EARTHQUAKES

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1.0 SCOPE

FM earthquake zone maps, as well as seismicity and earthquake background material, are presented in this data sheet. This data sheet also includes recommendations and information related to earthquake design and past earthquake performance of buildings and equipment.

Numerous other data sheets address earthquake in whole or in part. Most notably, Data Sheet 1-11, *Fire Following Earthquakes*, and Data Sheet 2-8, *Earthquake Protection for Water-Based Fire Protection Systems*. Most, but not necessarily all, other data sheets having additional earthquake-related recommendations are listed in Section 4.1. These include Data Sheet 1-42, *MFL Limiting Factors*; Data Sheet 3-2, *Water Tanks for Fire Protection*; Data Sheet 7-7 *Semiconductor Fabrication Facilities*; and Data Sheet 10-1, *Pre-Incident Planniing* .

1.1 Changes

January 2024. Interim revision. Minor editorial changes were made.

July 2023. Interim revision. This revision primarily incorporates technical guidance for the expanded scope of equipment including the following items:

- Raised access floors and supported equipment.
- Transformers and battery installations.
- HVAC and refrigeration equipment on structural floor.

Section C.5.5 is added in this revision to illustrate generic anchorage for equipment. This revision also incorporates updates from the American Society of Civil Engineers (ASCE) standards and clarifies some items in the data sheet. Minor editorial changes were made to Sections C.4.3, C.4.3.3, C.6.1, C.6.2 and Table C.4.3.3. These changes will have little or no effect on overall guidance.

July 2022. Interim revision. The worldwide earthquake zones visual overview shown in Appendix C, Figures C.6.3A and C.6.3B has been changed to correctly reflect the FM Worldwide Earthquake Map, the authoritative source that has been available online since January 2021 at https://www.fmglobal.com. There are no changes to the January 2021 earthquake zones, online map, or technical guidance.

January 2021. Interim revision. FM Global earthquake zones have been revised worldwide except for some small islands, and the FM Global Worldwide Earthquake Map, available online, is now the authoritative source.

Editorial revisions have been made to align with these changes, as follows:

A. Revised Section 2.1.

B. Revised Appendix C Sections C.6, C.6.1, C.6.2, and C.6.3.

C. Revised Appendix C Figures C.6.3A and C.6.3B.

D. Deleted Appendix C Table C.6.3.

E. Deleted Appendix C Figures C.6.3C to C.6.3I.

1.2 Hazards

An earthquake is a sudden shaking of the earth caused by shifting rock at or beneath the earth's surface. Seismic events generate both vertical and horizontal motion, but the horizontal motion usually governs earthquake performance of buildings and their contents.

Shake damage, typically the largest component of an earthquake-related loss, varies significantly among sites depending on the intensity of the shaking and the particular characteristics of the facility. Earthquake ground shaking at a facility can cause minor to moderate damage to many items, but also can include major damage to just a few items.

The type of building material and earthquake-resisting system used in a structure can have a significant effect on earthquake shake damage. Improperly protected equipment can move violently during an earthquake, resulting in toppling, sliding, swinging, or falling equipment, even when structures are not significantly damaged. A substantial reduction in earthquake shake damage is most cost-effectively achieved by establishing and applying seismic design criteria for new equipment installation and building construction. In-place contents and existing buildings can be retrofitted to increase their seismic resistance, but this can be more costly and at times is impractical.

Similarly, bracing sprinkler system piping, anchoring fire protection system water supply equipment, and protecting flammable gas and ignitable liquid systems when they are originally installed is the best way to avoid shake damage and consequent loss from water leakage, fire protection system impairment, and fire following earthquake. Finally, it is important to develop an effective earthquake emergency response plan that establishes procedures to repair damage that occurred, to prevent further damage, and to facilitate resumption of business.

2.0 LOSS PREVENTION RECOMMENDATIONS

2.1 Introduction

Recommendations are applicable to FM 50-year through 500-year earthquake zones shown in Figures C.6.3A and C.6.3B in Appendix C and in detail on the FM Worldwide Earthquake Map, online at https://www.fmglobal.com. Appendix C also provides details regarding recommendations.

Provide FM Approved equipment, materials, and services whenever they are applicable and available. For a list of products and services that are FM Approved, see the *Approval Guide*, an online resource of FM Approvals (www.approvalguide.com).

2.2 Construction and Location

2.2.1 Site Considerations

2.2.1.1 Assess site seismic hazards and their consequences to facility operation during the initial site selection to determine the earthquake-related risks and methods to reduce those risks. Consider hiring a qualified geotechnical engineer to assess the site.

2.2.1.2 Do not locate facilities on sites where the potential for earthquake-caused ground rupture, landslide, dam failure, etc. is significant.

2.2.1.3 Where new construction cannot be avoided on reclaimed land sites or sites having a significant liquefaction potential, support structures on appropriate foundations (e.g., deep piles) per the recommendations of a qualified geotechnical engineer. Install and detail important items located directly in or on the ground (e.g., tanks, piping and other utilities) to minimize damage from ground settlement or liquefaction.

2.2.2 Design Standards

2.2.2.1 Have buildings, structures, tanks, equipment, storage racks, piping-system bracing, mezzanines, non-structural elements, etc., designed by an engineer registered to practice structural design in the jurisdiction in which the project is located.

2.2.2.1.1 Provide earthquake anchorage details for important occupancy equipment and systems. Important occupancy equipment and systems may include, but are not limited to, the following:

- Electrical power: transformers, switchgear, batteries, independent circuit breakers, bus bars, cable trays.
- Data processing: server racks, UPS, data storage modules, cooling units, raised access floors, cable trays.
- Process: equipment, tanks, cylinders, piping, in-process storage, conveyors.
- Warehousing: pallet racks, material racks, shelving units.
- HVAC: chillers, boilers, cooling towers, air-handlers, rooftop package units, fans, heat exchangers, piping.
- Plumbing: water heaters, boilers, piping, water tanks.
- Architectural: suspended ceilings, partitions.
- Roof-mounted: antennas, exhaust stacks, pollution control, HVAC, photovoltaic panels (also see Data Sheet 1-15, *Roof Mounted Solar Photovoltaic Panels*).

For important items that cannot be adequately assessed using calculations alone (e.g., transformer bushings), use available data from shake table evaluations (e.g., based on AC 156, *Acceptance Criteria for Seismic Certification by Shake-Table Testing of Non-Structural Components*, or IEEE 693, *IEEE Recommended Practice for Seismic Design of Substations*).

2.2.2.1.2 Except as amended by other recommendations in this or other data sheets, design buildings and equipment and content load-resisting elements and anchorage in accordance with the requirements and design earthquake forces of ASCE 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, or a building code based on this standard (e.g., the International Building Code). Where earthquake maps are not provided in ASCE 7 (i.e., for locations outside of the United States, Puerto Rico, the Virgin Islands, and Guam), determine design acceleration parameters S_{DS} and S_{D1} for the location based on information from a reliable source that uses the ASCE 7 methodology. If these parameters cannot be determined, use the generic values of S_{DS} and S_{D1} provided below:

- FM 50-year earthquake zone: S_{DS} = 1.3 (g) S_{D1} = 0.8 (g)
- FM 100-year earthquake zone: S_{DS} = 0.9 (g) S_{D1} = 0.45 (g)
- FM 250-year earthquake zone: S_{DS} = 0.55 (g) S_{D1} = 0.25 (g)
- FM 500-year earthquake zone: S_{DS} = 0.55 (g) S_{D1} = 0.25 (g)

Where:

S_{DS} = the design spectral response acceleration parameter at a short (0.2-second) period (at 5% damping and adjusted for Site Class [soil] effects), expressed as a portion of the gravitational acceleration (g).

S_{D1} = the design spectral response acceleration parameter at a period of one second (at 5% damping and adjusted for Site Class [soil] effects), expressed as a portion of the gravitational acceleration (g).

2.2.2.1.3 Meet the following recommendations for attachments to concrete using post-installed concrete anchors (e.g., expansion anchors, screw anchors, and adhesive anchors installed in hardened concrete):

A. Use only post-installed concrete anchors that are prequalified for shear and tension in cracked concrete for seismic applications in regions of moderate and high seismic risk (e.g., Seismic Design Category C through F in ASCE 7) in accordance with American Concrete Institute (ACI) Standard 355.2, *Qualification of Post-Installed Mechanical Anchors in Concrete*, or Standard 355.4, *Qualification of Post-Installed Adhesive Anchors in Concrete*; and designed in accordance with ACI 318 *Building Code Requirements for Structural Concrete and Commentary*; or equivalent local building code or standards.

B. Design post-installed concrete anchors and establish quality control procedures (e.g., special inspection, including torque or tension testing, during installation) based on building code and manufacturers' requirements.

C. For post-installed concrete anchors, use a minimum nominal (i.e., before the anchor is set) embedment of at least 7 times the bolt diameter ($7 \bullet D_b$) for ¼ in. (6 mm) and ½ in. (12 mm) anchors, and ($6 \bullet D_b$) for other anchors; anchor spacing of at least ($8 \bullet D_b$); and distance from concrete edges of at least ($12 \bullet D_b$); unless different values are allowed or required by the manufacturer and the calculated capacity of the anchor is more than required.

D. In lieu of calculations, Allowable Stress Design (ASD) capacities for expansion or wedge anchors can be taken from Table C.5.4.2 in Appendix C. Alternatively, calculate the appropriate fastener size based on ASD capacities for post-installed concrete anchors selected from a manufacturer's product line meeting the conditions in Item A through Item C above as well as the following conditions:

1. Establish the shear and tension capacities of an anchor or group of anchors using ACI 318, or a similar local building code or standard. Reduce the ACI 318 shear and tension capacities to account for ASCE 7 overstrength (Ω_0) and load factors by multiplying ACI 318 Load and Resistance Factor Design (LRFD) values by 0.42 or by multiplying ACI 318 ASD values by 0.6.

2. The relationship between actual calculated shear and tension loads, and allowable shear and tension loads, must conform to the following equations:

$$(S_{ACT}/S_{ALL}) + (T_{ACT}/T_{ALL}) \leq 1.2$$
$$(S_{ACT}/S_{ALL}) \leq 1.0$$
$$(T_{ACT}/T_{ALL}) \leq 1.0$$

where

S_{ACT} = calculated actual shear load.

S_{ALL} = local governing jurisdiction-approved ASD allowable shear load capacity (including overstrength [Ω_0] and load factor reductions).

T_{ACT} = calculated actual tension load.

T_{ALL} = local governing jurisdiction-approved ASD allowable tension load capacity (including overstrength [Ω_0] and load factor reductions).

See Appendix C and Data Sheet 2-8, Section 3.1.9, for further information regarding post-installed concrete anchors.

2.2.2.1.4 Use cast-in-place (CIP) anchors or inserts (i.e., anchors or inserts cast into concrete during construction) designed in accordance with ASCE 7 and ACI 318, *Building Code Requirements for Structural Concrete,* (including overstrength [Ω_0] and load factor reductions); or equivalent local building code or standard; for use in cracked concrete for seismic application in regions of moderate and high seismic risk (e.g., Seismic Design Category [SDC] C through F in ASCE 7). Proprietary CIP inserts that are prequalified for use in cracked concrete in regions of moderate and high seismic risk using a combination of testing and calculations specified by an essentially similar standard (e.g., ICC-ES AC446, Acceptance Criteria for Headed Cast-in Specialty Inserts in Concrete) are acceptable. See Data Sheet 2-8, *Earthquake Protection for Water-Based Fire Protection Systems*, recommendation 2.2.1.3.6.5 and Section 3.1.9 for additional guidance.

2.2.2.1.5 Do not use powder-driven fasteners (see glossary) to attach to steel or concrete elements of the structure, except as allowed for lightweight acoustical tile ceilings in ASCE 7 and its referenced ASTM International standards (see Data Sheet 2-8, recommendation 2.2.1.6.2 and Section 3.1.11).

2.2.2.2 Design fire protection water tanks (e.g., suction tanks, gravity tanks) per the requirements, including the earthquake provisions, of Data Sheet 3-2, *Water Tanks for Fire Protection.*

2.2.2.3 Design fire protection systems, including piping and fire pump systems (i.e., the pumps and drivers, controllers, starter batteries, fuel tanks, emergency generators powering fire pumps, etc.) to meet the earthquake protection requirements in Data Sheet 2-8, *Earthquake Protection for Water-Based Fire Protection Systems*. See also 2.5.1.

2.2.2.4 Follow recommendations to prevent fire following earthquake, including pipe bracing, equipment anchorage, seismic shut off valves, etc., in Data Sheet 1-11, *Fire Following Earthquake*.

2.2.2.5 Design Maximum Foreseeable Loss (MFL) fire walls to meet the earthquake protection requirements in Data Sheet 1-42, *MFL Limiting Factors*.

2.2.2.6 Meet the earthquake requirements specific to certain occupancies or facilities that are contained in other FM data sheets (see Section 4.1 for a list of many, but not necessarily all, of these data sheets).

2.2.3 Other New Design Considerations

2.2.3.1 Design vital buildings and equipment, such as hospital structures, fire stations, and fire protection suction tanks (as well as those buildings and equipment critical to the facility or processes, even if not considered “essential” by traditional building code criteria) with increased seismic safety to resist damage and remain operational during and after an earthquake. This may include use of base isolation or dampening systems. Incorporate procedures to improve design and construction quality, such as peer review, submittal review, and frequent site observation by the engineer of record.

2.2.3.2 Where practical, equipment and piping that can leak fluid, corrosive gas, etc. if damaged should be located so as to limit the consequences of the leak (e.g., away from cleanrooms, valuable and damageable storage, etc.).

2.2.3.3 Design buildings housing on-premises fire services to be earthquake-resistant and provide them with very lightweight vehicle doors that are interlocked to automatically open upon warning of strong shaking via an earthquake early warning system, if available.

2.2.3.4 Provide a single common monolithic foundation pad to support interconnected equipment, such as drivers and pumps. Where this is not possible, provide connections with sufficient flexibility to accommodate expected differential movement.

2.2.3.5 When settlement would result in disorientation of equipment that must remain level, plumb, or aligned, do the following:

A. Incorporate a self-contained manual leveling mechanism into the equipment.

B. Establish a contingency plan for achieving efficient realignment.

2.2.3.6 Prior to making modifications to a structure (such as mounting heavy objects on or suspending them from roofs, removing braces, or cutting openings in walls), have the structure evaluated by a registered structural engineer.

2.3 Equipment and Processes

2.3.1 For tanks containing ignitable or corrosive substances, or that could cause extended process interruption if released, do the following:

A. Provide tank installations; designed using ASCE 7 and the concepts in Data Sheet 3-2, *Water Tanks for Fire Protection*, or other appropriate standards; that prevent the release of liquids due to seismic shaking, including structural failure of the tank or supports, horizontal sloshing forces on the tank walls, and vertical sloshing forces on the tank roof or cover.

B. Provide internal baffles to minimize sloshing forces if necessary to prevent damage to the tank.

C. Provide sufficient freeboard, as calculated using ASCE 7 or other appropriate standard, to prevent liquid release from sloshing when open tanks are necessary for process reasons. Where this is not practical due to process reasons, provide secondary containment (e.g., curb or trench drain) or implement other measures as necessary to minimize the consequences of a spill. Use guidance in Data Sheet 7-2, *Waste Solvent Recovery*, Data Sheet 7-32, *Ignitable Liquid Operations*, Data Sheet 7-83, *Drainage and Containment Systems for Ignitable Liquids* and other appropriate hazard or occupancy-specific data sheets.

2.3.2 For piping containing ignitable or corrosive liquids, or liquids that could cause extended process interruption if released, do the following:

A. Provide one or more seismic sensors that signal, directly or via a control panel, safety shutoff valves (SSOVs) installed in the pipe at locations that minimize the hazard of a spill (e.g., near the tank or before entry into a building). Arrange these for automatic operation to safely shutdown the flow of liquid in the event of strong ground motion. See Data Sheet 1-11 for information related to seismic shutoff systems. Where release of liquid does not present a fire-following earthquake hazard, it may be acceptable to signal SSOV via alerts of approaching strong ground motion from an extensive and reliable regional earthquake early warning system (EEWS), if available. See Data Sheet 10-1 for information regarding EEWS and earthquake response. Where a process safety review or guidance in other data sheets indicates that automatic shutdown would introduce unacceptable new hazards that cannot be mitigated (see examples in Data Sheet 1-11), automatic shutoffs may be omitted.

B. Provide seismic restraint and flexibility similar to that required for piping in Data Sheet 1-11, including, but not limited to, flexible couplings, flexibility across seismic joints, sway bracing, clearance and pipe hangers.

2.3.3 Design internal components of tanks that could cause extended process interruption if damaged (such as baffles in water treatment tanks) for seismic forces, including those induced by sloshing of liquid.

2.3.4 Provide a safe, remote shutoff for electrical service.

2.3.5 Provide an integrated seismic protection system (i.e., an arrangement of local seismic sensors and/or a reliable regional earthquake early warning system [EEWS]) that will withstand the effects of a severe earthquake and function to shut down major equipment in a safe condition in response to strong ground shaking. See Data Sheet 1-11 for information related to seismic sensors and Data Sheet 10-1 for information regarding EEWS.

2.3.6 Avoid the use of automatic-starting process equipment.

2.4 Occupancy

2.4.1 Keep heavier items on storage racks on the lower shelves or on pallets on the floor (but not in aisles).

2.4.2 Secure valuable storage kept on open shelves by installing a lip or horizontal barrier of appropriate height on the shelf.

2.4.3 Chain or fasten valuable or vital equipment used or stored on workbenches to the supporting surface. Brace or anchor the benches themselves to limit movement.

2.4.4 For portable liquid containers containing ignitable or corrosive substances, or that could cause extended process interruption if released, do the following:

A. Provide unbreakable containers, located as close to floor level as practical and restrained from falling from shelves or racks. If glass containers must be used for process reasons, place the glass container within an outer fixed container constructed of unbreakable material that is restrained from movement.

B. Locate glass containers of corrosive liquids in ventilated cabinets or dedicated rooms to restrict the spread of fumes.

C. Store chemicals that would react violently with one another in separate rooms or cabinets such that the release of liquids from containers will not allow the liquids to mix.

