EARTHQUAKE DESIGN LOADS

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Figures

1. GENERAL

1.01 This section discusses and provides standards for earthquake design loads that will be imposed on communication facilities located in seismic areas. These standards are provided for use in the design of new buildings or building additions that are intended to house telephone equipment that meets the requirements of Section 800-610-164, New Equipment-Building System (NEBS), General Equipment Requirements.

1.02 This section supersedes Section 5.3 of Specification X-74300, NEBS Building Engineering Standards (BES). Whenever this section is reissued, the reason for reissue will be listed in this paragraph.

2. EARTHQUAKE

2.01 Telephone facilities located in active seismic areas may be exposed to an environment of severe earthquake-induced vibration. To ensure their ability to survive and adequately perform vital services and also to protect plant investment, it is important to incorporate earthquake-resistant design into the telephone facilities located in active seismic areas. This section provides such relevant information as estimates of earthquake ground motion, induced loads for building design, cost aspects for providing earthquake protection, and some general seismic design considerations for both buildings and building equipment. Equipment considerations are further discussed in Sections 760-200-040 and 760-200-041.

A. *Geographic Regions of Strong Motion Earthquakes and Frequency of Occurrence*

2.02 The zones of the continental United States that have had major, moderate, minor, or no earthquakes are shown in a map prepared by the National Oceanic and Atmospheric Administration (NOAA). This map is included in the Uniform Building Code (UBC), while similar maps appear in the BOCA Basic Building Code and the National Building Code. The code map presents a qualitative seismic-risk guideline, but presents no information as to frequency of earthquake occurrence. Such information should not be overlooked, because the knowledge of how frequently earthquakes occur in a certain area is as important as the magnitude of the shaking.

2.03 The maps in Fig. 1 and 2 have been developed through state-of-the-art microzonation techniques and include frequency-of-occurrence information. These maps represent the best current estimate of earthquake zoning for communications facilities located in the United States. Expected peak ground acceleration levels within each of the four zones are shown in Table A. Statistically, these levels are based upon a 90 percent probability of not being exceeded during a 50-year service life of a communications facility.

2.04 The variation in acceleration shown in Table A for each zone represents the spread of earthquake severity across a zone. Acceleration levels for a specific location within a particular zone may be estimated by linear interpolation of the values shown in Table A.

2.05 Telephone facility designs should reflect the requirements stipulated in the building code of the area of construction. The seismic criteria outline in this specification should be used as complementary criteria and should be adhered to in those areas where local codes do not specify complete earthquake protection for either building or building equipment design.

B. Earthquake Ground Motion

2.06 Ground motions during an earthquake are transient vibrations that usually last from 10 to 60 seconds or more. A typical strong-motion earthquake accelerogram generally consists of 2 to 5 seconds initial buildup, 8 to 10 seconds of strong shaking, and a gradual decay that lasts from 18 to 45 seconds as shown in Fig. 3(b). The dominant frequency of typical strong-motion earthquake accelerograms generally ranges from 1 to 10 Hz though lower frequencies may be experienced at very soft soil sites and higher frequencies on hard rock sites. Structures possessing natural frequencies of vibration in this range are more responsive and susceptible to earthquake excitations than those with frequencies outside this range.

2.07 The maximum ground accelerations generally decrease with epicentral distance; that is, the distance between the building site and earthquake source as indicated by the epicenter or fault. Table B lists maximum ground accelerations in the vicinity of the fault for earthquakes of various magnitudes. The associated durations of the strong phases of ground motion are also given. At greater distances from the fault the total duration of shaking is longer and the intensity of shaking is less. The attenuation of the intensity of shaking with distance from the epicenter depends a great deal on the geology, the source mechanism, the magnitude, etc. In general, the acceleration amplitude decays exponentially with the focal distance. For example, for an earthquake of Richter magnitude 8, the peak ground acceleration can decrease from about 0.8g to approximately 0.2g 60 miles away and 0.1g 80 miles away.

2.08 The ground motion near the surface where a building is located is affected by the stiffness, strength, and layering properties of the

