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CASE STUDY OF DYNAMIC SOIL-STRUCTURE INTERACTION

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Presented to ASCE Annual and National Environmental Engineering Meeting, Houston, Texas

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Following the close of the World's Fair in New York, dynamic shaking tests were made on several buildings. The results for the Chimes Tower at the Belgian Pavillion were particularly interesting. The data revealed a small but definite amount of foundation rocking whose effect was not analyzed in the original study reported by Nielsen and Wiss. The present paper discusses the reported findings of Nielsen and Wiss and the results of this later case study of soil-structure interaction at the Chimes Tower. A rocking spring constant was determined directly from measurements of foundation movement. Using this rocking spring constant in a dynamic model for the structure improved the agreement between predicted and observed resonant frequencies. When a horizontal spring was also introduced, having a constant consistent with direct measurements of foundation movement, the dynamic model for the structure introduced a new mode that was not observed. Therefore, the damping associated with the horizontal spring must have been large enough to suppress the additional mode. Thus, in an indirect way, the test results for the Chimes Tower confirm that large damping is associated with horizontal translation interaction. The exact value of this damping cannot be deduced from the test data. It can only be concluded that the damping was quite large in general agreement with the theory for foundation interaction.

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CASE STUDY OF SOIL-STRUCTURE INTERACTION

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INTRODUCTION

The phrase dynamic soil-structure interaction denotes the effect of the soil immediately beneath a structure upon the dynamic response of the structure. Soil-structure interaction affects the natural periods of a structure, and the forces and motions induced in the structure. Evaluation of interaction is basic to the design of foundations for machines and for large dynamically loaded structures such as radar towers (5, 7). Recently, there has been considerable interest in the effect of soil-structure interaction upon the response of buildings to earthquakes (4, 9, 12).

Three different methods have been used to represent the soil in studies of soil-structure interaction: elastic half-space theory, finite elements, and equivalent foundation springs, masses and damping. All of these methods have their advantages, and important contributions have been made using each method. The use of equivalent foundation

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parameters has very important advantages for design studies and applied research: the simplicity of the method encourages parametric studies and allows the engineer to utilize his experience and judgement effectively.

Various procedures for evaluating equivalent foundation parameters have been proposed (1, 5, 10). Unfortunately, there have been very few well-documented case studies which can be used to judge the adequacy of these various procedures for determining equivalent parameters. As a result, there is contention about the values that should be used for some of these parameters. This is especially true of the choice of damping to be associated with the horizontal translation (swaying) of a foundation relative to the surrounding soil (Fig. 1). According to theory, this damping should be quite large (>20%). However, because of the lack of good field evidence, a conservative approach often is used (6, 9). For example, the Atomic Energy Commission currently (1972) permits a maximum of 10% damping in analysis of soil-structure interaction at nuclear power plants.

Following the close of the World's Fair in New York, dynamic shaking tests were made upon several buildings and excellent reports by Nielsen and Wiss are available (2, 3). The results for the Chimes Tower at the Belgian Pavilion were especially interesting. The data revealed a small but definite amount of foundation rocking whose effect was not analyzed in the original study. The present paper first shows that inclusion of rocking into the dynamic analysis improves the agreement between predicted and observed frequencies. Then the effect of foundation swaying is analyzed, leading to a conclusion concerning the magnitude of the damping associated with swaying.

THE CHIMES TOWER

The Chimes Tower was a seven-story structure constructed of rolled steel sections bolted and/or welded together (Fig. 2). In plan the structure was square. At the time of the tests, the cladding had been stripped from the structure so that there was only the bare steel frame.

The foundation (Fig. 3) consisted of a base slab with a diameter of 30 feet, which in turn supported a concentric reinforced concrete ring. The columns of the structure were supported on this ring. The center of the ring was filled with compacted soil and covered with a reinforced concrete slab 6 inches thick.

