DYNAMIC SOIL PROPERTIES, SEISMIC DOWNHOLE ARRAYS AND APPLICATIONS IN PRACTICE

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ABSTRACT

Downhole arrays are deployed worldwide to record seismic ground response in near-surface strata. The information supplied by these arrays is increasingly becoming the basis for verification, and for development and calibration of predictive tools and design procedures. Advances in sensors and information technologies will further expedite this learning process, opening the door for worldwide sharing and collaboration. In this paper, the following topics are addressed: i) A summary of downhole array installations and data in the U.S. and in the Taiwan, Hualien and Lotung sites, ii) An overview of related current research efforts worldwide, iii) Downhole array system identification analyses for lateral and vertical site amplification, iv) Downhole array analyses related to liquefaction and availability of Internet websites for conducting online computations (http://cyclic.ucsd.edu), and v) Summary of findings, and needs towards future advancements.

KEYWORDS:

Soil Dynamics, Vertical Arrays, Downhole Arrays, Liquefaction, System Identification

INTRODUCTION

Currently, downhole arrays are being increasingly deployed on a worldwide scale. Numerous existing installations are active, and valuable recorded data sets are becoming available. Some of the recorded sets (e.g., those related to liquefaction) have received much attention, and have already yielded invaluable information. Many more arrays have recorded data from small tremors, and are awaiting the occurrence of significant earthquake excitation (from the engineering point of view). In this paper, sections are devoted to the following issues:

1. Summary of downhole arrays in the U.S. (and the Hualien and Lotung arrays in Taiwan). Downhole data sets from centrifuge testing investigations are also discussed.
2. Literature overview to demonstrate the wide interest in this area.
3. Sample analyses related to lateral and vertical seismic response at soft as well as stiff soil sites.
4. Downhole studies related to liquefaction and associated soil response.
5. Summary of findings and discussion of future needs.

In compiling this document, much information was generously provided by many colleagues in the US, Japan, Taiwan, Mexico, and New Zealand. This assistance has been of great value and is most appreciated (please see Acknowledgements).

SUMMARY OF EXISTING DOWNHOLE ARRAYS IN THE US (AND TAIWAN, HUALIEN AND LOTUNG ONLY)

Seismic downhole-array data provide a unique source of information on actual soil behavior and local site amplification effects. This section provides a summary of existing geotechnical arrays in the US as well as Hualien and Lotung in Taiwan. Some information about data availability is also provided. Table 1 lists general information according to the best knowledge available to the authors. Thereafter, brief summaries are provided for the following specific sites covering geotechnical information and/or earthquake histories:

1. Borrego Valley Downhole Arrays, California, USA
2. Campus Laboratory Collaborative (CLC) Arrays, California
3. El Centro Downhole Array, California, USA
4. Garner Valley Array, California, USA
5. Hollister Borehole Array, California, USA
6. La Cienega Downhole Array, California, USA
7. San Francisco Bay Area Bridge Borehole Arrays, California
8. San Francisco City Borehole Arrays, California, USA
9. SCEC Los Angeles Borehole Arrays, California, USA
10. Tarzana Downhole Array, California, USA
11. Treasure Island Geotechnical Array, California, USA
12. Wildlife Refuge Array, Imperial County, California USA
13. Hualien Arrays, Taiwan
14. Lotung Arrays, Taiwan
Table 1: Summary of Borehole Arrays in the US and Taiwan (Hualien and Lotung only).

<table>
<thead>
<tr>
<th>No</th>
<th>Station Name</th>
<th>Year Installed</th>
<th>Sensor Depths, m</th>
<th>Geology</th>
<th>Agency</th>
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<td>1</td>
<td>Borrego Valley Downhole Array</td>
<td>1993</td>
<td>0, 9, 19, 139, 238</td>
<td></td>
<td>Kajima, UCSB</td>
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<tr>
<td>2</td>
<td>CLC U.C. Riverside</td>
<td>1998</td>
<td>0, 30, 100</td>
<td></td>
<td>UCCLC</td>
</tr>
<tr>
<td>3</td>
<td>CLC U.C. San Diego</td>
<td>1997</td>
<td>0, 46, 91</td>
<td></td>
<td>UCCLC</td>
</tr>
<tr>
<td>4</td>
<td>CLC U.C. Santa Barbara</td>
<td>1997</td>
<td>0, 75</td>
<td></td>
<td>UCCLC</td>
</tr>
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<td>El Centro Array</td>
<td>1998</td>
<td>0, 30, 100, 195</td>
<td>Deep alluvium</td>
<td>Caltrans, CSMIP</td>
</tr>
<tr>
<td>6</td>
<td>Eureka Array</td>
<td>1995-97</td>
<td>0, 19, 33, 16, 136</td>
<td>Deep soft alluvium</td>
<td>Caltrans, CSMIP</td>
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<td>7</td>
<td>Garner Valley Downhole Array</td>
<td>1989</td>
<td>0, 6, 15, 22, 50, 220, 500, 501</td>
<td>UCCLC</td>
<td></td>
</tr>
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<td>8</td>
<td>Hollister Downhole Array</td>
<td>1992</td>
<td>0, 10, 20, 50, 110, 192</td>
<td>UCCLC</td>
<td></td>
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<td>Los Angeles - La Cienega Array</td>
<td>1994-98</td>
<td>0, 18, 100, 252</td>
<td>Deep soft alluvium</td>
<td>Caltrans, CSMIP</td>
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<td>10</td>
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<td>1994-98</td>
<td>0, 3, 36.3, 57.6, 60.96, 150, 159.7</td>
<td>LLNL, Caltrans, UCB</td>
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<td>SF Bay area Bridge - Dumbarton Bridge</td>
<td>1993-94</td>
<td>0, 1.5, 71.6, 228</td>
<td>Rock</td>
<td>LLNL, Caltrans, UCB</td>
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<td></td>
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<td>189.2, pile cap 157.9</td>
<td></td>
<td></td>
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<td>12</td>
<td>SF Bay area Bridge - Golden Gate Bridge</td>
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<td>Rock</td>
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<td>0, 30, 88</td>
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<td>0, 30, 88 (Velocity)</td>
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<td>0, 8, 21, 42</td>
<td></td>
<td>USGS</td>
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<td></td>
<td>42 (Velocity)</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>8, 21 (Pressure)</td>
<td></td>
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<td>15</td>
<td>San Francisco - Embarcadero Plaza</td>
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<td>0, 10, 49, 79</td>
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<td>USGS</td>
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<td></td>
<td></td>
<td></td>
<td>79 (Velocity)</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>10, 49 (Pressure)</td>
<td></td>
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<td>16</td>
<td>San Francisco - Bessie Carmichael School</td>
<td></td>
<td>0, 10, 37, 90</td>
<td></td>
<td>USGS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 (Velocity)</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>10, 37 (Pressure)</td>
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<td></td>
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<tr>
<td>17</td>
<td>Tarzana - Cedar Hill B</td>
<td>1997</td>
<td>0, 60</td>
<td></td>
<td>ROSRINE, Caltrans, CSMIP</td>
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<tr>
<td>18</td>
<td>Treasure Island Array</td>
<td>1992-96</td>
<td>0, 7, 16, 31, 44, 104, 122</td>
<td>Fill, alluvium, rock</td>
<td>NSF, CSMIP</td>
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<td>Wildlife Refuge Array</td>
<td>1982</td>
<td>0, 7, 5</td>
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<td>USGS</td>
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<td>20</td>
<td>Parkfield - Turkey Flat #2</td>
<td>1987</td>
<td>0, 11, 23</td>
<td></td>
<td>Caltrans, CSMIP</td>
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<tr>
<td>21</td>
<td>Parkfield - Turkey Flat #1</td>
<td>1988</td>
<td>0, 24</td>
<td></td>
<td>Caltrans, CSMIP</td>
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<tr>
<td>22</td>
<td>Los Angeles - Vincent Thomas Array East</td>
<td>1998</td>
<td>0, 18, 46, 91</td>
<td>Deep soft alluvium</td>
<td>Caltrans, CSMIP</td>
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<tr>
<td>23</td>
<td>Los Angeles - Vincent Thomas Array West (two close sites combined)</td>
<td>1998</td>
<td>0, 15, 30, 91, 189</td>
<td>Deep soft alluvium</td>
<td>Caltrans, CSMIP</td>
</tr>
<tr>
<td>24</td>
<td>Los Angeles - Stone Canyon Reservoir</td>
<td>1998</td>
<td>0, 100</td>
<td></td>
<td>SCEC</td>
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<tr>
<td>25</td>
<td>Los Angeles - Griffith Park Observation</td>
<td>1998</td>
<td>0, 100</td>
<td></td>
<td>SCEC</td>
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<td>26</td>
<td>Los Angeles - Wonderland Ave. School</td>
<td>1998</td>
<td>0, 60</td>
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<td>SCEC</td>
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<td>27</td>
<td>Los Angeles - Mira Catalina School</td>
<td>1998</td>
<td>0, 30</td>
<td></td>
<td>SCEC</td>
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<td>28</td>
<td>Los Angeles - Obregon Park</td>
<td>1998</td>
<td>0, 70</td>
<td></td>
<td>SCEC</td>
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<tr>
<td>29</td>
<td>Central Fire Station, San Bernardino</td>
<td>1995</td>
<td>0, 100</td>
<td></td>
<td>USGS</td>
</tr>
<tr>
<td>30</td>
<td>Cerritos College Police Station</td>
<td>1995</td>
<td>0, 90</td>
<td></td>
<td>USGS</td>
</tr>
<tr>
<td>31</td>
<td>Hayward - San Mateo Bridge Array</td>
<td></td>
<td>0, 10, 23, 46, 91</td>
<td>Deep alluvium</td>
<td>Caltrans, CSMIP</td>
</tr>
<tr>
<td>32</td>
<td>Newhall - 14/5 Sep/OH (Ramp C)</td>
<td></td>
<td>10, 30 in pile</td>
<td>Rock</td>
<td>Caltrans</td>
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</table>
Borrego Valley Downhole Arrays (Archuleta and Steidl 1998)

The Borrego Valley downhole array, deployed by Kajima Engineering and Construction Corp. and Agbabian Associates (Dr. Robert Nigbor) in 1993, is located near San Jacinto Fault in Southern California. In this array there are four borehole instruments extending to a depth of 238m, and a remote rock site that includes a borehole sensor. At the main station, the shear wave velocity ranges from about 300m/s at the surface to 750m/s at the 230m granite interface (where it jumps to 2500m/s). The water table is at 92m depth (Archuleta and Steidl 1998).

Campus Laboratory Collaborative (CLC) Arrays (Archuleta et al., 2000b)

The Office of the President of the University of California (UCOP) initiated a program to install borehole arrays at a number of University of California campuses (Francois Heuze, P.I., LLNL). The program started in March 1996 as a partnership between four campuses of the University of California – Los Angeles, Riverside, San Diego, and Santa Barbara - and the Lawrence Livermore National Laboratory (LLNL). The current studies focus on the Riverside, San Diego, and Santa Barbara downhole arrays. Each of the boreholes is logged for P and S wave velocities, and for the collection of undisturbed samples (Archuleta and Steidl 1998, Minster et al. 1999, and Archuleta et al. 2000b). A number of low amplitude tremors were recorded at all these sites (contact: Dr. Francois Heuze at heuze@llnl.gov).

El Centro Downhole Array (Graizer et al. 2000)

The El Centro site consists of a deep soft alluvium profile, with shear wave velocities (Norris 1988) increasing from approximately 150m/s near the surface to 450m/s at a depth of 100m (silt, sand and clay). Surveys for P and S-wave velocities were performed by Caltrans. Table 2 lists 5 small amplitude earthquakes recorded at the surface, 30 and 100m.
Table 2: Earthquakes recorded by El Centro Geotechnical Array (after Graizer et al. 2000).

<table>
<thead>
<tr>
<th>No.</th>
<th>Date yr/mo/dy</th>
<th>Time(UTC) hh:mm:ss</th>
<th>ML</th>
<th>Lat</th>
<th>Long</th>
<th>Depth (km)</th>
<th>Epic dist. (km)</th>
<th>PGA, (g)</th>
<th>Acc. Recorded at Depth, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>99/07/24</td>
<td>02:01:26.0</td>
<td>3.9</td>
<td>32.770</td>
<td>115.560</td>
<td>15.4</td>
<td>10.6</td>
<td>0.015</td>
<td>0, 30, 100</td>
</tr>
<tr>
<td>2</td>
<td>99/10/16</td>
<td>09:46:44.1</td>
<td>7.1</td>
<td>34.594</td>
<td>116.271</td>
<td>6.0</td>
<td>216.0</td>
<td>0.015</td>
<td>0, 30, 100</td>
</tr>
<tr>
<td>3</td>
<td>00/04/09</td>
<td>10:48:09.7</td>
<td>4.3</td>
<td>32.692</td>
<td>115.392</td>
<td>10.0</td>
<td>10.4</td>
<td>0.042</td>
<td>0, 30, 100</td>
</tr>
<tr>
<td>4</td>
<td>00/06/14</td>
<td>19:00:20.0</td>
<td>4.2</td>
<td>32.896</td>
<td>115.502</td>
<td>5.1</td>
<td>14.6</td>
<td>0.014</td>
<td>0, 30, 100</td>
</tr>
<tr>
<td>5</td>
<td>00/06/14</td>
<td>21:49:18.0</td>
<td>4.5</td>
<td>32.884</td>
<td>115.505</td>
<td>4.9</td>
<td>13.5</td>
<td>0.008</td>
<td>0, 30, 100</td>
</tr>
</tbody>
</table>

Epic dist. (Epicenter distance)
Selected data sets available at ftp://ftp.consrv.ca.gov/pub/dmg/csmip/GeotechnicalArrayData/

Garner Valley Array (Archuleta and Steidl 1998)

The Garner Valley experiment (Mohammadioun and Gariel 1996) is sponsored jointly by the US Nuclear Regulatory Commission and the French Institut de Protection et de Surete Nucleaire (IPSN). This site is located in a seismically active area of southern California 7km east of the San Jacinto fault and within 35km of San Andreas fault (Archuleta et al. 1992). The array consists of three-component accelerometer stations at the surface and within the ground at depths of 6, 15, 22, 55, 220, 500 and 501m. It includes five surface instruments in a linear array, and a remote rock site that includes a surface and a shallow (30m) borehole sensor along with pore pressure transducers (Archuleta and Steidl 1998). At this location, the upper 18m of soil are followed by weathered granite (up to 45m), with solid granite bedrock thereafter. Shear-wave and P-wave velocity tests were conducted along with Standard Penetration and laboratory soil sample tests (Pecker 1995). The outcome of analyses of 218 recorded weak seismic motions was found to be in agreement with in-situ low-strain shear wave velocity measurements (Archuleta et al. 1992, and Mohammadioun and Pecker 1994). In addition, a damping mechanism proportional to the power 0.68 of the frequency was identified (Pecker 1995).

Hollister Borehole Array (Archuleta and Steidl 1998)

Agbabian Associates installed the Hollister Earthquake Observatory in 1991 with funding from Kajima. In 1998, Kajima Corporation donated this array to the University of California, Santa Barbara. It is located in the Salinas Valley of Southern California where alluvium overlies Tertiary sandstone overlying a granitic basement. The array consists of six accelerometers at the main station and three at remote rock stations - one on granite, one in the Tertiary sandstone and one in a borehole at 53m depth (Archuleta and Steidl 1998).

La Cienega Downhole Array (Graizer et al. 2000)

Sponsored by Caltrans, the array was installed to study the seismic response of a deep soil geologic structure in Los Angeles near the Santa Monica freeway (I-10) at La Cienega. The site consists of recent fluvial deposits of about 30m in thickness over marine deposits (sands, silts, clays and gravels). P and S-wave velocity surveys performed by Caltrans (suspension logging method) and the USGS indicate a 140m/s shear wave velocity near the surface with an increase to about 600m/s at a depth of 100m (Graizer et al. 2000). Eighteen earthquakes have been recorded at this site (Table 3).

San Francisco Bay Area Bridge Borehole Arrays (Hutchings et al. 1998)

This is a cooperative project between LLNL, UC Berkeley and Caltrans for instrumenting bridges in the San Francisco Bay region (Hutchings et al. 1998). A description of two borehole arrays is given in Table 1 (Bay Bridge and Dumbarton Bridge). The purpose of these arrays is to (Archuleta and Steidl 1998): 1) develop realistic predictions of strong ground motion at multiple input points along long span bridges; 2) examine ground motion variability in bedrock; 3) calibrate soil response models; 4) develop bridge response calculations with multiple support input motions; 5) evaluate the seismicity of potentially active faults in the San Francisco Bay; and 6) record strong ground motion.

San Francisco City Borehole Arrays (Borcherdt et al. 1999, 2000)

Four integrated borehole arrays and ten surface installations were deployed in the city of San Francisco, California to measure the seismic response of soft-soil deposits. Evidence of liquefaction during past earthquakes has been documented at three of the four sites (Embarcadero Plaza, Levi Strauss Plaza, Winfield Scott School in the Marina district, and Bessie Carmichael School). The borehole arrays extend through thick layers of soft water-saturated soils of Holocene age and older more consolidated soils of Pleistocene age into bedrock at depths up to 100m. Surface installations were configured in pairs to provide comparative surface measurements of soft soils and nearby rock (Borcherdt et al. 2000).