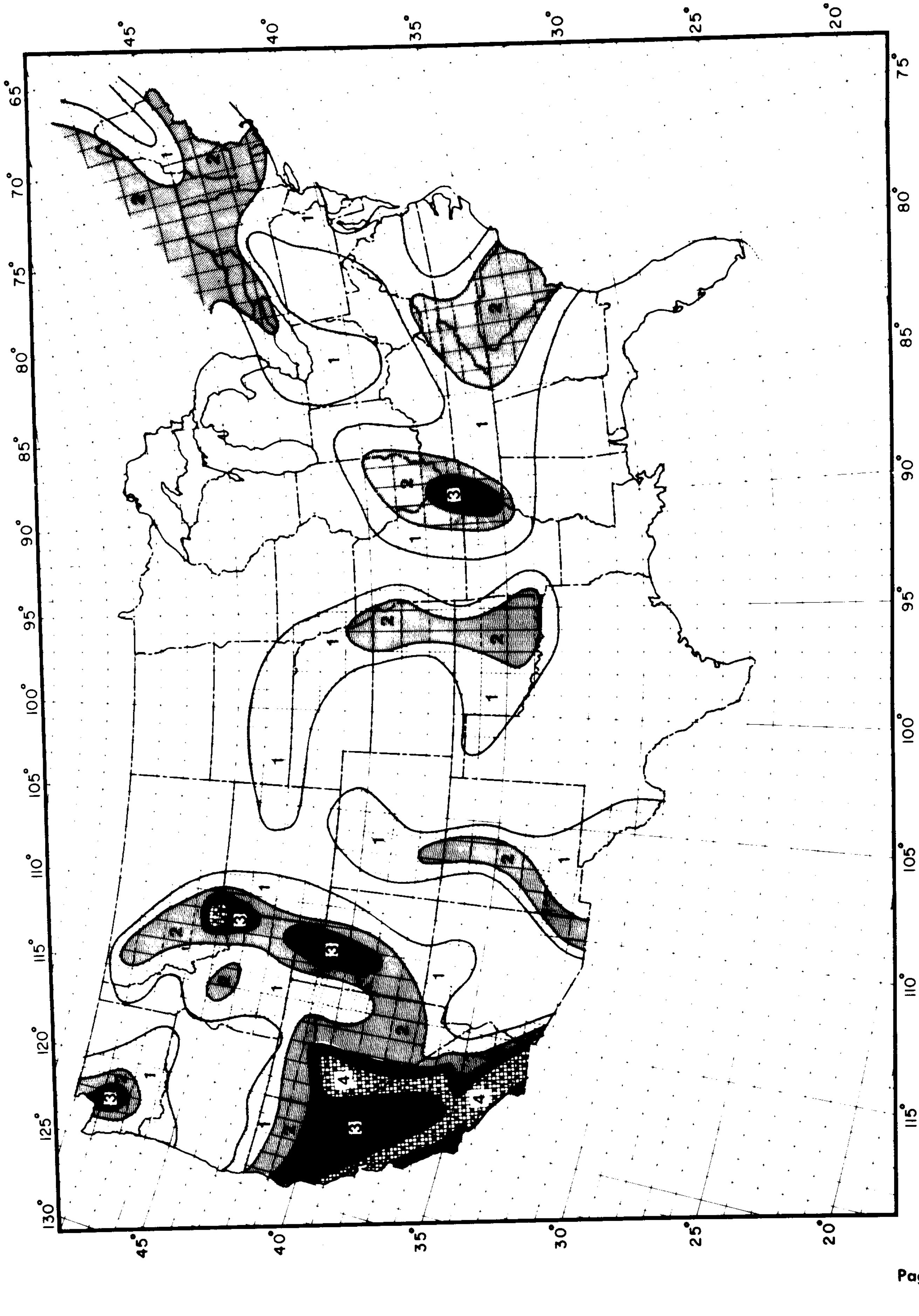


Fig. 1—Bell System Earthquake Zoning Map

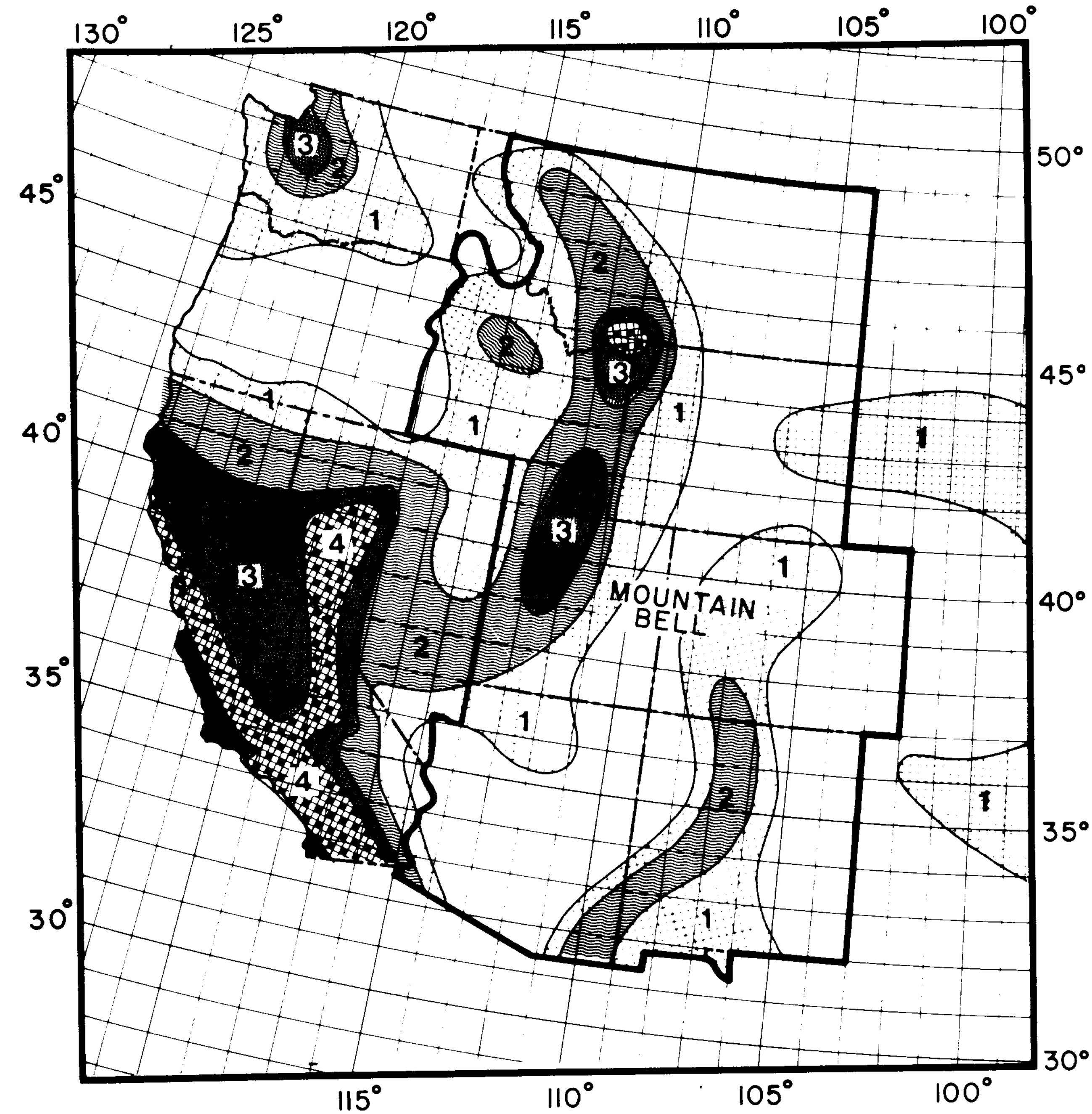


Fig. 2(a)—Western Area

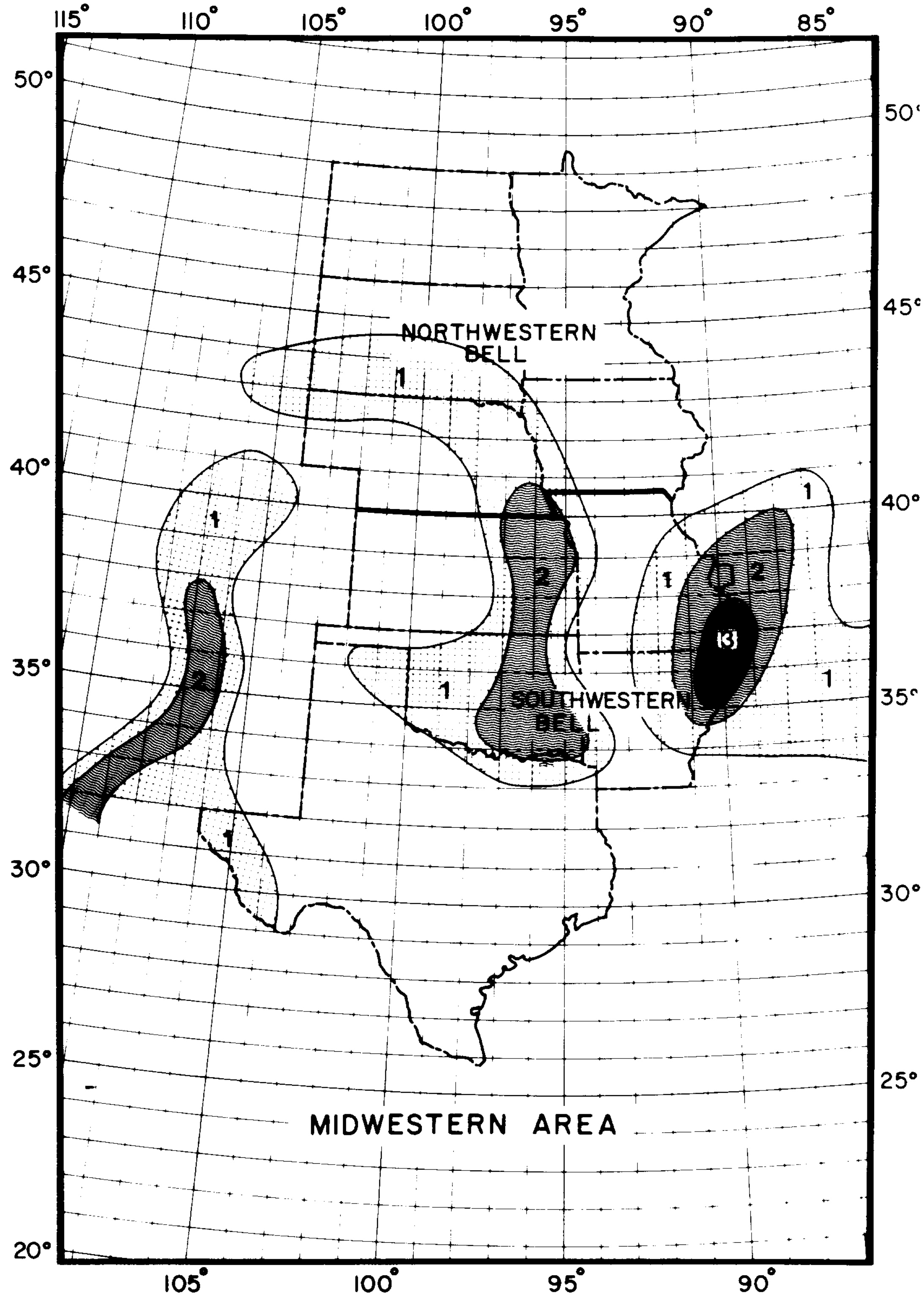


Fig. 2(b)—Midwestern Area

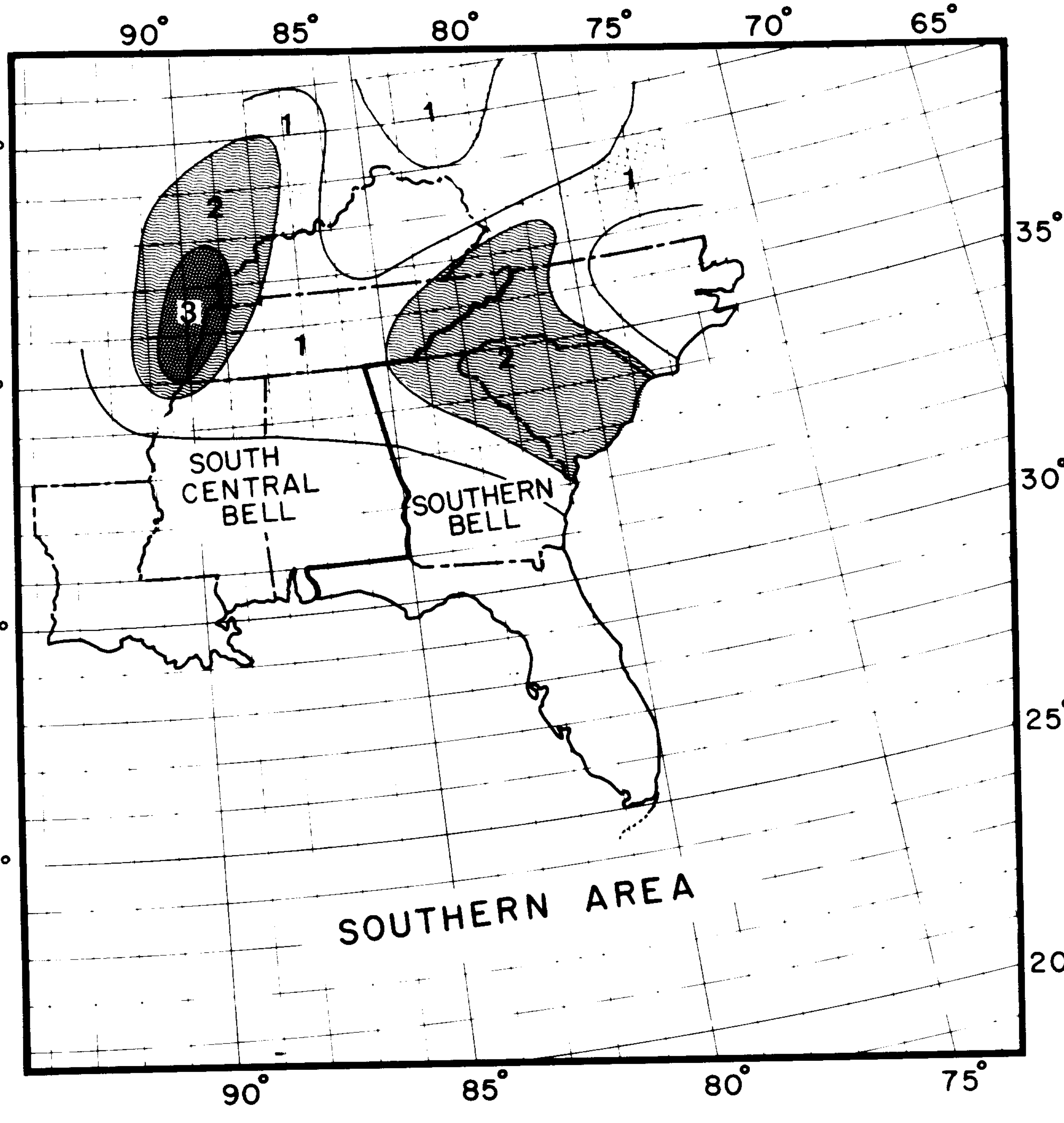


Fig. 2(c)—Southern Area

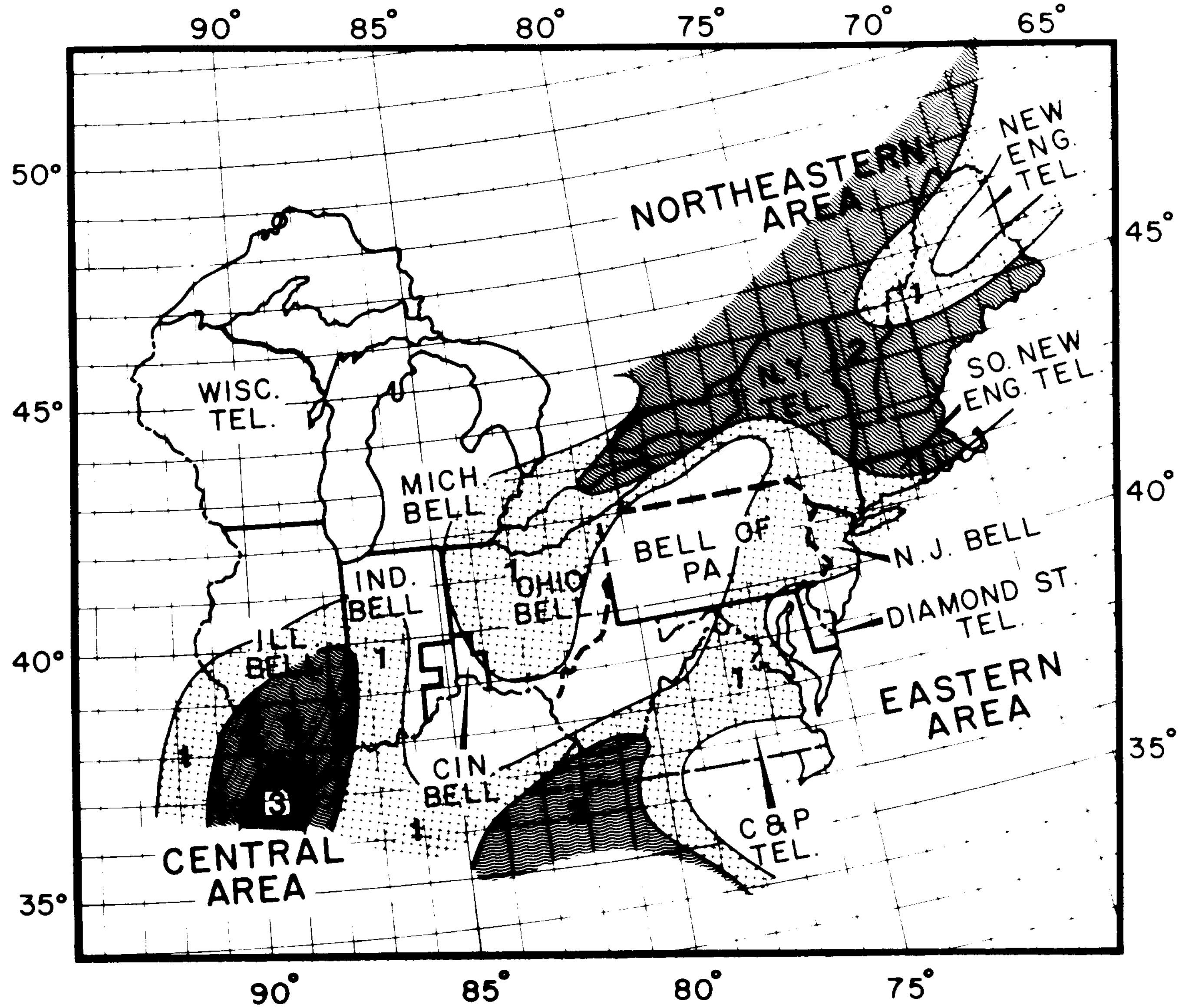


Fig. 2(d)—Northeastern Area

TABLE A

PEAK ACCELERATIONS VS ZONE FOR BELL SYSTEM ZONING MAPS (FIG. 1 AND 2)	
ZONE	PEAK GROUND ACCELERATION (g)
1	0.06 thru 0.10
2	0.11 thru 0.20
3	0.21 thru 0.40
4	0.41 thru 0.80

TABLE B

APPROXIMATE MAXIMUM GROUND ACCELERATIONS AND DURATIONS OF STRONG PHASE SHAKING NEAR (10 MILES) THE FAULT

RICHTER MAGNITUDE (M)	MAXIMUM ACCELERATION (g)	DURATION (SECONDS)
5.0	0.10	5
5.5	0.15	10
6.0	0.20	15
6.5	0.30	20
7.0	0.40	25
7.5	0.45	30
8.0	0.70	35
8.3	0.80	40

soil and rock strata between the building site and the earthquake epicenter. Deep layers of relatively compliant soils should be considered carefully in the design of buildings taller than three stories because of their high-acceleration-amplification characteristics and the tendency to induce into the building low-frequency motions that have a potential for structural and equipment damage.

