Dynamic Characteristics of Woodframe Structures

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California Institute of Technology

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Preface

The Northridge Earthquake of January 17, 1994 was in effect a full-scale dynamic test of thousands of wood structures. The Field Investigations element of the CUREE-Caltech Woodframe Project explores this information via Statistical Analysis (1) and this Case Studies report.

The Case Studies present observations of engineers who have investigated woodframe buildings damaged by the Northridge Earthquake. Their information is presented in this report in consistent content and format, designed to capture this valuable resource in a form that will help develop new design guidelines and code provisions.

The CUREE-Caltech Woodframe Project originated in the need for a combined research and implementation project to improve the seismic performance of woodframe buildings, a need which was brought to light by the January 17, 1994 Northridge, California Earthquake in the Los Angeles metropolitan region. Damage to woodframe construction predominated in all three basic categories of earthquake loss in that disaster:

- Casualties: 24 of the 25 fatalities in the Northridge Earthquake that were caused by building damage occurred in woodframe buildings (2);
- Property Loss: Half or more of the $40 billion in property damage was due to damage to woodframe construction (3);
- Functionality: 48,000 housing units, almost all of them in woodframe buildings, were rendered uninhabitable by the earthquake (4).

Woodframe construction represents one of society’s largest investments in the built environment, and the common woodframe house is usually an individual’s largest single asset. In California, 99% of all residences are of woodframe construction, and even considering occupancies other than residential, such as commercial and industrial uses, 96% of all buildings in Los Angeles County are built of wood. In other regions of the country, woodframe construction is still extremely prevalent, constituting, for example, 89% of all buildings in Memphis, Tennessee and 87% in Wichita, Kansas, with "the general range of the fraction of wood structures to total structures...between 80% and 90% in all regions of the US…" (5).

Funding for the Project is provided primarily by the Federal Emergency Management Agency (FEMA) under the Stafford Act (Public Law 93-288). The federal funding comes to the project through a California Governor’s Office of Emergency Services (OES) Hazard Mitigation Grant Program award to the California Institute of Technology (Caltech). The Project Manager is Professor John Hall of Caltech. The Consortium of Universities for Research in Earthquake Engineering (CUREE), as subcontractor to Caltech, with Robert Reitherman as Project Director, manages the subcontracted work to various universities, along with the work of consulting engineers, government agencies, trade groups, and others. CUREE is a non-profit corporation devoted to the advancement of earthquake engineering research, education, and implementation.
Cost-sharing contributions to the Project come from a large number of practicing engineers, universities, companies, local and state agencies, and others.

The project has five main Elements, which together with a management element are designed to make the engineering of woodframe buildings more scientific and their construction technology more efficient. The project’s Elements and their managers are:

1. **Testing and Analysis:** Prof. André Filiatrault, University of California, San Diego, Manager; Prof. Frieder Seible and Prof. Chia-Ming Uang, Assistant Managers
2. **Field Investigations:** Prof. G. G. Schierle, University of Southern California, Manager
3. **Building Codes and Standards:** Kelly Cobeen, GFDS Engineers, Manager; John Coil and James Russell, Assistant Managers
4. **Economic Aspects:** Tom Tobin, Tobin Associates, Manager
5. **Education and Outreach:** Jill Andrews, Southern California Earthquake Center, Manager

**Notes**


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Dynamic Characteristics of Woodframe Structures

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Summary

The main objective of Task 1.3.3 of the CUREE-Caltech Woodframe Project is to determine the dynamic characteristics of woodframe buildings and to develop a period formula specific for wood structures by regressing on appropriate structural characteristics. Much research has been done on the dynamic and cyclic characteristics of wood subsystems and connection panels, but a literature review shows that full-scale testing of wood shearwall buildings has been sparse. This scarcity emphasizes the importance of Task 1.3.3 because the dynamic characteristics of woodframe buildings as a whole must be better understood in order to establish code requirements for their safe and cost-effective design, and to improve current building practices.

Through analysis of recorded earthquake response and by forced and ambient vibration testing, Task 1.3.3 has developed a database of periods and damping ratios of woodframe buildings. The focus is on the fundamental mode behavior of low-rise shearwall buildings, the predominant type of woodframe construction in North America, as the fundamental mode tends to dominate the dynamic characteristics. A regression analysis was performed on the period database using building height to produce a simple but reasonably accurate period formula for design of woodframe buildings. This formula gives shorter periods than the 1997 UBC period formula, but it should be noted that all test results, including the seismic data, are at small drifts (in the order of 0.0004 or less). Despite these low amplitudes, the equivalent viscous dampings for the fundamental modes were usually more than 10% of critical during earthquake shaking.
Dynamic Characteristics of Woodframe Structures

In this project, the dynamic properties of wood shearwall buildings were evaluated, mainly modal parameters such as frequencies, damping and mode shapes of the structures and how these parameters change with motion amplitude. The focus was on the behavior of the entire structure in the elastic range (smaller amplitude of vibrations), and a database of fundamental mode parameters was compiled based on the reviewed literature and also based on vibration tests and analysis of recorded earthquake response.

Literature Review

Much research has been done on the dynamic and hysteretic characteristics of wood subsystems and connection panels (e.g. Cheung 1984; Falk 1986; Dolan 1989; Polensek and Schimel 1991), but that full-scale testing of wood shearwall buildings has been sparse.

Polensek and Schimel (1991) evaluated the degree of nonlinearity and degradation of damping and stiffness properties in wood subsystems with and without finish materials (gypsum wallboard). They observed that energy was dissipated by slipping interfaces of connected materials, and that damping tends to increase with increasing amplitude of vibration up to some limit, after which prior damage tends to reduce interface friction and therefore reduce damping and stiffness of shear wall, bending and connection panels. They also noted that the dynamic behavior of the panels was the same regardless of the lumber grade, suggesting that panel damping and stiffness depend mostly on nailed joints and less on the grade of lumber used in framing.

Seo et al. (1981) performed static and cyclic lateral load tests on wooden frames with tenon beam-column joints. The tests showed nonlinear and inelastic behavior, with estimated equivalent viscous damping ratios between 13% and 27% for these types of structures. The frame stiffness was significantly reduced with increased amplitude of displacement.

Hirashima (1988) tested a two-story building with diagonal bracings built in post-and-beam frames with no wall claddings, neither exterior nor interior. He used static loading tests to obtain spring constants to use in a mathematical model of the building and forced vibration tests to observe the dynamic behavior. He noted that the test building oscillated mainly in its fundamental mode of vibration in each direction, and that the corresponding periods of vibration were almost constant throughout the motion at 0.25 sec (4.0 Hz) transverse and 0.22 sec (4.5 Hz) longitudinal. The corresponding damping ratios were quite low, 2.4% transverse and 1.4% longitudinal, from a free vibration test with initial peak-to-peak displacements of about ½ mm. An earthquake record was also obtained in the test building with a 6%g peak acceleration at the roof. A Fourier amplitude spectrum of the roof accelerations showed a fundamental period for each direction of about 0.25 sec (4 Hz).
Yokel, Hsi and Somes (1973) performed full-scale tests on a two-story house with a partial brick veneer front at the lower story, stucco exterior finish and gypsum board interior. A series of tests were conducted to determine the dynamic response of the house to a single impulse load. The natural frequency of the structure was approximately 0.11 sec (9 Hz) and damping averaged 6% of critical, varying from 4% to 9%. The validity of these findings is questioned by the researchers, since the resolution of the displacement time history records was marginal.

Foliente and Zacher (1994) report on dynamic tests of timber structural systems. Table 1, taken from their paper, gives a summary of periods from tests performed in several different countries. Because of the differences in construction, results from other countries may not be especially relevant, and there are only a few tests of conventional North American woodframe residential construction. These show periods in the range 0.06 to 0.33 sec (3 to 18 Hz) (See Table 1), which is consistent with the values identified in the tests and analysis performed in this project.

Table 1:
Summary of Natural Periods and Frequencies of Low-Rise Wood and Wood-Based Buildings

<table>
<thead>
<tr>
<th>Building Type</th>
<th>Natural Period Tn (sec)</th>
<th>Natural Frequency (1/Tn) (Hz)</th>
<th>Reference(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two- and three-story N. American residential</td>
<td>0.14 to 0.33</td>
<td>3.0 to 7.0</td>
<td>Sugiyama (1984)</td>
</tr>
<tr>
<td>One-, one and a half- and two-story N. American residential and school buildings</td>
<td>0.06 to 0.25</td>
<td>4.0 to 18.0</td>
<td>Soltis et al (1981)</td>
</tr>
<tr>
<td>One- and two-story New Zealand residential</td>
<td>0.1 to 0.6</td>
<td>1.7 to 10.0</td>
<td>Dowrick (1987)</td>
</tr>
<tr>
<td>One-story truss-frame residential</td>
<td>0.14 to 0.26</td>
<td>3.8 to 7.2</td>
<td>Gavrilović and Gramatikov (1991)</td>
</tr>
<tr>
<td>Two-story residential (Greece)</td>
<td>0.18 to 0.22</td>
<td>4.5 to 5.6</td>
<td>Touliatos et al. (1991)</td>
</tr>
<tr>
<td>One-, two- and three-story Japanese residential</td>
<td>0.11 to 0.33</td>
<td>3.0 to 9.0</td>
<td>Arima et al (1990)</td>
</tr>
<tr>
<td>Three-story Japanese residential</td>
<td>0.16 to 0.20</td>
<td>4.7 to 6.2</td>
<td>Nakajima et al. (1993)</td>
</tr>
<tr>
<td>One- and two-story comm’l/industrial (plywood roof diaphragm and concrete/masonry walls)</td>
<td>0.20 to 0.80</td>
<td>1.2 to 5.1</td>
<td>Bouwkamp et al. (1994)</td>
</tr>
<tr>
<td>Range of Values for N. American Residential</td>
<td>0.06 to 0.33</td>
<td>3.0 to 18.0</td>
<td></td>
</tr>
</tbody>
</table>
Filiatrault, et. al. (2001) performed full-scale shake table tests on a two-story single-family woodframe house under Task 1.1.1 of the CUREE-Caltech Woodframe Project. The structure was tested during 10 phases of construction to determine the performance of the structure with fully sheathed shearwalls, symmetrical and unsymmetrical door and window openings, perforated shearwall construction, conventional construction, and with and without non-structural wall finish materials. The building had plan dimensions of 16’x20’ and height of 20’ (to top of roof). They performed four types of shake table tests: quasi-static in-plane floor diaphragm tests, frequency evaluation tests, damping evaluation tests, and seismic tests. For the fully-configured building (wall finish applied, Phase 10 tests), the fundamental transverse frequency was 6.5 Hz (from ambient vibrations), 6.3 Hz (shaking at PGA = 0.05g), 5.8 Hz (shaking at PGA = 0.36g), and 5.5 Hz (shaking at PGA = 0.50g and 0.89g). The equivalent viscous damping ratios were based on log-decrement method and increased from 3.1% at ambient levels to 12% at PGA = 0.22g shaking, then decreased to about 6% at PGA = 0.5g shaking and beyond.
Review of Current Code Period Formulas

Current building codes require a design earthquake load based on the building’s system characteristics, site location, occupancy, etc. The code specifies simplified formulas to approximate the building’s dynamic behavior. One important factor in determining how the building will behave during an earthquake is its fundamental period. This is used, for example, to help determine the appropriate seismic base shear coefficient for the design of a structure.