SCEC Los Angeles Arrays (Archuleta and Steidl 1998)

A major initiative by the Southern California Earthquake Center (SCEC) was undertaken to deploy borehole instruments throughout Southern California. The ultimate goal is to place approximately 10 borehole instruments in and around the Los Angeles basin in order to quantify the incoming wave field for studying basin effects as well as local site effects. Five of these sites have been drilled, sampled, logged and cased for instrumentation (Archuleta and Steidl 1998).
Table 3: Earthquakes recorded by La Cienega Geotechnical Array (after Graizer et al. 2000).

<table>
<thead>
<tr>
<th>No.</th>
<th>Date yr/mo/da</th>
<th>Time(UTC) hh:mm:ss</th>
<th>M_L</th>
<th>Lat</th>
<th>Long</th>
<th>Depth (km)</th>
<th>Epic dist. (km)</th>
<th>PGA, (g)</th>
<th>Acc. Recorded at Depth, m</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>95/06/26</td>
<td>08:40:28.9</td>
<td>5.0</td>
<td>34.390</td>
<td>118.670</td>
<td>13.3</td>
<td>47.6</td>
<td>0.011 0.009 0.002</td>
<td>0, 18, 100</td>
</tr>
<tr>
<td>2</td>
<td>97/03/15</td>
<td>15:24:47.7</td>
<td>5.1</td>
<td>34.970</td>
<td>116.820</td>
<td>1.8</td>
<td>176.7</td>
<td>0.003 0.003 0.007</td>
<td>0, 18, 100</td>
</tr>
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<td>3</td>
<td>97/04/04</td>
<td>09:26:24.5</td>
<td>3.3</td>
<td>33.980</td>
<td>118.350</td>
<td>4.2</td>
<td>6.7</td>
<td>0.076 0.058 0.013</td>
<td>0, 18, 100</td>
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<td>4</td>
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<td>09:35:09.5</td>
<td>2.4</td>
<td>33.990</td>
<td>118.360</td>
<td>4.5</td>
<td>6.4</td>
<td>0.010 (Lateral)</td>
<td>0, 18, 100</td>
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<td>33.990</td>
<td>118.360</td>
<td>4.1</td>
<td>6.4</td>
<td>0.021 0.013 0.003</td>
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<td>34.370</td>
<td>118.670</td>
<td>16.5</td>
<td>45.8</td>
<td>0.015 0.014 0.002</td>
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</tr>
<tr>
<td>7</td>
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<td>34.380</td>
<td>118.650</td>
<td>15.2</td>
<td>45.7</td>
<td>0.006 0.006 0.001</td>
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<td>34.190</td>
<td>118.470</td>
<td>11.3</td>
<td>19.1</td>
<td>0.005 0.008 0.001</td>
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<td>34.100</td>
<td>118.260</td>
<td>9.2</td>
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<td>0.008 0.013 0.002</td>
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<td>1.9</td>
<td>34.050</td>
<td>118.390</td>
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<td>1.9</td>
<td>0.012 (Lateral)</td>
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<td>01:11:50.1</td>
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<td>34.010</td>
<td>118.220</td>
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<td>34.010</td>
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<td>116.976</td>
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<td>0.035 (Lateral)</td>
<td>0, 18, 100, 252</td>
</tr>
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<td>0.007 (Lateral)</td>
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</tr>
<tr>
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</tr>
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</tr>
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<td>0.025 0.037 0.004</td>
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<td>0.021 0.062 0.005</td>
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</tbody>
</table>

Epic dist. (Epicenter distance)

Selected data sets available at ftp://ftp.cons.n.ca.gov/pub/dmg/csmip/GeotechnicalArrayData/

Tarzana Downhole Array (Graizer et al. 2000)

The Tarzana site is located on a gentle 20m high hill, about 500m in length by 130m in width. This site has been drilled and logged to a depth of 100m by Agbabian Associates (sponsored by CSMIP), and the accelerometers were installed at surface and at 60m depth. Low shear-wave velocities (about 200m/s) were found in the top 4m, with decomposed shale from 4 to 12m.

Highly to slightly weathered shale of the Modelo formation followed from 12m to 100m depth. Gypsum crystals were observed in the drill cuttings near 6m depth. Velocities increased gradually to near 750m/s around 80m depth, except in zones of hard shale (Darragh et al. 1997). The hill is well drained with a water table at a depth of 17m. Table 4 lists 11 earthquakes recorded at this array.

Table 4: Earthquakes recorded by Tarzana Array (after Graizer et al. 2000).

<table>
<thead>
<tr>
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<th>Time(UTC) hh:mm:ss</th>
<th>M_L</th>
<th>Lat</th>
<th>Long</th>
<th>Depth (km)</th>
<th>Epic dist. (km)</th>
<th>PGA, (g) Lateral direction</th>
</tr>
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<td>117.230</td>
<td>14.9</td>
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<td>2.3</td>
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</table>

Epic dist. (Epicenter distance)
Treasure Island Geotechnical Array (de Alba et al. 1994)

Treasure Island is a 160 ha reclaimed (man-made) island located in the San Francisco bay. It was constructed in the 1930’s (Pease and O’Rourke, 1995 and Power et al. 1995) of hydraulic fill over natural sand and Bay Mud. The fill (about 12m thick at the array location) is in a relatively loose condition, and is susceptible to liquefaction. Geologic formation at Treasure Island (including the upper loose hydraulic fill) is similar to that of nearby Marina District in San Francisco (Pease and O’Rourke, 1995, 1997). At both locations, widespread liquefaction with devastating consequences was documented during the 1989 Loma Prieta Earthquake (Pease and O’Rourke, 1995 and Power et al. 1995).

In the vicinity of the Treasure Island array, sand boils indicative of site liquefaction were observed (Hryciw et al. 1991, Darragh Instrumentation Program, and the National Science Foundation (de Alba et al. 1994). Among the main goals of installing this array (within a framework of a U.S. Geotechnical Test Site Network, Benoit and de Alba 1988) were: (1) to gather seismic data that would elucidate the mechanism of rock-motion amplification by deep soil deposits in the San Francisco area, and (2) to document the mechanisms of site liquefaction in the upper hydraulic fill strata (de Alba et al. 1994). The Treasure Island Geotechnical Array consists of six triaxial accelerometers located at the surface, 7, 16, 31, 44, 104 and 122m depths; and 6 piezometers located within the top 12m reclaimed hydraulic fill (de Alba et al. 1994, Shakal and Petersen 1992). Since the array installation, low amplitude data from 7 earthquakes have been recorded (Table 5). Site characterization and estimated P and S-wave velocity are described in Gibbs et al. (1992), Darragh et al. 1993, and Finn et al. 1993. The array site was instrumented in 1992 by the California Strong Motion Instrumentation Program, and the National Science Foundation (de Alba et al. 1994). Among the main goals of installing this array (within a framework of a U.S. Geotechnical Test Site Network, Benoit and de Alba 1988) were: (1) to gather seismic data that would elucidate the mechanism of rock-motion amplification by deep soil deposits in the San Francisco area, and (2) to document the mechanisms of site liquefaction in the upper hydraulic fill strata (de Alba et al. 1994). The Treasure Island Geotechnical Array consists of six triaxial accelerometers located at the surface, 7, 16, 31, 44, 104 and 122m depths; and 6 piezometers located within the top 12m reclaimed hydraulic fill (de Alba et al. 1994, Shakal and Petersen 1992). Since the array installation, low amplitude data from 7 earthquakes have been recorded (Table 5). Site characterization and estimated P and S-wave velocity are described in Gibbs et al. (1992), Darragh et al. 1993, and de Alba et al. 1994.

Table 5: Earthquakes recorded by Treasure Island Geotechnical Array (after Graizer et al. 2000).

<table>
<thead>
<tr>
<th>No</th>
<th>Date yr/mo/dy</th>
<th>Time(UTC) hh:mm:ss</th>
<th>ML</th>
<th>Lat</th>
<th>Long</th>
<th>Depth (km)</th>
<th>Epic dist. (km)</th>
<th>PGA, (g)</th>
<th>Acc. Recorded at Depth, m</th>
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</tr>
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</tr>
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<td>0.005</td>
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Epic dist. (Epicenter distance)

Selected data sets available at ftp://ftp.consrv.ca.gov/pub/dmg/csmip/GeotechnicalArrayData/
Table 6: Earthquakes recorded by the Hualien Array through 1995 (after Yang et al. 1995a,b).

<table>
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<th>Lat</th>
<th>Long</th>
<th>Depth (km)</th>
<th>Epic dist. (km)</th>
<th>PGA, (g)</th>
<th>Acc. Recorded at Depth, m</th>
</tr>
</thead>
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<tr>
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<td>12:18:45.36</td>
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<td>23.58</td>
<td>121.40</td>
<td>21.1</td>
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<td>49.5</td>
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<td>94/06/05</td>
<td>01:09:30.10</td>
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<td>24.27</td>
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<td>54.00</td>
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<td></td>
</tr>
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<td>31.3</td>
<td>95.8</td>
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<td></td>
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<td>21.83</td>
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<td>0.126</td>
<td>0.074, 0.023</td>
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</table>

Epic dist. (Epicenter distance)

**Figure 1: Hualien Downhole Array Sensor Arrangements (Yang et al. 1995a, b).**

Lotung Arrays, Taiwan (Tang 1987)

The Lotung site included constructions of ¼ and 1/12 scale models of a nuclear plant containment structure. The site was situated on a flat plain in a basin of triangular shape that is 15km wide and 8km long. Geological material at the site generally consists of recent alluvium Pleistocene material over a Miocene basement. The alluvium layer is 40-50m thick and the Miocene basement rock is located approximately 400m below ground surface at the test site (EPRI 1993). Extensive instrumentation was deployed to record seismic structural and ground responses, and to monitor soil pore-water pressure buildup. The ground instrumentation included (Fig. 2) three linear surface arrays and two downhole arrays (DHA and DHB) that extended to a depth of 47m below ground surface. Downhole triaxial accelerometers were installed at depths of 6, 11, 17, and 47m; oriented in the north-south (NS), east-west (EW), and vertical (UP) directions. Eighteen earthquakes were recorded during the period of 1985--1986 as listed in Table 7 (Tang 1987, Geomatrix Consultants 1990).
Table 7: Earthquakes recorded by Lotung Array (Tang 1987).

<table>
<thead>
<tr>
<th>No.</th>
<th>Date yr/mo/dy</th>
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<th>M_L</th>
<th>Depth (km)</th>
<th>Epic dist. (km)</th>
<th>PGA, (g)</th>
<th>Acc. Recorded at Depth, m</th>
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<td>EW</td>
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<td>LSST#3</td>
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<td>23.7</td>
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<td>31.4</td>
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<td>0.03</td>
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<td>LSST#11</td>
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<td></td>
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<td>0.05</td>
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<td></td>
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<td>0.10</td>
</tr>
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</tr>
</tbody>
</table>

Epic dist. (Epicenter distance)

![Diagram of instrument setup]

**Figure 2: Instrumentation at Lotung Site (Tang 1987, Elgamal et al. 1995).**
Soil Response from Centrifuge Testing Downhole Arrays (U.S.)

Rensselaer Liquefaction Research

Since 1992, the centrifuge facility at Rensselaer Polytechnic Institute (http://www.mi.edu/~dobry/centrifuge/) has generated many valuable liquefaction data sets (please see next section on literature overview). Contacts include Professor Ricardo Dobry (dobryr@rpi.edu), Director, and Dr. Tarek Abdoun (abdout@rpi.edu), Facility Manager.

UC Davis DKSO2 Centrifuge Experimental Arrays

Highly instrumented centrifuge experiments simulating a dense sand stratum were conducted recently at the University of California at Davis (UCD) to study the linear and nonlinear behavior of stiff soil sites during earthquake excitation (http://cgm.engr.ucdavis.edu/research/projects). This effort was initiated as a collaborative USGS funded project between UC Davis and UC San Diego. The experiments employed a Flexible Shear Beam (FSB) container mounted on the 9m-radius geotechnical centrifuge at UCD (Kutter et al. 1994). The FSB container dimensions are 1.651m x 0.787m x 0.584m in length, width and height respectively. Nevada sand at about 100% relative density was used to represent the stiff soil. Numerous accelerometers were installed to document the horizontal and vertical responses of the model (Fig. 3). The configuration of the FSB container and locations of the installed accelerometers are illustrated in Fig. 3. The recorded input-output accelerations provide an excellent opportunity for numerical model validation. The experiments were conducted at centrifugal acceleration levels of 10g, 20g and 40g respectively representing a prototype site of 5.5, 11, and 22m height respectively. At each acceleration level, the input motions included earthquake-like shaking, frequency sweeps and hammer impulse excitation. The peak ground surface acceleration among a total of 31 shaking events in prototype scale ranged from 0.05g to 0.99g in amplitude covering linear response as well as highly nonlinear response (Stevens et al. 1999, Lai et al. 1999, Elgamal et al. 2001, Stevens et al. 2001). Table 8 lists all earthquake events.

At UC Davis, 3 additional test (Stevens et al. 2001) configurations were modeled (rectangular rigid box, a saturated model, and an elliptic basin shaped model). This data is available at the UC Davis centrifuge website. At this website, a number of other valuable data sets are also available (please see http://cgm.engr.ucdavis.edu/)

Figure 3: Model Configuration of the DKSO2 FSB Container (after Stevens et al. 1999).
Table 8: USD DKS02 Centrifugal experiment central vertical array (Bruce Kutter, Center Director).

<table>
<thead>
<tr>
<th>No.</th>
<th>Event ID</th>
<th>Centrifuge Acceleration $g$ Level</th>
<th>PGA in Long., (g)</th>
<th>Acc. Recorded at Depth, m</th>
</tr>
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<tr>
<td>1</td>
<td>DKS02_c</td>
<td>10</td>
<td>0.073</td>
<td>0.25, 1.25, 2.5, 3.5, 4.75, 5.5</td>
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<td></td>
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<td></td>
</tr>
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<td>DKS02_f</td>
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</tr>
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<td>5</td>
<td>DKS02_ad</td>
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<td></td>
</tr>
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</tr>
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<td></td>
</tr>
</tbody>
</table>

Data available at http://cgm. engr. ucdavis. edu/ research/ projects/dks/ dks02/ index_ old. html

Long. (Longitudinal direction)
Seismic site response, manifested in the form of dynamic motion and ground failure, often results in costly and disastrous structural damage, e.g., during major earthquakes such as Sept. 19, 1985 Mexico (Seed et al. 1987), Oct. 17, 1989 Loma Prieta (Seed et al. 1990), Jan. 17, 1995 Kobe (EQE 1995, Sitar 1995, Bardet et al. 1995), Sept. 21, 1999 Ji-Ji, Taiwan (Uang et al. 1999), and Nov. 12, 1999, Dizce, Turkey (Ansal et al. 1999) earthquakes. In order to mitigate these disasters, it is crucial to understand the involved mechanisms of site response.

Soil shear stiffness characteristics at low-strain levels can be conveniently obtained from in-situ tests (e.g., Stokoe and Nazarian 1985, Nazarian and Desai 1993). At larger strain levels, soil dynamic characteristics are often evaluated through laboratory tests (NRC 1985). However, applicability of experimental results is somewhat restricted due to disturbance introduced during soil sampling, and difficulties in reproducing in-situ stress-state, seismic loading histories, site stratification conditions, and appropriate boundary conditions.

Site seismic response records are a valuable complement to the above in-situ and laboratory geotechnical investigation techniques. In particular, analyses of downhole seismic array data can provide unique information on actual soil and overall site dynamic characteristics, over a wide range of loading conditions that are not readily simulated by in-situ or laboratory experimentation procedures (Elgamal et al. 1997, Archuleta et al. 2000a, Baise and Glaser 2000c).

The earliest downhole arrays were perhaps deployed in Japan and the US. In the United States, an early downhole data set was recorded at Union Bay in Seattle, Washington (Seed and Idriss 1970a). This data was employed to verify site amplification procedures, and analyze the response of peat and clay deposits in Seattle. In Japan, an array of two surface and two downhole seismometers was installed on the premises of Tokyo Station, and recorded a set of seven earthquakes in the late 1950's (Shima 1962). Using these earthquake records, site resonance and damping characteristics were estimated (Shima 1962, Dobry et al. 1971). Other related notable efforts were directed towards earth dams in which crest and free-field earthquake records were employed (Abdel-Ghaffar and Scott 1978, 1979).

The above early efforts were followed by more comprehensive array installations worldwide. Using these arrays, soil dynamic characteristics are being increasingly examined through numerous downhole array records obtained from worldwide downhole array sites such as those in the US, Japan, Taiwan, Mexico, Greece, and New Zealand.

