VIBRATION STUDY OF THE U.S. COURT OF APPEALS BUILDING FOR SEISMIC ISOLATION RETROFIT

Steve Keowen[1], Navin Amin, S.E.[2], Anoop Mokha, Ph.D.[2], Paul Ibanez, Ph.D.[1]

[1]ANCO Engineers, Inc., 9937 Jefferson Blvd., Culver City, CA
[2]Skidmore, Owings & Merrill, 333 Bush St., San Francisco, CA

ABSTRACT

The historic U.S. Court of Appeals Ninth Circuit building located at 7th and Mission Street in San Francisco is currently being seismically retrofitted with seismic isolation and new shear walls. Ambient and forced vibration testing was performed to determine the dynamic characteristics of this century old, landmark structure in order to calibrate the analytical model of the structure for establishing design parameters. This paper presents a comprehensive review of the test objectives, program, setup and discusses the test results and their usefulness in calibrating the analytical models. Also discussed are the implications of extrapolating the observed dynamic response characteristics obtained at low levels of excitation to significantly larger levels of excitation expected during severe ground shaking.

1. INTRODUCTION

The U.S. Court of Appeals Ninth District Building located on 7th and Mission streets in San Francisco suffered damage to its structural and non-structural contents during the October 1989 Loma Prieta earthquake and was vacated thereafter. The building is currently being seismically retrofitted using base isolation. When completed in 1995, it will be the largest isolated structure in the United States.

1.1 Building Description. The original U-shaped building, constructed in 1905, structurally survived the devastating 1906 San Francisco earthquake and fire with minimal damage. In 1933, a fourth wing was added, giving the building a rectangular shape with a central atrium. Figure 1 shows a historic photograph of the building from the corner of Seventh and Mission street. Approximate plan dimensions are 100 m by 81 m (330 feet by 265 feet) and the total floor area is about 32,500 m^2 (350,000 square feet). The building is a five-story, 24.4 m (80 feet) tall structure with steel framing, concrete slabs, unreinforced granite masonry exterior walls and hollow clay tile interior partitions. The total weight of the building (dead load + reduced live load) is about 55 million kg (120,000 kips). Interior finishes are extremely ornate. They include carved marble figures, inlaid marble walls and floors, and highly intricate plaster ceilings. This Beaux Arts

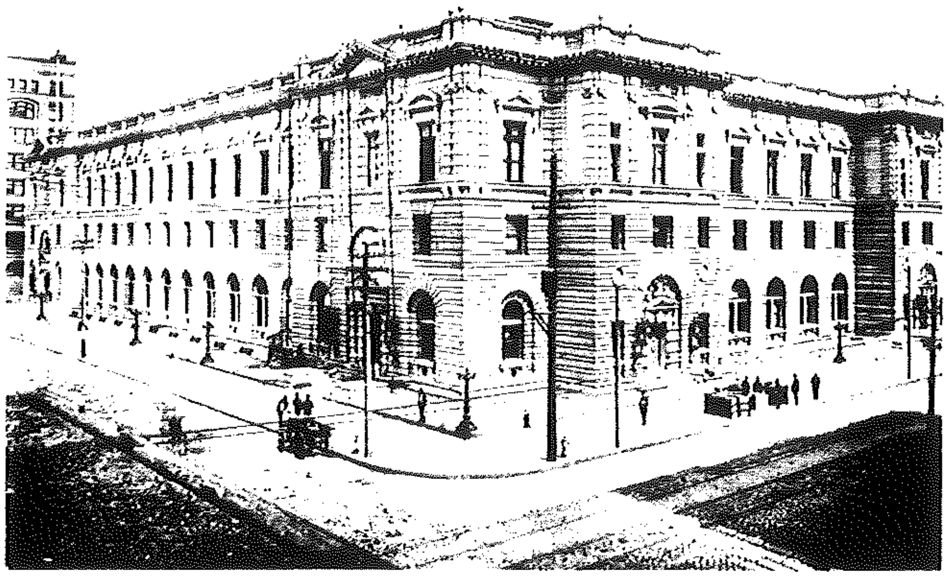


Figure 1: Photograph of U.S. Court of Appeals Ninth Circuit Building

building is on the National Register of Historic Places. When the seismic renovation is completed in 1995, the Court of Appeals will be the largest and the heaviest base-isolated building in the United States.

