2

Earthquake Resistant Design—Basic Principles

Whether we are designing for the structure of the building or the nonstructural components located in the building, the principles of earthquake resistant design are the same. Most existing codes involved with regulating nonstructural component design require static equivalents that derive from the dynamic principles discussed below. We discuss the relevance of this practice at a later time.

This Chapter is similar to the preceding familiarization chapter on earthquakes. It is written for those who are not familiar with the dynamic principles of earthquakes, their effect on buildings, and the nonstructural components within them. The reader who does have a basic understanding of this concept may wish to skip ahead to the next chapter. Further readings may also be found in books such as Alfred M. Kemper's *Architectural Handbook*, 1979.

To illustrate the difference between what is called a static load and a dynamic load, we can draw a parallel to a carpenter driving a spike with a 2 pound sledge-hammer as shown in Figure 2.1. Just resting the hammer on the spike (the static condition) will never get it driven into the piece of wood, while the dynamic pounding of the sledge will drive the spike fairly quickly. This example can be extended to building components during an earthquake because buildings and their components behave dynamically rather than statically.

As an example, test and monitoring equipment in communication centers is commonly mounted on portable carts as shown in Figure 2.2. During normal service, the cart remains stationary on the floor through the action of the earth's gravity. The dynamic effects of the earthquake cause the floor to vibrate, which puts the building components into motion. If the cart has wheel locks and they are not set, the cart can be expected to shift positions on the floor. Uncontrolled rolling can easily result in this piece of test equipment colliding with some of the exposed relay panels, and so on. This unnecessary collision can result in extensive damage to the communication center. Setting the wheel locks, however, does not assure seismic integrity of this equipment. It only keeps the equipment from rolling in an otherwise

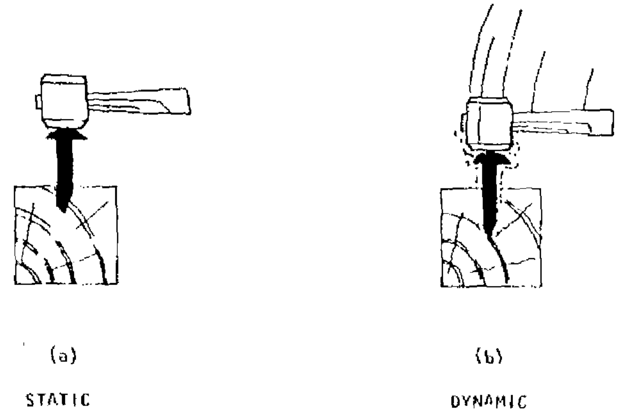

FIGURE 2.1. Static versus dynamic loading.

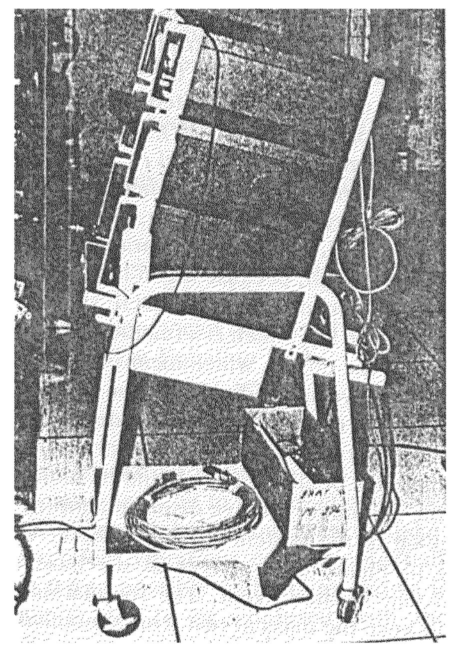

FIGURE 2.2. Communications test equipment.