Most downhole arrays in the US are deployed in California, (e.g., Garner Valley, the Wildlife Refuge in Imperial County, Borrego Valley, Treasure Island, Hollister Municipal Airport, San Francisco, McGee Creek, Los Angeles, Cholame Valley near Parkfield, and Cajon Pass). Other sites in the US include Paducah in Kentucky (Street et al. 1997) and an array at Northeastern University in Boston, Massachusetts (Yegian, M. 2000, personal communication). In Japan, numerous downhole array sites were deployed (e.g., Port Island, Rokko Island, Chiba, Ashigara Valley, Kushiro, Hokkaido, Hiyoshi, TKS, KNK, SGK in Kobe, Shuzenji, Choshi, Tateyama, Higashimatsuyama, Sunamachi, Etchujima, TTRL, Samukawa, Fujisawa, ShinFuji, Sandai, Tomioka, Fuchu, Westa, Shizukuza, Iwaki, Yokohama, Kanto, and Tokyo International Airport). Taiwan is another seismically active region with well-instrumented downhole array sites (e.g., Lotung, Hualien, Wuku, Dahan and other arrays in the Taipei basin). Downhole arrays are also available in many other regions of the world such as Mexico, Mainland China, Greece, and New Zealand.

The following sections present a brief overview of downhole array studies reported by many researchers worldwide. Some of the techniques used in analysis and identification of ground seismic response are highlighted. In view of the extensive available literature, it was impossible to document every published study. With the assistance of many colleagues (please see Acknowledgements), all materials available or brought to the authors attention are compiled below. Future literature review efforts will hopefully be even more comprehensive, with additional details of conducted investigations.

Garner Valley, CA

A number of earlier studies on the Garner Valley downhole array were conducted by Archuleta et al. (1992). The Garner Valley downhole seismographic array, sponsored jointly by the US Nuclear Regulatory Commission and the French Institut de Protection et de Surete Nucleaire (Archuleta et al. 1992, Archuleta 1998), was designed to improve understanding of the effects of a shallow soil column on ground motion (Steidl et al. 1998). This site is in a seismically active area of southern California located 7 km east of the San Jacinto fault. At this location, the upper 18m of soil is followed by weathered granite (up to 45 m), with solid granite bedrock thereafter. The array consists of 5 accelerometer stations that extend to a 220m depth. The observational instrumentation is complemented by geophysical and geotechnical logging and sampling surveys and laboratory dynamic testing of samples. Outcome of weak seismic motion analyses of 218 recorded events was found to be in agreement with in-situ low-strain shear wave velocity measurements (Archuleta et al. 1992).

Archuleta et al. (1996) found that peak spectral amplification was about 8 over the frequency range 2-20 Hz with resonances around 1.9, 3.1, 6.0 Hz. The general site response was matched using the measured elastic wave velocities and assuming Q factors from 10-30 in the upper 20 m (for damping). They found that (Steidl et al. 1998) downhole recordings from as far away as 20 km (Pinon flat) could be used to predict the surface response at the downhole site. A surface rock site record only about 5 km away was not successful. They suggested that a single downhole recording was much better as a reference ground motion than a surface rock site record to predict site amplification (linear). Finally, aftershock recordings were used to predict main shock recordings for surficial peak accelerations less than 0.1 g, and the response was found linear to this level.
Using data from this array, Archuleta and Steidl (1998) investigated the effect of nonlinearity on soil response, amplification and attenuation of seismic waves, effects of smooth versus discontinuous variation of material properties, and water saturated versus dry soil response. In addition, Archuleta et al. (2000a) reproduced the observed surface ground motion using the geotechnical site characterization data, the observed downhole motions, and a linear wave propagation technique. They showed the need to be cautious when using surface rock motion as the input motion in site response analysis due to the site effects present at the rock outcrop.

Baise and Glaser (2000a, b) identified the empirical Green's function from depth to the surface at Garner Valley by isolating the systematic effects of the soil profile on the recorded downhole array seismic motions. They showed that the site ground-motion was consistent at Garner Valley. They suggested that consistency of site response over a wide range of shaking levels indicated an "effective" range of linear soil behavior for peak ground acceleration (PGA) up to 0.1 g. They found that the location and magnitude of significant impedance contrasts within the soil profile controlled site response of Garner Valley.

Ching and Glaser (2001) developed an extended-Kalman filter/State Space approach for direct material property estimation. They directly identified shear velocity and viscous damping coefficients for a number of Garner Valley site earthquakes using this Extended Kalman filter/State Space model.

Kelner et al. (1999) characterized the fracturing of the anisotropic shallow granite (< 250 m in depth) present at Garner Valley. They concluded that the seismic anisotropy and attenuation observed at Garner Valley was due to the presence of fractures in the granite.

Mohammadioun et al. (1992) and Mohammadioun and Gariel (1995) found that the transfer functions computed from downhole array data sets at Garner Valley agreed with those derived from a mono-dimensional model. Mohammadioun and Gariel (1996) validated a number of available seismological and engineering computer codes for modeling earthquake ground motion using data set accumulated at Garner Valley since 1989. Mohammadioun (1998) used the seismic records to analyze soil nonlinearity, horizontal to vertical response ratios, and the potential for microzoning.

Pecker (1995) compared the motions computed from the downhole array records at Garner Valley and found that a carefully conducted geotechnical survey could yield an accurate shear wave velocity profile. It was reported that rate-independent damping might not be appropriate to represent soil behavior in the small strain range.

Additional studies on the Garner Valley downhole array were conducted by Coutant (1996), Theodulidis et al. (1996), Tumarkin (1998), Tumarkin and Archuleta (1997), and Volant et al. (1998).

**Wildlife Refuge, Imperial County, California**

Evidence of liquefaction at or near the Wildlife Refuge site was observed following the 1930, 1950, 1957, 1979, and 1981 Imperial Valley earthquakes (Youd and Wieczorek 1984, Holzer et al. 1989). The Wildlife Refuge array was specifically designed to simultaneously measure pore pressure and ground motion. It was instrumented in 1982 by the United States Geological Survey with a surface and a downhole accelerometer at 7m depth, below a liquefiable layer, and six piezometers that record seismic induced excess pore-pressures (Holzer et al. 1989). The surface records during the November 24, 1987 Superstition Hills earthquake (Mw = 6.6) displayed peculiar acceleration spikes associated with simultaneous instants of excess pore-pressure drop (Holzer et al. 1989, Youd and Holzer 1994, Scott and Hushmand 1995).

Data collected at this downhole array provided important information for studies on nonlinear ground motion (Archuleta 1998). Archuleta and Steidl (1998) employed the 1987 seismic records to assess the magnitude and effect of nonlinearity on soil response. Yoshida and Iai (1998) used the downhole data to assess the nonlinear behavior of a soft surface ground stratum during strong earthquakes. They pointed out that effective stress analysis was better than nonlinear total stress analysis, and nonlinear analysis was better than equivalent linear analysis.

Glaser (1996) and Baise and Glaser (2000b) studied the effects of liquefaction using the stationary and recursive versions of a black box system identification (SI) method. The black box SI method was demonstrated to consistently characterize the Wildlife site ground motion. They found the site exhibited linear behavior for the first portion of the record prior to the onset of liquefaction and that once the soil started to liquefy and change state, nonlinear soil behavior dominated site behavior. Ching and Glaser (2001) applied an extended-Kalman filter/State Space approach for direct material property estimation.

Zeghal and Elgamal (1994), Zeghal et al. (1996b), and Elgamal et al. (1997) used cross-correlation analyses to assess the site stiffness degradation associated with the phase of sharp pore-pressure rise. The seismic shear stress-strain and effective stress path histories were directly evaluated using the recorded downhole accelerations and pore pressures. The dramatic change in site response due to liquefaction was evident in the stress-strain history of the Superstition Hills earthquake. During the strong shaking phase, the site experienced clear and gradual stiffness degradation associated with the sharp increase in recorded pore water pressure. At low effective confining pressures (high excess pore pressures), the effective stress-path clearly exhibited a reversal of behavior from contractive to dilative at large strains (NRC 1985, Zeghal and Elgamal 1994). Thus, this case history clearly showed an in-situ mechanism of shear stress hardening at large strain excursions during liquefaction. Such a mechanism has been observed in a number of experimental studies and is a consequence of soil dilation at large strain excursions, which results in associated instantaneous pore-pressure drops. This observed phenomenon of hardening at large shear strain excursions (during liquefaction) is of
Zorapapel and Vucetic (1994) used the ground surface and subsurface accelerations and pore water pressures recorded during the 1987 earthquakes to evaluate the effects of gradual buildup of seismic pore water pressure and associated degradation of stiffness on the ground surface motion for shallow saturated liquefiable deposits. Ground surface displacements and accelerations, the Fourier Transforms of the acceleration-time histories and layer gain factors were analyzed.

**Borrego Valley, California**

The Borrego Valley Downhole Array (BVDA) is located in the northern part of Borrego Valley adjacent to the Coyote Creek branch of the San Jacinto fault in Southern California (Archuleta 1998, Archuleta and Steidl 1998). BVDA consists of three different arrays: (1) a downhole array at the main station in the center of the valley, (2) two pairs of surface/downhole arrays at a rock outcrop and sites in its vicinity, and (3) two linear surface arrays between the main station and the rock station. Observation of strong motions at BVDA was initiated in early 1993, and more than 200 earthquakes with various magnitudes have been recorded. Various types of geophysical and geotechnical methods have been used to characterize the 1-D to 3-D geological structure, providing calibration and verification of site response modeling techniques for stiff Quaternary soil sites (Kato et al. 1998, Archuleta 1998).

Archuleta and Steidl (1998) evaluated the nonlinear soil response at Borrego Valley. They found the low strain attenuation (known as $Q = 1/2b$ where $b$ is damping) was 10-20 in all soil layers. By comparing the Borrego Valley Downhole Array (unsaturated) with Garner Valley and Hollister downhole arrays (saturated), they observed the difference in low-strain attenuation between saturated and unsaturated soils.

Nigbor et al. (1999) used 1D, 2D, and 3D models to calculate the ground shaking for an earthquake recorded by the array. They showed the strengths and weaknesses of different site response estimation methods.

Olsen et al. (2000a, b) simulated 2 Hz wave propagation in a three-dimensional model of the Borrego Valley for an earthquake with epicenter 5 km north of the valley. The simulation reproduced the overall pattern of ground motions and showed a correlation of observed to synthetic waveforms. The 3D simulation reproduced the recorded peak motions, cumulative kinetic energies, and Fourier spectral amplitudes within a factor of 2 for most components at the individual sites. Their prediction required the inclusion of anelastic attenuation in the simulation with $Q$ values for P- and S-waves in the saturated alluvium of about 30.

**Treasure Island, California**

Treasure Island is a reclaimed island located in the San Francisco bay. It was constructed in the 1930's of hydraulic fill over natural sand and Bay Mud. The fill (about 12 m thick) is in a relatively loose condition, and is susceptible to liquefaction. Geologic formation at Treasure Island (including the upper loose hydraulic fill) is similar to that of the nearby Marina District in San Francisco (Pease and O’Rourke 1995, 1997). At both locations, widespread liquefaction with devastating consequences was documented during the recent 1989 Loma Prieta Earthquake (Hryciw et al. 1991, Finn et al. 1993, Power et al. 1995). The Treasure Island liquefaction case history and associated loss of soil stiffness and strength was documented by Pease and O’Rourke (1997) using surface accelerations recorded at Treasure Island, and representative bedrock accelerations recorded at the nearby outcrop of Yerba Buena.

The Treasure Island downhole array is located between the San Andreas and Hayward faults. The array consists of six accelerometer stations extending to a 104m depth, and 6 piezometers located within the top 12m reclaimed hydraulic fill. The array was installed in 1992 by the California Strong Motion Instrumentation Program, and the National Science foundation (de Alba et al. 1992, de Alba et al. 1994, de Alba and Faris 1998). This array is intended to provide data that can be used to identify the mechanism of rock-motion amplification by deep soil deposits in the San Francisco area, the mechanisms of site liquefaction in the upper hydraulic fill strata, and to develop more reliable methods for predicting permanent ground deformations and displacements.

Since its installation, the array recorded a number of low amplitude shaking events, and is currently active in anticipation of a future strong earthquake excitation. These low amplitude events provide a source of benchmark information on Treasure Island low strain dynamic response characteristics. Elgamal et al. (1996a) employed the 1993 downhole records to evaluate: (1) shear wave velocity profile, (2) site shear stress-strain response, and (3) low-strain soil dynamic properties. The identified stiffness and damping parameters showed a noticeably higher damping ratio in the upper loose hydraulic-fill layer, possibly reflecting the soft nature of the fill, along with other surface wave propagation characteristics that were not modeled by the employed simple 1D shear wave propagation concept (Zeghal and Elgamal 2000). Baise (2000) and Yoshida and Iai (1998) also studied the Treasure Island site behavior using downhole array data.

**Hollister Municipal Airport, California**

At Hollister, the near-surface sediment consists of clay, sand, and gravel (Archuleta 1998). The downhole arrays were installed in 1991. The site is located in the Salinas Valley where alluvium overlies Tertiary sandstone overlying a granite basement. The array consists of nine accelerometers, six at the main station and three remote rock stations - one on granite, one on Tertiary sandstone and one in a downhole at 53m depth (Archuleta 1998). Archuleta and Steidl (1998) summarized the installation and data analysis of the Hollister downhole arrays. Archuleta et al. (1998) studied the records from two downhole arrays and two surface rock sites and found that a single downhole recording was much
better as a reference ground motion than a surface rock record to predict site linear amplification. More recent studies were conducted by Glaser and Baise (personal communication 1999).

San Francisco, California

A four-level downhole array of three-component instruments was established on the southwest shore of San Francisco Bay (Borcherdt et al. 1999). The deepest instrument is at a depth of 186 m, 2 m below the top of the Franciscan bedrock (Joyner et al. 1976). The arrays are designed to address the issue of anelastic and nonlinear site response at high strain levels (Borcherdt et al. 1999). Borcherdt et al. (1999) developed anelastic models to account for contrasts in characteristics at boundaries and the resultant inhomogeneity of propagating wave fields. They found that significant amounts of energy might be trapped in soil basins with resultant larger amplifications than those obtained from conventional homogeneous wavefield models.

McGee Creek, California

At the McGee Creek, California, site, three-component strong-motion accelerometers are located at depths of 166 m, 35 m, and 0 m. The surface material is glacial moraine, to a depth of 30.5 m, overlying hornfels. Accelerations were recorded from two California earthquakes: Round Valley, M_L 5.8, Nov. 23, 1984, and Chalfant Valley, M_L 6.4, July 21, 1986. Anti-plane shear strains have been determined for the Round Valley and Chalfant Valley events (Seale and Archuleta 1989). Seale and Archuleta (1989) applied a constant phase velocity Haskell-Thomson model to generate synthetic SH seismograms at the surface using the accelerations recorded at 166 m. They found that for low amplitude strain, a downhole time history of acceleration could be linearly propagated to the surface based on geotechnical logs of P and S-wave velocities in the vertical column. The synthetic acceleration time history could match the observed response both in phase and amplitude up to 10 Hz. In the frequency band 0.10 to 10.0 Hz, they found that the seismic amplification at the surface was a result of the resonance of the surface layers and the contrast of impedance (shear stiffness) in the near surface materials.

Los Angeles, California

Hauksson et al. (1987) described the downhole array operating in the seismically active Newport-Inglewood fault zone of the Los Angeles basin. Results of observations and measurements were given for an M_L 2.8 earthquake that occurred 0.9 km from the array on July 31, 1986, generating large vertical ground shaking. They reported results derived from P- and S-wave pulse shapes, near-surface site response and resulting amplifications of the P and S waves, Q values, wave spectra, corner frequencies (determined from displacement spectra), and the effects on f_0 on near-surface attenuation.

Cholame Valley near Parkfield, California

A wedging system for coupling downhole accelerometers into cased downholes has been developed and deployed at an Electric Power Research Inst. (EPRI)/U.S. Geological Survey (USGS) instrument array located in the Cholame Valley near Parkfield, California. The wedging system allows downhole accelerometers to be set and retrieved by hand. With this system, accelerometers can be readily retrieved from cased holes for inspection, maintenance, or redeployment (Youd et al. 1989).

Cajon Pass, California

Early estimates of Cajon Pass site response were limited to shallower holes, where the surface reflection interferes with the upgoing direct wave, and the deepest sensor was not below the severe near-surface effects, in bedrock (Abercrombie 1997). Following early efforts, deep downhole arrays were deployed, penetrating 500 m of Miocene sandstone and then crystalline, granitic basement rock. Near surface attenuation and site effects have been investigated using the seismograms of 17 local earthquakes recorded at depths of 0, 0.3, 1.5, 2.5, and 2.9 km (Abercrombie 1997).

Paducah, Kentucky

The downhole vertical accelerometer array VSAP near Paducah, Kentucky (Street et al. 1997), consists of three-component accelerometers at the surface, the top of the McNairy Formation (41 m depth), and the top of the Paleozoic bedrock (99 m depth). The array is at the northern end of the Mississippi Embayment. The array is intended to verify ground-motion modeling for the site, assuming a significant earthquake in the New Madrid Seismic Zone. Accelerograms from M_L 2.0 and 4.2 earthquakes have been used to check aspects of the modeling pertaining to linear behavior of the soil column. The soil column models derived by drilling and geotechnical methods and through the use of high-resolution P- and SH-wave seismic refraction and reflection techniques have been examined (Street et al. 1997).