2.09 For massive structures founded on compliant soils, there may be some feedback of the motion from the structure into the founding soils. This may result in energy dissipation and in lower natural frequencies than those that might occur in an equivalent building founded on rock. The energy dissipation also tends to reduce the seismic forces that are transmitted to a structure founded on compliant soil. These phenomena are generally referred to as the soil-structure interaction.

2.10 The piles of a building constructed on a pile foundation will displace laterally with the surrounding soil during an earthquake and consequently have little influence on horizontal motions of the building. However, piles may improve the ability of the building to resist the effects of ground motions by reducing both static settlement and dynamic rocking motion. According to the recent Uniform Building Code recommendation, individual pile or caisson footings of every building are to be interconnected by ties, each of which can carry by tension and compression a horizontal force equal to 10 percent of the largest pile cap loading.

2.11 Constructing buildings on saturated sands is undesirable because earthquake shaking may cause the rearrangement of the sand particles, which may lead to sinking or tilting of the building —a phenomenon known as liquefaction.

C. Earthquake Load

2.12 The magnitude of earthquake loading on buildings in both horizontal and vertical directions is a function of (1) properties of the building structure, such as rigidity, mass, energy-absorption characteristics, and geometrical configuration, and (2) characteristics of the ground motion, such as amplitude, duration, and predominant frequency. The latter are in turn dependent on Richter magnitude, local geology, and epicentral distance.

2.13 Horizontal motions cause the most damage to buildings; vertical motions, though important to consider, are of less significance. The simplest design procedure is to consider horizontal load factors such as the seismic coefficients specified in the Uniform Building Code. The seismic coefficients enable computation of the equivalent static load for which a structure must be designed. However, it is generally desirable to check the capability of multistory Central Office buildings by performing a dynamic response analysis of the building to earthquakes. Dynamic earthquake responses can be evaluated if structural designers elect either of two methods for seismic analysis of buildings: the response-spectrum method or the time-history method. The cost to perform such analyses is often negligible when compared with the cost of the building.

Response Spectrum

2.14 The response spectrum is defined as the maximum displacement, velocity, or acceleration responses of a series of simple mass-spring systems of variable fundamental frequencies subjected to a specific earthquake input. Response spectra for various degrees of structural damping and ductility can be developed and many are available that have been developed from strong earthquake motions recorded in the United States. These spectra can be used for building design, particularly at sites where subsurface conditions and selected earthquake characteristics are similar to those measured at strong-motion recording stations.

Time History

2.15 The time-history method involves the computation of the time-varying response of a building to earthquake ground motions by considering an appropriate mathematical model of the building and an actual earthquake accelerogram (time history of the earthquake motion). This method is suitable for structures that respond nonlinearly to the earthquake excitation. It also facilitates computation of time-domain response and floor response spectra, which are used to determine the seismic effects on equipment located within the building. The frequency content and duration of the earthquake-motion input can be varied to suit the particular earthquake conditions that a particular structure is designed to withstand.

2.16 For telephone buildings no more than five stories high, it is recommended that the seismic-coefficient method of design (as in the Uniform Building Code) be used. For buildings, taller than five stories, either the response-spectra [Fig. 3(a)], or the time-history method [Fig. 3(b)], may be used to verify or modify designs established through use of the seismic-coefficient method. Both Fig. 3(a) and 3(b) correspond to locations where the peak ground acceleration is 0.3g. For other locations the response spectra and ground accelerograms can be constructed by linearly scaling the ordinate of Fig. 3(a) and 3(b) by a factor, ie, peak acceleration from Table A divided by 0.3g. Response spectra for buildings on typical sites in Fig. 3(a) are derived for critical damping ratios of 2, 5, and 10 percent. The spectra are computed for elastically responding systems and, thus, are primarily applicable for systems that do not undergo inelastic deformation.

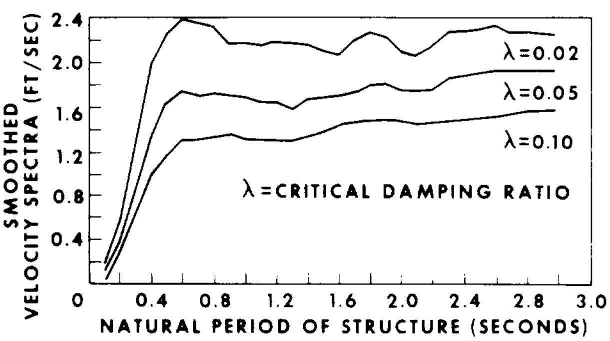


Fig. 3(a)—Smoothed Elastic Velocity Spectra of an Accelerogram Corresponding to an Earthquake With a Peak Acceleration = 0.3g

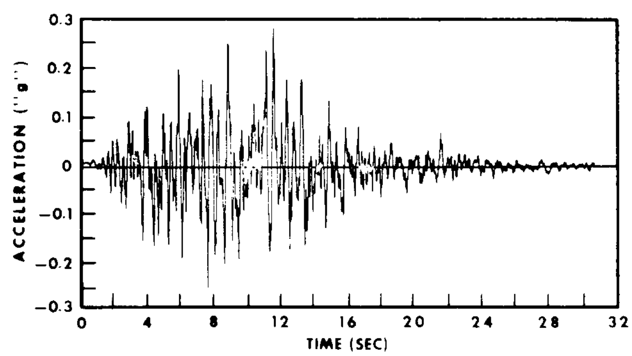


Fig. 3(b)—Synthetic Earthquake Accelerogram With a Peak Acceleration = 0.3g

2.17 Response spectra are provided in References 6, 15, 16, and 18 for structural systems that may deform inelastically and absorb significant amounts of energy through ductile action. Elastic response spectra, such as those in Fig. 3(a), may be used to compute the response of inelastic systems; however, acceleration and hence, force levels so obtained may be somewhat conservative. On the other hand, elastic-system spectra generally underestimate the displacement when used for inelastic-system response analyses.

2.18 In all cases, the equivalent static lateral loads specified in the local governing codes shall be used as the minimum requirements for earthquake-resistant design. Allowance must be made for the heavy mass of telephone equipment when determining the total building weight used in establishing the ***lateral forces*** prescribed by Code formulae. Studies show that the entire floor area of a building is never fully loaded with equipment and that a good approximation to the maximum ***average*** equipment weight is 50 percent of the design live load less 10 psf. Thus, for a floor designed for a live load of 150 psf, 65 psf is to be used as equivalent dead load in the seismic code formulations for lateral load effects. (See also Section 760-200-100.)

2.19 Floor-supported telephone equipment on upper stories of telephone buildings generally are subjected to more severe earthquake-induced vibration than is the building itself, because of amplification of the free-field ground motions that usually occur within a building. This amplification depends on many factors, such as number of stories, building frequencies, damping, the characteristics of the free-field ground motion, and soil properties. Dynamic response studies of typical telephone buildings indicate that the in-building motion for equipment on the second floor and above may be estimated by multiplying the free-field ground motions by the amplification factors in Fig. 4. Methods and recommendations for earthquake protection of building equipment and other nonstructural items are given below in the paragraphs on equipment protection.

D. Cost of Providing Earthquake Protection

2.20 Information concerning the cost trends for seismic design of various building systems is important for making design decisions. There has been very little published information on the cost increases of buildings for seismic design. In the past, it has been judged that this cost would not exceed 5 percent of the total contract cost. A recent study using UBC seismic design shows costs for increasing intensity levels for steel and reinforced concrete buildings. The results shown in Fig. 5 include costs of seismic protection of the building foundation, structure, architectural treatment, and mechanical and electrical facilities. They do not include the seismic protection costs for the equipment housed within the building.