The soil at the site consisted of an upper layer of cinders, whose density varied from loose to dense, overlaying a thick layer of organic clay silt. During most of the dynamic tests, the soil surrounding the foundation was excavated away from the sides of the foundation.

The total weight of the building was 25 tons (22,700 Kg), and the foundation also weighed 25 tons.

RESULTS FROM FIELD TESTS

The dynamic excitation was provided by a mechanical oscillator mounted on the 4th floor. Both the frequency and the magnitude of the oscillation could be varied independently. Accelerometers were mounted at all floors of the structure and also on the foundation; both vertical and horizontal motions of the foundation were recorded.

The first and second resonant peaks were identified by scanning a range of frequencies. Then a series of tests were carried out in a which the first resonant peak was studied in detail using small increments in frequency. Table 1 summarizes the resonant frequencies determined by the tests.

The strong direction refers to the direction of excitation parallel to the webs of the columns. Since the resonant frequencies were quite similar for the two directions of excitation, the lateral resistance apparently resulted primarily from truss actions; that is, the bending resistance of the columns was unimportant.

During the series in the strong and weak directions, the magnitude of excitation was gradually increased, and as a result, the first resonant frequency decreased somewhat. During the later stages of each series, welds were broken by the vibrations, and then the first resonant frequency decreased sharply. The welds broken by excitation in the strong direction were repaired prior to the series with excitation in the weak direction.

Fig. 4 shows the deflections observed in one of the early tests with excitation in the strong direction. The vertical and horizontal motions measured at the foundation approached the limit of sensitivity of the accelerometers. However, the measured foundation movements were reasonably consistent from test to test. The pattern of vertical motion of the foundation suggests that the foundation was not completely rigid. The rotation of the base of the tower is related to the difference between the vertical movements of the two inner measurement points. For three tests, the contributions of the base rotation to the total movement at the top of the tower were:

Test 4	Test 6	Test 7
11%	14%	16%

These percentages suggest the possible importance of soil-structure interaction.

In addition to the dynamic tests, the tower was subjected to static horizontal forces of various magnitudes by means of two cables attached to the 7th floor and pulled from a distance of 250 feet. Deflections of the structure and of the foundation were recorded. Again there were detectable foundation movements, although they were not much greater than the precision of the measuring devices.

DYNAMIC ANALYSES IGNORING SOIL-STRUCTURE INTERACTION

The dynamic response of the Chimes Tower was discussed at length in the original reports by Nielsen and Wiss (2, 3). The tower was modeled as a linear lumped mass system with each floor assumed to be completely rigid. The stiffness matrix was determined assuming that the tower behaved as a pin ended truss, with the diagonal braces in compression contributing nothing to the stiffness. The computed fundamental and second natural frequencies are given in Table 2. These results assume complete fixity at the foundation level. The computed natural frequencies were judged to be in reasonable agreement with the observed resonant frequencies. There was also reasonable agreement between the shape of the computed mode slopes and the shape of the observed deflection curves at the resonant frequencies. In addition, the deflections during the static pull tests could be predicted reasonably well using the stiffness matrix developed for the dynamic analysis.

Nielsen and Wiss determined damping from the amplitudes of motion at resonance. For the first resonant frequency, the damping ratio was between 2% and 3% with a slight trend toward larger values with the increasing exciting force. The damping ratio for the second resonant frequency was evaluated as 5.2% in one test and 6.2% in another. The writers employed a method based upon the width of the response peaks, and found a much smaller (1.5%) damping ratio for the second resonant peak.

The writers repeated the earlier analysis using a slightly different approach. Again the tower was modeled as a truss with pin connections ignoring the stiffness of diagonal braces in compression, but now a full two-dimensional analysis was made using the dynamic capability of ICES STRUDL. The mass at each floor level was lumped in equal parts at each of the pin connections. The results of this analysis are also given in Table 2. As expected, the results are very similar to the earlier results of Nielsen and Wiss.

