D I S C U S S I O N

BY J.G. BOUWKAMP *

Research on the Structural Damping of Steel Trussed and Framed Multi-Storey Buildings **

ABSTRACT

This paper describes the general objectives of an investigation on the dynamic response of steel multi-storey buildings during different stages of construction. Information regarding the test sequence, and the test arrangements are given. Preliminary results on a 15-storey bare moment-resistant steel frame with concrete floor slabs indicated a low percentage of critical damping. The preliminary results show an excellent correlation between experimentally obtained frequencies and analytically derived values. The anticipated program of analysis to supplement the experimental research is discussed briefly.

DESCRIPTION OF FACILITIES

The main portion of the new structural complex at the Medical Center of the University of California in San Francisco consists of two multi-storey buildings which will house laboratories and other educational facilities, see Figure 1. The total heights of the East Building and the West Building are 195 ft. (15 stories) and 208 ft. (16 stories), respectively. The dimensions of the floor plan for these two identical buildings is 107 ft. by 107 ft. The steel columns, placed along a line 6 ft. 8½in. in from the exterior glass walls, enclose a space of 93ft. 7in. by 93 ft. 7 in., entirely free from columns. These columns - 12 in total - are placed along the perimeter of this center core only and are located on center distances of 30ft. 1⅝in., 33ft. 4in. and 30ft. 1⅝in., respectively (total 93ft. 7in.).

The corridors around the center core provide the space for the horizontal traffic flow. The vertical flow is made possible by a single free standing elevator tower which serves both multi-storey structures. The plan of this tower is approximately 30ft. by 30ft.

Since, in case of earthquake, breakage of the glass plumbing for the chemical liquids necessary for the several laboratories, would be disastrous for the building, and also in order to maintain the free floor space in the main buildings, a separate, free-standing, mechanical service tower for

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** As requested by Professor Emilio Rosenblueth, Reporter Session IV, this paper was presented following the presentation of the paper: "Investigation for the Earthquake Resistant Design of a Large-size Welded and Bolted Girder to Column Connections" by J.G. Bouwkamp.

each building has been designed. The plan of these towers is approximately 20ft. by 36ft.

The multi-storey buildings are moment-resistant steel frames, see Figure 2. The composite floors are designed as cast-in-place reinforced concrete slabs on steel girders, as shown in Figure 3. The elevator tower and the two mechanical service towers are steel towers with trussed walls, which are stiffened by encasing them entirely in reinforced concrete. The height of each of these towers is approximately 230 ft. Figures 4 and 5 show the bare steel truss for the East mechanical service tower and the completed concrete encased service tower for the West Building.

PROGRAM OBJECTIVES

Although present knowledge in the field of earthquake engineering assures the safe design of the structures mentioned before the described research was undertaken to obtain detailed information about the actual behavior of these structures when subjected to forced vibrations. Therefore the moment-resistant frame of the main structure (East Building), the East service tower, and the elevator tower were subjected to vibrating loads. The bare steel trussed frame of the East service tower was observed under a free-vibration condition.

The results from these experimental studies will be used to investigate the applicability of presently available programs of analysis to evaluate the complete response of such structures. The described structures offer the advantage of having almost complete symmetry. Also absence of shear walls in the East Building is advantageous for an accurate analysis. To obtain information about the basic structural system under dynamic loads, the bare frame with concrete floor slabs was investigated first. The second phase of the program will deal with a study of the response of the structure after floating partitions and exterior glass walls have been installed.

With the established correlation between theory and experiment for small loads, as applied to these structures, more reliable predictions about the dynamic response of these buildings under actual earthquake conditions can be expected. This information in turn can be used for similar structures with more complicated configurations.

PROGRAM OF INVESTIGATION

In order to evaluate the dynamic response of the several buildings it was decided to investigate these structures at different stages of construction.

The first phase of the program, which was carried out during the summer of 1964, dealt with the East Building and the associated service tower. The 15-storey main building at that time consisted of the bare steel moment-resistant frame and the reinforced concrete floor slabs. These slabs were connected to the steel girders by means of shear connectors. Figure 6 shows a schematic section through this building. The first floor of this building was actually a basement floor as noted in tne same figure.