Recent research has shown that the current 1997 Uniform Building Code period formulas substantially underestimates the building periods for concrete and steel moment-resisting frame buildings, as well as that for concrete shearwall buildings (Goel and Chopra, 1997, 1998). An important objective of this task is to evaluate and improve the current code period formulas for wood structures.


The 1997 UBC prescribes the following period formulas for buildings:

Method A:

\[ T = C_t h_n^{3/4} \]  

where \( T \) = period, \( C_t \) = period factor, \( h_n \) = height, in feet, above the base to the uppermost level in the main portion of the structure.

The values of \( C_t \) vary according to material and framing characteristics:

- \( C_t = 0.035 \) (Steel Moment Resisting Frames)
- \( C_t = 0.030 \) (Reinf. Concrete Moment Resisting Frames and Eccentric Braced Frames)
- \( C_t = 0.020 \) (All other buildings)

or, for Concrete or Masonry Shear Wall Buildings, the following may be used:

\[ C_t = \frac{0.1}{A_e^{0.5}} \quad \text{where} \quad A_e = \sum A_e \left[ 0.2 + \left( \frac{D_e}{h_n} \right)^2 \right] \quad \text{and} \quad \frac{D_e}{h_n} \leq 0.9 \]

where \( A_e \) = minimum cross-sectional area, in sq. feet, of the shear walls in the first story of structure.

\( D_e \) = length, in feet, of a shear wall in the first story in the direction parallel to the applied forces.
Method B:

\[
T = 2\pi \sqrt{\sum_{i=1}^{n} w_i \delta_i^2} \sqrt{\sum_{i=1}^{n} g f_i \delta_i}
\]

where:

\( w_i \) = that portion of the total seismic dead load located at or assigned to level i.

\( \delta_i \) = horiz. displacement at level i relative to the base due to applied lateral forces, f.

\( f_i \) = lateral force at level i.

\( g \) = acceleration due to gravity.

\( n \) = uppermost level in the main portion of the structure.

The UBC-97 limits the maximum period obtained from Method B (simplified structural analysis) to 1.3 times the period obtained by Method A for Zone 4 buildings or 1.4 times period obtained by Method A period for Zones 1, 2 and 3.

**FEMA-273**

This document presents approaches for the seismic rehabilitation of buildings that will limit the expected earthquake damage due to a certain level of ground shaking. In FEMA-273 Section 3.3 – Analysis Procedures, it offers a linear static method equivalent to that of the UBC-97, and the period can be determined by one of three methods:

**Method 1:** eigenvalue (dynamic) analysis of mathematical model

**Method 2:** similar to UBC-97 Method A,

\[
T = C_i h_a^{3/4}
\]

The values of \( C_i \) are same as in UBC-97 except:

\( C_i = 0.060 \) (for wood buildings)
Method 3: for a 1-story building with single span flexible diaphragm,

\[ T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \]  \hspace{1cm} (4)

where:

\[ \Delta_w = \text{in-plane wall displacement in inches due to lateral load equal to the weight tributary to diaphragm} \]
\[ \Delta_d = \text{in-plane diaphragm displacement in inches due to lateral load equal to the weight tributary to diaphragm} \]

The period obtained from eq. (4) for various diaphragms and walls that maximizes the design base shear (pseudo lateral load) is to be used.

Other Building Codes

Many other building codes, such as the National Building Code of Canada and the Australian Standard, have based their period formulas on older versions of the Uniform Building Codes (see IAEE, 1996). These formulas do not differentiate between building materials or structural systems, for example:

\[ T = 0.09 \frac{h}{\sqrt{L}} \]  \hspace{1cm} (5)

or

\[ T = 0.1N \text{ for moment-resistant space frames} \]  \hspace{1cm} (6)

where:

\[ L = \text{the overall building dimension in the direction of the earthquake forces or the length of the wall or braced frame parallel to the lateral forces, depending on the country’s interpretation of the UBC formula.} \]
\[ N = \text{number of stories/levels.} \]

Other codes, such as the New Zealand Standard, do not prescribe a simplified period formula. They prescribes a formula based on Rayleigh’s method, similar to the “Method B” formula in the UBC-97.
Recent Developments

Goel and Chopra (1997, 1998) have presented alternative period formulas for reinforced concrete and steel moment-resisting frame buildings and for concrete shearwall buildings. They obtained information about the fundamental modes of vibration of a number of buildings by analyzing their recorded motion from various California earthquakes. These structures were shaken strongly but not so strongly as to enter the inelastic range. They were divided into two categories depending on the strength of the earthquake shaking they experienced, i.e. whether or not the peak ground acceleration was less than 0.15 g. After determining that the current code formulas substantially underestimated the natural vibration periods for these structures, they re-evaluated the theory upon which the code formulas were based and derived new formulas by regression analysis. These period formulas lead to a best fit, in the least-squares sense, to the measured period data. The final recommended period formulas were derived looking at the trend obtained from only the buildings experiencing peak ground accelerations of 0.15g or greater. At smaller acceleration levels, the periods tend to be smaller because the non-structural components contribute significantly to the lateral stiffness. This fact should also be kept in mind when interpreting the period formula derived during this project because all data on which it was based came from low-amplitude response.

Goel and Chopra concluded that Rayleigh’s method was sufficient to give a good approximation of the dynamic behavior of moment resisting frame buildings. Based on this method, the period formula should be of the form \( T = C h_n^\gamma \), where \( C \) and \( \gamma \) are to be determined from regression analysis in the form \( \ln T = \ln C + \gamma \ln h_n \). The least-squares estimates of \( \ln C \) and \( \gamma \) then give the median estimate of the period, that is, there is a 50% probability that the actual period of the building is greater than the period estimated by the regression formula. Since for determination of design base shear the formula should provide lower values of the period (to be conservative), Goel and Chopra chose a lower bound of a standard deviation from the best fit line. Also, they provided an upper limit for periods found using rational analysis rather than the code formula. The resulting lower bound period formulas were the following, with standard error estimates of \( s_e = 0.209 \) for R/C MRF and \( s_e = 0.233 \) for steel MRF:

**Reinforced Concrete Moment Frames:**

\[
T = 0.016 h_n^{0.90} \quad (7)
\]

no larger than 1.4 T if using rational analysis

**Steel Moment Resisting Frames:**

\[
T = 0.028 h_n^{0.80} \quad (8)
\]

no larger than 1.6 T if using rational analysis

where

\( h_n = \) the total building height from base of structure, in feet.
Rayleigh’s method was not sufficient to give a good estimate for the dynamic characteristics of shearwall buildings, so Goel and Chopra chose to use various other well established analytical procedures, such as Dunkerley’s method, which combines both flexural and shear deformations of a cantilever. Based on this particular method, the period formula should be of the following form:

\[ T = C \frac{h_n}{\sqrt{A_e}} \]

where

\[ A_e = 100 \frac{A_{e}}{A_B} \]

and

\[ A_e = \frac{A}{\left[ 1 + 0.83 \left( \frac{h_n}{D} \right)^2 \right]} \]

\[ h_n = \text{the total building height from base of structure, ft} \]
\[ C = \text{constant to be defined by regression} \]
\[ A_B = \text{plan area of building, ft}^2 \]
\[ A = \text{total area of shear walls, ft}^2 \]
\[ D = \text{building dimension parallel to direction being considered, ft} \]

Regression analysis yielded the following formula, with an error of estimate \( s_e = 0.143 \) while the UBC-97 error estimate is \( s_e = 0.546 \):

Concrete Shear Wall:

\[ T = 0.0019 \frac{h_n}{\sqrt{A_e}} \]

no larger than 1.4 \( T \) if using rational analysis
A period and damping database was developed by performing system identification on response time histories recorded in woodframe buildings. These vibration records consist of available seismic records and also of ambient and forced test vibration data. The earthquake and ambient vibration data were analyzed using the Caltech program MODE-ID, which uses a well-established system identification procedure to estimate the modal parameters of the dominant contributing modes for the building being evaluated. The forced vibration data were analyzed by a nonlinear least-squares fit of the model frequency response curves to the recorded frequency response curves (the forcing excitation was sinusoidal), from which modal frequencies and damping values were obtained.

**Method for System Identification**

Modal identification is an important application of system identification in structural dynamics where modal parameters based on a model with linear dynamics are estimated using dynamic data from a structure. Modal identification can be performed in the time domain without the need to develop a structural model involving mass, stiffness and damping matrices (Beck, 1978). The method was initially applied to the measured seismic response from tall buildings where only a single input (the recorded base acceleration) was used (Beck and Jennings, 1980). The method was then extended to handle multiple inputs in order to find the modal parameters from seismic motions recorded on a bridge (Werner et al, 1987). The computer program, called MODE-ID, that implements this approach has been extensively applied to earthquake and other dynamic data. MODE-ID is based on a nonlinear least-squares output-error method, which utilizes a class of models defined as follows.

Structural motion at the $N_0$ observed degrees of freedom is modeled as a superposition of $N_m$ dominant modes:

$$x_i(t) = \sum_{r=1}^{N_m} x_{ir}(t), \quad i = 1, \ldots, N_0$$  \hspace{1cm} (11)

where $x_{ir}$ is the contribution of the $r^{th}$ mode to the response at the $i^{th}$ degree of freedom.

The response for the ($N_m - 1$) dynamic modes of vibration is calculated numerically using a very accurate discrete-time recursive approximation (Beck and Dowling, 1988) of the well-known equation of motion:

$$\ddot{x}_{ir} + 2\zeta_r \omega_r \dot{x}_{ir} + \omega_r^2 x_{ir} = \phi_{ir} \sum_{k=1}^{N_k} p_{rk} f_k(t)$$  \hspace{1cm} (12)

$$x_{ir}(0) = \phi_{ir} c_r; \quad \dot{x}_{ir}(0) = \phi_{ir} d_r; \quad \sum_{i=1}^{N_0} \phi_{ir}^2 = 1$$
where the \( f_k, k = 1,\ldots,N_I \) are the measured accelerations at the \( N_I \) structural supports (e.g. defining the motion at the base of the structure).

A pseudostatic “mode” is also necessary:

\[
\ddot{x}_i = \sum_{k=1}^{N_I} r_{ik} f_k (t)
\]  \hspace{1cm} (13)

This accounts for the quasi-static contributions to the structural motions induced by the support motions during the earthquake, ignoring inertial and damping effects since these are accounted for in the dynamic response contributions (Werner et al, 1987). The simplest pseudostatic mode is rigid-body motion such as the direct contributions from rocking and translation of the base of a building.