1.2 Structural System. The structural system for the 1905 U-shaped building consists of built up columns with laced steel channels. The floor framing system consists of a beam and girder system, with beams spaced at about 1.5 m (5 feet). Concrete slabs of arch form span between the beams. Interior partition walls consist of hollow clay tiles and exterior walls are made up of non-reinforced granite masonry supported on steel beams. The foundation consists of steel grillage footings encased in non-reinforced concrete.

The structural system of the 1933 addition is entirely different from that of the 1905 building. The columns are made up of wide flange sections. The floor framing system consists of a beam and girder assemblage with 153 mm (6 inches) thick reinforced concrete slab. The interior partitions are of hollow clay tiles and the exterior wall is made up of non-reinforced granite masonry. The foundation system for the 1933 wing consists of reinforced concrete piles with reinforced concrete pile caps.

The building was damaged during and repaired after the 1906 earthquake. Damage was limited to portions of the exterior granite masonry and interior hollow clay tile walls (including a settlement at the corner of Seventh and Mission street). Significant damage to the exterior granite masonry and interior walls also resulted from the Loma Prieta earthquake of October 1989. Structural response and subsequent damage to exterior granite masonry walls and interior hollow clay partition walls demonstrated the contribution of strength and stiffness of these elements to the overall dynamic characteristics of the structure.

1.3 Pretest Analysis. Two cases were considered: 1) Model with rigid floors; and 2) Model with flexible floors. Table A shows the dynamic characteristics of the existing structure. These values were calculated based on (best guess) assumptions of structural material properties and connectivity. As can be seen, the modeling assumptions regarding the floor have a significant effect on the resulting natural frequencies.

2. TEST OBJECTIVES

The test objectives included, but were not limited to, the following:

- To determine the natural frequencies and modal damping ratios of the structure for modes of vibration having frequencies of 12 Hz or less;
- To determine the response shapes of the identified modes of vibration;
- To determine any amplitude-dependent trends in the identified modes of vibration;
- To determine the interaction between different building sections;

TABLE A: DYNAMIC CHARACTERISTICS OF THE EXISTING STRUCTURE

Summary - Dynamic Characteristics of the Existing Structure Calculated Periods							
Flexible Model				**Rigid Model**			
Frequency (Hz)	Period (sec)	Dominant Dir.	Mass Part (%)	Frequency (Hz)	Period (sec)	Dominant Dir.	Mass Part (%)
1.40	0.715	E-W	72.0	3.02	0.331	E-W	75.1
1.57	0.638	N-S	62.7	3.46	0.289	N-S	71.9
2.28	0.439	Torsion	---	4.55	0.220	Torsion	---
2.95	0.339	E-W	3.1	6.94	0.144	N-S	1.5
3.89	0.257	N-S	4.4	7.52	0.133	Torsion	---
5.62	0.178	Torsion	---	7.63	0.131	E-W	1.4

- To identify, and, if possible, quantify nonlinear phenomenon (such as slippage and friction, etc.);
- To evaluate the flexible modes of vibration of large span floors at selected locations (in- and out-of-plane); and
- To identify any soil-structure interaction.

The results of these tests were used to confirm the detailed analytical models of the structure to reduce the uncertainty in predicted earthquake-induced loads. Large uncertainties were possible for this case in estimating the stiffness contribution due to interior partition and exterior masonry walls and in how the different structural elements are connected and how they interact as indicated in Table A. The "modified" analytical model was used to assess the relative merit of different retrofit schemes.