Two vibration generators, or "snaking machines" were mounted on the roof of the building as shown schematically in Figure 7. These machines consist each of an electric motor and a drive mechanism that rotates two pie-shaped baskets, or rotors, producing a centrifugal force as a result of the rotation. The two rotors are mounted on a common vertical shaft and rotate in opposite directions thus producing a periodic sinusiodal load. When these baskets are lined up as shown in Figure 8, a maximum centrifugal force will be exerted. Based on the design criteria for the machine the maximum permissible load to be produced by each machine is 5000 lbs. The magnitude of the loads is directly related to the rotational speed of the machines and the weight of the lead plates which can be inserted in the baskets. The maximum load of 5000 lbs. can be reached for a minimum frequency of approximately 2.5 cps. In that instance however all the load weights need to be placed in the cells of the baskets. In order not to surpass the design load of 5000 lbs. at higher frequencies it is necessary to reduce the weight in the baskets. The maximum operating speed is 10 cps. The minimum practical speed is approximately 0.5 cps. With all the weights placed in the baskets the maximum load at that frequency is only approximately 200 lbs.

To obtain the maximum possible force out-put the two machines were fully synchronized and the loads exerted along either of the two axis of the structure. Figure 7 shows the synchronized load scheme for vibrations in the N-S direction. By adjusting the orientation of the baskets with respect to their common shaft the machines produced synchronized loads in the E-W direction. By synchronizing the machines 180 degrees apart, torsion loads were applied to the main building.

Lateral motions of the structures were measured by accelerometers and recorded continuously during shaking on a multi-channel galvanometer type recorder. The initial studies on the main building dealt with the N-S motion. During these tests four of the six accelerometers used in these studies were placed on the roof of the building above the column caps on the south side of the structure; (numbers 1,2,3, and 4 in Figure 7). The two remaining accelerometers (numbers 5 and 6) were placed along the center line of the roof. The latter two meters were removed after the initial tests since the record showed a pick up of secondary motions due to the vibratory action of lateral floor loads. The four remaining accelerometers showed a pure and constant translationary movement of the building in the N-S direction. By sweeping the frequency and observing the accelerometers for different machine weights, information for several acceleration-frequence response curves was obtained. To obtain the information for these curves the machine speed was increased stepwise with 0.03 to 0.10 cps. At each frequency the accelerometers were recorded only after the steady state vibration of the structure was evident. These response curves will also show the possible influence of the magnitude of the dynamic load on the damping of this structure.

Also the free vibration response of the structure was recorded after the structure had been brought in resonance and the machines had been stopped.

Following the N-S direction the force-orientation of the machines was changed to provide an E-W movement of the building. In that instance the four standard accelerometers were placed on the column caps in the west wall of the structure in order to record E-W accelerations. The same positions were maintained for the torsional studies.

In order to obtain information regarding the mode-shapes in the second phase of the field studies, four accelerometers were placed on the 5th, 8th, 11th and 14th floors as shown in Figure 6. In addition two accelerometers were placed on the roof of the building. With the information obtained from these six accelerometers the mode shapes for the first four resonance frequencies in the N-S and E-W directions will be determined.

Table 1 shows preliminary results for frequencies and % of critical damping as obtained from the free vibration tests.

During the summer of 1964 also the bare-steel trussed frame of the mechanical service tower East was tested under a force-vibration condition. Both the concrete encased elevator towers and the East service tower were tested during December 1964.

The service tower consists of three hollow shafts of which the middle one houses the stairs. (See Fig. 9). The accelerometers were placed on the top floor (two) and on four additional floors distributed over the height of the structure. Both the motions in the E-W and N-S directions under the influence of the synchronized shaker forces were studied.