Port Island, Japan

Port Island is a reclaimed island located on the southwest side of Kobe, Japan. Soil in the reclaimed layer consisted of decomposed weathered granite fill with grain sizes ranging from gravel and cobble-sized particles, to fine sand. A 4-station downhole accelerometer array extending to 83 m depth was installed at Port Island in August 1991 (Iwasaki 1995, Iwasaki and Tai 1996). The array site consists of: (1) a reclaimed, loose surface layer down to about 19 m depth, (2) an alluvial clay layer between 19 m and 27 m depth, (3) sand and sand with gravel strata interlayered with clay between 27 and 61 m depth, (4) a diluvial clay layer between 61 m and 82 m depth, and (5) sand with gravel layers interlayered with clay starting at about 82 m depth. The water table was located at a depth of 4 m approximately. The Port Island downhole array data have been the subject of numerous recent studies as described below.

Aguirre and Irikura (1995) showed the strong influence of nonlinear effects in horizontal components, and the linear amplification of vertical components. Thus explained why the vertical motions had disproportionate large amplitudes in
comparison to the horizontal ones. Aguirre (1996) used linear and nonlinear simulations to study the nonlinear site effects at Port Island during the 1995 Hyogo-ken Nanbu earthquake. Aguirre and Irikura (1997) used a spectral ratio technique and a nonlinear inversion process for the velocity structure.

Ansary et al. (1995) and Yamazaki et al. (1995) conducted analyses of the downhole records using a nonlinear dynamic effective stress method taking into account liquefaction under multi-directional shearing. Computed and recorded ground motions and associated response spectra were found to be in good agreement. Coupled effects of the two horizontal motions on the dynamic response and liquefaction of the ground were examined.

Baise and Glaser (2000a, b, c) identified the empirical Green's function at Port Island by isolating the systematic effects of the soil profile on the recorded ground motions. They indicated that strong nonlinear behavior (liquefaction) controlled site response. The stationary and recursive versions of the black box system identification method were used to show that the site exhibited linear behavior for the first portion of the record prior to the onset of liquefaction. Using foreshocks and aftershocks, they showed that the liquefied ground gradually returned to pre-liquefied site response conditions.

Cubrinovski et al. (1996) conducted a series of fully coupled effective stress analyses to investigate the ground response associated with the recorded accelerations. They showed a governing influence of the excess pore pressures on the ground response.

Davis and Berrill (1998a) and Davis (2000) presented a scheme for interpolating displacement and acceleration measurements using Port Island downhole array records to provide higher accuracy approximations for subsurface shear strain and stress as continuous functions of time. Davis and Berrill (1998b) analysed the Port Island downhole records from the Hyogo-ken Nanbu earthquake to obtain approximate histories of shear stress, shear strain and dissipated energy at a range of depths. They showed that a rapidly developing zone of liquefaction was initiated at a depth of roughly 15 meters in the Port Island reclaimed soils.

Elgamal et al. (1996c, 1997) and Zeghal et al. (1996a, b) evaluated shear stress-strain response using the recorded downhole accelerations. They found that two different response patterns were exhibited at the site. Below 32m depth, the shear stress-strain histories showed an essentially linear soil response, with no appreciable reduction in soil stiffness. On the other hand, at shallow depths (near the ground surface) the stress-strain histories indicated an abrupt sharp loss of stiffness and reduction of yield strength, evidently associated with site liquefaction. This liquefaction response mechanism was marked by the virtual absence of very large strains and associated hardening (near ground surface), in contrast to the Wildlife case.

Haddadi and Kawakami (1998a, b, 2000) used the normalized input-output minimization (NIOM) method to model wave propagation and identify dynamic characteristics of subsurface layers at the Port Island downhole array site. Variations of wave amplification and damping were discussed. The stiffness characteristics of the liquefied portion recovered gradually from the time of the main shock of the Hyogoken-Nanbu earthquake to the earthquake of February 18, 1995.

Igarashi (1996) derived an explicit formula to evaluate the factor of safety against liquefaction $F_l$ from energy-related parameters of soil and ground motion, namely, the liquefaction energy amplitude, dynamic friction angle, root mean square shear stress, a bandwidth factor, and effective number of cycles. He applied the formula to the liquefaction at Port Island using the measured downhole accelerations.

Iwasaki and Tai (1996) analyzed downhole array records obtained at Kobe Port Island during the Hyogo-ken Nanbu earthquake. The peak horizontal acceleration was minimum at the ground surface and increased with depth, whereas, the peak vertical acceleration, the peak horizontal velocity, and displacement all decreased with depth.

Kamiyama and Matsukawa (1998) and Kamiyama et al. (1998, 2000) proposed a method for estimating the dynamic variation of stiffness and damping of soils related closely to the nonlinear ground earthquake response. The downhole array records obtained at Port Island during the 1995 Kobe earthquake were used to investigate the capability of the method. They concluded that the method was effective for estimating the nonstationary variation of soil stiffness and damping and could be applied to various phenomena associated with nonlinear seismic behavior.

Kawase et al. (1995, 1996) studied the strong motion records of the Hyogo-ken Nanbu earthquake observed by the downhole array at Port Island. They simulated the observed accelerograms by using 1-D linear, equivalent linear, and effective stress analyses with reasonable soil constants derived from the boring survey.

Kokusho et al. (1994) studied seismic response of deep soil by means of downhole array data with special focus on soil damping evaluated in-situ and in the laboratory. Kokusho et al. (1995) estimated S-wave velocity and damping ratio corresponding to the main shock as well as a small aftershock by an inversion technique. Their study revealed the effect of soil liquefaction and nonlinear soil characteristics on the strong motion amplification mechanism at the site.

Mok et al. (1998) utilized the ground motion recordings at Port Island downhole array to back-calculate the compression-wave velocities ($V_p$) during strong seismic events and to examine the effects of soil damping on vertical site response. They suggested that dynamic soil behavior was related to the degree of saturation.

Nozu and Uwabe (2000) applied the empirical Green’s function (EGF) method to the records obtained at Port Island during the 1995 Hyogo-ken Nanbu earthquake and its aftershocks, to evaluate the nonlinear effect on the mainshock records. Nonlinear ground response analysis showed that amplitude
reduction during reverberation could exist under certain conditions.

Numata et al. (1998, 2000) estimated ground motion behavior of an improved site and an unimproved site during the 1995 Kobe earthquake using downhole array data. A simplified liquefaction evaluation approach was used.

Oka et al. (1996, 2000) studied the nonlinear ground motion amplification, focusing on its time-dependent characteristics due to the decrease of excess pore water pressure after the large event. They applied a frequency-dependent equi-linearized technique, and an effective stress-based liquefaction analysis method. Oka et al. (1997) carried out a three-dimensional effective-stress liquefaction analysis in order to understand the basic behavior of the reclaimed island subjected to liquefaction. The analytical results were found to be in good agreement with the array records.

Sato et al. (1996), Kokusho et al. (1996), Kokusho and Matsumoto (1997, 1998), and Kokusho (1999) analyzed downhole array records corresponding to the 1995 Hyogoken-Nambu earthquake including Port Island with an inversion technique (to estimate S-wave velocity and damping ratio corresponding to the main shock as well as small aftershocks). They showed the effects of soil liquefaction and nonlinear soil characteristics on the peculiar seismic amplification mechanism at this site.

Shiono (1978, 1979) analyzed early downhole array records to identify wave types. They found that the vertical component was composed of vertical incident standing P-waves while the horizontal component was composed of S-waves with the same behavior. The wave types did not change throughout the length of the records although the frequency contents varied significantly with time.

Soeda et al. (1999) studied the nonlinearity and irregularity on strong seismic motions using equivalent linear models. The stiffness degradation due to strong motions was evaluated based on considerations of irregularity of the recorded motions.

Sugito (1995), Furumoto et al. (1999), and Sugito et al. (2000) investigated the nonlinear ground motion amplification using a frequency-dependent equivalent linearization technique. An effective stress-based liquefaction analysis was also conducted.

Tanaka (1996, 2000) discussed the liquefaction damage at Port Island based on the downhole array seismic records. Tanaka (1997) showed how the stiffness of the Port Island ground dropped and recovered during and after the Great Hanshin earthquake of 17 January 1995.

Uetake and Satoh (1998) showed that Port Island downhole array records directly revealed the nonlinearity of soil sediments, and that the effects of nonlinearity on strong motions were not negligible with or without liquefaction. They indicated that downhole array records were useful for estimating underground topography, S-wave velocities, and damping factors.

Using the downhole array records obtained from the 1995 Kobe earthquake, Yanagisawa and Kazuma (1996) estimated average stress-strain relationships in the ground at Kobe Port Island. The stress was directly evaluated from earthquake acceleration records. The strain was evaluated from the relative displacement divided by the distance between adjacent observation points. The hysteretic deformation characteristics were analyzed from the following points of view: (1) Softening of the ground stiffness during the earthquake, (2) Liquefaction process of the reclaimed ground during the main shock, and (3) Nonlinear characteristics during the main shock and the aftershocks.

Yang and Sato (1999, 2000a) and Yang et al. (2000a, b) studied the response of the liquefiable Port Island site. They suggested that the stress-strain response and the build-up of pore pressure were well correlated to the variation of the characteristics of ground motion during the shaking history. Yang and Sato (2000b) presented a mechanism to explain the observed vertical amplification at Port Island in order to understand why the horizontal peak accelerations were reduced, but motions in the vertical direction were significantly amplified at the surface. They showed a mechanism of large amplification caused by incomplete saturation of near-surface layers and suggested that the condition of saturation in the study of vertical site amplification might need to be carefully examined.

Yoshida (2000) conducted effective stress dynamic analysis to evaluate the effect of liquefaction of improved ground at Port Island using the downhole array records. He reported that the existence of the improved ground did not significantly affect the downhole array records. Yoshida et al. (1996) studied the nonlinear behavior of Port Island ground using downhole arrays. It was reported that severe nonlinear behavior occurred in two materials, the Holocene clay and the fill material which was shown to affect the response of the ground by the effective stress analysis. Their back analysis of the vertical array records indicated that nonlinear behavior with shear strain larger than 1% occurred.

Additional studies at Port Island were carried out by Fukushima and Irikura (1997), Kaniyama and Matsukawa (1998), Kazuma (1996), Miwa et al. (2000), and Sugito et al. (1996a, b), Taguchi et al. (1996), Wang et al. (2001), and Yoshida and Iai (1998).

**Chiba, Japan**

The prospect for conducting 3-dimensional (3D) studies using downhole array data is illustrated by the Chiba downhole array (Katayama et al. 1990). This array is located at Chiba Experiment Station of the University of Tokyo, Japan. Ground surface at the array location is essentially flat and the site is dry. Geological material consists of a top loam layer 4-5m in thickness, followed by a 4m thick clayey layer. A sand layer lies under this clayey stratum. Site instrumentation included a dense network of downhole accelerometers, that constitute a system of 9 downhole arrays extending to 40m depth. Extensive data was recorded by the Chiba array (Nagata et al. 1990). Most of these events produced low shaking levels with amplitudes below 0.05g. However, peak ground accelerations of about 0.1g were recorded.
Farjoodi et al. (1983) used finite element interpolations to estimate 3D seismic strains at Chiba. They evaluated the Chiba site seismic strain field using linear finite element shape functions and found the calculated strains were in good agreement with those measured in-situ by displacement and strain instruments.

Ghayamghamian and Kawakami (1996) investigated nonlinear response of the soil by comparing the spectral ratios (uphole/downhole) using weak and strong motions from the Chiba downhole array. Reduction in the predominant frequency of the transfer function with increase in excitation level reflected the nonlinear response of the soil. Ghayamghamian and Kawakami (2000a, b) used strong motion records from the Chiba downhole array to examine soil shear modulus and material damping as a function of shear strain during large earthquakes. Acceleration data from the site were processed directly for evaluation of site shear stress-strain hysteresis curves for different time windows of the record. They illustrated the significance of the site nonlinearity during strong ground motions as well as the accuracy of the dynamic soil characteristics obtained from laboratory tests.

Ikemoto et al. (1999, 2000) performed an inverse analysis of records from the Chiba downhole array. The dynamic soil parameters were identified using horizontal and vertical ground motions in the downhole array records. Applications of the method to actual vertical acceleration records as well as horizontal ones were discussed.

Kurita and Matsui (1997) proposed a formula to estimate the confidence region of identified parameters in Chiba. Their technique was based on the propagation law of errors and the sensitivity of the identified parameters with respect to the model parameter errors.

Sawada et al. (1995) developed a localized identification method to calculate dynamic soil characteristics such as shear wave velocity and quality factor of the specified subsurface layers, using the Chiba downhole array records. Sawada et al. (1998) investigated the frequency dependency of Q-value in subsurface layers, and found that the frequency dependency of the Q-value for Chiba was clearly identified with stable results of shear wave velocity.

Sykora and Bastani (1998) evaluated the influence of soil stiffness and its distribution with depth at Chiba. For this analysis, they used peak horizontal acceleration (PHA) and peak horizontal particle velocity (PHPV) obtained from acceleration time histories measured in the free-field Chiba downhole strong motion array.

Yoshida and Iai (1998) assessed the recorded nonlinear behavior at Chiba. They showed the importance of taking nonlinear behavior into account in predicting site response. They found that equivalent linear analysis was a good approximation when the maximum strain is less than about 0.5%.

Many other researchers also studied the downhole array data at Chiba including Baise and Glaser (2000a, b, c), Haddadi and Kawakami (1998b), Miura et al. (2000), and Tsujihara et al. (1990).

Rokko Island, Japan

The Rokko Island downhole array instruments were installed at depths of 0.0 m, 35.0 m, 98.0 m, and 154.5 m (Oka et al. 2000).

Baise and Glaser (2000c) examined the variability of site response using parametric autoregressive moving-average (ARMA) models between the geologically similar Port Island and the Rokko Island sites, which are separated by approximately 5 km and are both man-made islands in the same sedimentary basin. They found that site response estimated using the downhole array recordings at the two islands were similar. The Port Island site response amplified the ground motion slightly more than observed at Rokko Island, but the Port Island filter could be used to adequately predict the Rokko Island site response.

Sugito et al. (1996a, b) evaluated the orientation error of accelerographs using the data obtained during the South Hyogo earthquake. An error of 22 degrees in the horizontal plane was detected for the sensor installed at 83m depth. The peak acceleration of each time history was found to be significantly affected by the correction of the error (of the order of 60 % larger or smaller than the original peak value). Based on correcting the downhole array records, Furumoto et al. (1999), Sugito et al. (1996b, 2000), and Oka et al. (2000) investigated the nonlinear ground motion amplification using a frequency-dependent equivalent linearization technique for frequency domain analysis, and an effective stress approach for liquefaction analysis. They discussed the time-dependent amplification characteristics of the reclaimed Rokko Island depending on reclamation history.

Ashigara Valley, Japan

The Ashigara Valley downhole instruments were installed at depths of 0, 10, 30, 100, and 467m in Odawara, Japan, by the Earthquake Research Institute, University of Tokyo (Yao 1999).

Sato et al. (1998a, b) investigated Ashigara Valley site amplification using downhole array records corresponding to the 1995 Hyogoken-Nambu earthquake with the inversion technique to estimate S-wave velocity and damping ratio. They found the effects of source radiation, geotechnical structure, and nonlinear and anisotropic soil characteristics on the seismic amplification mechanism at the site.

Sato et al. (1993) pointed out based on an inversion method that the main part of the strong motion had the potential of making...
the surface soil nonlinear, but that the soil returned to the linear range just after the main part. Satoh and Kawase (1995) and Satoh et al. (1999) studied the amplification characteristics of soil sediments by using strong and weak motion downhole records in the Ashigara Valley. They found that the amplification factors for the main part of the weak and strong motions could be simulated by nonlinear one-dimensional site analysis for the alluvial surface layers and the diluvial layers. They also performed two-dimensional finite element soil analysis to simulate the amplification factors. Spectral ratio analysis revealed the nonlinearity of soil during the strong ground shaking. In order to quantify the nonlinear behavior of soil sediments, they identified their S-wave velocities and damping factors by minimizing the residual between the observed spectral ratio and the theoretical amplification factor (calculated from a one-dimensional wave propagation theory).

Yao (1999) investigated near-surface attenuation and site effects using the strong motion downhole array records. He showed that accelerograms near the ground surface have peak amplitudes 3 to 4 times larger than those at the deep downhole areas. Q values for S-waves in sedimentary layers obtained by fitting the theoretical transfer functions (1-D multiple reflection theory of SH-wave) to the observed spectral ratios were very low for the near-surface layers, but showed strong frequency dependence.

Kushiro, Japan

Different acceleration records of the surface motion were observed at two adjacent observation stations during the Kushiro-oki earthquake on January 15, 1993, one of which was set up on the building foundation of the Kushiro Meteorological Observatory (JMA) and the other was on the ground. In order to investigate an amplitude discrepancy between two nearby observation stations, two downhole array stations were set up after the earthquake (Yahata et al. 2000). Analyses of the downhole array data revealed that the Kushiro site behaved nonlinearly during strong earthquakes (Yahata et al. 2000, Yoshida and Iai 1998, Kanda and Motosaka 1997, Uetake and Satoh 1998).

