# PERIODS OF A MODERN MULTI-STORY OFFICE BUILDING DURING CONSTRUCTION

By

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## SYNOPSIS

A fifteen-story steel-frame office building for the Bethlehem Steel Company, Pacific Coast Division is being completed in San Francisco. As the frame was erected and later as floors, fireproofing, partitions and other elements were added, the natural periods of vibration were measured with various instruments.

The structure, at corresponding stages of construction, was analyzed for mass, stiffness and other characteristics and natural periods of vibration were computed. Reconciliation of measured and calculated periods and of the changes in periods provided confirmation of the calculation techniques. Data were also obtained on the participation of the various structural and architectural elements in the vibration response of this modern building.

Instrumentation is being provided by the owner for the automatic recording of future earthquake acceleration and amplitude at several levels of the building.

## INTRODUCTION

Numerous multi-story buildings are being constructed in active seismic areas of the world. A few of these tall buildings have instruments installed to record building movements during earthquakes. As a consequence, some records are available of the response accelerations of tall buildings subjected to earthquake motions.(1)(2) The dynamic response characteristics of buildings have been calculated using certain assumptions, based on test and other data, as to the relative participation of the various building elements(3). One method of checking some of these assumptions is to measure the natural periods of a structure as it is constructed and hence the initial or primary effect of some of the various elements can be determined as they are added to the structure.

The construction of the fifteen-story steel framed office building for Bethlehem presented such an opportunity for measuring the natural periods of a tall structure as erection of the steel frame progressed and later as the floors, fireproofing, walls and partitions were added. The measuring of the vibration response of this building is the first phase of a continuing research program. Two strong motion accelerometers and two sets of maximum story displacement recorders will be installed to record building movements during future earthquakes.

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Much research has been conducted on the Alexander Building in San Francisco,(1)(2)(3) which is located within six blocks of the Bethlehem Building. Seismic forces will be resisted primarily by the steel frame in the Bethlehem Building while the Alexander Building involves the participation of exterior masonry filler walls, interior shear walls, and stairways. As a result there will be a traditional building and a modern building within close proximity which will have basic data available as well as instrumentation for future research.

## DESCRIPTION OF THE BUILDING

The Bethlehem Steel Company - Pacific Coast Division office building, located in downtown San Francisco, at California and Davis Streets, is a modern fifteen-story building which will be completed in May of 1960. The building has a moment resistant steel-frame, light-weight concrete floor fill on cellular steel decking, exterior marble facing over fireproofed columns and concrete encased spandrel sections and no conventional concrete or masonry filler walls. The steel frame was designed to resist both vertical and lateral loads and conformed to the San Francisco Building Code 1956 seismic provisions(4). Another criterion used in the frame design was a maximum allowable wind drift of .002 times either the story or building height. The moment capacity of the rigid frame joints is developed by high-strength bolted connections. To limit the lateral wind drift as well as to provide for seismic resistance the girders in both directions were haunched at the columns. There is a portion of the building west of the tower which is two stories high. This section is separated from the tower by an expansion joint and therefore has no effect on the dynamic properties of the tower.

Special problems in the design of the building were (1) the exterior columns on column lines A and D (see Fig. 1) which were placed outside of the spandrel line for architectural reasons, thus necessitating a specially designed rigid "torque-box" to transfer the spandrel girder moments to the columns; and (2) the abrupt and unsymmetrical change in column stiffness below the third floor level. This latter situation required horizontal steel bracing in addition to concrete slab floor diaphragms at the second, third and fourth floor levels to distribute the resulting shear forces at those levels. The steel frame is symmetrical above the third floor through the fourteenth floor.

Construction of the building commenced in the summer of 1958 with the driving of steel H-section friction piles approximately 80 feet in length, through fill material into old bay deposits and varied clay and sand strata. Erection of the steel by Bethlehem Steel Company began early in October of 1958 and continued through February of 1959. From that time until July 1959 concrete spandrel and column fireproofing, beams and slabs and metal deck fill were poured. After July the glass and marble facing were set into place. (See Figs. 7 and 8).