2.5 Protection

2.5.1 Locate fire pumps in a structure that is earthquake-resistant. Provide diesel-powered pumps where possible. If pumps are electric-powered, furnish an automatically activating emergency power supply that is properly protected against earthquakes. (See also 2.2.2.2 and 2.2.2.3).

2.6 Operation and Maintenance

2.6.1 Have a qualified person inspect the following at least annually to detect damage and identify needed repairs or maintenance:

A. Buildings and structures significant for operations

B. Fire protection systems

C. Warehouse storage racks

D. Other equipment significant for operations

2.7 Human Factor

2.7.1 Earthquake Emergency Response

2.7.1.1 Establish an earthquake emergency response team within the overall facility emergency response team, and a comprehensive earthquake emergency plan to provide guidelines for control of hazards, fire safety, repairs, and salvage. See Data Sheet 10-1 for specific guidance.

2.7.1.2 Monitor the regional earthquake early warning system (EEWS), where available, to alert personnel and allow manual or automatic shutdown of critical processes or equipment prior to arrival of strong ground motion. See Data Sheet 10-1 for further guidance.

2.7.1.3 Implement a building occupancy resumption program (BORP), or back-to-business program, to facilitate return to business following an earthquake. See Data Sheet 10-1 for further guidance.

3.0 SUPPORT FOR RECOMMENDATIONS

3.1 General

Refer to Appendix C, Supplemental Information, for general comments on recommendations.

4.0 REFERENCES

4.1 FM

Approval Guide, an online resource of FM Approvals

Approval Standard Class Number 1950, *Approval Standard for Seismic Sway Braces for Pipe, Tubing and Conduit*

Approval Standard Class Number 4020, *Approval Standard for Steel Tanks for Fire Protection*

Country Building Codes Index, Publication P15105

Data Sheet 1-6, *Cooling Towers*
Data Sheet 1-8, *Antenna Towers and Signs*
Data Sheet 1-11, *Fire Following Earthquakes*
Data Sheet 1-15, *Roof Mounted Solar Photovoltaic Panels*
Data Sheet 1-35, *Vegetative Roof Systems, Occupied Roof Areas and Decks*
Data Sheet 1-42, *MFL Limiting Factors*
Data Sheet 1-56, *Cleanrooms*
Data Sheet 1-62, *Cranes*
Data Sheet 2-0, *Installation Guidelines for Automatic Sprinklers*
Data Sheet 2-8, *Earthquake Protection for Water-Based Fire Protection Systems*
Data Sheet 3-2, *Water Tanks for Fire Protection*
Data Sheet 3-6, *Lined Earth Reservoirs for Fire Protection*
Data Sheet 3-7, *Fire Protection Pumps*
Data Sheet 4-2, *Water Mist Systems*
Data Sheet 5-14, *Telecommunications*
Data Sheet 5-17, *Motors and Adjustable Speed Drives*
Data Sheet 5-23, *Design and Protection for Emergency and Standby Power Systems*
Data Sheet 5-32, *Data Centers and Related Facilities*
Data Sheet 7-7, *Semiconductor Fabrication Facilities*
Data Sheet 7-12, *Mining and Ore Processing Facilities*
Data Sheet 7-32, *Ignitable Liquid Operations*
Data Sheet 7-38, *Loss Prevention in Ethanol Fuel Production Facilities*
Data Sheet 7-39, *Materials Handling Vehicles*
Data Sheet 7-54, *Natural Gas and Gas Piping*
Data Sheet 7-55, *Liquefied Petroleum Gas (LPG) in Stationary Installations*
Data Sheet 7-64/13-28, *Aluminum Smelting*
Data Sheet 7-79, *Fire Protection for Gas Turbines and Electric Generators*
Data Sheet 7-83, *Drainage and Containment Systems for Ignitable Liquids*
Data Sheet 7-88, *Ignitable Liquid Storage Tanks*
Data Sheet 7-101, *Fire Protection for Steam Turbines and Electric Generators*
Data Sheet 7-106, *Ground-Mounted Solar Photovoltaic Power*
Data Sheet 7-108, *Silane*
Data Sheet 8-9, *Storage of Class 1, 2, 3, 4 and Plastic Commodities*
Data Sheet 8-18, *Storage of Hanging Garments*
Data Sheet 8-23, *Rolled Nonwoven Fabric Storage*
Data Sheet 8-33, *Carousel Storage and Retrieval Systems*
Data Sheet 10-1, *Pre-Incident Planning*
Data Sheet 10-5, *Disaster Recovery Planning*
Data Sheet 13-10, *Wind Turbines*

4.2 Other

American Concrete Institute (ACI), ACI 318, *Building Code Requirements for Structural Concrete and Commentary*.

American Concrete Institute (ACI), ACI 355.2, *Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary*.

American Concrete Institute (ACI), ACI 355.4, *Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary*.

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APPENDIX A GLOSSARY OF TERMS

Acceleration: Rate of change in velocity with respect to time resulting from earthquake ground motion.

Accelerometer: A seismograph designed to measure ground motion earth particle accelerations, from an earthquake or other source.

Active Fault: A fault that has experienced displacements in recent geological time (generally considered to be within the last 11,000 years but ranges of 10-15,000 years are sometimes used), and the potential for future displacement is great enough for concern.

Allowable Stress Design (ASD): A method of designing structural members such that computed stresses produced by normal gravity design loads (e.g., the weight of the building and usual occupancy live loads) do not exceed allowable stresses that are typically below the elastic limit of the material (e.g., in steel these are typically well below the yield point). Normal allowable stresses are sometimes increased by a factor (often a one-third increase is used) when design includes extreme environmental loads such as earthquakes. (Also called working stress design or elastic design).

Alluvial Soil (Alluvium): Soils carried and deposited by water, such as those found at the deltas of rivers reaching lakes or oceans.

Amplitude: the distance from a peak to the baseline in a time series.

FM Approved: Products and services that have satisfied the criteria for FM Approval. Refer to the *Approval Guide*, an online resource of FM Approvals, for a complete listing of products and services that are FM Approved.

Attenuation: The decrease in seismic energy, or amplitude of seismic waves, with distance from its source through absorption and scattering.

Base Shear: Total design lateral force or shear at the base of a building.

Bearing Wall: When a wall carries floor or roof loads, this load-carrying wall is defined as a bearing wall. If only supporting itself, it is termed a nonbearing wall.

Bond Beam: A horizontal course of U-shaped (lintel) masonry with steel reinforcement embedded in concrete core fill to provide structural integrity to a masonry wall.

Braced Frame: An essentially vertical truss system having bracing to resist lateral forces and in which the members are subjected primarily to axial stresses.

Creep: Fault movement without recorded earthquakes.

Damping: The decreasing of ground or building earthquake motions due to friction generated within the earth's crust or within a building.

Dead Fault: An inactive fault; a fault that shows no evidence of movement in recent geological time.

Design Acceleration: A specific ground acceleration at a site; used for the earthquake-resistant design of a structure.

Design Earthquake Ground Motion: A specific seismic ground motion at a site; used for the earthquake-resistant design of a structure.

Design Spectra: A set of response spectrum (acceleration, velocity and/or displacement) used for design.

Diaphragm, Horizontal: The wood sheathing, concrete slab or fill, or metal deck at a roof or floor capable of transferring earthquake forces to vertical lateral force-resisting elements (e.g., shear walls, braced frames, or moment frames).

Displacement: Change in position relative to former position, such as that resulting from earthquake ground motion or relative movement of two sides of a fault.

Diving Plates: See subduction zone.

Drift (Story Drift): Relative movement between one floor and the floor or roof above it.

Ductile Detailing: Special requirements (usually in building codes) needed so that an element remains ductile. In concrete and masonry, for example, closely spaced hoops around longitudinal reinforcement confine the concrete core so that it can still resist forces after being severely cracked.

Ductile Element: A (structural) element capable of sustaining large cyclic deformations and stresses (e.g., beyond the yield point) without any significant loss of strength.

Earthquake: The shaking caused by an abrupt release of elastic energy stored in rocks within the earth.

Elastic: A mode of structural behavior in which a structure displaced by a force will return to its original state upon release of the force.

Elastic Design: See Allowable Stress Design.

Epicenter: The point on the earth's surface directly over the focus or hypocenter.

Equivalent Lateral Force Seismic Design Procedure: A simplified method of earthquake design in which a single seismic response coefficient is determined and multiplied by the building mass to determine the design base shear. The seismic response coefficient is based mainly on building characteristics (e.g., use, lateral force-resisting system and natural period) and the design earthquake ground shaking at the site.

Essential Facility: A facility where buildings and equipment are intended to remain operational in the event of extreme environmental loading from flood, wind, snow, or earthquakes.

Fault (see also active and dead faults): A fracture or fracture zone along which there has been displacement of the sides relative to one another.

Fault scarp: The cliff formed by a fault.

Focal Depth: The depth to the focus (hypocenter) of an earthquake below the earth's surface.

Focus: See hypocenter.

Frequency. The number of oscillations (cycles) in a second, expressed in Hertz. The frequency is the inverse of the period of a cyclic event.

Geologic Hazard: Any geologic process (e.g., landslide or liquefaction) that causes damage to the built environment.

Gravity (g): Acceleration due to the earth's gravity (32.2 ft/s^2 [9.81 m/s^2]).

Hypocenter: The point of origin of an earthquake in three spatial dimensions.

Importance Factor: A factor used in building codes to increase, for example, the usual wind or earthquake design forces for important or essential structures, tending to make them more resistant to those phenomena.

Inactive Fault: See dead fault.

Inelastic: A mode of structural behavior in which a structure displaced by a force exhibits permanent unrecoverable deformation upon release of the force.

Intensity: A measure of the observed effects of an earthquake at a specific location (e.g., Modified Mercalli Intensity and European Macroseismic Scale.

Isoseismal Lines: Imaginary lines connecting points on the surface of the earth experiencing the same seismic intensities.

Lateral Force-Resisting System: A structural system for resisting horizontal forces due, for example, to earthquakes or wind (as opposed to the vertical load-resisting system, which provides support against gravity).

Lateral Spread: Landslides that occur on mildly sloping sites due to liquefaction of soil.

Lift Slab Construction: A construction process in which whereby reinforced concrete floor and roof slabs are cast one upon another, then lifted into place.

Liquefaction: A process in which well-sorted, water-saturated sands or silts rotate out of point-to-point contact under earthquake shaking, lose their shear strength, and behave as slurrys.

Load and Resistance Factor Design (LRFD): A method of designing structural members such that computed stresses produced by service design loads multiplied by load factors do not exceed the theoretical nominal member strength multiplied by a strength reduction (resistance) factor. (Also called strength design or ultimate strength design).

Long Period: A relatively long time period (e.g., greater than about 0.5 s for structural design) to complete one oscillation of ground motion or building vibration.

Magma: Molten rock material within the earth.

Magnitude: A quantitative measure of the total energy released by an earthquake independent of the place of observation (commonly designated with “M” as in M6.6). Currently the most commonly used measure is the moment-magnitude (M_w).

Masonry: Brick, stone, tile, or concrete block bonded together with mortar (with reinforcing steel, it is defined as reinforced masonry; without reinforcing steel it is defined as unreinforced masonry [URM]).

Mean Recurrence Interval: The average time between events (e.g., earthquakes of magnitude ≥7 on a given fault).

Moment-Resisting Frame (Moment Frame): A vertical structural frame comprised of beams and columns in which the members and beam-column joints are capable of resisting lateral forces primarily by flexure (also called a rigid frame).

Natural Period: The interval of time required for an oscillating body in free (i.e., unforced) vibration to complete a cycle.

Non-Ductile Elements: Elements lacking ductility or energy absorption capacity due to the lack of ductile detailing—the element is able to maintain its strength only for smaller deflections and/or fewer cycles (by comparison to ductile elements).

Peak Ground Acceleration (PGA): The maximum amplitude of recorded acceleration at ground level during an earthquake.

Period: The interval of time, usually in seconds, required for an oscillating body to complete a cycle. The period is the inverse of the frequency of a cyclic event.

Plasticity: The property of a soil (or other material) which allows it to deform continuously under a constant load and to retain its deformed shape when the load is removed**.**

Pounding: The collision of adjacent buildings during an earthquake due to insufficient lateral clearance.

Powder-driven fastener: A fastener that is shot (propelled) into a concrete or steel base, usually by the explosion of chemicals (e.g., gun powder) in a small cartridge, similar to the process that discharges a firearm.

The end entering the concrete or steel is similar in shape to a wood nail and resists forces via friction between the fastener and the base material. Also known as a power-driven fastener, an explosive-driven fastener, a powder-actuated fastener, or a gas-actuated fastener.

Resonance: an abnormally large response of a system having a natural vibration period to a stimulus of the same frequency.

Response Spectrum: A set of curves calculated from an earthquake accelerogram that plot maximum amplitudes of acceleration, velocity, or displacement of a single-degree-of-freedom oscillator as a function of its period of vibration and damping.

Restraint: Elements (e.g., anchors, braces, bumpers, chains, and cables) that prevent equipment, piping, contents, etc. from excessive movement (e.g., sliding, swinging, and overturning) relative to their supporting structure during an earthquake.

Rigid Frame: See moment-resisting frame.

Seiche: Oscillations of confined bodies of water due to earthquake shaking.

Seismic: Pertaining to or produced by earthquake or earth vibrations.

Seismic Design Category (SDC): A category used in building codes to classify buildings based on their use and the expected seismic acceleration at a site. In the American Society of Civil Engineers Standard SEI/ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, these are designated as Seismic Design Categories A to F (not to be confused with Site Class A to F based on soil type). The building code provisions for a lower Seismic Design Category (e.g., SDC A) are typically less restrictive than those for higher categories (e.g., SDC D). Ordinary buildings at sites with small expected accelerations require little or no seismic design (SDC A and B); facilities at sites with higher expected accelerations require some (SDC C) or full (SDC D and above) seismic design.

Seismic-Design-Load Effects: The actions (axial forces, shears, or bending moments) and deformations induced in a structural system due to a specified criteria (time history, response spectrum, or base shear) of seismic design ground motion.

Seismic-Design Loading: The prescribed criteria (time history, response spectrum, or equivalent static base shear) of seismic ground motion to be used for the design of a structure.

Seismic Hazard: Any physical phenomenon (e.g., ground shaking, ground failure) associated with an earthquake that may produce adverse effects on human activities.

Seismic Risk: The probability that social or economic consequences of earthquakes will equal or exceed specified values at a site, at several sites, or in an area, during a specified exposure time.

Seismic Waves: The vibrations traveling through the body, and along the surface, of the earth, generated by an earthquake.

Seismic Zone: A generally large area within which seismic design requirements for structures are constant.

Seismograph: An instrument for recording the motion on or within the earth as a function of time.

Seismometer: An instrument for measuring ground motion.

Sensitive (Quick) Clay: A clay soil that has a very low strength when disturbed (e.g., by earthquake shaking) and so fails or flows.

Shear Wall: A wall designed to resist lateral (e.g., earthquake) forces parallel to the plane of the wall.

Short Period: A relatively short time period (e.g., less than about 0.5 s for structural design) to complete one oscillation of ground motion or building vibration.

Sinkhole: An underground hole that develops when water-soluble underground rocks (typically limestone) dissolve. Development of a sinkhole is a non-seismic occurrence, but collapse of the overlying soils into the sinkhole may be hastened by an earthquake.

Snubbers: Resilient and strong anchored blocks placed next to equipment to prevent earthquake forces from moving it laterally.

Soft Story: A story of a building significantly less stiff than adjacent stories (some codes define this as a lateral stiffness 70% or less than that in the story above, or 80% of the average stiffness of the three stories above).

Strength Design: See Load and Resistance Factor Design.

Subduction Zone: A region where one of the earth's lithospheric plates descends beneath another plate.

Tectonics: Pertaining to deformation of the earth's crust.

Tilt-Up Construction: Reinforced concrete walls that are cast horizontally, usually on a concrete floor slab, then lifted (or tilted up) into place.

Tsunami: Long period ocean waves, usually generated by large-scale seafloor displacements associated with large earthquakes or major submarine slides.

Ultimate Strength Design: See Load and Resistance Factor Design.

Unreinforced Masonry: Masonry construction (e.g., bricks, concrete blocks) that does not incorporate steel reinforcement.

Velocity: The rate of change in displacement with respect to time resulting from earthquake ground motion.

Vertical Load-Resisting System: The structural system providing support against gravity (as opposed to the lateral force-resisting system, which resists horizontal forces from earthquakes or wind).

Working Stress Design: See Allowable Stress Design.

Yield Point: The stress at which there is a decided increase in the deformation or strain without a corresponding increase in stress. The strain is inelastic resulting in permanent deformation.

APPENDIX B DOCUMENT REVISION HISTORY

The purpose of this appendix is to capture the changes that were made to this document each time it was published. Please note that section numbers refer specifically to those in the version published on the date shown (i.e., the section numbers are not always the same from version to version).

January 2024. Interim revision. Minor editorial changes were made.

July 2023. Interim revision. This revision primarily incorporates technical guidance for the expanded scope of equipment including the following items:

- Raised access floors and supported equipment.
- Transformers and battery installations.
- HVAC and refrigeration equipment on structural floor.

The following changes have been made:

a) New data sheet Section C.5.5 has been added, which shows generic equipment anchorage.

b) OS 1-2 was updated per the latest version of American Society of Civil Engineers (ASCE) standards, ASCE 7-22 and ANSI/RMI MH16.1-2021.

c) Minor editorial changes were made to Sections C.4.3, C.4.3.3, C.6.1, C.6.2 and Table C.4.3.3.

July 2022. Interim revision. The worldwide earthquake zones visual overview shown in Appendix C, Figures C.6.3A and C.6.3B has been changed to correctly reflect the FM Global Worldwide Earthquake Map, the authoritative source that has been available online since January 2021 at https://www.fmglobal.com. There are no changes to the January 2021 earthquake zones, online map, or technical guidance.

January 2021. Interim revision. FM Global earthquake zones have been revised worldwide except for some small islands, and the FM Global Worldwide Earthquake Map, available online, is now the authoritative source.