The model parameters \( \alpha \) to be estimated are the modal parameters for each of the identified \( (N_m-1) \) dynamic modes, that is, the natural frequencies and damping ratios, \( \omega_r \) and \( \zeta_r \), the initial modal displacement and velocity, \( c_r \) and \( d_r \), the modeshape components at the observed degrees of freedom \( \{ \phi_{ir}, i = 1,\ldots,N_o \} \), and the input participation factors \( \{ p_{ik}, k = 1,\ldots,N_I \} \); together with the pseudostatic influence coefficients \( \{ r_{ik}, i = 1,\ldots,N_o, k = 1,\ldots,N_I \} \). The latter parameters may be fixed on a theoretical basis in some situations (e.g. for the pseudostatic response due to rocking and translation of the base of a building). Only the modeshape components at the observed degrees of freedom can be identified since the “missing” modeshape components at the unobserved degrees of freedom cannot be identified directly without introducing a structural model as a basis for the “interpolation.”

The model parameters \( \alpha \) are estimated by minimizing the mean square of the prediction errors at all the observed degrees of freedom, that is:

\[
J(\alpha) = \frac{1}{N_o N} \sum_{i=1}^{N_o} \sum_{n=1}^{N} (\hat{y}_i(n) - x_i(n; \alpha))^2
\]  \hspace{1cm} (14)

Typically, the discrete system output \( \{ \hat{y}_i(n): n = 1,\ldots,N; i = 1, \ldots, N_o \} \) in (13) consists of measured acceleration time histories at the \( N_o \) observed degrees of freedom for some sampling interval \( \Delta t \). The model output \( x_i(n; \alpha) \) in (13) is a nonlinear function of the parameters and so the minimization of \( J(\alpha) \) must be done numerically by an iterative optimization algorithm. The algorithm used in the MODE-ID program is a robust one exploiting the linearity of the model dynamics (Beck, 1978; Werner et al, 1987).
Available Seismic Records

Several earthquake records were obtained from woodframe buildings instrumented by the California Strong Motion Instrumentation Program (for earthquake time histories, see Appendix A). This section describes each record, the building in which it was recorded and the identified modal system parameters. The results are summarized in Table 2 at the end of this subsection.

San Bernardino – 3-Story Motel

Three sets of earthquake records obtained at this site were analyzed. This building is highly irregular, forming an asymmetrical T. Built in 1986, the building had plywood shearwalls in the first story along the transverse directions and gypsum board on the upper stories and along the longitudinal directions.

Figure 1: San Bernardino – 3-Story Motel

Elevation

1st Floor Plan
There were nine channels in the N-S direction, five in the E-W direction, and one channel recording vertical motion. Channels 2 and 3 were taken as the excitation when using MODE-ID (See Figure 1). The three earthquakes recorded included two near field earthquakes (epicentral distance less than 1 km). The horizontal maximum accelerations generated in the structure were 9.2%g (from the $M_L = 4.2$ magnitude, June 28, 1997 earthquake), 7.8%g (from the $M_L = 3.7$ magnitude, July 26, 1997 earthquake), and 7.1%g (from the $M_L = 4.7$ magnitude, March 11, 1998 earthquake).

Modal analysis of these earthquake records has revealed that the structure’s average fundamental periods (frequencies) are 0.19 sec (5.2 Hz) in the N-S direction and 0.21 sec (4.7 Hz) in the E-W direction. Detailed analysis of the earthquake records showed a gradual elongation in the natural periods of vibration, which was longest at the time of strongest shaking and which gradually returned to the original values (See Appendix B).

The average fundamental damping ratio was 12.0% in the N-S direction and 11.8% in the E-W direction. Damping ratios gradually increased during the course of the earthquake and were highest at the time of strongest shaking, after which they gradually returned to their original.

Parkfield – Elementary School

Two sets of earthquake records obtained at this site were analyzed. This is a one-story, rectangular building built in 1949, with plywood shear walls in the longitudinal direction.

There were three channels in the N-S (transverse) direction and three in the E-W (longitudinal) direction. Channels 3 and 6 were taken as the excitation when using MODE-ID (See Figure 2). The two earthquakes were both within 10 km from the site. The horizontal maximum accelerations generated in the structure were 12.3%g (from the $M_L = 4.2$ magnitude, April 4, 1993 earthquake) and 20.1%g (from the $M_L = 4.7$ magnitude, December 20, 1994 earthquake).

Modal analysis of these earthquake records has revealed that the structure’s average fundamental periods (frequencies) are 0.12 sec (8.3 Hz) in the N-S direction and 0.14 sec (6.9 Hz) in the E-W direction. Detailed analysis of the earthquake records showed a gradual elongation in the natural periods of vibration, which was longest at the time of strongest shaking and which gradually returned to the original values (See Appendix B).

The average fundamental damping ratio was 14.7% in the N-S direction and 11.2% in the E-W direction. Note that the higher damping ratio is in the N-S direction, which has considerably more shear wall length. Damping ratios gradually increased during the course of the earthquake and were highest at the time of strongest shaking, after which they gradually returned to their original value.
Figure 2:  
Parkfield – Elementary School

One set of earthquake records obtained at this site was analyzed. This site is a one-story, rectangular fire station with large door openings on both N-S walls. Built in 1983, the structure has plywood shear walls along building perimeter with gypsum board interior finish.

There were three channels in the N-S (transverse) direction and three in the E-W (longitudinal) direction. Channels 1 and 2 were taken as the excitation when using MODE-ID (See Figure 3). The earthquake of May 17, 1993, recorded at this site, had magnitude $M_L = 6.0$. The horizontal maximum acceleration generated in the structure was $4.4\%g$.

Modal analysis of this earthquake record has revealed that the structure’s fundamental periods (frequencies) are 0.18 sec (5.6 Hz) in the N-S direction and 0.11 sec (8.7 Hz) in the E-W direction. Note that the longer period is in the N-S direction, which has considerably less shear walls than the E-W direction. Detailed analysis of the earthquake records showed a gradual elongation in the natural periods of vibration, which was longest at the time of strongest shaking and which gradually returned to the original values (See Appendix B).
The damping ratio is 7.0% in the N-S direction and 12.2% in the E-W direction. Note that the higher damping ratio is in the E-W direction, which has considerably more shear wall length. Damping ratios gradually increased during the course of the earthquake and were highest at the time of strongest shaking, after which it gradually returned to the original value.

**Eureka – 2-Story Office Building**

One set of earthquake record obtained at this site was analyzed. This site is a two-story building built in 1992 with plywood shear walls along perimeter walls and at interior wall on first floor, and with gypsum board interior wall on second floor.

There were three channels in the N-S (longitudinal) direction and three in the E-W (transverse) direction. Channels 2 and 3 were taken as the excitation when using MODE-ID (See Figure 4). The earthquake of February 8, 1995, recorded at this site, had magnitude $M_L = 3.9$. The horizontal maximum acceleration generated in the structure was 6.2%g.
Modal analysis of this earthquake record has revealed that the structure’s fundamental periods (frequencies) are 0.17 sec (5.8 Hz) in the N-S direction and 0.20 sec (4.9 Hz) in the E-W direction. Note that the longer period is in the E-W direction, which has less shear walls than the N-S direction. Detailed analysis of the earthquake records showed a gradual elongation in the natural periods of vibration, which was longest at the time of strongest shaking and which gradually returned to the original values (See Appendix B).

The damping ratio is 16.5% in the N-S direction and 14.9% in the E-W direction. Note that the higher damping ratio is in the N-S direction, which has more shear wall length than the E-W direction. Damping ratios gradually increased during the course of the earthquake and were highest at the time of strongest shaking, after which it gradually returned to the original value.
**Indio – 1-Story Hospital**

One set of earthquake records obtained at this site was analyzed. This site is a one-story building built in 1981 with plywood shear walls distributed along the first floor.

There were three channels in the N-S direction and three in the E-W direction. Channels 1 and 2 were taken as the excitation when using MODE-ID (See Figure 5). The earthquake of July 26, 1997, recorded at this site, had magnitude $M_L = 4.9$. The horizontal maximum accelerations generated in the structure were $8.3\%g$.

Modal analysis of this earthquake record has revealed that the structure’s fundamental periods (frequencies) are $0.14$ sec ($7.1$ Hz) in the N-S direction and $0.13$ sec ($7.9$ Hz) in the E-W direction. Detailed analysis of the earthquake records showed a gradual elongation in the natural periods of vibration, which was longest at the time of strongest shaking and which gradually returned to the original values (See **Appendix B**).

The average damping ratio was $6.3\%$ in the N-S direction and $8.9\%$ in the E-W direction. Damping ratios gradually increased during the course of the earthquake and were highest at the time of strongest shaking, after which it gradually returned to the original value.
Table 2:
Summary of Building Dynamic Characteristics
From Earthquake Records

<table>
<thead>
<tr>
<th></th>
<th>San Bernardino</th>
<th>Parkfield</th>
<th>Bishop</th>
<th>Indio</th>
<th>Eureka</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Height</td>
<td>29.9’</td>
<td>13.2’</td>
<td>17’</td>
<td>13.7’</td>
<td>26.0’</td>
</tr>
<tr>
<td></td>
<td>(top of roof)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length:</td>
<td>180.5’</td>
<td>48’</td>
<td>62’</td>
<td>298’</td>
<td>80’</td>
</tr>
<tr>
<td>(Longit.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>132’</td>
<td>30’</td>
<td>50’</td>
<td>148’</td>
<td>54’</td>
</tr>
<tr>
<td>(Transv.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5/17/93</td>
<td>7/25/97</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2/8/95</td>
<td></td>
</tr>
<tr>
<td>Magnitude (M&lt;sub&gt;L&lt;/sub&gt;)</td>
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<td>3.7</td>
<td>4.5</td>
<td>4.2</td>
<td>4.7</td>
</tr>
<tr>
<td>Peak Response</td>
<td>9.2%g</td>
<td>7.8%g</td>
<td>7.1%g</td>
<td>12.3%g</td>
<td>20.1%g</td>
</tr>
<tr>
<td>Total Drift (mm):</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
<td>0.5</td>
<td>0.9</td>
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<tr>
<td>(Longit.)</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>0.7</td>
<td>0.7</td>
<td>0.5</td>
<td>1.5</td>
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<tr>
<td>(Roof w.r.t. Base)</td>
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<td></td>
<td></td>
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<tr>
<td>Periods (sec):</td>
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<td>0.20</td>
<td>0.23</td>
<td>0.14</td>
<td>0.15</td>
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<tr>
<td>(Longit.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.19</td>
<td>0.21</td>
<td>0.18</td>
<td>0.11</td>
<td>0.13</td>
</tr>
<tr>
<td>(First Mode)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frequency (Hz):</td>
<td>4.6</td>
<td>5.0</td>
<td>4.4</td>
<td>7.3</td>
<td>6.6</td>
</tr>
<tr>
<td>(Longit.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.4</td>
<td>4.8</td>
<td>5.6</td>
<td>8.7</td>
<td>8.0</td>
</tr>
<tr>
<td>(First Mode)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Damping Ratio (%)</td>
<td>13.6</td>
<td>14.1</td>
<td>7.7</td>
<td>11.6</td>
<td>10.8</td>
</tr>
<tr>
<td>(Longit.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>17.3</td>
<td>6.9</td>
<td>11.7</td>
<td>14.2</td>
<td>15.3</td>
</tr>
<tr>
<td>(First Mode)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Discussion of Results from the Seismic Records