## DESCRIPTION OF THE VIBRATION RECORDINGS

From the earliest phases of the steel erection, commencing when the building was three stories high and continuing until the building was nearly completed in February of 1960, measurements were made of wind induced micro-vibrations of the building. A total of twenty sets of measurements were made from October 22, 1958, to February 11, 1960, the results of which are tabulated in Table 1. The number of individual instrument runs for each phase of construction varied from two to seven with the instruments being used in a variety of different combinations; individually, adjacent to each other, and coupled together. The instruments were set-up in numerous locations; at the center of the building, at the corners of the building, and on different floors.

The instruments used to record the vibrations were (1) a three component, battery powered, photographically recording portable seismograph belonging to the U. S. Coast and Geodetic Survey (Fig. 3a), (2) a similar portable seismograph belonging to John A. Blume & Associates Research Division, and (3) a horizontal, two component torsion pendulum vibration meter system belonging to Mr. Ralph S. McLean (Fig. 3b). The USC&GS portable seismograph has inverted pendulums of 0.75 second periods to measure the horizontal components of vibration, the Blume instrument has 2.5 second periods. Both instruments have a dual static magnification of 100 to 200. The McLean instruments have natural periods ranging from 1.5 to 3.0 seconds with a static magnification of 76.

The intent of the vibration measurements was to isolate several modes in each direction of the building as they existed at the numerous stages of construction. Determination of the actual natural periods enabled a comparison with calculated periods and provided a good measure of the contribution of the various structural and non-structural elements to the total story and building characteristics.

The positions of the instruments recording natural periods was often determined by convenience to the contractor and the availability of a suitable mounting surface for the recording instruments. In order to minimize secondary vibrations, the instruments were set up on the top flange of the steel girders adjacent to the columns, and later, on the concrete floor, in the same general location, as construction progressed. Throughout the program wind induced vibrations were satisfactory because of the prevailing westerly winds which provided a disturbing force. Only horizontal motion was noted throughout the records and the translational modes through the fourth were at times isolated (Table 1). No vertical building vibrations were observed, and torsional periods were undoubtedly recorded but their isolation was not feasible in this program.

## ANALYSIS OF THE RECORDED PERIODS

A history of the change in the recorded natural periods of the building with different phases of construction is shown in Figures 7 and 8 for the longitudinal and transverse directions of the building, respectively. Similar trends of period variation corresponding to additional construction can be noted for each direction of the building.

As the steel frame erection progressed the fundamental periods lengthened quite rapidly because of the rapid increase in building height. Toward the end of the steel erection a plateau is noted which was partially due to a stoppage in the erection work for a short time. The concrete encased spandrel girder sections and slabs added stiffness and weight. At the lower floors which were poured first, the stiffness and weight increased in nearly the same ratio so that the periods did not lengthen appreciably. Later when the spandrels and slabs were poured in the upper stories, the effect of the added weight was greater than the increase in stiffness and therefore, the periods increased considerably between phases 10 through 16. The exterior column concrete fireproofing and the gypsum block walls surrounding the fire tower contributed to the stiffness relatively more than to the weight. The exterior and interior finishes (marble, glass, plaster and ceilings) had the net effect of adding more stiffness than weight for small amplitudes so that the building periods decreased to the values shown for phase 20.

The higher modes follow the same general pattern noted for the fundamental with respect to phases of construction. The ratio between fundamental, second and third mode recorded periods was approximately 1:2:4 at the early phases of construction and changed to 1:3:5 at the final phases.

## METHOD OF PERIOD CALCULATION

The building is assumed to undergo shear-type deflection only wherein axial deformations of columns are neglected. This is not strictly the case as the building deflects in flexure as well as it rocks and moves laterally. However, it is assumed for this type of building, without heavy filler walls and partitions that the dominant movement is a shearing displacement and the first, second and third modes of vibration are calculated on this basis. Calculations made indicate that neglecting flexure for this building involves an error of 2 or 3 percent which is minor. The results are shown in Table 3 and the corresponding measured values are included for comparison.