Yahata et al. (2000) carried out analyses by applying one and two-dimensional models considering nonlinear soil behavior for the downhole strong motions during the Hokkaido-Toho-oki earthquake of October 4, 1994. They showed that the different surface motions at two sites were generated by the influence of a topographic cliff during strong seismic excitation.

Iai et al. (1995) reported data from a downhole array at Kushiro Port. The data was employed for calibration of a state-of-the-art computational code.

Hokkaido, Japan

Hayashi et al. (2000) studied liquefaction array observations to measure ground acceleration and pore water pressure during earthquakes simultaneously (on soft ground on the south side of Lake Utunai, Hokkaido, Japan). It was found that the dynamic characteristics of the ground were not affected by the existence of an adjacent road embankment through 1D and 2D equivalent linear analyses.

Hiyoshi, Japan

Kawakami and Haddadi (1998) employed the method of normalized input-output minimization (NIOM) to analyze the ground motion records of three earthquakes detected at the Hiyoshi downhole array in Japan. They presented the amplification characteristics of soil layers at the site.

TKS, KNK, and SGK, Kobe, Japan

Sato et al. (1996), Kokusho et al. (1996), Kokusho and Matsumoto (1997, 1998), and Kokusho (1999) analyzed TKS, KNK, and SGK downhole array records with the inversion technique to estimate S-wave velocity and damping ratio corresponding to the main shock as well as a small aftershock (Kobe, Hyogo-ken Nanbu earthquake). They showed the effects of nonlinear soil characteristics on the peculiar seismic amplification mechanism at these sites. Strain dependent changes in the shear modulus and the damping ratio were back-calculated from the downhole array records.

Shuzenji, Choshi, Tateyama, and Higashimatsuyama, Japan

The four downhole array sites, Shuzenji, Choshi, Tateyama, and Higashimatsuyama at suitable rock sites surrounding Tokyo were equipped with strong-motion instruments (Omote et al. 1980). A large overall array is composed of four local arrays. Each local array comprises a single sub-array in a triangle with 500-m side length and a downhole array. Three-component sensor units were employed. A control and monitoring center located in Tokyo was connected to the sub-arrays via telephone lines. Recorded accelerograms have been analyzed (Omote et al. 1980).

Sunamachi, Japan

Davis and Berrill (2001) studied the dynamic pore pressure recordings obtained from downhole arrays at Sunamachi. The calculated stress and strain wavefields were integrated to produce time histories of dissipated energy density for comparison with measured values. Good agreement between measured and calculated pore pressures was reported.

Etchujima, TTRL, Samukawa, Fujisawa, and Shinfuji, Japan

Ghayamghamian and Kawakami (1996) investigated nonlinear response of the soil by comparing the spectral ratio (uphole/downhole) using weak and strong motions from Etchujima, TTRL, Samukawa, Fujisawa, and Shinfuji downhole arrays. The reduction in predominant frequency in the transfer function with the increase in excitation level reflected nonlinear soil response.

Ghayamghamian and Kawakami (2000a, b) used strong motion records from Etchujima, TTRL, Samukawa, Fujisawa, and Shinfuji downhole arrays to examine soil shear modulus and material damping as a function of shear strain during large

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earthquakes. Acceleration data from the sites were processed directly for evaluation of site shear stress-strain hysteresis curves for different time windows of the records. Additional studies on Etchujima, TTRL, and Shinfuji downhole arrays were carried out by Haddadi and Kawakami (1998a, b) and Yoshida and Iai (1998).

Sendai, Japan

Satoh et al. (1995) evaluated local site effects due to surface layers overlying bedrock using one-dimensional soil models. They identified S-wave velocities and frequency-dependent quality factors Q from amplification factors between surface records and downhole records observed at 10 sites at depths of several tens of meters. They estimated the so-called engineering bedrock waves from downhole records by using 1D models with identified soil constants. Uetake and Satoh (1998) showed that Sendai downhole array records directly revealed the nonlinearity of soil sediments.

Tomioka, Japan

The downhole arrays at a site in Tomioka city were installed in the vertical direction from bedrock to ground surface in Fukushima Prefecture, Japan. Ghaeampahian and Kawakami (1996, 2000b) investigated nonlinear response of the soil by comparing the spectral ratios (uphole/downhole) using weak and strong motions from the Tomioka downhole arrays in Japan. Reduction in the predominant frequency with the increase in excitation level was noted as evidence of nonlinear soil response.

Iwata et al. (1992) compared the downhole array records obtained from 0 m to about 1 km in depth at Tomioka. Synthetic seismograms were computed using the full-wave theory for a multilayered structural model with a deterministic source. Uetake and Satoh (1998) studied the Tomioka downhole array records as well.

Fuchu, Japan

Kinoshita (1999) studied the Fuchu site effects using a stochastic method for investigations by means of downhole array observation. The wave transfer function for SH waves, the separation of upcoming and downgoing waves and the effect of fundamental mode Love waves on a surface seismogram were investigated without any knowledge of the velocity structure beneath the site.

Westa, Japan

Power et al. (1986) studied the downhole array data from free-field ground motion in Westa and found that both peak accelerations and response spectra of ground motions decrease significantly with depth in the range of typical embedment of nuclear power plant structures and other buildings. They concluded that appropriate variations of ground motion with depth should be included in carrying out soil-structure interaction analyses and characterizing foundation input motions for embedded structures.

Yamada (1990) suggested that generation of local surface waves with multiple horizontal reflection between underground boundaries might account for periodic seismic pulses in the coda part of seismic waves by studying the records from the Wesda downhole array, Tokyo. The effect of repetition of seismic pulses on artificial structures was examined.

Shizuoka, Japan

Nozawa et al. (1989) investigated the strain dependence of soil characteristics by interpolating the strong-motion accelerograms recorded at the Shizuoka downhole array. They indicated that the accelerograms recorded by the downhole array with peak horizontal ground acceleration exceeding 400 gal were affected by saturation.

Iwaki, Japan

The downhole arrays at a site in Iwaki city are installed from bedrock to the ground surface in Fukushima Prefecture, Japan. Sugawara et al. (1991) investigated the characteristics of vertical earthquake ground motions. The relationships between wave motions of vertical and horizontal (radial) components, wave propagation aspects of vertical components from bedrock to ground surface, and amplification functions of vertical components through soil deposits were examined.

Yokohama, Japan

The Yokohama downhole array stations are located in and around Yokohama, the southwestern part of Tokyo metropolitan area. Stations are located within 20x20 km². All stations have P- and S-wave profiles by elastic wave explorations. The deepest accelerographs were installed in soft rock with S-wave velocity of about 600 m/s at depth between 75 and 100m. Efforts were carried out to identify S-wave and Qs profiles. With the identified profiles, site amplification characteristics can be removed and incident spectra from bedrock can be evaluated (Toshinawa et al. 2000).

Kanto, Japan

The Kanto downhole array sites are located around the Tokyo gulf in the Kanto district, Japan. Kurita et al. (2000) used the downhole array records to evaluate local site effects. Local site effect was calculated as the geometric average of the acceleration response spectral ratios between the estimated values from an attenuation relation and observed seismic ground motions. The amplification characteristics of surface geology were considered as the theoretical transfer function. A theoretical transfer function between the wave at ground surface and incident wave at the base layer was calculated from the optimum ground structure model. The optimum ground structure models were identified from vertical array observation records.

Tokyo International Airport, Japan

A horizontal seismometer array having six observation points along a straight line of 2500m length has been established at the
Tokyo International Airport. Each observation point is equipped with two horizontal seismometers. Downhole seismometer arrays have also been established at two points, one at an end of the observation line and the other at a point 500m inside the other end of the line. The observations started in April 1974 and 28 earthquakes had been recorded as of June 1975. Correlations among the ground motions at the points on the ground surface and the two points in the ground where the downhole seismometers were installed have been studied (Tsuchida and Kurata 1975, Tsuchida and Kurata 1976, Tsuchida et al. 1977, Hayashi et al. 1977, Tsuchida and Iai 1978).

In 1988, a three-dimensional seismic array was deployed at the Tokyo International Airport, Japan. The array extends 2550 meters along a runway in one wing accommodating six observation stations, with another wing extending normal to the runway about 500 meters and accommodating three observation stations. Each observation station has four downhole seismometers down to a depth of about 80 meters with three pore water pressure meters. Phase velocities and directions of incidence to the base of the ground at a depth of 80 meters were obtained from the seismic array records (Iai et al. 1994).

Lotung, Taiwan

The U.S. Electric Power Research Institute in cooperation with the Taiwan Power Company conducted a Large-Scale Seismic Test (LSST) at a site near Lotung (Tang 1987). Two models (1/4-scale and 1/12-scale) of a nuclear-plant containment structure were constructed on a flat and vast alluvium basin (in the vicinity of seismically active faults). Soil at this site consists predominantly of interlayered silty-sand and sandy-silt, and ground-water level was at or within 1m of ground surface. Extensive instrumentation was deployed to record both structural and ground seismic responses. The ground instrumentation included three linear surface arrays (arms 1, 2, and 3), and two downhole arrays (DHA and DHB) that extended to a depth of 47m below ground surface (Tang 1987). Eighteen earthquakes were recorded during the period 1985-1986 (Tang 1987). This wealth of data constituted a basis for a number of valuable research efforts. Numerous analysis techniques and ground response models have been developed to identify the soil dynamic parameters and evaluate the Lotung site seismic performance.

Baise and Glaser (2000a) identified the empirical Green's function from depth to the surface at Lotung. They suggested that consistency of site response over a wide range of shaking levels indicated an "effective" range of linear soil behavior for PGA up to 0.21 g at Lotung. Baise and Glaser (2000c) compared the Lotung site responses with Chiba and found that they were dissimilar although the geologic profile showed consistencies.

Beresnev and Wen (1995) examined the nonlinearity in P-wave amplification using the strong-motion data from a downhole array at the Lotung site where a significant nonlinear response to S waves had been found. They analyzed soil transfer functions and found that the response remained linear in the shear strain range of $7 \times 10^{-5}$. Beresnev and Wen (1996a) examined the relationship of spectral ratios between soft soil and reference rock sites. They suggested that the soil-to-rock spectral ratios could be considered reliable estimates of the real site response. Beresnev and Wen (1996b) indicated that nonlinear site effects were more common than previously recognized in strong-motion seismology.

Borja et al. (1999a) showed that soil structure interaction effects were partly responsible for the reduced peak north-south ground surface acceleration recorded by a downhole array near the containment structure. Lin and Borja (2000) noticed the pore pressure records in Lotung were erratic and suggested the presence of artesian conditions during the LSST12 and LSST16 seismic events. They proposed the possibility of double drainage (i.e., water was draining not only to the top but also to a sand layer at a depth between 8-9 meters, just above the clayey silt layer). A coupled analysis using a u-w-p finite element formulation was employed. They argued that the moduli and damping ratio curves developed by Zeghal et al. (1995) tended to underpredict the Arias intensities of the ground motion.

Borja et al. (1999b, 2000) used a nonlinear FE model (accounting for soil column effects) to examine the nonlinear ground response at the LSST site in Lotung. They showed that the recorded downhole motion was dominated by nonlinear response. They suggested that it was not necessary to have a complete damping ratio-shear strain curve to model nonlinear ground response (the zero shear-strain damping ratio asymptote was sufficient to obtain a measure of true viscous damping, and the remainder of the damping ratio curve was redundant information).

Chang et al. (1989) found evidence of significant nonlinear soil response for peak accelerations greater than about 0.15g at the ground surface using the downhole ground motion data. Effective shear moduli or shear-wave velocities were found to decrease as the level of shaking increased. Chang et al. (1990) evaluated the equivalent linear and nonlinear analysis techniques for modelling nonlinear soil behavior using downhole ground motion data from Lotung. They compared computed motions with recorded motions. Both deconvolution and upward wave propagation analyses were performed using equivalent linear techniques (only upward wave propagation analyses were performed for the nonlinear analysis technique). Chang et al. (1994) analyzed vertical motions recorded at the Lotung downhole array in Taiwan during three large events having magnitudes ranging from 6.2 to 7.0. The recorded motions were first used to examine any change in locations of peak Fourier spectral ratios between motions at the ground surface and at depth for different events having various levels of excitation. Using the frequencies of the peak Fourier spectral ratios, values of P wave velocity for the soil profile were estimated and compared with the geophysical measurements.

Chang et al. (1996) evaluated equivalent-linear dynamic shear moduli from the recorded downhole earthquake accelerations. The identified reduction in shear modulus, as a function of effective shear strain, was shown to be in agreement with laboratory test data, and provided evidence of nonlinear soil
behavior during earthquake excitation. They evaluated the accuracy of dynamic characteristics obtained from geophysical measurements and laboratory tests.

Chen and Chen (1989) presented seismic wave velocity analysis using the acceleration time-histories recorded at the Lotung downhole array. The frequencies of maximum amplification of soil layers were identified using the spectral ratios between the surface and downhole accelerograms. They proposed an elastic 1-D wave propagation model to estimate the ground seismic wave velocities. They indicated that significant reduction of shear wave velocity as well as soil modulus was induced by earthquake excitation as compared to the values obtained from low strain level soil tests.

Davis and Berrill (2001) studied the dynamic response and pore pressure recordings obtained from downhole arrays at Lotung. The calculated stress and strain wave fields were integrated to produce time histories of dissipated energy density for comparison with measured values. Good agreement between measured and calculated pore pressures was reported. Shen et al. (1991) also evaluated excess pore pressures during three different LSST earthquakes.

Glaser (1995), Glaser and Baise (2000), and Ching and Glaser (2001) estimated soil characteristics at the Lotung downhole array site with a mapping to a lumped mass model using both time-invariant and time-variant parametric modeling methods (system identification). They found that the estimated shear stiffness ranged between 0.5 and 6 MN/m, inversely proportional to PGA, and the estimated equivalent viscous damping ratio varied from 2% to 30% of critical damping (in proportion to PGA). They noticed the degradation of soil behavior, while less pronounced with increasing depth, consistently occurred above a peak input acceleration of 0.07g.

Huang et al. (2000) studied the linear and nonlinear behavior of soft soil layers. Spectral analyses showed that the strong motion caused peaks in spectral ratios to shift to lower predominant frequencies. Compared to the weak motions, the strong motions also decreased the amplification factor. The maximum reduction in amplification reached 50%. A waveform simulation showed that the linear model based on the Haskell method could predict well the weak motions (PGA < 60 gals) at various depths. This model was not good for the strong motion data (PGA > 150 gals). They showed that the nonlinear numerical scheme could significantly improve the simulation results although PGA at the surface station was still underestimated.

Li et al. (1998) used two major earthquakes at Lotung within a 4-month period to perform numerical analyses with a fully coupled, multidirectional dynamic procedure incorporating a bounding surface hypo-plasticity model. The calculated responses for the two different events showed equally reasonable agreements with the field records in both 3D acceleration and pore-pressure time histories.

Wen et al. (1994, 1995b) studied the nonlinear seismic response at Lotung by comparing the spectral ratios of surface to downhole horizontal accelerations during weak and strong motion. The dependence of soil amplification on the amplitude of input excitation was established. Wen (1994) investigated the issues of linear versus nonlinear soil response and isotropy versus anisotropy at the Lotung site.

Mok et al. (1998) utilized the ground motion recordings at Lotung to back-calculate the compression-wave velocities ($V_p$) during strong seismic events and to examine the effects of soil damping on vertical site response. They suggested that dynamic soil behavior was related to the degree of saturation.

Shen et al. (1991), Elgamal et al. (1996a) and Zeghal et al. (1996b) employed the Lotung downhole seismic records to calibrate linear and nonlinear computational site response models and showed downhole records to be a valuable means of advancing the state-of-the-art in site amplification computations.

Zeghal and Elgamal (1993), Zeghal et al. (1995), Elgamal et al. (1995), Elgamal et al. (1996a), and Elgamal et al. (1997) performed cross correlation and spectral analyses of the recorded Lotung downhole accelerations to evaluate shear wave propagation characteristics, variation of shear wave velocity with depth, and site resonant frequencies and modal configurations. They found that a shear-beam model, calibrated by the identified site characteristics can represent the site dynamic response characteristics over a wide frequency range. They utilized strong-motion earthquake records to investigate the Lotung site large-strain soil response characteristics. A shear stress strain imaging technique was developed to evaluate soil shear stress-strain histories directly from the free-field downhole accelerations. From these histories, they found two salient response features: (1) stiffness reduction due to the increase in shear strain amplitude and (2) stiffness reduction due to pore pressure buildup. The identified stiffness and damping characteristics provided quantitative evidence of nonlinear site amplification during strong-motion earthquakes.

Many other researchers also conducted studies on the Lotung downhole array including Lin (1994), Loh and Yeh (1992), Sykora and Bastani (1998), and Yoshida and Iai (1998).

Hualien, Taiwan

Hualien is located south of Lotung on the east coast of Taiwan within a highly active seismic zone. The Hualien LSST was initiated in 1993 by an international consortium of industrial and research enterprises from five countries (Japan, USA, China, France and Korea). In contrast to the Lotung soft soil conditions, the Hualien experiment was located at a relatively stiff site (Tang et al. 1991, Graves et al. 1996). Extensive instrumentation was deployed to record seismic structural and ground responses, and to monitor soil pore-water pressure buildup. The ground instrumentation (installed around a one-quarter scale nuclear-plant containment structure) included fifteen surface accelerometer stations, and three downhole accelerometer arrays. Each downhole array consisted of 5 accelerometers and extended to a 52.6m depth. During the period July 1993 to May 1995, seven earthquakes were recorded at the Hualien site (Yang et al. 1998).
excitation. Actually, motions from the recent 1999 Ji-Ji Taiwan earthquake were recorded by this array (C. Chen and H. Graves, personal communication 2000).