With the forces applied in the N-S direction a coupled resonance motion of both the tower and the main building was noted around 3.00 cps. An observation of the structure showed that a small beam at the 3rd floor level formed the only connection between the two structures and caused the force transfer from the tower to the building. A coupling of the predominantly torsional resonance in the main building seems the main reason for this strong response. It was noted that the 2nd torsional mode frequency of the main building (see Table 1) coincided with the resonance frequency of the tower.

Future tests to be carried out during the summer of 1965 call for a further investigation of the main building after both the floating partitions and the glass curtain walls are installed.

ANALYTICAL PROGRAM

The analytical program (3) will be carried out in four phases. The first phase is directed to establishing the stiffness characteristics of each building. Comparison between the several resonance frequencies and the normalized mode shapes will provide the necessary information for an accurate analytical presentation of the true structures. Preliminary results of these analytical studies show excellent agreement with experimentally determined frequency values.

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(3) This program is carried out under the direction of Professor R.W. Clough, University of California, Berkeley, California. U.S.A.

The second phase will deal with the damping values as evaluated experimentally and substituted in the steady state program of analysis. A comparison between the motions of the actual structure and those obtained analytically from a steady state program will determine the accuracy of the present program. With the experimentally determined damping value of 0.6% of critical, a preliminary comparison of the analytically determined steady state response of the structure showed for the natural resonance frequency a very good agreement with the actually observed response.

In the third phase of this program after the stiffness characteristics and damping values are determined, the structures will be subjected analytically to actual earthquake ground acceleration records. These studies will involve the evaluation of the range of the elastic response of these structures.

The final phase of the analytical studies will deal with the non-linear response of these structures to determine their expected behavior under severe earthquakes.

CONCLUSIONS

The described structures lend themselves in an exceptional manner for both field and analytical studies. For those reasons a program of investigation which is presently carried out on three buildings, was sponsored by the American Iron and Steel Institute, New York. The preliminary results show good agreement between experimentally and analytically determined values. The strucutral steady state vibrations are induced by two shaking machines. These vibration generators have a frequency range from 0 to 10 cps. and a maximum load output of 5000 lbs. each. (4)

(4) The machines were developed under the guidance of the Earthquake Engineering Research Institute (U.S.A.) under a grant of the State of California Office of Architecture and Construction, Department of General Services.

TABLE 1 : PRELIMINARY RESULTS

Force		Frequency	Mode	% Damping
Direction	Magnitude (lbs) Single Load	cps	Number	
N.S.	485	0.78	1	—
N.S.	3200	2.25	2	0.690
N.S.	4500	2.29		0.650
N.S.	1520	2.16		0.735
N.S.	1500	2.13		0.735
N.S.	2100	3.92	3	0.735
N.S.	2000	3.75		0.690
N.S.	3200	3.72		0.583
N.S.	2850	5.40	4	0.690
N.S.	3050	5.55		0.790
N.S.	4100	5.37		0.650
N.S.	2600	7.18	5	0.920
N.S.	2610	7.20		0.920
E.W.	3200	2.25	2	0.552
E.W.	1700	2.27		0.395
E.W.	3500	2.26		0.582
E.W.	2800	2.13		0.614
E.W.	3400	3.90	3	0.735
E.W.	1550	5.39	4	0.552
N/S	–	1.32	1	—
N/S	2950	2.92	2	1.846
N/S	3350	4.89	3	1.382
N/S	—	7.17	4	—

N.S – Translational mode North-South

E.W – Translational mode East-West

N/S – Torsional mode

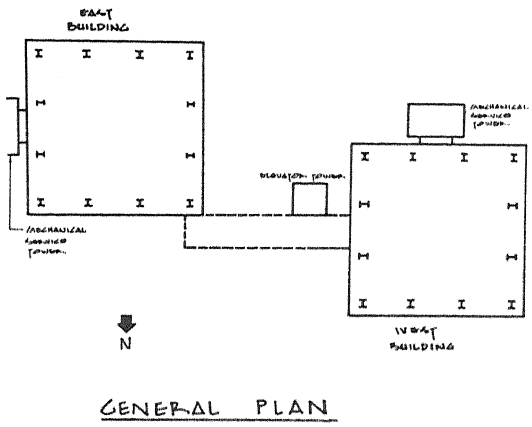


Figure 1. (General Plan)

