. Oblique view of sensor locations with respect to the Garri Observatory Tunnel.
Dashed lines indicate vertical, parallel to, and perpendicular to the tunnel axis.
The tunnel door, which is the origin of the coordinate system used in Table 2-1,
is at the small dot indicated by the arrow.

From Claussen and Eberhardt (1992)
Average Velocity Estimates

Frequency (Hz)

Velocity (km/sec)

- SMART1
- Chiba
- LSST
- Average
- EPRI
- Garni
Surface, 10 m, and 20 m depths

Ratio of slowness*PGV to Window 1

Time Window

Legend:
- □ 8519
- ○ 8722
- △ 8806
- ⦿ 8816
Depth = 10 m

![Graphs showing slowness, peak velocity, and PGV vs. time window for different sites.](image)
Depth = 20 m

Slowness (sec/km)

Peak Velocity (cm/s)

PGV * Slowness x 10^5

Time Window
Appendix F
Spatial Coherency Functions

Introduction
Scattering and complex wave propagation can lead to significant variations in the ground motion along the length of large structures even if the site conditions are similar (e.g., rock). This variability typically includes variability in both the Fourier phases and Fourier amplitudes; however, since each ground motion is modified to be spectrum compatible, the variation in the Fourier amplitude is not considered. The variation in the Fourier phase affects the shape of the time history. This variation can be described by the spatial coherency which is a statistical model of the ground motion variability.

Coherency
The coherency of ground motion is parameterized by a coherency function. The coherency is a complex number given by

\[ \gamma_{ij}(\omega) = \frac{S_{ij}(\omega)}{S_{ii}(\omega)S_{jj}(\omega)} \]

where

\[ S_{ij}(\omega) = \sum_{m=M}^{M} a_m u_i(\omega_m) \bar{u}_j(\omega_m), \]
The absolute value of the complex coherency is called the “lagged coherency” because it is equivalent to lining up (lagging) the ground motion at the two locations so that the wave passage effect is removed.

\[ |h_{ij}(\omega)| = \left| \frac{S_{ij}(\omega)}{S_{ii}(\omega)S_{jj}(\omega)} \right| \]

The lagged coherency does not restrict the alignment of the ground motions to be consistent from frequency to frequency; that is, the apparent wave speeds can be different at each frequency. At high frequencies, the wave speed implied by the complex coherency becomes more random. However, in the application of the wave passage (see section 4), a constant wave speed is used for all frequencies. To be consistent with this application, the coherency function needs to correspond to the coherency that would be computed by aligning the ground motion to a single wave speed. This coherency is called the “plane-wave coherency”:

\[ |h_{pw}(\omega,\xi_j,\xi_p)| = |h(\omega,\xi_j)\ h(\omega,\xi_p)| \]

where

\[ \frac{\text{Re} \ \gamma_{jk}(\omega,\xi_j)}{|h_{jk}(\omega,\xi_j)|} = h(\omega,\xi_j) \cos (2\pi \xi_j z) \]

Coherency Model

Array data from the dense arrays listed in Tables F-1 were used to develop the spatial coherency model. A previous evaluation (Abrahamson, 1992) has shown that the coherency is does not depend strongly on earthquake
magnitude, distance, or site condition. Therefore, all of the array data was used to develop a single generic coherency model that is applicable to the Bay Bridge site. A summary of the data set from each array is given in Table F-2.

The empirical modeling is developed using tanh⁻¹ of coherency during the curve fitting. The final model is presented in terms of the linear coherency for ease of application. This final model is listed in Table F-3 and is plotted in Figures F-1 and F-2.

References
<table>
<thead>
<tr>
<th>Array</th>
<th>Location</th>
<th>Site Class</th>
<th>Surface Stations</th>
<th>Spacing (m)</th>
<th>Spacing (m)</th>
<th>Spacing (m)</th>
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<td>Taiwan</td>
<td>Soil</td>
<td>15</td>
<td>3</td>
<td>85</td>
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<td>Rock</td>
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<td>Japan</td>
<td>Soil</td>
<td>15</td>
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<td>25</td>
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# Dense Array Data Sets

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<tr>
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<td>3.9</td>
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<td>15</td>
<td>0.04g</td>
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<td>0.21g</td>
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<td></td>
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<td>60</td>
<td>0.06g</td>
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</tbody>
</table>
Table F-3. Empirical Functions for Plane-Wave Coherency

\[
\gamma_{pw}(f, \xi) = \left[ 1 + \left( \frac{f}{a_1 f_0(\xi)} \right)^{n_1} \right]^{-1/2} \left[ 1 + \left( \frac{f}{a_2 f_0(\xi)} \right)^{n_2(\xi)} \right]^{-1/2}
\]

where

<table>
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<tr>
<th>Coeff</th>
<th>Horizontal</th>
<th>Vertical</th>
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<tr>
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</tr>
<tr>
<td>a2</td>
<td>1.01</td>
<td>1.0</td>
</tr>
<tr>
<td>n1</td>
<td>7.02</td>
<td>4.95</td>
</tr>
<tr>
<td>n2(\xi)</td>
<td>5.1 - 0.51 \ln(\xi+10)</td>
<td>1.685</td>
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where \( f \) is frequency in Hz and \( \xi \) is the separation distance in m.

For the horizontal component:

\[
f_0(\xi) = \left[ 1.885 - 2.2209 \ln \left( \frac{4000}{\xi} + 1.5 \right) \right]
\]

For the vertical component:

\[
f_0(\xi) = \exp \left( 2.43 - 0.025 \ln (\xi+1) - 0.048 \left[ \ln (\xi+1) \right]^2 \right)
\]
Figure 3-14a. Plane wave coherency model for the horizontal component.
Figure 3-14b. Plane wave coherency model for the vertical component.
VARIATION OF GROUND MOTIONS ACROSS INDIVIDUAL SITES

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Consultant
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with

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ABSTRACT

The spatial variation of site response over short distances is characterized by the variation of response spectral values from dense array recordings. Strong and weak motion recordings from nine dense arrays in California, Taiwan, and Japan with station separations less than 100 m are analyzed. The standard error of the natural logarithm of response spectral values for station separations less than 100 m at soil sites is between 0.1 and 0.2 (eg. 10-20%). The variability for rock sites is generally larger than or equal to the variability for soil sites, indicating that it is not appropriate to simply combine the variability of rock motions with the variability of soil site response calculations to estimate the total variability of ground motion at soil sites. The variability of the site response can be used by geotechnical engineers to guide 1-D characterization of site response.

INTRODUCTION

Site response can vary even over short distances (<100 m). The planning and/or evaluation of a geotechnical site characterization should consider the inherent variability of the site response over the dimensions of a large structure or group of structures. In this paper, the variation of site response over short distances is quantified using empirical recordings of seismic ground motion at dense arrays. In a previous study, Schneider et al. [1] used dense array recordings to analyze the variation of Fourier amplitude spectra over short distances. Here, the variation of the response spectra is analyzed for the same data set.

ESTIMATION OF GROUND RESPONSE VARIABILITY

The variability of site response is estimated as follows. Let $S_{ijk}(f)$ be the average horizontal component acceleration response spectrum for the $i$th station and the $j$th earthquake. Let $\Delta S_{ijk}(f)$ be the difference between the log spectral values of the $i$th and $k$th stations from the $j$th event. That is

$$\Delta S_{ijk}(f) = \log S_{ijk}(f) - \log S_{ikj}(f)$$

Let $\sigma(f,\xi)$ be the standard deviation of $\Delta S_{ijk}(f)$ where $\xi$ is the separation distance between stations $j$ and $k$. Assume that $\sigma(f,\xi)$ is independent of the event (but may be magnitude dependent) so that $\Delta S_{ijk}(f)$ from different events can be analyzed together. Then $\sigma$ represents the standard deviation of the difference in site response between two points separated by distance $\xi$ and it is used to quantify the variability in ground response.

DATA SET

The largest set of dense-array strong-motion recordings are from the SMART-1 and L3ST arrays in

Fourth DOE Natural Phenomena Hazards Mitigation Conference - 1993
Taiwan, however, there are several other dense arrays in California and Japan that have recorded strong motions. In addition, there are several dense arrays that have recorded weak motions.

We considered only arrays with minimum station separations of less than 100 m and obtained recordings from nine dense arrays for use in the analysis. The arrays and their general characteristics are listed in Table 1. For this study, the arrays have been grouped only by the general site classes of soil and rock, with five arrays on rock and four on soil. The data sets for each array are summarized in Table 2. Six of the arrays have recorded strong motion and three have recorded only weak motion.

### Table 1. Dense Array Characteristics.

<table>
<thead>
<tr>
<th>Array</th>
<th>Location</th>
<th>Site Class</th>
<th>Surface Stations</th>
<th>Spacing (m) Min</th>
<th>Spacing (m) Max</th>
<th>Reference</th>
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<td>15</td>
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<td>Abrahamson et al. [2]</td>
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<td>10</td>
<td>191</td>
<td>EPRI [3]</td>
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### Table 2. Dense Array Data Sets.

<table>
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<td>0.04g</td>
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<td>4.7</td>
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MODEL

As the separation distance, \( x \), goes to zero, \( c \) goes to zero by definition. Based on the previous study of variation of Fourier amplitudes by Schneider et al. (1992) and a preliminary analysis of the site response variability, the site response variation as a function of frequency, separation distance, and magnitude range is modeled by

\[
c(f, x) = c_1(f, M) \left( 1 - \exp \left( -\frac{x}{c_2(f)} \right) \right)
\]

where \( c_1(f) \) and \( c_2(f) \) are constants for each frequency and magnitude range. Using the dense array data, the
### Table 3: Regression Results for Parameter \( c_1 \)

| Frequency (Hz) | Soil (M4.1-4.6) | Soil (M5.0-5.6) | Soil (M6.0-7.8) | Rock (M3.0-4.1) | Rock (M4.1-4.7) | Rock (M5.2) |
|---------------|-----------------|-----------------|-----------------|-----------------|-----------------|**********|
| 0.7           | 0.085           | 0.087           |                 |                 | 0.15            |             |
| 1.5           | 0.12            | 0.10            | 0.095           |                 | 0.21            | 0.16        |
| 2.5           | 0.16            | 0.13            | 0.12            | 0.18            | 0.21            | 0.16        |
| 3.5           | 0.30            | 0.19            | 0.18            | 0.46            | 0.28            | 0.26        |
| 5.0           | 0.33            | 0.20            | 0.19            | 0.48            | 0.30            | 0.38        |
| 7.0           | 0.37            | 0.20            | 0.20            | 0.50            | 0.32            | 0.26        |
| 9.0           | 0.30            | 0.20            | 0.18            | 0.58            | 0.33            | 0.16        |
| 11.0          | 0.30            | 0.20            | 0.15            | 0.65            | 0.40            | 0.14        |
| 15.0          | 0.30            | 0.20            | 0.13            | 0.74            | 0.49            | 0.14        |
| 20.0          | 0.30            | 0.20            | 0.13            | 0.80            | 0.54            | 0.14        |
| 25.0          | 0.30            | 0.17            | 0.13            | 0.76            | 0.53            | 0.14        |

The constants \( c_1 \) and \( c_2 \) are estimated by regression using a maximum likelihood approach.