Based on correlation analyses and identified seismic stress-strain histories, Chen and Chiu (1998) employed the free-field motions of six earthquakes to identify the predominant frequencies of soil layers and estimate the shear wave velocities. They found that the soils at the Hualien LSST site were anisotropic such that the earthquake responses exhibited different characteristics in two horizontal principal directions. Chen and Chiu (1999) presented the results of ground response analyses by using the free-field motions recorded at the site during the earthquake of May 1, 1995.

Gunturi et al. (1998) found the shear wave velocity profile estimated in the EW direction to be lower than in the NS direction. They showed that this anisotropic soil response had been manifested during earlier in-situ experimental studies of foundation impedance functions (de Barros and Luco 1995). The identified linear soil characteristics adequately represented the recorded seismic site response at small strains.

Pires and Higgins (1998) found that the site exhibited cross-anisotropy while studying the soil-structure interaction response observed under dynamic lateral load testing. Similar conclusions were reported by de Barros and Luco (1995).

Wuku, Taiwan

This downhole array was installed in a sewage disposal plant in the Wuku Industrial Area at the end of 1993. It is a sub-project of “An Integrated Survey of the Subsurface Geology and Engineering Environment of the Taipei basin” initiated in 1991. The array is located at the western part of the Taipei Basin. It consists of accelerometers, a GPS timing system and a digital recording system. The main purpose of this downhole array project is to study the basin effects on seismic waves (Wen et al. 1995a, Wen and Peng 1998). Wen et al. (1995a) conducted travel time analysis of seismic waves and calculated $V_p$ and $V_s$ for the soil layer from surface to the depth of 352 meters using weak motions records. By comparing the peak ground motions at different depths with ground surface motions they found that the main amplification existed in the top 60 meters of the soft soil layer and that no nonlinear amplification effect occurred in the weak motion data set. They obtained an equation representing the variation of horizontal peak ground acceleration with respect to depth using regression analysis. They also performed ground response analysis in the frequency domain.

Dahan, Taiwan

The Dahan downhole array was installed at depths of 0, 50, 100 and 200 m. Chiu et al. (1994) showed that the polarization of later arrivals could be used to estimate the orientation error and the tilt of the downhole accelerometers in the Dahan downhole strong-motion array. Huang and Chiu (1996) used Four-level Dahan downhole recordings to investigate site amplification due to the near-surface structure. They selected eight sets of data presenting well-separated incident and reflected waves (from the free surface) to study site amplification. They also tested five techniques of spectral ratios to determine the most reliable way to estimate site response.

Mexico City, Mexico

The Central de Abasto Oficinas (CAO) downhole arrays in Mexico and related analyses have been reported by Taboada et al. (1999a, b, 2000).

The valley of Mexico City is frequently affected by strong ground motions due to a combination of high rate subduction-related seismicity at regional distances, and unique soft clay site material conditions. The CAO site is located within the central lacustrine deposit zone of the valley, close to the downtown of Mexico City. Geological material of the site consists of quaternary soft clayey and silty soil over a partially cemented gravel and sandy alluvial stratum. The clayey deposit is 40-50m thick and it contains not only important fractions of silt, but also some thin layers of fine sand and volcanic glass at various depths. It may be observed that water content varies from 50 to 200% within the top 10 meters; and reaches 350 to 450% between 10m and 40m. The CAO downhole acceleration array is located near the central part of the valley. The downhole array of the site includes one superficial accelerometer and three more located at 12m, 30m and 60m below ground surface (Taboada et al. 2000).

Faccioli et al. (1996) used data from the Jalapa building in the soft clay zone of Mexico City, equipped with a local array of 14 digital accelerographs (including one free-field and two downhole instruments at 20 m and 45 m depth), to study salient aspects of site response and soil-structure interaction (SSI) at PGA levels in free-field up to about 0.05 g. The analyses were focused on the interpretation of the peak amplitude reduction of spectral ratios between surface and intermediate depth downhole recordings and on the influence of surface waves on ground motion.

Taboada et al. (1999a, b, 2000) used the CAO downhole array records from the 3/31/93, 24/10/93, 23/05/94, 10/12/94 and 09/10/95 seismic events to investigate the soil stiffness and damping ratio as a function of shear strain amplitude. The shear stress-strain histories were evaluated directly from the downhole array records, employing a technique of system identification. These histories were used to obtain the variation of shear modulus and damping ratio with shear strain amplitude. A shear-beam model, calibrated by the identified properties, was found to represent the site dynamic response characteristics.
Tangshan, Mainland China

The Tangshan downhole array is located at Xiangtang town in Tangshan city where a destructive earthquake of $M_L = 7.8$ occurred in 1976 (Xie et al. 1999). This is a three dimensional strong-motion array instrumented in 1994. It consists of five digital strong-motion accelerographs with two on a surface rock site and the other three on a soil site with different depths in the downholes. The array was designed to study local soil effects on seismic ground motion. A number of strong motion accelerograms have been recorded from 60 earthquakes of magnitudes $M_L = 1.5$ to $M_L = 5.9$ and the maximum peak ground acceleration reached up to 60 cm/sec² (Xie et al. 1999).

Thessaloniki, Greece

Raptakis et al. (1998a, b) employed an accelerogram data set recorded at the Euroseistest array in the Mygdonia graben (Lake Volvi area) near Thessaloniki, Greece, during the period April 1994 to June 1996 to study the effectiveness of the standard spectral ratio and the horizontal-to-vertical spectral ratio techniques in investigating and quantifying the influence of geological conditions on strong ground motion. They showed that the horizontal-to-vertical spectral ratio technique could reveal the fundamental resonant frequency of alluvial deposits by using only a single strong motion station. Raptakis et al. (1998a, b) also examined the probable influence of the dominant soil formation, composed of stiff red clay, on the wavefield produced by either weak or strong earthquakes. They noted that this soil formation, having surprisingly high shear wave velocity and a low-quality factor, acts like a protection shield.

Centrifuge-Testing Downhole Arrays

Centrifuge modeling is a very useful technique for simulation of the effects of earthquakes on geotechnical structures (Kutter et al. 1994, Kutter 1995). The basic principle is to produce stresses, strength, and stiffness in a small-scale model that are similar to those in a prototype. A number of centrifuge model tests instrumented with downhole arrays and corresponding analyses have been carried out to study seismic ground response, seismic soil-structure interaction, and liquefaction, among other problems (e.g., Balakrishnan and Kutter 1999, Dobry and Abdoun 1998, Elgamal et al. 1996b, Elgamal and Yang 2000, Fiegel and Kutter 1992, Kutter and Balakrishnan 2000, Taboada and Dobry 1998, and Wang et al. 1998).

At the Rensselaer Polytechnic Institute (RPI) Centrifuge Facility, downhole acceleration arrays, embedded pore-pressure transducers, and LVDTs along the model height (to measure lateral displacements in laminar container experiments) have provided a wealth of information on liquefaction, associated lateral deformations, and related soil-pile interaction phenomena. This work is detailed in a number of Ph.D. theses (R. Dobry, academic advisor) including those of Liu (1992), Taboada (1995), and Abdoun (1997).

Elgamal et al. (1996b) used centrifuge downhole arrays to study dynamically induced liquefaction of a saturated loose sand stratum. The experiments were conducted at Rensselaer Polytechnic Institute (Taboada and Dobry 1998) and California Institute of Technology (Scott et al. 1993). The observed acceleration, pore pressure and lateral displacement responses showed a high level of consistency, and were used to estimate the corresponding dynamic shear stress-strain histories. Elgamal and Yang (2000) used centrifuge downhole arrays to calibrate a liquefaction constitutive model. They found that there was significant difference in soil response between level-ground and sloping sites, due to the strong influence of soil dilatancy, an important mechanism in possibly limiting soil lateral spreading during liquefaction. Such dilatancy effects were also documented by many other centrifuge experiments instrumented with downhole arrays (Fiegel and Kutter 1992, Taboada and Dobry 1998, Dobry and Abdoun 1998, and Balakrishnan and Kutter 1999).

Fiegel and Kutter (1994a) presented centrifuge model tests with downhole arrays for studying the mechanism of liquefaction in layered soils. Fiegel and Kutter (1994b) used centrifuge downhole arrays to evaluate liquefaction induced lateral spreading of mildly sloping ground.

In centrifuge model tests and field case histories, the soil stiffening due to dilatancy has been observed to be accompanied by large magnitude, short duration spikes in acceleration time histories (Kutter and Wilson 1999). Kutter and Wilson (1999) proposed de-liquefaction as a term to describe the solidification due to dilatancy. By studying data from dense downhole accelerometer arrays in their model tests, slow traveling (about 5 to 30 m/s) but spatially sharp (0.1m wavelength or less) de-liquefaction wave-fronts were observed.

Wang et al. (1998) analyzed downhole array centrifuge test data to evaluate the beam on nonlinear Winkler foundation (BNWF) model (soil-pile analysis). Placing the linear viscous dashpots (representing radiation damping in the far-field) in series with the hysteretic component of p-y elements (representing nonlinear soil-pile response in the near-field), was found to be technically preferable to a parallel arrangement of the viscous and hysteretic damping components.

In a pile-soil interaction investigation, Wilson et al. (1998) described the signal processing procedures when conducting centrifuge model tests instrumented with downhole arrays. Boulanger et al. (1999) also used centrifuge downhole arrays to study seismic soil-pile structure interaction. In addition, Wilson et al. (2000) used centrifuge downhole arrays to study seismic lateral resistance of liquefying sand.

The above is only a brief summary of related research efforts at RPI and UC Davis. Obviously, many other seismic/dynamic centrifuge data sets have been generated worldwide (with much data lately from Japan). The intent of this section is only to draw attention to the large role of centrifuge-testing downhole arrays, particularly in liquefaction studies.

SOAP-6
SOIL PROPERTIES IDENTIFIED FROM DOWNHOLE RECORDINGS

In this section, the following studies will be discussed:
1) Lateral and vertical response at Hualien,
2) Lateral and vertical response at Lotung, and
3) Lateral vibration of dense sand in a UC Davis centrifuge experiment.

Site Response Analysis and System Identification Technique

The employed procedure is based on the following steps (Gunturi 1996):
1. All site response analyses were conducted using the computer program SHAKE (Schnabel et al. 1972). In the conducted investigations, the effective strain was taken as 0.5 of the maximum strain.
2. Total motion was prescribed at the shear beam base as the accelerations actually recorded at this location.
3. Linear soil stiffness properties were defined by the following relationship: $v = v_0 + v_H \left( \frac{z}{H} \right)^b$ in which $v_0$ is wave velocity at ground surface, $(v_0 + v_H)$ is wave velocity at the base, $z$ is depth coordinate, $H$ is beam height, and $0 \leq b \leq 1.0$ dictates a smooth variation of $v$ with depth.
4. A hyperbolic relationship (Kondner 1963) with a reference strain parameter $\gamma_m$ was used to define the modulus reduction with strain amplitude.
5. Damping ratio was composed of a constant strain-independent term and a strain-dependent term derived from another hyperbolic relation (with a reference strain parameter $\gamma_d$).
6. SNOPT, a numerical optimization code (Gill et al. 1997) was employed (Zeghal 1990) to minimize the difference between the acceleration response spectra (ARS, with 5% damping) of the SHAKE results and the ARS of downhole records.
7. For each case, a weighted ensemble of available recorded motions was used simultaneously in the minimization process. Soil modeling parameters were defined such that an optimal match was achieved for the entire ensemble.
8. Once the optimal soil properties were defined, a so-called indifference interval (Bard 1974) for a given modeling parameter may be sought. In this section, variability in a parameter was defined such that additional incurred difference (between computed and recorded acceleration response spectra) was within 10% of the minimized difference, for each individual earthquake in the ensemble.

Generally, the reported results may be considered as a first attempt in defining modeling parameters (rather than actual soil properties). More refined system identification techniques (and the availability of additional earthquake records) will continue to improve the quality of the calculated results.

Results for stiff as well as soft soil sites are reported below. In addition, lateral as well as vertical vibration is addressed each separately using the one-dimensional code SHAKE as explained above. In some cases, SHAKE was used to conduct a linear elastic analysis (when only small tremors are available), in which damping ratio and shear modulus were both strain-level independent.

Model Properties from Lateral Excitation Records

Hualien Site

The Hualien site has demonstrated an azimuthal anisotropy as reported earlier by a large number of investigators. This section highlights this phenomenon and defines a range of optimal damping values for the site.

The Hualien site was modeled as a 52.6m shear beam, and the total motion at the 52.6m deep was defined as the actual recorded acceleration at this depth. Data of the DH2 free-field vertical array was used for the events of Feb 23, 95, May 1, 95 and May 2, 95 (Fig. 1, Table 6). The original data were corrected with a high pass filter (0.1Hz) and a low pass filter (20Hz) to remove any noise in the signal. The horizontal peak ground surface accelerations among the three events ranged from 0.035g to 0.126g in amplitude, representing predominately linear response (for the purpose of our study). As such, only a linear material model was used for this identification process.

The analysis strongly suggested an azimuthal anisotropy at this site. Figure 4 illustrates the difference of the optimized shear wave velocity profiles in the longitudinal and transverse directions and the associated 10% indifference interval. However, no marked difference in damping and its 10% indifference was observed between the two horizontal directions (Fig. 5). For the effective strain range shown in the damping figure, it is seen that the optimized damping is rather high in comparison to earlier reported laboratory test results. In this figure, the range of optimized damping ratios was between 7% and 10%. As mentioned earlier, peak recorded accelerations were in the range of 0.035g to 0.126g.

The optimized properties were used to reproduce the site response of each earthquake by exciting the 52.6m shear beam at the base. Figures 6-13 show representative results comparing recorded and computed acceleration time histories and ARS (with 5% damping).
Gunturi et al. 1998 (Long. & Trans. respectively)

In-situ measurements:

- Gunturi et al. 1998 (Long. & Trans. respectively)
- CRIEPI, 1991
- IES
- IES & Penetration Test reported by de Barros and Luco (1995)

*Figure 4: Optimized Shear Wave Velocity Profiles and 10% Indifference at the Hualien DH2 Array.*
Figure 5: Optimized Damping and 10% Indifference Region at Hualien.
Figure 6: Recorded and Computed Longitudinal Accelerations during Haulout Feb. 23, 95.

Figure 7: Recorded and Computed Longitudinal ARS during Haulout Feb. 23, 95.
Figure 8: Recorded and Computed Longitudinal Accelerations during Hualien May 02, 95.

Figure 9: Recorded and Computed Longitudinal ARS during Hualien May 02, 95.
Figure 11: Recorded and Computed Transverse ARS during Haulen Feb 23, 95.

Figure 10: Recorded and Computed Transverse Accelerations during Haulen Feb 23, 95.
Figure 12: Recorded and Computed Transverse Accelerations during Hualien May 01, 95.

Figure 13: Recorded and Computed Transverse ARS during Hualien May 01, 95.
The Lotung site (Fig. 2) was modeled as a 17m one-dimensional shear beam, with the total motion at the 17m depth defined as the actual recorded acceleration at this depth. Out of the 18 earthquakes listed in Table 7, acceleration recordings at DHB of six events (LSST 6, 7, 8, 13, 16, 17) in two horizontal directions were selected for identification of soil shear wave properties. The horizontal peak ground surface accelerations among the selected six events ranged from 0.03g to 0.21g in amplitude covering linear response as well as moderate nonlinear response. Figures 14–16 show the optimized properties for shear wave velocity profile, shear modulus and damping versus shear strain level applicable to all six events. In the Lotung case, the stiffness profile is in agreement with geophysical measurements. In addition, the EW and NS properties showed no appreciable difference (no azimuthal anisotropy). Shear modulus variation with strain amplitude is also in good agreement with laboratory testing results. However, the lowest optimized damping ratio was about 5%. At higher strains, damping follows and exceeds published upper-bound estimates. It is noted here that the lowest damping ratio of 5% produced a satisfactory match for minor shaking events (of as little as 0.03g in peak acceleration). The optimized properties were used to reproduce the site response of each earthquake by exciting the 17m shear beam at the base. Figures 17–28 show some representative results comparing recorded and computed acceleration time histories and ARS (with 5% damping). As shown in Figs. 23 and 27, only the first 23 seconds of LSST16 excitation were studied (in order to avoid the effect of pore-pressure buildup during the remainder of this event which lasted for about 56 seconds, Elgamal et al. 1997).
Figure 15: Optimized Shear Modulus Reduction at Lotung.

Figure 16: Optimized Shear Wave Related Damping and 10% Indifference Region at Lotung.
Figure 19: Recorded and Computed EW Accelerations during LSST7.

Figure 20: Recorded and Computed EW ARS during LSST7.
Figure 23: Recorded and Computed EW Accelerations during LSST16.