3. TEST PROGRAM

Data were acquired and analyzed to meet project goals while the building response was induced by both ambient (wind, local traffic, distant earthquakes, etc.) and forced vibration (mechanical excitation) sources. Ambient excitation was used to gain initial insight into the dynamics of the structure prior to forced vibration testing and to identify out-of-plane floor modes of vibration. Forced vibration testing relied on mechanical excitation using an eccentric mass shaker system. The shaker was used at three different locations on the fourth floor to preferentially excite different modes of vibration. Response of the building to this controlled forced (about .001 g) was several to tens of times higher than the response of the building to ambient forces.

3.1 Test Equipment and Test Methods. A total of six Kinemetrics Model SS1 (Ranger) seismometers were employed to sense vibration amplitudes. Velocity proportional seismometer signals were passed through medium gain amplifiers to a two-channel Hewlett-Packard Model 3582A real-time (spectrum) analyzer for data analysis. The spectrum analyzer was operated in several ways to present either time domain data or frequency domain data as appropriate. During ambient excitation tests, the spectrum analyzer was used in the root-mean-squared (RMS) average mode to compute and average the moduli of the Fourier transforms of the response over many sampling intervals. During sine sweep testing, the analyzer was operated in the peak hold mode to compute and store the maximum sinusoidal RMS value which occurred in each of the analyzer's 128 frequency bands during the entire time of testing. The analyzer was operated in the real-time mode to compute and report the transfer function amplitude and response phase angle between a fixed reference seismometer and a seismometer which was moved to many different locations on the structure to map the response of the structure while it was being forced sinusoidally at an identified resonant frequency by the eccentric mass shaker.

3.2 The Eccentric Mass Shaker. The eccentric mass shaker (ANCO Model MK 12.8-4600) consisted of a matched pair of weight sets which rotate in opposite directions about parallel vertical shafts.

The eccentricity of the shaker was continuously adjustable between 0 and 100% of its maximum eccentricity of 4,600 lb-in. Upon rotation of the weights, a unidirectional sinusoidal force (up to 10,000 lbs) is imparted in a chosen direction in the horizontal plane.

The peak force output of this shaker (or any rotating imbalance) varies in proportion to its eccentricity and running speed squared. An expression for the force output is:

$$F = 0.102(WR)f^2 \text{ lbs}$$

where:

F = peak force in lb_f
WR = eccentricity (lb-in.)
f = running speed (Hz)

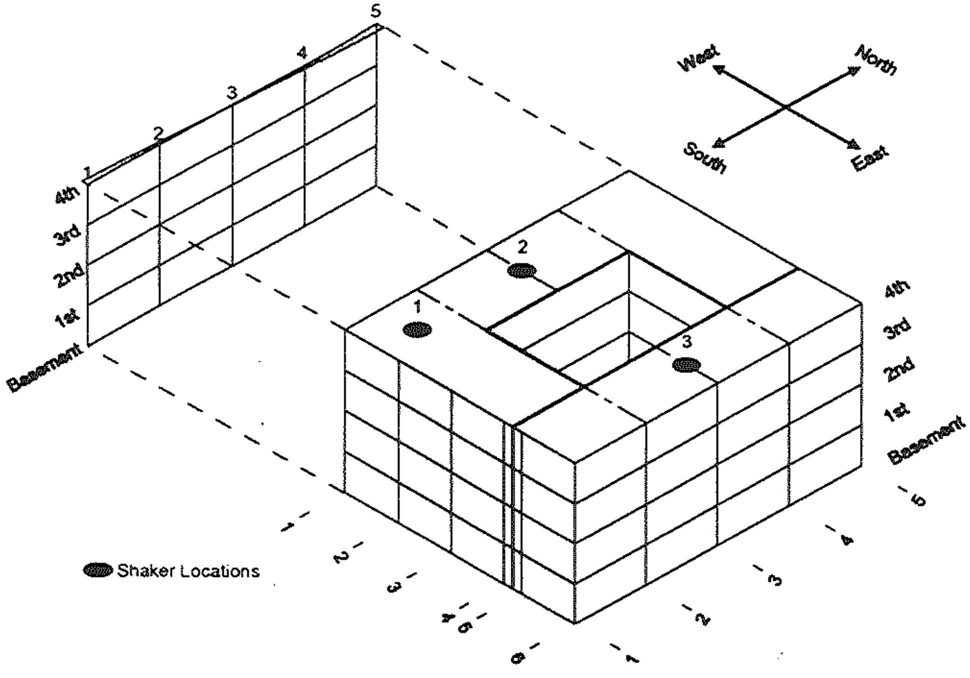
Thus, the applied dynamic load imparted to the test structure could be calculated given knowledge of the shaker's eccentricity and speed. Three shaker locations were selected to preferentially excite the building modes of interest as shown in Figure 2.