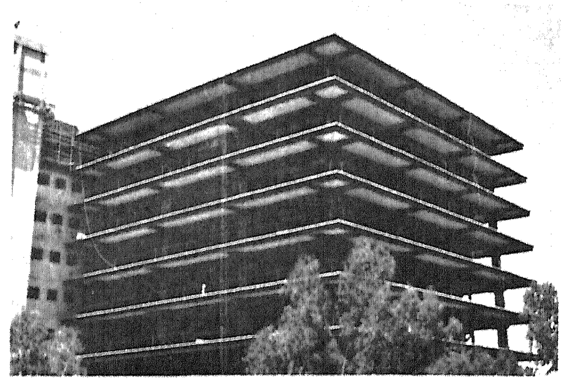


Figure 2. Upper Floors East Building

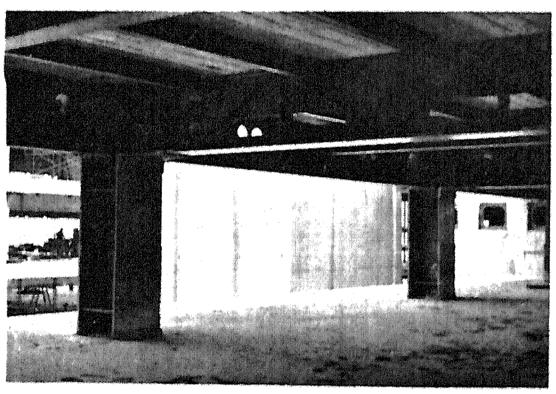


Figure 3. Interior East Building

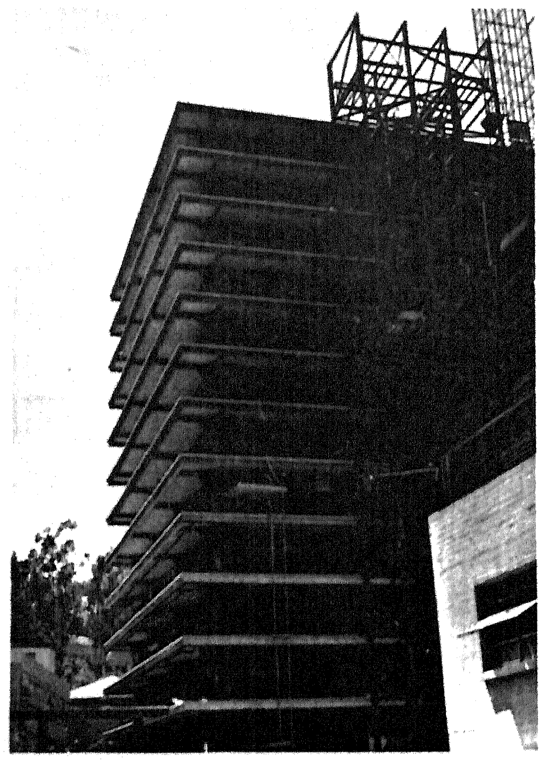
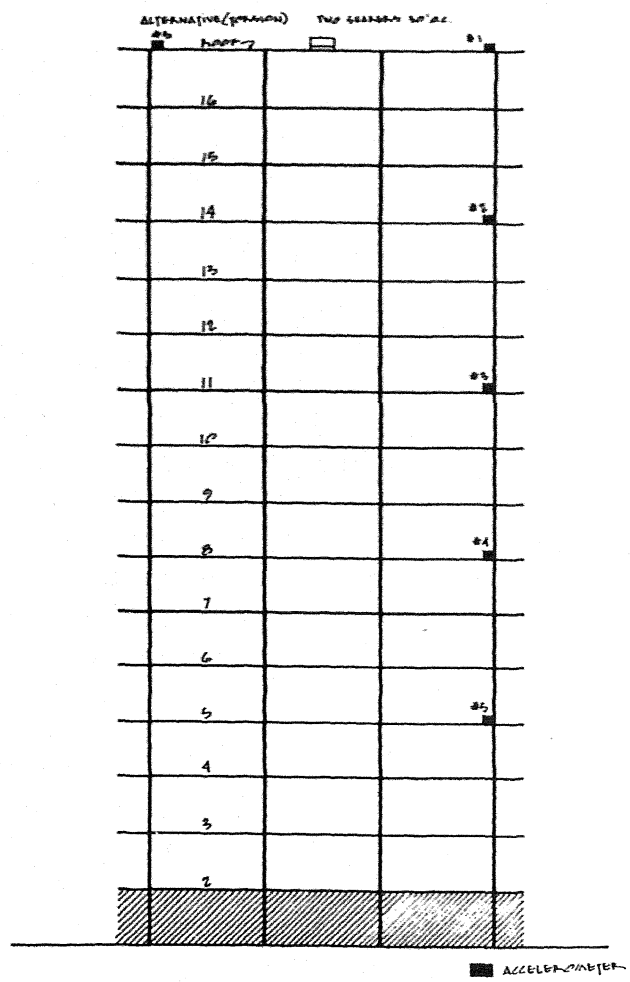