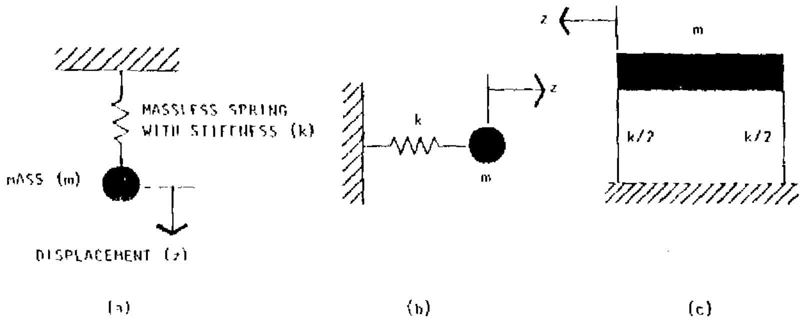

FIGURE 2.3. Typical SDOF system dynamic models without damping. (*a*) Vertical configuration (*b*) Horizontal configuration (*c*) With two supports.

moderate seismic environment. As the intensity of shaking increases, the portable equipment tends to wobble when the wheels are locked. If the earthquake is strong enough, the cart may skid or possibly overturn. This toppling can also damage critical equipment within the facility or the test equipment itself or may injure people in the vicinity. To adequately protect equipment of this type, architectural details should be provided for anchoring it in a fixed position when it is not in use.

The following is a discussion describing the mathematical relationships experienced by building equipment in the earthquake environment. For the sake of brevity, the discussion is limited to an undamped freely vibrating single degree of freedom system (SDOF), such as the typical dynamic model examples shown in Figure 2.3 (*a*, *b*, and *c*).

Damping is an inherent method of energy absorption through particle friction, wobbling connections, and so on, that would only serve to complicate our simplified model. The damping parameter must be considered in the final seismic design though, because it can have a great effect on the degree of response of an equipment item. The SDOF system is defined to have only one direction of motion (z), which is a function of time. The dynamic models use the mass (m) of the object, which means that in the real problem, where the weight is given, we must divide by the gravity constant (386 inches/second²). The mass in our example is considered to be infinitely stiff, while the column is considered to be a massless spring (k) that is a function of its length (l), its modulus of elasticity (E), and its moment of inertia (I). Figure 2.4(*a*) shows the maximum relative displacement (z) with respect to the rest position. Rocking of the system is ignored here. We know that the earthquake energy in Figure 2.4*b* arrives at the base of our idealized model from some distant seismic event. The foundation material shakes, while inertia tends to keep the mass at rest. Our simplified model shows the foundation at rest while the mass does all the moving. A detailed examination of the dynamic properties of the system would show that it does not matter if

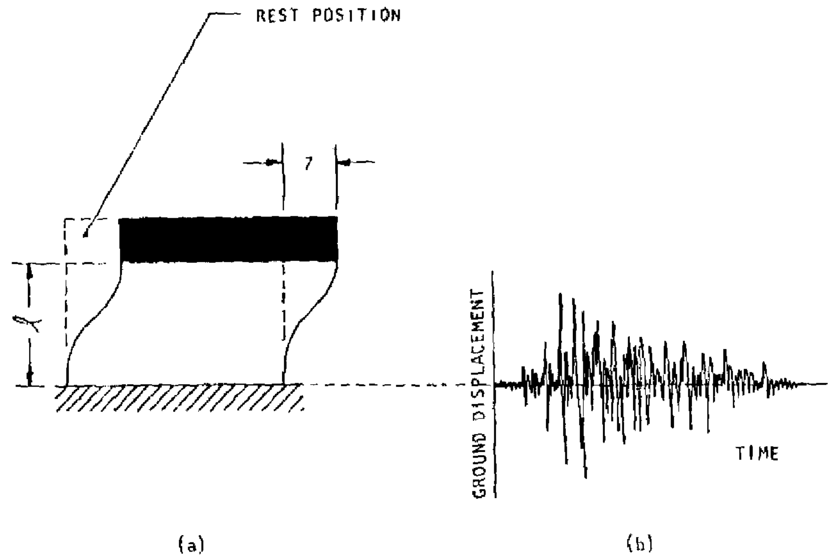

FIGURE 2.4. Maximum displacement. (*a*) SDOF model (*b*) Ground displacement

we assume either the ground or the mass to vibrate, since it is the relative displacement between the mass and the ground that is significant. The assumption that the mass moves only tends to simplify the problem. Technical proofs of this assumption are available in many texts that deal with the dynamic properties of earthquakes.