To obtain the natural periods, the method outlined by Blume(5) for shear vibration is followed. This procedure requires the mass and the shearing stiffness for each story, see Table 2. The mass of each story is assumed to be concentrated at the floor level. The spans of the girders and the floor construction are such that joint rotation of the girders and columns are included in the analysis, however, axial deformation of the columns due to flexure is not.

"Non-structural" components, such as fireproofing and partitions, are items that should be taken into consideration in calculating the stiffness. Overlooking such items results in considerable amount of error. In Table 3, the calculated transverse first mode period, phase 18, is 38% greater than the measured period when the gypsum walls, surrounding the smoke stack, are neglected. When the stiffness of these walls is included the variation is only 6.7%. Similarly, for the longitudinal first mode period, when the column fireproofing is neglected the variation is 39%. When the fireproofing is included, the difference is only 2.2%.

The total building weights used in design were also used for calculating periods for phase 18. Therefore, much of the variation between calculated and measured periods may be due to the fact that the actual mass is lighter than that assumed in design. In addition, the stiffness factors are probably slightly greater than assumed due to interaction of the various materials such as the steel decking (which is welded to the steel).

There are two gypsum walls acting in the transverse direction. Each wall is hollow, 4 inches thick, and has one-half inch plaster on each face. Thus the gross thickness of the two walls is 10 inches. It is assumed that 7 inches are effective for calculating the stiffness of the wall.

Field Measurement phase 5, February 27, 1959, was recorded when the steel frame was completed and concrete was cast on the second floor and on the spandrels of the fifth floor. These measurements were analytically checked using the method of this paper. Phase 18, December 29, 1959, was recorded when the building was completed except for part of the ceiling, partitions, glass, and similar items. This phase was also checked analytically. The results of the calculations are shown in Table 3, and the relative changes for the first, second and third modes are shown in Figs. 7 and 8.

## DISCUSSION OF RESULTS

1) The computed periods are in general concurrence with the measured values.

2) Non-structural items, such as partitions, fireproofing, etc., should be included in the calculations to obtain realistic values of the period. While such materials are often neglected in the design of a structure they will participate up to their capacity in resisting lateral forces. Accordingly, they must be designed to withstand the stresses imposed on them by virtue of their existence as part of the building, be detailed not to participate, or be expected to fail.

3) Table 3 shows that for phase 18, the nearly completed building, the fundamental period T, for the transverse direction is approximately equal to 0.10 N. This confirms the recommendation of the Seismology Committee of the SEAOC[6] for modern rigid frame buildings.

4) A study of the ratios of fundamental to higher mode periods for this building indicates that deformation due to flexure or "column-shortening" is minor compared to shearing deformation; calculations confirmed this conclusion.

5) The effect of soil on building periods has been neglected because the friction piles provide measurable resistance to rocking.

## FUTURE RESEARCH

Further vibration recordings are contemplated after the building is completed to evaluate the effect of additional floor to ceiling partitions, furnishings and office equipment, and of general weathering of the building materials, such as the spandrel walls, concrete, etc. In order to evaluate seismic response of the building during future earthquakes two accelerometers have been purchased by Bethlehem Steel Company, one to be installed on the 12th floor, the other in the basement. These instruments will form a part of the strong motion stations maintained by the Seismological Field Survey of the U. S. Coast and Geodetic Survey and will contribute to the general fund of engineering knowledge regarding building response when future earthquakes are recorded. In addition story displacement gages have been installed between the 6th and 7th floors and between the 12th and 13th to record intra-story deformations in both longitudinal and transverse directions. This building is thought to be the first moment resisting steel-frame structure without the benefit of concrete shear walls and other resisting elements in which such instrumentation has been installed. The accelerometers and gages will enable response characteristics of this steel-frame building to be determined from future earthquake motions. In addition forced vibration research and calculation of periods of all phases of construction would be valuable.

## ACKNOWLEDGEMENTS

The authors appreciate the aid received from Mr. William K. Cloud and his staff of the Seismological Field Survey of the United States Coast and Geodetic Survey in making vibration recordings. Mr. Ralph S. McLean is to be thanked for the loan of his vibration meters which were used to record periods during the construction. The assistance provided by the contractors at the building site to facilitate the field recording is appreciated: Bethlehem Steel Company was the steel erector, Swinerton and Walberg Company were the general contractors. The structural engineers for the building were Hayes and Little, and John A. Blume & Associates, Engineers, and the architect was Welton Becket & Associates. Messrs. Eric Elsesser, Abe Mohammed and Roland L. Sharpe of John A. Blume & Associates Research Division conducted the field measurements, reduced the data and made the calculations.