Editorial revisions have been made to align with these changes, as follows:

A. Revised Section 2.1.

B. Revised Appendix C Sections C.6, C.6.1, C.6.2, and C.6.3.

C. Revised Appendix C Figures C.6.3A and C.6.3B.

D. Deleted Appendix C Table C.6.3.

E. Deleted Appendix C Figures C.6.3C to C.6.3I.

October 2018. Interim revision. Revised FM Global earthquake zones in China. The change is documented in Table C.6.3 and Figures C.6.3B and C.6.3G. Made editorial changes in Recommendations 2.2.2.1 and 2.2.2.1.1.

April 2018. Made editorial and technical revisions throughout the data sheet. The most significant editorial and technical revisions include the following:

A. Revised Section 1.0, Scope and added Section 1.2, Hazards. Renumbered all figures and tables, and renumbered or consolidated some recommendations.

B. Provided new guidance (2.2.1.3) for sites subject to liquefaction.

C. Added guidance (2.2.2.1.1) enumerating equipment that should be restrained for earthquake forces.

D. Consolidated and clarified guidance (2.2.2.1.2) regarding use of the ASCE 7 design standard.

E. Revised and added guidance for anchorage to concrete using post-installed anchors (2.2.2.1.3), cast-in-place anchors and inserts (2.2.2.1.4) and powder-driven fasteners (2.2.2.1.5).

F. Revised guidance for fire protection tanks (2.2.2.2). Added guidance for other tanks (2.3.1 and 2.3.3), piping (2.3.2) and portable liquid containers (2.4.4).

G. Revised earthquake emergency response requirements (2.7.1) and moved details previously in Appendix C (in Section C.6) to Data Sheet 10-2, *Emergency Response*.

H. Revised Section 4 references and Appendix A definitions.

I. Made revisions throughout Appendix C, Section C.1 (earthquakes and seismicity), to clarify the information. Included are extensive revisions to Section C.1.6 regarding earthquake intensity (including Table C.1.6A and Table C.1.6B for Modified Mercalli Intensity).

J. Made revisions throughout Appendix C, Section C.4 (earthquake performance of buildings), to clarify the information. Included are construction cost of seismic design (Section C.4.1 and Table C.4.1), strengthening existing buildings (Section C.4.3.2 and Section C.4.3.3), and comparing the generic earthquake building types from this data sheet with those used in FEMA 154 (Table C.4.3.3).

K. Made revisions throughout Appendix C, Section C.5 (earthquake performance and restraint of contents), to clarify the information. Included are added information on past earthquake performance of contents (Section C.5.1); and extensive revisions regarding equipment restraint (Section C.5.2), storage rack restraint (Section C.5.3) and anchorage to concrete (Section C.5.4 including Table C.5.4.2).

L. Revised earthquake zones for miscellaneous worldwide islands, including: Antipodes Island, Ascension Island, Auckland Island, Black Rock, Bounty Island, Bouvet Island, British Indian Ocean Territory, Campbell Island, Christmas Island, Clipperton Island, Coral Sea Islands, Europa Island, Faroe Islands, Gough Island, Jan Mayen, Johnston Atoll, Macquarie Island, Midway Islands, Nightingale Island, Norfolk Island, Paracel Islands, Saint Helena, South Georgia Islands, South Orkney Islands, South Sandwich Islands, Spratly Islands, Tristan da Cunha and Tromelin Island; also the southernmost islands of the Maldives and some Indonesian islands (those near North Maluku and West Papua, and the Sangihe Islands). The changes have been documented in Table C.6.3 and in Figure C.6.3A through Figure C.6.3I.

April 2016. Interim Revision. Revised FM Global earthquake zones in the continental United States (i.e., the United States excluding Alaska and Hawaii).

July 2012. Made the following changes related to earthquake zones for the continental United States and for worldwide islands/island groups:

- Revised FM Global earthquake zones in the continental United States (i.e., the United States except for Alaska and Hawaii).
- Revised FM Global earthquake zones for thirty-nine worldwide islands/island groups including: American Samoa, Azores, Baker Island, Bermuda, Canary Islands, Cape Verde, Cocos (Keeling) Islands, Cook Islands, Easter Island, Fiji, French Polynesia, Galapagos Islands, Greenland, Howland Island, Iceland,

Jarvis Island, Kiribati, Madeira, Maldives, Malta, Marshall Islands, Mauritius, Federated States of Micronesia, Nauru, New Caledonia, Niue, Palau, Palmyra Atoll, Phoenix Islands, Pitcairn Islands, Réunion, Samoa, Seychelles, Tokelau, Tonga, Tuvalu, Vanuatu, Viringili Island, Wallis and Futuna.

- Confirmed that no changes to the previous FM Global earthquake zones are needed for the island/island groups of Guam and Northern Mariana Islands, which remain 50-year zones; and for the Lakshadweep Islands, which remain a >500-year zone.
- Replaced forty-two maps (Figure 2 through Figure 8D) with nine completely revised maps, including: Figure 2 (World - Western Hemisphere), Figure 3 (World - Eastern Hemisphere), Figure 4 (North America and Caribbean), Figure 5 (South America and Caribbean), Figure 6 (Europe), Figure 7 (Europe, Africa and Middle East), Figure 8 (Middle East and Asia), Figure 9 (Australia and Surrounding Area), and Figure 10 (Pacific Ocean Islands).
- Modified Table 3 to document the dates that earthquake zones were revised or confirmed in the countries/regions noted above. Also removed the Table 3 column showing on which map(s) each country/region can be found.

Made minor changes and clarifications in Section 2.1, Section 2.2.2.1.3, Section 4.2, Section C.3.3, Section C.7.1, Section C.7.2 and Section C.7.3.

July 2011. Made the following changes related to earthquake zones in Central Asia and Africa:

- Revised FM Global earthquake zones in the Central Asian countries of Bangladesh, Bhutan, China, India, Kazakhstan, Kyrgyzstan, Mongolia, Nepal, North Korea, Pakistan, Russia (east of 50° longitude), South Korea, Tajikistan, Turkmenistan, and Uzbekistan; and in the African countries of Algeria, Burundi, Democratic Republic of the Congo, Djibouti, Eritrea, Ethiopia, Kenya, Libya, Malawi, Morocco and Western Sahara (including Ceuta and Melilla), Mozambique, Rwanda, Somalia, Sudan, Tanzania, Tunisia, Uganda, Zambia, and Zimbabwe.

  The changes have been documented in Table 3 and in maps showing the rezoned areas including Figure 4 (Africa); Figure 5 (Europe); Figure 5A (Eastern Europe); Figure 5A, Part 3 (Middle East); Figure 6 (Asia); Figure 6, Part 2 (West Asia); Figure 6, Part 3 (Central Asia); Figure 6, Part 4 (East Asia); Figure 7 (Western China and Mongolia); Figure 7, Part 2 (Eastern China and Mongolia); and Figure 8 (Oceania).
- Confirmed, and documented in Table 3, that no change to the previous FM Global earthquake zones is needed for the Central Asian country of Sri Lanka; and for the African countries of Angola, Benin, Botswana, Burkina Faso, Cameroon, Central Africa Republic, Chad, Comoros (including Mayotte), Republic of the Congo, Cote d'Ivoire (Ivory Coast), Equatorial Guinea, Gabon, The Gambia, Ghana, Guinea, Guinea-Bissau, Lesotho, Liberia, Madagascar, Mali, Mauritania, Namibia, Niger, Nigeria, Sao Tome and Principe, Senegal, Sierra Leone, South Africa, Swaziland and Togo.

April 2011. For the following countries in Europe, the Middle East, Asia and Africa, and near Australia:

- Revised FM Global earthquake zones in Afghanistan, Armenia, Azerbaijan, Bahrain, Belarus, Brunei, Burma, Cambodia, Egypt, Georgia, Indonesia, Iran, Iraq, Kuwait, Laos, Malaysia, Moldova, Oman, Papua-New Guinea, Philippines, Russia (west of 50°E longitude), Saudi Arabia, Singapore, Solomon Islands, Thailand, Ukraine, United Arab Emirates (UAE), Vietnam and Yemen. Revised Table 3 and maps showing the rezoned area including Figure 4 (Africa); Figure 5 (Europe); Figure 5A (Eastern Europe); Figure 5A, Part 2 (East Europe); Figure 5A, Part 3 (Middle East); Figure 6 (Asia); Figure 6, Part 2 (West Asia); Figure 6, Part 3 (Central Asia); Figure 6, Part 4 (East Asia); Figure 8 (Oceania); and Figure 8A (Australia) to reflect the change.
- Confirmed that no change to the previous FM Global earthquake zones is needed for Estonia, Finland, Latvia, Lithuania, Qatar and Timor-Leste, and revised Table 3 to reflect this.

April 2010. The following changes were made:

- Revised FM Global earthquake zones for Australia. Revised the maps showing the rezoned area, including Figure 6, Part 1 (Asia); Figure 8 (Oceania); and Figure 8A, Part 1 (Australia). Added Figures 8A, Parts 2, 3, and 4 (Australia).
- Revised Table 3 entries for Australia to reflect earthquake zone changes.
- Revised Table 3 entries for Japan to reflect confirmation of current earthquake zones.

- Modified Section 2.2.2.1.3 regarding post-installed concrete anchors.
- Modified Section 2.2.2.2 regarding suction tanks.
- Updated Section 4.0, References.
- Updated Appendix A, Glossary of Terms
- Revised Section C.3.3 regarding equipment/nonstructural component anchorage and post-installed concrete anchors.
- Revised terminology in Sections C.7.1 through C.7.3.

January 2009. Revised FM Global Earthquake Zones for Taiwan. Revised maps showing the rezoned area including: Figure 6 (Asia), Figure 6-part 4 (East Asia), Figure 8 (Oceania) and Figure 8C (Taiwan). Revised Table 3 to reflect the above change.

May 2008. Revised FM Global Earthquake Zones for the following countries/territories in Asia and Oceania: Bangladesh, Bhutan, China, India, Kazakhstan, Kyrgyzstan, Mongolia, Nepal, New Zealand, North Korea, Pakistan, Russia (east of 60°E longitude), South Korea, Tajikistan, Turkmenistan and Uzbekistan. Confirmed that no change to the previous FM Global Earthquake Zones is needed for Sri Lanka.

Revised maps showing, or overlapping into, the rezoned countries/territories identified above including: Figure 2 (North America), Figure 5A Part 3 (Middle East), Figure 6 Part 1 (Asia), Figure 6 Part 2 (West Asia), Figure 7 Parts 1 and 2 (China and Mongolia), Figure 8 (Oceania) and Figure 8B (New Zealand). The mapped area in Figure 6 Parts 1 and 2 has been revised from the previous version of this data sheet.

Added new maps: Figure 6 Part 3 (Central Asia) and Figure 6 Part 4 (East Asia). Deleted maps: Figure 7 Parts 3 and 4 (Eastern China).

Revised Table 3 to reflect the above changes.

July 2007. Revised FM Global Earthquake Zones for the following countries/territories in South America and the Caribbean: Argentina, Bolivia, Brazil, Chile, Colombia, Ecuador, Falkland Islands (Islas Malvinas), Guyana, Paraguay, Peru and Venezuela (all in South America); and Anguilla, Aruba, Bahamas, Cayman Islands, Cuba, Dominican Republic, Haiti, Navassa Island, Netherlands Antilles, Puerto Rico, St. Kitts & Nevis, St. Lucia, St. Vincent & the Grenadines, Trinidad & Tobago, Turks & Caicos Islands, and the Virgin Islands (all in the Caribbean).

Confirmed that no change to the previous FM Global Earthquake Zones is needed for the following countries/territories in South America and the Caribbean: French Guiana, Suriname and Uruguay (all in South America); and Antigua & Barbuda, Barbados, Dominica, Grenada, Guadeloupe, Jamaica, Martinique, and Montserrat (all in the Caribbean).

Revised maps showing, or overlapping into, the rezoned countries identified above including: Figure 2 (North America), Figure 3 (South America), Figure 3A (Central America), Figure 3B (Northern South America and the Caribbean) and Figure 3C (Southern South America). The mapped area in Figures 3B and 3C has been revised from previous version of DS/OS 1-2.

Revised Table 3 to reflect the above changes.

May 2007. Revised FM Global Earthquake Zones for the following countries/territories in Europe and Asia/Middle East: Bulgaria, Cyprus, Israel (includes Gaza Strip, Golan Heights and West Bank), Jordan, Lebanon, Romania, Syria and Turkey. Confirmed the FM Global Earthquake Zone (>500-year) determined previously for the European countries of Finland, Norway and Sweden. Revised FM Global Earthquake Zones for the following countries in Central America: Belize, Guatemala, Honduras, Nicaragua and Panama. Confirmed the FM Global Earthquake Zone (50-year) determined previously for the Central American countries of Costa Rica and El Salvador. Revised maps showing, or overlapping into, the rezoned countries identified above including: Figure 3 (South America), Figure 3A (Central America), Figure 4 (Africa), Figure 5A (Eastern Europe), Figure 5A – part 2 (East Europe), Figure 5A – part 3 (Middle East) and Figure 6 (Asia).

January 2007. The following changes were made for this revision:

- Section 2.2.2.1 – Design requirements have been made more specific and seismic parameters have been defined that correspond to FM Global earthquake zones. These requirements are the basis for building and equipment anchorage seismic design.

- Section C3.3 of Appendix C – Technical details used in the design of equipment and storage-rack anchorage have been added.
- Table 3 - corrected the effective dates of FM Zones for Denmark, Ireland, Luxembourg and United Kingdom.

May 2006. Revised FM Global Earthquake Zones for the following countries/territories in Europe: Albania, Andorra, Austria, Belgium, Bosnia and Herzegovina, Croatia, Czech Republic, Denmark, France, Germany, Gibralter, Greece, Guernsey, Hungary, Ireland, Italy, Jersey, Liechtenstein, Luxembourg, Macedonia, Man (Isle of), Monaco, Netherlands, Poland, Portugal, San Marino, Serbia and Montenegro, Slovakia, Slovenia, Spain, Switzerland, United Kingdom and Vatican City. Updated and changed the mapped area shown in Figure 5, Figure 5B, Figure 5C, Figure 5D, Figure 5E and Figure 5F. Revised Figure 5A and Figure 5A - part 2. Deleted Figure 5D - part 2. Added Figure 5G. Revised Figure 4 (Africa) and Figure 6 (Asia) where they show the countries/territories identified above. Revised Table 3 and Section C.7.2.

July 2005. Updated maps for Mexico (including Figures 2, 2D, 3 and 3A), Table 3 and section C.7.2.

March 2005. Updated maps for Canada (Figures 2 and 2B [Parts 1 to 4]), maps for Australia (Figures 8 [Part 1] and 8A) and Table 3. Made editorial changes.

November 2004. Updated with editorial changes.

September 2004. The previous FM Global zones used to define relative earthquake hazards worldwide have been replaced by FM Global zones based on recurrence intervals of ground shaking. Legends for all maps have been revised to reflect the new earthquake zone designations. The map color scheme for portraying the new zones has also been revised. For most areas, earthquake zones have not been reevaluated. In these areas, earthquake zone boundaries shown on the maps remain unchanged, but old earthquake zones have been converted to new zones based on an approximate correlation (see section C.7).

Earthquake zones in the continental United States, Alaska, Hawaii, and the area around Vancouver, Canada have been reevaluated using a revised methodology and the latest seismicity information available. For these areas only, earthquake zone boundaries shown on the maps differ from those in the previous version of this data sheet.

In addition to updating the maps, the text throughout the document has been revised to reflect the new zones and has also been extensively updated. Sections have been renumbered. Some information previously missing from Appendix B has been added.

September 2001. Some of the earthquake zones maps were clarified by completing zone hatching where missing and adding cross-references. Figure 9 and 9A were renumbered as Figures 8 (part 2) and 8D, respectively. Figure references in C.14.2 were changed to agree with the revised figure numbers of the redrawn maps for May 2001 revision.

May 2001. All earthquake zones maps were redrawn for improved resolution and mapping accuracy.

January 2001. Revision of earthquake zones for New Zealand and Southeast Canada were added. The zoning note in Figure 8 was revised.

September 2000. The document was reorganized and revisions of earthquake zones for Andorra, Canada, France, Germany, Monaco, Portugal, Spain and Switzerland were added.

May 2000. Revisions of earthquake zones for the United States, Alaska and Hawaii were added.

1999. Revisions of earthquake zones for Taiwan, Mexico, Venezuela and Colombia were added.

1996. Earthquake zones were revised for much of Europe, Eurasia, and the Middle East, including Albania, Belarus, Bosnia and Herzegovina, Bulgaria, Croatia, Czech Republic, Denmark, Estonia, Finland, Georgia, Germany, Greece, Hungary, Italy, Latvia, Lithuania, Luxembourg, Macedonia, Moldova, Norway, Poland, Romania, San Marino, Serbia and Montenegro (Yugoslavia), Slovakia, Slovenia, Sweden, Ukraine, Vatican City (Holy See), Russia, Turkey, Armenia, Azerbaijan, and Cyprus.

1992. Australia earthquake zones were revised.

February 1987. The following changes were made:

1. A glossary of earthquake related terms was added.
2. Earthquake zones where recommendations apply were added.

APPENDIX C SUPPLEMENTAL INFORMATION

C.1 Earthquakes and Seismicity

C.1.1 General

Earthquakes are essentially oscillating ground movements, having horizontal and vertical components, caused by sudden displacements along faults of adjacent strained rock masses.

The majority of earthquakes are best understood in the context of global plate tectonics. Plate tectonics explains that the outer layer of the earth (the lithosphere) consists of several large and many smaller plates that are drifting over the surface of the earth. The propelling force is convection in the mantle, or interior, of the earth as it slowly cools.

The drifting plates collide with each other. Where collisions occur, two plates may grind laterally against each other (as in the case of the San Andreas fault), or one plate may dive down under the other in a subduction zone (as in the Pacific Northwest of the U.S.). About 90% of the world's earthquakes occur at the boundaries of adjacent plates.

Shallow-focus (less than about 20 miles [32.0 km]) earthquakes generally occur where plates are sliding past each other.