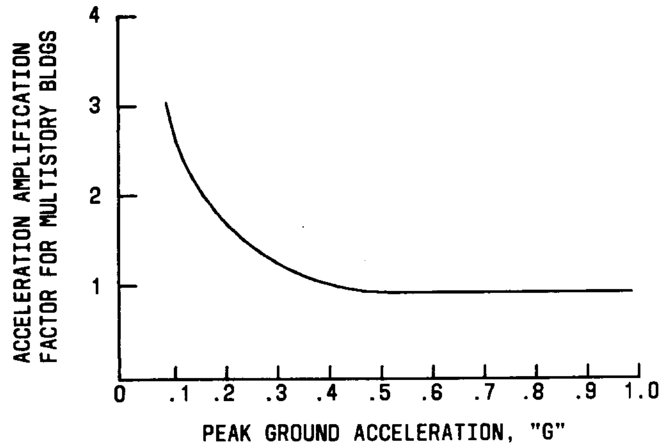


Fig. 4—Amplification of Earthquake Horizontal Acceleration Over That Which Occurs in the Ground for Typical Floors in Multistory Telephone Buildings

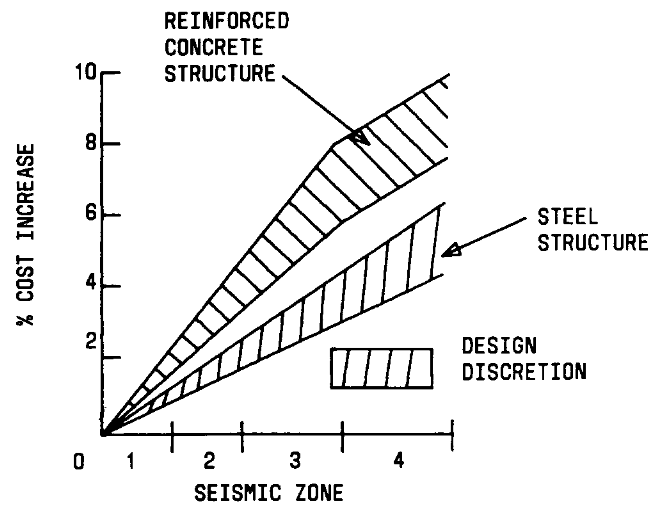


Fig. 5—Approximate Seismic Cost Increase for Medium-Rise Commercial Buildings

2.21 Figure 5 shows that for the UBC high seismic-risk Zone 3, the maximum estimated cost of earthquake protection over the total (nonseismic) construction cost is 3 percent for a structural steel building, 5.5 percent for a reinforced-concrete building, and, in addition, 2 percent for the nonstructural systems such as heating, ventilating, air-conditioning, elevators, ceilings, partitions, and lighting. Reinforced concrete structures have higher costs than steel structures owing to stringent design criteria and placement details. Also, for an equal-volume structure, a concrete structure requires higher seismic design forces because of its greater mass.

2.22 It should be noted that the cost estimates are not based on restrengthening the lateral force-resisting system. The estimates are based on a complete redesign, which may include changing the building frame type to accommodate changing force constraints. The curves in Fig. 5 represent average costs for earthquake-protecting a medium-rise (2-story to 10-story) building and use current UBC seismic design that provides minimum seismic design requirements. Design based on methods different from the UBC obviously may lead to different costs. If other factors are incorporated to meet specific soil conditions and structural needs, maximum design forces may increase and, therefore, the maximum seismic design cost also may increase somewhat.

2.23 In general, Fig. 5 defines the upper limits for the cost increase due to seismic design for telephone structures. These come about because exacting Bell System building specifications and quality control procedures produce a structure more reliable than its commercial counterpart. Also, because of use of reinforced core areas and shear walls in telephone equipment buildings, such structures are expected to be capable of resisting higher seismic forces better than most commercial buildings. Adequate earthquake protection is achieved in many instances at negligible increase in cost for new telephone buildings located in minor and moderate risk areas.

E. General Building-Design Considerations

2.24 Some general principles for building planning in earthquake-prone areas are:

(a) The building should be simple in form and have balanced loads and shear walls. Complex building plans and elevations should be avoided. Under circumstances where the building must have an irregular shape or frame system, the distribution of lateral forces and differences in lateral resistance between adjacent stories or other unusual structural features are to be determined by considering the dynamic characteristics of the building.

(b) The building should not be located upon or near an active fault insofar as is practicable. In selecting the building site, the possibilities of sinking, landslides, and ground displacement, all of which would greatly endanger foundation stability, should be investigated and evaluated.

2.25 Some specific practices related to the earthquake-resistant design of reinforced-concrete telephone buildings are given in Part 4 of this section.

F. Building Equipment Protection

2.26 Mechanical and electrical building equipment may be supported either directly on the base-slab floor (ground floor) or on upper floors within the structure. Equipment supported on the ground floors can be analyzed as damped single-degree or multidegree-of-freedom systems subjected to free-field earthquake excitation. The systems are designed using the seismic coefficient, the response-spectra, or the time-history methods.

2.27 Equipment situated on floors above the base-slab floor will experience building motions that may differ considerably from the seismic motion in the free field and, therefore, must be designed with stronger restraints.

2.28 The degree to which the building response motions differ from the free-field motions depends upon the structural characteristics of the building, its mass distribution, and the nature of the founding medium. Multistory buildings with fundamental frequencies close to the dominant frequency of the earthquake may experience displacements and accelerations on upper levels that are significantly higher than those in the free field as a result of amplifications due to resonance (Fig. 4).

2.29 Equipment processing frequencies close to the primary building frequency and free-field earthquake frequency may be subjected to further response amplifications, which are also caused by the effects of resonance. Maximum resonance effects generally occur at relatively low frequencies, ie, at frequencies that are 15 Hz or lower. Typical examples are a compressor or engine situated on vibration isolators, suspended ceilings, tall slender cabinetry, heavy fluid-filled piping, or air-conditioning ducts that are hung from the building ceilings. Such building components may be excited in resonance by the building dynamic motions. Near the point of resonance between the equipment and building, the maximum acceleration of this type of equipment may be several times higher than the acceleration at its supporting points. Experience has shown that such components may be vibrated to failure unless special bracing, holddown, or snubbing devices are employed to prevent vibration response from building up to destructive amplitudes.

2.30 For example, limit stop snubbers would be used to prevent excessive vibration buildup in a component supported upon vibration isolators. Thus, the isolators would do their intended job of filtering out excessive mechanical vibrations, but they could not be stretched to failure by earthquake motions. Another example is a suspended ceiling system that needs special horizontal support in earthquake areas to prevent excessive lateral swaying. Similarly, massive counterweights used in elevators must be properly designed so that they do not separate from their guiderails.

2.31 Rigid equipment assemblies which possess frequencies higher than 15 Hz, such as small motors, compressors, pumps, or other relatively stiff components that are rigidly attached to the building, will experience essentially the same motions as that of the building points to which they are attached.