3.3 Data and Data Interpretation. As mentioned, both ambient and sine sweep test data consist of frequency versus velocity response plots such as that illustrated in Figure 3. There is a lot of information on the plot. During this sine sweep test, modes of vibration were identified at 1.68, 2.44, 3.56 and 4.44 Hz with a small peak at 3.08 Hz. In parentheses are the relative response amplitudes expressed in millivolts (mV). This plot was made with three different vertical scales (160, 800 and 1,600 mV full-scale) so that greater amplitude resolution was made possible. Damping was estimated using the half-power bandwidth method from the data in Figure 3 to range between about 2 and 5% of critical for the identified modes.

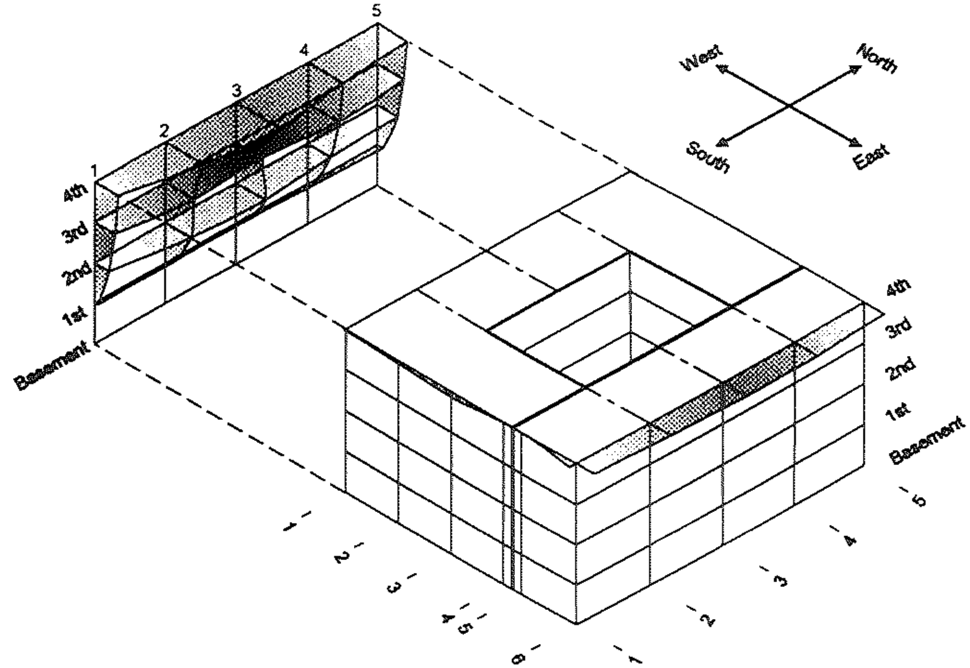
Sine sweeps were made at up to three different values of shaker eccentricity (hence, three different force amplitudes) so that trends in resonant response for the identified modes could be determined.

Response shapes of the building were mapped at the identified resonant frequencies. The procedure was as follows. With the shaker set to force at a location and direction which adequately excited the mode of interest, the shaker was set to run at a speed corresponding to the identified resonant frequency while the relative amplitude and phase angle between a reference seismometer and signals acquired from a roving seismometer were compared. The reference seismometer was generally located where building response was high.