Intermediate and deep-focus earthquakes generally occur in subduction zones. The subducting plate compresses and thrusts upward the overriding plate, forming mountains. Offshore, the subduction zone forms a deep trench. As the subducting plate descends, it becomes heated. Water and lower melting point minerals become fractionated and rise to the surface within the overriding plate, producing volcanoes and also large masses of granite rock intrusions.

Most earthquakes occur at plate boundaries. Some earthquakes occur in plate interiors ("intraplate earthquakes"). Locations of some intraplate earthquakes within the North American plate are Charleston, South Carolina; New Madrid, Missouri; and along the St. Lawrence Valley. These earthquakes occur on ancient faults, driven by stresses transferred from the plate boundaries.

C.1.2 Faults

Nearly all large earthquakes are associated with faults. A fault is a fracture zone along which the two sides are displaced relative to each other. Most faults are readily recognized by trained geologists or by seismological methods. Not all faults are active, and there is currently no method of accurately predicting the future activity of a given fault; however, faults associated with a plate boundary can be expected to have future activity.

Displacement along a fault may be vertical, horizontal, or a combination of both. Movement may occur very suddenly along a stressed fault, producing an earthquake, or it may be very slow, the rock undergoing what is called "creep," unaccompanied by seismographic evidence. Permanent displacement of the terrain during an earthquake might be several inches (centimeters), or it might be tens of yards (meters). The Assam, India Earthquake of 1897 produced 35 ft (10.7 m) of vertical displacement; the 1906 San Francisco earthquake produced 21 ft (6.4 m) of horizontal displacement.

The total area of fault rupture varies with magnitude. Likewise, if the rupture reaches the earth's surface, the surface rupture length varies with magnitude. A magnitude (M) 6 earthquake may produce five miles (eight km) of surface rupture, whereas an M8.8 earthquake may produce 1000 miles (1600 km) of surface rupture.

Faults are frequently described by the way one side moved relative to the other. When there has been no lateral movement, the fault is "normal," or "reverse," according to whether the overlying side slipped down or was thrust upward. If there has been lateral movement, the fault is termed a "strike-slip" fault. The fault is "left lateral" if the opposite side moved to the left when the viewer faces the fault. Combinations of normal or reverse and lateral faults are possible (see Figure C.1.2).

The width of the fault zone varies widely. Major strike-slip systems, such as the San Andreas system, may be tens of miles (km) wide. Individual faults may be several hundred yards (meters) wide.

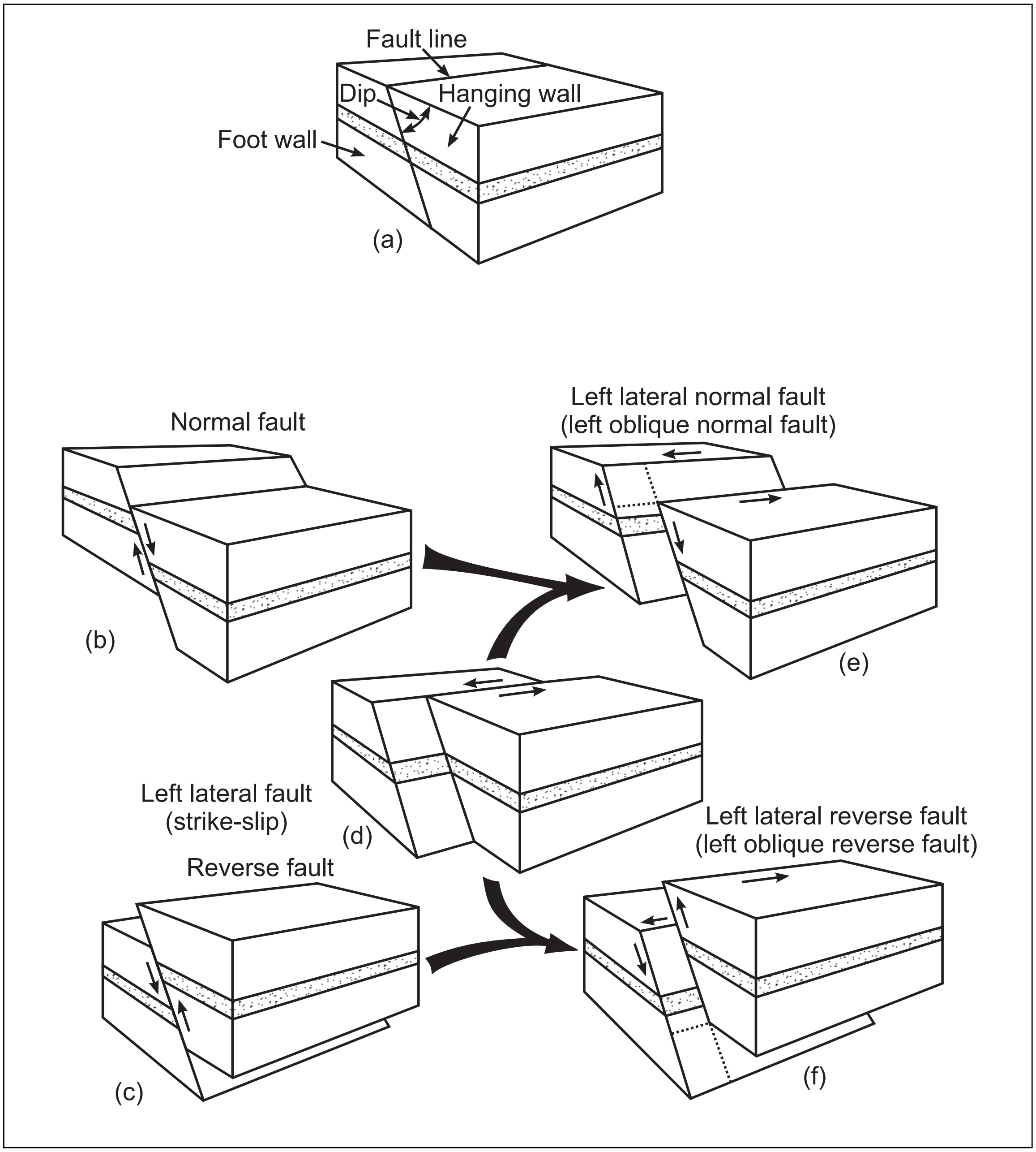

Fig. C.1.2. Types of fault movement

(a) Names of some of the components of faults. (b) Normal fault, in which the hanging wall has moved down relative to the foot wall. (c) Reverse fault, sometimes called thrust fault, in which the hanging wall has moved up relative to the foot wall. (d) Lateral fault, sometimes called strike-slip fault, in which the rocks on either side of the fault have moved sideways past each other. It is called left lateral if the rocks on the other side of the fault have moved to the left, as observed while facing the fault, and right lateral if the rocks on the other side of the fault have moved to the right, as observed while facing the fault. (e) Left lateral normal fault, sometimes called a left oblique normal fault. Movement of this type of fault is a combination of normal faulting and left lateral faulting. (f) Left lateral reverse fault, sometimes called a left oblique reverse fault. Movement of this type is a combination of left lateral faulting and reverse faulting. Two types of faults not shown are similar to those shown in (e) and (f). They are a right lateral normal fault and a right lateral reverse fault (a right oblique normal fault and a right oblique reverse fault, respectively).

C.1.3 Seismic Waves

When a fault ruptures, seismic waves are propagated in all directions, causing the ground to vibrate; much of the energy is at frequencies ranging from about 0.1 to 30 Hertz. There are two basic wave types: body waves that travel within the earth, and surface waves that travel along the earth's surface.

There are two types of body waves: P and S waves, known as compression and shear waves, respectively. These waves cause high-frequency (greater than 1 Hertz) vibrations, which are more effective than low-frequency waves in causing low-rise buildings to vibrate. Surface waves are produced by body waves interacting with the surface. Surface waves may be subdivided into Love (L) waves and Rayleigh (R) waves. These waves mainly cause low frequency vibrations, which are more effective than high frequency waves in causing tall buildings to vibrate. Because of the long wavelengths, decay of the amplitudes of low frequency vibrations require a longer distance to occur than those of high frequency vibrations; therefore, tall buildings located at a relatively great distance from a fault, e.g., 250 miles (400 km), may be damaged.

The velocity of seismic waves varies with the density and elastic properties of the material through which it travels. P waves are fastest, followed by S waves, then L waves and then R waves. The hypocenter can be determined by measuring the time interval between arrivals of the P and S waves on seismographs, or by inverting the arrival times of a single wave type.

C.1.4 Ground Motion

The ground shaking in an earthquake depends mainly on the strength of the earthquake (magnitude and related duration), the distance from the fault, and local geologic conditions. The important parameters of ground vibration due to earthquakes include acceleration, velocity, displacement, and frequency or period. Acceleration and velocity can be measured directly with seismometers. Displacement can be measured geodetically, or can be obtained from integrating velocity or acceleration records. A very useful engineering tool, the response spectrum, can then be derived by plotting spectral values of acceleration, velocity, and displacement against varying periods to determine maximum peaks at differing levels of damping for a given natural period of vibration.

Ground acceleration during earthquakes can range from imperceptible to more than two times gravity (g), depending on the size of the earthquake and proximity to the fault.

Most strong earthquakes will produce accelerations from 10% to 20% of g at considerable distances away from the fault. Acceleration near the fault can be substantial, from 30% to 50% g or more, but more than 50% g is infrequent. Damage to individual items will vary, depending on inherent strength, natural frequency, and built-in damping capability.

Vertical acceleration in most earthquakes is from one-third to two-thirds the horizontal acceleration, and usually is of higher frequency, or about double the number of horizontal pulses.

Occasional sharp peaks of acceleration can result in a high reported peak ground acceleration, but might be too short a duration to be damaging. A true measure of damage potential is the area under the accelerograph curve, which is an indicator of maximum ground velocities. Sustained high acceleration will produce high velocities, perhaps in excess of 3 ft (1 m) per second. Integrating the acceleration record with respect to time will give the record of velocity. Peaks of the latter curve will give maximum velocities.

The displacement during an earthquake will vary considerably. Maximum displacement in strong earthquakes can range from inches (centimeters) to about 50 yards (meters).

Earthquake waves decay, or attenuate, with each vibration cycle. Short-period waves require many cycles to travel a short distance, and so attenuate quickly as a function of distance. However, long-period energy does not necessarily quickly attenuate, and acceleration impulses of a long period, e.g., from 1 to 3 seconds, will be sustained, and potentially damaging, quite some distance from the epicenter. These long-period accelerations can produce serious damage to any building having a correspondingly long natural period (that is, the building resonant period coincides closely with that of the long period wave). For example, the 1952 Kern County, California earthquake damaged tall (taller than 5 story) steel and concrete frame buildings in downtown Los Angeles some 75 miles (121 km) from the epicenter.

Local geologic conditions affect the strength and frequency content of shaking. Generally, soft ground shakes more strongly than firm soil or bedrock, and thicker sediment layers amplify shaking more than thinner layers. An extreme example of this exists in Mexico City, Mexico, where unconsolidated lakebed sediments amplified motions from a distant (about 250 miles [400 km] away) M8.1 earthquake in 1985. Several hundred buildings collapsed and several thousand buildings were damaged in Mexico City. A large percentage of the buildings damaged in Mexico City were between 8 and 18 stories high, indicating a resonance effect, with the long period horizontal ground accelerations exciting motions in the lakebed sediments, which in turn resonate with the buildings.

Depth of the earthquake also affects surface ground motion in the same way as distance from the fault. Where soils are homogeneous, ground motions attenuate (decrease) as the distance from the earthquake hypocenter increases. The difference between the distance to the epicenter and the distance to the hypocenter is not as great for a shallow earthquake as it is for a very deep earthquake. The deeper the energy release, the less energy is available per unit surface area. The 1949 and 1965 Puget Sound, Washington earthquakes (magnitudes 7.1 and 6.5) had focal depths of about 35 miles (56 km) and produced maximum Modified Mercalli Intensity (MMI) levels of VIII and VII, respectively. The 1971 San Fernando, California, Earthquake (M6.6) had a focal depth of about 8 miles (13 km), and produced a maximum MMI of XI.

C.1.5 Earthquake Measurement–Magnitude

Historically, the most common quantitative method of measuring earthquakes was the Richter magnitude scale (also known as the local magnitude [M_L]). In 1935 Dr. Charles Richter at Caltech developed the scale, and he explained it in 1958 as follows: magnitude is intended to be a rating of a given earthquake, independent of the place of observation. Because it is calculated from measurements on seismograms, it is properly expressed in ordinary numbers and decimals. Richter magnitude was originally defined as the logarithm of the maximum amplitude, corrected to a distance of 100 kilometers (62 miles) from the epicenter, on a seismogram written by an instrument of specified standard type. Because the scale is logarithmic, every upward step of one magnitude unit means the recorded amplitude is a factor of 10 larger.

The magnitude also can be related to the earthquake's energy. A 1-unit increase in magnitude corresponds roughly to a 30-fold increase in energy. The magnitude scale necessarily relates only with energy radiated as seismic waves. Additional energy is converted into heat by friction along the fault, or moving material against gravity.

Several kinds of instrumentally-derived magnitude scales have been used by earth scientists (e.g., surface-wave magnitude [M_S], body-wave magnitude [m_b] and moment magnitude [M_w]). M_L, M_S and m_b each use different seismic wave characteristics to measure magnitude, and each has an upper earthquake magnitude beyond which it cannot measure (i.e., saturates). M_L and m_b saturate at about magnitude 6.2 to 6.5, and M_S saturates at about magnitude 7.5. Because they saturate, M_L, M_S and m_b will typically underestimate the magnitude of large earthquakes.

Currently earthquake magnitudes are commonly given in terms of moment magnitude (M_w). This scale is derived from the seismic moment (which is based on the area of fault rupture, the average amount of slip, and the force that was required to overcome the strength of the fault). Moment magnitude is uniformly applicable to all sizes of earthquakes and is the most reliable magnitude scale. For large earthquakes, the reported M_w is the most accurate. Although magnitude scales are open-ended, the upper magnitude of earthquakes is limited by the strength of crustal rocks. The largest instrumentally recorded earthquake to date is the 1960 Chile earthquake of M_w9.6.

The number of earthquakes decreases rapidly as the magnitude increases (see Table C.1.5). Richter found that, for the world at large, the frequency of shocks at any given magnitude level was roughly 8-10 times that of about one magnitude higher. A later worldwide study by J.P. Rothe covering the period 1953-1965 found this frequency to be roughly 13-22 times, while 2012 USGS data shows it to be roughly 9-15 times (based on catalogs of M7+ earthquakes since 1900, and of M5 to M6.9 earthquakes since 1990).

Table C.1.5. Average Number of Earthquakes per Year

	Average Number of Earthquakes Per Year		
Magnitude	Richter Study	Rothe Study	USGS Data
6.0-6.9	150	195	134
7.0-7.9	18	15.5	15
8.0 and over	2+	0.7	1

Earthquakes of M6.0 or greater generate ground motions sufficiently severe to be damaging to well-built structures, whereas the threshold for damaging poorly-built structures (e.g., unreinforced masonry walls) may be as low as M5.0.

C.1.6 Earthquake Measurement–Intensity

Another term commonly used to describe the size of an earthquake is intensity. Intensity and magnitude are often confused, and it is important to understand the difference.

The severity of ground shaking at a specific location is the intensity. It is often categorized by assigning intensity values, usually expressed in Roman numerals (e.g., I to XII), based on the observed strength of shaking and resulting damage to the built environment. Intensity of shaking can also be defined in terms of peak ground acceleration (PGA), peak ground velocity (PGV), or other instrumentally-determined measures of ground motion.

The earthquake intensity scale presently used in the United States is a 12-level scale (I to XII, from least to most severe) developed first by G. Mercalli in 1902 and modified at Caltech by H. Wood and F. Neumann in 1931 and Richter in 1956. It is known today as the Modified Mercalli Intensity (MMI) scale. This scale is based on the observed effects of earthquakes: the perceptions of persons in the quake-stricken area determined through interviews, the effect on the built environment determined by damage surveys, and geologic observations. Because intensity has historically been determined by observers reviewing earthquake effects, the MMI scale (and other intensity scales) may be subjective and relative. However, use of nontechnical intensity values continues to be useful since they convert complex ground motions into a few groupings that convey earthquake damage in a straightforward way.

Although there can be a fair amount of PGA/PGV scatter within areas reported to have the same MMI, rough conversions of PGA/PGV to MMI have been developed (e.g., Trifunac and Brady 1975; Wald, et al. 1999; and Worden, et al. 2012 - see Section 4.2 references). Following significant earthquakes such conversions are useful to quickly develop regional shaking intensity (e.g., MMI) maps from a limited set of recorded ground motions (e.g., PGA and PGV). These shaking intensity maps allow post-earthquake response to focus on areas most likely to have significant damage.

In Table C.1.6A, descriptions of shaking and potential damage for each MMI level (taken from the United States Geological Survey [USGS] publication, *The Severity of an Earthquake*) are given. The “Definition” column in the table is from the European Seismological Commission report, *European Macroseismic Scale 1998 (EMS-98)*. The approximations of PGA and PGV in the table are based on Worden, et al. 2012, “Probabilistic Relationships between Ground-Motion Parameters and Modified Mercalli Intensity in California.” The Worden, et al. 2012, PGA and PGV vs. MMI relationships are also used to develop USGS ShakeMaps, which provide near-real-time maps of shaking intensity following significant earthquakes.

Other intensity scales are, or have been, in common usage throughout the world. Several are 12-level scales (again from I to XII) that are reasonably equivalent to the MMI scale. These include: the 1964 Medvedev-Sponheuer-Karnik scale (MSK or MSK-64 scale) that is used, for example, in India and Russia, and formerly in Europe; the 1998 European Macroseismic Scale (EMS-98); and the China Seismic Intensity Scale (CSIS) or China Liedu scale. The Japan Meteorological Agency (JMA) Seismic Intensity Scale or JMA Shindo scale has 10 basic levels (0, 1, 2, 3, 4, 5 lower [5L], 5 upper [5U], 6 lower [6L], 6 upper [6U] and 7). The intensity using the JMA Shindo scale is reported using Arabic numbers to one decimal point. A rough correlation between the MMI and JMA Shindo scales is shown in Table C.1.6B.