The writers also examined the assumption that only the tension diagonals contributed to stiffness. This assumption is quite important; including the stiffness of the compression diagonals would increase the natural frequencies to 3.1 and 11.4 cps. It was concluded that the compression diagonals must indeed have buckled, so as not to contribute to lateral resistance, for the amplitudes of motion in the field tests.

EVALUATION OF FOUNDATION STIFFNESS

Rotational Spring Constant

This spring constant, k_R, expresses the ratio of the dynamic moment, M_f, on the base of the foundation to the corresponding dynamic rotation, ϕ_f:

$$k_R = \frac{M_f}{\phi_f} \tag{1}$$

The dynamic moment can be evaluated directly from the test results as:

$$M_f = \Sigma m_i h_i \ddot{x}_i + m_f h_f \ddot{x}_f + I_f \ddot{\phi}_f \tag{2}$$

where m_i = mass of ith floor

h_i = height of ith floor above base of foundation

$\ddot{x}_i$ = horizontal acceleration of ith floor

m_f = mass of foundation

h_f = height of CG of foundation above base of foundation

$\ddot{x}_f$ = horizontal acceleration of foundation

I_f = moment of inertia of foundation about horizontal axis through its base

$\ddot{\phi}_f$ = rotational acceleration of foundation.

At the first resonant frequency, $\ddot{x}_i$, $\ddot{x}_f$, and $\ddot{\phi}_f$ were all measured. Since the remaining quantities can all be determined from the properties of the structure, it should be possible to determine M_f with considerable accuracy.

Eqs. 1 and 2 have presumed that the foundation is completely rigid, but, as shown by the measured motions in Fig. 4, the foundation apparently was not actually rigid. This apparent lack of rigidity was confirmed by an approximate analysis using the theory for a beam resting upon an elastic foundation. Non-rigidity introduces two complications: care must be taken in evaluating ϕ_f in Eq. 1 and the third term of Eq. 2.

The third term of Eq. 2 was evaluated as $I_{ef}\ddot{\phi}_f$, where I_{ef} is an equivalent moment of inertia. I_{ef} was determined using the assumed deflected shape shown in Fig. 5 in place of the usual assumption of rigid body rotation. Thus, the control core of the foundation was assumed to rotate as a rigid body, while the slope of the outer part of the foundation was 0.3 times the rotation of the core. This reduction factor was selected from examination of measured deflections such as those in Fig. 4. The resulting value of I_{ef} was about 40% of the moment of inertia for a completely rigid foundation. Similar results were obtained using other assumptions concerning the deflected shape of the foundation.

On this basis ϕ_f is the rotation of the central part of the foundation. This is the rotation actually experienced by the bottom of the structure. For each test, ϕ_f was determined by fitting the deflected slope in Fig. 5 to the measured deflections.

Using these values of I_{ef} and ϕ_f, k_R was computed using the results from Tests 4, 6 and 7. The results ranged from 3.03 x 10^9 lb-ft/radian to 3.13 x 10^9 lb-ft/radian (4.11 x 10^9 to 4.25 x 10^9 N-m/rad.) -- an extremely small scatter indeed! It should be noted that the value used for I_{ef} was rather unimportant, since the first term on the right side of Eq. 1 contributed over 98% of the dynamic moment.

The rotational spring constant was also evaluated from the results of the static pull tests. Using vertical motions measured at the edges of the foundation, the apparent k_R was 1.5 x 10^9 lb-ft/rad. (2 x 10^9 N-m/rad.). Because of the non-ridigity of the foundation, the actual static spring constant was greater than this value. Also, as already noted, the static foundation movements were not measured precisely. In view of these difficulties, the static value of k_R is in reasonable agreement with the values deduced from the dynamic tests.