The results of applying MODE-ID to the 8 sets of building records are summarized in Table 2. Note that the damping levels are higher than initially expected, but they are consistent with the damping levels exhibited in the UCSD shaking table tests of Task 1.1.1 (Filiatrault et. al., 2001). The fundamental frequency estimates from different earthquakes for the San Bernardino and Parkfield buildings are quite consistent, as expected, since the drifts are at similar levels and it is known from experience that the uncertainty in the estimates of the fundamental frequencies is relatively small if the drift levels are comparable from test to test. On the other hand, the fundamental damping estimates vary significantly for the San Bernardino building. This may be partly because the instrumentation layout does not allow the excitation of the structure to be well captured. However, experience has shown that the uncertainty in the damping estimates is relatively large even when the excitation is well defined; it is thought that this is partly because the assumed linear viscous damping is not a good model for the actual damping mechanisms and partly because the seismic response of the model is not very sensitive to changes in the damping level. The periods listed in Table 2 have been used in developing this Task’s period regression formula based on structural height that is presented later in this report.
Shake Table Tests

UC San Diego – 2-Story House

Thirteen sets of records obtained from shake table tests were analyzed (testing performed under Task 1.1.1 of this project, see Filiatrault et. al., 2001). The records selected correspond to seismic tests at levels 1, 2, 3, 3r, 4 and 5 for Phase 9 testing (no stucco), and at levels 1, 2, 3, 4, 4r, 5 and 5r for Phase 10 testing (exterior stucco) of the 2-story house. Channels D1, D2 and D3 were taken as input in direction of shake table motion (NS), and channels E1 and E3 were taken as input in direction perpendicular to shake table motion (EW). The output channels selected were D9, D11, D13 in NS direction and E4, E6 in EW direction at 2nd floor, and D14, D16, D18 in NS direction and E7, E9 in EW direction at roof. For complete testing details, please see Filiatrault et. al, 2001.

Figure 6A shows a comparison between the frequencies and dampings obtained from MODE-ID and that obtained from the frequency and damping tests performed by UCSD for the Phase 9 structure. Figure 6B shows the same comparison for the phase 10 structure. Note that MODE-ID and the UCSD logarithmic decrement method compute the equivalent viscous damping for the structure and account for hysteretic damping. However, the UCSD damping values are for much smaller amplitude levels and therefore the hysteretic damping contribution will be smaller. Figures 6C through F compare the recorded time histories for channel D16 during shaking levels 1 and 5 to the predicted response based on the MODE-ID modal parameters and on the UCSD frequency and damping. The MODE-ID analysis is presented in Appendix C.

Discussion of Results from the Shake Table Tests

In Figures 6A and 6B, note that the repeated tests performed with the same input time history for the shake table excitation (3+3r during Phase 9, and 4+4r, 5+5r during Phase 10) yield different results. The lower fundamental frequency values can presumably be attributed to “damage” (i.e. permanent loss of stiffness) in the previous test with the same input. It should be noted that the UCSD frequency and damping values were obtained from vibrations at smaller amplitudes (0.025g to 0.04g RMS input amplitude for frequency tests, and peak response at roof typically around 0.05g for damping tests) than those observed during the seismic tests. Therefore it is reasonable that the MODE-ID identified frequencies obtained from the higher amplitude seismic tests are considerably lower than those obtained from low-level white noise shaking. Similarly, the MODE-ID identified damping ratios are considerably higher at stronger shaking.

In Figures 6C through 6F, it is important to emphasize that the dynamic characteristics obtained from UCSD’s frequency and damping tests were at low level shaking (white noise input and free vibration), and the dynamic characteristics obtained from MODE-ID give the parameters for a linear dynamic model which best fits the recorded response, and therefore the predicted response based on the MODE-ID frequency and damping values is more accurate.
Figure 6A: UCSD 2-Story House: Phase 9

### Seismic Level vs. Fundamental Frequency (Hz)

**Task 1.1.1 Phase 9**

<table>
<thead>
<tr>
<th>Seismic Level</th>
<th>Fundamental Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>4.16</td>
</tr>
<tr>
<td>0.05</td>
<td>3.58</td>
</tr>
<tr>
<td>0.10</td>
<td>2.88</td>
</tr>
<tr>
<td>0.15</td>
<td>2.63</td>
</tr>
<tr>
<td>0.20</td>
<td>2.23</td>
</tr>
<tr>
<td>0.25</td>
<td>1.88</td>
</tr>
</tbody>
</table>

**MODE-ID**

- 1: 4.16
- 2: 3.58
- 3: 2.88
- 3r: 2.63
- 4: 2.23
- 5: 1.88

**UCSD**

- 1: 3.91
- 2: 3.71
- 3: 3.66
- 3r: 3.42
- 4: 2.93
- 5: 2.93

### Seismic Level vs. Damping Ratio

**Task 1.1.1 Phase 9**

<table>
<thead>
<tr>
<th>Seismic Level</th>
<th>Damping Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>7.4%</td>
</tr>
<tr>
<td>0.05</td>
<td>20.1</td>
</tr>
<tr>
<td>0.10</td>
<td>19.2</td>
</tr>
<tr>
<td>0.15</td>
<td>15.7</td>
</tr>
<tr>
<td>0.20</td>
<td>16.3</td>
</tr>
<tr>
<td>0.25</td>
<td>15.1</td>
</tr>
</tbody>
</table>

**MODE-ID**

- 1: 4.3%
- 2: 4.2%
- 3: 3.9%
- 3r: 6.2%
- 4: 7.2%
- 5: 8.7%

**UCSD**

- 1: 4.3%
- 2: 4.2%
- 3: 3.9%
- 3r: 6.2%
- 4: 7.2%
- 5: 8.7%

### Seismic Level vs. PGA (g)

**Task 1.1.1 Phase 9**

**MODE-ID Results**

- 0.00: 0.2
- 0.05: 0.4
- 0.10: 0.6
- 0.15: 0.8
- 0.20: 1

### Seismic Level vs. Equivalent Damping Ratio

**Task 1.1.1 Phase 9**

**MODE-ID Results**

- 0.00: 0.2
- 0.05: 0.4
- 0.10: 0.6
- 0.15: 0.8
- 0.20: 1

---

Construction of Period and Damping Database | 23
Figure 6B:
UCSD 2-Story House: Phase 10

Task 1.1.1 Phase 10

MODE-ID Results

- PGA (g)
- Fundamental Frequency (Hz)

- Damping Ratio
- Equivalent Damping Ratio
Figure 6C:
UCSD 2-Story House: Phase 9

Actual Response for Channel D16 – Phase 9 Level 1

Predicted Response for Channel D16 (MODE-ID Freq. & Damp.)– Phase 9 Level 1

Predicted Response for Channel D16 (UCSD Freq. & Damp.)– Phase 9 Level 1
Figure 6C:  
UCSD 2-Story House: Phase 9

Actual Response for Channel D16 – Phase 9 Level 5

Predicted Response for Channel D16 (MODE-ID Freq. & Damp.) – Phase 9 Level 5

Predicted Response for Channel D16 (UCSD Freq. & Damp.) – Phase 9 Level 5
Figure 6D: UCSD 2-Story House: Phase 10

Actual Response for Channel D16 – Phase 10 Level 1

Predicted Response for Channel D16 (MODE-ID Freq. & Damp.) – Phase 10 Level 1

Predicted Response for Channel D16 (UCSD Freq. & Damp.) – Phase 10 Level 1
Figure 6D:
UCSD 2-Story House: Phase 10

Actual Response for Channel D16 – Phase 10 Level 5

Predicted Response for Channel D16 (MODE-ID Freq. & Damp.)– Phase 10 Level 5

Predicted Response for Channel D16 (UCSD Freq. & Damp.)– Phase 10 Level 5
Ambient Vibration Tests

Ambient vibration tests were performed on a number of houses and condominiums in the Los Angeles area. These tests measured naturally occurring ambient vibrations induced by wind, traffic, and/or other sources. The results are summarized in Table 3.

Analysis of ambient vibration data consisted of examining the Fast Fourier Transform of the recorded time histories of duration 60 seconds each and of processing the data using MODE-ID. The FFT method provided information regarding the frequency content of the data and was especially helpful in setting up for the forced vibration tests. In order to use MODE-ID, first the data was cross-correlated with a reference channel because the theoretical cross-correlations for a linear system satisfy the equation of motion for free vibrations with the time lag as the pseudo-time (Beck et. al., 1995). Then MODE-ID was used to analyze the cross-correlated data as if it was free vibrations. See Figure 7 for a sample plot of empirical and identified (i.e. best fit) cross-correlation functions (it should be noted that, at longer time lags, empirical cross-correlation functions are poor estimates of the theoretical ones), and Figures 8A and 8B for examples of ambient vibration FFT plots.

Table 3: Summary of Building Parameters Dynamic Characteristics From Ambient Vibration Tests

<table>
<thead>
<tr>
<th>Test Site</th>
<th>Building Height</th>
<th>Test Date</th>
<th>1st Mode Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-Story House (South Pasadena)</td>
<td>13'</td>
<td>02-May-00</td>
<td>10.9</td>
</tr>
<tr>
<td>1-Story House 95th st. (Los Angeles)</td>
<td>10'</td>
<td>09-May-00</td>
<td>13.0</td>
</tr>
<tr>
<td>1-Story House/Office (Los Angeles)</td>
<td>10'</td>
<td>09-May-00</td>
<td>12.7</td>
</tr>
<tr>
<td>1-Story House 99th st. (Los Angeles)</td>
<td>10'</td>
<td>11-May-00</td>
<td>12.7</td>
</tr>
<tr>
<td>2-Story House (South Pasadena)</td>
<td>20'</td>
<td>25-Apr-00</td>
<td>9.0</td>
</tr>
<tr>
<td>2-Story House: S. Catalina Ave. (Pasadena)</td>
<td>20'</td>
<td>23-Jun-00</td>
<td>6.5</td>
</tr>
<tr>
<td>3-Story Townhouse (Pasadena)</td>
<td>30'</td>
<td>13-Apr-00</td>
<td>5.1</td>
</tr>
<tr>
<td>3-Story Apartment Building (Pasadena)</td>
<td>30'</td>
<td>07-Jul-00</td>
<td>4.5</td>
</tr>
</tbody>
</table>
Discussion of Results from Ambient Vibration Tests

The results of applying MODE-ID to the ambient vibration test data are summarized in Table 3. In a plot of fundamental periods against building height, the ambient vibration results are significantly below periods estimated from the earthquake records and forced vibration tests (see Figure 27). This is consistent with the drifts being much smaller for the ambient vibration tests, which yielded drifts 100 times lower than those obtained from the earthquake analyses and forced vibration tests (drift ratios in the order of 0.000001). Also, because the excitation is not measured in ambient vibration tests, the uncertainties in the damping estimates are too large for these estimates to be meaningful. Thus, the main value of the ambient vibration tests is when they precede forced vibration tests and can give the frequency range to investigate for resonance.

