# ANALYTICAL AND EXPERIMENTAL DYNAMIC ANALYSES OF A TEN STORY BUILDING

by

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## ABSTRACT

The ten story Engineering School building at U.C. Irvine has been subjected to both ambient and forced vibration tests in order to provide a data base in anticipation that interpretations of changes in dynamic properties measured after a future earthquake will enable assessment to be made of the degree of deterioration experienced by the structural system. Additionally several mathematical models have been investigated, ranging from simple shear buildings to substantial three dimensional finite element representations. Whereas simple models were found to provide estimates of lower mode frequencies within 10% of those measured, and hence are deemed satisfactory for use as design tools, more complex models were necessary to allow computation of torsional characteristics and it was found that only when allowances for the additional in-plane stiffness contributions of the internal partition walls were made could reasonable correlation with measured properties be achieved.

In this paper the details of the building examined are outlined, the experimental procedures are summarized and a selection of the measured properties are presented. The theoretical analyses are explained and the derived results are compared with measured values. Conclusions are drawn regarding the particular characteristics of this structure, including a propensity for modal interference to occur, and concerning the validity of the various mathematical models.

## BACKGROUND

As it is situated close to the Newport/Inglewood fault-zone, and also within the range of expected severe shaking when the southern portion of the San Andreas fault next moves, it is anticipated that this structure will suffer strong seismic excitation within its economic life. Strong motion recorders have been permanently installed at selected positions in the building and the free field with the object of monitoring the seismic response. The availability of the results of the controlled dynamic testing prompted an investigation of the validity of the mathematical modeling techniques involved in the extensive idealisation of prototype structures which are invariably necessary before analyses can be performed.

## THE STRUCTURE

The Engineering building is a nine-story plus penthouse pedestal-type structure. Typical section and plan views of both the structural and foundation systems are given in Figures 1 and 2. A moment-resistant ductile steel frame forms the top six stories of the structure whereas a frame constructed of a combination of post-tensioned and normal reinforced concrete girders, columns and walls makes up the bottom three stories. The building forms the larger part of a two-building complex with the Computer Science building, the first story between the two structures being continuous.

The steel frame of the building has overall plan dimensions of 144 feet (N-S) by 128 feet (E-W) and overhangs the bottom three stories on each side by approximately 37 feet (N-S) and 29 feet (E-W). The building has a common central area of approximately 70 feet square on each level. The structure stands 126 feet above grade on the west side, 111 feet on the east side and has an overall height of 132 feet above basement level, excluding a 16 foot high penthouse.

The large reinforced concrete columns of the bottom three stories have an inverted "L" shape and appear to tilt outward from the vertical centerline of the building as a result of tapered cross-sections. In the N-S direction, these columns are approximately 4 feet wide by 10 feet deep at fourth floor level reducing in depth to 6 feet at the first floor while in the E-W direction their dimensions are approximately 3 feet wide by 9 feet deep reducing in depth to 6 and 5 feet at the east and west grade levels respectively. Each of these sixteen columns is supported on a footing typically 13 feet square by 4 feet deep. Further support to the structure is provided by four centrally located 24 inch diameter belled caissons extending approximately 15 feet into the subsoil below.

"L" shaped shear walls 12 inches thick between the first and fourth floors form the sides of the central core of the building which houses three elevators, two stairways and a central mechanical equipment shaft. At the fourth floor level, large post-tensioned reinforced concrete girders tie the main columns and walls together in each direction. These girders measure 4 feet wide by 7 feet deep (N-S) and 3 feet wide by 7 feet deep (E-W).

The floor slabs of the second, third and fourth floors are of waffle-type construction and typically consist of a 4.5 inch thick deck supported by 8 inch wide by 20 inch deep joists in both directions. All girders, columns, joists and slabs above the third floor are of 110 pounds per cubic foot concrete while those below the third floor, including the 12 inch thick shear walls are of 150 pounds per cubic foot concrete.