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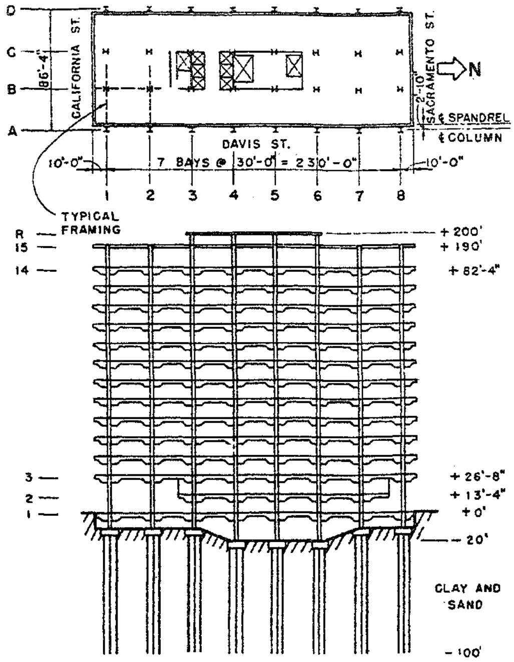



FIGURE 1 - SCHEMATIC ELEVATION AND PLAN

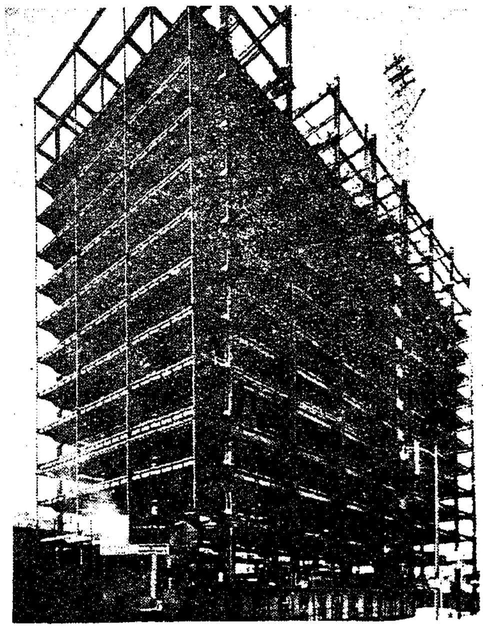
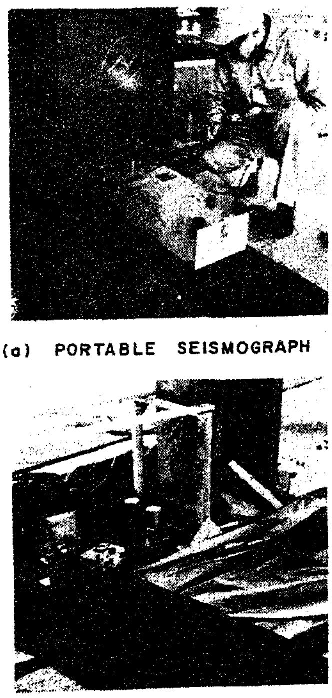