According to elastic theory, the rotational spring constant for a circular foundation is (1, 5, 10):

$$k_R = \frac{8GR^3}{3(1-u)} \tag{3}$$

where G = shear modulus

R = radius of foundation

u = Poisson's ratio

Using k_R = 3 x 10^9 lb-ft/rad. (4.07 x 10^9 N-m/rad), R = 15 ft. (4.57m) and u = 0.4, the shear modulus G = 200,000 lb/ft^2 (9.58 x 10^6 N/m^2). Assuming a unit weight of 100 pcf (1.6 x 10^3 Kg/m^3), this modulus corresponds to a shear wave velocity of 250 ft/sec (77 m/sec). No field measurements are available for comparison with this backfigured value, which seems low but not unreasonable for cinders and organic clay silt. The backfigured modulus should be less than the actual modulus because of the non-rigidity of the foundation; that is, Eq. 3 overestimates the spring constant for a non-rigid foundation.

Horizontal Spring Constant

In a similar fashion, the horizontal (swaying) spring constant k_H is the ratio of the dynamic base shear V_f to the corresponding horizontal motion x_f of the foundation:

$$k_H = \frac{V_f}{x_f} \tag{4}$$

The base shear can be evaluated from the measured accelerations by:

$$V_f = \Sigma m_i \ddot{x}_i + m_f \ddot{x}_f \tag{5}$$

However, the determination of k_H was not as satisfactory as the determination of k_R, since the second term in Eq. 5 was important and involved the product of a large mass and a small and rather inaccurate acceleration. Resulting values of k_H ranged from 0.75×10^7 lb/ft (1.1×10^8 N/m) for test 4 to 1.05×10^7 lb/ft (1.5×10^8 N/m) for test 6.

According to elastic theory, the horizontal spring constant for a circular foundation is (5, 10):

$$k_H = \frac{8GR}{2-u} \quad . \tag{6}$$

Combining Eqs. 3 and 6 gives:

$$k_H = \frac{3(1-u)}{2-u} \frac{k_R}{R^2} \tag{7}$$

Using $k_R = 3 \times 10^9$ lb-ft/rad, R = 15 ft. and u = 0.4, the horizontal spring constant is $k_H = 1.5 \times 10^7$ lb/ft (2.2×10^8 N/m). Because non-rigidity of the foundation reduces k_R more than k_H, an argument can be made that this computed k_H is too low.

DYNAMIC ANALYSES INCLUDING SOIL-STRUCTURE INTERACTION

The dynamic analysis made using ICES STRUDL was modified to include the effect of foundation flexibility and foundation inertia, and a number of computer runs were made. The same rotational spring constant was used in all analyses: $k_R = 3 \times 10^9$ lb-ft/rad (4.07×10^9 N-m/rad). The horizontal spring constant and the rotational inertia of the foundation were varied to cover the uncertainties in these parameters. The following values were used:

k_H: 0.96×10^7 and 1.6×10^7 lb/ft and infinity
(1.39×10^8 and 2.32×10^8 N/m)

I_{ef}: 210,000 and 480,000 lb-ft-sec^2 (8850 and 20,200 Kg-m^2)

The variation in I_{ef} covered the possible effective inertia of the soil as well as the uncertainty in the moment of inertia of the foundation itself. The mass of the foundation was the same in all runs, since theory indicates that the effective mass of the soil is small for swaying (5,10). The natural frequencies computed by these analyses are given in Table 2.

The fundamental computed frequency was affected very little by the assumptions concerning k_H and I_{ef}. Introducing the rocking spring thus decreased the computed fundamental frequency from 2.76 cps to 2.44 cps, a reduction of about 12%. As a result of this reduction, the computed fundamental frequency was brought into excellent agreement with the first observed resonant frequency. The computed mode shape agreed well with the observed deflections (Fig. 6).

Similarly, the third computed natural frequency was affected little by the choice of k_H and I_{ef}. The frequency and modal shape for this third mode were very similar to the frequency and deflected shape of the second observed resonant frequency, and to the frequency and modal shape of the second mode for a fully fixed tower. Introducing the foundation flexibility actually improved the agreement between observed and predicted frequencies.

Thus, for two modes of the structure, computed results were still consistent with observed results when foundation flexibility was introduced. In fact, introducing the foundation flexibility substantially improved the agreement at the first resonant frequency, while the agreement at the second observed resonant frequency was somewhat improved.