The resulting values of \( c_1(f,M) \) are listed in Table 3 and the resulting model for \( c_2(f) \) is

\[
\begin{align*}
    c_2(f) &= 0.2 f / 3.5 & \text{for } f \leq 3.5 \text{ Hz}, \\
    c_2(f) &= 0.2 & \text{for } f > 3.5 \text{ Hz}.
\end{align*}
\]

The site response variation model is plotted as a function of frequency in Figure 1 and 2 for separation distances of 10m and 100m, respectively and as a function of separation distance in Figure 3. The variation is strongly dependent on earthquake magnitude with larger magnitude events having less variation. The magnitude dependence is strongest at the higher frequencies. This result is consistent with recent results of analyses of large empirical strong motion data bases for peak acceleration [11]. The variability of site response for soil sites is between 10% and 20% for moderate to large magnitude (M>5) events.

**COMPARISON OF SOIL SITE AND ROCK SITE GROUND RESPONSE VARIABILITY**

The majority of the dense array data used in this study is from soil site arrays, however, there is some data from rock sites. The variability of ground response on rock and soil sites shown by the solid and open symbols, respectively, in Figure 2. This figure shows that the variability of ground response at rock sites is larger than or equal to the variability at soil sites for most frequencies and magnitudes. The small magnitude (M4.1-4.7) rock site variability is larger than the small magnitude (M4.1-4.6) soil site variability at low frequencies and high frequencies, whereas, they are similar at moderate frequencies (3 to 7 Hz). The large magnitude rock site curve is for only a single event from the Coalinga array so it is not as robust as the other curves, but it also shows larger variability than the M5 soil site variability in the frequency range of 1 to 7 Hz. One possible source for difference in variation on soil and rock is that a slight shift in resonance across a site can easily generate large variations in amplitude at a given frequency. In this regard, small changes in layer thickness would produce more predominant shifts in resonance for shallow layers; thus shallow soil sites and rock sites with complex geology would tend to experience the largest amplitude variations.

**CONCLUSIONS**

The difference in soil site and rock site ground response variability has important practical consequences. In site response studies, the variability of the rock ground motion at the base of the soil column is often assumed to be the same as the variability at a rock outcrop. If the total variability of ground motion at soil sites is computed by simply adding the variance of the soil amplification to the variance of the rock response spectrum, then the variability of response spectra at soil sites would be larger than the variability of the response spectra at rock sites. The dense array data, however,
Figure 2. Model of ground response variability as a function of frequency for a separation distance of 100 m. The open symbols are for soil sites and the solid symbols are for rock sites.
Figure 3. Model of ground response variability as a function of separation distance for large magnitude events at soil sites.
suggest that the opposite is true. This suggests that either the soil has a homogenizing effect on the ground motion or that the bedrock motion is less variable than the outcrop motion.

These results also have implications for empirical site response studies. If small magnitude events recorded at the site are used to characterize the site response, then such empirical site response estimates will have large variability across the site. Therefore, several recording stations should be used to get a stable estimate of the site response.

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Abstract: This manuscript presents an overview of the literature on transient seismic ground strains: their empirical estimation as the ratio of particle velocity over the apparent propagation velocity of the motions; their analytical evaluation from numerical codes; and techniques for the evaluation of seismic strains from data recorded at a single station or at an array of stations. Advantages and limitations of these strain estimation techniques are discussed. The significance of the spatial variation of seismic ground motions is highlighted; spatial variation of seismic ground motions includes the effects of their propagation on the ground surface as well as the amplitude and phase variation in the seismic data recorded over extended areas. For illustration purposes, the causes for the spatial variability of the motions are briefly described and coherency models, analytical and empirical, for shorter and longer separation distances between stations, presented. Simulations of seismic ground strains based on spatial variability models are generated and the relative contribution of coherency vs. apparent propagation of the motions in the strain estimates illustrated. The response of a rigid foundation subjected to seismic ground motions that exhibit loss of coherency as they propagate on the ground surface is described.
I. Introduction

Post Northridge earthquake damage evaluation revealed cracking of pavement and foundation at sites with stable soil. The question that was posed for the “Workshop on the Effect of Earthquake-Induced Transient Ground Surface Deformations on At-Grade Improvements” in Oakland, CA, on May 28, 2003, was “Under what circumstances, if any, should transient ground surface deformations be considered as a potential cause of damage to improvements?” More specifically, the questions that needed to be considered were:

- Can the magnitude of earthquake-induced transient ground surface strains at an arbitrary site be reasonably estimated given the current state-of-science?
- If so, what is necessary to develop an efficient methodology to relate common measures of ground motion to the magnitude of transient ground surface strains? If not, what research is needed to develop such a capability?
- What is the nature of earthquake-induced transient ground surface strains experienced at a given site with stable soil (i.e. no earthquake-induced ground failure)?
- What is the nature of demands (force and deformation) due to earthquake-induced transient ground surface strains on a concrete plate on the ground surface that is the size of a typical residential slab (less than 100 ft/30 m in any dimension)?
- Is it possible, given the current state-of-science, to identify any correlation between intensity of ground shaking (MMI or instrumental intensity) and potential for damaging earthquake-induced ground surface deformations?

To address some of the above questions, this manuscript presents an overview of the current state-of-science on the following topics:

- **Empirical maximum strain evaluation** – Seismic ground strains are evaluated from estimates of the maximum particle velocity divided by a measure of the apparent propagation velocity of the motions for either body or surface waves.

- **Analytical seismic ground strain evaluation** – Analytically evaluated longitudinal strains and tilts in the vicinity of a thrust fault are presented. Additionally, strain attenuation relationships, based on numerical codes, as functions of distance and magnitude are highlighted.

- **Seismic strain estimates from recorded data** – The commonly used single station estimate technique and its application to the Northridge earthquake is described. The accuracy of the single station estimate approach in comparison to the seismogeodetic one is illustrated.
• **Spatial variability of seismic ground motions** – The causes for the spatial variability of seismic ground motions, namely apparent wave propagation velocity, phase variability (lagged coherency) and amplitude variability are discussed. Some commonly used coherency models are presented. Frequency-wavenumber techniques for the identification of the apparent propagation velocity of the motions are also briefly described.

• **Foundation response to seismic ground motions** – A pioneering approach for the evaluation of the response of a rigid foundation subjected to ground motions propagating on the ground surface and experiencing loss of coherency is described.

• **Seismic ground strain simulations** – The stochastic simulation of seismic ground strain time histories and their comparison with the strain estimates, obtained from the ratio of the particle velocity over the apparent propagation velocity of the motions, is presented.

• **Spatial coherency at shorter and longer separation distances** – The differences in the behavior of coherency at shorter and longer separation distances is briefly illustrated.

• **Local coherency** – An analytical model for the evaluation of the coherency at a specific site, for which the subsurface characteristics are known, is presented. Its advantages and limitations are discussed.

• **Coherent propagating motions vs. actual data** – The comparison of coherent motions, which conform with the assumption that strain can be estimated from the ratio of particle velocity over the apparent propagation velocity of the motions, with actual data is highlighted.

### II. Empirical Maximum Strain Evaluation

Recent evaluations of seismic ground strains still rely on observations made in the late 60’s – early 70’s. Newmark and Roesenblueth (1971) noted that buried pipelines follow the motion of the ground, and that maximum axial strains induced in buried pipelines can be adequately approximated by the maximum ground strains, which are given by:

$$\varepsilon_{\text{max}} = \frac{(v_L)_{\text{max}}}{C}$$

where \((v_L)_{\text{max}}\) is the maximum horizontal velocity in the longitudinal direction of the pipeline and \(C\) is the component of the apparent velocity of the waves with respect to the ground surface in the same direction. Equation 1 is based on the modeling of the seismic excitation by a single plane wave traveling with velocity \(C\) on the ground surface. The torsional response of buildings can also be approximated by a similar expression (Newmark and Roe-
senblueth, 1971), in which the maximum angle of rotation of the ground, $\alpha_{\text{max}}$, is approximated by:

$$\alpha_{\text{max}} = \frac{(v_T)_{\text{max}}}{C}$$

(2)

where $(v_T)_{\text{max}}$ is now the maximum horizontal velocity in the transverse direction.

Whereas the particle velocity may be available from recorded data and/or attenuation relations, an appropriate estimate of the apparent propagation velocity is critical for the estimation of seismic strains and rotations. O’Rourke and his co-workers presented approaches for the identification of appropriate values for $C$ evaluated from body and surface waves.

(a) Evaluation of seismic strains from body waves

For the identification of the apparent propagation velocity of body waves on the ground surface, O’Rourke et al. (1982) first evaluated the principal directions of the ground motions using a moving time window intensity tensor, $G(t)$, with elements defined as:

$$g_{ij}(t) = \int_{t-0.5\delta}^{t+0.5\delta} a_i(\tau)a_j(\tau) d\tau$$

(3)

where $a_i(t)$, $i=1, 2, 3$, are ground accelerations in three mutually perpendicular directions, and $\delta$ is the width of the moving window. The eigenvalues of $G(t)$ correspond to the principal values of the moving time window intensity, and the components of the ground motions along the three principal directions are statistically independent of each other. The eigenvector corresponding to the largest eigenvalue indicates the predominant direction of the motions in the time window considered. Figure 1 illustrates the collateral angle $\phi(t)$ and the longitudinal angle $\theta(t)$ associated with the major principal eigenvector of $G(t)$ at a specific time $t$.

![Figure 1. Illustration of predominant direction of ground motions (from O’Rourke et al., 1982; Copyright © 1982 John Wiley & Sons, Ltd.)](image)

$\phi(t)$, however, cannot be used directly for the evaluation of the apparent propagation velocity of the motions: O’Rourke et al. (1982) noted that the angle of incidence of incoming body
waves to the ground surface, \( \gamma_p \) and \( \gamma_s \) for compressional and shear waves, respectively, (Fig. 2), is not the same as \( \phi(t) \), because of the reflection of the waves on the ground surface. Utilizing an approximate analysis and considering small angles — less than 0.44 rad (25°) —, they proposed the following expression for the apparent propagation velocity of shear waves on the ground surface:

\[
C = \frac{C_s}{\sin \gamma_s} \approx \frac{C_s}{0.87\phi}
\]

where \( C_s \) is the shear wave velocity of the top layer. It is noted that, for a site that can be approximated by horizontal layers (Fig. 2), the incident waves impinging from the bedrock shift direction, and their path tends to become vertical, so that the assumption that \( \gamma_p, \gamma_s, \) and \( \phi \) are small is valid.

![Figure 2. Illustration of angle of incidence of incoming body waves to the ground surface (after O’Rourke et al., 1982; Copyright © 1982 John Wiley & Sons, Ltd.)](image)

Based on the aforementioned methodology, O’Rourke et al. (1982) estimated the apparent propagation velocity of shear waves for the 1971 San Fernando and the 1979 Imperial Valley earthquakes. For the 1971 San Fernando earthquake, which is more closely associated with the Northridge event, they estimated the apparent propagation velocity of the motions in the range of 1.26 to 9.33 km/sec, with a median value of 2.12 km/sec. O’Rourke and Castro (1981) reported strains of the order of 0.83–8.2 \( \times 10^{-4} \) for near-field sites during the San Fernando earthquake.