FIG. 2 — VIEW OF NEARLY COMPLETED STEEL FRAME



(a) PORTABLE SEISMOGRAPH

(b) VIBRATION METERS

FIG. 3 - VIBRATION RECORDING INSTRUMENTS

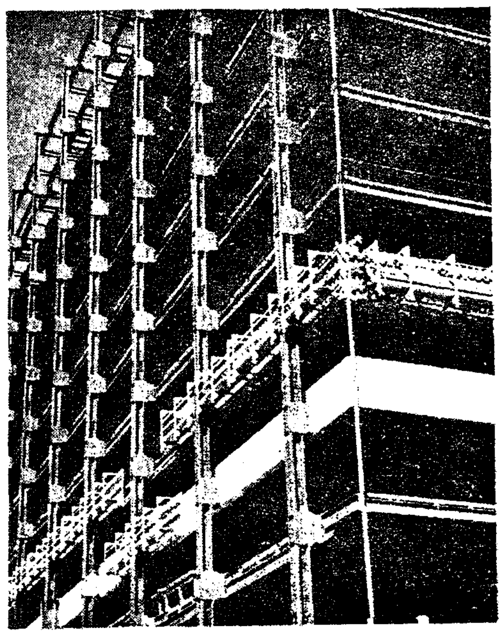


FIG. 4 - VIEW OF BUILDING CONSTRUCTION

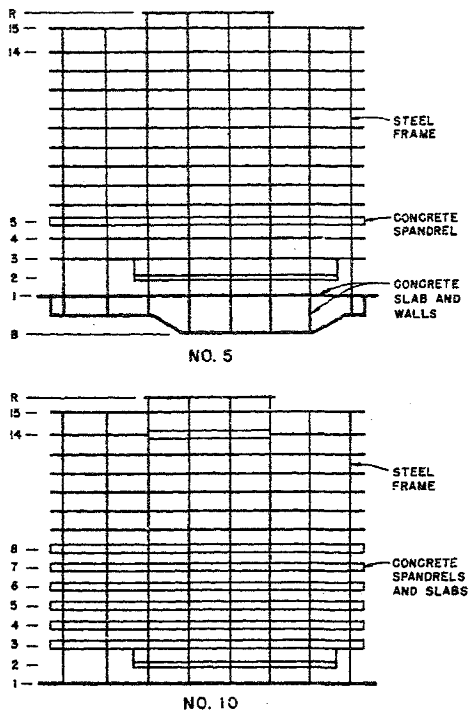



FIG. 5 - PHASES OF CONSTRUCTION

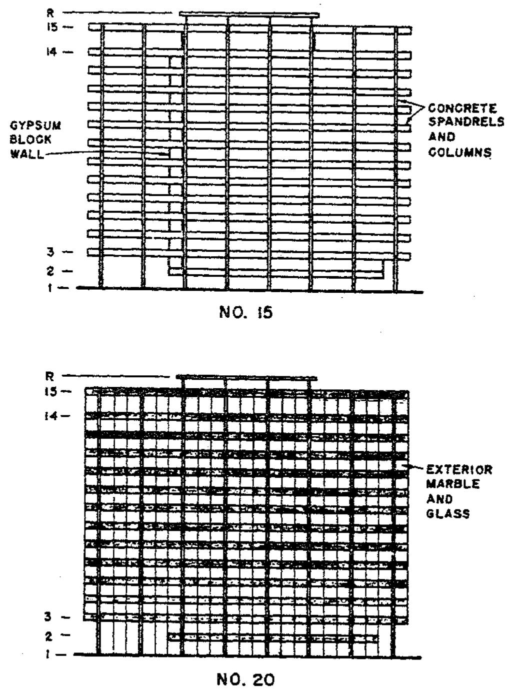



FIG. 6 - PHASES OF CONSTRUCTION

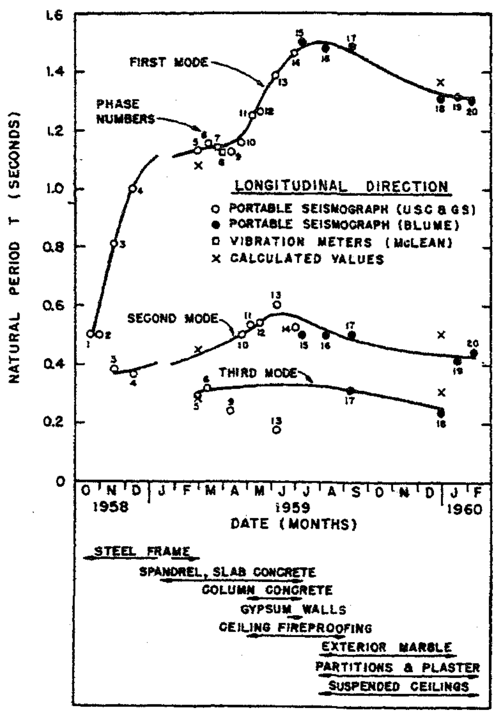



FIG. 7 – VARIATION OF PERIOD WITH CONSTRUCTION
LONGITUDINAL DIRECTION

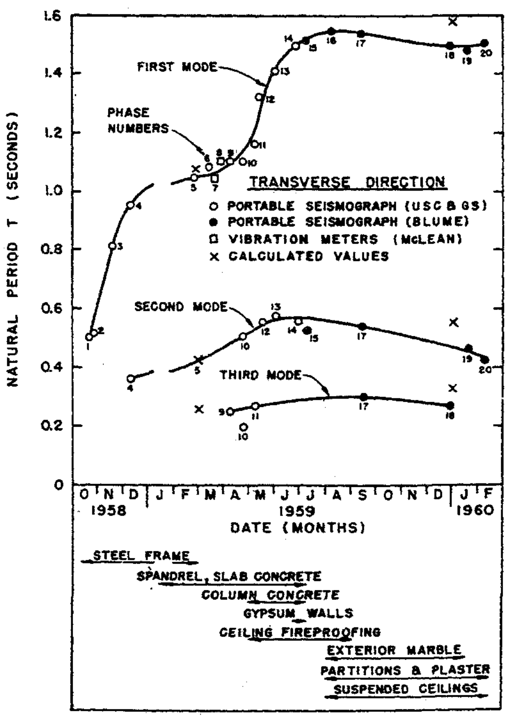



FIG. 8 — VARIATION OF PERIOD WITH CONSTRUCTION
TRANSVERSE DIRECTION

TABLE 1

SUMMARY OF MEASURED PERIODS

| Phase No. | Date of Measurement | Period (Seconds)* | | | | | | | | Recording Instrument Used |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Transverse | | | | Longitudinal | | | | |
| | | 1st Mode | 2nd Mode | 3rd Mode | 4th Mode | 1st Mode | 2nd Mode | 3rd Mode | 4th Mode | |
| 1 | Oct. 22, 1958 | .50 | | | | .50 | | | | A |
| 2 | Oct. 27, 1958 | .51 | | | | .50 | | | | A |