Figure 4. Trussed Frame of East Mechanical Service Tower



LOCATION OF ACCELEROMETERS FOR MODE-SHAPE

MAIN BUILDING

Figure 6. (Main Building)

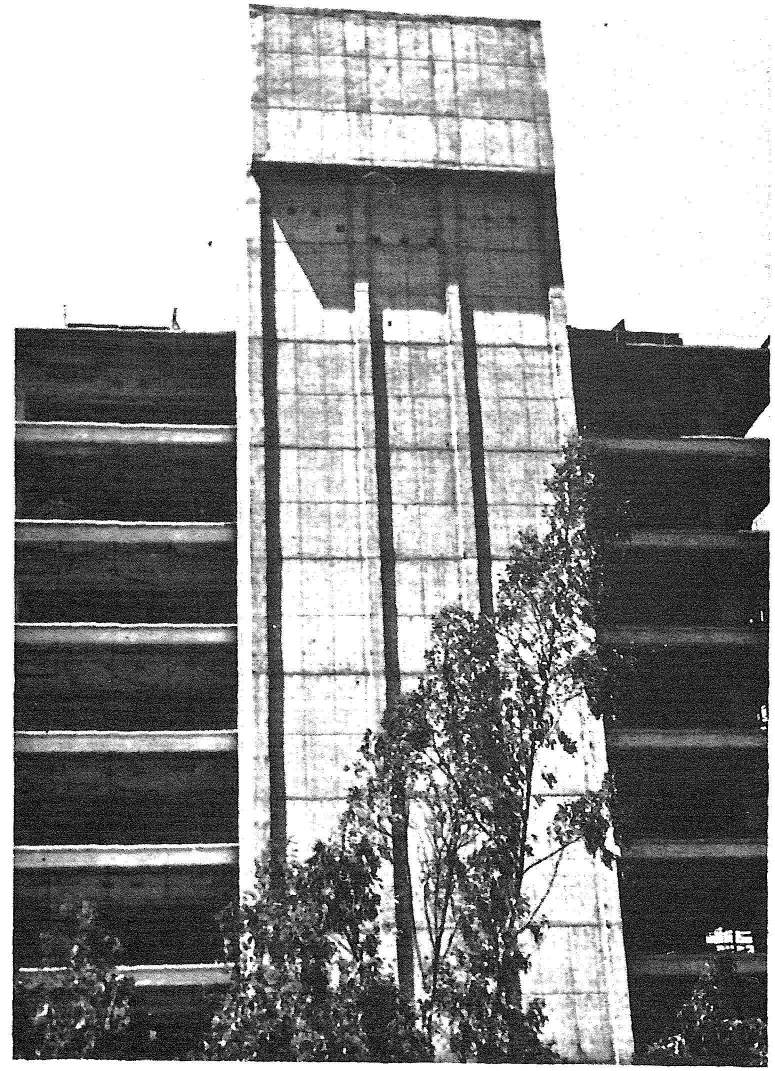
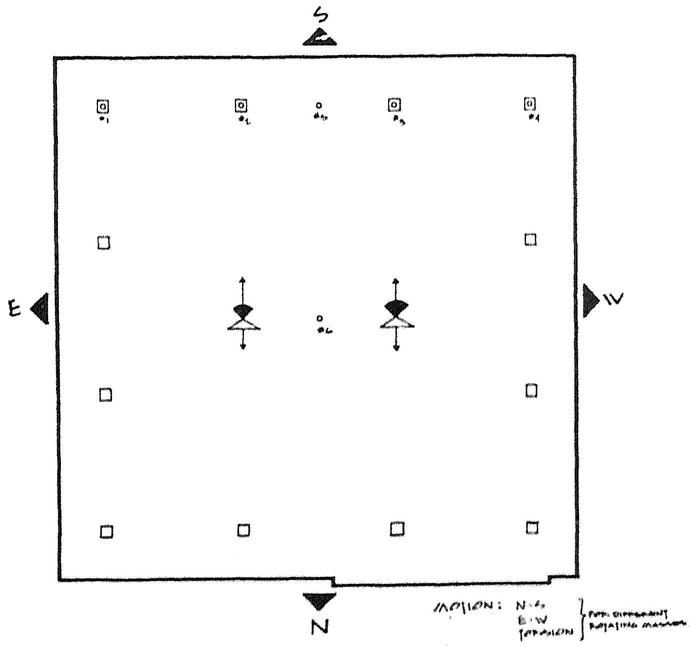