Despite their many shortcomings, intensity scales are an important consideration in areas where no seismographs have been installed, and they afford the only means for interpreting historical information. Additionally, expected damage levels can be quickly identified based on post-earthquake isoseismic intensity maps prepared and issued by various agencies (such as the USGS as described above). Damage to unreinforced masonry structures, unbraced fire protection sprinkler piping and similar systems built with little or no consideration for earthquake resistance begins at about MMI of VI (JMA Shindo 5L [4.5-4.9]) and becomes significant at an intensity of VII (JMA Shindo 5U [5.0-5.4]). As described in Data Sheet 1-11, *Fire Following Earthquake*, seismic shutoffs compliant with the 2006 edition of ASCE 25, *Earthquake-Actuated Automatic Gas Shutoff Devices*, are expected to actuate at about MMI of VII (JMA Shindo 5U [5.0-5.4]). Likewise, in the 1994 Northridge, California, USA earthquake (M6.7), most gas fires occurred at MMI VII (JMA Shindo 5U [5.0-5.4]) or higher according to ATC 74, *Collaborative Recommended Requirements for Automatic Natural Gas Shutoff Valves in Italy*. Generally speaking, MMI of VIII (JMA Shindo 6L [5.5-5.9]) is the threshold of serious damage to well-built structures.

Magnitude and maximum intensity of an earthquake are interdependent to some degree, but there is no direct correlation between them. For example, an earthquake might have relatively low magnitude but, because of shallow focus or poor soil condition, it might cause a great deal of damage (i.e., have a high intensity) at certain locations. A larger earthquake with a deeper focus or that shakes a region of firm soil or rock may generate lower intensities.

While an earthquake can have only one magnitude, the intensity is a function of location. The intensity is typically highest near the epicenter and, if geologic conditions are uniform, it gradually decreases as distance from the epicenter increases. However, intensity may vary considerably at two points that are equidistant from the epicenter because it is so dependent on the particular ground (i.e., local soil conditions). For this reason, it is difficult to equate magnitude with estimated intensity.

In order to develop shaking intensity maps for earthquakes that may occur in the future, crude correlations have been developed for the relationship of an earthquake's magnitude with the area enclosed by each MMI isoseismal. These correlations can be complex and depend on variables such as the earthquake magnitude, the distance to the earthquake hypocenter, the type of faulting (e.g., subduction vs. crustal), the subsurface bedrock energy absorption (i.e., attenuation) characteristics, and the surface soil type.

Table C.1.6A. Modified Mercalli Intensity (MMI) Scale

MMI	*Definition*[1]	*MMI Description*[2]	*Approximate Range of Ground Motions*[3]	
			PGA, %g	PGV, in/sec(cm/s)
I	Not felt	Not felt except by a very few under especially favorable conditions.	<1.3	<0.25 (<0.6)
II	Scarcely felt	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.		
III	Weak	Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration similar to the passing of a truck. Duration estimated.		
IV	Largely observed	Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.	1.3-4.5	0.25-1.2 (0.6-3.1)
V	Strong	Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.	4.5-8.5	1.2-2.6 (3.1-6.7)
VI	Slightly damaging	Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.	8.5-16	2.6-5.5 (6.7-14)
VII	Damaging	Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.	16-29	5.5-11 (14-29)
VIII	Heavily damaging	Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.	29-55	11-24 (29-60)
IX	Destructive	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.	55-102	24-49 (60-124)
X	Very destructive	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.	>102	>49 (>124)
XI	Devastating	Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.		
XII	Completely devastating	Damage total. Lines of sight and level are distorted. Objects thrown into the air.		

Note:
1. "Definition" is taken from the European Seismological Commission report, *European Macroseismic Scale 1998 (EMS-98)*.
2. "MMI description" is taken from the United States Geological Survey publication, *The Severity of an Earthquake*.
3. "Approximate range of ground motions" is based on Worden, et al., 2012, *"Probabilistic Relationships between Ground-Motion Parameters and Modified Mercalli Intensity in California."* The low and high values of the range are determined using half of an MMI intensity unit above and below the MMI in the table (i.e., for MMI VI, the range values are based on MMI from 5.5 to 6.5).

Table C.1.6B. Modified Mercalli Intensity (MMI) Scale vs. Japan Meteorological Agency (JMA) Shindo Seismic Intensity Scale

MMI	*Approximate Corresponding JMA Shindo Intensity*
Less than V	0 to 3 (0.0 to 3.4)
V	4 (3.5 to 4.4)
VI	5L (4.5-4.9)
VII	5U (5.0-5.4)
VIII	6L (5.5-5.9)
IX to X	6U (6.0-6.4)
XI to XII	7 (6.5-7.0)

C.2 Site Specific Geologic Considerations

Unfavorable site-specific geologic conditions include close proximity to known active faults, soft soil conditions, artificial fill, and high potential for major geologic deformation (e.g., liquefaction, landslide, lateral spreading, and sinkholes). There are many sources that can provide information on the hazard these conditions present in a general area. An investigation by a geotechnical engineer is usually needed to confirm the actual hazard at the site and to determine design parameters (e.g., soil bearing capacities or appropriate types of foundations).

The best sources for local geological information of a particular occupied or unoccupied site are soil exploration or foundation reports, prepared by a geotechnical engineer during the site investigation. In some cases, records of site soil borings are included in the plans or specifications for a building site or development.

Some faults are actually zones of movement that can be hundreds of yards (meters) in width. The hazard to a site located close to a fault can be higher than the hazard to sites further away. First, sites located directly over traces of faults can be damaged from the relative movement of the ground on either side of the fault (due to non-earthquake fault creep or fault rupture during an earthquake). Second, expected intensity of shaking is higher near the earthquake source.

There are at least two generic types of soil that could lead to settlement or other poor response: alluvium and loess. Alluvium is clay, silt, sand, or gravel that has been deposited by water or glaciers. Loess is soft soil deposited by wind. Flat areas of alluvial soil adjacent to rivers, or valleys filled with unconsolidated alluvium or loess, generally will respond at least one point higher in MMI than otherwise. Highly water-saturated soft soils may respond two points higher. Tertiary (geologically older) sediments generally are fairly well consolidated and should give a more favorable response than Quaternary (geologically younger) sediments. The depth of the soil is also a factor. Shallow soil beds on rock generally give a better response (i.e., lower shaking intensity) than deep soils.

Soils of man-made fill are very dangerous unless properly engineered and compacted with heavy earth-rolling equipment. Many such fills have been placed by hydraulic pumping methods, without compaction. They may settle or lurch during strong earthquakes, thus causing severe damage to buildings. Many waterfront areas along coastal zones have been enlarged with hydraulic fill, which is a poor building foundation particularly in seismic areas, and that typically requires the use of deep foundations (i.e., piles). Even when appropriate foundations are used to support structures, damage to grade-supported items (such as concrete slabs, asphalt paving, cranes, and utilities) can be great.

Alluvial soils can lose their capacity to support loads if they are nearly or completely water saturated (that is, the space between individual soil particles is completely filled with water) and are strongly shaken. The water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together. Earthquake shaking can cause some of the loose soil to compact, displacing and pressurizing the water in other areas to the point where the soil particles can readily move with respect to each other. This process, called “liquefaction”, essentially produces quicksand. Buildings may then sink into the soil and settle differentially, making repairs very difficult. A classic example of soil liquefaction problems was in the 1964 Niigata, Japan, Earthquake, which caused severe building settlements, including one four-story concrete building that tilted as it sank, ending up on its side.

Alluvial soil should be considered suspect to potential liquefaction when the water table is near the surface. Short of liquefaction, some alluvial soils may be displaced horizontally (lateral spread) or form cracks, fissures, and sags during strong ground motion. Structures may suffer from differential settlement.

Structures built on hillsides need special consideration, as a landslide could occur to the soil on which they rest. Retaining walls of concrete, sheet piling, etc. can be utilized to stabilize the foundation soil downhill of the building, or to prevent a landslide uphill of the structure. The hillside may require dewatering to prevent landslides.

A geotechnical engineer can determine the soil classification and the potential for seismically induced soil failures (e.g., liquefaction, landslide, lateral spread, or sinkholes). The site classification is typically based on Vs_{30} (i.e., the average shear wave velocity in the upper 30 m of soil) or blow counts (i.e., from a Standard Penetration Test, the number [N] of blows needed to advance a sample tube 12 in. [300 mm] into the soil). Site classifications taken from ASCE 7-22 are shown in Table C.2A (see ASCE 7-22 for more information about shear wave velocity, blow counts, etc.,).

Table C.2A. Site Classification from ASCE 7-22

Site Class (Not to be confused with Seismic Design Categories A to F)	*Soil Profile Name*	*Soil Profile Description (For Unnamed Soil Profiles)*
A	Hard rock	
B	Medium hard rock	
BC	Soft rock	
C	Very dense sand or hard clay	
CD	Dense sand or very stiff clay	
D	Medium dense sand or stiff clay	
DE	Loose sand or medium stiff clay	
E	Very loose sand	
	Soft clay	More than 10 ft (3 m) of soil having high plasticity, high moisture content or very low strength
F	Unnamed	Soils vulnerable to potential failure or collapse under seismic loading (e.g., liquefiable, highly sensitive [quick] clays, collapsible weakly cemented soils) Peats and highly organic clays greater than 10 ft (3 m) thick Very high plasticity clays greater than 25 ft (7.6 m) thick Very thick soft/medium clays (greater than 120 ft [36.6 m])

Sites ideally should be located in areas of firm soils (Site Class A to D). Soft soil and soil subject to failure (Site Classes E and F) are very undesirable. Site selection should be made to limit the potential for damage to the facility due to fault rupture or creep and other soil failures (landslide, etc.). While the FM l earthquake risk map incorporates soil effects including amplification of shaking intensity, it does not consider soil failures such as liquefaction landslide, lateral spreading, or sinkholes.

C.3 Building Codes

C.3.1 Building Code Design Philosophy

Minimum building code provisions are intended mainly to safeguard against collapse or failure that can result in loss of life in a major earthquake. Minimum code provisions will not necessarily limit damage or limit business interruption, or allow for easy repair.

The design earthquake ground motion set by building codes has a low probability of occurrence during the life of the structure. Historically, an earthquake ground motion having a mean recurrence interval of approximately 500 years has commonly been used as a basis for design. Currently, some codes base design on more severe earthquake ground motion with approximately a 2500-year return period or, in the case of ASCE 7, using a 1% in 50-year probability of collapse. When actual design forces are determined, the theoretical forces from this more severe earthquake ground motion are reduced (e.g., for ASCE 7, design accelerations are multiplied by a factor of 2/3) and can be fairly comparable to the 500-year return period earthquake ground motion. In certain cases, the reduced 2500-year (or 1% in 50-year collapse probability) ground motions are significantly different (usually higher) than the 500-year ground motions.

While it is possible to design a structure to respond in the elastic range for these earthquake ground motions, building codes are based on the assumption that it is impractical to do so in most cases. The code-required lateral design forces typically are a fraction of the theoretical forces that would be developed for the design earthquake ground motion in recognition that this ground motion is a rare event. Because this means buildings will respond in the inelastic range for the design earthquake ground motion (and possibly for other more frequent events), building codes also have detailing provisions that are intended to provide the required level of inelastic capacity. The level of force reduction and special detailing required varies depending on the return period of the design earthquake ground motion (e.g., 500-year, 2500-year), the type of lateral force-resisting system, the building use, the likelihood that damage to lateral force-resisting elements will result in building instability (e.g., damage to shear walls that are also bearing walls is more likely to trigger a vertical collapse), and observed performance of similar buildings in past earthquakes.

C.3.2 Building Code Provisions

Building codes establish the seismic design criteria through formulas and provisions that specify:

- the strength and stiffness of the structural system required to resist the seismic design loading (which is some fraction of the theoretical maximum forces from the design earthquake).
- the requirements for structural detailing that will allow the expected level of inelastic behavior to occur.

It is the designers' responsibility to design into the structures sufficient strength, stiffness, and detailing. Competent structural engineers are aware of the potential effects of strong earthquakes on buildings and equipment, and will make allowances for the special problem areas in their static or dynamic analyses and designs.

Many countries develop and enforce their own earthquake code provisions; often these are based on earthquake requirements in codes established in the United States (e.g., the *Uniform Building Code* and its replacement, the *International Building Code*), Japan (*Building Standards Law*), New Zealand (*Building Standards Law*), and Europe (*Eurocode 8*).

In the United States, the *Uniform Building Code* (UBC) was historically the most commonly used code in seismically active areas prior to 1997 when the last edition of this code was issued. Currently, the Building Seismic Safety Council (BSSC) develops the National Earthquake Hazard Reduction Program (NEHRP) *Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. The NEHRP provisions are then used to develop the earthquake requirements in the American Society of Civil Engineers (ASCE) standard *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7).* ASCE 7 is then typically adopted (possibly with some revisions) by the actual building codes. The main United States building codes are the *International Building Code* (IBC) and the National Fire Protection Association *Building Construction and Safety Code* (NFPA 5000); some states also issue their own codes.

Although each individual code has many unique provisions, in general all are based on and incorporate similar concepts. The following main steps commonly are used to establish code design criteria:

- Determine the appropriate seismic zone or seismic acceleration, typically from a map (more seismically-hazardous areas will have correspondingly higher accelerations).
- Determine the site soil classification (poorer soils, in general, result in increased design forces).
- Apply basic seismicity modification factors (e.g., increases for “near-fault” locations, reductions when a 2500-year earthquake ground motion is used for design).
- Determine whether the building or equipment is essential or hazardous (essential or hazardous buildings/ equipment typically are designed using increased forces and more restrictive detailing).
- Determine other needed parameters (e.g., natural period of vibration).
- For buildings, determine the appropriate reduction factor to be applied based on the lateral force-resisting system and system detailing (systems having greater redundancy, ability to dissipate energy without serious degradation and damping utilize larger reduction factors).
- For equipment, determine appropriate amplification factors (flexible equipment generally amplifies earthquake shaking) and reduction factors (based on the overstrength and deformability of the component's structure and attachments).

- For equipment, determine the location in the building (forces are higher at upper floors than they are at grade–in the latest IBC, design forces at the roof can be up to three times those at grade).
- Determine building or equipment weight and weight distribution.
- Determine other relevant information (e.g., vertical or plan irregularities, redundancy factors).
- Determine the type of analysis allowed or required (i.e., equivalent static lateral force or dynamic analysis).
- Analyze the system based on the above parameters and compare to code provisions for strength and deformation limits.

C.3.3 Local Code Provisions

There is often wide latitude given at the local level (e.g., city, county, or state) with respect to adopting and enforcing a building code. Often, local authorities having jurisdiction (AHJ) are not required to enforce seismic design provisions that are established in other consensus documents (e.g., the ASCE 7 Standard). In some cases, commonly used construction techniques long established as having poor seismic performance (e.g., unreinforced masonry buildings) are allowed by the local authorities in seismically active areas. Specifying that a facility must meet local building code requirements does not necessarily assure even minimal earthquake performance.

Even when the local building code adopts the seismic provisions of a state-of-the-art consensus document (e.g., the ASCE 7 Standard), relying only on the minimum earthquake provisions of these building codes may lead to significant property damage and downtime following a major earthquake.

C.3.4 Enhancing Building Code Provisions to Minimize Seismic Risk

The degree of seismic vulnerability can be lessened by establishing performance objectives that go beyond code minimums (e.g., using performance-based design), developing project specifications in support of those objectives, and communicating this performance-based philosophy in a way that the owner and all members of the design team share a clear understanding of the earthquake performance objectives established for the completed facility.

Seismic design measures can be taken that mitigate the consequences of many less than desirable conditions (e.g., using pile-supported structures at sites subject to liquefaction). However, strictly from the perspective of earthquake performance, it usually is better to minimize or eliminate conditions that have a high probability of increasing damage in a seismic event rather than designing to accommodate them. Whether or not minimizing or eliminating objectionable seismic conditions is desirable to the overall project typically is a matter of comparing the costs and benefits of all options. As a minimum, seismic issues must be considered, with the goal of lowering the potential for damage during an earthquake.

The owner should include in project specifications requirements for owner involvement in decision-making if and when provisions intended to minimize potential earthquake damage conflict with other design considerations. The following items outline some ways to limit the potential for earthquake damage by minimizing the risk and/or enhancing minimum code provisions:

- **Site Selection**: When possible, locate the facility at a site that has competent soils (site class A [hard rock] to site class D [stiff soil] as defined in the IBC) and as far from active earthquake faults as possible. Sites to be avoided if practical include those:
  - crossed by faults
  - having a significant potential for soil failures (e.g., liquefaction, landslide, lateral spreading, sinkholes, etc.)
  - having soils classified as site class E (soft soil) or site class F (see definition above)
  - that are very close to active faults (i.e., within 6 miles [10 km] of major faults and within 3 miles [5 km] of other active faults)
  - in areas affected by earthquake-related hazards such as tsunamis, debris from adjacent sites subject to landslides, dam failures, hazardous adjacent sites, etc.
- **Design Code**: Where the local building code does not adopt or relaxes the requirements of broader consensus codes containing more restrictive earthquake requirements, the design should be performed in

accordance with a recognized building code that incorporates state-of-the-art seismic provisions. Examples include the *International Building Code* (United States), Japan *Building Standards Law*, New Zealand *Building Standards Law* and *Eurocode 8*.

- **Critical Facilities**: Examples of typical essential or hazardous facilities in building codes are hospitals, fire stations, and police stations. Code design provisions for these facilities (and similarly for equipment that is essential, is needed for life safety, or contains hazardous material) are more stringent than for ''normal'' facilities. Design forces typically are higher (through the use of an importance factor to increase design forces) and detailing is often more restrictive. Buildings and equipment designed and constructed incorporating these essential facility criteria will have a lower probability of earthquake-induced loss of functionality and operability.

  In some cases non-traditional methods, such as base isolation or energy dissipating devices installed within the structure, should be considered for critical facilities. Base isolators are devices added between the building or equipment and the foundation to separate or isolate the motion of the building or equipment from the motion of the ground, thus reducing (but not eliminating) seismic forces and deflections. Energy dissipating devices can be installed within the building to dampen earthquake motions.

