DYNAMIC RESPONSE OF A 90-FT STEEL FRAME TOWER

Abstract

by N. Norby Nielsen*

After the end of the New York World's Fair a total of 17 steady-state vibration tests were carried out on the Chimes Tower in the Belgian Village. The tower is seven stories (87 feet) high and 10 feet square, it is constructed of rolled steel sections bolted together. A 30 feet diameter base slab, 2½ ft. thick supports a reinforced concrete ring 7 ft. in diameter and 3 ft. high. Four pedestals which support the column base plates are placed 90° apart with two anchor bolt per pedestal. A vibration generator was attached at the fourth floor level. In each of the principal directions of the tower, the two lowest translational modes were excited. In addition, modes were excited in which the tower moved in a diagonal plane containing two diametrically opposed columns. A total of 23 accelerometers were installed; 8 of these were located at the base of the structure to investigate in detail the translational and rotational response of the foundation. In addition, 25 strain gages were installed to measure axial and bending strains in selected members of the structure.

Representative response curves showing acceleration or strain as a function of frequency are shown in graphs, mode shapes are also shown. For all of the modes excited, complete response curves were determined at several different force levels. The resonant frequency is found to decrease with increasing stress levels. An approximate analysis of the tower was used to determine the theoretical natural frequencies and mode shapes. The theoretically determined natural frequencies are found to be about 10% higher than the experimentally determined values. Damping values were determined for all of the modes excited. In the lowest translational mode damping is between 2.0 and 3.0 per cent of critical damping, with a slight but consistent trend for damping to increase with increasing stress levels. In the second lowest translational mode damping is between 5.0 and 6.0 per cent.

At the highest level of excitation the top of the tower was displaced 1.5 inch, corresponding to an acceleration of .8 g. An earthquake spectrum analysis of the tower was made in order to compare the force levels of the dynamic tests with that which the structure would encounter during an earthquake of El Centro intensity. It was found at resonance during the highest amplitude test, the tower responded at a level corresponding to about 75% of the level the El Centro earthquake would have induced. In determining the response of the foundation it was found that rotation of the base accounted for about 10% of the motion at the top of the tower while translation of the base accounted for about 3% of this motion.

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Synopsis. A number of steady-state vibration tests were carried out on a 90-ft steel frame tower. In each of the principal directions of the tower, the two lowest translational modes were excited. Representative response curves showing acceleration or strain as a function of frequency are shown in graphs, mode shapes are also shown. Damping values were determined for all of the modes excited. At the highest level of excitation the top of the tower was vibrating with a single amplitude acceleration of 0.8 g. It was found that rotation of the base accounted for about 10% of the motion of the top story.

Introduction. In most cases where full-scale structures become available for dynamic tests the amplitudes of induced vibrations must be kept to rather low levels. Before demolition of a seven-story steel frame tower after the conclusion of the New York World's Fair permission was obtained to carry out a series of dynamic tests; in these tests the only restriction on amplitude levels were the limitations of the vibration exciter.

At the highest level of excitation the top of the tower was vibrating with a single amplitude acceleration of 0.8 g, corresponding to a single amplitude displacement of 1.5 inches. An earthquake spectrum analysis of the tower was made in order to compare the force levels of the dynamic tests with those the structure would encounter during an earthquake of El Centro intensity. It was found that at resonance during the highest amplitude test, the tower responded at a level corresponding to about 75% of the level the El Centro earthquake would have induced. Careful attention was given to instrumenting the base of the structure. In determining the response of the foundation it was found that rotation of the base accounted for about 10% of the motion at the top of the tower while translation of the base accounted for about 3% of this motion.