The New Mode

However, when a finite value of k_H was used, a new mode appeared that was not observed in the tests and was not predicted by the theory for a fully-fixed foundation. As seen in Fig. 6, this mode involves considerable horizontal deflection at the foundation; that is, the swaying spring participates strongly in this mode. As might be expected, the frequency of this mode is quite sensitive to the choice of k_H. When k_H is assumed very large, this mode disappears.

At first sight, the appearance of this mode might seem to discredit the theory for soil-structure interaction. However, quite the reverse is true. In an indirect way, the fact that the computed mode was not observed provides proof of the theory for damping associated with swaying of a foundation.

Fig. 7 gives response curves computed using the ICES STRUDL program including foundation flexibility. Dampings determined from the field tests by Nielsen and Wiss were assigned to the 1st and 3rd modes. Two different assumptions concerning damping were made for the 2nd mode.

1. A value of 3.5% intermediate between the values for the 1st and 3rd modes. This value would appear to be suitable if there were no soil-structure interaction.
2. An arbitrarily selected large value of 50%. The theory for weighted modal damping says that the damping in this mode should be approximately equal to: (a) the damping for pure rigid-body swaying of this structure, and (b) the contribution of the swaying spring to the total energy stored in the mode (11). The damping for pure swaying may be estimated from the theory for an elastic half-space (1, 5, 10); in this case it is greater than 50%. The energy contribution of the swaying spring to the 2nd mode calculates to be 80% in this case. Hence, the general theory for soil-structure interaction says that 50% is the correct order of magnitude for this mode. (This theory also says that foundation interaction will have little effect upon the damping for the 1st and 3rd modes.)

If the first assumption were correct, there should have been a resonance at about 5 cps to 6 cps, and the resonance would have been sharp enough that it certainly would have been observed. However, the resonance was not observed. Hence, apparently there was damping large enough to suppress it, in accordance with the second assumption.

SUMMARY AND CONCLUSIONS

The results of the study of soil-structure interaction at the Chimes Tower may be summarized as follows:

1. There were measurements of foundation movement, from which a rocking spring constant could be determined directly without having to use the theory of foundation interaction. The resulting spring constant was in reasonable agreement with what would be expected for the size of foundation and type of soil, although no direct measurements of soil-stiffness were available for the sake of comparison.
2. Using this rocking spring constant in a dynamic model for the structure improved the agreement between predicted and observed resonant frequencies.
3. When a horizontal spring was also introduced, having a constant consistent with direct measurements of foundation movement, the dynamic model for the structure introduced a new mode that was not observed.
4. Therefore, the damping associated with the horizontal spring must have been large enough to suppress the additional mode.

Thus, in an indirect way, the test results for the Chimes Tower confirm that large damping is associated with horizontal translation interaction. The exact value of this damping cannot be deduced from the test data; it can only be concluded that the damping was quite large in general agreement with the theory for foundation interaction.

From the practical standpoint, these results mean that two choices are open when modeling soil-structure interaction:

1. Omit the swaying spring, and use only a rocking spring.

2. Include both swaying and rocking springs, but assign large damping to the swaying spring and use weighted modal damping to determine the proper damping for the various modes.

The first choice is suitable for a tall slender building, while the second choice may be necessary for heavy low buildings. Use of a swaying spring without large damping can lead to unnecessarily conservative predictions of dynamic response (6, 9).

Finally, two observations can be made concerning the determination of swaying damping from tests on actual structures:

1. The conclusions concerning the response of the foundation of the Chimes Tower could be reached only because of the unusually complete test program and because of the simplicity of the structure itself.
2. If the theory for swaying damping is correct, swaying of a foundation seldom will be important. Thus, the correctness of the theory can only be proved indirectly.

It is little wonder that there is little or no direct evidence concerning swaying damping.