Figure 24: Recorded and Computed EW ARS during LSST16.
Figure 25: Recorded and Computed NS Accelerations during LSST7.

Figure 26: Recorded and Computed NS ARS during LSST7.
Figure 27: Recorded and Computed NS Accelerations during LSST16.

Figure 28: Recorded and Computed NS ARS during LSST16.
Model Properties identified from UCD DKS02 Centrifuge Experiment (Stevens et al. 1999, 2001)

The DKS02 Flexible Shear Beam (FSB) container in the longitudinal direction was modeled as a one-dimensional shear beam down to accelerometer location A17 instead of the actual aluminium container base (Fig. 3). This modification was made in order to avoid potential relative slip movement between the container base (accelerometer) and the overlying soil. As such, the experiments conducted at centrifugal acceleration levels of 10g, 20g and 40g represent a prototype site of 4.75m-, 9.5m- and 19m-depth respectively (Lai et al. 1999). All the earthquake-like shaking events listed in Table 8, covering linear response as well as highly nonlinear response, were utilized for the identification of the dense sand properties (http://cgm. engr.ucdavis.edu/research/projects/dks/). Evidence of 3D effects was noted at higher frequencies, and frequency windows (Hz) of [0, 11], [0, 6.5] and [0, 4] were applied to all recorded motions under centrifugal accelerations of 10g, 20g and 40g respectively (to minimize those effects on the conducted analyses). Figures 29-31 show the optimized properties for the dense sand (Nevada sand with Dr of about 100%) shear wave velocity profile, shear modulus and damping ratio versus shear strain level (using all shaking events simultaneously). It is noted that the optimized stiffness profile is somewhat lower than expected. However, modulus variation is found to track the upper bound of published data. The unexpected surprise appears in the optimized damping, which exceeds upper bound values. Damping in this case did not fall below 7% even for the smallest recorded tremors. Professor Ken Stokoe of the University of Texas has indicated (Personal Communication 2000) that damping ratio might show some increase with increase in excitation frequency. This frequency dependence of damping ratio might be partially responsible for the larger optimized damping values in the current study, since the centrifuge soil system resonances occurred at very high frequencies (>70Hz).

The optimized properties were used to reproduce the site response during each shaking experiment by exciting the shear beam at A17 with its recorded motion. Figures 32-43 show selected comparisons of recorded and computed acceleration time histories and ARS (with 5% damping), one small and one large shaking event at each of the employed centrifugal acceleration levels. In general, good agreement is noted between recorded and optimized response.
Figure 30: Optimized Shear Modulus Reduction for Stiff Sand (DKS02).

Figure 31: Optimized Shear Damping Value and 10% Indifference for Stiff Sand (DKS02).
Figure 32: Filtered Recorded and Computed Accelarations of DXS02-c (10g).

Figure 33: Filtered Recorded and Computed ARS of DXS02-c (10g).
Figure 34: Filtered Recorded and Computed Accelerations of DKS02_ah (10g).

Figure 35: Filtered Recorded and Computed ARS of DKS02_ah (10g).
Figure 36: Filtered Recorded and Computed Accelerations of DKS02-n (20g).

Figure 37: Filtered Recorded and Computed ARS of DKS02-n (20g).
Figure 38: Filtered Recorded and Computed Accelerations of DKS02_bb (20g).

Figure 39: Filtered Recorded and Computed ARS of DKS02_bb (20g).
Figure 41: Filtered Recorded and Computed ARS of DKS02_u (40g).

Figure 40: Filtered Recorded and Computed Accelerations of DKS02_u (40g).
Figure 42: Filtered Recorded and Computed Accelerations of DKS02_by (40g).

Figure 43: Filtered Recorded and Computed ARS of DKS02_by (40g).
Model Properties for Vertical Seismic Excitation

Hualien Site

Recordings of DH2 during three events (Feb. 23, 95, May 1, 95 and May 2, 95) (Fig. 1 and Table 6) were selected for identification of model properties under vertical excitation conditions. The vertical peak ground surface accelerations are less than 0.023g in amplitude, indicating a predominately linear response. The site was modeled as a 52.6m one-dimensional (1D) column as described earlier. Figure 44 shows the optimized properties for vertical motion in terms of stiffness profile. In this case, the optimization process was inconclusive. Two different stiffness profiles were found to yield nearly equal minimal error. Quality of the data may be questioned since the peak accelerations are quite small (among other reasons). However, in both cases, the optimized damping ratio was extremely large. Damping in this case was in the range of 14% to 25%. This outcome suggests that the underlying physical mechanisms are not well understood, and calls the validity of the employed 1D modeling procedure in question.

Nevertheless, the optimized properties were used to reproduce the site response of each earthquake. Figures 45 and 46 show a comparison of recorded and computed acceleration time histories and ARS (with 5% damping). Similar quality matches were observed for the other two shaking events as well. In summary, further studies are needed before conclusion can be drawn.

![Optimized Vertical Motion Velocity Profile at Hualien](image-url)
Figure 45: Recorded and Computed Vertical Accelerations of Hualien Feb. 23, 95.

Figure 46: Recorded and Computed Vertical ARS during Hualien Feb. 23, 95.
Lotung Site

Recordings of DHB (Fig. 2 and Table 7) of the same six events (LSST 6, 7, 8, 13, 16, 17) were selected for identification of model properties for vertical excitation. The vertical peak ground surface accelerations are less than 0.10g in amplitude, suggesting a predominately linear response. The site was modeled as a 17m 1D column as described earlier. The site response was simulated with the recorded total vertical motion at 17m depth specified at this location. Figure 47 shows the optimized stiffness profile for vertical excitation. In this case, the optimization process consistently produced a stiffness profile that increased gradually with depth. However, the upper bound of this profile is about 25% lower than reported from in-situ measurements (Tang 1987). At Lotung, damping for vertical excitation was also high (optimal value of 18% with an indifference range of 14% - 19%). Overall, the combined findings about damping from Hualien and Lotung suggest that large damping ratios are needed when a 1D model is to be used for vertical vibration analyses (14% as a lower limit).

The optimized model properties were used to reproduce the site response for each earthquake. Figures 48-53 show comparisons of recorded and computed acceleration time histories and ARS (with 5% damping). Good agreement is noted. Similar match quality was observed for all other investigated events. As mentioned before only the first 23 seconds of event LSST16 were used (Fig. 49).

![Figure 47: Optimized Vertical Motion Velocity Profile and its 10% Indifference at Lotung.](image-url)
Figure 48: Recorded and Computed Vertical Accelerations during LSST6.

Figure 49: Recorded and Computed Vertical ARS during LSST6.
Figure 31: Recorded and Computed Vertical ARS during LSST.

Figure 30: Recorded and Computed Vertical Accelerations during LSST.
Figure 52: Recorded and Computed Vertical Accelerations during LSST16

Figure 53: Recorded and Computed Vertical ARS during LSST16
LIQUEFACTION AND DOWNHOLE ARRAYS

Liquefaction (excess pore pressure ratio \( r_e = u_p / \sigma_v' \) approaching and reaching 1.0, where \( u_p \) = excess pore pressure and \( \sigma_v' \) is effective vertical stress) and associated deformations remain among the main causes of damage during earthquakes (Seed et al. 1990, Bardet et al. 1995, Sitar 1995, Japanese Geotechnical Society 1996, Ansal et al. 1999). Indeed, dramatic unbounded (flow failure) deformations due to liquefaction in dams and other structures (Seed et al. 1975, 1989, Davis and Bardet 1996) have highlighted the significance of this problem in earthquake engineering research.

In this section, samples of downhole acceleration data in which liquefaction response is involved are presented. Many of these records point to the significance of an associated cyclic mobility phenomenon (described briefly below). Computational modeling of this important phenomenon is addressed thereafter. Finally, an available website for conducting live online site amplification and liquefaction computational modeling is presented (http://cyclic.ucsd.edu).

Soil Liquefaction from Field Downhole Arrays

**The Wildlife Refuge (California, USA):** This site is located on the West Side of the Alamo River in Imperial County, southern California. Evidence of liquefaction was observed at or near the site following the 1930, 1950, 1957, 1979, and 1981 Imperial Valley earthquakes (Youd and Wieczorek 1984). These observations triggered an interest in the site, which was instrumented in 1982, in an insightful effort (Youd and Wieczorek 1984) by the United States Geological Survey (USGS). In 1987, the site was shaken by two main earthquakes (Holzer et al. 1989), including the Superstition Hills earthquake (Mw 6.6), which caused a sharp increase in recorded pore-water pressure. In addition, subsequent field investigations showed evidence of site liquefaction and ground fissures. The surface records displayed peculiar acceleration spikes (Holzer et al. 1989) associated with simultaneous instants of excess pore-pressure drop (Fig. 54).

Using the simple approach of Koga and Matsuo (1990), the site lateral seismic response during liquefaction was inferred from the stress-strain and effective-path histories (Figs. 55 and 56) of the Superstition Hills earthquake (Zeghal and Elgamal 1994). At low effective confining pressures (high excess pore pressures), the effective stress-path clearly exhibited a reversal of behavior from contractive to dilative (Fig. 55), as the line of phase transformation (see below for further discussion) was approached (National Research Council, NRC 1985). Such a mechanism is a consequence of soil dilation at large strain excursions, resulting in associated instants of pore-pressure drop.

**Aomori Harbor (Japan):** Towhata et al. (1996) presented (Fig. 57) a ground surface lateral acceleration history recorded at Aomori Harbor during the 1968 Tokatchi-Oki earthquake. This history was recorded at a location where liquefaction of subsoil was observed. It may be noted that the Wildlife site acceleration record (Fig. 54) shows a pattern of spikes, which is quite similar to that of this Aomori Harbor record (Fig. 57). Thus, a shear stress-strain response of similar characteristics to that at Wildlife might have occurred at Aomori Harbor during this earthquake.

**Port Island (Kobe, Japan):** Port Island is an artificial (reclaimed) island located on the west-south side of Kobe, Japan. In the phase completed by 1981, 436 ha were reclaimed by bottom dumping from barges (Nakakita and Watanabe 1981). Soil in the artificial reclaimed layer (O'Rourke 1995, Sitar 1995, Japanese Geotechnical Society 1996) consisted of decomposed weathered granite fill (Masado soil mined from the nearby Rokko mountains) with grain sizes ranging from gravel and cobble-sized particles, to fine sand (2 mm mean particle size, with silt-sized particles or smaller of less than 10% by weight).

A downhole accelerometer array was installed at the Northwest corner of Port Island in August 1991 (Iwasaki 1995). The array consisted of triaxial accelerometers located at the surface, 16 m, 32 m, and 83 m depths. This array documented the observed widespread liquefaction of reclaimed ground during the 1995 Hyogoken-Nambu earthquake. Using the recorded downhole accelerations, the corresponding one-dimensional shear stress-strain response was evaluated (Elgamal et al. 1996c, 1997). At shallow depths, the stress-strain histories indicated (Fig. 58): (1) a noticeable reduction in stiffness with slight but visible signs of shear strain hardening at elevation 24 m, and (2) an abrupt sharp loss of stiffness and reduction of yield strength near the surface at 8 m depth. It is noted here that no clear signs of permanent lateral deformations were observed at the array site.

**Kushiro Port (Japan):** Iai et al. (1995) presented downhole accelerations recorded during the 1993 Kushiro-oki earthquake. The downhole array is located in Kushiro port. Upper layers at this site included saturated sand with shear wave velocities of 250 m/sec. The recorded accelerations showed clear spikes at ground surface and at 2 m depth (Fig. 59). These spikes were attributed to sand dilation (during the lateral seismic shear process), and were successfully simulated by a noteworthy computational model (Fig. 60) as will be discussed below. In this case, acceleration spikes do not necessarily denote \( r_e \) values approaching or equal to 1.0, but may nevertheless be associated with a phase of significant increase in excess pore-water pressure (and relatively high dynamic shear stress).
Figure 54: Recorded response at the Wildlife refuge site, Superstition Hills earthquake (Holzer et al. 1989, Zeghal and Elgamal 1994).

Figure 55: Wildlife-refuge NS shear stress-strain and effective-stress histories during the Superstition Hills 1987 earthquake (evaluated from acceleration histories, Zeghal and Elgamal 1994).
Figure 56: Selected stress-strain loops of Wildlife-refuge response during Superstition Hills earthquake (Zeghal and Elgamal 1994).

Figure 57: Aomori Harbor NS, 1968 Tokachi-oki earthquake (Towhata et al. 1996).
Figure 58: Selected stress-strain loops, Port Island, Hyogoken Nambu earthquake (Elgamal et al. 1996c).

Figure 59: Recorded acceleration, Kushiro-oki earthquake 1993 (Iai et al. 1995).

Figure 60: Kushiro-oki 1993, recorded and computed earthquake accelerations at the ground surface (Iai et al. 1995).
Soil Liquefaction from Centrifuge Downhole Arrays

A large body of centrifuge experiments using Nevada Sand (Arulmoli et al. 1992, Taboada and Dobry 1992) has also shown clear evidence of cyclic dilative soil response effects. Many of these experiments are described in (Arulanandan and Scott 1993, 1994) as part of the VELACS project (Taboada and Dobry 1993, Scott et al. 1993, Wilson and Kutter 1993, Dobry and Taboada 1994, Whitman and Ting 1994). Relative density of the Nevada sand varied within the range of about 40% to 75% in these tests. A representative set of recorded accelerations is shown in Figs. 61-63. In addition to VELACS, a number of RPI Ph.D. theses contain extensive evidence of this response (e.g., Liu 1992, Taboada 1995, Adalier 1996, Abdoun 1997). A large number of studies (mostly using Nevada Sand) conducted at the University of California, Davis (e.g., Divis et al. 1996, Arulanandan et al. 1977, Balakrishnan et al. 1997, Wilson et al. 1997) also show a similar response (e.g., Fig. 64).

The basic nature of associated shear stress-strain response is clearly manifested by back-calculated shear stress-strain lateral-deformation histories as reported by Dobry et al. (1995) and Elgamal et al. (1996b). These histories (Fig. 65) are based on the recorded acceleration and displacement responses of infinite mild-slope dynamic simulations (Fig. 61) in a set of pioneering tests conducted by Taboada (1995), and duplicated by Scott et al. (1993).

Cyclic Liquefaction

Summarizing the above observations from field and centrifuge downhole array data, it may be concluded that liquefaction frequently results in limited but objectionable levels of deformation (Bartlett and Youd 1995, Youd et al. 1999). Such response has been well documented in the pioneering work of Seed and Lee (1966), and Castro (1975). In such situations (Fig. 66), the deformation process may be characterized mainly as cyclic straining with limited amplitudes (Seed 1979), commonly known as cyclic mobility (Castro and Poulos 1977) or cyclic liquefaction (Casagrande 1975).

Much other laboratory experimental data sets also suggest the importance of the cyclic liquefaction mechanism. A valuable comprehensive survey of experimental research (triaxial, shear, and shake-table tests), documenting the significance of cyclic mobility during liquefaction (Fig. 66) was compiled by Seed (1979). Based on this survey, clean sands with a relative density $D_r$ of 45% or more, appear to exhibit the mechanism of limited strain cyclic mobility during liquefaction. At and above $D_r = 45\%$, the tendency for soil-skeleton dilation at large shear strain excursions instantaneously reduces $r_m$, allowing for significant regain in soil stiffness and strength, and eventually arresting further deformations.

In addition to the above, a survey compiled by Elgamal et al. (1998) indicates that a large body of laboratory experiments (e.g., Ishihara 1985, Arulmoli et al. 1992, Boulanger and Seed 1995), shake-table, and centrifuge tests (Dobry et al. 1995, Taboada 1995, Dobry and Abdoun 1998, Fiegel and Kutter 1992, Kutter and Balakrishnan 1998, Balakrishnan and Kutter 1999) all point to the same pattern of deformation (clean sands and non-plastic silts). In these experimental observations, clean uniform cohesionless soils with a reported $D_r$ of as low as 37%, may accumulate large liquefaction-induced cyclic shear strains, but do not exhibit flow-type failures (in laboratory sample, shake-table, and centrifuge tests).

Figure 61: RPI and CalTech Model 2 acceleration histories at free surface (AH3), 2.5m depth (AH4), and 5m depth (AH5), and input acceleration (AH1) at 10m depth (Taboada and Dobry 1993, Scott et al. 1993).
Figure 62: Recorded horizontal acceleration in UCD Model 7 (Wilson and Kutter 1993).

Figure 63: Pore-pressure and acceleration near surface of backfill, test 2e (Whitman and Ting 1994).

Figure 64a: Recorded lateral accelerations at UCD (Divis et al. 1996).
Figure 64b: Recorded lateral accelerations at UCD (Balakrishnan et al. 1997).

Figure 65: RPI (Dobry and Taboada 1994) and CalTech Model 2 (Scott et al. 1993) shear stress-strain histories (Elgamal et al. 1996b).
Computational Modeling for Liquefaction Analysis

Available Constitutive Models


At this point, few results have been published to show the performance of most constitutive models for the important case of an acting driving shear stress (e.g., lateral spreading situations, near and below slopes, the retaining wall problem). Such situations demand a high degree of control over accumulated cycle by cycle deformations as depicted in Fig. 68 (Tateishi et al. 1995).