Description of Structure, Vibration Exciter and Instrumentation. At the time of the dynamic tests the tower was stripped leaving only the bare steel frame. The tower is seven stories high and constructed of rolled steel sections bolted together. The tower is 10 feet square with a total height of 87 feet. A general view of the tower is shown in Fig. 1. Four pedestals which support the column base plates are placed 90° apart with two anchor bolts per pedestal. The foundation consists of a base slab 30 feet in diameter, the $2\frac{1}{2}$ ft. thick base slab supports a reinforced concrete ring which is 7 ft. in diameter and 3 ft. in height. The center of the ring contains compacted soil. The ground level floor of the tower is a 6 inch. thick concrete slab placed over the reinforced concrete ring and the compacted soil. Details of the foundation can be seen in Fig. 2. All of the columns of the tower are 8 WF sections; all diagonal braces are single angles with the long leg vertical. The cross braces are connected at the floor levels and are held together at the midpoints by a single rivet.

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The vibration exciter uses counter-rotating weights so the inertia forces will be additive in one direction while they will cancel each other in the perpendicular direction. The speed control of the vibration exciter is extremely accurate. The speed can be controlled to an accuracy of about 0.1%; i.e., after running the exciter at a specific frequency, and recording the response, it is possible to change the frequency of excitation to a new value that only differs 0.1% from the previous value. The vibration exciter was attached at the fourth floor level. A detailed description of the vibration exciter has been given by Hudson (1).

A total of 23 accelerometers were installed; 8 of these were located at the base of the structure to investigate in detail the translational and rotational response of the foundation. In addition to the accelerometer readings throughout the structure, 25 strain gages were installed to measure axial and bending strains in selected members of the structure. The signals representing strain and acceleration were recorded on direct writing, magnetic oscillographs.

Results of Tests. A total of 17 steady-state vibration tests were carried out. In each of the principal directions of the tower the two lowest translational modes were excited at various force levels. In addition, modes were excited in which the tower moved in a diagonal plane containing two diametrically opposed columns; this type of motion occurred after prior tests had resulted in breaking some of the welds connecting one of the columns to its base plate. Fig. 3 shows typical response curves as the structure is excited close to and at resonance for the lowest translational mode in one of the principal directions of the tower. It can be seen how the resonant frequency decreases as the force at resonance is being increased. The behavior is typically that of a softening spring. The resonant frequency at the lowest level of excitation is 2.42 cps; decreasing to 2.32 cps at the highest level of excitation. This 5% reduction in resonant frequency corresponds to approximately a 10% reduction in stiffness of the structure.

Fig. 4 shows the response curves obtained from strain measurements. The strain gages, both for the measurement of axial strain as well as for bending strain, were attached about one foot above the ground floor to one of the columns. It can be seen that the axial strain is about seven times as large as the bending strain. It is interesting to note that if one measures damping from the width of the response curves almost identical results are obtained whether acceleration response or strain response curves are used.

Analysis of Structure. The structure was analyzed in order to compare the results from the dynamic tests with the results that would be predicted from an analysis of the structure. The tower is assumed to be a pin-ended truss with the columns pinned to a rigid base. For any displaced configuration only the cross braces in tension are assumed to be effective. The assumptions mentioned above are reasonably in view of the experimental results since the recorded axial and bending strains clearly show that bending strains are rather small compared with axial strains. Also, the motion of the foundation was found to be rather small. A comparison of the mode shapes and frequencies is shown in Fig. 5. The analytically determined mode shape of the lowest translational mode is almost identical to that determined experimentally while there is some deviation in the similarly determined mode shapes of the second lowest translational mode. The theoretically determined frequencies are roughly 10% higher than those found experimentally. Considering the number of simplifying assumptions made in the analysis, the agreement is quite good.

Damping. It is well known that only by using very accurate steady-state response curves is it possible to obtain accurate values of damping. For structures with low values of damping, this will require an exceedingly accurate frequency control of the vibration exciter. The high degree of accuracy in the determination of response curves for the present tests is clearly indicated from the response curves shown in Figs. 3 and 4. Damping is probably a combination of several types of energy absorption. However, as long as the total damping in a structure is reasonably small, say less than 10%, mixed damping can be treated satisfactorily by the use of equivalent viscous damping.