ACKNOWLEDGEMENT

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APPENDIX I.--NOTATION

G = shear modulus

h_f = height of center of gravity (CG) of foundation above base of foundation

h_i = height of ith floor above base of foundation

I_f = moment of inertia of foundation about horizontal axis through its base

I_{ef} = effective moment of inertia of non-rigid foundation

k_H = horizontal translational (swaying) spring constant

k_R = rotational (rocking) foundation spring constant

M_f = dynamic rocking moment on base of foundation

m_f = mass of foundation

m_i = mass of ith floor

R = radius of foundation

V_f = dynamic shear on base of foundation

x_f = horizontal translation (swaying) of foundation

$\ddot{x}_f$ = horizontal acceleration of foundation

$\ddot{x}_i$ = horizontal acceleration of ith floor

ϕ_f = rotation (rocking) of foundation

$\ddot{\phi}_f$ = rotational acceleration of foundation

u = Poisson's ratio

β = damping ratio

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TABLE 1

OBSERVED RESONANT FREQUENCIES

(1) Tests	(2) Direction of Excitation	(3) Status of Foundation	(4) Observed resonant frequencies: First	(5) Observed resonant frequencies: Second
1-3	Strong	Unexcavated	2.51-2.54	8.50-8.55
4-7	Strong	Excavated	2.39-2.45	---
7-11*	Strong	Excavated	1.95-2.40	---
12-16	Weak	Excavated	2.32-2.44	7.86
17*	Weak	Excavated	2.17	---

All frequencies in cps (Hz). *Denotes welds broken.

TABLE 2

PREDICTED AND OBSERVED FREQUENCIES

	Calculated		
	Fixed base		With interaction
Observed	Nielsen(2.3)	Writers	Writers
2.4-2.5	2.8	2.76	2.41-2.44
			4.73-6.01
7.9-8.5	9.2	8.95	8.76-8.97

All frequencies in cps (Hz).