Figure 7:
Plot of a Cross-Correlation Function from Ambient Vibration Data
3-Story Townhouse (Pasadena)
Each window shows a plot of the Fast-Fourier Transform for the recorded time history at each of the channels. For channel location, see Figure 11.
Figure 8B:
Example of Ambient Vibration Survey FFT
3-Story Apartment Building (Pasadena)

Each window shows a plot of the Fast-Fourier Transform for the recorded time history at each of the channels. For channel location, see Figure 20.
Forced Vibration Tests

The purpose of the forced vibration tests was to fill gaps in the data obtained from the analysis of the earthquake records, providing a more reliable regression analysis result. These tests measured harmonic vibrations induced by a shaking machine in a 3-story and two 2-story woodframe buildings. The results are summarized in Table 4 at the end of this subsection. For two of the buildings, the naturally occurring ambient vibrations were also measured and compared the ambient data analysis results to that of the forced vibration analysis results.

The force delivered by the shaking machines is generated by the centrifugal acceleration of the weights, so it is proportional to the frequency squared and also to the weight eccentricity around the rotating shafts (see Figure 9). Ranger seismometers (voltage output proportional to velocity response) and accelerometers (output proportional to acceleration response) were used to sample the building response at each driving frequency, then a least-squares fit was made of a sinusoidal curve to the time history data to obtain the best approximation to the response amplitude, frequency and phase shift (amplitudes were normalized by the driving frequency squared to account for the increase in force between frequency samplings). Finally, by plotting the frequency-response curves for all data channels, the modal frequencies of the building were identified and the corresponding damping ratios were computed by using a curve-fitting approach involving nonlinear least-squares matching of the model and measured frequency response amplitudes.

Finding buildings suitable for forced vibration testing was not an easy task because of the potential for some cosmetic damage. Test candidates were limited to buildings scheduled for demolition or buildings whose owners were not concerned about any cosmetic damage that might occur. The L.A. City Department of Airports offered a listing of woodframe buildings owned by L.A. City that are in the airport’s flight path and which are scheduled for demolition or relocation to a different site. The list includes many single-story houses and multi-story apartment buildings. However, during the time window for the forced vibration tests, only one house was available for shaking. The Nevada Test Site (Nevada Testing Institute) also offered single-story warehouse buildings with plan areas typically over 2000 square feet, but due to time constraints this site could not be utilized. The best-suited test candidates were offered by the California Institute of Technology, which owns several buildings in the vicinity of the campus. All forced vibration tests performed are described next.
1-Story House (Los Angeles)

This site was scheduled for demolition by the L.A. City Department of Airports. This was a single-family residence on cripple walls, built circa 1940, with plan area of approximately 800 square feet.

At this site this task’s first forced vibration test was performed using the Caltech shaker, which had never previously been used on woodframe structures. It was difficult to use the Caltech shaker due to its high profile, which induced large overturning moments. It was a challenge to secure the machine to the hardwood floors to accommodate the large rocking moments as the shaking level increased, which required keeping the forcing frequency below 6 Hz since higher frequencies induced severe rocking of the machine. Unfortunately this frequency was well below the building’s natural frequency, estimated around 9 Hz. Although no shaking data could be obtained at this site, valuable insight was gained regarding the logistics of shaking woodframe buildings, specially regarding the connection of the shaking machines onto flexible flooring.
This two-story house, owned by the California Institute of Technology and located in the vicinity of the campus, is being used as an undergraduate dormitory (See Figures 10 through 12). It is built on cripple walls, and built circa 1940, with first floor plan area nearing 2000 square feet. There is a stairway leading to the roof attic. It has two brick fireplaces, which have been seismically retrofitted and anchored to the roof. The building has exterior wood shingles, the interior finish is stucco, and the original hardwood floors are still in place.

This building was shaken successfully using a shaker borrowed from Harvey Mudd College (See Figure 9), which has a much lower profile and therefore presented significantly less rocking instability. The equipment was setup for shaking at the second floor level, placing the shaker at the top of the main stairway over a sheet of plywood and wedged between planks of wood, which were then screwed into the plywood below. The intent was to avoid any damage to the carpeting beneath, so the assembly could not be bolted to the floor. Instead, the shaker assembly was fixed between the landing walls by squeezing it into place, in order to provide maximum shear transfer to the building. See Figure 13 for a picture of the shaker setup. Six seismometers and ten accelerometers were located as shown in Figures 11 and 12 respectively.

Ambient vibration tests prior to the shaking in order to determine the frequency range to be used and also to observe any shift in fundamental frequency due to loss of non-structural stiffness at stronger shaking amplitudes. A frequency scan was then performed, where the shaker winds down from high shaking frequencies while the response time histories are monitored in order to visually identify the resonant frequencies.

During the first day of testing, the shaker weights were placed at 2.5% eccentricity. The building response was recorded for shaking in the East-West direction between 5.0 Hz and 12.6 Hz, at 0.2 Hz increments, and between 4.6 Hz and 10.0 Hz for the North-South direction, also at 0.2 Hz increments. During the next day of testing, the building was again shaken at 2.5% eccentricity and the building response was re-recorded for shaking between 4.5 Hz and 10.0 Hz in each direction, this time at 0.1 Hz increments. Next, the shaker eccentricity was raised to 10% to increase the force levels by a factor of 4, and the building was shaken in the frequency range between 4.0 Hz and 7.0 Hz in the East-West direction. Finally, the shaker eccentricity was raised to 20%, a further doubling of the force levels, and the response was recorded for shaking between 4.0 Hz and 7.0 Hz also in the East-West direction and with 0.1 Hz increments. Each recording was taken for 5 seconds at 1000 Hz sampling frequency.

Analysis of the data obtained from the ambient vibration surveys and frequency scans consisted of taking the Fast Fourier Transform of the measured time histories, in order to observe the frequency content of the building response. This analysis was done during the testing, and it helped determine the frequency range for the shaking. See Figure 14 for a plot of the frequency scan FFTs, which can be compared with the corresponding ambient vibration FFT plot in Figure 7 (note the shift in frequency content that can be observed in all the channels).
Plots of the test results can be found in **Figures 15 through 17**. **Figure 18** shows a sample of the comparison of the identified model and test frequency-response curves. The identified fundamental modal frequencies and damping ratios are presented in **Table 4** at the end of this subsection where the results are discussed.

**Figure 10:**
2-Story House (Pasadena)
Figure 11:
2-Story House (Pasadena)
Location of Seismometers (6/27/00)

Arrows indicate seismometer locations with corresponding channel numbers (dotted arrows indicate seismometers were placed in the attic).
Figure 12:
2-Story House (Pasadena)
Location of Accelerometers (6/27/00)

Arrows indicate accelerometer locations with corresponding channel numbers (dotted arrows indicate accelerometers were placed in the attic).
Figure 13:
2-Story House (Pasadena)
Experimental Setup
Each window shows a plot of the Fast-Fourier Transform for the recorded time history at each of the channels. For channel location, see Figure 11.
This is a plot of the seismometer channel response amplitude (proportional to velocity) at each forcing frequency (in increments of 0.1 Hz). The amplitudes are normalized by the square of the frequency. Odd number channels face North, even number channels face West. For channel location, see Figure 11.
Figure 16:
2-Story House (Pasadena)
Forced Vibration tests (6/27/00) -- Seismometers

This is a plot of the seismometer channel response amplitude (proportional to velocity) at each forcing frequency (in increments of 0.1 Hz). The amplitudes are normalized by the square of the frequency. Odd number channels face North, even number channels face West. For channel location, see Figure 11.
This is a plot of the accelerometer channel response amplitude (proportional to acceleration) at each forcing frequency (in increments of 0.1 Hz). The amplitudes are normalized by the square of the frequency. For channel location, see Figure 12.
The test data plotted is for EW seismometer channels 4, 6, and 2 (from highest to lowest amplitude) during EW shaking at 20% eccentricity. For channel location, see Figure 12. “Jnormal” is the mean squared error normalized by the mean squared frequency response from all channels.
3-Story Apartment Building (Pasadena)

This apartment building is owned by the California Institute of Technology and is located in the vicinity of the campus (see Figures 19 and 20). It is currently being used as graduate student apartments. It was built circa 1960. There is an underground parking garage with concrete shear walls below ground level around three sides and the East side of the garage is open. The first floor plan area is approximately 5000 square feet, with an aspect ratio of approximately 3:1. The student apartments are located at the first and second floor levels, and there is a penthouse apartment occupying the third floor. The exterior wall finish is stucco, the interior finish is plaster on drywall, and the flooring is a soundproofing topping (probably lightweight concrete) over sheathing. The shaker was placed at the third floor (see Figure 20).

This building was shaken successfully using the Harvey Mudd shaker, which was placed over a sheet of plywood and wedged between wood planks. Screws then were driven through the planks, the plywood and the floor deck below to secure the shaker assembly. See Figure 20 for the seismometer locations and Figure 21 for a picture of the experimental setup.

Figure 19:
3-Story Apartment Building

North-East view of building (note underground garage).
Arrows indicate seismometer locations with corresponding channel numbers.
The testing procedure was similar to that at the 2-story house. During the first day of testing, the shaker weights were placed at 2.5% eccentricity. The building response was recorded for shaking between 3.6 Hz and 13.0 Hz, at 0.2 Hz increments, in each direction. During the next day of testing, the building was again shaken at 2.5% eccentricity and the building response was re-recorded for shaking between 4.5 Hz and 10.0 Hz in each direction, this time at 0.1 Hz increments. Next, the shaker eccentricity was raised to 10% and the building was shaken in the frequency range between 4.0 Hz and 7.0 Hz in the East-West direction. Finally, the shaker eccentricity was raised to 20% and response recorded for shaking between 4.0 Hz and 7.0 Hz also in the East-West direction, again with 0.1 Hz increments. Each recording was taken for 5 seconds at 1000 Hz sampling frequency.

Plots of the results obtained can be found in Figures 22 through 24. Figure 25 shows a sample of the comparison of the identified model and test frequency-response curves. The identified fundamental modal frequencies and damping ratios are presented in Table 4.
Figure 22:  
3-Story Apartment Building (Pasadena)  
Forced Vibration tests (7/7/00)

This is a plot of the channel response amplitude at each forcing frequency (in increments of 0.1 Hz). The amplitudes were normalized by the square of the frequency. Not every channel has been plotted in each direction. For channel location, see Figure 20.
This is a plot of the channel response amplitude at each forcing frequency. The amplitudes were normalized by the square of the frequency. Not every channel has been plotted in each direction. For channel location, see Figure 20.
Figure 24:
3-Story Apartment Building (Pasadena)
Forced Vibration tests (7/10/00)

This is a plot of the channel response amplitude at each forcing frequency. The amplitudes were normalized by the square of the frequency. Not every channel has been plotted in each direction. For channel location, see Figure 20.
Figure 25: 3-Story Apartment Building (Pasadena)

The test data plotted is for seismometer channels E3, E2, E1, N4, N5 (from highest to lowest amplitude) on upper plot, and for seismometer channels E2, E1, N6, N7 (from highest to lowest amplitude) on lower plot. For channel location, see Figure 20. “$J_{\text{normal}}$” is the mean squared error normalized by the mean squared frequency response from all channels.
2-Story Office Building (Pasadena)

This building is owned by the California Institute of Technology and is located in the vicinity of the campus. It was under construction at the time of the testing, with all plywood sheathing, asphalt and tile roofing, and concrete flooring in place, but no interior or exterior finishes (no plaster or stucco at time of testing). This is a rectangular building with steel ridge beam and center columns, and a small steel moment frame supporting the East end of the ridge beam due to window openings. There are no interior shear walls (all interior walls are drywall partitions on metal studs). The first floor plan area is approximately 5600 square feet (140’x40’). See Figure 26 for a sketch of the floor plan and seismometer locations and Figure 27 for a picture of the building.