The values of damping from some of the tests are shown in table 1. Tests 1 and 2 were made before excavation of the soil surrounding the foundation pedestal. In table 1 "strong" direction refers to the direction parallel to the web of the columns; "weak" refers to the perpendicular direction. It can be seen that the values of damping stay quite constant in the lowest translational modes; for all levels of excitation damping values fall in the range of 2 - 3% of critical damping. There is some indication of an increase in damping as the force level is being increased, but even at the highest level of excitation, the value of damping does not exceed 3.1% of critical damping. Damping in the second lowest translational mode is 5 - 6% of critical. Since damping increases almost proportional to resonant frequencies, it can be concluded that a reasonable presentation of the damping mechanism would involve interfloor dashpots i.e., relative damping.

It is of interest to compare the damping values above with those obtained from dynamic tests of two other full-scale structures. It should be noted that while the damping values listed above were obtained in tests at very high levels of amplitude, the damping values that follow were obtained at much lower amplitude levels. One of the buildings tested is a nine-story steel frame building with plan dimensions 220 ft. by 40 ft. At the time of the vibration tests, the building was completed structurally, but non-structural elements such as curtain walls, windows and interior partitions were not yet installed. A complete description of the structure and the results of the dynamic tests are given in (2).

From steady-state vibration tests in the short direction of this building, damping in the lowest translational mode was found to be 0.5% with only a slight indication of an increase in damping as the force level was increased. The second lowest translational mode had damping ranging from 1.1% to 1.6% increasing with an increase in force level. The third lowest translational mode had a damping value of 3.7%. The values of damping are close to being proportional to the resonant frequencies, again here it is concluded that the damping mechanism is best described by interfloor dashpots.

The other building subjected to an extensive series of dynamic tests is a modern five-story reinforced concrete building, plan dimensions are 125 ft. by 25 ft. The vibration tests were carried out when the building was ready for occupancy. A description of the structure and the results of the vibration tests are given in (3). Damping in the lowest translational mode was found to be 2.2% with no indication of an increase as the force level was increased. In the second lowest translational mode damping was found to be 2.1%. The values of damping in the two modes are nearly equal. This indicates that the mathematical model that represents the damping mechanism will have not only relative (interfloor) dashpots, as in the case of the two steel frame structures, but absolute dashpots as well.

Earthquake Spectrum Analysis of the Tower. It is of interest to make an approximate comparison of the level of excitation reached during the dynamic tests with the level of excitation the tower would experience during a strong motion earthquake. It is here assumed that the structure can be analyzed as a single-degree-of-freedom system responding according to the experimentally determined frequency, mode shape and damping in its lowest translational mode. The earthquake response spectrum chosen was an average spectrum of California strong motion earthquakes, each scaled to have a maximum acceleration of .33 g, which is the intensity recorded at El Centro 1940. Using the experimentally determined values of frequency (2.32 cps) and damping (2.8%) for the lowest translational mode the maximum values of displacement, velocity and acceleration for a one-degree-of-freedom system can be determined from the earthquake spectrum. Using the experimentally determined mode shape, the participation factor for the mode and using d'Alembert's principle the forces acting at each floor level can be determined. The results are shown in table 2. The total base shear from this analysis was found to be 12.57 kips while the total base shear corresponding to the dynamic test with the largest amplitude (Test 16, Fig. 3) is 9.55 kips. The total base shear corresponding to test 16 is about 75% of the base shear that would correspond to the El Centro earthquake. It should be noted that the weight of the stripped tower was about 25 kips while the tower in its original state weighed about 100 kips. The lateral wind forces for which the tower was designed correspond to a base shear of almost 25 kips.