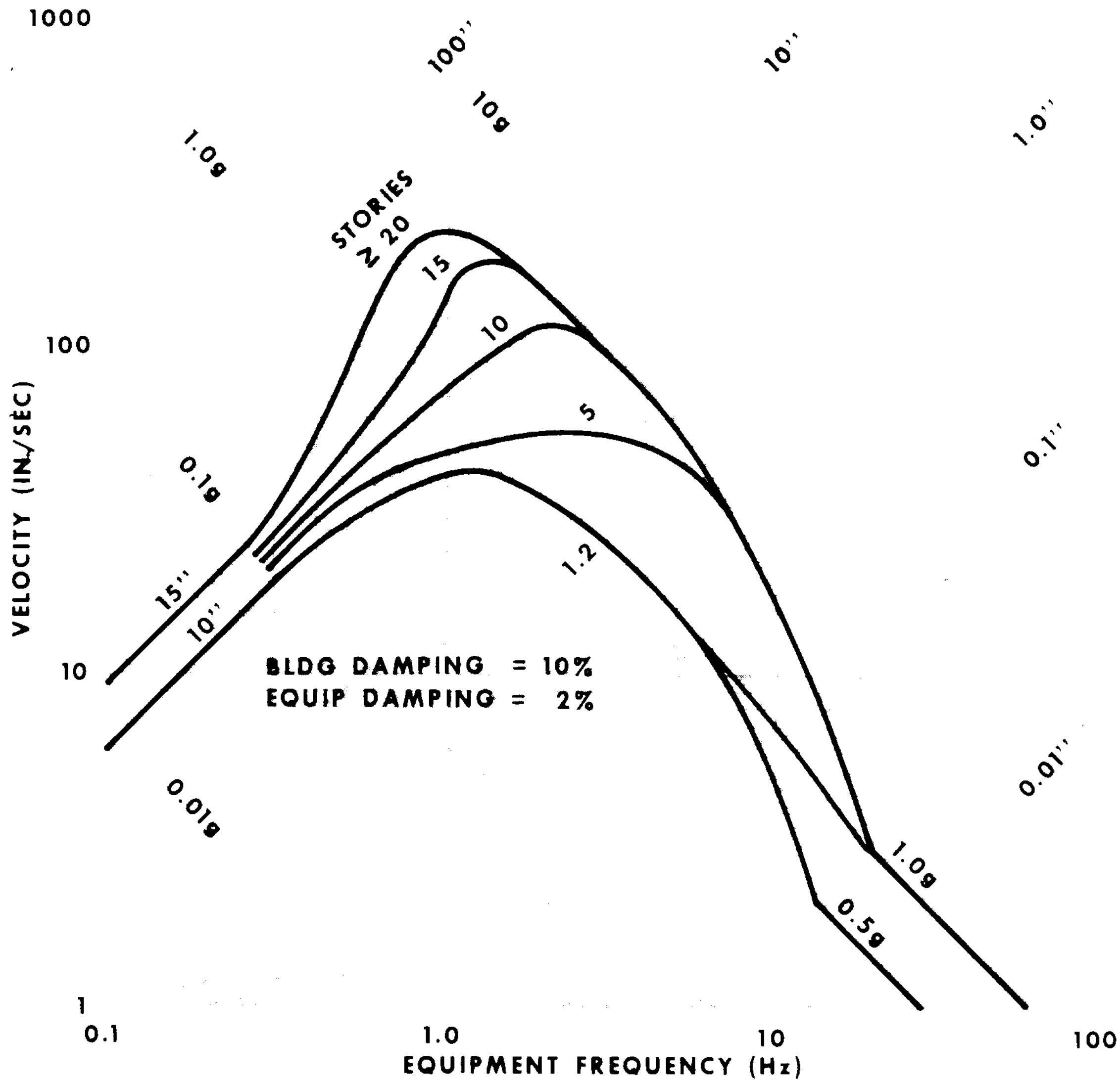
G. Seismic Analysis of Building Equipment

2.32 A commonly used method of seismic analysis and design of equipment is the floor-response-spectra method. Basically, this method takes into account the properties of both the building and the equipment, and the characteristics of the earthquake ground excitation, to determine the amplification at the point where the equipment is attached. The floor response spectra can be constructed if the complete time history of the acceleration response of the building at the equipment location is known. However, this approach is rather complicated, since it involves the detailed modeling of the combined building and equipment system and a complete dynamic analysis of this model using a given earthquake accelerogram.

2.33 Dynamic studies of some typical buildings have led to the development of the earthquake horizontal response spectra shown in Fig. 6. These spectra bound the response of elastic systems that possess about 2 percent critical damping. The spectra are applicable to single and multistory equipment buildings located in areas of major seismic activity. Response spectra for equipment systems having a greater amount of damping, or for those equipment systems that may respond inelastically, will indicate a lower seismic response than the spectra shown in Fig. 6.

2.34 Equipment possessing more than one significant mode of vibration may be analyzed by modal techniques using the Fig. 6 spectra and responses summed by root-mean-square methods. Coupled response in the vertical direction in multistory buildings may be assumed to be the same as that of single-story buildings (eg, the ground response spectrum) or $\pm 0.7g$ over gravity loadings, whichever is more severe. Spectra for buildings in other geographic regions may be constructed by scaling these curves linearly with the spectra reduction factors in Fig. 6.

2.35 The static seismic design factors tabulated on Table C may be used in lieu of the response spectra in Fig. 6 to establish equipment strength requirements. The factors in Table C are derived for equipment possessing fundamental frequencies in five bands which cover installations ranging from flexible to relatively rigid. All of the components in the equipment assembly shall be capable of sustaining the accelerations tabulated. Further care must be taken to avoid designs that possess internal components that may resonate at frequencies below 10 Hz. Damping ratios used to establish the proper seismic factor given in Table C may be determined from physical tests or known data; otherwise, it shall be assumed that the minimum level of 1 to 3 percent of critical exists. In some cases, the design of equipment to meet the acceleration factors under completely elastic (nonyielding) conditions may result in unduly heavy supporting hardware; this is particularly true for flexible (low frequency) equipment that may resonate with the building. Allowance for a lighter supporting system is made by permitting inelastic (ductile) deformations to take place. Where ductile supporting systems are considered desirable to reduce cost, the reduced acceleration forces shown on Table C may be used if calculations or tests show that the ductility levels assumed can and do occur without impairing equipment performance under the application of the reduced (ductile) seismic loadings. When the ductile supporting seismic load factors are used, it shall be shown through calculation or test that



GEOGRAPHIC ZONE (FIG. 2)	4	3	2	1
SPECTRA REDUCTION FACTOR	NONE	1/2	1/4	1/8

Fig. 6—Horizontal Spectra for Equipment in Multistory Buildings in Zone 4

ductile distortions are incipient at the stresses associated with the acceleration force levels used.

2.36 The design of equipment to meet the acceleration force levels shown in Table C will ensure sufficient strength to resist major earthquake loadings. Once designs for strength are completed, calculations or tests shall be performed to assure that equipment deformations under load are not excessive. Fig. 6 shows that low-frequency assemblies will undergo significant displacement and, therefore, care must be taken to provide sufficient "rattle space" for this motion as well as sufficient slack in interconnections to other equipment or to the building. In the design of interconnections, out-of-phase motions must be considered unless rigid structural ties are used to force interconnected equipment to vibrate in a common direction.

TABLE C

EQUIPMENT HORIZONTAL STATIC DESIGN LOADS

FUNDAMENTAL FREQUENCY OF EQUIPMENT	EQUIPMENT LOCATION IN BUILDING	NONYIELDING SUPPORTS			DUCTILE SUPPORTS		
		DAMPING RATIO			DUCTILITY RATIO†		
		1-3%	4-6%	7-10%	u = 2	u = 3	u = 5
$f_e \geq 15$ Hz	Below 3rd floor	0.5g	0.5g	0.5g	0.3g	0.2g	0.2g
(Rigid)	3rd floor & above	1.0	1.0	1.0	0.6	0.3	0.2
$f_e = 13$ Hz	Below 3rd floor	0.8	0.6	0.6	0.4	0.2	0.2
	3rd floor & above	2.0	1.5	1.2	1.2	0.6	0.3
$f_e = 11$ Hz	Below 3rd floor	1.0	0.7	0.6	0.6	0.3	0.2
	3rd floor & above	3.0	2.0	1.5	1.7	0.8	0.4
$f_e = 7$ Hz	Below 3rd floor	1.3	0.8	0.6	0.7	0.3	0.2
	3rd floor & above	4.0	2.5	1.7	2.3	1.1	0.5
f_e 1-5 Hz	Below 3rd floor	1.5	1.0	0.7	0.9	0.4	0.2
(Flexible)	3rd floor & above	5.0	3.0	2.0	2.9	1.3	0.7

f_e = The fundamental mechanical vibration frequency of the equipment assembly.

*The tabulated acceleration forces may be further reduced by multiplying by the geographic region factors in Fig. 6, where applicable. Static design levels in the vertical direction shall be taken as 0.7g above gravity and may also be reduced by the factors shown in Fig. 6. The horizontal and vertical forces shall be assumed to act in the most adverse direction.

†The ductility ratio is defined as the ratio of the maximum displacement under load to the displacement at which major support yielding first occurs. Refer to discussion in text and References 2, 16, 18.

2.37 The response spectra of Fig. 6 and the seismic static design factors of Table C show considerable reduction of acceleration load as the fundamental frequency of the equipment is increased. This is because resonance effects between the equipment and the building decrease with increasing equipment frequency, especially when the equipment frequency is greater than about 7 Hz. As a result, rigid, stiff equipment assemblies are generally desirable. On the other hand, a degree of equipment flexibility achieved through supporting hardware, which may distort in a ductile manner, also serves to reduce acceleration levels. An optimum condition is reached when equipment that is rigid in nature is attached to the building with connections capable of some yielding under load.