We can begin to develop the required mathematical relationships by assuming that the ends of the idealized columns (which may represent a shelving unit or a communication relay panel) of Figure 2.4*a* are fixed at both top and bottom. The column shear (V) can be obtained by referring to Figure 2.5 and Equation 2.1.

$$V = \left(\frac{12EI}{l^3}\right)z \tag{2.1}$$

The free body diagram of Figure 2.6 shows the forces applied to the mass of Figure 2.4*a*. The elastic restoring force (kz) tries to overcome the applied force $[F(t)]$ and restore the mass to its original position. If we assume that a unit displacement occurs, then Equation 2.1 becomes the unitless quantity of Equation 2.2

$$V = \left(\frac{12EI}{l^3}\right)1 = k \tag{2.2}$$

which is the definition of the system stiffness (k). Formulas for arriving at k for other models can be found in elementary dynamics texts. Where more

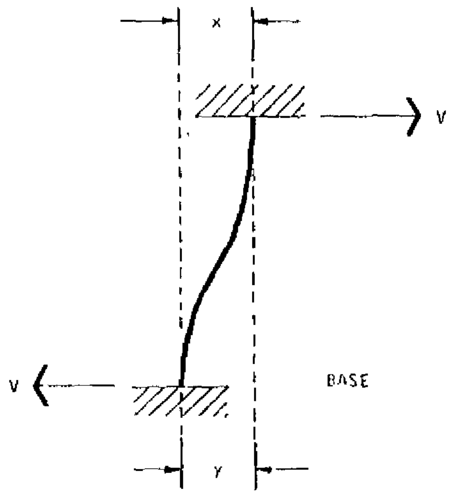

FIGURE 2.5. Shearing in fixed end column

than one column is considered as in Figure 2.3c, the column stiffnesses are summed

Newton's law states that the sum of the applied force in Figure 2.6 is equal to the mass times the acceleration, or

$$\Sigma F = ma \tag{2.3}$$

where the acceleration (a) is in units of distance per second per second. The dot ($\cdot$) format is used to represent all time derivatives. With it, we can define the following relationships:

$x(t)$ = the absolute displacement of the mass (distance)

$\dot{x}(t) = \frac{dx(t)}{dt}$ = the absolute velocity of the mass (distance/second)

$\ddot{x}(t) = \frac{d^2x(t)}{dt^2}$ = the absolute acceleration of the mass (distance/second^2)

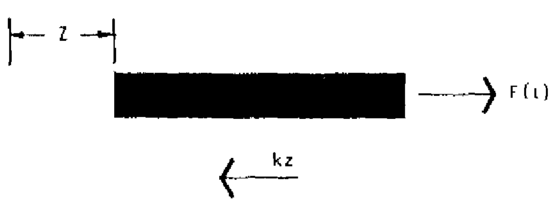

FIGURE 2.6. Free body diagram.

Figure 2.5 shows the variables x to be the absolute displacement of the mass and y to be the absolute displacement of the ground. Figure 2.4a shows the variable z to be the maximum relative displacement or

$$z = x - y \tag{2.4}$$

Newton's law can be applied to the mass in Figure 2.4a. The dynamic equation of motion for an undamped SDOF system is expressed in Equation 2.5.

$$F(t) - k(x - y) = m\ddot{x} \tag{2.5}$$

By subtracting $m\ddot{y}$ from both sides of Equation 2.5, it becomes $F(t) - m\ddot{y} = m\ddot{z} = kz$. For free vibration, $F(t)$ = equals 0, and y equals 0. If we next divide both sides of Equation 2.5 by the mass m we have:

$$\ddot{z}(t) = \frac{k}{m}z(t) = 0 \tag{2.6}$$

We can now define the undamped natural frequency (ω_n) of our SDOF system in radians/second as represented in Equation 2.7.

$$\omega_n = \sqrt{\frac{k}{m}}\ \text{radians/second,} \tag{2.7}$$

Next we define the undamped natural frequency (f_n) in cycles/second, or more conventionally, hertz, as shown in Equation 2.8:

$$f_n = \frac{\omega_n}{2\pi}(\text{cycles/second = hertz}) \tag{2.8}$$

Finally, we can define the undamped natural period (T_n) in seconds as shown in Equation 2.9:

$$T_n = \frac{2\pi}{\omega_n} = \frac{1}{f_n}(\text{seconds}) \tag{2.9}$$

The undamped natural frequency can be arrived at directly from the weight (W) of the system as shown in Equation 2.10:

$$f_n = \sqrt{\frac{9.8k}{W}} \tag{2.10}$$

This saves in the conversion from weight to mass.