Motion of Foundation. The foundation of the tower was supported on rather poor soil, loose to medium dense cinder and miscellaneous fill to a depth of about 7 m. For all of the tests, both translational and rotational movements of the foundation were measured. It was found that the base motion accounted for only a minor portion of the total response. Rotation of the base accounted for about 10% of the total response of the top floor, while translation of the base accounted for about 3% of the total response of the top floor. The vertical accelerations measured on the base slab are shown in Fig. 6. The two accelerometers were placed at the edges of the base slab with a 30 ft. distance between them. The record showed clearly the 180° phase angle between the two accelerations. Different scales are used in Fig. 6 in order to separate the two response curves. Considering the poor soil the motion of the base was less than expected. However, as noted above the weight of the tower at the time of the dynamic tests was only one quarter of the original weight so the normal bearing stress on the soil was very low. It is of interest to compare the motion of the base with that determined by Japanese investigators. Kawasumi and Kanai (4) report that in small amplitude vibration tests of a seven-story reinforced concrete building the translation of the base accounted for 40% of the total response of the roof, while rotation of the base accounted for about 20% of the total response of the roof. The large difference in test results is probably due to the fact that the Japanese building is a much more rigid building; this would tend to facilitate the rigid body translation and rotation to a higher degree than would be the case for a flexible structure.

Motion in a Diagonal Direction. As noted earlier very high accelerations were induced in test 16 (Fig. 3); at the very end of this test it was noted that the welds at the base of one of the columns were broken. It was decided to run an additional test with the tower in this deteriorated condition. The top portion of Fig. 7 shows the acceleration response of the sixth floor in the direction parallel to the principal direction of the tower in which the maximum force is applied. The response shows two peaks, at 2.17 cps and at 2.36 cps. Fortunately, in all of the tests the acceleration response of the fifth floor level was recorded in both principal directions of the tower. In all of the tests, up to test 16, the acceleration record from the accelerometer positioned perpendicular to the direction in which the maximum force was applied would show almost no response. As can be seen in the lower portion of Fig. 7 the magnitudes of the accelerations measured in the two perpendicular directions are almost identical at both 2.17 cps and at 2.36 cps. The two responses are in phase at 2.17 cps and 180° out of phase at 2.36 cps. It is evident that the resonance at 2.17 cps corresponds to a motion in a diagonal direction while the motion at 2.36 cps occurs in the other diagonal direction.

Acknowledgements. The firm of Wiss, Janney, Elstner and Associates was responsible for the execution of the entire test program involving several structures at the site of the New York World's Fair. John F. Wiss of the same firm was in charge of the dynamic test program on the Chimes Tower. The program was conceived by the American Society of Civil Engineers. The responsibility of raising funds and subsequent administration of the project were assumed by the National Academy of Sciences through the Building Research Advisory Board. Professor V. J. McDonald, University of Illinois, was in charge of all instrumentation. Much of the instrumentation was provided by the U.S. Corps of Engineers through the Experimental Station at Vicksburg, Mississippi, and the Aberdeen Proving Grounds at Aberdeen, Maryland. The vibration exciter and auxiliary equipment, which belong to the Division of Architecture, State of California, were supplied by the Department of Engineering, University of California at Los Angeles, California. Two graduate students in the Department of Civil Engineering. University of Illinois, Mr. Jerry O. Blackburn and Rudolf J. DuPreez, reduced the data and helped in analyzing the data.