The three-dimensional steel frame of the top six stories is formed by six planar frames in each of the N-S and E-W directions. All members are of A 36 structural steel and support reinforced concrete floor slabs 5 inches thick. Metal stud walls 6 inches thick form the sides of the central core of the building between the fourth floor and roof and typically provide interstory partitioning within the steel frame. The exterior facing of the top six stories of the structure is completed by aluminum wall and window framing panels.

## THE TESTING

To improve interpretation of earlier ambient-test results (1), forced vibration investigations (2) were aimed at determining responses to each of north-south, east-west and torsional excitation. The Slave unit of a Kinemetrics Model VG-1 vibration generator system consisting of two counter-rotating masses fixed to a common vertical axis was used. The resulting sinusoidal force may be aligned in any fixed direction in the horizontal plane. Up to 5,000 pounds of horizontal force can be generated but the maximum allowable stress in the shakers limits the excitation frequency to a peak of about 9.5 Hz. The control unit incorporates a solid state speed control system which limits fluctuations in excitation frequency to $\pm 0.005$ Hz.

The shaker was bolted to the floor slab at three different locations during the course of a testing as shown in Figure 2. N-S excitation at roof level was achieved from position A, E-W and torsional excitations at roof level from position B and N-S excitation at fourth floor level from position C. The structural layout of the roof on the E-W centerline prevented selection of positions on the center lines of the building. However, one advantage of using position B was that both E-W and torsional modes could be excited at different times simply by changing the phase relationship between the two counter-rotating buckets of the shaker. Since the fourth floor marks the transition between the reinforced concrete frame below and the steel frame above, it was thought worth-while to excite the concrete sub-frame directly to determine whether this would give rise to a different response from that induced by excitation applied directly to the steel frame at roof level.

Measurements made during the tests used up to five Model SS-1 Ranger seismometers each set at 70 percent of critical damping to give a transfer function of approximately unity over the frequency range of interest. The seismometers were used primarily to determine relative motion and phase between measurement points. However, approximate values for the foundation stiffness of the building were derived from absolute seismometer calibrations provided by the manufacturer.

Relative calibrations between the seismometers were made at all of the frequencies of interest by aligning them side by side when the building was excited at a known frequency, and measuring the outputs simultaneously. Kinemetrics SS-1/SC-1 attenuator boxes were used to decrease the signals from the Rangers before amplifying and filtering by the SC-1 signal conditioners.

The signals were recorded on one of two Gould Brush ink strip chart recorders. Both recorders provided very clean and readable double amplitude traces which could be read to within several percent.

The testing procedure involved one seismometer being kept at roof level as a reference for each series of N-S, E-W and torsional tests. For the measurement of N-S modes excited at either roof or fourth floor level, the remaining four seismometers were moved in a systematic pattern on lower floors to determine the translational mode shapes of interest.

The measurement of the response of each floor to torsional excitation involved a distribution of the Ranger seismometers in which four instruments per floor were placed in a pinwheeltype configuration around the periphery of the building, the seismometer at the north edge pointing west, the seismometer on the west edge pointing south, the seismometer on the south edge pointing east and the seismometer on the east edge pointing north. With the limited number of instruments available, this arrangement resulted in the response of only one floor at a time being measured relative to the roof. At roof level, the reference seismometer was placed on the west edge of the building pointing south. The responses of the seismometers placed at the north and south edges of each floor were used to determine the torsional mode shape amplitudes.

Comparisons of the responses of the seismometers on the north, south and west edges with each other and with the roof reference were used to yield the mode shape phases. The seismometer arrangement described above was also used to measure modal coupling and modal interference of the upper four even-numbered floors induced by N-S excitation at roof level.

By interpretation of measurements made using two seismometers in a vertical position at basement level, an estimate of the foundation compliance, and hence foundation stiffness, was made. For the lowest two modes in each of the N-S and E-W directions of movement the effect of foundation compliance was found to be small although more flexibility in the N-S direction is evident than in the E-W one. This behavior is consistent with both the nature of the foundation system appropriate to a good bearing situation - and with the detailed configuration of the basement layout.