These relationships are important to the seismic qualification of the building equipment. We have learned that the initial earthquake generates random wave patterns. The random vibrations contain potentially destructive frequencies from about 0.5 to possibly 40 hertz as shown in Figure 2.7. This mechanical spectrum is not to be confused with the electromagnetic spectrum (light, radio waves, etc.), which is also measured in hertz. As a general rule, the lower frequencies in the range of 0.5 to 10 hertz (with higher

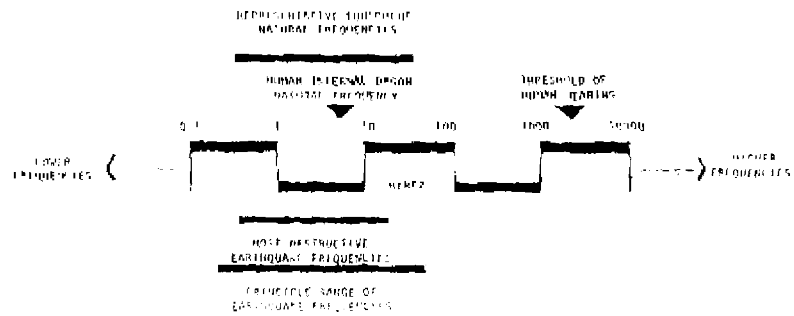

FIGURE 2.7. Representative frequencies

amplitudes associated) cause more problems to building equipment. Every material through which the earthquake wave travels affects the wave. This is true of the various geologic frameworks, as well as the building structure and even the equipment. The effects tend to attenuate some of the system response to the frequency content and to amplify others. By use of terms that are discussed shortly, such as ground response spectra and floor response spectra, it is possible to predict with some degree of confidence what the seismic frequency distribution at each equipment location within the structure will be. A knowledge of the expected seismic frequency distribution will tell us what associated equipment natural periods are critical. When seismic frequency distribution overlays equipment natural period, extreme equipment oscillations usually occur. This is similar to the slight push required at the correct timing intervals for a child on a swing to attain the highest arc. The more motion the equipment experiences, the more likely that the system will have a failure. The failure can result from internal component malfunctions, from overturning of the equipment, from hammering against a wall or from other pieces of equipment colliding with it.

The concept of the response spectrum has been extremely valuable to designers of earthquake resistant materials. It is developed as in Figure 2.8 from a plot of the maximum response to a SDOF system for a particular earthquake with variable frequency and variable damping.

The response spectrum is applicable to the free field environment (ground response spectrum), the building (structure response spectrum), and the equipment within the building (equipment response spectrum). The particular format most often chosen to represent the response spectrum is a four coordinate logarithmic plot. By referring to Figure 2.9, we can see these coordinates. They are the frequency (the multiplicative inverse of the period), the spectral displacement (S_d), the spectral velocity (S_v), and the spectral acceleration (S_a). The smoothed curves in Figure 2.9 are for $\frac{1}{2}$ and 5 percent critical damping. One hundred percent critical damping is the