### (b) Evaluation of seismic strains from surface waves

For the evaluation of peak ground strains from surface (Rayleigh) waves, O’Rourke, Castro and Hossain (1984) used Eq. 1 rewritten as:

\[
\varepsilon_{\text{max}} = \frac{v_{\text{max}}}{C_{p\phi}}
\]
where $C_{ph}$ is the phase velocity of the wave. For an arbitrary layer of thickness $H$ and shear wave velocity $C_L$ overlying a half-space with shear wave velocity $C_H$, O’Rourke et al. (1984) provided the following semi-empirical dispersion expression for $C_{ph}$:

$$C_{ph} = \begin{cases} 
0.875C_H & \text{if } Hf / C_L \leq 0.25 \\
0.875C_H - (0.875C_H - C_L)(Hf / C_L - 0.25)/0.25 & \text{if } 0.25 \leq Hf / C_L \leq 0.5 \\
C_L & \text{if } Hf / C_L \geq 0.5 
\end{cases}$$  \hspace{1cm} (6)

The rationale behind Eq. 6 is that, for low frequencies ($Hf / C_L \leq 0.25$), i.e., large wavelengths, the waves do not “see” the layer and propagate with a velocity close to that of the half-space. For large frequencies ($Hf / C_L \geq 0.5$), i.e., short wavelengths, the layer dominates and the waves propagate with the shear wave velocity of the layer. In between these two extremes ($0.25 \leq Hf / C_L \leq 0.5$), there is a linear transition region.

The above expression can be extended to more than one layers. O’Rourke et al. (1984) presented an example for seismic strain evaluation in a medium consisting of two layers overlying a half-space. The seismic strain was then used to estimate differential displacements between the abutments of a bridge with a span length of 21 m; because the span length of this bridge is comparable to the dimensions of foundations (< 50 m – pertinent to the problems addressed in this workshop), their example is presented here in its entirety. Consider that the site consists of two layers with thickness of 9.1 m and 15.2 m and shear wave velocities of 122 m/sec and 228 m/sec, respectively, overlying a half-space with shear wave velocity of 386 m/sec, as indicated in Fig. 3a.

Figure 3. Steps for the development of dispersion curve for a two layer over a half-space model: (a) actual soil profile; (b) soil profile for high frequency wave components; (c) approximate dispersion curve for the soil profile in (b); (d) soil profile for low frequency wave components; (e) approximate dispersion curve for the soil profile in (d). (from O’Rourke et al., 1984; Copyright © ASCE, ISSN 0733-9410/84/0009-1173)

1 Wavelength, $\lambda$, is defined as the ratio of velocity, $C$, over frequency, $f$, or the product of velocity, $C$, times period, $T$, i.e., $\lambda = C/f = CT$
For high frequencies (short wavelengths), the incoming waves do not “see” the half-space. Instead the second layer acts as the half-space in Eq. 6 (Fig. 3b), and the resulting approximate dispersion curve is presented in Fig. 3c; the solid line part of the curve is valid for these higher frequency components. On the other hand, for low frequencies (long wavelengths), the incoming waves “see” a single layer, equivalent to the two actual ones \( C_L = (C_1H_1 + C_2H_2) / (C_1 + C_2) = 188 \text{ m/sec} \), and the half-space. The approximate dispersion curve in this case is the solid line part of Fig. 3e. The approximate dispersion curve for the entire profile is presented in Fig. 4. O’Rourke et al. (1984) compared this approximate curve with the exact one (for Poisson’s ratio and material density of 0.30 and 2.0 g/cm\(^3\), respectively) and found that the two curves are in good agreement (Fig. 4).

![Figure 3c](image)

![Figure 3f](image)

Figure 3c. A labeled image for the text.

Figure 3f. Another labeled image for the text.

O’Rourke et al. (1984) observed that a wavelength of four times the separation distance, \( L \), between two locations \( \lambda = 4L \) would create a 90° out-of-phase motion between the locations. (A consideration of \( \lambda = 2L \) would create an 180° out-of-phase motion between the locations, but this would lead to unrealistically large differential motions and strains). For the span length of 21 m between the bridge abutments, the corresponding wavelength causing this out-of-phase motion would be \( \lambda = 84 \text{ m} \). Figure 4 then suggests that for a 3 Hz frequency, the phase velocity would be \( C_{ph} = 252 \text{ m/sec} \); this combination of values is appropriate, as the resulting wavelength \( \lambda = C_{ph} / f \) becomes 84 m. Consider next that the estimated particle velocity at the site is 0.34 m/sec. The associated strain, based on Eq. 5 would then be \( \varepsilon = 1.30 \times 10^{-3} \) and the relative displacement between the example bridge abutments in the direction of the span \( \Delta D = \varepsilon \times L = 2.73 \times 10^{-2} \text{ m} \).

The above analysis applies to the horizontal component of Rayleigh waves, which is considered to coincide with the direction of the span. If vertical strains and relative displacements are of interest, the relation between the horizontal and vertical components of the Rayleigh wave needs to be determined. For example, for a homogeneous half-space with Poisson’s ratio \( \nu = 0.25 \), the ratio of horizontal \((A_1)\) to vertical \((A_3)\) motion of the Rayleigh wave becomes \( A_1 / A_3 = 0.42 / (-0.62) = -0.68 \) (Newmark and Roesenblueth, 1971).
Expressions such as Eqs. 1 and 5, namely that transient seismic ground strains can be obtained from the ratio of particle velocity over the apparent propagation velocity of the motions, have been extensively used for the evaluation of seismic ground strains, as shown in the subsequent sections.

III. Analytical seismic ground strain evaluation

In the analytical evaluation of seismic ground strains, it is generally considered that the site can be approximated by horizontal layers overlying a half-space. Thus, site effects associated with irregular subsurface topography are neglected. Examples of results of such analyses are provided below.

(a) Longitudinal strain and tilt in the vicinity of a thrust fault

Bouchon and Aki (1982) evaluated analytically seismic ground strains, tilts and rotations in the vicinity of a strike-slip and a thrust fault buried in layered media. The thrust fault, with characteristics similar to the San Fernando earthquake, is shown in Fig. 5. In their model, the rupture starts at 13 km depth, propagates upward and stops 1 km below the surface; the width of the fracture zone is 12 km, and the velocity of rupture propagation is 2 km/sec. Longitudinal strains and tilts were evaluated at the 11 stations shown in Fig. 5 right above the trace of the fault.

\[ \rho = 2.3 \text{ gr/cm}^3 \text{ for the layer and } \rho = 2.8 \text{ gr/cm}^3 \text{ for the half-space.} \]

Figures 6 and 7 indicate that, close to the fault, the maximum longitudinal strain and the maximum tilt reach values of \(8 \times 10^{-4}\) and \(5 \times 10^{-4}\), respectively. From the tip of the fault
forward, the approximation that strain equals ground velocity over apparent propagation velocity is valid. Bouchon and Aki (1982) indicated that the apparent propagation velocity used in their comparison was close to the rupture velocity at the source and cautioned that the apparent propagation velocity needed in Eq. 1 is related to the velocity in the bedrock and the rupture velocity at the source rather than the velocity of the uppermost layer. It is noted that this comment does not contradict the rationale of Section IIa, where $C$ is evaluated from the propagation velocity of the top layer, divided, however, by a factor that increases its value.

**Figure 6.** Longitudinal strain and horizontal ground velocity at the stations of Fig. 5 (from Bouchon and Aki, 1982; Copyright © 1982 Seismological Society of America)
(b) Strain attenuation relations

Using the numerical codes developed by their research group, Trifunac and Lee (1996) presented attenuation relationships for ground strains evaluated as functions of peak ground velocity. They note that radial ($\varepsilon_r$), shear ($\varepsilon_\theta$) and vertical ($\varepsilon_z$) strains can be expressed as $\varepsilon = A\varepsilon_{max}/\beta$, where $A$ is a site specific scaling function and $\beta$ is the shear wave velocity of the top layer. They indicate that, under strike slip faulting conditions at the location of Westmoreland in Imperial Valley, CA, a 50 m layer with $\beta \sim 300$ m/sec, would require $A \sim 0.4$ for $\varepsilon_r$, $A \sim 0.2$ for $\varepsilon_\theta$, and $A \sim 1.0$ for $\varepsilon_z$. The distance and magnitude dependence of the radial and shear strains is presented in Fig. 8.
Figure 8. Radial and shear strain attenuation relations as functions of peak ground velocity with distance and magnitude (from Trifunac and Lee, 1996; Copyright © 1996 Elsevier Science Limited)
Extending the approach further, Todorovska and Trifunac (1996) presented a methodology for the development of peak strain microzonation analyses of large areas. In their example application they conducted a normalized peak strain microzonation evaluation of the Los Angeles metropolitan area. Their model considered that all the seismicity is associated with known faults (until the 1980’s) and a diffused seismic zone, and they utilized a modified scaling equation for peak velocity originally developed by Trifunac (1976). The normalized strain estimates are given by $c \varepsilon_{\text{max}}$, where $c$ is a measure of velocity (in cm/sec). Figure 9 presents the spatial distribution of the logarithm of the normalized peak strains with equal probability of exceedance ($p = 0.9$) in 50 years of exposure. The range of values for the normalized maximum strain in Fig. 9 is $\log_{10}(c \varepsilon_{\text{max}}) = 0.7 - 1.7$. For an estimate of $c \sim 500$ m/sec, the range of strains would be $0.1 \times 10^{-3} - 1 \times 10^{-3}$. For a very low probability of exceedance ($p = 0.01$), the highest value for the normalized maximum strain estimate was reported as $\log_{10}(c \varepsilon_{\text{max}}) = 2.9$, yielding, for the same value of $c$ as above, a maximum peak strain of $1.59 \times 10^{-2}$. Todorovska and Trifunac (1996) also note that larger strains ought to be expected in soft and deep soils, due to smaller shear wave velocity in the layer, and smaller strains ought to be expected in the basement rock.

Figure 9. Spatial distribution of the logarithm of the normalized peak strains with equal probability of exceedance ($p = 0.9$) in 50 years of exposure (from Todorovska and Trifunac, 1996; Copyright © 1996 Elsevier Science Limited)
IV. Seismic Strain Estimates from Recorded Data

Nearly all techniques for the estimation of strains from seismic motions rely on seismometer data with certain approximations for the propagation of the waveforms on the ground surface. This is due to the fact that there are relatively few direct measurements of strains. This section describes the commonly used techniques for strain evaluation from recorded data and highlights results of published work on strain estimates evaluated during various earthquakes including the Northridge earthquake.

(a) Single station strain estimates

The following assumptions are implicit in all approaches using (seismometer) recorded time histories at a single station to estimate seismic ground strains:

- seismic energy travels as plane waves
- medium is laterally homogeneous
- azimuth and horizontal velocity of the motions are known.