2.38 Communications elements, such as cabling and support works, which span between two adjacent (but unconnected) multistory buildings must be designed to accommodate the relative drift (sway) that will occur during an earthquake. The drift of a building during a major earthquake may range from 0.002 to 0.005 times the height above ground, and since adjacent buildings may be out of phase, relative motions twice this amount may be experienced. The building designer should be consulted to obtain seismic drift motions so as to provide for sufficient flexibility or expansion.

2.39 Earthquake criteria to establish equipment mounting procedures are under active study at Bell Laboratories. The recommendations stated here represent the results of some of these studies and may change when all investigations are complete. Preliminary criteria are being presented at this time because of the great importance of adequately mounting building electrical and mechanical equipment, as evidenced by the many failures in the recent Anchorage, Alaska, and San Fernando, California, earthquakes.

H. Specific Earthquake Design Practice for Reinforced Concrete Buildings

2.40 ***Columns:*** Spirally reinforced columns have proved to be superior in strength and ductility to columns reinforced with lateral ties. If the latter are used, however, the following column details are required in addition to the conventional code requirements:

(a) The maximum reinforcement ratio should be 4 percent.

(b) Reinforcing bars should be tied together at intervals not exceeding 12 inches with ties not less than No. 3 bar. No. 4 bars should be used at tops and bottoms of columns. Ties should be anchored either by 90-degree hooks plus an extension of at least 12 bar diameters but not less than 6 inches at the free end of the bar, or by 135-degree hooks plus an extension of at least 6 bar diameters, but not less than 4 inches at the free end of the bar. In members of the lateral-force-resisting system, the free ends of the hooks extend into the column core.

(c) Laps should be 40 diameters minimum for 40,000-pound-yield reinforcing, and correspondingly increased lengths should be used for high-strength reinforcing.

(d) Column splices should not be made in the beam-column joint. Splices are made by lapping (within the column spirals or ties), full-strength welding, or by approved devices. Welding and device splices should be staggered as much as is reasonably possible.

(e) Column ties at not over 4 inches on centers must be carried through all beam-column joints and through column-capital areas in flat slabs.

2.41 ***Walls:*** The walls should have 1/4 percent minimum reinforcement in each direction and anchors around marginal and trim bars. Shear walls must be 9 inches or thicker and must have two curtains of reinforcement. Walls around stairs and elevators should be reinforced concrete of 8-inch minimum thickness with two curtains of reinforcing steel.

2.42 ***Flexural members:***

(a) Minimum positive and negative reinforcement should be 1/2 percent.

(b) Beams and Girders—A minimum of two No. 5 bars, continuous on top and bottom, with 40-diameter laps are to be used. At exterior spans, bars are anchored with 90-degree hooks and have a minimum 32-diameter embedment. The steel reinforcement shall be continuous across the top and bottom layers and shall not be less than 1/4 percent. Stirrups are to be used throughout the beam—spacing shall not exceed d/2 at ends (1/6 span) or 18 inches for the remainder of the span.

(c) Joists—Twenty-five percent of the joists in each direction are to have at least one No. 5 bar top and bottom continuous. Joists should also conform to the recommendations for reinforcement ratio, lap, hooks, and stirrups that apply to the beams and girders.

(d) Slabs are to have 25 percent of positive reinforcing steel continuous throughout the span, and be lapped and anchored at the exterior. Approximately 25 percent of the negative steel should be continuous. Standard provisions must be made for all of the normal forces such as unbalanced loadings, continuity bars over points of abnormal support, and temperature and shrinkage stresses. Because many one-way slabs with wide-flange beams are used in telephone buildings, it is important that slab stresses be calculated at the edge of the beam, and that beam centerline moments be increased to compensate for the haunch effect of the beam on the slab. The proportion of live load in telephone buildings is greater than in conventional office buildings, and this should be provided for in the reinforcing-steel patterns by detailing negative steel farther from the supports and positive steel closer to the supports.

2.43 ***Floors and cable slots:*** Cable slots should not be cut into concrete slabs because this destroys the continuity of the reinforcing steel. Absolute minimum reinforcing on top of the slab should be No. 4 bars at not over 3-foot centers. The bottom layer should have more continuous reinforcing. The absolute minimum slab thickness for a floor or roof is 5 inches even for a very short span.

2.44 ***Trim bar at openings:*** All openings must have trim bars of adequate size and length to accommodate not only the usual vertical loads and temperature shrinkage stresses, but also the diaphragm shears, or shear-wall stresses. The extensions (anchorages) of these bars, as determined by bond stresses, are not sufficient. In addition to the normal cable and duct openings and plugged cable holes (see Section 760-200-032), the requirement for trim bars is especially important at major discontinuities of the slab diaphragm, such as at stairs, elevators, re-entrant corners of the building, light wells, etc.

2.45 The tensile forces of the slab diaphragm should be carried entirely in chord bars at the edge of the diaphragm, not in the typical slab reinforcing. In the event of future building growth the larger concentrated chord bars provide something substantial to tie the old and new portions of the structure together.

2.46 If the chord-reinforcing bars are separated from the body of the diaphragm by a series of slots, the horizontal shear through the slots must be resisted by a substantial calculable shear system.

2.47 ***Collector (drag) reinforcing:*** An important element of earthquake-resistant design is the tying together of all elements of the building. This is not specified in any building code because there are no known criteria for the amount of tying; nonetheless, this does not mean that the item is unimportant. Substantial ties are required between different units of a building. Furthermore, it is of utmost importance that various stages of a building (those built at different times) be adequately tied together. The minimum tie in Zones 3 and 4 should be capable of resisting 10 percent of the total weight of the lighter structure, but additional strength may be required if the two (or more) units are to act together.

2.48 Telephone buildings characteristically have numerous slots in the floor and also have shear walls or other bracing elements that are not uniformly spaced around the diaphragms. These conditions require that "collector" or "drag" bars be provided.

2.49 The partial interior shear wall (Fig. 7) requires reinforcing to collect the shears or drag the forces in the diaphragm into the shear wall.

2.50 A common configuration for telephone buildings is shown in Fig. 8. A long series of cable slots cuts the floor diaphragm next to the shear walls. The diaphragm shears are more or less uniformly distributed throughout the width of the diaphragm. The connection of the diaphragm to the walls is primarily through the continuous portions (shown as X in Fig. 8) of the floor slab. The collector reinforcing bars are needed to collect the uniform shears of the diaphragm and deliver them to the solid-floor portions, X, at the ends. There is an eccentricity between the collector bars and the shear wall that causes reinforcing bars T to be required. If the shear wall is solid, or has uniform rigidity throughout its length, then collector bars will also be required in the wall to redistribute the forces from X to the entire length of the wall.

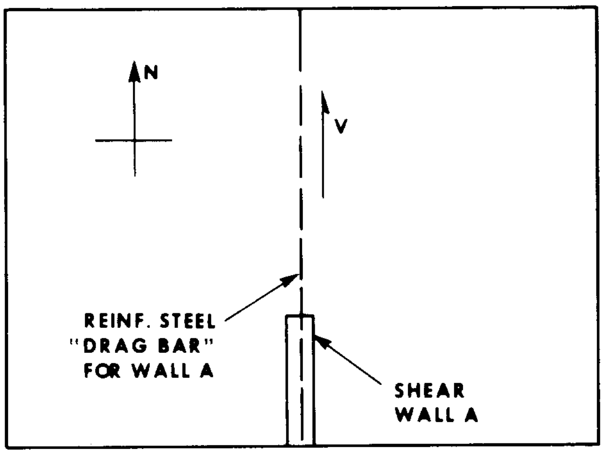


Fig. 7—Plan of Building Showing Reinforcement Required to Collect the Shear in the Diaphragm into the Partial Shear Wall

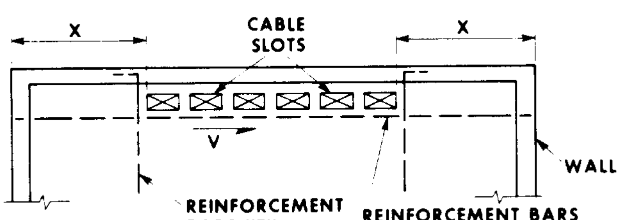


Fig. 8—Common Configuration of Telephone Building Showing the Collector Reinforcing Bars Needed for Shear Force Transfer

2.51 A very common condition in many buildings is shown in Fig. 9. For east-west loadings, bar A is needed to collect diaphragm shears, as discussed above. Bars B must be provided for the same reason as bars T were provided in Fig. 8. In additions, bars B serve as collector bars for diaphragm shears in the north-south direction. Bars C act as collector bars for north-south loadings, but also act as east-wall chord bars under east-west loadings.