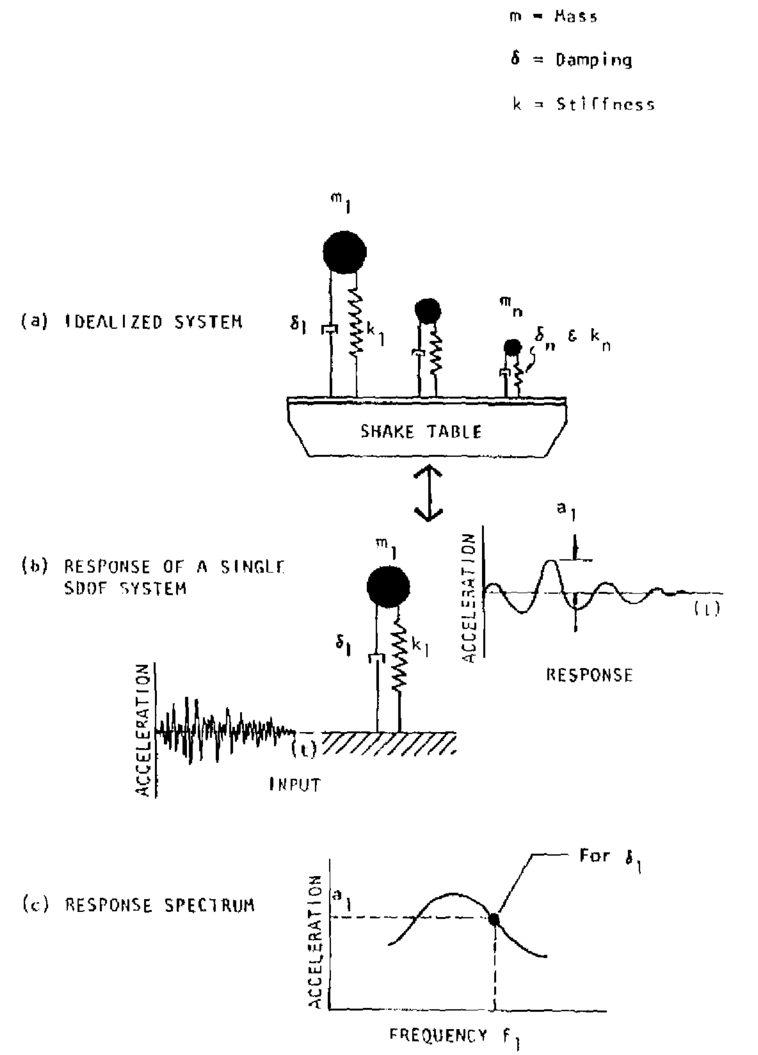

FIGURE 2.8. Development of response spectrum. (*a*) Idealized system. (*b*) Response of a single SDOF system. (*c*) Response spectrum

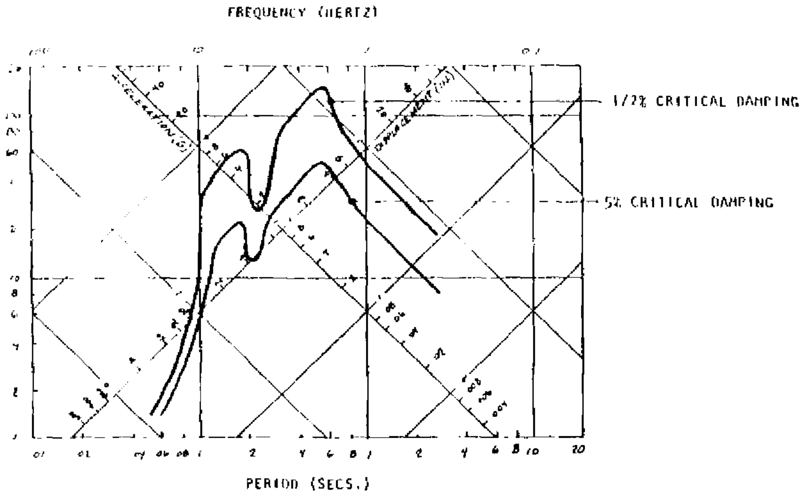

FIGURE 2.9. Example of smoothed floor response spectrum.

amount of energy dissipation required to bring a freely vibrating system to a halt after one full oscillation.

The response spectrum has definite limitations. It can tell us the maximum response to a SDOF array, the peak amplitude of input, and the bandwidth of the input. It cannot tell us the duration of the earthquake (that must come from the strong motion recording), the type of wave of the input (in natural earthquakes, we can assume random), how a particular piece of equipment will respond or if a particular piece of equipment will continue to operate during and after the earthquake. The use of the response spectrum is only as good as the assumptions made in the initial math model.

The spectral velocity (S_v) depends on the particular characteristics of the earthquake under consideration, the natural frequency of the system, and the damping characteristics. The spectral displacement (S_d) and the spectral acceleration (S_a) are associated with the spectral velocity as shown in Equations 2.11 and 2.12. Damping has again been ignored for the sake of simplicity.

$$S_d = \left(\frac{T_n}{2\pi}\right) S_v \qquad (2.11)$$

$$S_a = \left(\frac{2\pi}{T_n}\right) S_v \qquad (2.12)$$

The maximum base shear (V_b) of a piece of equipment can be obtained from the relationship shown in Equation 2.13. The spectral velocity must be