Based on the aforementioned assumptions, Gomberg and Agnew (1996) suggested the following relationships for radial, $\varepsilon_r$, tangential, $\varepsilon_{\theta\theta}$, and shear, $\varepsilon_{r\theta}$, strains for a single mode of Rayleigh and Love waves as:

$$
\varepsilon_r \approx \frac{1}{C_R} \frac{\partial u_r}{\partial t}; \quad \varepsilon_{\theta\theta} = 0; \quad \varepsilon_{r\theta} \approx \frac{1}{2C_L} \frac{\partial u_\theta}{\partial t}
$$

where $C_R$ and $C_L$ are the Rayleigh and Love wave phase velocities, respectively, and $t$ is time. Equations 7 indicate that the tangential strain, $\varepsilon_{\theta\theta}$, should be zero, a consequence of plane wave propagation.

Gomberg and Agnew (1996) then proceeded with the comparison of the application of Eq. 7 to seismograms recorded at the Pinyon Flat Observatory in California with seismic strains measured directly by strainmeters. They considered three regional earthquakes: St. George, Utah, at a distance of 470 km; Little Skull Mountain, Nevada, at a distance of 345 km; and Northridge, California, at a distance of 206 km. The range of frequencies was $f = 0.05 – 0.5$ Hz. Figure 10 presents the comparison between the single station strain estimates from seismograms and the strainmeter data.

Figure 10 suggests that, although $\varepsilon_{\theta\theta} = 0$ from the analytical considerations, the strainmeter data provide $\varepsilon_{\theta\theta} \neq 0$. Gomberg and Agnew (1996) attributed the existence of a non-zero tangential strain to the distortion and scattering of the strain field by lateral material heterogeneities and topography. Furthermore, they noted that the overall differences between the strain estimates from seismograms and those from strainmeter data can be attributed to the contribution of higher surface wave modes that propagate with veloci-
ties that vary differently with frequency for each mode. Indeed, they indicate that the accuracy of the phase velocities limits the accuracy of strain estimates from seismic data.

Figure 10. Comparison of single station strain estimates from seismograms (dashed lines) and strainmeter data (solid lines); \( \varepsilon_{00} = 0 \) for single station estimates (after Gomberg and Agnew, 1996; Copyright © Seismological Society of America)

(b) Single station estimates for the Northridge earthquake

Using a single station estimate approach, Trifunac et al. estimated peak velocities and peak ground strains (Trifunac, Todorovska and Ivanovic, 1996), and correlated the density of pipe breaks with ground strains (Trifunac and Todorovska, 1996) during the 1994 Northridge earthquake. They estimated seismic ground strain factors from peak ground velocities using the expression \( v_{\text{max}}/C_s \) for horizontal (radial, \( \varepsilon_{rr} \), and shear, \( \varepsilon_{r\theta} \)) strains, and \( v_{\text{max}}/1.73C_s \) for vertical, \( \varepsilon_z \), ones. The peak ground velocities were obtained from 169 recordings of the Northridge earthquake. They estimated the average \( C_s \) as 275 m/sec for sites over quaternary deposits and 475 m/sec over geological rock. Figure 11 presents contours of their strain estimates for horizontal (Fig. 11a) and vertical strains (Fig. 11b). The maximum strain factors that they determined were \( \sim 10^{-2.2} \) (\( = 6.31 \times 10^{-3} \)) for horizontal strains (at the Rinaldi Receiving Station) and \( \sim 10^{-3} \) for vertical strains (Trifunac, Todorovska and Ivanovic, 1996). To get a first approximation for an estimate of the actual
peak strains they suggest that, assuming that mostly surface waves contribute to the motions, radial strain factors are multiplied by 0.36, shear ones by 0.22 and vertical ones by 1.7. This would yield peak strains $\varepsilon_{rr} = 2.27 \times 10^{-3}$, $\varepsilon_{r\theta} = 1.39 \times 10^{-3}$ (if the same strain factor can be used for both horizontal directions), and $\varepsilon_z = 1.7 \times 10^{-3}$.

Figure 11. Horizontal and vertical peak strain factors of strong ground motion at the ground surface during the Northridge earthquake (from Trifunac et al., 1996; Copyright © 1996 Elsevier Science Limited)
The single station estimation technique was also used by Gomberg (1997) to evaluate principal strains during the Northridge earthquake. Figure 12 presents her results. She used ground velocities resulting from processed recorded accelerograms, and, for the apparent propagation of the waveforms, the standard southern California model assuming a first-arriving S-wave. To account for the source finiteness, she considered that the entire energy is emitted from one of three sub-events (A, B, and C) of the Northridge earthquake, according to the source model of Wald et al. (1996). Three different analyses were conducted, each utilizing a different sub-event. The estimated apparent propagation velocity was 3 – 5 km/sec, in agreement also with the results of Zhang and Papageorgiou (1996), for the large amplitude S-wave pulses that dominated many of the traces (Gomberg, 1997).

Figure 12. Estimates of horizontal principal strains and vertical strains at strong motion stations during the Northridge earthquake. Positive traces correspond to extension and negative ones to compression. The vertical strains are plotted underneath the principal horizontal ones. Three strain time histories are plotted for station U55; they correspond to the three different sub-events, A, B and C. For the remaining stations only
the plots corresponding to sub-event A are plotted; the numbers next to the peaks indicate maximum values for each sub-event. The shape of the strains determined from the different sub-event analyses varied significantly only for stations located close to each sub-event (from Gomberg, 1997; Copyright © 1997 Elsevier Science Limited).

The peak horizontal strain was observed at the Rinaldi Receiving Station, Granada Hills, for sub-event B and had a magnitude of $2.29 \times 10^{-4}$. This is about an order of magnitude less than the results reported by Trifunac et al. (1996). Parenthetically, it is noted that Zhang and Papageorgiou (1996) reported strains of the order of $10^{-4}$ in the major epicentral area, whereas Hutchings and Jarpe (1996) estimated that strains were of the order of $10^{-3}$ at the Interchange between Highways 14 and I-5, where several of the interchange structures collapsed. Gomberg (1997) also noted that vertical strains were less than half the value of the horizontal ones. This result is not in agreement with the strain factor estimates of Fig. 11.

A small part of the differences between Fig. 11 (and the crude strain estimates derived herein from the work of Trifunac et al., 1996) and Fig. 12 may be due to possible differences in the processing of the data. However, the large differences may be attributed to the contributing waves and the estimates of their apparent velocities: one study (Trifunac et al., 1996) assumes that mostly surface waves contribute to the motions and propagating with a velocity controlled by the surface layer divided by a factor less than one for horizontal strains (i.e., \( \{275–475 \text{ m/sec}\}/\{0.36–0.22\} = \{0.76–2.16 \text{ km/sec}\} \)), whereas the other (Gomberg, 1997) considers that shear waves dominate the motions with a bedrock shear wave velocity of 3–5 km/sec. The differences in the vertical strains may also be attributed to the same cause.

(c) Seismo-geodetic analysis

An alternative approach for seismic ground strain estimation that waives some of the single station method restrictions but introduces some new ones, and requires, basically, array data has been borrowed from geodecy and used, among others, by Spudich et al. (1995) and Bodin et al. (1997).

Following Bodin et al. (1997) for the description of the approach, the displacement gradient tensor, \( \mathbf{D} \), with elements \( \partial u_i / \partial x_j \), \( u_i \) indicating displacement in the \( i \)th direction and \( x_j \) distance in the \( j \)th direction, can be evaluated from an array of \( N \) sensors as follows:

\[
\Delta u_i^{(n)} = u_i^{(n)} - u_i^{(0)} = \frac{\partial u_i}{\partial x_j} \Delta x_j^{(n)} \; \; \; i,j = 1,2,3; \; \; n = 1, ..., N-1 \tag{8}
\]

where \( \Delta u_i^{(n)} \) is the differential displacement in the \( i \)th direction between station \( (n) \) and the reference station \( (0) \) and \( \Delta x_j^{(n)} = x_j^{(n)} - x_j^{(0)} \) is the separation distance between the stations in the \( j \)th direction. The displacement gradients, \( \partial u_i / \partial x_j \), fully describe strains and rotations, i.e., \( [\partial u_i / \partial x_j + \partial u_j / \partial x_i] / 2 \) and \( [\partial u_i / \partial x_j - \partial u_j / \partial x_i] / 2 \), respectively. At the
free surface, the stress free boundary conditions impose the following constraints on the displacement gradients:

\[
\frac{\partial u_1}{\partial x_3} = -\frac{\partial u_3}{\partial x_1}; \quad \frac{\partial u_2}{\partial x_3} = -\frac{\partial u_3}{\partial x_2};
\]

\[
\frac{\partial u_3}{\partial x_3} = -\lambda \left( \frac{\partial u_1}{\partial x_1} + \frac{\partial u_2}{\partial x_2} \right) / (\lambda + 2\mu)
\]  \hspace{1cm} (9)

where \(\lambda\) and \(\mu\) are the Lame parameters and the subscripts for direction are now specified as 1 for east, 2 for north and 3 for up. The remaining six independent displacement gradients can then be evaluated by solving Eq. 8 at \(x_3 = 0\). If more than three stations are used in the identification, a least-squares minimization scheme should be utilized.

The seismo-geodetic approach considers that deformation is spatially uniform and displacements vary linearly. Furthermore, the frequency range of the strain estimates is limited by the aperture of the array. The ratio of the estimated over the true displacement gradient is given by:

\[
\frac{[\frac{\partial u_i}{\partial x_j}]_{\text{geodetic}}}{[\frac{\partial u_i}{\partial x_j}]_{\text{exact}}} = \frac{\sin[\pi L/\lambda]}{[\pi L/\lambda]} = \frac{\sin[\pi L f/V]}{[\pi L f/V]}
\]  \hspace{1cm} (10)

where \(L\) is the distance between stations, \(\lambda\) indicates wavelength and \(V\) is the velocity of the contributing wave (Gomberg, 1997). Equation 10 suggests that, for \(~ 90\%\) accuracy in the displacement gradient estimates, \(L\) should be less than approximately \(1/4\) of the wavelength of the dominant energy in the signal (Bodin et al., 1997). An additional consideration that affects the accuracy of the seismo-geodetic method is that it utilizes differences of displacements (Eq. 8): When taking differences of measured displacements, the coherent signal subtracts out, but noise does not. Furthermore, the method requires at least three three-component stations with synchronous timing.