## FORCED VIBRATION TEST RESULTS

The forced vibration tests conducted on the U. C. Irvine Engineering building principally involved the determination of natural frequencies and mode shapes for ten modes of vibration. Damping values were derived from the lower modes while foundation compliance was estimated for all modes. Modal coupling and modal interference in the E-W direction under N-S excitation were briefly investigated, as was the non-linearity associated with increasing levels of response. An attempt was made to quantify any variation in the results obtained between August and November 1981 and those derived from the subsequent tests in April 1982. Sample results are presented in Table 1 and Figures 3 and 4.

Table 1 indicates that the building is only slightly stiffer than might have been predicted. A rule of thumb commonly used is that the fundamental period of a multistory building is about 0.1 N where N is the number of stories. For the Engineering building this approximation suggests a fundamental period of 0.9 seconds as compared to 0.75 seconds (N-S) and 0.81 seconds (E-W) from forced vibration tests and 0.71 seconds (N-S) from the ambient tests.

An obvious feature of the first two mode shapes in each of the three principal directions is the pronounced kink at fourth floor level. This is the level at which the structure changes from the flexible steel frame above to the very stiff reinforced concrete frame below. The corresponding discontinuity in the third and fourth modes tends to be masked by the close proximity of a nodal point to the fourth floor.

Non-structural elements such as exterior window frames, metal stud-wall partitions, electrical and mechanical ducts, stairwells and fireproofing are considered to have had a significant effect on the translational and torsional response of the building to forced vibration. In combination with several other factors, including three 3000 gallon distilled water tanks located on the roof 30 to 35 feet southeast of the plan center of the building, the contribution to interstory stiffness from these elements is consistent with the kink in the upper two stories of the first torsional mode shape.

Particularly in the case of the higher modes, the non-orthogonal nature of the response made interpretation difficult. Convincing evidence of modal interference (3) was found both in the form of the resonance curves and the difficulty in isolating individual mode shapes during the testing. Also significant foundation compliance was established. Although the foundation stiffnesses determined were only approximate, they compared well with values determined by earlier experimentors (4, 5).

Fourth floor excitation of the lowest three N-S modes was undertaken for purposes of comparison with the roof excitation values. The first mode was found to increase by some 8%, the second by 1% and no significant change was observed in the third one. However, the relative amplitudes and phases of the raw data for the third N-S mode were found to be more clearly defined for fourth floor excitation since there is little interaction between the energies supplied by the shaker to the steel frame above and the reinforced concrete frame below. The mode shapes derived are somewhat smoother than those for the roof excited case.

The aim of the second set of forced vibration tests undertaken six months later than the main set was to clarify a number of uncertainties and this objective was achieved but the opportunity was taken to test the repeatability of a selection of the earlier tests. A range of differences was established. Typically the frequencies repeated within 1 or 2%. The mode shapes were found to be essentially unaltered but the actual measured displacements differed by values ranging from 5% to 30%. Seasonal variation in the structure proper coupled with calibration shifts could possibly account for these changes. Investigators of other buildings have observed a non-linear response to different levels of forced vibration (6,7). This behavior usually involved a decrease in the natural frequencies of vibration of a structure as the exciting force is increased. Such a response is typical of a softening dynamic system. Somewhat larger changes than previously reported of the order of a 10% reduction in natural frequency - were measured between the cases of the minimum and maximum exciting force used in the tests on the U.C.I. building. A possible explanation for this trend is the very light nature of the internal partitions and secondary structure in the building tested. The equivalent viscous damping was observed to approximately double, from around 2% to 5%, with the increase in exciting force.

## MATHEMATICAL ANALYSES

Two different types of mathematical models were constructed to compute the dynamic properties of the Engineering building. The first type involved simple planar models based on a shear building idealization of the structure while the second type was a complete three-dimensional finite element model. The natural frequencies and mode shapes predicted by both types of models were compared.

The finite element model was established based on the SAP IV analysis code (8). The model was refined to reflect the actual value of building stiffness determined from the dynamic testing. In particular clear span lengths were used for all column heights and the stiffness of the boundary elements at ground level were chosen on the basis of the compatability of the static lateral load deflected shape.