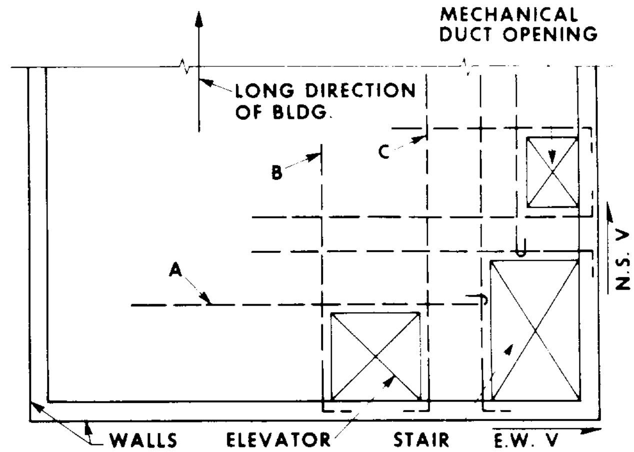


Fig. 9—Building Floor Plan Showing Collector Reinforcing Bars A, B, and C

2.52 ***Torsion:*** Observation of earthquake effects confirms that torsion is a cause of much damage. For buildings with unbalanced shear walls and an open end, major torsions will probably occur in the buildings. Buildings with a length-to-depth ratio (L/D) of about 1/2 or less should have little trouble from torsion in an earthquake, since the total deflections including torsion will be about the same as the symmetrical loading of the earthquake in the perpendicular direction. With ratios of L/D above 1/2, the torsional deflections increase rapidly and damage will occur at the open end unless specific precautions are taken. Unfortunately, with the need for buildings that can be extended, an open-ended structure is usually required for the first or even later stages of construction to permit growth in that direction. Consequently, the structural engineer must provide the maximum possible strength and rigidity at the open end without precluding growth.

2.53 If a rigid element is not possible within the other constraints of the construction of this wall, then the structure must be expected to deform and the architectural details must accommodate this deformation. The engineer and architect must detail this portion of the building with the knowledge that major deflections will occur. This will usually invalidate use of ordinary field-welded connections.

2.54 When removing walls while extending a building, the engineer must provide adequate seismic resistance for both the building and the equipment housed therein. In cases where building shear walls are removed, temporary bracing must be provided to carry earthquake loadings prior to completion of the extended structure. Also, in such cases the floor above the removed shear wall may be required to carry shear stresses and moments in the horizontal plane so as to transfer loads to the new building wall. When equipment is anchored to a wall that is to be moved, care must be taken to provide temporary equipment support as well as final support against seismic forces.

2.55 ***Future extension:*** Telephone buildings are expected to grow as population increases and average telephone usage increases. This causes serious problems for the structural engineer in arranging his load-resisting elements. The growth may not follow the pattern that was predicted when the original structure was designed, so the structural engineer must design ahead for other reasonable possibilities. All of the reasonable eventualities must be considered when the first unit is designed. Because building extensions will probably be constructed years after the original design was made, it is important that structural drawings be prepared that indicate the growth pattern originally considered and the design tributary area of each of the major resisting elements. Obviously these records should be preserved by the Operating Company building engineer.

2.56 ***Overturning, J factor, and shear walls:*** Present codes do not fully account for the vertical loadings exerted by earthquakes; there are changes contemplated and discussed in the engineering committees that recommend code changes. For that reason, and because of observed damage patterns of recent earthquakes, the J factor for overturning must not be less than 1.0. Overturning stresses must be accounted for in columns and foundations. Because of vertical-acceleration effects, the steel column splices, anchors, reinforcing steel in columns, etc, must be designed for conditions of minimum as well as maximum load. It is recommended that the minimum loads on columns that can be relied upon for uplift, reduction of tensile steel, etc, be 50 percent of the design dead load in Zones 3 and 4.

2.57 Shear walls are to be fully designed to account for all stresses. This includes bending, shear, and torsion in all columns and spandrels as well as joint details and intersections. The overturning components through spandrels or horizontal elements must at least equal the overturning stresses in the columns. Shears are to be limited to design stresses allowed by code. Bond stresses must be calculated.

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4. EARTHQUAKE PROTECTION STANDARDS

A. Buildings

4.01 Determine from Fig. 2 and Table A the expected peak ground acceleration for a telephone facility constructed in a given geographic location.

4.02 Use the seismic-coefficient method of design as outlined in building codes such as the UBC for all telephone buildings regardless of the number of stories. However, use the response-spectrum or time-history methods, based on an earthquake of a magnitude determined from 4.01 above, to verify the design of buildings taller than five stories.

4.03 Use the UBC-specified seismic coefficients and equivalent static lateral loads for risk zones outlined by Fig. 2, or other local governing codes if they are more conservative, as the minimum requirements for earthquake-resistant design of telephone buildings.

4.04 When calculating lateral loads, add to the dead load a loading equal to 50 percent of the vertical design live load (150 psf) less 10 psf.

4.05 Design buildings to be simple in form, with balanced loads and shear walls, and to be founded on stable ground. Do not locate upon or near an active fault.

4.06 After conforming to established building codes, give special attention to the following design details:

(a) Reinforced concrete in columns, walls, joists, beams, girders, and slabs.

(b) Trim bars and reinforcing-steel continuity around cable holes and major openings such as stairs, elevators, and re-entrant corners of the building.

(c) Collector or drag reinforcing.

(d) Reinforcing steel for torsion resistance.

(e) Consideration of possible future building extensions and additions.

4.07 For building equipment that is vital to the survivability of communications, design the equipment using the floor-response-spectra approach to avoid overstress in equipment supports and to prevent overturning and dislocation during earthquakes.

B. Building Electrical and Mechanical Equipment

4.08 All vital electrical and mechanical equipment and installations shall be adequately designed to perform satisfactorily after the earthquake and be safe against overturning or dislocation during earthquakes. All vital equipment shall meet the following:

(a) Equipment in buildings shall be designed to meet the horizontal response spectra in Fig. 6 together with the ground level vertical response spectrum. A vertical support motion of $\pm 0.7g$ over gravity loadings should be considered in lieu of the ground level vertical response spectrum if this causes a more severe loading. In lieu of using the Fig. 6 response spectra, the horizontal force factors in Table C may be used as an acceptable alternative. The spectra reduction factors shown in Fig. 6 may be used to reduce the Fig. 6 spectra or the static design factors (Table B in areas of lower seismic risk).

(b) When the response spectra technique is used, equipment situated on basement, ground level, and the first above-grade level may be designed to the one- to two-story building criteria. Equipment located on floors above the first above-grade level in multistory buildings shall be designed to the response spectrum corresponding to the actual floor level. Where response spectra for particular floor levels are not shown in Fig. 6, the response spectrum shown for the next higher level shall be used.

4.09 Combined state of stresses, including those caused by earthquake loads such as tension and shear on anchor bolts, shall be investigated in accordance with established design codes, such as UBC.

4.10 Equipment shall be secured by bolts, embedment, or other acceptable schemes. Reliance on friction only is not allowed as an acceptable method. Equipment supports made of brittle material, such as cast iron, are not permitted.

4.11 Where possible, bracing that increases the stiffness of the equipment assembly should be used to reduce motion, lateral loads, and stresses.

4.12 Snubbing devices with limit stops shall be used on all equipment supported by vibration isolators to prevent excessive movement of the isolator. Limit stops shall be solidly anchored to the building structure and shall be capable of withstanding vertical, horizontal, and overturning loads caused by the response of the isolated component.