Gomberg et al. (1999) compared single station and seismogeodetic displacement gradient estimates using the data of the Geyokcha, Turkmekistan, array. The signal-to-noise ratio (SNR) of the array data was SNR \(\geq 100\), and the epicentral distance of the events analyzed were 64 to 904 km. Gomberg et al. (1999) used available frequency-wavenumber (F-K) analyses results for the azimuthal direction and the apparent propagation velocity of the incoming waves. (A brief description of F-K techniques is presented subsequently in Section Va). Figure 13 presents the comparison of single station and geodetic estimates of displacement gradients for an earthquake with epicentral distance \(~ 64\) km, azimuth 325° and apparent wave propagation velocity of 2.72 km/sec. The frequency range is 0.5 – 1.0 Hz. Four single station estimates and one geodetic estimate using the data of the four stations are shown in the figure; the maximum separation distance between the recording stations was 400 m. The agreement between the estimates in Fig. 13 is satisfactory, suggesting that the plane wave assumption is valid in this frequency range. Gomberg et al. (1999) noted, however, that for events with longer epicentral distances, more significant differences can be observed a few seconds after the onset of the S-wave, possibly due to the forward scattering and dispersive surface waves, for which propagation velocity decreases for later arriving frequencies.
Gomberg et al. (1999) then proceeded with the comparison of the displacement gradients estimated by the two techniques at higher (4.0 – 8.0 Hz) frequencies. They first evaluated single station estimates at four array stations with maximum separation distance of approximately 50 m. The resulting displacement gradients were very similar for all four stations (Gomberg et al., 1999). Figure 14 presents displacement gradients evaluated from one representative station. Additionally, Fig. 14 presents the seismo-geodetic estimates determined from the use of all four stations. The epicentral distance of the earthquake was ~ 136 km, with azimuth of 320° and apparent wave propagation velocity of 3.84 km/sec. Figure 14 shows large differences between the single station and the geodetic estimates. Furthermore, Gomberg et al. (1999) report that the geodetic estimates are unstable, i.e., they depend on the selection of the reference station in Eq. 8. The authors conclude that, in this higher frequency range, displacements vary nonlinearly over distances of ~50 m, scattering at the local structures underneath the array becomes important, and the assumption of plane wave propagation is no longer valid.
Figure 14. Comparison of a single station (solid line) and a geodetic estimate (dashed line) of displacement gradients in the frequency range of 4.0 – 8.0 Hz. The geodetic estimate was estimated from the data at the four stations with maximum separation distance of ~ 50 m. (from Gomberg et al., 1999; Copyright © 1999 Seismological Society of America)

The conclusions of Gomberg et al. (1999) regarding the variability in the estimates over relatively short separation distances at higher frequencies points directly to the significance and contribution of the coherency of seismic ground motions for strain estimation.

V. Spatial Variability of Seismic Ground Motions

Just the consideration of the apparent propagation of the motions would then provide a lower bound for actual seismic ground strains. Indeed, Bodin et al. (1997), from their analysis of seismic strains in the Valley of Mexico, suggest that Eq. 1 may, in certain cases, be uncertain within a factor of 2 to 3. Additional differential displacements and strains are caused by changes in the amplitudes and phases of the motions as well as arrival time perturbations of the waveforms at the various locations on the ground surface.
These effects are generally termed “spatial variability of seismic ground motions”. An extensive treatise on spatial variability has been presented in Zerva and Zervas (2002); a brief description is presented in the following.

Consider, e.g., that seismic ground motions (accelerations) at a location \( r_i \) on the ground surface can be approximated by the sum of sinusoidal functions:

\[
\ddot{a}(r_i, t) = \sum_{m=1}^{M} A_m \sin(\kappa_m \cdot r_i + \omega_m t + \phi_m)
\]  

where \( \kappa_m \) is the discrete wavenumber \( = \omega_m / \bar{s} \), \( A_m \) and \( \phi_m \) are the corresponding amplitude and phase of the sinusoids, respectively, and \( M \) is the number of discrete frequencies used in the approximation. The slowness \( \bar{s} \) may be identified by means of any of the F-K spectra estimation techniques described below. The term \( \kappa_m \cdot r_i \) then denotes the phase in the estimate that is caused by the apparent propagation of the waves. \( \kappa_m \cdot r_i \) is deterministic and caused by the average propagation velocity of the waves on the ground surface; there is also a random part in the wave propagation effect, which is caused by the arrival time perturbations of the waveforms that are particular for each recording station. Similarly, \( A_m \) and \( \phi_m \) consist of a part that is common to all stations as well as a random part that is, again, particular for each recording station. These differences constitute the spatial variation of the motions.

(a) Frequency-Wavenumber (F-K) analysis

As indicated in Eq. 11, part of the changes in the phase of the motions is caused by the apparent propagation of the waveforms on the ground surface. If Eq. 11 is further simplified, and the seismic motions at two stations \( i \) and \( j \) are described by a single sinusoidal wave with frequency \( \omega \), the delay in the arrival of the wave at the further away station \( j \) due to its propagation with velocity \( v_{app} \) would produce a phase difference between the motions at the two stations of \( \omega t_{ij} = \omega x_{ij} / v_{app} \), with \( t_{ij} \) indicating the time delay and \( x_{ij} \) the separation distance between the stations in the direction of wave propagation.

Frequency-wavenumber (F-K) analysis of array data provides estimates for the azimuth and the apparent propagation velocity of the waves impinging the array. The analysis requires data recorded at dense instrument arrays. The most celebrated array for the evaluation of the spatial variability of seismic ground motions is the SMART-I (Strong Motion ARray in Taiwan, Phase I), built by the University of California at Berkeley and the Institute of Earth Sciences at Taipei in Lotung, Taiwan (Bolt et al., 1982). The array, located on deep sedimentary soil layers, has been operational from 1980 to 1991. It consisted (Fig. 15) of a center station C00 and 36 additional ones arranged on three concentric circles (each with 12 equidistant stations), the inner-ring stations denoted by I, the middle-ring ones by M and the outer-ring ones by O; the radii of the circles are 200 m, 1000 m and 2000 m, respectively. In 1983 two stations E01 and E02 were added to the array.
Frequency-wavenumber analyses using two commonly used techniques, the ConVen tional (CV) method and the MUltiple SIgnal Characterization (MUSIC) are presented in Fig. 16 (Zerva and Liao, 2002). The F-K analysis was conducted for the data of Event 40 (magnitude \( M_t = 6.5 \); focal depth 16 km; source-site distance 67 km; azimuthal direction \( 195^\circ \)) during the strong motion S-wave window of duration 5.12s (12.50s-17.62s in the data recordings) in the E-W direction.

Figures 16 a and b are contour plots of the F-K estimates as functions of slowness in the two horizontal directions. Red color indicates the highest elevation and blue/black color the lowest one. The peak of each plot identifies the azimuth and apparent propagation of the body wave that controls the motions for the window analyzed. The line connecting the peak with the slowness origin identifies the direction of propagation of the waves, and the distance of the peak from the origin is the inverse of its apparent propagation velocity. Both techniques in Fig. 16 identify the peak of the contour plot at \( \bar{s} = \{0.1 \text{ sec/km}, -0.25 \text{ sec/km}\} \), which implies that the waves impinge the array with an azimuth of \( \phi = 201.8^\circ \) and an apparent propagation velocity of \( v_{app} = 3.71 \text{ km/sec} \); the identified azimuth is close to the epicentral direction (195\(^\circ\)) of the earthquake. Such estimates for the azimuth and the apparent propagation velocity of the motions can then be used in the single station strain estimates for the evaluation of seismic ground strains. It is noted, however, that such estimates for the slowness of the motions reflect only the average propagation
velocity of the waves but do not capture the arrival time perturbations at individual stations.

![SMART I, Event 40, Conventional, East-West](image1)

(a) Conventional Method

![SMART I, Event 40, MUSIC, East-West](image2)

(b) Multiple Signal Characterization Method

**Figure 16.** CV and MUSIC slowness results applied to the strong motion S-wave window in the E-W direction of Event 40 at the SMART-1 array (after Zerva and Liao, 2002)
Changes in the phase of the motions, as well as changes in their amplitudes contribute to the spatial variation of the seismic ground motions. Changes in the amplitudes of the motions have started being investigated much later than changes in the phases (Schneider et al., 1992; Zerva and Zhang, 1997; Zerva and Liao, 2002). On the other hand, the random variability in phases started being investigated essentially right after the SMART-I array became operational.

Coherency is a statistical measure of the variability in phases between seismic ground motions recorded at two stations. It is defined by the following complex function of frequency:

$$\gamma_{ij}(\omega) = \frac{S_{ij}(\omega)}{\sqrt{S_{ii}(\omega)S_{jj}(\omega)}}$$  \hspace{1cm} (12)

where $S_{ij}(\omega)$ and $S_{ji}(\omega)$ denote the auto-spectral density of the ground motions at each station, and $S_{ij}(\omega)$ represents the cross-spectral density between the motions at two stations. Alternative, but similar, descriptions for the coherency are also used; they are: the unlagged coherency, $\text{Re}[\gamma_{ij}(\omega)]$, with “Re” indicating the real part of complex values; the coherence: $|\gamma_{ij}(\omega)|^2$, and, very commonly, the lagged coherency, $|\gamma_{ij}(\omega)|$. The complex part of Eq. 12 reflects the propagation of the motions on the ground surface. The lagged coherency, $|\gamma_{ij}(\omega)|$, describes the correlation of the two time histories in the frequency domain. It is obvious that the lagged coherency, obtained from the absolute value of Eq. 12, is identically equal to unity. Smoothing over near-neighbor frequencies on the spectral estimates needs to be performed, and the smoothed lagged coherency satisfies the condition $0 \leq |\gamma_{ij}(\omega)| \leq 1$. The amount of smoothing applied to the raw spectral densities of the data should depend not only on the statistical properties of the coherency, but also on the resolution that needs to be maintained for accuracy in the structural response, for which the coherency model will be used (Abrahamson et al., 1991). Recently, Zerva and Beck (2003) presented an approach that bypasses the strict requirement of spectral smoothing in parametric estimates of the coherency.

Lagged coherency is a measure of “similarity” in the seismic motions, and indicates the degree to which the data recorded at the two stations are related by means of a linear transfer function. If, for example, one process can be obtained by means of linear transformation of the other and in the absence of noise (Newland, 1984), coherency is equal to one; for uncorrelated processes, coherency becomes zero. It is expected that at low frequencies and short separation distances, the motions will be similar and, therefore, coherency will tend to unity as $\omega \to 0$ and the separation distance between stations, $\xi$, also tends to zero, i.e., $\xi \to 0$. On the other hand, at large frequencies and long station separation distances, the motions will become uncorrelated, and coherency will tend to zero.
The value of the coherency in-between these extreme cases will decay with frequency and station separation distance. This observation has been validated from the analyses of recorded data, and the functional forms describing the lagged coherency at any site and any event are exponential functions decaying with separation distance and frequency. However, the functional forms for the coherency and its exponential decay with frequency and separation distance can vary significantly for different sites and for the same site but different events (Zerva and Zervas, 2002).