Three separate sets of computed frequencies are presented in Table 1 for comparison with the measured values. The Type I results were derived from the simple shear building models. The Type II results were obtained from the second phase of dynamic analysis of the finite element model, in which the major axis bending rigidities of the steel floor beams were multiplied by a factor of three to account for composite action with the concrete floor slabs. Transformed section calculations based on a total effective flange width of 15 percent of the beam span indicatd that a factor of only 2 to 2.5 was appropriate. The factor of three was chosen, however, after an extra allowance was made for the bending rigidity of the slabs outside the region of influence of the main steel floor beams. The Type III results were the final dynamic properties derived from the finite element model after allowances were made for the non-structural panel elements within the real structure. Specifically the use of shear stiffness estimates for the wall partitions and window frames were based on the results of recent tests on similar types of panels (9). A stiffness of $1.5 \times 10^3$ lb/in was assigned to each section of wall partition or window frame having a height-to-length ratio of two. Since the panel elements in the Engineering building are approximately 12 feet high, a subdivision length of 6 feet was used for analysis purposes. Allowances were made for the fact that continuous panels are stiffer than the sum of the contributions from their 6 foot long "subpanels". On this basis, the total interstory stiffness from panel elements was estimated to be

$2.2 \times 10^6$ lb/in in the N-S direction and $1.4 \times 10^6$ lb/in in the E-W direction. These were approximately 65 percent and 40 percent respectively of the typical steel frame interstory stiffness in each direction.

## COMPARISON OF RESULTS

Several interesting conclusions may be drawn from a comparison of these three sets of computed results with those measured during forced vibration tests. The first, is that for most practical purposes a simplified shear building analysis gives adequate estimates of the natural translational frequencies of the building. Comparison of the measured lower translational mode shapes with those derived from the simple models shows similar adequate agreement. The significant discrepancy between the measured and computed third mode shapes would be unimportant for practical analysis since the effect of third and higher modes on the total response of a structure is normally very small. It should, however be noted that in conjunction with the neglect of foundation compliance effects, these simple models predict a considerably stiffer behavior of the building's reinforced concrete frame than was measured during forced vibration tests. In addition, these models give no estimate of the torsional properties of the building.

The results for the Type II finite element model show reasonable agreement with the measured characteristics. Providing that an allowance is made for composite action of floor systems and bending rigidity of floor slabs, it may be concluded that currently available computer models allow prediction of dynamic properties of a multistory building sufficiently accurately for design use.

The Type III finite element model results demonstrate the importance of non-structural panel elements in the measured response of the Engineering building to forced vibration. Without allowances for non-structural panel elements being made in the model, a match between measured and computed natural frequencies closer than 10 percent would not have been achieved.

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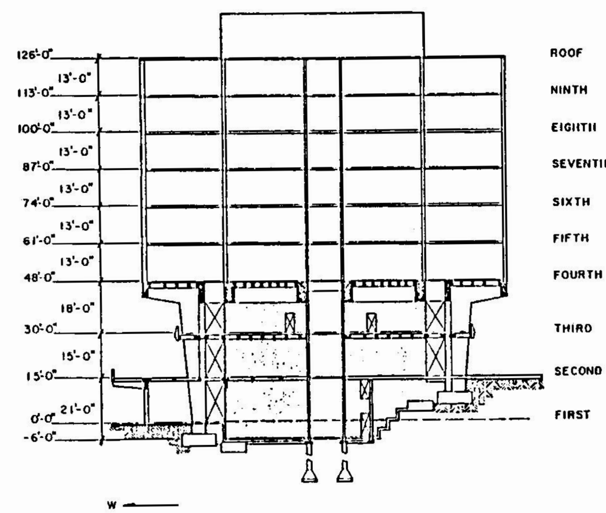


Figure 1 Typical Section View Engineering Building

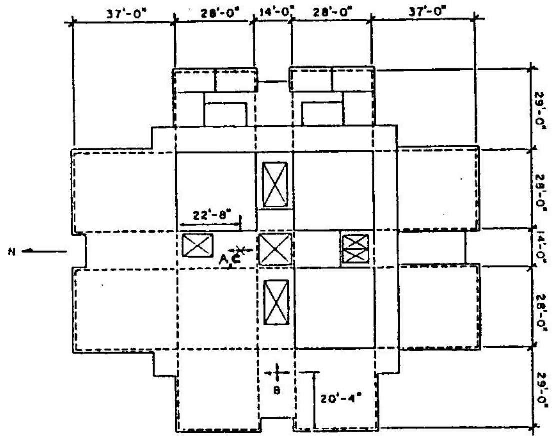


Figure 2 Typical Floor Plan of the Engineering Building

( A, B and C denote Shaker locations )