  Where a corporation considers a facility or building or equipment as essential (although these items may not be classified as essential or hazardous per building code criteria), or for important one-of-a-kind or hard to replace items, limiting damage and lessening downtime should be corporate objectives. Using ''essential'' or ''hazardous'' facility building code provisions (or possibly even more restrictive owner-imposed criteria) or using non-traditional techniques (e.g., base isolation), along with peer review and construction observation (see below), should be seriously considered to meet these objectives.

- **Peer Review**: Peer review is an independent professional assessment of the design assumptions, analysis, and construction documents to judge whether the design is adequate to meet the owner-established performance objective. It is performed by an engineering consultant or consultants having recognized technical expertise in the design and seismic performance of buildings, equipment, and nonstructural components similar in configuration to the project being reviewed. The reviewing engineer(s) should be hired directly by the owner to guarantee impartiality. The final responsibility for the design remains with the engineer of record, not the peer reviewer. Unresolved issues should be brought to the attention of the owner for resolution. Independent peer review should be strongly considered for:

  - facilities, buildings, and equipment designated by the owner as critical
  - buildings having highly irregular or unusual designs
  - one-of-a-kind or hard to replace items that are important to production
  - facilities located in areas without experienced building officials
  - facilities having high overall property and business interruption values;

- **Building Design**: Symmetrical, regular building configurations incorporating ductility and redundancy should be used where possible. Uniform, continuous distribution of strength, stiffness, ductility, and energy dissipation capacity is desirable. Designs that incorporate horizontal or vertical irregularities, such as weak or soft stories, roofs or floors with abrupt discontinuities, etc., have not performed satisfactorily in past earthquakes because concentrated forces or drifts tend to occur in buildings having these features. Irregular structures need to be well engineered and detailed, especially at discontinuities, for satisfactory results.

  Some specific considerations with respect to building design include the following:

  - Unreinforced masonry should not be used in any way, even if allowed by the local building code–not as part of the lateral force-resisting system nor as non-structural infill, partitions, or facade.
  - When concrete or masonry frames are used, they should be detailed using the most stringent code ductility requirements (less restrictive detailing such as that allowed for "ordinary" or "intermediate" frames should not be used).
  - Plain or minimally reinforced concrete or masonry walls should not be used.
  - Structural irregularities should be avoided (in particular, vertical lateral force-resisting elements should be continuous to the foundations, weak or soft stories should be avoided, and horizontal diaphragms having abrupt discontinuities or variations in stiffness should not be used).

- Use of a lateral system that relies on cantilevered column elements for lateral resistance should be avoided.
- Rigid nonstructural elements that resist forces before more flexible structural elements (e.g., masonry infill walls that restrict the movement of steel or concrete moment-resisting frames) should not be used.
- Ideally, the maximum span (i.e., the distance between shear walls, braced frames, etc.) and the maximum span to depth ratios (s:d) of horizontal diaphragms (i.e., roofs and floors) should not exceed the following:
  - 200 ft (61 m) and 3:1 for flexible wood diaphragms (e.g., plywood)
  - 300 ft (91 m) and 3:1 for flexible bare metal deck diaphragms or rigid diaphragms of a concrete topping slab over precast concrete elements
  - 400 ft (122 m) and 4:1 for other rigid diaphragms (e.g., cast-in-place concrete or concrete on metal deck)
- Precast concrete diaphragms without a topping slab should not be used.
- Ideally, straight or diagonal wood sheathing should not be used and plywood, oriented strand board, or similar sheathing should be at least ½ in. (13 mm) thick for shear walls and ⅝ in. (16 mm) thick for horizontal diaphragms.
- Use of walls sheathed with lath and plaster or gypsum board as shear walls should be avoided if possible (if these materials must be used, the wall height to length ratio should be less than one [i.e., the wall should be at least as long as it is tall]).
- All detailing and design requirements imposed by the building code for buildings with irregular features should be met.
- Drift should be limited as required by the code and there should be no pounding of adjacent structures.
- Special attention should be given to the system anchoring concrete and masonry walls to roof and floor diaphragms (ties from the roof or floors to walls, and all connections within the diaphragm necessary to develop the anchorage forces into diaphragms and tie each diaphragm together to act as a unit).

- **Equipment Restraint/Location and Nonstructural Component Anchorage**: Anchorage of equipment is an often overlooked item. When anchorage requirements are specified at the time the equipment is ordered and installed, it is typically a low- or no-cost item. However, it can be costly to anchor equipment later. Advantageously locating equipment and piping is another low- or no-cost way to reduce the potential for earthquake-induced damage.

  Anchorage of all equipment is strongly recommended, including in those buildings in which base isolation or energy dissipating devices have been installed. It may be reasonable to leave non-critical equipment unanchored if it will not overturn, fall from supports or be damaged from swinging or impact, and if misalignment or utility damage caused by sliding or swinging are not concerns. In addition, anchorage of some equipment with unusually fragile or sensitive interior components (e.g., steppers and diffusion furnaces in semi-conductor manufacturing occupancies) may need to account for the unique qualities of this equipment. In some cases, damage to the internal components may not be preventable; for these items preventing damage to attached utilities and piping, and preventing gross failure (e.g., overturning, falling) may be the only practical solution.

  An engineering consultant experienced in the seismic performance of equipment and nonstructural components should base the design of these items on reliable codes and considering the equipment importance (see discussion above). Anchorage design may be provided by the equipment vendor or installer but should be confirmed by the engineer of record for the project.

  Some specific considerations with respect to equipment anchorage include the following (see Section C.5 for more information):

  - Equipment should be located at grade or as low in the structure as possible to avoid the amplified forces that occur at higher levels in the building.

- It is best to locate equipment and piping subject to leaking away from sensitive areas such as clean rooms, sterile areas, susceptible high-value storage (e.g., pharmaceuticals), etc.–if this is not possible it is important to specifically review this equipment and piping for adequacy.
- It is important to specifically review equipment and piping used to store or move hazardous materials.
- Items that may pose a fire hazard (e.g., gas-fired equipment, high-voltage electrical equipment that could arc) should have earthquake protection and seismic shutoff valves per Data Sheet 1-11.
- Fire protection systems (e.g., sprinkler piping, suction tanks, fire pumps and their associated batteries) should have earthquake protection per Data Sheet 2-8 and Data Sheet 3-2.
- Anchorage of equipment that is expensive or unique and is critical for production, perishable product storage, or emergency operations merits special attention, particularly if the item will overturn or fall from its supports, or if sliding or swinging will result in significant damage due to impact or damage to rigidly connected utilities.

- **Site Observation and Review of Submittals**: Construction errors caused by misinterpreted drawings and specifications can result in significant earthquake-induced damage to an otherwise well-designed structure or equipment item. Observation of the construction by a qualified engineer with a good understanding of the building design has been determined to be critical to limit construction errors that could affect earthquake performance. For similar reasons, the design engineer should also review shop drawings and other submittals during construction to confirm the design intent is being met. Generating shop drawings should be required only if they are necessary to properly perform the work.

  The geotechnical engineer should provide an appropriate level of foundation review/inspection during construction. The design engineer should review shop drawings, special inspection reports and other submittals during construction. Structural observation (separate from and in addition to any code-required special inspection and testing requirements performed by a qualified inspector) should be performed by the design engineer or another qualified engineer for all buildings and major equipment. Structural observation by the design engineer during the course of construction should consist of visual reviews of critical elements of the lateral force-resisting system for general conformance with the approved plans. Observations should be made at significant construction stages and should be timed to allow for identification and correction of deficiencies without substantial effort or uncovering of the work involved. The engineer should provide his written record of the observation to the owner noting conformance or nonconformance with the approved plans.

C.4 Earthquake Performance of Buildings

C.4.1 General

The objective of the earthquake design is to provide a continuous load path capable of transferring all seismic loads and forces from their points of origin to the final points of resistance for the building (including nonstructural elements).

Designing for wind loading will provide very little earthquake resistance in the lateral force-resisting systems. Except for very light weight buildings (e.g., light metal structures), this resistance is not sufficient to resist major earthquakes. For common buildings in the most severe seismic zones, today’s building codes with earthquake design requirements apply an Allowable Stress Design (ASD) static loading in the range of 5%-20% of the building’s mass horizontally (about 7%-28% for Load and Resistance Factor Design [LRFD]). The percentage varies due to code, zone location, building type, foundation soil, natural period of vibration, etc. Design forces for taller (i.e., more flexible) buildings and more ductile lateral systems (e.g., well-reinforced concrete moment frames - see Section C.4.3) are typically lower than design forces for shorter (i.e., stiff) buildings and less ductile lateral systems (e.g., poorly reinforced concrete moment frames).

An example of the approximate increase in construction cost for buildings designed to resist earthquake, relative to the cost if they were designed for wind resistance only, is addressed for six typical buildings in the study entitled *Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee* (NIST GCR 14-917-26, December 2013). The average S_{DS} and S_{D1} values used in the study were 0.67g and 0.37g, respectively. These are about midway between the generic S_{DS} and S_{D1} values, given in 2.2.2.1.2, for FM 100-year earthquake zones and those for FM 250/500-year earthquake zones.

For Memphis, the premium to incorporate seismic resistance (which includes the earthquake lateral system for the structure as well as detailing of connections and anchorage of building support equipment) relative to the design for wind only is shown in Table C.4.1. The “structure only” cost includes the structural framing (beams, columns, floor slabs, shear and perimeter structural walls, braces, etc.) and foundation. The “total building” costs include the structure, non-structural cladding/curtain walls, roofing, stairs, partitions, building support equipment, etc. but excludes grading, landscaping, parking lots, buried utilities, furnishings and tenant improvement.

Table C.4.1. Increase in Construction Cost When Incorporating Earthquake Design in Memphis, Tennessee[1]

Building Description	*Building Earthquake Lateral System*	*ASCE 7-10 Seismic Design Premium Relative to Building Designed for Wind Only*	
		Structure Only	Total Building
Three-story (54,000 ft^2 [5015 m^2]) wood-framed apartment building	Stucco and wood-sheathed shear walls	4.1%	1.2%
Four-story (100,000 ft^2 [9290 m^2]) steel-framed office building	Steel braced frames	19.6%	2.8%
One-story (38,000 ft^2 [3530 m^2]) concrete tilt-up retail building	Flexible steel deck roof to perimeter concrete shear walls	0.6%	0.5%
One-story (400,000 ft^2 [37,160 m^2]) concrete tilt-up warehouse	Flexible steel deck roof to perimeter concrete shear walls and interior steel braced frames at an expansion joint	2.9%	1.4%
Six-story with basement (162,000 ft^2 [15,050 m^2]) steel-framed hospital	Steel braced frames	17.1%	2.5%
One-and two-story (51,200 ft^2 [4760 m^2]) masonry elementary school	Concrete slab second floor and steel deck roof to masonry shear walls	4.4%	1.4%

Note:
[1] NIST GCR 14-917-26 Study, Memphis, Tennessee (Average S_{DS} = 0.67g; Average S_{D1} = 0.37g)

Stresses are introduced into buildings and other structures when subjected to forces from earthquake vibration because of the inertia of the structure (i.e., the tendency of a body at rest to remain at rest). Designing buildings to resist earthquakes requires that ground motions be translated into forces acting upon a building. Earthquake forces are called lateral forces (and the system that resists these forces is called the lateral force-resisting system) because their predominant effect is to apply horizontal loads to a building. Vertical earthquake forces are usually only accounted for in special cases because reserve strength in the vertical load-resisting system (i.e., the system resisting gravity loads such as the weight of the building, snow, etc.) normally provides sufficient resistance.

The response of a structure to ground shaking depends on several building characteristics. For this reason, different buildings can respond differently even if the ground shaking to which they are subjected is exactly the same. The response of a building depends in large part on its fundamental period of vibration (which in turn depends on the building height, the structural system stiffness, and the building mass distribution). Seismic waves are amplified when the building’s fundamental period of vibration is similar to that of the ground shaking motion. Since earthquake shaking is dominated by high frequency waves, short-period buildings (e.g., low-rise shear wall or braced frame structures) are designed for higher forces than long-period buildings (e.g., tall moment frame structures). In some cases, longer period earthquake waves that are more dominant farther from the earthquake or on softer soils can cause more flexible (e.g., taller moment frame) buildings to experience amplified forces.

Another important building characteristic is whether the building is regularly or irregularly shaped. Earthquake force distribution in a simple rectangular building will be fairly uniform. In other buildings having vertical (e.g., “soft” stories) or plan (e.g., L-shaped) irregularities, forces will tend to concentrate at the irregularities. The stiffness of individual structural elements also has an effect. When structural elements having different rigidities are used in combination, the stiffer elements (e.g., walls) will attract higher forces than the more flexible structural elements (e.g., moment frames) whether they are part of the lateral force-resisting system or not.

To prevent pounding of adjacent structures against each other, a seismic joint is provided with a clearance or gap greater than the combined (seismic) displacements of each structure. Seismic joints should not be confused with expansion joints. Expansion joints are “slip joints” within the structure provided to compensate for thermal expansion and contraction. They are usually points of damage concentration during earthquakes. In rare cases a specially designed combination of a seismic and slip joint is utilized.

Finally, though less common, the response of the building to ground shaking can be modified using base isolation (i.e., devices added between the building and the foundation to separate or isolate the motion of the building from the motion of the ground) and energy dissipating devices installed within the structure (to dissipate and dampen earthquake motions).

Additional helpful information regarding design of buildings for earthquake, generic earthquake building types and retrofit (Section C.4.3), and earthquake irregularities (Section C.4.4) is available in the following United States Federal Emergency Management Agency (FEMA) documents:

- FEMA 454/December 2006, *Designing for Earthquakes, a Manual for Architects*
- FEMA P-749/December 2010, *Earthquake-Resistant Design Concepts*
- FEMA 154/January 2015, *Rapid Visual Screening of Buildings for Potential Seismic Hazards*
- FEMA 547/October 2006, *Techniques for the Seismic Rehabilitation of Existing Buildings*

These documents are available for download at: www.fema.gov.

C.4.2 Foundations

The type of foundation is extremely important to the overall earthquake performance of a structure. The lateral and vertical motion of an earthquake will tend to make the structure rock on its foundation, thus increasing the pressure on the soil. The foundation must be capable of transferring lateral forces from the structure to the supporting rock or soil, and preventing excessive settlement. The type of foundation system selected depends on the soils, loads, and the structural system. Foundations are typically constructed of reinforced concrete although piles are sometimes wood or steel.

Foundations can be classified in two main groups: shallow and deep.

Shallow foundations are the most common and are generally appropriate for most buildings unless the soils are poor or column/wall loads are very large. Typical shallow foundations include the following:

- Spread footings: square or rectangular concrete pads, usually reinforced, supporting a single or multiple columns. The plan dimensions of the footing must be large enough to spread out the load such that the allowable soil pressure is not exceeded. In some buildings individual footings may be tied together to form a grid foundation.
- Continuous footings: a narrow concrete pad, unreinforced or reinforced, supporting a wall or a row of columns.
- Mat foundation (also known as a ‘‘raft’’ or ‘‘floating’’ foundation): generally a thick reinforced concrete slab designed to spread the structure’s load over the greatest possible area and minimize settlement. Mat foundations are often used when the soils are relatively poor and the total area of individual spread footings would constitute a very large percentage of the plan area of the building, or when deep foundations are not feasible.

Deep foundations are commonly used where soils are poor or foundation loads are high. For example, structures at sites having soils subject to liquefaction should be supported on deep foundations. The deep foundations usually are used only to support the building. Concrete slabs, asphalt paving, utilities, tanks, etc. supported directly on or in the ground may be damaged if soils liquefy. Typical deep footings include:

- Piles: slender vertical structural members that are forced into the ground by impact (from a machine called a ‘‘pile driver’’). They are driven through poor soils to bedrock or, more commonly, ‘‘to refusal’’ into firmer soils beneath. Piles distribute their loads at the tip or by friction on the sides of the pile, or a combination of both. Piles most commonly are made of reinforced or prestressed concrete but can be wood (usually only in older structures) or steel. As wood needs to be constantly wet or dry for decay prevention, the ground water table must be above the top or below the tip of wood piles. Piles usually are driven in clusters or groups, and there is a heavy pad or cap of reinforced concrete placed over their tops to distribute building loads into the piles.

Building codes usually require pile caps to be tied together with concrete beams in seismic areas to prevent lateral movement of the pile caps. For example, in ASCE 7-22 the tie design force in tension or compression is 10% of S_{DS} times the largest factored dead plus live vertical load on any one of the connected pile caps.

- Caissons (or drilled piers): foundations in which deep holes are drilled and then filled with concrete. Steel reinforcement usually is used for at least a portion of the caisson length. Caissons usually are bigger in diameter than the columns they support and are larger members than piles. Unlike the pile, which may get its support from the sides, the caisson depends mainly on its bearing capacity in the soil or rock that supports it. When caissons rest on soil, they generally are ‘‘belled’’ at the bottom to spread the load over a wider area. The greater the bell diameter, the larger the area and the greater the bearing capacity.

C.4.3 Generic Building Types

C.4.3.1 Lateral Force-Resisting Systems

Lateral force-resisting systems (i.e., the systems that resist horizontal earthquake forces) typically align with the major axes of the building. In buildings, lateral forces most often are transferred by roof and floor sheathing or topping slab (diaphragms) that act like deep beams spanning horizontally between shear walls, braced frames, or moment frames. Although shear walls, braced frames, or moment frames are often located at the perimeter of a building as shown in Figure C.4.3.1, they do not necessarily have to be placed there, as discussed below. The lateral forces in walls or frames are then transferred to foundations and into supporting soils. Lateral force-resisting systems are sometimes identified by text on structural drawings but usually must be visualized from observations or structural drawings.