Abrahamson (1992) presented the relation between lagged coherency and random phase variability, which is summarized in the following: Let \( \phi_j(\omega) \) and \( \phi_k(\omega) \) be the (Fourier) phases at two stations \( j \) and \( k \) on the ground surface at frequency \( \omega = \omega_m \) (Eq. 11), after the wave passage effects have been removed. The relation between \( \phi_j(\omega) \) and \( \phi_k(\omega) \) can be expressed as:

\[
\phi_j(\omega) - \phi_k(\omega) = \beta_{jk}(\omega) \pi \varepsilon_{jk}(\omega)
\]

(13)

in which, \( \varepsilon_{jk}(\omega) \) are random numbers uniformly distributed between (-1,+1], and \( \beta_{jk}(\omega) \) is a deterministic function of the frequency \( \omega \), and assumes values between 0 and 1. \( \beta_{jk}(\omega) \) indicates the fraction of random phase variability that is present in the Fourier phase difference of the time histories. For example, if \( \beta_{jk}(\omega) = 0 \), there is no phase difference between the stations, and the phases at the two stations are identical and fully deterministic. In the other extreme case, i.e., when \( \beta_{jk}(\omega) = 1 \), the phase difference between stations is completely random. Based on Eq. 13 and neglecting the amplitude variability of the data, Abrahamson (1992) noted that the mean value of the lagged coherency can be expressed as:

\[
E[\gamma_{jk}(\omega)] = \frac{\sin(\beta_{jk}(\omega)\pi)}{\beta_{jk}(\omega)\pi}
\]

(14)

where \( E \) indicates mean value. It is easy to verify from Eq. 14 that when the lagged coherency tends to one, \( \beta_{jk}(\omega) \) is a small number, i.e., only a small fraction of randomness appears in the phase difference between the motions at the two stations; as coherency decreases, \( \beta_{jk}(\omega) \) increases, and, for zero coherency, \( \beta_{jk}(\omega) = 1 \).

In the following, the term “coherency” will indicate the lagged coherency of the data. Two of the most widely referenced coherency models that will also be used later on herein are the models of Luco and Wong (1986) and Harichandran and Vanmarcke (1986). A brief description of these models follows:

(i) Luco and Wong (1986) coherency model:

A simplified version of Luco and Wong’s expression for coherency is given by:
\[ |\gamma(\xi, \omega)| = \exp(-\alpha^2 \omega^2 \xi^2) \] (15)

where the coherency drop parameter \( \alpha \) controls the exponential decay of the function. The model is based on wave propagation through random media, as discussed further in Section VI. Luco and Wong suggested values for \( \alpha \) in the range \( 2 - 3 \times 10^{-4} \) sec/m, so that the exponential decay of the model conforms with the exponential decay of coherency observed in recorded data; it is noted, however, that \( \alpha \) is not an inverse measure of velocity (see Section VI). Figure 17 presents the exponential decay of Eq. 15 with separation distance and frequency for \( \alpha = 2.5 \times 10^{-4} \) sec/m. It is noted that this coherency model produces full correlation in the motions at low frequencies for all separation distances and decays to zero as frequency and separation distance increase.

**Figure 17.** Luco and Wong’s coherency model for \( \alpha = 2.5 \times 10^{-4} \) sec/m at separation distances of \( \xi = 100, 300 \) and 500m

(ii) **Harichandran and Vanmarcke (1986) coherency model:**

Harichandran and Vanmarcke (1986) used a double exponential function to match the coherency obtained from data recorded at the SMART-I array. Their coherency expression is given by:

\[
|\gamma(\xi, \omega)| = A \exp\left(-\frac{2B|\xi|}{a\theta(\omega)}\right) + (1 - A) \exp\left(-\frac{2B|\xi|}{\theta(\omega)}\right)
\]

\[
\theta(\omega) = k\left[1 + \left(\frac{\omega}{2\pi f_0}\right)^b\right]^{-1/2}; \quad B = (1 - A + aA)
\] (16)

The values of the parameters in Eq. 16 for Event 20 were determined as: \( A = 0.736, \ a = 0.147, \ k = 5210 \) m, \( f_0 = 1.09 \) Hz, \( b = 2.78 \). The exponential decay of the model for these
values of the parameters is presented in Fig. 18. It is noted that this coherency model produces values for the coherency less than one for zero frequencies, and tends to finite values as frequency and separation distance decrease. This behavior is due to the smoothing process for the evaluation of coherency from recorded data.

Figure 18. Harichandran and Vanmarcke’s coherency model from the data of Event 20 at the SMART-I array at separation distances of $\xi = 100, 300$ and $500m$

**VI. Foundation Response to Seismic Ground Motions**

In evaluating the seismic response of foundations subjected to seismic ground motions, researchers have used, among other techniques, the boundary element method and the finite element method. The foundations were considered to be rigid, flexible, single or multiple ones. In most cases, the seismic load was considered to be a single sinusoidal function with frequency varying over a certain range. Of special importance is the study by Luco and Wong (1986), who were the first to consider the effect of wave propagation and loss of coherency in seismic ground motions on the response of a rigid foundation. Their work is summarized in the following.

Luco and Wong (1986) considered the response of a flat, rigid, massless foundation bonded on a viscoelastic half-space (Fig. 19) and treated the problem as a boundary value one. As input excitations, they considered spatially variable seismic ground motions and developed transfer functions for the foundation response in six directions (Fig. 19): three translational ones: $\{U_{01}, U_{02}, U_{03}\}$ and three rotational ones $\{U_{04} = a \theta_{01}, U_{05} = a \theta_{02}, U_{06} = a \theta_{03}\}$, where $a$ is the characteristic length of the foundation. $U_{04}$ and $U_{05}$ indicate rocking, whereas $U_{06}$ indicates torsion.
Luco and Wong (1986) based their coherency expression on wave propagation over a distance $H$ in a random medium (Uscinski, 1977). For a three-dimensional domain, the spatial coherency function between two locations on the ground surface is given by:

$$
\gamma(x, x', \omega) = \exp[-(v\omega | x - x' | / \beta)^2] \exp[-i \omega (\frac{x_1}{c_m} - \frac{x_1'}{c_n})]
$$

(17)

The first exponential function in Eq. 17 is the lagged coherency, and the second represents wave propagation. In the expression, $x$ and $x'$ indicate (vector) location on the ground surface, and $x_1$ and $x_1'$ are the location coordinates in the source-site direction (direction of wave propagation); $v = \mu (H / r_0)^{1/2}$, with $r_0$ being the scale of random inhomogeneities along the wave path and $\mu$ a measure of the relative variation of elastic properties; $\beta$ is an estimate of the elastic velocity in the medium; and $c_m = c_n = c$ is the apparent propagation velocity of the motions in the $x_j$ direction (not to be identified with $\beta$). A simplified one-dimensional version of Eq. 17 was presented in Fig. 17 and Eq. 15, where $v/\beta = \alpha$. 

Figure 19. Layout of foundation and coordinate system (from Luco and Wong, 1986; Copyright © 1986 John Wiley & Sons, Ltd.)
Luco and Wong (1986) then proceeded with separate analyses of the effects of loss of coherency and wave propagation on the foundation response. They presented their results in the form of transfer functions for a square foundation of dimensions $a \times a$. The amplitude of the transfer functions is indicated by $\sqrt{A_{ii}^{jj}}$, where subscripts $ii$ indicate the component of the excitation and superscripts $jj$ the response at the $j$-th degree of freedom. Figures 20 and 21 present the amplitudes of the transfer functions for variable $\nu$ (loss of coherency effect only with seismic ground motions arriving simultaneously at all foundation locations, i.e., $c \rightarrow \infty$), and variable $\beta/c$ ($\nu = 0$, i.e., identical seismic motions propagating on the ground surface), respectively, as functions of the dimensionless frequency $a_o = \omega a / \beta$.
As can be seen from Fig. 20a, loss of coherency in a specific direction results in significant reduction of translation in the corresponding direction \((\sqrt{A_{11}^{41}}, \sqrt{A_{22}^{42}}, \sqrt{A_{33}^{43}})\). On the other hand, loss of coherency in one direction contributes only minimally to the translation in the other two directions. Figure 20b suggests that the rocking response is mostly affected by the vertical component of the motions \((\sqrt{A_{33}^{44}}, \sqrt{A_{33}^{55}})\), whereas the torsional response is associated with the horizontal motions.
Figure 20b also suggests that coherency affects the rotational response, which increases, at low values of $a_o$, as coherency decreases ($\nu$ increases).

Figure 21 presents the foundation response when only propagation effects are considered. Luco and Wong (1986) assumed in this case that the motions propagate along the $x_1$ direction. As can be seen from Fig. 21, decreasing propagation velocity ($\beta/c$ increasing) also reduces the translational response ($\sqrt{A_{ii}^{\theta \theta}}$, $\sqrt{A_{ij}^{\theta \theta}}$, $\sqrt{A_{ij}^{\theta \theta}}$). However, only the $x_2$ component of the motion introduces torsion ($\sqrt{A_{22}^{\theta \theta}}$) and the vertical component introduces rocking only about the $x_2$ axis ($\sqrt{A_{33}^{\theta \theta}}$).

Figure 21. Foundation response to seismic ground motions propagating in the $x_1$ direction (variable $c$, $\nu = 0$) (from Luco and Wong, 1986; Copyright © 1986 John Wiley & Sons, Ltd.)
Figures 20 and 21 then emphasize the importance of the spatial variability of ground motions on the foundation response. Luco and Wong (1986) note that loss of coherency produces effects comparable to the deterministic effects of wave passage including reduction of the translational components of the response at high frequencies and creation of rocking and torsional response components. Harichandran (1987) conducted a similar analysis for the evaluation of the translational response of square foundations (50 × 50 m to 71 × 71 m) subjected to spatially variable ground motions using base averaging. For the spatial coherency description he used the model of Harichandran and Vanmarcke (1986), Eq. 16, and considered wave propagation effects as well. He concluded that there is reduction in the translational structural response both due to the coherency decay and the traveling wave effects and that the latter effect is, in general, more significant.

Both analyses conclude that loss of coherency as well as wave propagation effects on the foundation response are significant, although their relative contribution differs in the work of Luco and Wong (1986) and Harichandran (1987). These differences can be attributed to the different types of solution approaches (boundary value vs. base averaging), the three-dimensional vs. one-dimensional analysis, and also to the differences associated with the coherency models used.

An example of the latter difference is illustrated in the following: Differential displacements control the quasi-static internal forces in extended structures. Consider then two sets of input excitations: one is following the (one-dimensional) coherency model of Luco and Wong (1986), Eq. 15 and Fig. 17, and the other of Harichandran and Vanmarcke (1986), Eq. 16 and Fig. 18. Root-mean-square (rms) differential displacements resulting from the two models (Zerva, 1994) are presented in Fig. 22a; a normalized Clough-Penzien spectrum (Clough and Penzien, 1975) was used in the evaluation. It can be seen from Fig. 22a that the Harichandran and Vanmarcke model (blue line) produces significantly higher values for the differential displacements than the Luco and Wong one (black line). This is due to the fact that the Harichandran and Vanmarcke model is only partially correlated in the low frequencies that control displacements (Fig. 18), whereas the model of Luco and Wong is fully correlated in that frequency range (Fig. 17). The two models also induce different dynamic response in the structures: Figure 22b presents the differential response spectra induced by the two models for separation distances of 100 and 300m. The differential response spectrum can be loosely described as the contribution of the spatial variability in the motions to the rms dynamic response of a single-degree-of-freedom oscillator resting at two supports that undergo such spatially variable ground motions during an earthquake (Zerva, 1992a); the differential response spectrum is a function of the natural frequency of the oscillator. The long separation distances (100 and 300m) in the figure are not to be identified with the distance between the supports of a single-degree-of-freedom oscillator, but rather as the contribution of the spatial variation of the motions underneath bridge supports at long separation distances to the dynamic response of the structure. Figures 22b suggest that when a coherency model produces higher correlation (Figs. 17 and 18), it also produces higher contribution to the dynamic response of the structure. Indeed, when the motions at the supports are fully correlated (coherency is identically equal to one), these identical motions at the structures supports would yield the highest contribution to the dynamic response of the structure.
lines in Figs. 22b). It should be noted, however, that equal support excitations would not necessarily yield the highest response in the structure, as they do not induce any quasi-static response (differential displacements, in this case, are equal to zero). Indeed, the selection of a coherency model in the dynamic response of extended structures has an important effect on their quasi-static and dynamic response (Zerva, 1990; 1992a).