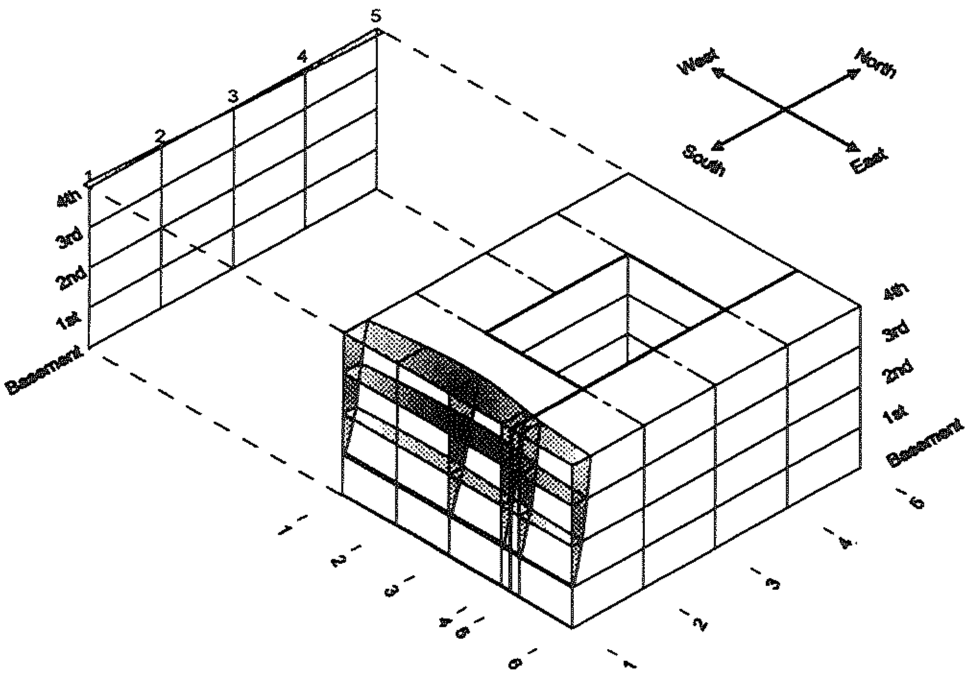
If the phase angle between the reference seismometer and a seismometer at a specific location was less than $\pm$ 90°, the signal was considered in-phase and the relative amplitude between the two declared positive. If the phase angle was greater than $\pm$ 90°, the signal was considered out-of-phase and the relative amplitude declared negative. The collection of positive and negative relative amplitudes, when normalized to the reference seismometer, constituted the response shape of the building at the resonant frequency of interest. With well separated modes of vibration, the response shape is essentially the same as the mode shape.