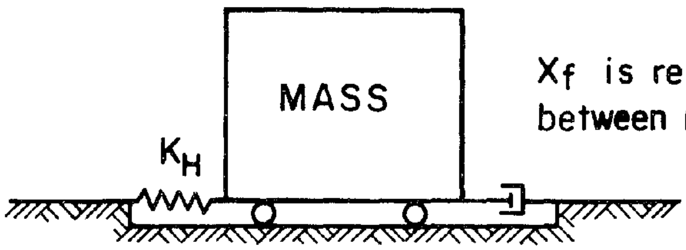

FIGURE 1: MODEL FOR FOUNDATION SWAYING

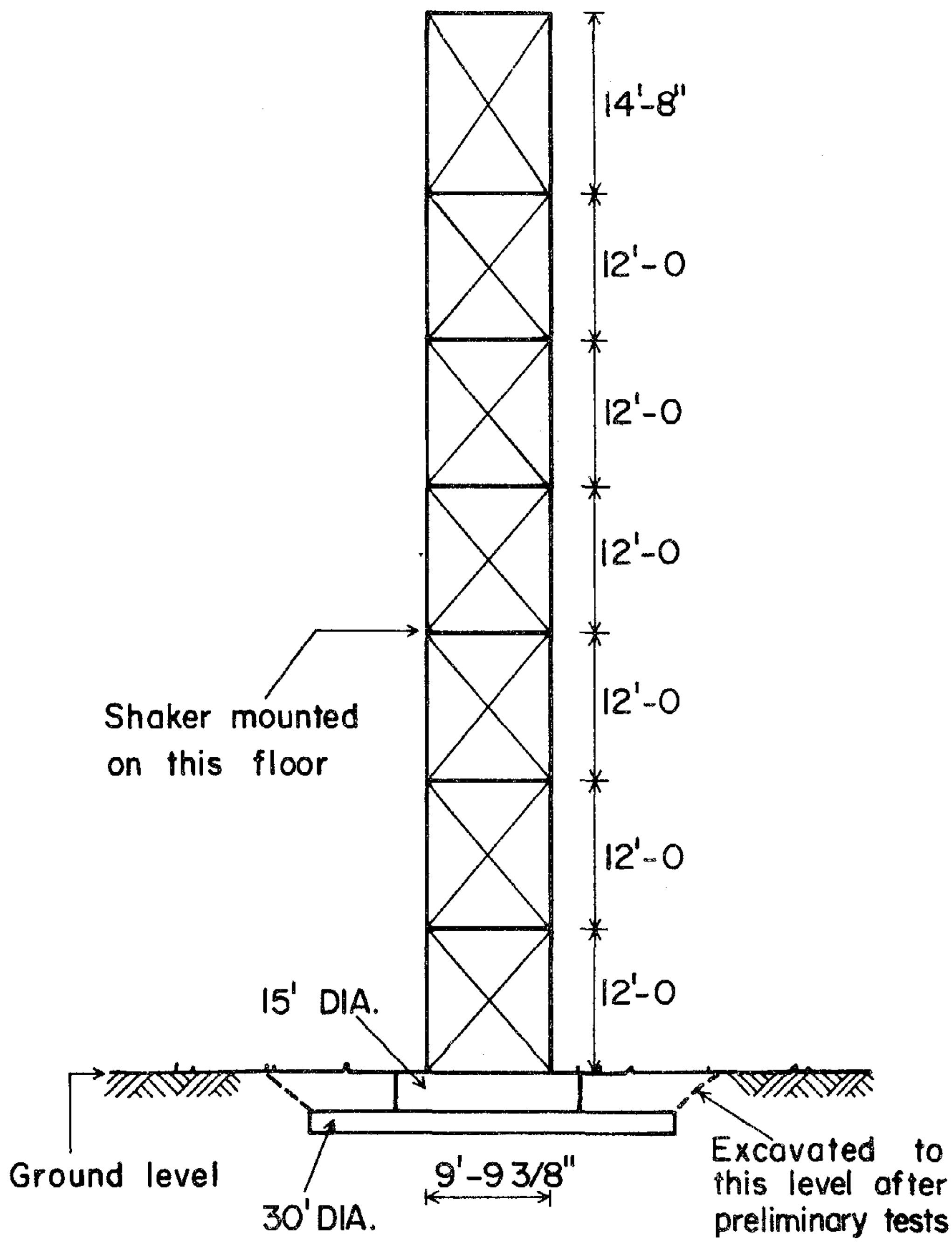

FIGURE 2 : ELEVATION OF CHIMES TOWER

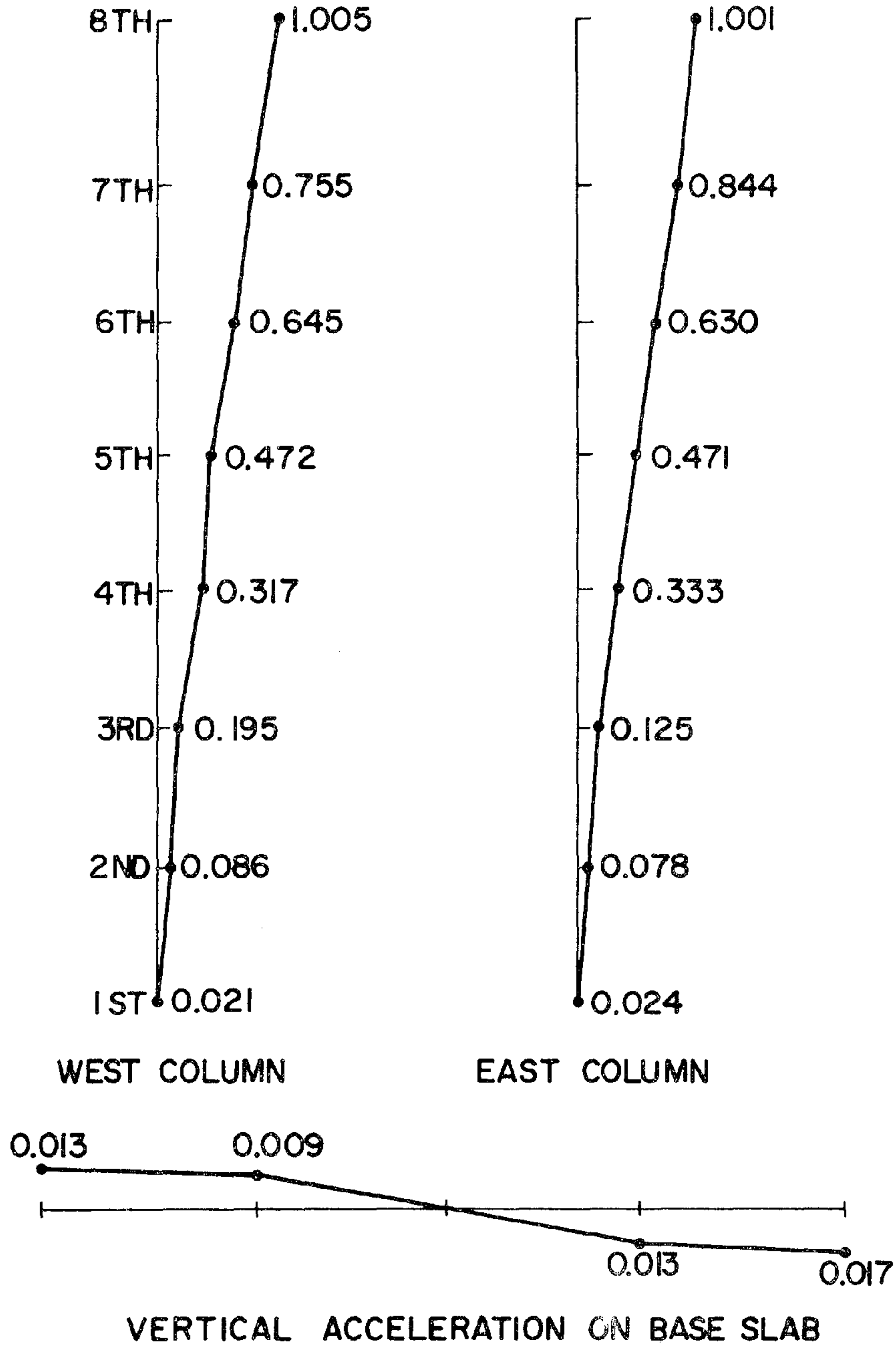