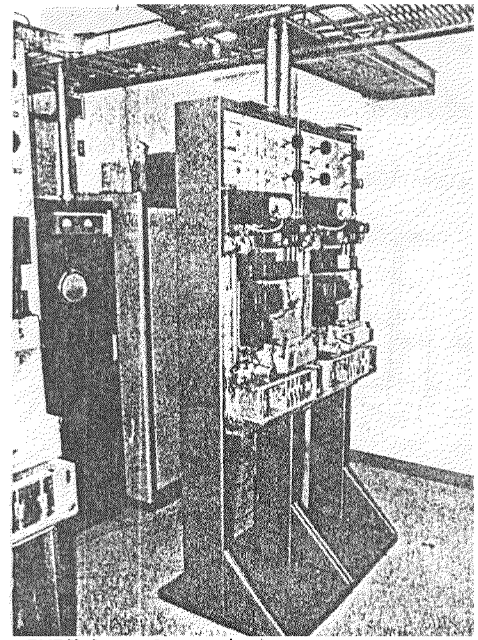

FIGURE 2.10. Communications rack.

derived from a specific design earthquake response spectrum such as Figure 2.9. The maximum base

$$V_b = kS_d = k\left(\frac{T_n}{2\pi}\right)S_v \qquad (2.13)$$

shears can also be obtained from the 1979 *Uniform Building Code* relationship shown in Equation 2.14:

$$V_p = ZIC_pW_p \qquad (2.14)$$

where Z = seismic zone coefficient
I = building importance factor
C_p = equipment horizontal force coefficient
W_p = weight of the equipment

To compare these two methods, we take as a simple example a communication relay panel located in an emergency operation center. The piece of equipment shown in Figure 2.10 is similar to that which we are considering.

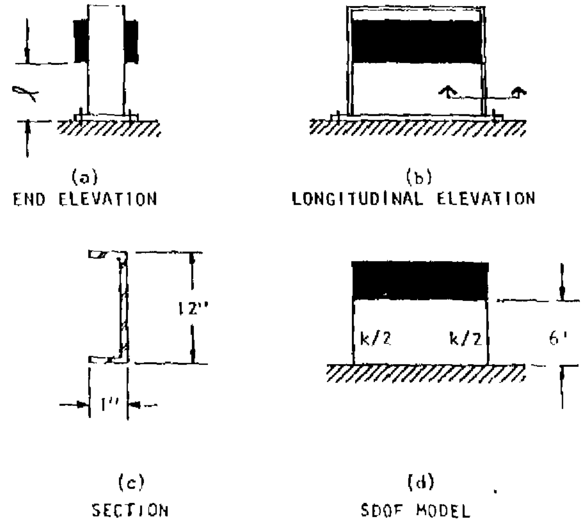

FIGURE 2.11. Example equipment. (*a*) End elevation (*b*) Longitudinal elevation. (*c*) Section (*d*) SDOF model

The actual configuration of the equipment in our example is shown in Figure 2.11 (*a*–*d*).

The following information is given with respect to Figure 2.11:

Z = 1.0 (seismic zone 4)
I = 1.5 (essential facility: emergency operating center)
C_p = 0.3 (equipment required for essential facility operation)
W_p = 1200 pounds

Calculate for both 0 and 5 percent critical damping in the longitudinal direction only. Calculating the *Uniform Building Code* value first, we have

$$V_b = ZIC_pW_p = (1) \times (1.5) \times (0.3) \times (1200)$$

$$V_b = 540 \text{ pounds}$$

The *Uniform Building Code* method does not take into account the dynamic properties of the system. To do this, we use the response spectrum approach and make a simple dynamic analysis for our idealized SDOF equipment system.

Determining the stiffness for the columns, the natural frequency, and natural period of the system

$$k = \frac{12EI^*}{l^3} = \frac{(12)(29 \times 10^6)(0.015)}{(6 \times 12)^3} = \frac{5.2 \times 10^6}{72^3} = 139$$