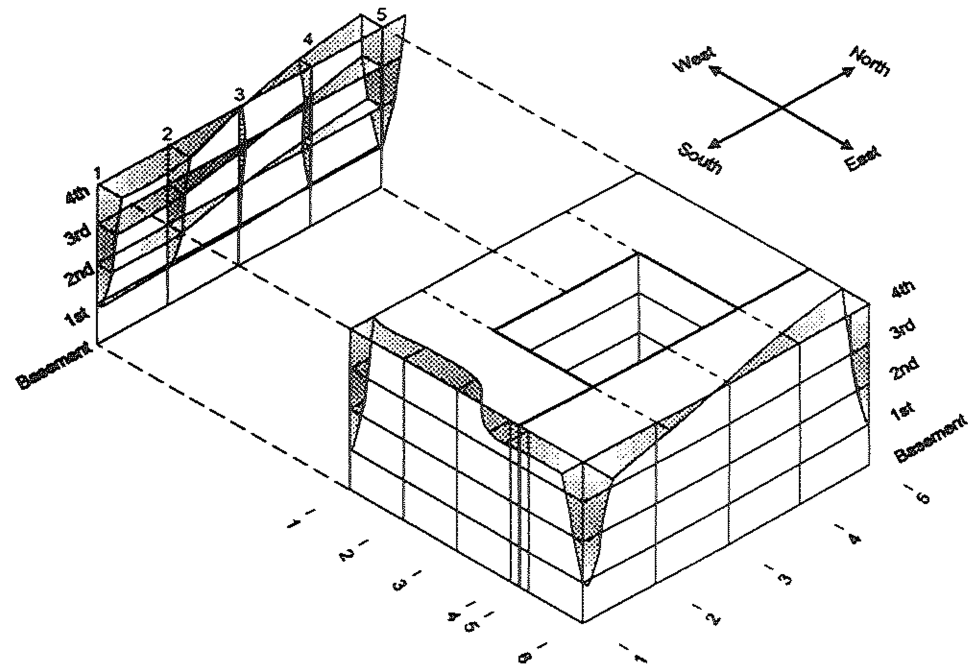
Response Shape Measurement Locations



Mode No. 1, Response Shape at 1.72 Hz (Test 6, Run 2)



Mode No. 2, Response Shape at 1.94 Hz (Test 6, Run 2)



Mode No. 3, Response Shape at 2.16 Hz (Test 6, Run3)

Figure 2: Response Shapes

4. TEST RESULTS

Test results are in the form of identified frequencies of the modes of vibration of the structure and their corresponding response shapes. Some amplitude-dependent trends in response are apparent in the data, however, changes were small over the range of test amplitudes.

Twenty or so modes of vibration were identified below 12 Hz. Fundamental modes of the structure were identified as low as 1.7 Hz. In-plane floor flexibility was evident in all modes of vibration but was particularly evident in modes of vibration above about 2.5 Hz. In-plane floor flexibility often masked the presence of second and third order structural modes of vibration. Out-of-plane floor modes of vibration were found to range between about 8 and 14 Hz. Response shapes were mapped for the majority of the identified modes.

4.1 Ambient Vibration Test Results. Ambient test data proved most useful in planning subsequent forced vibration testing. Fundamental modes were noted at about 1.76, 2.00 and 2.16 Hz. Response amplitudes of a few micro g were sufficient to gain insight into bending shapes of the lowest modes of vibration only.

4.2 Forced Vibration Test Results. Table B presents an abbreviated summary of the identified modes of vibration of the structure resulting from forced vibration sine sweep testing. Presented in the table is the frequency of the mode of vibration, its period (reciprocal of frequency), the test and run number where the resonant peak was well defined, the location and direction of the forcing function, the eccentricity of the shaker for that test, and a verbal description of the shape of the building response. Frequencies varied slightly with test amplitude, hence the values given are "nominal" values. Response shapes corresponding to the first three modes of vibration are illustrated in Figure 2. In all, nineteen modes of vibration were mapped.

4.3 Out-of-Plane Floor Modes of Vibration. Ambient vibration was sensed vertically and averaged over many sampling intervals at 21 selected floor locations to identify out-of-plane modes of vibration. Frequencies of these modes were found to range between about 8 and 14 Hz. Damping ranged between 4 and 7% of critical. A small constant amplitude out-of-plane peak at 3.6 Hz was recorded at most vertical measurement locations which suggested that the building was bouncing vertically on the soil at that frequency.

4.4 Interaction Between the 1933 Wing and 1905 Building. Tests were performed to look specifically at the interaction between the old building (built in 1905) and the new wing (added in 1933). Sine sweeps were made in the N-S and E-W from 1 to 10 Hz, while the shaker was at Location No. 3 so that the transfer