FIGURE 4: DOUBLE AMPLITUDE ACCELERATIONS IN TEST 6 AT FUNDAMENTAL FREQUENCY, IN G'S

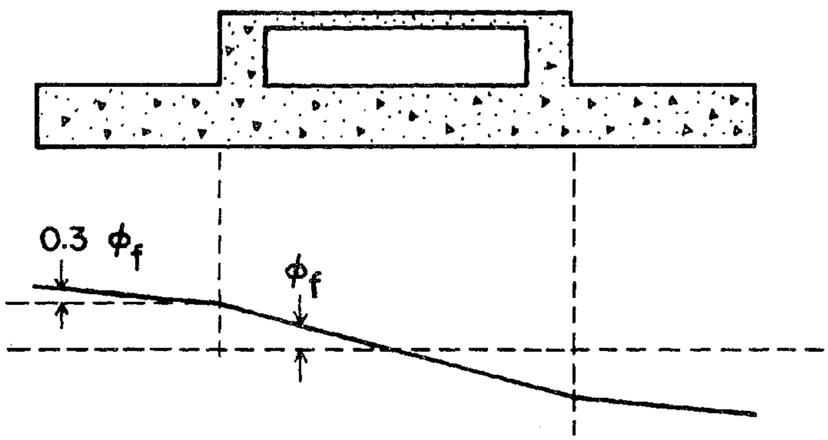

FIGURE 5: DEFLECTED SHAPE OF FOUNDATION