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Comparison of Analytical and Experimental Results

| Mode | Experimental | | | Analytical | | |
|---|---|---|---|---|---|---|
| | Frequency (Hz) | Period (Seconds) | Damping (% Critical) | Computed Frequency (Hz) Model I | Model II | Model III |
| N-S 1 | 1.33 | 0.75 | 2.5 | 1.33 | 1.19 | 1.33 |
| N-S 2 | 4.04 | 0.25 | 3.3 | 3.69 | 3.66 | 4.10 |
| N-S 3 | 6.02 | 0.17 | – | 5.75 | 5.52 | 5.76 |
| E-W 1 | 1.23 | 0.81 | 2.7 | 1.30 | 1.17 | 1.23 |
| E-W 2 | 3.66 | 0.27 | 3.8 | 3.67 | 3.58 | 3.78 |
| E-W 3 | 5.32 | 0.19 | – | 5.75 | 5.29 | 5.40 |
| Torsion 1 | 1.29 | 0.78 | 2.6 | – | 0.99 | 1.08 |
| Torsion 2 | 3.54 | 0.28 | – | – | 2.58 | 2.68 |
| Torsion 3 | 4.88 | 0.20 | – | – | 3.65 | 3.97 |
| Torsion 4 | 6.66 | 0.15 | – | – | 5.43 | 5.66 |

Table 1

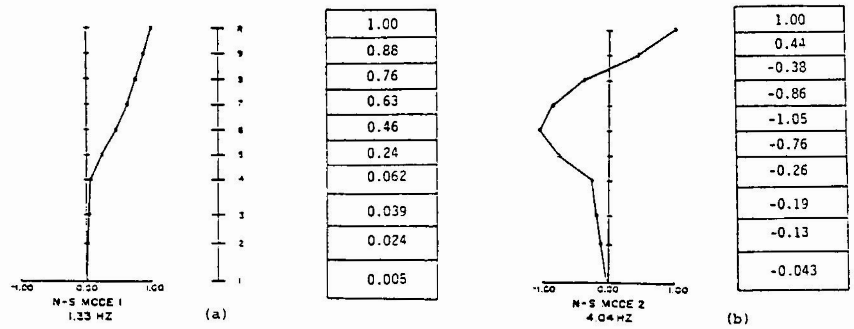


Figure 3 First and Second N - S modes of Engineering Building

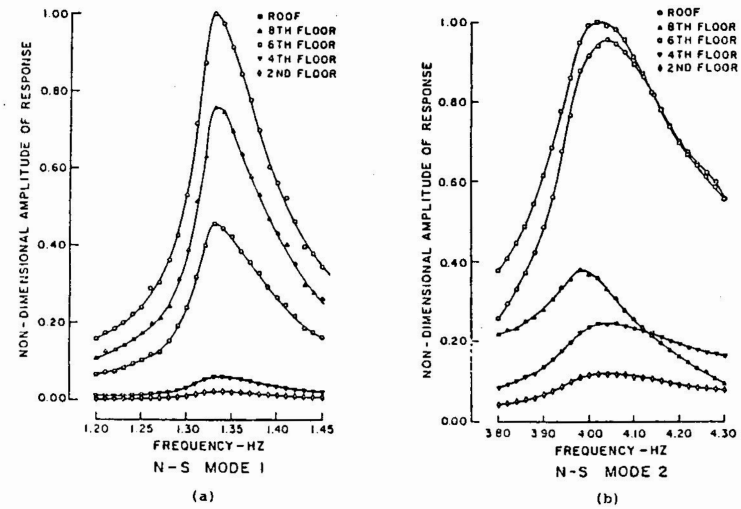


Figure 4 Resonance Curves for First and Second N - S Modes