*Not *UBC I*

$$\omega_n = \sqrt{\frac{k_1 + k_2}{m}} = \sqrt{\frac{(139 + 139)\,386}{1200}} = 9.48 \text{ radians/second}$$

$$f_n = \frac{\omega_n}{2\pi} = 1.5 \text{ hertz}$$

$$T_n = \frac{2\pi}{\omega_n} = 0.66 \text{ seconds}$$

Referring to Figure 2.9, we find that the spectral velocity is approximately 90 inches/second for ½ percent critical damping and 38 inches/second for 5 percent critical damping. The base shear Equation 2.13 is then:

$$V_b = k\left(\frac{T_n}{2\pi} S_v\right)_{1/2\%} = 278\left(\frac{0.66}{2\pi} \times 90\right) = 2630 \text{ pounds}$$

V_b = 2630 pounds for ½ percent critical damping

V_b = 1110 pounds for 5 percent critical damping (calculations omitted)

Thus we can see that the *Uniform Building Code* base shear value is significantly less than that of the response spectrum solution. Neither of these methods, however, can confirm that equipment will remain operational during and after an earthquake. To assure equipment operability for most items during and after an earthquake, it must be subjected to a dynamic testing procedure (earthquake simulations).

Above we discuss only single degree of freedom systems. Many of the building components encountered by the design professional are more complex in their dynamic response to an earthquake. These complex systems are commonly termed multiple degree of freedom (MDOF) systems. Figure 2.12 shows an example of a typical MDOF system that needs to be given seismic consideration. This type of equipment item is commonly required for the effective operation of facilities after an earthquake.

There is more than one method to determine the associated forces and response characteristics of the MDOF system. We discussed the response

FIGURE 2.12. Multiple degree of freedom example.

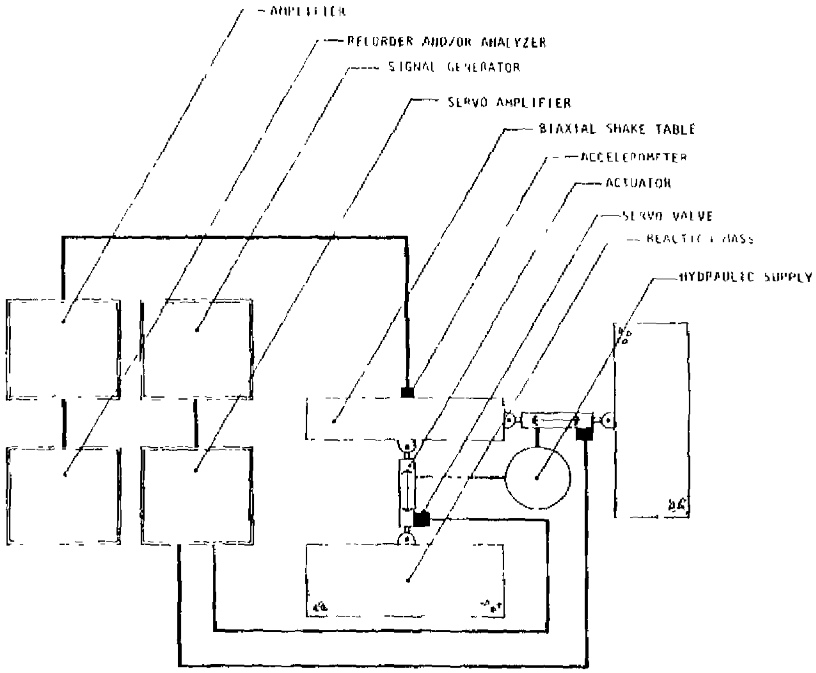

FIGURE 2.13 Diagrammatic biaxial seismic simulator

spectrum method for the SDOF system earlier. The principal difference in calculating for the MDOF as opposed to the SDOF is the number of natural frequencies (a dynamic system having the same number of natural frequencies and associated modes of vibration as there are degrees of freedom) and the physical method of analyzing the math model. The response spectrum method of analysis usually requires the least expenditure of engineering and computer time.

The other principal method of analysis is called the time-history method. This method is complicated requires extensive computer application, and may be beyond the scope of the architectural profession. This, however, does not preclude the architect from being aware that this method exists. The time-history method yields a more exact approximation of the expected behavior of the MDOF system. Where the response spectrum method yields results that are usually conservative, the time-history method of analysis yields results that take into account each increment of time for the seismic event, thus providing results that are generally more indicative of the true system response.