Modeling the Phase Transformation Phenomenon

As described earlier, during a shear loading process, a saturated undrained cohesionless soil initially exhibits a tendency for contraction (phase 0-1 in Fig. 69), leading to development of excess pore-pressure and reduction in effective confinement (Lambe and Whitman 1969). As the shear strain increases and the shear stress approaches the failure envelope (or more precisely the so-called phase transformation (PT) envelope, Ishihara 1985, 1996, Vaid and Thomas 1995, Vaid and Sivathayalan 1999, Iai 1991, 1998, Dobry and Abdoun 1998, Kramer and Arduino 1999, Fig. 69), the contractive tendency changes to a dilative tendency that increases effective confinement (Vaid and Thomas 1995, Vaid and Sivathayalan 1999) and allows the soil to resist increased levels of shear stress (by moving upwards along the failure envelope, e.g., phase 2-3 in Fig. 69). For the purpose of liquefaction-induced shear deformations, medium-dense clean granular soils are found to exhibit this type of response (Elgamal et al. 1998). During a dilative phase (phase 2-3 in Fig. 69), the increased confinement may generate significant shear stiffness and strength in the soil, which progressively prevents further shear deformation.
Dilation can result in significant increases in shear stress and mean effective confining stress (Fig. 69). This increase will be limited by (Casagrande 1936, 1975):

1. Fluid Cavitation: If soil response is essentially undrained (fluid migration is relatively slow), the tendency for dilation can eventually drop pore-pressure to the minimum value of \(-1.0\) atmospheric pressure (i.e., cavitation). Cavitation will prevent the effective confining pressure from further increase (Iai 1998).

2. Critical void-ratio or constant volume soil response: If the soil is partially or fully drained (relatively rapid flow of pore-fluid), dilation-induced expansion of the soil skeleton will occur. To this end, the soil will eventually reach a critical void-ratio or "constant volume state", whereupon further shear deformation continues to develop without additional volume change. At this state, effective confinement will remain constant (due to shear loading).

![Stress-strain](image1)

![Stress path](image2)

*Figure 67: Measured and numerically simulated shear behavior of Fuji River Sand (Ishihara 1985, Iai 1991).*
Figure 68: Computed undrained torsional shear test response (Tateishi et al. 1995).

Figure 69: Schematic of constitutive model response showing the octahedral stress $\tau$, the effective confinement $p$, and the octahedral strain $\gamma$ relationship (after Parra 1996).

UCSD Liquefaction Model
A liquefaction soil constitutive model has been developed recently (Parra 1996, Yang 2000). In this model, emphasis is placed on controlling the magnitude of cycle-by-cycle permanent shear strain accumulation in clean medium-dense sands (Parra 1996, Yang 2000). Specifically, the experimentally observed accumulation of permanent shear strain (e.g., Arulmoli et al.)
1992, Fig. 66) was modeled by using strain-space parameters (Yang 2000), within a multi-yield surface stress-space model (Prevost 1985). Furthermore, appropriate loading-unloading flow rules were devised to reproduce the observed strong dilation tendency (Arulmoli et al. 1992, Fig. 66), and resulting increase in cyclic shear stiffness and strength. The main components of this model are summarized below.

Yield function

Following the classical plasticity convention (Hill 1950), it is assumed that material elasticity is linear and isotropic, and that nonlinearity and anisotropy result from plasticity. The selected yield function (Prevost 1985, Lacy 1986) forms a conical surface in stress space with its apex at \( p_0 \) along the hydrostatic axis (Fig. 70). In the context of multi-yield-surface plasticity (Iwan 1967, Mroz 1967, Prevost 1985), a number of similar yield surfaces with the common apex \( p_0 \) and different sizes form the hardening zone (Fig. 70). The outmost surface is designated as the failure surface. Each surface is associated with a constant stiffness (elasto-plastic modulus), and the stiffness value typically decreases with the surface size (Fig. 71).

Hardening rule

A purely deviatoric kinematic hardening rule (Prevost 1985) is employed to account for the Bauschinger effect exhibited by soil under cyclic loading. All yield surfaces but the outermost may translate in stress space (Prevost 1985, Parra 1996, Yang 2000).

When subjected to a drained monotonic shear loading, this hardening rule generates a piecewise-linear, gradually softening shear stress-strain curve (Fig. 71), the so-called back-bone curve (Kramer 1996). Therefore, the elasto-plastic moduli associated with the yield surfaces can be calibrated by matching a computed back-bone curve to that from experimental data. Furthermore, when subjected to a drained cyclic shear loading, this hardening rule generates a piecewise-linear shear stress-strain loop with hysteretic damping.

Flow rule

During shear loading, the soil contractive/dilative behavior is handled by a non-associative flow rule (Parra 1996, Yang 2000) so as to achieve appropriate interaction between shear and volumetric response. In particular, nonassociativity is restricted to the volumetric component \( P'' \) of the plastic flow tensor (outer normal to the plastic potential surface in stress-space). Therefore, depending on the relative location of the stress state (Fig. 69) with respect to the phase transformation (PT) surface, different expressions for \( P'' \) were specified for (Parra 1996):

1. The contractive phase, when the stress-state lies inside the PT surface (Fig. 69, phase 0-1).
2. The dilative phase during loading, if the stress-state lies outside the PT surface (Fig. 69, phase 2-3), and
3. The contractive phase during unloading, with the stress-state starting outside the PT surface (Fig. 69, phase 3-4).

At low effective confining pressure, when the stress state reaches the PT surface while loading, permanent shear strain may accumulate rapidly with essentially no change in shear stress (Fig. 69, phase 1-2). This is achieved by activating a perfectly plastic zone (PPZ, Fig. 69, phase 1-2) before the initiation of dilation outside the PT surface (Fig. 69, phase 2-3). The PPZ is defined in deviatoric strain space as a circular, initially isotropic surface (Yang et al. 2000). Depending on the current strain state and plastic loading history, the PPZ may enlarge and/or translate in deviatoric strain space to model the accumulation of permanent shear deformations (Yang 2000).

Model Calibration

The developed model has been extensively calibrated for No. 120 Nevada sand at a relative density \( D_r \) of about 40% (Parra 1996, Yang 2000). The calibration procedure for medium Nevada sand (\( D_r = 40\%) \) was carried out in the following stages. First, the data from a monotonic Consolidated Isotropically Drained Compression triaxial test (Arulmoli et al. 1992) was employed to define elastoplastic moduli and sizes of the multi-yield surfaces. Next, the data from a monotonic Consolidated Isotropically Undrained Compression triaxial test (Arulmoli et al. 1992) was used to define constitutive model parameters that control contraction and dilation behavior. Thereafter, the remaining model parameters (especially the parameters that control the extent of cyclic accumulation of liquefaction-induced large yield strain) were calibrated based on matching recorded responses from a Consolidated Anisotropically Undrained Cyclic loading test (Arulmoli et al. 1992) and two (Taboada and Dobry 1993, Dobry and Taboada 1994) dynamic centrifuge tests (dealing with liquefied site response and lateral spreading).

Model Performance

Figures 72 and 73 illustrate the mechanism of model response. These figures depict a simulation of a biased simple shear stress-strain history. A static driving "locked-in" deviatoric stress was simulated by applying load cycles in the range of 0.0 kPa to 60 kPa (Figure 72). Under this loading history, gradual pore pressure buildup and liquefaction occurs (i.e., effective confinement approaches zero, Figure 73). During liquefaction, the model reproduces a stable cycle-by-cycle accumulation of shear deformation. For engineering applications, three performance scenarios were selected (Elgamal et al. 1999) to represent clean medium, medium-dense and dense sand (or non-plastic silt) situations (relative density in the range of about 40% to 90%). In Figure 72, maximum accumulated cycle-by-cycle shear deformations are about 1.3% (medium), 0.5% (medium-dense) and 0.3% (dense).

The medium sand response was calibrated according to the procedure described above. At this point, the deformation characteristics for the medium-dense and dense sand situations are based on engineering judgment (motivated by the literature review in Elgamal et al. 1998). Further data is needed to refine the estimates of the proposed model (Figs. 72 and 73), particularly for the medium-dense and dense sand situations.
Figure 70: Conical yield surface in principal stress space and deviatoric plan (after Prevost 1985, Lacy 1986, Parra 1996, and Yang 2000).

Figure 71: Hyperbolic representation of soil nonlinear shear stress-strain response.
Figure 72: Simulation of undrained biased cyclic simple shear tests (Elgamal et al. 1999, Yang et al. 1999).

Figure 73: Computed stress path during undrained biased cyclic simple shear (Elgamal et al. 1999, Yang et al. 1999).
Website for Site Amplification/Liquefaction

Introduction
A large number of computational programs have been and continue to be developed for assessing seismic ground response and liquefaction effects (e.g., DYNAFLOW (Prevost 1998), DYSAC2 (Muraleetharan et al. 1988), and other notable codes (see Arulanandan and Scott 1993, 1994). Ease of use of such programs is among the most important concerns. One efficient solution is being increasingly facilitated by the Internet in the form of an interactive web page. An interactive web page for a computer code CYCLIC 1D (http://cyclic.ucsd.edu) allows remote users via a web browser to select the desired model parameters and excitation signal, run the computational code CYCLIC, and view/retrieve the computation results. Thus, regardless of the specifics of a particular computational program, all interested users can gain access around the clock.

Website
The website (http://cyclic.ucsd.edu) allows remote users to operate a nonlinear finite element program CYCLIC that is specially developed for numerical simulation of earthquake ground response and liquefaction effects. CYCLIC is a two-phase (solid and fluid) fully coupled, two-dimensional Finite Element program. Fig. 74 depicts a typical element employed in CYCLIC. This program has been evolving over the past 10 years with extensive calibration through numerous sources including field downhole array records, laboratory tests, and centrifuge experiments (some of which are mentioned above).

Currently Available Options
(1) Soil profile height (any value up to 100 m).
(2) Number of elements (10 - 100 elements).
(3) Depth of water table.
(4) Inclination angle of the soil profile (any value from 0.0 to 10.0 degrees can be chosen, with 0.0 degree representing level ground). This option allows for mild infinite slope simulations.
(5) Bedrock stiffness. If "Rigid", the input motion is treated as a total motion; otherwise, the input motion is handled as an incident motion.
(6) There are 15 materials available (3 cohesive and 12 cohesionless soils), covering a wide range of densities and permeabilities. A different material may be assigned to individual elements.
(7) Base excitation can be chosen from an input motion library.
(8) The amplitude of input motion can be scaled by a factor ranging from 0.01 to 1.0.

Sample Web-Site Results
Figures 75 - 77 show the numerical simulation results of a 10m saturated medium sand stratum with an inclination angle of 6.0 degrees, subjected to 10 cycles of 0.2g sinusoidal base excitation. The results (Figs. 76 and 77) show the involved cyclic-mobility lateral spreading response characteristics.

In Figure 76, acceleration near ground surface is seen to display asymmetric response with spikes that are directly related to the instants of excess pore-pressure drop during liquefaction. Associated lateral spreading is also seen to accumulate on a cycle-by-cycle basis. In Figure 77, the cyclic mobility characteristics are displayed in terms of: i) high stiffness and strength in the shear stress-strain response during liquefaction, and ii) phases of increase in shear strength (and increase in effective stress) as the stress path travels above the phase transformation line during cyclic loading.

Figure 74: Typical 9-4-node element employed in CYCLIC (effective stress u-p coupled formulation).
Profiles of Maximum, Minimum, and Final Computed Results

Horizontal Displacement (Relative to the base, m)

Horizontal Acceleration (m/s²)

Excess Pore Pressure (kPa)

Shear Strain

Shear Stress (kPa)

Effective Confinement (kPa)

Figure 75 Profiles of maximum, minimum, and final computed results (http://cyclic.ucsd.edu).

Computed Results at 1 m Depth

Horizontal Acceleration Time History (m/s²)

Response Spectrum of Acceleration (m/s²)

Fourier Transform Amplitude of Acceleration (m/s)

Figure 76a: Computed acceleration, lateral displacement and excess pore-pressure histories at 1 m depth (http://cyclic.ucsd.edu).
Figure 76b: Computed acceleration, lateral displacement and excess pore-pressure histories at 1m depth (http://cyclic.ucsd.edu).

Figure 77: Computed shear stress-strain and stress path at 1.5m depth (http://cyclic.ucsd.edu).
SUMMARY AND DISCUSSION OF FUTURE NEEDS AND APPLICATIONS

In the earlier sections, downhole array data have shed light on the following main site response characteristics:

1. Definition of damping ratio ranges that result in satisfactory computational response of lateral and vertical 1D seismic site response.
2. The possible presence of azimuthal site anisotropy, in the form of different stiffness characteristics in different horizontal directions (at Hualien, Taiwan stiff soil site).
3. The significance of cyclic mobility in liquefaction response with possible consequence such as:
   - Appearance of high amplitude acceleration spikes in the recorded surface motions.
   - Possible asymmetry in recorded accelerations with the spikes appearing in a direction that corresponds to evolution of cyclic lateral deformation (down-slope cyclic deformation).
   - Ability of soil (clean sands and non-plastic silts) to restrain the magnitude of shear deformation as a consequence of this phenomenon (i.e., undergo a number of limited deformation cycles before possible evolution of a flow-failure).

Much remains to be explored based on downhole array deployments. The involved high expenses of installation and maintenance are still a major challenge. In this regard, it is hoped that advances in sensor technology will eventually result in economical solutions and allow for wide deployment.

Additional issues of current relevance to the quality and value of recorded motions include:

1. Development of standards for installation techniques of downhole instruments into the ground (nature of connection between instruments and borehole casing, and construction procedures of boreholes in different soils).
2. Accuracy in definition of sensor depth and accounting for sensor package size (some 3-accelerometer downhole units are of the order of 0.5m in length).
3. Relative orientation between downhole sensor instruments. This problem is known as azimuthal error (e.g., a downhole sensor assumed to be aligned in the NS-EW directions is actually significantly off).
4. Accuracy in deployment in terms of maintaining correct polarity for all recorded data (e.g., positive amplitudes are North and negative are South for all sensors).
5. Need for more closely-spaced downhole instruments. In many cases today, instruments are sparse covering depths of 100m or more with 2 or 3 sensor locations. This lack of sufficient instrumentation is especially detrimental when the soil profile is of significant variability.
6. Increased accuracy by employing a sufficiently small digitization time step (e.g., 0.005 second or smaller). In addition, accuracy in time synchronization between all instruments in the array is of utmost importance. This accuracy allows for tracking propagating waves, among other time domain analysis applications.
7. Need for more accuracy in measuring seismic ground displacement. Currently, integration of recorded accelerations necessitates the removal of useful long-period signals. This has always been a tough problem to overcome and Geographical Positioning System (GPS) or similar techniques might eventually offer a viable solution.
8. In situations related to liquefaction, additional concerns related to occurrence of relative motion between sensor and surrounding soil (both vertically and laterally) and proper saturation of piezometers must be addressed carefully. When excess pore-pressure remains after dynamic excitation, piezometer data should be collected (at a slow rate) to document the important dissipation phase (which might be accompanied by significant liquefaction-induced deformations).
9. Finally, the data-processing phase is currently a major chore. Much effort is currently expended in attempts to acquire available recorded data. Often, many questions arise thereafter and must be answered before the data is ready for use. Meticulous book-keeping is needed in handling the data files, and new versions of the data files must be prepared for use in analysis packages (e.g., MATLAB) or home-made codes. Simplification of the above process is of utmost importance. In this regard, a web-based environment equipped with data processing and visualization tools can be of much value. In this environment, duplication of work between researchers can be minimized, and all users can consistently and conveniently have access to the same data. This extra investment of resources is well worth it.

Looking into the future, synchronized and more elaborate 1D, 2D and 3D downhole arrays (e.g., the Chiba Array in Japan, Katayama 1990) will eventually contribute much increased knowledge about:

1. 3D wave propagation mechanisms and analyses related to surface and body waves.
2. Increased understanding of lateral and vertical vibration including considerations related to inhomogenity, anisotropy and topography.
3. Major advances in soil-structure interaction when a superstructure (including foundation) and underlying ground are instrumented as one integrated entity. Examples of ongoing applications currently include bridges, buildings and dams. Deployment for other critical facilities (e.g., Power Plants, Ports and Harbor facilities, and water-front structures) will be of great value.

Advances in technology may also allow speculation about:

1. Massive (compared to the situation today) deployment of downhole sensors allowing fine resolution for highly stratified or inhomogeneous sites and for assessment of liquefaction countermeasure techniques.
2. Real-time or near real-time data availability and automated levels of data processing for early warning, decision-making purposes, and emergency response applications.
3. Possibility of more global synchronization of sensor data which may become a basis for larger scale studies such as seismic response of basins underlying a city, and other similar applications.

SOAP-6
Finally, many downhole arrays are active today and are ready to record earthquake excitations. In most cases, strong shaking of engineering significance is yet to be recorded. In the heavily instrumented areas, our database will increase substantially with each new earthquake. In the meantime, efforts are underway (e.g., at the University of Texas, Prof. Ken Stokoe) to develop in-situ dynamic excitation sources that can be used to trigger downhole installations and generate additional (man-made) tailored data sets (to augment our currently available database).

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