**Figure 22.** Root-mean-square differential displacements and differential response spectra at separation distances of 100 and 300m based on the Harichandran and Vanmarcke (HV) and the Luco and Wong (LW) coherency models (after Zerva, 1994)
VII. Seismic Ground Strain Simulations

Proper descriptions of the spatially variable nature of seismic ground motions allow the direct simulation of seismic ground strains to be used in design. In the following, seismic ground strains are evaluated from spatial variability models developed for strong motion shear wave windows. A similar approach can be employed for seismic strains resulting from vertical motions using appropriate spatial variability models; the number of coherency models for vertical motions is, however, very limited.

The purpose of this section is to analyze the contribution of coherency and wave propagation in seismic ground strains. For this purpose, Zerva (1992b) used Luco and Wong’s (1986) model with two different values of $\alpha$ ($\alpha = 2 \times 10^{-4} \text{ sec/m}$ and $\alpha = 1 \times 10^{-3} \text{ sec/m}$); the graphical representation of the models is presented in Fig. 23.

![Figure 23](image-url)

**Figure 23.** Luco and Wong’s (1986) coherency models for two values of the coherency drop parameter $\alpha$ (from Zerva, 1992b; Copyright © 1992 Elsevier Science Publishers Ltd.)
Figure 24 presents the comparison of seismic ground strains ($\varepsilon$) evaluated using both loss of coherency and wave propagation with the strain estimate evaluated as the ratio of particle velocity over the apparent propagation velocity of the motions ($V/C$). The strain estimate considers that contributions to strains result only from the propagation of the waves on the ground surface. Specifically, Figure 24a presents the comparison of the two strain estimates for $\alpha = 2 \times 10^{-4}$ sec/m and $C = 1500$ m/sec; Fig. 24b for $\alpha = 1 \times 10^{-3}$ sec/m and $C = 1500$ m/sec; and Fig. 24c for $\alpha = 1 \times 10^{-3}$ sec/m and $C = 500$ m/sec.

(a) $\alpha = 2 \times 10^{-4}$ sec/m and $C = 1500$ m/sec

(b) $\alpha = 1 \times 10^{-3}$ sec/m and $C = 1500$ m/sec
Figure 24. Simulations of seismic ground strains ($\varepsilon$) and strain estimates ($v/C$) for various values of the coherence drop parameter $\alpha$ of Luco and Wong’s model and the apparent propagation velocity of the motions (from Zerva, 1992b; Copyright © 1992 Elsevier Science Publishers Ltd.)

Figure 24a demonstrates that the strain ($\varepsilon$) and the strain estimate ($v/C$) time histories, although not identical, are in fairly good agreement, implying that for seismic motions that do not exhibit significant loss of coherence (Fig. 23 for $\alpha = 2 \times 10^{-4}$ sec/m), the actual strains can be reasonably well represented by the strain estimate and Eq. 1 is valid. This is not the case, however, when the motions exhibit significant loss of coherence, as is the case for motions described by the model for Luco and Wong (1986) for $\alpha = 1 \times 10^{-3}$ sec/m (Fig. 23) and an apparent propagation velocity of $C = 1500$ m/sec (Fig. 24b). Figure 24b shows that the strain estimate grossly underestimates the actual strain, which, in this case, assumes significantly higher values than those of the strain resulting from motions with low degree of coherency loss (Fig. 24a). Only when the apparent propagation velocity of the motions is reduced to the value of $C = 500$ m/sec does the strain estimate approximate again fairly satisfactorily the actual strain (Fig. 24c). It is also noted that, in Fig. 24c, strain assumes its highest value.

The comparison of Figs. 24a-c then suggests that (i) the strain estimate of Eq. 1 underestimates seismic ground strains when the motions exhibit significant loss of coherency; and (ii) coherency cannot be neglected in the estimation of seismic ground strains.
VII. Spatial Coherency at Shorter and Longer Separation Distances

Evaluation of seismic ground strains requires coherency models that are valid at short separation distances. However, most coherency models have been evaluated for distances longer than 100m. These models cannot be readily extrapolated to shorter separation distances: Abrahamson, et al. (1990) investigated the same events recorded at the SMART-I array (separation distances of 200-4000 m) and the LSST array (separation distances of 0-100 m), which is located within the SMART-I array. They noted that coherency values extrapolated at shorter separation distances from the SMART-I data tended to overestimate the true coherency values obtained from the LSST data. They provided the following expression for the description of coherency of the LSST data:

$$\tanh^{-1} |\gamma(\xi, \omega)| = (2.54 - 0.012\xi)[\exp((-0.115 - 0.00084\xi)\omega) + \omega^{-0.578}/3] + 0.35$$  (18)

Harichandran (1991) also observed this phenomenon, and proposed the following relationship:

$$|\gamma(\xi, \omega)| = A \exp(-\frac{2|\xi|(1-A)}{ak})[1 + \left(\frac{\omega}{2\pi f_0}\right)]^{1/2} + (1-A)$$  (19)

to complement the Harichandran and Vanmarcke expression (Eq. 16) for shorter separation distances. Similarly, Riepl et al. (1997) observed, from their analyses of an extensive set of weak motion data recorded at the EUROSEISTEST site in northern Greece, that the loss of coherency with distance for their data was marked by a “cross over” distance, that distinguished coherency for shorter (8-100 m) and longer (100-5500 m) separation distances.

Another point worth noting regarding the behavior of coherency at shorter and longer separation distances is the following: Abrahamson et al. (1990), Schneider et al. (1992) and Vernon et al. (1991) from analyses of data at close by distances (< 100 m) observed that coherency is independent of wavelength and decays faster with frequency than with separation distance. On the other hand, independent studies of data at longer separation distances (> 100 m) by Ramadan and Novak (1993), and Toksöz et al. (1991) observed that the decay of coherency with separation distance and frequency is the same. It appears then that different factors control the loss of coherency in the data at shorter and longer separation distances.

IX. Local Coherency

Since coherency at short separation distances is of importance for strain evaluation, it can be considered worthwhile to develop coherency models that are particular for the location of interest. Based on this rationale, Zerva and Harada (1997) introduced a methodology to model coherency at short distances if the subsurface characteristics are known.
Consider, e.g., that the soil profile at a site of 1200 m width is known over a depth of 70 m; the cross section of the profile is shown in Fig. 25. Zerva and Harada (1997) considered shear (SH) wave propagation. The waves impinge the array at such angles that their propagation within the uppermost 70 m can be considered to be almost vertical (see Fig. 2); vertical shear wave propagation is then considered in the layer. They, furthermore, assumed that the waves arriving at the site have already lost full correlation due to their propagation in the bedrock, and their loss of coherency in the bedrock follows the model of Luco and Wong (Eq. 15) with \( a = 2.5 \times 10^{-4} \text{ sec/m} \). It is also noted that, although vertical propagation of the shear waves is considered in the layer, the apparent propagation velocity of the waves on the ground surface can be considered finite, due to the shift in the direction of their propagation as they travel through the softer surface layers (Fig. 2).

### Figure 25. Soil profile for local coherency methodology evaluation (from Zerva and Harada, 1997; Copyright © 1997 Elsevier Science Limited)

Zerva and Harada (1997) then conducted a statistical analysis of the soil profile by considering the characteristics of the individual “columns” in Fig. 25. They established a mean value for the soil layer’s predominant frequency as \( \omega_0 = 5.64 \text{ rad/sec} \) with an associated standard deviation of \( \sigma_{\omega} = 0.101 \) and scale of correlation \( b_{\omega} = 155.56 \text{ m} \). For the soft layer of the figure they assumed a value of critical damping of 20%. They then conducted a random vibration analysis to evaluate the soil layer transfer function and the ground response. Figure 26 presents their estimate of the ratio of the rms ground strain, \( \sigma_{xx} \), at the site over the rms of particle velocity, \( \sigma_{vv} \), as function of the apparent propagation velocity of the motions. Three variations of the seismic strains are presented in the figure: the first, termed “incidence motion coherence” in the figure, corresponds to incident wave effects only (i.e., ground motions exhibiting loss of coherency according to the Luco and Wong model and propagating on the ground surface, but the random variability in the layer properties is not considered); the second, termed “site coherence”, corresponds to site effects only (i.e., the incident motions are fully coherent and loss of coherency results only from the variability in the layer properties while motions still propagate on the ground surface); and the third, termed “total coherence”, incorporates the contributions of both the incident motion variability and the surface layer stochasticity.
Figure 26. Variation of rms strain ($\sigma_{\text{rms}}$) over rms particle velocity ($\sigma_{\text{vp}}$) with the apparent propagation velocity of the motions (from Zerva and Harada, 1997; Copyright © 1997 Elsevier Science Limited)

Figure 27. Variation of incident and surface motion coherency with frequency at separation distances of 40, 100, 200 and 400 m (from Zerva and Harada, 1997; Copyright © 1997 Elsevier Science Limited)
Figure 26 suggests that, for low values of the apparent propagation velocity, which is the case for surface rather than body wave propagation, the apparent propagation effects overshadow those of loss of coherency, and Eq. 1 is valid. For higher values of the apparent propagation velocity, which are more appropriate for the shear waves considered in the analysis, loss of coherency starts becoming important and, eventually, fully controls the values of the seismic ground strains as is reflected by the essentially constant values of the ratios in Fig. 26 at the high apparent propagation velocity range. Figure 26 further suggests that the effects of the site stochasticity contribute more to the total values of seismic ground strains than those of the incident motion variability. The aforementioned analysis then suggests that a simple approach may suffice for the site-specific evaluation of seismic ground strains. There is, however, a draw-back in the methodology, which is illustrated in Fig. 27.