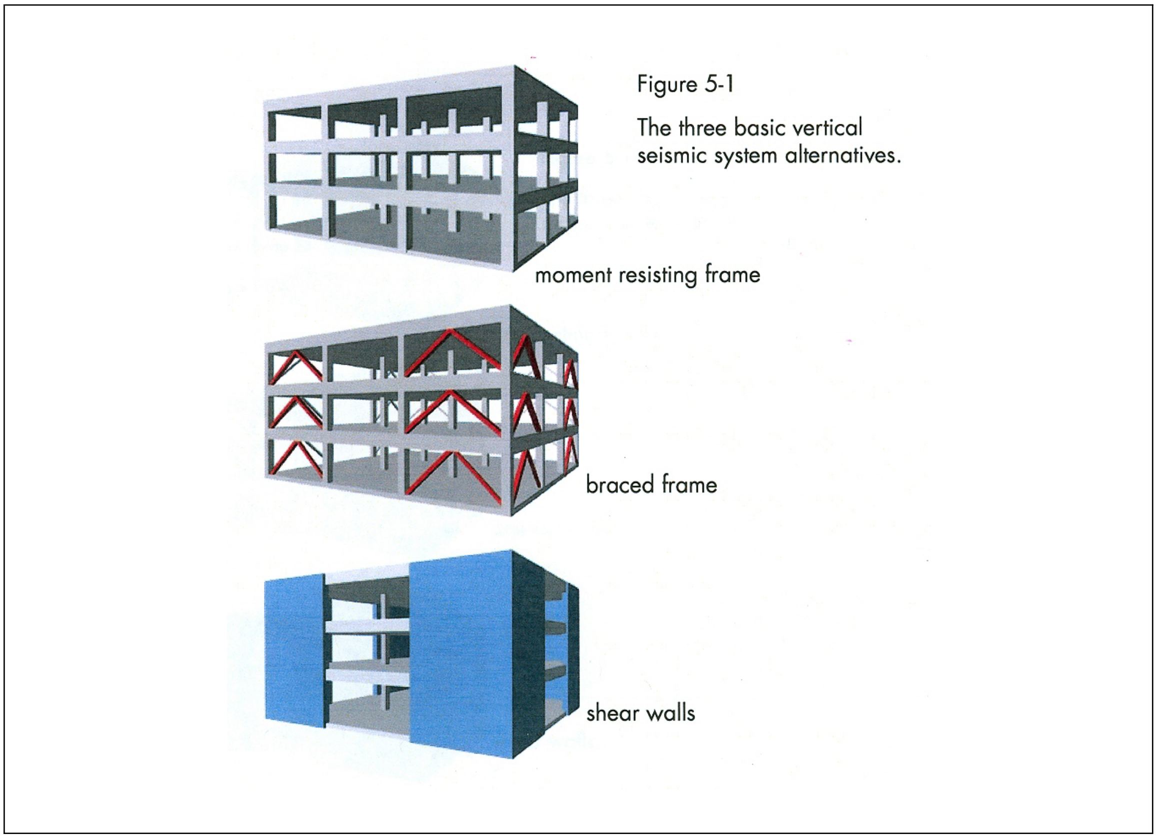

Fig. C.4.3.1. The three basic vertical seismic system alternatives (Figure 5-1 from FEMA 454)

Roof and floor diaphragms are typically divided into those that are “flexible” and those that are “rigid”. Further divisions, such as “semi-rigid”, also can be made but will not be used here. The diaphragm type is associated with the floor or roof sheathing or topping slab, not the type of supporting members. Flexible diaphragms are roofs/floors sheathed with wood (plywood, oriented strand board [OSB], or wood boards), metal deck without concrete fill, metal deck with insulating concrete fill, and poured gypsum (poured gypsum is fairly rare; although it is classified for our purposes as flexible, it is sometimes considered to be semi-rigid). Although not technically a diaphragm, horizontal steel bracing also acts similarly to a flexible diaphragm. A flexible diaphragm does not transfer significant forces via rotation and therefore must have lateral force-resisting elements (shear walls, braced frames or moment frames) at or very near each edge of its perimeter. Rigid diaphragms are roofs/floors of concrete-filled metal deck (not insulating concrete) or concrete slabs. Because a rigid diaphragm can transfer significant forces via rotation, lateral force-resisting elements (shear walls, braced frames or moment frames) do not necessarily have to be provided near its perimeter.

Shear walls (i.e., walls that resist lateral earthquake forces) and braced frames often are located at roof or floor boundaries, at elevator and stair cores, and along interior walls. Moment frames (a.k.a., moment-resisting or rigid frames) have truss or rigid beam-to-column connections that transfer bending to resist lateral earthquake forces. They can be located at discrete locations (often near roof or floor boundaries) or can be distributed throughout the building plan.

C.4.3.2 Strengthening Existing Lateral Force-Resisting Systems

Existing buildings can be strengthened (retrofit) to address some or all of their seismic weaknesses. Generally, not every weakness can be eliminated, so retrofitted buildings are not usually as seismically resistant as a new structure. For some buildings discussed in Section C.4.3.3 (e.g., concrete tilt-up buildings with retrofit roof-to-wall anchors and interior continuity ties) the performance difference may be relatively small. In other cases (e.g., unreinforced masonry [URM] buildings) with structural elements that are inherently weak, the retrofit may prevent collapse but not total economic loss in strong ground shaking.

Retrofits should ideally mitigate all weaknesses that could lead to partial or total collapse, or major damage. Most commonly, retrofits involve adding elements. An example is a building with major irregularities (e.g., having soft stories or vertical elements that don’t stack, or having an open front - see Section C.4.4) where the retrofit involves adding lateral force-resisting elements (shear walls, braced frames or moment frames). Other retrofits may include locally strengthening elements (e.g., wrapping concrete columns with carbon fiber/epoxy composite fabrics to confine concrete that cracks during an earthquake), improving connections between components (e.g., roof-to-wall anchors in concrete tilt-ups), removing components (e.g., replacing hollow clay tile partitions with gypsum-sheathed partitions), and reducing demand (e.g., by base-isolating the structure).

Some guidance regarding appropriate retrofit of generic building types to prevent major damage is discussed in Section C.4.3.3. Comprehensive conceptual retrofit details are available in FEMA 547, *Techniques for the Seismic Rehabilitation of Existing Buildings*. Again, in some cases, such as URM structures, a retrofitted structure may still experience substantial damage due to the inherent vulnerability of the construction.

A rigorous methodology for determining how structures and equipment should be seismically retrofitted is provided in the American Society of Civil Engineers standard ASCE 41, *Seismic Evaluation and Retrofit of Existing Buildings*. Building retrofits can be performed to meet a particular performance target (i.e., collapse prevention, life safety, immediate occupancy, or operational). Recommend building retrofits be made at least to the life safety level, which would (according to ASCE 41) result in “slightly more damage and slightly higher life safety risk” compared to a new code-compliant building. ASCE 41 also has retrofitting guidance for equipment and non-structural systems - see Section C.5.2.

C.4.3.3 Generic Building Types and Common Retrofits

The building code classification of building lateral force-resisting systems is somewhat complex. For example, it depends on the type of vertical elements used (shear walls, braced frames, or moment frames), the material used (concrete, masonry, steel, wood, etc.), whether vertical elements resist only seismic forces or also resist vertical gravity loads, whether a dual system is used (moment frames combined with another system), and specific details (how reinforcement is configured, how moment connections are made, etc.). Building code design forces and detailing requirements are related to these classifications.

As a simplification, the following eight categories can be used to classify most buildings. The categories below, combined with information on age of the structure and building-specific features (e.g., structural irregularities), can give a reasonable understanding of the expected building performance in an earthquake. A listing of common, but not necessarily comprehensive, retrofits is included for each generic earthquake building type.

- **Light Metal Structures**. Light metal buildings (also known as pre-engineered metal buildings [PEMB] or all-metal building systems [AMBS] or “Butler” buildings) typically are one-story prefabricated, light-gage steel structures. Roofs can be flat or pitched. Siding and roof decks most commonly are metal, although other materials can be used. The vertical load-carrying system typically consists of metal deck supported by steel beams (often light gage) spanning between steel moment frames.

  The lateral force-resisting system in the longitudinal direction typically consists of steel rod or cable bracing in the roof plane, which acts as a simply supported truss spanning between longitudinal braced frames. In the transverse direction, the roof deck typically spans between the transverse moment frames along each column line.

  Light metal buildings have generally performed well in past earthquakes, usually experiencing only minor structural, and often somewhat greater architectural, damage.

  Retrofits of light metal structures are not typically required except in unusual cases (e.g., removed braces, addition of heavy façade). Adding or strengthening steel braced frames or moment frames to properly resist the forces would likely be the most common retrofits.

- **Wood Frame Structures**. Wood frame structures typically have vertical load-carrying systems of wood sheathing (possibly with lightweight concrete fill) over wood framing at the roof and elevated floors, supported by wood stud walls or columns. The lateral force-resisting system usually consists of the flexible wood-sheathed roof and floor diaphragms that transfer lateral forces to shear walls sheathed with wood, gypsum board (drywall), or cement plaster (stucco).

  Wood frame buildings, when properly designed and constructed, and properly maintained, are highly resistant to earthquake damage. This type of structure tends to possess a great deal of redundancy, design overstrength, and energy absorption capability.

  Although most wood frame buildings performed well during the 1994 Northridge, California (USA) earthquake, a significant number experienced substantial damage. Shear walls using stucco or gypsum board performed poorly in numerous two- and three-story apartment and condominium buildings. Damage to wood frame buildings also appears to have occurred due to inadequacies in the design of plywood shear walls, poor installation of hold-down hardware at the ends of shear walls, and the irregular placement of shear walls that resulted in diaphragm rotation. Because of the poor performance of some wood frame structures, some building codes reduced allowable stress values for stucco, drywall and plywood shear walls, and hold-down hardware; disallowed the use of stucco and drywall shear walls in the bottom of multi-story buildings; and disallowed the use of diaphragm rotation to transfer forces.

  Retrofits of wood frame structures commonly address items such as:

    - Adding elements to correct significant irregularities in plan and elevation (e.g., tuck-under parking)
    - Strengthening inadequate bracing or poorly-designed shear wall systems, especially in buildings having heavy tile roofing, or in the bottom floor of multi-story buildings
    - Strengthening of large wall and floor openings and inadequate connections between floors and walls.
    - Adding bolts from the structure to its foundation
    - Strengthening short walls between a slightly raised first floor and the ground (called cripple walls) that can tip

- **Steel Braced Frame Structures**. Many different vertical load-carrying systems can be used in a steel braced frame structure. A common construction is concrete-filled metal floor and roof decks that are supported on steel beams, girders and columns. Other systems having concrete or wood floors and roofs are possible.

  The lateral force-resisting system typically consists of rigid concrete diaphragms that distribute earthquake forces to steel braced frames. Wood sheathing or steel bracing at floors and roofs also can serve as flexible diaphragms to transfer lateral forces.

Braced steel frames are commonly used in mid- and low-rise steel construction. These structures derive their lateral strength through the presence of bracing between their beams and columns. These braces resist lateral forces primarily through tension and compression. Braced frame structures tend to be much stiffer than moment-resisting frames, another common form of structural steel construction, and therefore are often used in buildings to reduce earthquake and wind-induced swaying. The rigidity inherent in this system tends to minimize the amount of damage experienced by architectural elements.

Braces commonly will yield and buckle, and may even fracture, during strong ground shaking; however, such damage is easily repaired. If the braces are not adequately connected to the beams and columns, the connections themselves may fail at premature load levels. Modern codes (e.g., 1973 and subsequent editions of the *Uniform Building Code* [UBC]) usually require that connections be designed to be stronger than the braces themselves to prevent such failures. Braces that terminated in the middle of a column (e.g., "K" braced frames) have performed poorly in past earthquakes because they induce large stresses on critical column elements. Particularly in buildings greater than two stories in height, ''K'' braced systems and tension-only bracing (e.g., rods or cables) have performed poorly and usually are not permitted for earthquake load resistance in modern codes.

Retrofits of steel braced frame structures commonly address items such as:

- Strengthening braced frames or adding new elements (e.g., replacing tension-only bracing, strengthening connections at ends of braces, or adding braced frames or shear walls)
- Adding elements to correct significant irregularities in plan and elevation (e.g., soft stories)
- Adding collectors in the roof or floor diaphragms to transfer forces at openings or to braced frames
- Strengthening connections of frames to foundations.

- **Steel Moment Frame Structures**. Steel moment-resisting frames commonly have a vertical load-carrying system of concrete-filled metal floor and roof decks supported on steel beams, girders, and columns. The lateral force-resisting system typically consists of rigid concrete-filled metal deck diaphragms that distribute earthquake forces to steel moment frames at discrete locations at each level in the structure. Typically only some of the building frames are utilized as moment frames. At these locations the beams are rigidly attached to columns, and lateral forces are resisted by bending of the beams and columns through the rigid beam-column joints. Early steel moment frame structures made these connections with bolts; since the 1950s connections have been made by heavy welds from beams to columns. Moment frames (steel or concrete) can be designed with various levels of ductility. ''Special'' (i.e., ductile) moment frames are typically required by current building codes in areas of high seismicity. ''Intermediate'' (i.e., limited ductility) and "ordinary" (i.e., little ductility or nonductile) moment frames are usually allowed only in areas of relatively low seismicity.

  Prior to the 1994 Northridge, California (USA), earthquake, steel moment frames were generally considered by many in the structural engineering profession to be among the most reliable seismic force resisting systems. During the Northridge earthquake, over 200 steel moment frame buildings experienced cracking in the welds of the moment connections (located at beam-column joints), which led to further cracking of beam and column flanges and webs. None of the damaged steel moment frame buildings collapsed or caused serious injuries or death. However, at least two buildings were subsequently demolished, and the repairs for the other identified structures were expensive. Some structural engineers believe a larger earthquake with longer duration would have resulted in at least partial collapse of some of these structures. Following the earthquake, the industry quickly issued interim guidelines for the design of new moment frame structures; these guidelines were incorporated into subsequent codes. Newer steel moment connections (e.g., having a reduced beam cross-section near the column, or employing cover or side plates) have been developed to mitigate the concern with brittle fracture of welds at beam ends.

  Retrofits of steel moment frame structures commonly address items such as:

  - Strengthening moment frames (especially the beam-to-column connections) or adding new elements (e.g., braced frames or shear walls)
  - Adding elements to correct significant irregularities in plan and elevation (e.g., soft stories)
  - Adding collectors in the roof or floor diaphragms to transfer forces at openings or to moment frames
  - Strengthening connections of frames to foundations.

- **Rigid Shear Wall/Rigid Diaphragm Structures**. This generic building type includes buildings with reinforced concrete or masonry walls and concrete diaphragms. Common vertical systems include concrete slabs supported by concrete framing and columns, or concrete-filled metal deck supported by steel framing and columns.

  The lateral force-resisting system typically consists of the rigid concrete diaphragms that distribute earthquake forces to reinforced concrete or masonry shear walls. Concrete walls can be cast-in-place or precast.

  The performance of rigid shear wall/rigid diaphragm buildings is highly dependent on the number of concrete or masonry shear walls, their location within the building, their configuration, the size and number of openings in the walls, and steel reinforcing details.

  Well-designed concrete or masonry walls have adequate reinforcing throughout (both horizontally and vertically) as well as special reinforcing around openings and at edges. Walls with extensive openings often are subject to significant cracking and spalling of the masonry units or concrete around openings. For walls greater in height than width, vertical reinforcing steel at edges typically is provided with horizontal steel ties that wrap around this steel and the concrete within to hold it together. The pattern in which the masonry is laid-up also is important. Most concrete masonry is laid-up in a running bond pattern, in which the joints of the masonry units in each layer are staggered relative to the layers above and below. This is a preferred form of construction. Walls incorporating a masonry pattern known as stack bond, in which the joints between units align vertically from the top of the wall to the bottom, will probably suffer a higher level of damage than walls utilizing running bond.

  Rigid shear wall/rigid diaphragm buildings with abrupt changes in lateral resistance often have performed poorly in earthquakes. Damage tends to be located in weak or flexible stories, or at locations where shear walls at upper levels do not continue to the foundation level. Buildings with walls distributed primarily at only two or three sides are subject to large torsional displacements (twisting) and have been severely damaged in past earthquakes.

  Retrofits of rigid shear wall/rigid diaphragm structures commonly address items such as:

  - Adding new elements (usually shear walls, sometimes steel braced frames)
  - Strengthening existing shear walls (e.g., by concrete overlay)
  - Adding elements to correct significant irregularities in plan and elevation (e.g., discontinuous shear walls or soft stories).

- **Rigid Shear Wall/Flexible Diaphragm Structures**. This generic building type includes buildings with reinforced concrete or masonry walls and wood-sheathed or metal deck diaphragms. They have historically had wood roofs in California (USA) and metal roofs elsewhere, but metal roofs are becoming more prevalent in California. They typically are low-rise structures (less than three stories) and often are one-story buildings. A common vertical system is wood sheathing supported by wood or steel members/trusses/columns. In many buildings metal deck without concrete fill (or with non-structural insulating concrete fill) is supported by steel framing and columns. Concrete tilt-up structures (buildings constructed with large, site-cast concrete panels that are “tilted-up” to form the exterior walls) are a typical example of this type of structure.

  The lateral force-resisting system usually consists of the flexible wood-sheathed or metal deck roof diaphragm that distributes lateral forces to concrete (often tilt-up) or masonry shear walls. These walls will be located at the building perimeter and also can be located at the interior of the building.

  The performance of rigid shear wall/flexible diaphragm buildings, particularly those with timber floors and roofs, is strongly related to the capacity of the wall anchorage to the roof and floor diaphragms. The heavy concrete or masonry walls tend to separate from the roof framing when subjected to seismic ground shaking perpendicular to the plane of the walls unless engineered connections (referred to as wall anchors) are provided to tie them together. When provided with good wall anchorage details, these buildings often perform well.

  Proper wall anchorage was not required by United States codes until after the 1971 San Fernando, California, earthquake, when many concrete tilt-up and masonry walls separated from their roof diaphragms. A series of code changes requiring improved anchorage of concrete and masonry walls to diaphragms was enacted following that earthquake to avoid these failures. Design forces for positive, direct roof-to-wall anchorage and continuity ties (first required in the 1973 *Uniform Building Code* [UBC]) were increased in the 1976 UBC for many structures, and were again increased in the 1991 UBC. In the 1994 Northridge, California earthquake, over 400 late-vintage concrete tilt-up structures experienced either partial or total collapse of their roof framing systems and the relevant UBC provisions were again revised in the 1997 edition.