The Harvey Mudd shaker was placed at the second floor, over the bare concrete flooring with enough weights over the shaker to prevent sliding and rocking. During the first day of testing, the shaker weights were placed at 5% eccentricity. The building response was recorded for shaking in the North-South direction (transverse direction), for a frequency range between 4.5 Hz and 13.0 Hz, at 0.1 Hz increments. During the next day of testing, a few channels were lost due to damage to one of the signal conditioners, and the testing had to be finished with only 3 out of 7 channels working properly. The building was shaken at 5% eccentricity in the East-West direction (longitudinal direction) for the same frequency range, and then the shaker eccentricity was increased to 10% and the building was shaken between 4.5 Hz and 7.6 Hz in the East-West direction with 0.1 Hz increments. Each recording was taken for 5 seconds at 1000 Hz sampling frequency.

Figures 28 and 29 show plots of the results. The identified fundamental modal frequencies and damping ratios are presented in Table 4.
Figure 26:  
2-Story Office Building (Pasadena)  
Location of Seismometers

Arrows indicate seismometer locations with corresponding channel numbers. Shaker location has been labeled. At the time of testing, perimeter walls have plywood sheathing on exterior but no interior sheathing, all interior walls have gypsum wallboard.
Figure 27:
2-Story Office Building (Pasadena)
(photo taken on 5/21/2001)
This is a plot of the channel response amplitude at each forcing frequency. The amplitudes were normalized by the square of the frequency. Not every channel has been plotted in each direction. For channel location, see Figure 26.
This is a plot of the channel response amplitude at each forcing frequency. The amplitudes were normalized by the square of the frequency. Not every channel has been plotted in each direction. For channel location, see Figure 26.
Table 4:
Summary of Building Parameters And Dynamic Characteristics
From Forced Vibration Tests

<table>
<thead>
<tr>
<th>Test Site</th>
<th>Building Height</th>
<th>Test Date</th>
<th>Ambient / Forced (shaker used)</th>
<th>Shaker Eccentricity</th>
<th>Direction of Shaking</th>
<th>1st Mode Frequency (Hz)</th>
<th>1st Mode Damping Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-Story House (Pasadena)</td>
<td>20’</td>
<td>June 23, 2000</td>
<td>AVS</td>
<td>--</td>
<td>--</td>
<td>7.8</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>2.5%</td>
<td>Transv. (EW)</td>
<td>5.7</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>2.5%</td>
<td>Longit. (NS)</td>
<td>5.6</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td></td>
<td>June 27, 2000</td>
<td>HMC Shaker</td>
<td>2.5%</td>
<td>Longit. (NS)</td>
<td>5.5</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>2.5%</td>
<td>Transv. (EW)</td>
<td>5.7</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>10%</td>
<td>Transv. (EW)</td>
<td>5.2</td>
<td>5.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>20%</td>
<td>Transv. (EW)</td>
<td>5.2</td>
<td>4.9</td>
</tr>
<tr>
<td>3-Story Apartment (Pasadena)</td>
<td>30’</td>
<td>July 7, 2000</td>
<td>AVS</td>
<td>--</td>
<td>--</td>
<td>5.5</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>2.5%</td>
<td>Longit. (NS)</td>
<td>5.3</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>2.5%</td>
<td>Transv. (EW)</td>
<td>--</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>2.5%</td>
<td>Longit. (NS)</td>
<td>5.3</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td></td>
<td>July 10, 2000</td>
<td>HMC Shaker</td>
<td>2.5%</td>
<td>Transv. (EW)</td>
<td>--</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>10%</td>
<td>Longit. (NS)</td>
<td>5.2</td>
<td>4.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>20%</td>
<td>Longit. (NS)</td>
<td>5.1</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>20%</td>
<td>Transv. (EW)</td>
<td>--</td>
<td>4.2</td>
</tr>
<tr>
<td>2-Story Office (Pasadena)</td>
<td>20’</td>
<td>Sept. 9, 2000</td>
<td>HMC Shaker</td>
<td>5%</td>
<td>Transv. (NS)</td>
<td>--</td>
<td>7.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>HMC Shaker</td>
<td>5%</td>
<td>Longit. (EW)</td>
<td>6.7</td>
<td>7.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sept. 10, 2000</td>
<td>HMC Shaker</td>
<td>10%</td>
<td>Longit. (EW)</td>
<td>6.6</td>
<td>7.0</td>
</tr>
</tbody>
</table>

Discussion of Results from Forced Vibration Tests

The EW and NS fundamental frequencies of the 2-story house are very close together, and Figures 15, 16 and 17 show that these modes are coupled: all channels are excited in both modes as observed from the double peaks, one at each resonant frequency. Figure 15 shows that the fundamental mode in the EW direction has a torsional component in the mode-shape, exciting all channels. The fundamental mode in the NS direction has a similar effect. Note that seismometer channel 1 (NS), which was placed near the West fireplace, has a resonant peak at 6.4 Hz that does not show up in the other channels (see Figure 15). This peak may be the fundamental frequency of the West chimney since channel 4 (EW), which was placed near the North fireplace, is also excited at this frequency. The identified transverse and longitudinal fundamental frequencies of 5.5 Hz and 5.7 Hz at 2.5% eccentricity are significantly lower than the corresponding fundamental frequency of 6.5 Hz and 7.8 Hz identified from the ambient
vibration survey. Figures 16 and 17 and Table 4 show the downward shift in fundamental frequencies with increasing force amplitude, as stronger shaking reduces stiffness in the non-structural components.

The 3-story apartment building showed strong torsional behavior when shaken in the NS direction due to the lack of walls on the East side of the parking garage. Figures 22, 23 and 24 show that under shaking in the first NS mode, both NS and EW channels have peak amplitudes at the NS fundamental frequency (i.e. there is a strong torsional component to that mode). This shows that the floor diaphragm is undergoing almost pure torsion rigidly as it translates in the NS direction in the first NS mode. Note that the ratio of East side to West side NS motion in the NS mode is large because of the torsional component of the modeshape and because of the lower NS stiffness of the East side compared with that of the West side. This ratio reduces as the shaker eccentricity is increased from 2.5% to 20%. The fundamental EW modeshape has the highest component of EW motion at the mid-span of the building (E3 in Figures 22 and 23), which indicates that the floor is bending in plane as a flexible diaphragm. The EW modeshape also has a torsional component since channels E1 and E2 have significantly different amplitudes (see Figures 22 and 23). It is interesting that the diaphragm behavior is different for shaking in the EW and NS fundamental modes. This is most likely due to the aspect ratio of the floor diaphragm (3:1 in this building). The identified transverse and longitudinal fundamental frequencies of 4.4 Hz and 5.3 Hz at 2.5% eccentricity are close to the corresponding frequencies of 4.5 Hz and 5.5 Hz identified from the ambient vibration survey. Note that the fundamental frequencies of this building were lowered as the shaking amplitude increased (see Figure 24 and Table 4).

Shaking the 2-story office building shows that both NS and EW fundamental frequencies are quite similar. The first NS mode has substantial in-plane bending and rotation of the second floor (see Figure 28), as shown by the large modeshape component at the NS channel 7 (located at the mid-span of the building). In contrast, in the first EW mode, there is very little rotation of the second floor (see Figure 28). In Figure 29 and Table 4, the downward shift in fundamental frequencies with increasing force amplitude is noticeable.

The fundamental frequencies and dampings identified from analysis of the forced vibration test data are summarized in Table 4. The maximum total drifts in the forced vibration tests were computed using the calibrated accelerometer data and found to be 0.54 mm at the 2-story house (transverse shaking at 20% eccentricity) and 0.52 mm at the 3-story apartment building (longitudinal shaking at 20% eccentricity). These drift values for shaking at 20% eccentricity are comparable to the drifts produced by the earthquakes in the database (see Table 2). Figure 30 shows how the periods shifted as a function of the amplitude of shaking force, which is proportional to $0.102*980*\text{eccentricity}^2$. 
The frequencies listed in Table 4 are consistent with those obtained from analysis of the earthquake records (see Table 2) and have been used in developing this Task’s period regression formula based on structural height. The damping ratios were more difficult to obtain. First, the approximate damping values were estimated using the half-power bandwidth of the resonant peaks of the amplitude response curves. Since these values were much lower than those obtained from the analysis of the earthquake data, a more sophisticated curve-fitting approach involving non-linear least-squares matching of the model and test frequency response curves was used. In those cases where there are clearly modes whose frequencies are close, this method is expected to give more accurate damping values. The damping values obtained from the curve-fitting were still significantly lower than those obtained from the earthquake records. The damping values obtained from the forced vibration tests should have been higher than the dampings from earthquake records due to additional energy dissipated by soil-structure interaction expected in forced vibration tests (note that in the analysis of the earthquake records, the input channel is taken at the base of the structure, and therefore the input motion already accounts for the soil-structure interaction). This difference may be due to the fact that the buildings tested differ significantly from those from which the earthquake records were obtained.
Period Formula by Regression Analysis on Database

Methodology

To determine a period formula by regression, a Maximum Likelihood estimation method based on a lognormal distribution for the periods at each value of the selected regressor was used. Thus, a period formula similar to equation (1) is derived from a statistical model of the form:

\[ \ln T = \ln c + \gamma \ln x + s_e^2 \varepsilon \]  

(15)

where \( \ln c \) and \( \gamma \) are parameters to be estimated, the regressor \( x \) is a structural characteristic, \( \varepsilon \) is a Unit Normal random variable (i.e. zero mean and unit variance) and \( s_e^2 \) is the variance in the predicted value of \( \ln T \), taken to be independent of \( x \). The maximum likelihood estimates \( \hat{c} \) and \( \hat{\gamma} \) minimize \( \sum_{i=1}^{N} [\ln T_i - (\ln \hat{c} + \gamma \ln x_i)]^2 \) and the standard error estimate \( \hat{s}_e \) in \( \ln T \) is calculated from:

\[ \hat{s}_e = \sqrt{\frac{\sum_{i=1}^{N} [\ln T_i - (\ln \hat{c} + \gamma \ln x_i)]^2}{(N - 2)}} \]  

(16)

where \( N \) = total number of data points \((T_i, x_i)\) in the period database.