Figure 5. Completed West Mechanical Service Tower



POSITION OF SHAKERS AND ACCELEROMETERS

Figure 7. (Roof Plan)

Figure 8. Vibration Generator

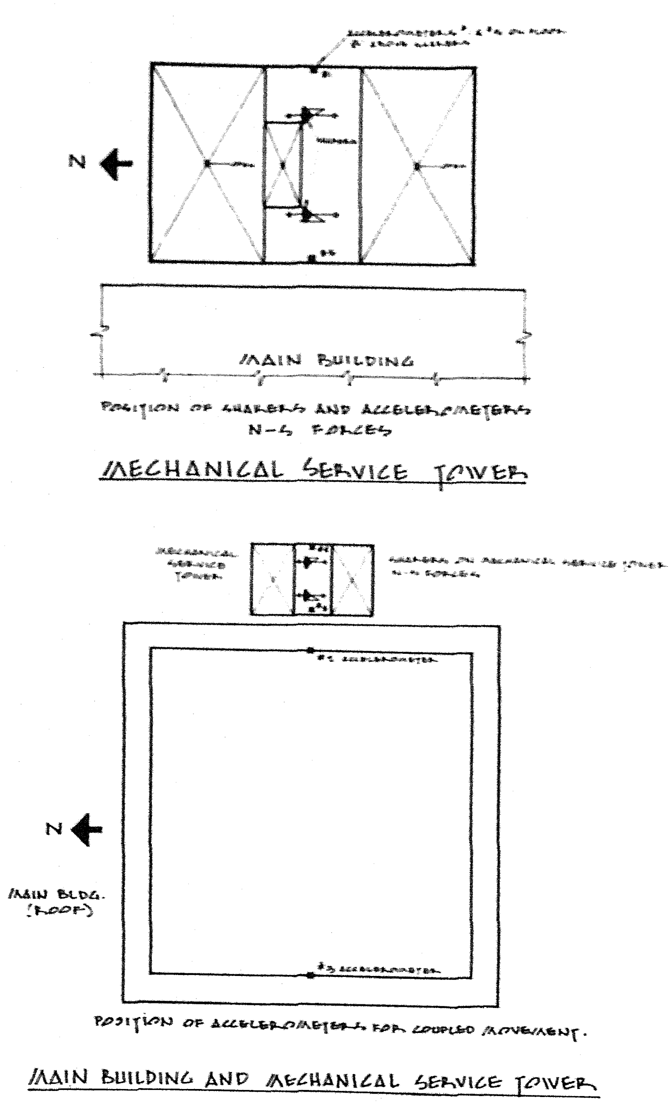


Figure 10. Placement of Accelerometers

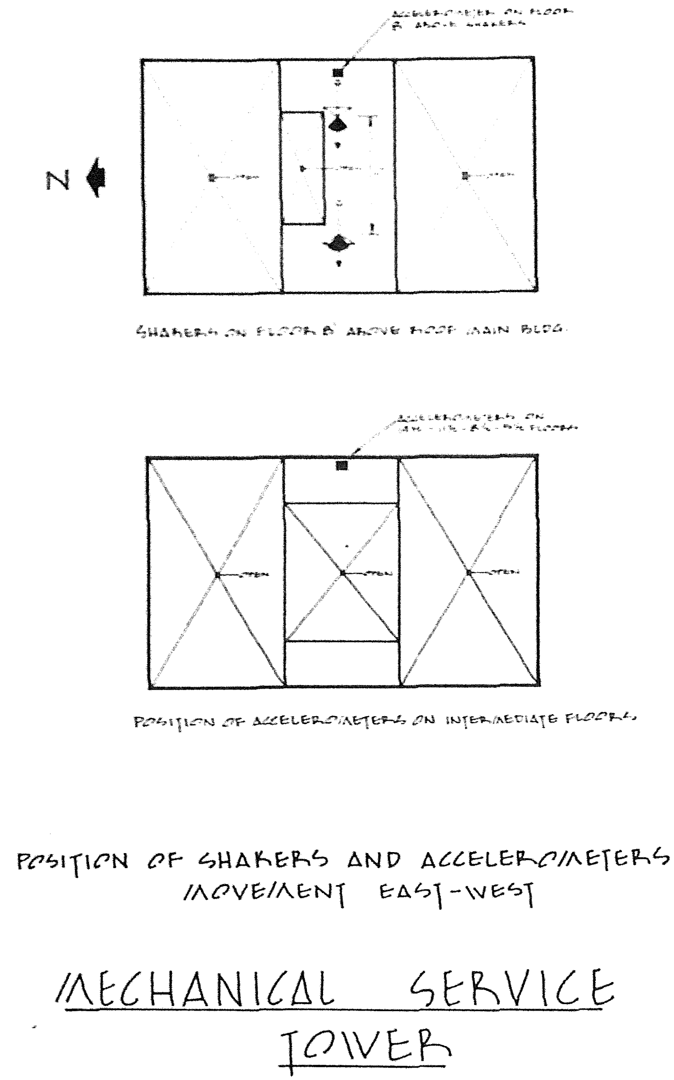


Figure 9. (Mechanical Service Tower)

RESEARCH ON THE STRUCTURAL DAMPING OF STEEL TRUSSED AND FRAMED MULTI-STOREY BUILDINGS

BY J.G. BOUWKAMP

QUESTION BY: J. PRINCE - MEXICO

I would like to ask Professor Bouwkamp why the value of damping corresponding to the first mode was not included in the table presenting the results obtained from forced vibration tests of the actual building. Secondly would it be feasible to determine this value by means of some other type of test, for instance, a pull-back test?

AUTHOR'S REPLY: The percentage of critical damping for the first mode was omitted from Table 1, since we were unable to excite the structure sufficiently during resonance. Consequently we could not observe the free vibration of the structure after the machines had come to a complete halt immediately after resonance. At higher resonance modes the machine loads producing resonance were 3 to 10 times the load which could be excited for the first mode resonance. With completely filled baskets the load at the first mode frequency of 0.78 c.p.s. was only 485 lbs. per machine.

Although it would have been possible to perform a pull-back test, it was felt that the damping values for all resonance frequencies could be determined more accurately from the frequence-response curves. During the tests scheduled for next summer we might however consider the possibility to perform a pull-back test.