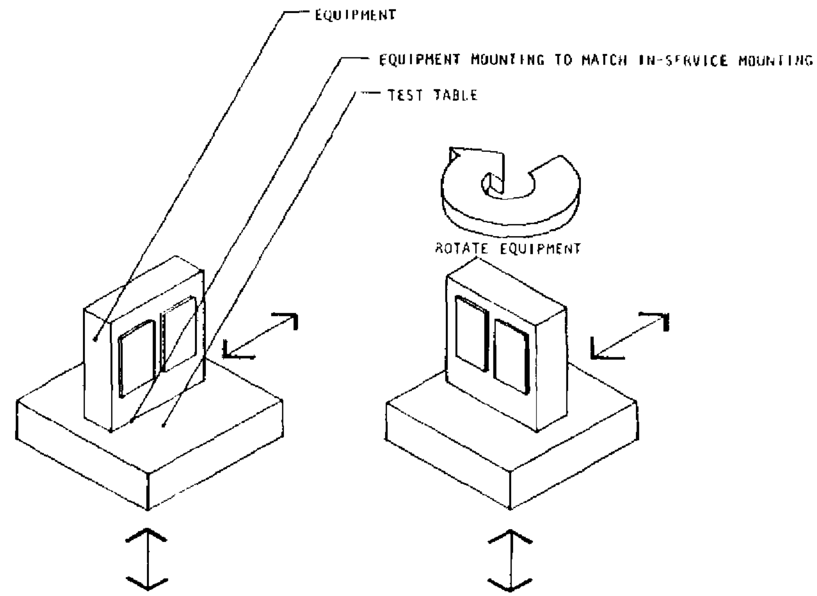

FIGURE 2.14 Equipment rotation on the seismic test table.

There are specific pieces of equipment that must remain functional after an earthquake for the safe and efficient operation of an essential facility. A seismic analysis cannot always guarantee that this equipment will operate. When this is the case, an earthquake test on a seismic simulator is required. Because of equipment limitations, it is sometimes necessary to combine the test procedure with an analysis. Figure 2 13 is a schematic drawing of a seismic test machine.

The nonstructural component under consideration is mounted on the test table in a fashion similar to that of the actual mounting in the essential facility. During and after the earthquake test, the equipment is subjected to functional tests. This is principally the only method of assuring that the equipment will be able to function after an earthquake.

A number of options are available with the earthquake test First is the choice of the number of axes to be tested Current technology limits seismic testing to only one or two axes of excitation. Biaxial simulators usually have one horizontal component and the vertical component operating simultaneously. To test the two equipment plan axes (transverse and longitudinal), the test specimen is rotated between tests as in Figure 2.14. The next major decision with respect to seismic testing is the type of waveform to be used.

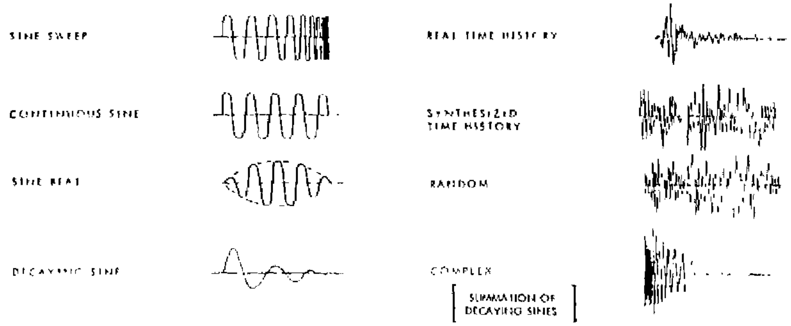

FIGURE 2.15. Waveforms available for testing.

Figure 2.15 shows the eight basic waveforms that are available for testing purposes. Each of these waveforms has its own particular use when applied to the test specimen. For instance, the sine sweep is used to determine the specimen's natural frequency. The sine dwell test can be used to determine failure characteristics of the specimen. The time histories and random waveforms are used most often for analysis verification and proof testing. Proof testing is the most valuable asset of the seismic test philosophy. This is the method by which a particular piece of critical equipment can be tested to relatively guarantee that it will be able to perform its function after an earthquake. The equipment is subjected to its required operational functions during and/or after the earthquake test to be certain that it will perform adequately.

This concludes our discussion on the basic principles associated with earthquake resistant design. The next two chapters present and discuss fairly specific details on the qualification approaches and procedures for many types of equipment found in modern buildings.