| 3 | Nov. 19, 1958 | .81 | | | | .82 | .38 | | | A |
| 4 | Dec. 10, 1958 | .95 | .36 | | | 1.01 | .36 | | | A |
| 5 | Feb. 27, 1959 | 1.05 | .43 | | | 1.13 | | .29 | | A |
| 6 | Mar. 13, 1959 | 1.08 | | | | 1.15 | | .30 | | A |
| 7 | Mar. 17, 1959 | 1.04 | | | | 1.14 | | | | B |
| 8 | Mar. 31, 1959 | 1.10 | | | | 1.13 | | | | B |
| 9 | Apr. 10, 1959 | 1.10 | | .25 | | 1.13 | | | .24 | A |
| 10 | Apr. 24, 1959 | 1.10 | .51 | | .19 | 1.16 | .50 | | | A |
| 11 | May 4, 1959 | 1.16 | | .27 | | 1.25 | .53 | | | A |
| 12 | May 15, 1959 | 1.32 | .55 | | | 1.30 | .54 | | | A |
| 13 | June 5, 1959 | 1.41 | .57 | | | 1.38 | .60 | | .20 | A |
| 14 | June 29, 1959 | 1.51 | .55 | | | 1.47 | .53 | | | A |
| 15 | July 8, 1959 | 1.53 | .52 | | | 1.50 | .50 | | | C |
| 16 | Aug. 7, 1959 | 1.55 | | | | 1.48 | .50 | | | C |
| 17 | Sept.10, 1959 | 1.54 | .53 | .30 | | 1.48 | .50 | .32 | | C |
| 18 | Dec. 29, 1959 | 1.49 | | .27 | | 1.32 | | .23 | | C |
| 19 | Jan. 21, 1960 | 1.46 | .46 | | | 1.32 | .41 | | | C |
| 20 | Feb. 11, 1960 | 1.50 | .42 | | | 1.31 | .43 | | | C |