The estimated relationship \( \ln \hat{T} = \ln \hat{c} + \gamma \ln x \) (or, equivalently, \( \hat{T} = \hat{c} x^{\hat{\gamma}} \)) gives the median period for the given regressor value. Curves for the 84\(^{th}\) and 16\(^{th}\) percentiles can be obtained based on an amount \( \hat{s}_e \) above or below the logarithm of the median period (84\% and 16\% probability that data will lay below the respective curves). The equations for these curves are:

\[ T_{16} = \hat{T} e^{\hat{s}_e} \]  

(17)

\[ T_{84} = \hat{T} e^{\hat{s}_e} \]  

(18)
**Period Formula Regressed on Height**

The data obtained from the analysis of the earthquake records and the forced vibration tests was used to perform a regression analysis with respect to the height of the buildings. It was felt that the number of data was insufficient to regress on additional structural characteristics, such as total area of shear walls in the direction of each building axis. Ambient vibration survey results were not used in developing this period formula since the natural periods are significantly lower, and the interest has been in the behavior for stronger shaking of these buildings (See **Figure 31**). The best-fit curve for the median period based on the earthquake records and the strongest shaking in the forced vibration tests can be represented by the following formula:

\[
\hat{T} = 0.032h^{0.55}
\]  

**Figure 31** shows a comparison between the periods found from the earthquake records and forced vibration tests and those given by equations (1a), (1b) and (19). The 16- and 84-percentile curves are given by (17) and (18), where \( \delta_e = 0.129 \) is the standard error in \( \ln T \).
Conclusions

This task has developed an improved period formula based on the periods of woodframe buildings obtained from the analysis of earthquake records and from dynamic tests of these structures. This new period formula is expected to represent the behavior of these structures more accurately than the current UBC formula for miscellaneous woodframe/masonry buildings, and perhaps it is more realistic than the FEMA-273 period formula for moderate shaking levels. Valuable insight was gained regarding the dynamic behavior of these buildings, which showed much shorter natural periods than intuitively expected, between 0.11 and 0.24 sec (4.2 and 8.7 Hz) and high damping ratios, averaging 7.2% and ranging from 2.6% to 17.3%. The median period formula in (19) was derived from low-amplitude shaking (inter-story drifts less than 1.5 mm), since strong earthquake shaking data is not currently available. The periods are expected to be significantly longer for stronger shaking of these structures (see, for example, the increase in fundamental period with increasing amplitude that is apparent in Figure 30 and the increase in periods shown in Figure 31 for UCSD shake table tests). Hopefully, instrumented woodframe structures will provide additional data in future earthquakes and supplement this current database.

Acknowledgments

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The authors are grateful for Professor Ziyad Duron’s generosity in lending the Harvey Mudd shaker. His expertise and that of his research assistants, Joshua Switkes and Annie Tran, were invaluable. Also, thanks to Jay Shih, project engineer responsible for the demolition of the structures in the LAX flight path, who was very generous with his time and helped greatly during the testing at the LAX site. Several faculty, staff and graduate students at Caltech assisted with setting up the testing equipment, taking pictures and preparing the test sites, with special thanks to Dr. Alfred Mason, Ayhan Irfanoglu, Raul Relles, Javier Favela, and Andy Guyader for all their assistance. Finally, many thanks to the CUREE-Caltech advisory panel, whose input and comments were extremely helpful.
References


APPENDIX A
Earthquake Records
### Figure A1:
**List of CDMG/CSMIP Instrumented Woodframe Buildings as of 5/99**

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Station Number</th>
<th>No. of Stories</th>
<th>Floor Dimension</th>
<th>Lateral Force Resisting System</th>
<th>Year of Construction</th>
<th>Date Instrumented</th>
<th>No. of Sensors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parkfield - Elementary School</td>
<td>36531</td>
<td>1</td>
<td>48' x 30'</td>
<td>plywood shear walls in longitudinal direction</td>
<td>1949</td>
<td>6/87</td>
<td>6+FF</td>
</tr>
<tr>
<td>Bishop - Fire Station</td>
<td>54545</td>
<td>1</td>
<td>62' x 50'</td>
<td>perimeter plywood shear walls</td>
<td>1983</td>
<td>9/88</td>
<td>6</td>
</tr>
<tr>
<td>Eureka - Office Building</td>
<td>89687</td>
<td>2</td>
<td>80' x 48'</td>
<td>perimeter plywood shear walls</td>
<td>1992</td>
<td>2/95</td>
<td>11</td>
</tr>
<tr>
<td>Tempieton - Hospital *</td>
<td>36695</td>
<td>1</td>
<td>87' x 51'</td>
<td>distributed plywood shear walls</td>
<td>1977</td>
<td>6/94</td>
<td>9+FF</td>
</tr>
<tr>
<td>San Bernardino - Motel</td>
<td>23701</td>
<td>3</td>
<td>181' x 48'</td>
<td>plywood shear walls in 1st story</td>
<td>1986</td>
<td>9/94</td>
<td>15</td>
</tr>
<tr>
<td>Fremont - Motel</td>
<td>57720</td>
<td>2</td>
<td>145' x 38'</td>
<td>plywood shear walls in long. direction in the 1st story</td>
<td>1989</td>
<td>7/95</td>
<td>11</td>
</tr>
<tr>
<td>Indio - Hospital *</td>
<td>12759</td>
<td>1</td>
<td>298' x 244'</td>
<td>distributed plywood shear walls</td>
<td>1981</td>
<td>6/97</td>
<td>8+FF</td>
</tr>
</tbody>
</table>

* Instrumented under OSHPD/CSMIP Hospital Instrumentation Project.

### Figure A2:
**List of Records from CSMIP-Instrumented Woodframe Buildings**

<table>
<thead>
<tr>
<th>Date of Earthquake</th>
<th>Time (UTC) hr:min:sec</th>
<th>Magnitude (M)</th>
<th>Epicentral Distance (km)</th>
<th>Maximum Acceleration (g)</th>
<th>Ground</th>
<th>Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parkfield - Elementary School</td>
<td>05/04/93 05:23:25.3</td>
<td>4.2</td>
<td>7</td>
<td>7.8% H, 12.3% H</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>12/20/94 10:27:47.2</td>
<td>4.7</td>
<td>4</td>
<td>8.9% H, 20.1% H</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bishop - Fire Station</td>
<td>05/17/93 23:20:48.8</td>
<td>6.0</td>
<td>61</td>
<td>5.8% H</td>
<td></td>
<td></td>
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<td>Eureka - 2-story Office Building</td>
<td>07/08/95 09:36:51.1</td>
<td>3.9</td>
<td>13</td>
<td>3.7% H, 0.8% V, 6.2% H</td>
<td></td>
<td></td>
</tr>
<tr>
<td>San Bernardino - 3-story Motel</td>
<td>06/28/97 21:45:25.1</td>
<td>4.2</td>
<td>1</td>
<td>6.4% H, 3.5% V, 9.2% H</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>07/26/97 10:24:16.9</td>
<td>3.7</td>
<td>0</td>
<td>3.8% H, 2.7% V, 7.8% H</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>05/11/98 12:16:51.8</td>
<td>4.5</td>
<td>18</td>
<td>2.3% H, 1.1% V, 7.1% H</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Indio - 1-story Hospital</td>
<td>07/26/97 03:14:56.0</td>
<td>4.9</td>
<td>33</td>
<td>2.4% H, 8.3% H</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Earthquake of Sat Jul 26, 1997 03:24 PDT
San Bernardino - 3-story Motel Sta No. 23701
Frequency Band Processed: 2.0 secs to 46.0 Hz
- CSMIP AUTOMATED PRELIMINARY STRONG MOTION PROCESSING -

Chn 10 Roof, N. Wing, W. Wall - N
Max = 0.073 g

Chn 12 Roof, N. Wing, Center - N
-0.077 g

Chn 11 Roof, N. Wing, E. Wall - N
-0.075 g

Chn 13 Roof, S. Wing, N. Wall - N
-0.052 g

Chn 9 3rd Floor, N. Wing, W. Wall - N
-0.049 g

Chn 4 2nd Floor, N. Wing, W. Wall - N
-0.038 g

Chn 6 2nd Floor, N. Wing, Center - N
0.043 g

Chn 5 2nd Floor, N. Wing, E. Wall - N
0.035 g

Chn 2 1st Floor, N. Wing, W. Wall - N
-0.036 g

Time (sec)
Earthquake of Sat Jul 26, 1997 03:24 PDT
San Bernardino - 3-story Motel Sta No. 23701
Frequency Band Processed: 2.0 secs to 46.0 Hz
- CSMIP AUTOMATED PRELIMINARY STRONG MOTION PROCESSING -

ACCELERATION (g)

Chn 14 Roof, N. Wing: Center - E
Max = .028 g

Chn 15 Roof, S. Wing: S. Wall - E
-.077 g

Chn B 2nd Floor, N. Wing: Center - E
.017 g

Chn 7 2nd Floor, S. Wing: S. Wall - E
-.032 g

Chn 3 1st Floor, N. Wing: W. Wall - E
.018 g

Chn 1 1st Floor, N. Wing: W. Wall - Up
-.027 g

Time [sec]
Earthquake of Wed Mar 11, 1998 04:19 PST
San Bernardino - 3-story Motel Sta No. 23701
Frequency Band Processed: 2.0 secs to 46.0 Hz
- CSMIP AUTOMATED PRELIMINARY STRONG MOTION PROCESSING -

Chn 10 Roof, N. Wing: W. Wall - N
Max = .071 g

Chn 12 Roof, N. Wing: Center - N
-.039 g

Chn 11 Roof, N. Wing: E. Wall - N
-.060 g

Chn 13 Roof, S. Wing: N. Wall - N
.038 g

Chn 9 3rd Floor, N. Wing: W. Wall - N
.055 g

Chn 4 2nd Floor, N. Wing: W. Wall - N
.032 g

Chn 6 2nd Floor, N. Wing: Center - N
.025 g

Chn 5 2nd Floor, N. Wing: E. Wall - N
-.023 g

Chn 2 1st Floor, N. Wing: W. Wall - N
-.019 g

Time (sec)
Earthquake of Wed Mar 11, 1998 04:19 PST
San Bernardino - 3-story Motel Sta No. 23701
Frequency Band Processed: 2.0 secs to 46.0 Hz
- CSMIP AUTOMATED PRELIMINARY STRONG MOTION PROCESSING -

ACCELERATION (g)

Chn 14 Roof, N. Wing: Center - E
Max = -.053 g

Chn 15 Roof, S. Wing: S. Wall - E
.068 g

Chn 8 2nd Floor, N. Wing: Center - E
.027 g

Chn 7 2nd Floor, S. Wing: S. Wall - E
-.029 g

Chn 3 1st Floor, N. Wing: W. Wall - E
.023 g

Chn 1 1st Floor, N. Wing: W. Wall - Up
-.011 g

Time [sec]
California Earthquake of April 4, 1993  CSMIP Preliminary Processing
Parkfield - Elementary School  Sta No. 36531
Frequency Band Processed:  3.3 secs to 40.0 Hz
- CSMIP AUTOMATED STRONG MOTION PROCESSING -