Figure 27 compares the coherency at distances of 40, 100, 200 and 400 m for the incident and the surface motions. The plots for the coherency at the various separation distances are identical except for the drop of coherency in the vicinity of the dominant frequency of the site ($\omega_0 = 5.64$ rad/sec). This is conceptually correct: coherency is a measure of correlation between the motions at two ground locations. With all things considered being identical except for the dominant layer frequency, which varies randomly around its mean value for the different “columns” at the site (Fig. 25), it should be expected that the motions would lose correlation close to the frequency of the layer. This type of behavior was noted by Kanasevich (1981), who suggested that site resonances can be identified from “holes” in the coherence spectra of motions at adjacent locations, and by Cranswick (1988), who further indicated that perturbations with small deviations in the layer characteristics will produce the greatest changes in the response functions, and, since coherency is a measure of similarity in the motions, it will be low at the resonant frequencies. Liao and Li (2002) also reached a similar conclusion from their analytical evaluation of wave propagation through media with variable properties. However, coherency evaluated from recorded data indicates that coherency “troughs” are associated with amplitude “troughs” at the same frequencies – if the coherency troughs were occurring at the dominant frequencies of the site, they would be associated with amplitude peaks rather than troughs. It appears then that there are additional considerations that need to be taken into account for the proper evaluation of local coherency.

X. Coherent Propagating Motions vs. Actual Data

Seismic ground motions recorded over extended areas incorporate, in addition to the effect of wave passage with constant propagation velocity, perturbations in the arrival time of the waves at the various locations as well as variabilities in their amplitudes and phases. These latter effects can be a significant additional cause for differential motions and strains. It would be interesting at this point to compare coherent, propagating motions with actual seismic data. Coherent, propagating motions incorporate only the wave passage effect with constant velocity on the ground surface and produce strains fully consistent with Eq. 1.
(a) Approach

To illustrate the differences between coherent, propagating motions and actual seismic data, the approach developed by Zerva and Zhang (1997) and extended by Zerva and Liao (2002) is utilized: Consider that seismic data at an array of sensors are available. The strong motion shear wave window is analyzed. Frequency-wavenumber techniques are applied to the data to estimate the slowness, $\tilde{s} = k / \omega$, of the broad band shear wave. The ground motions at all recording stations of interest are then approximated by:

$$\tilde{a}(\tilde{r}, t) = \sum_{m=1}^{M} A_m \sin(\tilde{k}_m \cdot \tilde{r} + \omega_m t + \phi_m)$$  \hspace{1cm} (20)

an expression similar to Eq. 11. In Eq. 20, $\tilde{k}_m$ is the discrete wavenumber ($= \omega_m \tilde{s}$), $A_m$ and $\phi_m$ are the corresponding amplitude and phase of the sinusoids, respectively, and $M$ is the number of discrete frequencies used in the approximation. It is noted that the amplitudes and phases of the sinusoids, $A_m$ and $\phi_m$, in Eq. 20 are only functions of frequency and not of location, as was the case in Eq. 11. This is because Eq. 20 is used to approximate the ground motions at all locations of interest. The amplitude and phase of the sinusoids can then be evaluated from the least-squares minimization of:

$$E = \sum_{l=1}^{L} \sum_{n=1}^{N} [a(\tilde{r}_l, t_n) - \tilde{a}(\tilde{r}_l, t_n)]^2$$ \hspace{1cm} (21)

where $a(\tilde{r}_l, t_n)$ indicates the actual time histories, $L$ is the number of stations used in the minimization and $N$ is the number of time steps of the strong motion shear wave window considered. Because the amplitude and phase estimates are evaluated from information at all stations of interest by means of Eq. 21, the resulting ground motion approximation of Eq. 20 represents a coherent motion that propagates with constant velocity ($v = 1/|\tilde{s}|$) in the $-\tilde{s}$ direction.

Zerva and Zhang (1997) applied the methodology to SMART-I data. For the strong motion shear wave window of the motions recorded in the N-S direction of Event 5 ($M_L = 6.3$, epicentral distance of 30 km and focal depth of 25 km), their F-K analysis identified the slowness of the shear waves as $\tilde{s} = \{0.1 \text{ sec/m}, -0.2 \text{ sec/m}\}$, which implies that the waves impinge the array at an azimuth of 153° with an apparent propagation velocity of 4.5 km/sec. The comparison of the actual recorded data during the shear wave window with the coherent estimate of the motions, termed reconstructed motions, evaluated from Eqs. 20 and 21 using the center array station C00 and four inner ring stations, I03, I06, I09 and I12, of the SMART-I array (Fig. 15) is presented in Fig. 28.
Figure 28 suggests that the gross characteristics of the recorded time histories are captured by the “best fit” reconstructed ones, but not the details; these details constitute the random variability in the seismic ground motions. In other words, the reconstructed motions of Fig. 28 would produce seismic strains captured by Eq. 1; as can be seen from the figure, however, the differential motions between stations, and, consequently, the seismic strains are enhanced by the additional variability in the actual waveforms. These changes include the aforementioned perturbations from the constant slowness in the arrival of the waves at the various stations, and, also, variations in amplitudes and phases of the motions at the individual stations (consistent more with Eq. 11 for the ground motion approximation rather than Eq. 20).

(b) Arrival time perturbations

Figure 28 suggests that the actual motions at station I12 arrive slightly earlier than what the constant apparent velocity assumption would predict, whereas they arrive slightly later at stations I09 and C00. These arrival time perturbations are particular for each recording station and are caused by the upward propagation of the waves through the horizontal variation of the layers underneath the array. The character of these delays can be
random. Boissières and Vanmarcke (1995), based on a large number of SMART-I data, modeled the time delays due to wave propagation between two stations $j$ and $k$, $\Delta t_{jk}$, as the sum of a deterministic, $\Delta t_{jk}^{dp}$, and a random part, $\Delta t_{jk}^{r}$:

$$\Delta t_{jk} = \Delta t_{jk}^{dp} + \Delta t_{jk}^{r}$$ (22)

The deterministic part of the time delays is given by:

$$\Delta t_{jk}^{dp} = \xi_{jk} / v_{app}$$ (23)

i.e., the delay in the arrival of the waves propagating with a velocity $v_{app}$ between two stations having a separation distance of $\xi_{jk}$. Boissières and Vanmarcke (1995) modeled the random part, $\Delta t_{jk}^{r}$, of the arrival times of the motions as a normally distributed random variable with zero mean and standard deviation of:

$$\sigma = 2.7 \times 10^{-2} + 5.41 \times 10^{-5} \xi_{jk}$$ (24)

with $\xi_{jk}$ given in m.

A simple way to partially remove these arrival time perturbations is through alignment of the seismic motions. In the alignment process, a reference station is selected (the obvious choice of a reference station for the SMART-I array would be the center station C00, Fig. 15). The cross correlation between the reference station time history and the motions at individual stations is evaluated, the time lag associated with the peak value of the cross correlation identified, and the time histories are shifted as much as the identified time lag, so that propagation effects are partially removed. The term “partially” is used herein because the alignment of the motions still considers that the time histories propagate with constant velocity at all frequencies. In this case, the constant velocity is evaluated from the propagation of the motions between the two stations rather than from an average velocity considering the propagation of the motions through the array, as is the case in the slowness analyses. In reality, the apparent propagation velocity of the motions may vary slightly with frequency even for the broad-band shear wave window.

**(c) Amplitude and phase variability**

Once the seismic motions are aligned, Eqs. 20 and 21 can still be used for the identification of the amplitudes and phases, $A_m$ and $\phi_m$, of the coherent component of the aligned motions. In this case, however, the term $\tilde{k}_m \cdot \tilde{r}$ in Eq. 20 is set equal to zero, as, for the aligned motions, the apparent propagation of the waveforms is infinite and the motions arrive simultaneously at all stations. Figure 29 presents the variation with frequency of the amplitude and phase of the common component (thicker, solid line in the figure) of the aligned motions recorded in the N-S direction at C00 and four inner ring stations of
the SMART-I array (Fig. 15) during the strong motion shear-wave window of Event 5 (Fig. 28). Additionally, the figure presents the amplitudes and phases of the motions recorded at each individual station (thinner dashed lines); the amplitude and phase at each station in the figure are, basically, the Fourier amplitude and phase of the time history for the particular station and for the window analyzed.

Figure 29 suggests that amplitudes and phases at the individual stations vary around the common amplitude and phase. This variability gives rise to additional considerations for differential motions and strains. It is noted that in the low frequency range, the amplitudes and phases of the motions at the individual stations agree fairly well with the common component ones. This may be attributed to the long wavelength of the motions, which do not “see” the site irregularities. In this frequency range, coherency models assume values close to 1 (Figs. 17 and 18), and the evaluation of seismic strains from recorded data (e.g., Fig. 13) validate Eq. 1. As frequency increases in Fig. 29, the amplitudes and phases of the motions at the individual stations start deviating from the common amplitude and phase; increasing range of variability is observed as frequency increases. In this range of frequencies, the coherency models decrease with frequency (Figs. 17 and 18). These differences in the amplitudes and phases give rise to higher strains and can be, possibly, the cause for the discrepancies in the strain estimates obtained from recorded data in Fig. 14.

The behavior of amplitudes and phases around the common coherent component of the motions (Fig. 29) has been observed from additional data at the SMART-I array (Zerva and Zhang, 1997; Zerva and Liao, 2002). The pattern was also verified with data from the Chi-Chi earthquake recorded at the SMART-II array, a seismic event with small magnitude recorded at the PINYON array, a chemical explosion recorded at the ZAYA array, and the Northridge earthquake recorded at the Leona Valley stations (Zerva and Liao, 2002).
XII. Conclusions

According to the overview of current trends in seismic ground strain estimation presented herein, it appears that, still, Eq. 1, namely that strain can be approximated by the ratio of particle velocity over the apparent propagation velocity of the motions, is the most commonly used approximation. The validity, however, of Eq. 1 may be limited: Its basic assumption is that differential motions, and, consequently, seismic strains are caused only by the propagation of the motions, which, furthermore, are considered to propagate unchanged on the ground surface. This assumption may be appropriate for low frequency ground motions, but cannot be validated for higher frequency components. Additional causes for differential displacements and strains are the variation in the amplitudes and phases of seismic ground motions over extended areas, as well as perturbations in the arrival time of the waves at various locations from that predicted with the consideration of constant wave propagation velocity. The spatial variability of seismic ground motions describes these effects and its use in the estimation of seismic strains appears to be necessary.

It was also shown that the approximation of Eq. 1 is sensitive (to an order of magnitude) to the proper identification of the dominant wave component in the contributing motions and the estimation of its apparent propagation velocity. Indeed, the selection of the appropriate apparent propagation velocity of the motions is a complicated task, and the accuracy of its estimation limits the accuracy of strain estimates from recorded data.

The particular problem at hand, namely “Under what circumstances, if any, should transient ground surface deformations be considered as a potential cause of damage to improvements?” may require specific evaluations, including: State-of-the-art wave propagation analyses at representative sites where damage during the Northridge was observed utilizing complex site topography and realistic soil material modeling; validation of analytical results with recorded and/or experimental data where feasible; and soil-structure interaction analyses incorporating spatially variable seismic ground motions and advanced nonlinear models of structural response. Such analyses may start with the specific sites, but, should the outcome indicate that, indeed, transient seismic ground strains were the cause of the damage, the results of the analyses should be generalized, so that they can be realistically extrapolated to different sites and events to prevent damage in man-made structures during future earthquakes.

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