A - Portable Seismograph (U. S. Coast and Geodetic Survey, Seismological Field Survey)
B - Vibration Meters (Ralph McLean)
C - Portable Seismograph (John A. Blume & Associates Research Division)
* - Average readings

TABLE 2

CALCULATED WEIGHTS AND STIFFNESSES

| Story | Construction Phase 5 | | | Construction Phase 18 | | |
|---|---|---|---|---|---|---|
| | Weight (Kips) | Stiffness K ($lbs/in \times 10^6$) | | Weight (Kips) | Stiffness K ($lbs/in \times 10^6$) | |
| | | Longit. | Trans. | | Longit. | Trans. |
| 15 | 58 | 0.093 | 0.046 | 130 | 0.200 | 0.200* |
| 14 | 200 | 0.796 | 1.011 | 1704 | 2.795 | 5,928 |
| 13 | 346 | 1.946 | 2.236 | 2858 | 6.312 | 7.602 |
| 12 | 471 | 2.313 | 2.514 | 1762 | 6.828 | 7.373 |
| 11 | 480 | 2.213 | 2.411 | 1695 | 6.793 | 7.283 |
| 10 | 507 | 2.577 | 2.898 | 1688 | 7.811 | 7.615 |
| 9 | 532 | 2.702 | 2.951 | 1722 | 7.959 | 7.719 |
| 8 | 549 | 2.919 | 3.217 | 1885 | 8.740 | 7.838 |
| 7 | 575 | 3.205 | 3.270 | 1923 | 8.937 | 7.914 |
| 6 | 591 | 3.333 | 3.498 | 1924 | 9.292 | 8.018 |
| 5 | 629 | 3.438 | 3.920 | 1975 | 9.361 | 8.540 |
| 4 | 1143 | 6.096 | 4.366 | 1975 | 10.078 | 8.715 |
| 3 | 661 | 3.579 | 4.212 | 2011 | 10.071 | 8.701 |
| 2 | 660 | 4.609 | 5.253 | 3175 | 10.792 | 10.001 |
| 1 | 1320 | 3.090 | 2.968 | 1439 | 8.076 | 3.000 |
| Average | 623 | 3.050 | 3.20 | 1960 | 8.132 | 7.589 |

*** As 15th story is considerably smaller than rest of floors, this story has negligible effect on period.**

TABLE 3

COMPARISON OF CALCULATED AND MEASURED PERIODS

| Phase No. | Direction | Assumed System Contributing to Weight & Stiffness | First Mode | | | Second Mode* | | | Third Mode* | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Calc. | Meas. | % Var. | Calc. | Meas. | % Var. | Calc. | Meas. | % Var. |
| 5 (1) | Longit. | Rigid steel frame + concrete encased spandrel @ 5th floor | 1.09 | 1.13 | 3.5 | 0.44 | 0.43 | 2.3 | 0.27 | 0.29 | 6.9 |
| | Transv. | Rigid steel frame only | 1.07 | 1.05 | 0.9 | 0.43 | 0.43 | 0 | 0.26 | 0.22 | 18 |
| 18 (2) | Longit. | Rigid steel frame + concrete encased spandrels + concrete column fireproofing | 1.36 (3) | 1.32 | 2.2 | 0.50 | 0.42 | 19 | 0.31 | 0.23 | 24 |
| | Transv. | Rigid steel frame + concrete column fire-proofing + gypsum block walls | 1.59 (4) | 1.49 | 6.7 | 0.56 | 0.47 | 19 | 0.33 | 0.27 | 22 |

(1) Phase 5: Weight of steel frame and cellular steel deck plus weight of concrete poured around 5th floor spandrels. The typical concrete spandrel section encases the structural steel and adds three feet of concrete wall above the steel.

(2) Phase 18: Total weight of nearly completed building.

(3) With only steel frame contributing to stiffness $T_1 = 2.23$ sec. With only steel frame + encased spandrels contributing to stiffness $T_1 = 1.83$ sec.

(4) With only steel frame + concrete column fireproofing contributing to stiffness $T_1 = 2.06$ sec.

* Where measured values are not available, values are scaled from Figures 7 and 8.