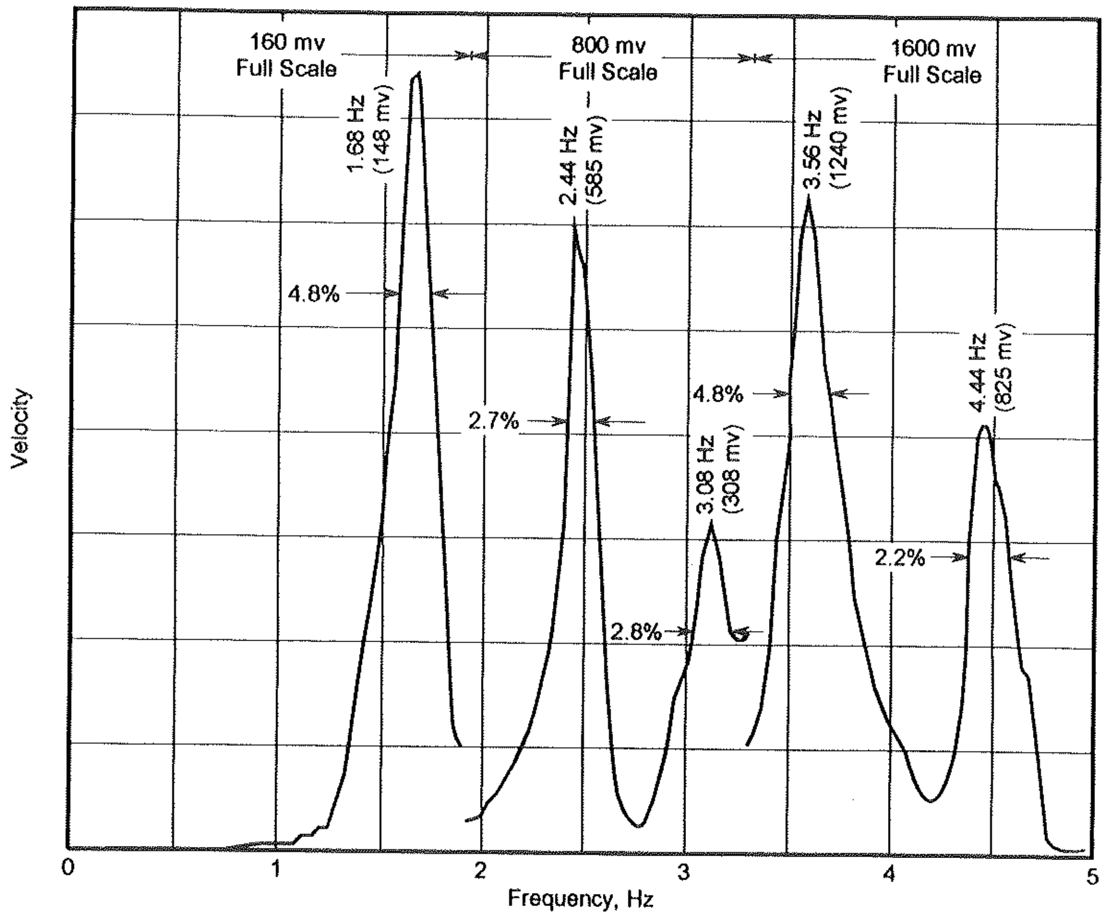


Figure 3: Typical Sine Sweep Test Data

TABLE B: ABBREVIATED SUMMARY OF THE LOWEST FEW IDENTIFIED MODES OF VIBRATION

Mode No.	Frequency (Hz)	Period (sec)	Test and Run No.	Force Location	Force Direction	Shaker Eccentricity (lb-in.)	Response Shape Description
1	1.76	0.57	T1R2R	1	E-W	980	First E-W bending.
2	1.92	0.52	T2R2R	1	N-S	980	First N-S bending.
3	2.16	0.46	T3.2R2	2	N-S	980	First torsional mode.
4	2.30	0.43	T4R3	3	N-S	4600	First N-S bending of the east wing.
5	2.48 2.56	0.40 0.39	T3R2R T4R5	2 3	E-W E-W	980 980	In-plane bending of east and west wings. All floors in-phase. East and west wings out-of-phase.
6	3.12	0.32	T3R3R	2	E-W	980	In-plane bending of east and west wings. East and west wings out-of-phase. Second order bending of south wing.
7	3.60 3.64	0.28 0.27	T3R3 T2R2R	2 1	E-W N-S	4600 980	In-plane bending of east and west wings. East and west wings in-phase. Second order bending of south wing.
8	4.64	0.22	T4R5	3	E-W	980	Second order E-W bending of east wing, i.e., Floors 1, 2 and 3 out-of-phase with Floor 4.

function across the intersection of the two building sections on the south-side, fourth floor could be recorded. It should be noted that some interaction had been noted in the data gathered while forcing the structure from Location No. 1. These results indicated the need to look at the phenomenon in greater detail. As a result of the subsequently performed test, it was concluded that in either direction below about 5 Hz, there was no measurable differential motion recorded between the old and new sections of the building. Between 5 and 10 Hz, some flexibility was recorded across the intersection of these two elements. The transfer function amplitude varied between about 0.5 and 2.0 at building resonances but the phase angle between the 1905 and 1933 buildings never exceeded about $\pm$ 60°, indicating that the two buildings moved in-phase with some differential motion.