ACCELERATION (g)

Chn 1 Roof: East Wall - N  Max = -0.09 g

Chn 2 Roof: Center - N  -0.10 g

Chn 3 Ground Floor - N  0.05 g

Chn 4 Roof: South Wall - E  -0.12 g

Chn 5 Top of South Shear Wall - E  -0.08 g

Chn 6 Ground Floor - E  -0.07 g

Time (sec)

Appendix A: Earthquake Records | 77
California Earthquake of Dec. 20, 1994  CSMIP Preliminary Processing
Parkfield - Elementary School  Sta No. 36531
Frequency Band Processed: 3.3 secs to 40.0 Hz
- CSMIP AUTOMATED STRONG MOTION PROCESSING -

ACCELERATION (g)

Chn 1 Roof: East Wall - N  Max = .17 g

Chn 2 Roof: Center - N  .16 g

Chn 3 Ground Floor - N  -.09 g

Chn 4 Roof: South Wall - E  -.20 g

Chn 5 Top of South Shear Wall - E  -.11 g

Chn 6 Ground Floor - E  -.08 g

Time (sec)
California Earthquake of May 17, 1993  CSMIP Preliminary Processing
Bishop - Fire Station  Sta No. 54545
Frequency Band Processed: 5.0 secs to 40.0 Hz
- CSMIP AUTOMATED STRONG MOTION PROCESSING -

ACCELERATION (g)

Chn 4 Roof Level: East Wall - N
Max = -0.044 g

Chn 5 Top of South Wall - N
-0.039 g

Chn 6 Roof Level: West Wall - N
-0.044 g

Chn 1 Ground Floor - N
0.019 g

Chn 3 Top of South Wall - W
0.030 g

Chn 2 Ground Floor - W
0.016 g

Time (sec)
Earthquake of Wed Feb 8, 1995 01:36 PST
Eureka - 2-story Office Bldg  Sta No. 89687
Frequency Band Processed: 2.0 secs to 40.0 Hz
- CSMIP AUTOMATED STRONG MOTION PROCESSING -

ACCELERATION (g)

<table>
<thead>
<tr>
<th>Channel</th>
<th>Type</th>
<th>Time (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chn 1</td>
<td>1st Floor: Center - Up</td>
<td>0 - 30</td>
</tr>
<tr>
<td></td>
<td>Max = .007 g</td>
<td></td>
</tr>
<tr>
<td>Chn 3</td>
<td>1st Floor: Center - N</td>
<td>0 - 30</td>
</tr>
<tr>
<td></td>
<td>Max = .037 g</td>
<td></td>
</tr>
<tr>
<td>Chn 7</td>
<td>2nd Floor: Interior Wall - N</td>
<td>0 - 30</td>
</tr>
<tr>
<td></td>
<td>Max = .045 g</td>
<td></td>
</tr>
<tr>
<td>Chn 11</td>
<td>Roof: Interior Wall - N</td>
<td>0 - 30</td>
</tr>
<tr>
<td></td>
<td>Max = .062 g</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX B
Earthquake Records: MODE-ID Analysis Details
**Figure B1:**
San Bernardino 3-Story Motel (6/28/1997)
MODE-ID Analysis Details

### E-W (longitudinal)

<table>
<thead>
<tr>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
<th>Damping Ratio (%)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>--</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
<tr>
<td>6-12</td>
<td>4.6</td>
<td>13.2</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>4.8</td>
<td>11.7</td>
<td></td>
</tr>
<tr>
<td>18-24</td>
<td>5.3</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
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</table>

### N-S (transverse)

<table>
<thead>
<tr>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
<th>Damping Ratio (%)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>5.4</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>6-12</td>
<td>5.4</td>
<td>17.3</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>5.8</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td>18-24</td>
<td>6.0</td>
<td>12.7</td>
<td>Low amplitudes</td>
</tr>
</tbody>
</table>

---

![Graph showing frequency and damping ratio over time](image1)

**San Bernardino 3-Story Motel (6/28/97)**

**X** EW (longitudinal)  ○ NS (Transverse)
Figure B2:
San Bernardino 3-Story Motel (7/26/1997)
MODE-ID Analysis Details

<table>
<thead>
<tr>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
<th>Damping Ratio (%)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>--</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
<tr>
<td>6-12</td>
<td>5.0</td>
<td>13.3</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>5.3</td>
<td>6.1</td>
<td></td>
</tr>
<tr>
<td>18-24</td>
<td>5.5</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
</tbody>
</table>

N-S (transverse)

<table>
<thead>
<tr>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
<th>Damping Ratio (%)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>6.0</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
<tr>
<td>6-12</td>
<td>4.8</td>
<td>5.8</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>5.3</td>
<td>11.7</td>
<td></td>
</tr>
<tr>
<td>18-24</td>
<td>5.5</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
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Figure B3:
San Bernardino 3-Story Motel (3/11/1998)
MODE-ID Analysis Details

<table>
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<th>Natural Frequency (Hz)</th>
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<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>5.3</td>
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<td>Low amplitudes</td>
</tr>
<tr>
<td>6-12</td>
<td>4.4</td>
<td>7.2</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>5.1</td>
<td>4.8</td>
<td></td>
</tr>
<tr>
<td>18-24</td>
<td>5.2</td>
<td>--</td>
<td></td>
</tr>
</tbody>
</table>

N-S (transverse)

<table>
<thead>
<tr>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
<th>Damping Ratio (%)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>5.5</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
<tr>
<td>6-12</td>
<td>5.5</td>
<td>9.9</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>6.1</td>
<td>11.8</td>
<td></td>
</tr>
<tr>
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<td>--</td>
<td>Low amplitudes</td>
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**Figure B4:**
Bishop Fire Station (5/17/1993)
MODE-ID Analysis Details

### E-W (longitudinal)

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<th>Natural Frequency (Hz)</th>
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<th>Comments</th>
</tr>
</thead>
<tbody>
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<td>9.0</td>
<td>8.9</td>
<td></td>
</tr>
<tr>
<td>6-12</td>
<td>8.6</td>
<td>12.7</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>8.8</td>
<td>10.9</td>
<td></td>
</tr>
<tr>
<td>18-24</td>
<td>9.1</td>
<td>8.6</td>
<td></td>
</tr>
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</table>

### N-S (transverse)

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<th>Damping Ratio (%)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>5.9</td>
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<tr>
<td>6-12</td>
<td>5.5</td>
<td>6.8</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>5.7</td>
<td>4.9</td>
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<td>5.8</td>
<td>5.1</td>
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### Figure B5:
Parkfield 1-Story School (4/4/1993)
MODE-ID Analysis Details

#### E-W (longitudinal)

<table>
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<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
<th>Damping Ratio (%)</th>
<th>Comments</th>
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</thead>
<tbody>
<tr>
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<td>Strongest shaking</td>
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<tr>
<td>6-12</td>
<td>7.2</td>
<td>12.1</td>
<td>Strongest shaking</td>
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<tr>
<td>12-18</td>
<td>8.6</td>
<td>7.2</td>
<td></td>
</tr>
<tr>
<td>18-24</td>
<td>8.8</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
</tbody>
</table>

#### N-S (transverse)

<table>
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<th>Natural Frequency (Hz)</th>
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</thead>
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</tr>
<tr>
<td>12-18</td>
<td>9.4</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
<tr>
<td>18-24</td>
<td>--</td>
<td>--</td>
<td>Low amplitudes</td>
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</table>
Figure B6: Parkfield 1-Story School (12/20/1994)
MODE-ID Analysis Details

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<th>Natural Frequency (Hz)</th>
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<th>Comments</th>
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<td>10.4</td>
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<td></td>
<td>6-12</td>
<td>6.6</td>
<td>10.2</td>
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<td>12-18</td>
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<td>7.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>18-24</td>
<td>8.7</td>
<td>--</td>
<td>Low amplitudes</td>
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</table>

<table>
<thead>
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<th>N-S (transverse)</th>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
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<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-6</td>
<td>7.8</td>
<td>17.2</td>
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<tr>
<td></td>
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<td>8.3</td>
<td>12.6</td>
<td></td>
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<td>12-18</td>
<td>9.3</td>
<td>7.7</td>
<td></td>
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<tr>
<td></td>
<td>18-24</td>
<td>9.4</td>
<td>7.7</td>
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Figure B7:  
Eureka 2-Story Office (2/8/1995)  
MODE-ID Analysis Details

<table>
<thead>
<tr>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
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<tbody>
<tr>
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<td>18-24</td>
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**E-W (transverse)**

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<th>Natural Frequency (Hz)</th>
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<th>Comments</th>
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<tbody>
<tr>
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</tr>
<tr>
<td>18-24</td>
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<td>5.2</td>
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</tr>
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**N-S (longitudinal)**

Eureka 2-Story Office  
(2/8/95)

× EW (transverse)  ○ NS (longitudinal)
## Figure B8:
**Indio 1-Story Hospital (7/25/1997)**
**MODE-ID Analysis Details**

### E-W (longitudinal)

<table>
<thead>
<tr>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
<th>Damping Ratio (%)</th>
<th>Comments</th>
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</thead>
<tbody>
<tr>
<td>0-6</td>
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<td>Low amplitudes</td>
</tr>
<tr>
<td>6-12</td>
<td>7.9</td>
<td>11.0</td>
<td>Strongest shaking</td>
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<tr>
<td>12-18</td>
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<td>7.1</td>
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<tr>
<td>18-24</td>
<td>8.0</td>
<td>--</td>
<td>Low amplitudes</td>
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### N-S (transverse)

<table>
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<tr>
<th>Time Range (sec.)</th>
<th>Natural Frequency (Hz)</th>
<th>Damping Ratio (%)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6</td>
<td>7.9</td>
<td>--</td>
<td>Low amplitudes</td>
</tr>
<tr>
<td>6-12</td>
<td>6.8</td>
<td>9.5</td>
<td>Strongest shaking</td>
</tr>
<tr>
<td>12-18</td>
<td>7.1</td>
<td>6.0</td>
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</tr>
<tr>
<td>18-24</td>
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<td>5.1</td>
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</table>
APPENDIX C

Task 1.1.1: MODE-ID Analysis Details
### Figure C1:
#### Task 1.1.1 Seismic Tests -- Phase 9
#### MODE-ID Analysis Details

<table>
<thead>
<tr>
<th>Test Level</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>3r</th>
<th>4</th>
<th>5</th>
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</thead>
<tbody>
<tr>
<td>PGA (g)</td>
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<td>0.36</td>
<td>0.36</td>
<td>0.50</td>
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<td>Freq. (Hz)</td>
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<td>2.88</td>
<td>2.63</td>
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<td>1.88</td>
</tr>
<tr>
<td>Damp. (%)</td>
<td>7.4%</td>
<td>20.1%</td>
<td>19.2%</td>
<td>15.7%</td>
<td>16.3%</td>
<td>15.1%</td>
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<tr>
<td>NMS Error (J)</td>
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<td>0.1435</td>
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### Figure C2:
#### Task 1.1.1 Seismic Tests -- Phase 10
#### MODE-ID Analysis Details

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