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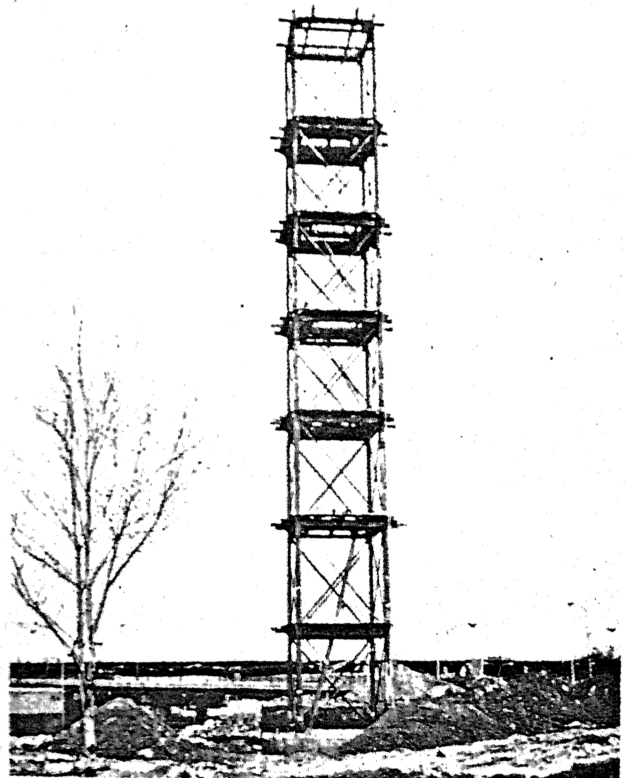
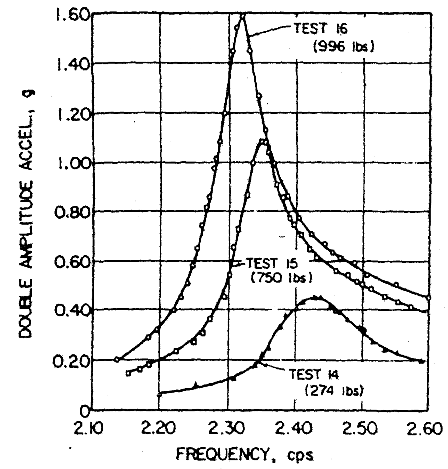