4.5 Differential Wall Motion. Tests were performed to detect differential motion between the outer stone facade and an interior floor slab. Seismometers were placed across the west side outer stone wall/floor slab interface during a 1-10 Hz frequency sweep to note the amplitude of the transfer function and its phase angle to define differential motion between the two locations. These data indicate that at all frequencies where structural modes of the building had been noted below 10 Hz, there was no differential motion across the outer stone wall and floor slab interface. However, at 8.64 Hz, the transfer function was noted to rise steeply to a value of 1.52 and the phase angle between the two seismometers rose to 78°, suggesting that some nonlinear phenomenon or some differential motion may have been present.

5. IMPLICATIONS OF TEST RESULTS

The data suggest that, over the range of test amplitudes in all modes of vibration, the structure behaves as a softening system. A softening system is defined as one whose frequency decreases as response amplitude increases. Test response amplitudes were small. In some modes of vibration, motion could be sensed by test personnel and in others it could not. In general, damping was found to range between about 2.5 and 5.0%. Extrapolation to design levels of response require judgment.

5.1 Extrapolation to Higher Levels of Response. Extrapolating the dynamic properties of a structure from low-level test data to high-level earthquake response involves judgment, the evaluation of load paths, and the review of past test and earthquake data. The forced vibration tests at the court house involved peak accelerations on the order of 0.001 g. The response during a large earthquake may be on the order of 0.5 g. Will the properties measured in the low-level test be appropriate at the higher seismic levels?

Experience from tests and from measured earthquake responses of many structures indicates that the properties will remain essentially the same, with some decrease in frequencies and increase in damping (mild "softening" behavior).

This assumption is valid as long as no highly nonlinear mechanism is at work and no severe damage occurs. Hence, the resonant frequencies measured at the court house will most likely decrease 10 to 20% at the higher excitation level, and the damping will probably increase by a factor of 1.5 to 2.0.

Could a highly nonlinear mechanism be at work? This is unlikely. The stiffness of the court building appears to be dominated by the stiffness of the outer granite walls, acting partially as a shear wall through the steel extension beams from the main steel frame. This load path involves no strong nonlinearities, such as soil-structure interaction, impact, or closing of cracks. As long as the granite (and granite mortar) does not fail in a significant number of locations, stiffness will not significantly degrade.

Experience from the 1906 and 1989 earthquakes suggests that only some localized granite failure will occur at similar ground motions. At these levels, it is reasonable to project that the dynamic properties measured during the test (slightly changed as discussed above) and the above postulated load path will be valid.

6. COMPARISON WITH ANALYTICAL RESULTS AND DISCUSSIONS

Based on results obtained from extensive testing of existing structural material, an extensive analytical model of the non-isolated structure was developed using the program SAP90. The tests utilized cores of concrete slabs, exterior stone and brick masonry to determine their compressive and shear strength. In-phase shear tests were performed on brick and stone masonry to evaluate the existing lateral force-resisting capacity of the building. The fundamental period of the non-isolated structure was calculated as 0.46 seconds (2.17 Hz) using experimentally obtained material values. The fundamental period obtained from the forced vibration testing was 0.57 seconds (1.75 Hz). Further parametric studies were conducted to assess the change in fundamental period of non-isolated structure on the overall response of the isolated structure, and it was observed that a shift in structure period from 0.46 seconds to 0.57 seconds had negligible effect on the response of the isolated structure.

The forced vibration testing results were very helpful in understanding structural behavior. Although the measured dynamic properties were not directly used in modeling the complex superstructure, they were utilized in confirming various assumptions in the analytical modeling of the superstructure.