Fig.1 GENERAL VIEW

Fig.2 FOUNDATON



ACCELERATION RESPONSE OF 8 th FLOOR

Fig. 3 RESPONSE CURVES, LOWEST TRANSLATIONAL MODE

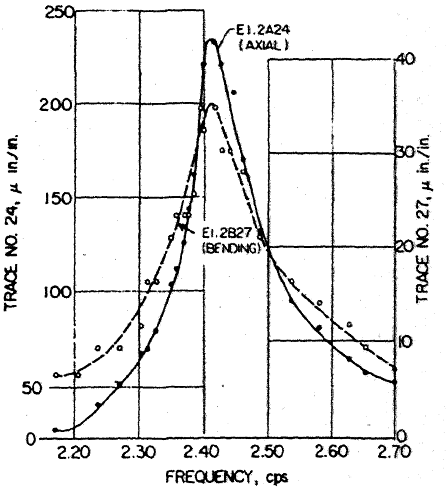


Fig. 4 RESPONSE CURVES, STRAIN MEASUREMENTS (Double amplitude) TEST 5

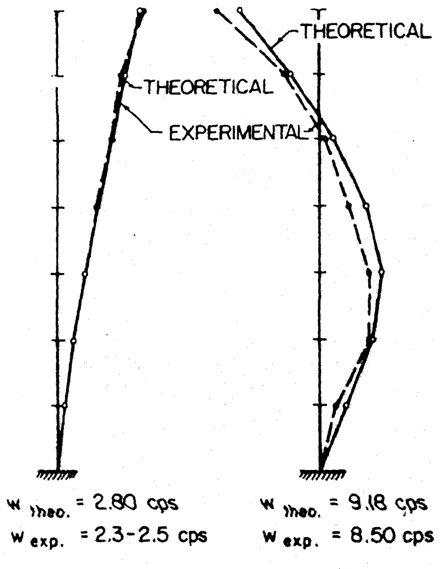


Fig. 5 MODE SHAPES AND FREQUENCIES

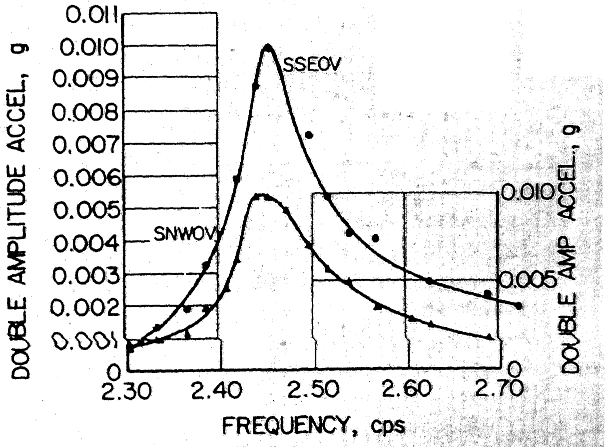


Fig. 6 VERTICAL ACCELERATION, BASE SLAB, TEST 4

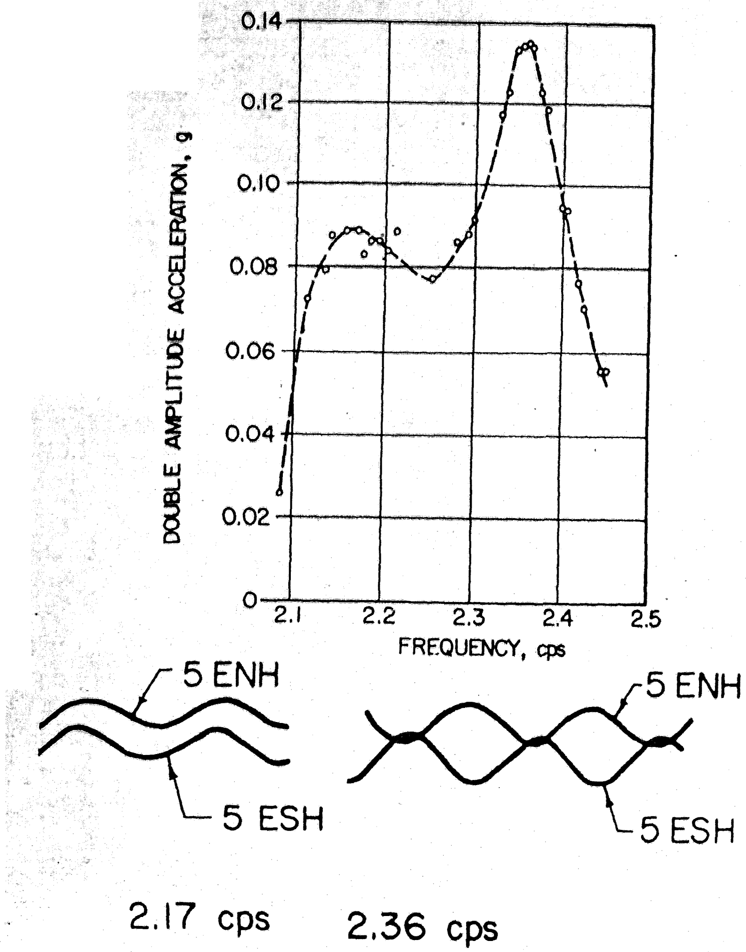


Fig. 7 RESPONSE OF SIXTH FLOOR, TEST 17

DAMPING, LOWEST TRANSLATIONAL MODE, "STRONG" DIRECTION

TEST NO.	FORCE AT RESONANCE LBS	8 th FLOOR SING AMPL. ACCEL., g	RESONANT FREQ. CPS	DAMPING PERCENT
1	49.0	.057	2.54	1.9
2	285.5	.29	2.51	2.2
4	276.0	.29	2.45	2.1
5	525.0	.44	2.38	2.7
		"WEAK" DIRECTION		
12	276.0	.21	2.44	3.1
13	34.0	.037	2.46	2.0
14	274.0	.22	2.43	2 9
15	750.0	.55	2.32	3.1
16	996.0	.80	2.32	2.8

TABLE 1 DAMPING VALUES

	TEST 16	EL CENTRO EQ.	WIND DESIGN
8 th FLOOR	1.91 kips	2.50 kips	3.0 kips
7 th	2.13	2.85	4.0
6 th	1.82	2.42	5.0
5 th	1.42	1.91	3.4
4 th	1.35	1.69	3.7
3 rd	.69	.90	2.9
2 nd	.23	.30	2.8
TOTAL BASE SHEAR :	9.55	12.57	24.8

TABLE 2 LATERAL FORCES