Retrofits of rigid shear wall/flexible diaphragm structures commonly address items such as:

- Adding or strengthening anchorage from concrete or masonry walls to floor and roof diaphragms (roof-to-wall or floor-to-wall connections), and adding connections between members at the diaphragm interior (continuity ties)
- Adding collectors in the roof or floor diaphragms to transfer forces at openings or to shear walls
- Strengthening floor or roof diaphragms (e.g., by increasing the nailing of plywood sheathing to roof members)
- Adding elements (often shear walls or steel braced frames) to correct significant irregularities in plan (e.g., reentrant corners or open fronts).

- **Concrete Moment Frame Structures**. Concrete moment frames resist seismic forces by the bending of their beams and columns through the rigid beam-column joints, similar to steel moment frames. The vertical load-carrying system often consists of concrete slabs supported by concrete framing (joists, beams, and girders) and concrete columns. Rigid concrete diaphragms transfer earthquake forces to concrete moment frames, which may be distributed throughout the building or only in discrete locations.

  The performance of concrete moment frame structures in earthquakes is strongly dependent on detailing of reinforcing steel in beams, columns and connections. Frames that are designed to dissipate energy from major earthquakes without failure are termed “ductile”. Some concrete frame buildings constructed without ductile detailing or using precast concrete moment frames have been severely damaged and/or have collapsed in past earthquakes. Columns with insufficient ties to prevent buckling of longitudinal reinforcing steel after the concrete cracks and spalls frequently have been the source of major damage.

  Few considerations for seismic detailing were incorporated into the design of these structures until the early 1970s. Extensive modifications of code detailing requirements to provide ductility in concrete frames were made in the 1967 and subsequent editions of the *Uniform Building Code* (UBC). However, provisions requiring that concrete frames be designed as ductile frames in areas of high seismicity were first adopted in the 1973 UBC. Good detailing of reinforcement to provide ductility did not become common in California (USA) until the late 1970s. It is expected that concrete-frame structures designed after adoption of these criteria will perform substantially better than those designed prior to the more stringent codes. Ductile design of concrete moment frames may not have been required in other jurisdictions until much later; even today some allow construction of nonductile reinforced or precast concrete moment frames in high seismicity areas.

  Retrofits of concrete moment frame structures commonly address items such as:

  - Adding new elements (usually shear walls, sometimes steel braced frames or concrete moment frames), especially when existing concrete moment frames are non-ductile or precast
  - Strengthening existing non-ductile concrete frame or gravity columns (e.g., using a concrete/steel jacket or fiber composite wrap)
  - Adding elements to correct significant irregularities in plan and elevation (e.g., soft stories).

- **Unreinforced Masonry Structures**. The “unreinforced masonry” (URM) in this type of structure refers to building walls, which have no steel reinforcement. A vertical load-carrying system of wood sheathing supported by wood members and the masonry walls is common. Buildings of confined masonry construction would also be considered URM structures. In these buildings, unreinforced masonry wall panels (that will eventually bear loads from concrete floor and roof slabs) are constructed and have horizontal and vertical reinforced concrete ties (typically about the same thickness as the wall) cast directly against them on all four sides to confine the masonry. URM construction can also include a building having concrete or steel vertical load-carrying elements with bearing or infill URM shear walls (in these structures the masonry walls are constructed after the frame).

Walls constructed without steel reinforcing have consistently suffered severe damage during earthquakes. Walls can crack extensively due to in-plane shear forces, or can fall away from the building due to forces perpendicular to the wall. For URM structures without frames or confining ties, only the existence of numerous wood-frame interior partitions-typical of older brick office and apartment buildings but less typical in industrial structures has prevented collapse during strong ground shaking. Confined or infilled unreinforced masonry structures are less likely to collapse than structures where entire walls are URM, but in-plane cracking or out-of-plane failure of masonry walls can still result in large economic losses.

Construction of URM buildings generally has not been allowed in California (USA) since the 1933 Long Beach earthquake; however, they may not have been phased out in some seismically-active California, Oregon and Washington state (USA) localities until the mid-1950s. Later vintage URM construction was allowed in other regions of the United States (e.g., into the 1970's in Utah, and into at least the 1980's in the New Madrid Seismic Zone [NMSZ] states of Missouri and Tennessee). URM is also one of the least expensive building techniques and for that reason is still common, particularly confined masonry construction, even today in some places having a high seismic risk (e.g., Mexico, and Central and South America). Retrofits of URM structures typically are for collapse prevention only. Unreinforced masonry buildings located in areas of high seismicity probably will suffer high losses whether they have been retrofitted or not.

Retrofits of URM structures commonly address items such as:

- Adding or strengthening anchorage from URM walls to floor and roof diaphragms (roof-to-wall or floor-to-wall connections) and sometimes to partition walls, and adding connections between members at the diaphragm interior (continuity ties)
- Adding steel (usually) vertical members (strongbacks) or horizontal members (similar to wind girts) to transfer wall out-of-plane forces to the structure
- Stabilizing or strengthening URM walls with concrete overlay or fiber composite fabric
- Removing non-structural URM partition walls
- Adding new elements (usually shear walls, sometimes steel braced frames)
- Adding collectors in the roof or floor diaphragms to transfer forces at openings or to shear walls
- Strengthening floor or roof diaphragms (e.g., by increasing the nailing of wood sheathing to roof members or applying plywood over wood boards)
- Adding elements (often shear walls or steel braced frames) to correct significant irregularities in plan (e.g., open fronts)
- Bracing parapets (preventing them from falling - does not significantly reduce the economic loss).

Additional information regarding generic earthquake building types can be found in FEMA 154, which uses similar building classifications. Table C.4.3.3 correlates the nine categories discussed above (the concrete moment frame category above is split into reinforced concrete and precast concrete in the table) with FEMA 154 categories.

Table C.4.3.3. Comparison of Data Sheet 1-2 With FEMA 154 Earthquake Building Types

Data Sheet 1-2 EQ Building Type	*FEMA 154*	
	EQ Building Type	Description
Light Metal (LM)	S3	Light metal frame
Wood Frame (WF)	W1, W1A	Light wood frame, residential/commercial, <5000 ft^2/floor (465 m^2/floor)
	W2	Wood frame buildings, >5000 ft^2/floor (465 m^2/floor)
Steel Braced Frame (SBF)	S2	Steel braced frame
Steel Moment Frame (SMF)	S1	Steel moment-resisting frame
Rigid Shear Wall/Rigid Diaphragm (RSW/RD)	S4	Steel frame with cast-in-place concrete shear walls
	C2	Concrete shear wall
	RM2	Reinforced masonry with rigid diaphragms
Rigid Shear Wall/Flexible Diaphragm (RSW/FD)	PC1	Tilt-up construction
	RM1	Reinforced masonry with flexible floor and roof diaphragms
Reinforced Concrete Moment Frame - Ductile, Non-Ductile, or Unknown Ductility (CMF/RC/D, ND, or U)[2]	C1	Concrete moment-resisting frame
	C3	Concrete frame with unreinforced masonry infill
Precast Concrete Moment Frame (CMF/PC)[1]	PC2	Precast concrete frame
Unreinforced Masonry (URM)[2]	S5	Steel frame with unreinforced masonry infill
	C3	Concrete frame with unreinforced masonry infill
	URM	Unreinforced masonry bearing-wall buildings

1. FEMA 154 PC2 might sometimes be a RSW/RD
2. FEMA 154 C3 might sometimes be either a CMF/RC (likely non-ductile) or URM

C.4.4 Effects of Building-Specific Features on Generic Building Performance

The previous section provided basic information on performance of generic building types and some building-specific features that can change the typical performance expected. A discussion of some of the more common features (termed ″irregularities″) that may modify typical building performance follows. Additional information regarding building irregularities can be found in FEMA 154, FEMA 389 (*Primer for Design Professionals: Communicating with Owners and Managers of New Buildings on Earthquake Risk*, January 2004) and FEMA 454.

- **Vertical and Plan Irregularities**: Descriptions of vertical and plan irregularities and design requirements to be applied are commonly provided in building codes. If these irregularities are not properly accounted for in design, the consequences on performance can be severe. If the design is modified to account for these features (through special detailing, etc.) the effect of the irregularity on performance can be minimized. However, damage still may be locally heavier at the irregularity. Common examples of vertical and plan irregularities are listed below.

  Vertical irregularities:

  - Stories that are significantly weaker/softer than adjacent stories (i.e., ″soft stories″); commonly occurs in the first story of a building
  - Vertical elements (e.g., shear walls) that are not continuous to the foundation, don't ''stack'', or that have radically different lengths in adjacent stories
  - Very significant mass changes between adjacent floors (not including lighter roofs)
  - Short concrete columns (e.g., at parking garage ramps and where partial height masonry infill adjacent to columns restricts column movement)

  Plan irregularities:

- Torsionally irregular rigid or semi-rigid diaphragm (i.e., the layout of lateral elements [e.g., shear walls] is such that there will be significant twisting or rotating of the floors or roof)
- Large re-entrant corners (e.g., L- or U-shaped configurations)
- Significant discontinuities or variations in floor or roof diaphragm stiffness or strength (e.g., large cutouts in the floor or roof interior)
- Vertical elements (e.g., shear walls) not aligned with the major orthogonal axes of the lateral force-resisting system (i.e., skewed walls)

- **Poor condition or modifications**: While newer structures may have been built in conformance with earthquake code provisions, seemingly minor modifications may severely reduce the structure's ability to withstand an earthquake if they are performed without the review of a knowledgeable structural engineer. Such modifications as retrofitted doorways and windows (that may result in removed braces or weakened shear walls); or added mezzanines, penthouses, and roof-supported heavy equipment would be examples of this. A large loss in the 1971 San Fernando, California (USA) earthquake was attributed to a concentration of air-handling equipment added in one corner of the roof. Similarly, poor maintenance of the structure that results in significant deterioration of structural members may have an adverse effect on seismic performance of these members.
- **URM Partitions/Infill**: Non-structural unreinforced masonry (URM) infill or partitions (e.g., hollow clay tile) most often occur in buildings of similar vintages as discussed for URM buildings in Section C.4.3.3, but can be found in buildings having any type of building lateral force-resisting system. Often the URM materials are covered with plaster or stucco, or are painted such that it is difficult to recognize them in an existing building. During an earthquake the non-structural URM cracks in-plane due to deflections of the building, and can crack or fail outofplane as well. Extensive cracking of non-structural URM has occurred in relatively moderate shaking and can require costly removal and replacement of substantial amounts of URM even if the building structure itself is relatively undamaged.
- **Other factors**: Several other factors can modify generic performance (for better or worse). Examples include:

  Negative factors:

  - Unusually heavy façade, mezzanines, etc. in an otherwise light weight building (e.g., light metal structure)
  - Pounding between adjacent buildings
  - Unusually long or narrow roofs or floors
  - Incomplete or inadequate lateral load paths
  - Poor quality building materials
  - Use of relatively earthquake-vulnerable material (e.g., some precase concrete)

  Positive factors:

  - Enhanced design (e.g., utilizing building code criteria for critical buildings)
  - Base isolated buildings
  - Utilization of energy dissipation devices within a building

C.5 Earthquake Performance and Restraint of Contents

The following information regarding past earthquake performance of contents, design of restraint (e.g., anchorage or bracing) for equipment and single- and double-row racks, and post-installed concrete anchors is intended to provide a brief overview of some more important and common aspects of earthquake performance and design for these items.

Although most industrial equipment is fairly rugged and will perform well if anchored, some objects may contain fragile internal components (e.g., steppers or diffusion furnaces in semiconductor fabrication facilities) that can be affected by strong shaking. Methods to mitigate damage to these fragile components is not addressed below.

It is not intended that the discussion below will cover all earthquake vulnerabilities, code provisions or manufacturer requirements. Consult publications documenting past earthquake performance (e.g., by the Earthquake Engineering Research Institute [EERI] and the New Zealand Society for Earthquake Engineering [NZSEE]), building codes, publications explaining earthquake design of equipment restraint (e.g., those of the U.S. Federal Emergency Management Agency [FEMA]) and manufacturers' literature for a full understanding of earthquake design provisions.

Examples of documents showing generic anchorage techniques are the following United States Federal Emergency Management Agency (FEMA) documents:

- FEMA E74/December 2012, *Reducing the Risks of Nonstructural Earthquake Damage*
- FEMA 412/December 2002, *Installing Seismic Restraints for Mechanical Equipment*
- FEMA 413/January 2004, *Installing Seismic Restraints for Electrical Equipment*
- FEMA 414/January 2004, *Installing Seismic Restraints for Duct and Pipe*

These documents are available for download at: www.fema.gov.

C.5.1 Performance of Contents in Past Earthquakes

Performance of equipment observed by FM in recent earthquakes has been similar to the performance documented by others (e.g., EERI, NZSEE). Most properly-restrained (i.e., anchored or braced) equipment has sustained relatively minor damage during strong ground shaking. Equipment is often inherently rugged due to operational considerations (e.g., pumps, compressors, generators, shop equipment), but most other equipment (e.g., electrical switchgear and MCC, dry transformers, batteries) has also shown sufficient internal ruggedness to withstand earthquake shaking. There are, however, some equipment items for which the equipment itself has been vulnerable to damage during earthquake shaking. Observations from past earthquakes have demonstrated that:

- Tall narrow objects (e.g., electrical cabinets, computer servers, some shop equipment) are most prone to overturning, especially when mass is concentrated near the top of the object. Similarly, pedestals of tall raised access floors may collapse during strong ground shaking, due to a lack of lateral bracing. Damage to toppled equipment or to items it impacts, and damage to equipment supported on failed raised access floors, can be very high.

- Low profile equipment typically slides or shifts. Unless it falls from its support (e.g., table, equipment pad, vibration isolators) or impacts adjacent items, damage to the equipment itself is often minor. However, loss can become significant if the equipment falls, if movement damages attachments to the equipment (e.g., piping, conduit, conveyors, rigid connections between batteries) or if large amounts of equipment require realignment.

- Equipment hung from overhead structures (e.g., suspended ceilings, piping) will swing in an earthquake. In some cases (e.g., individual runs of welded or soldered piping with short hangers, cable trays with short hangers, conduit, individual light fixtures), bracing is not necessary to prevent damage as long as vertical support is maintained and the items don't impact or impose large loads on other equipment. However, piping systems having threaded or grooved mechanical couplings and smaller pipe branching off larger pipe (similar to fire sprinkler systems), unbraced suspended ceilings, and items that can swing into other objects have proven to be highly damageable. Additionally, distribution systems (e.g., piping) that span across seismic joints or between individual buildings can be damaged if the differential movement between the structures is not accommodated (e.g., by a flexible pipe loop).

- Vibration-isolated equipment can be particularly vulnerable to earthquake damage. Horizontal forces are amplified in such equipment and the vibration isolators themselves are often not adequate internally to resist horizontal forces. In addition, vibration isolators, or independent bumpers, can be tall and narrow. Forces applied at or near the top of these (i.e., where the equipment is attached or impacts) will tend to overturn the vibration isolators or bumpers themselves. If they are narrow, this can result in high tension forces in the bolts that attach the vibration isolator or bumper to the structure.

- Forces are amplified by structures; items at the tops of buildings are more likely to shift, overturn or swing than items at grade.

- In some cases, the equipment itself may be vulnerable to damage from earthquake shaking such that rigid anchorage is not a total solution or such that rigid anchorage is not a preferred solution. Some examples include:
    - Tanks - For leg-supported tanks, buckling of legs and damage at their attachment to the tank sometimes occurs if legs are not adequately braced, or if legs or the tank shell are constructed of thin steel or weak material (e.g., plastic). For base-supported tanks (e.g., stainless steel wine tanks) damage to thin shells (e.g., “elephant foot” buckling) or ripping of shell-to-floor plate connections are common. In addition, tanks can be damaged if pipe is rigidly attached or if access walkways are rigidly attached between the tanks.
    - Storage racks are often marginally designed for earthquake forces. Racks have failed even when they are anchored, especially when they are heavily loaded or damaged (e.g., from forklift impact). But, except for providing base anchorage and replacing damaged frames, retrofit of existing racks themselves is usually not cost effective. Damage to inadequately-designed racks may include column or brace buckling as well as tearing of beam-to-column connections or column-to-base plate welds. Retrofit measures to prevent items from falling from racks are often not easily implemented due to their impact on operations.
    - Some equipment is inherently fragile, or has fragile internal components. In certain cases, rigidly anchoring this equipment may invalidate its warranty (rigid anchorage increases the internal forces within the equipment) and more complex solutions, like base isolation, are needed to mitigate earthquake damage (an example might the steppers used in semiconductor fabrication). At times, the only practical solution to mitigate earthquake losses for equipment incorporating fragile components may be to stock spare parts (examples are tall porcelain transformer bushings [which can be qualified for use in seismic zones but may still be seismically vulnerable] and quartz tubes in diffusion furnaces used in semiconductor fabrication).

C.5.2 Equipment Restraint

All connections should be positive direct connections for good seismic performance. Friction resulting from gravity loads should not be considered to provide resistance to seismic forces except at the foundation-to-soil interface as allowed by the geotechnical engineer.

In ASCE 7-22 restraint (e.g., anchorage or bracing) of typical architectural, mechanical, and electrical components and systems (e.g., electrical cabinets and transformers, manufacturing machinery, piping systems, walls, and raised access floors) is designed for forces from earthquake shaking using the following method. Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) are included in the discussion below. See the Appendix A glossary for full definitions of LRFD and ASD.

Similar to the retrofit criteria discussed in Section C.4.3.2 for structures, ASCE 41 also addresses seismic retrofit of existing equipment and non-structural systems. For these, the performance levels are life-safety, position retention and operational. Position retention, according to ASCE 41, “essentially mirrors the requirements of ASCE 7 non-structural seismic provisions for cases where I_p is taken as 1.0.” By contrast, the life safety performance level focuses mainly on preventing equipment from falling or blocking egress paths and in many cases will allow significant and costly damage. Therefore, it is recommended that retrofits of typical equipment be made to at least the position retention performance level (i.e., to the ASCE 7 provisions discussed below).

Restraint of equipment, piping, etc. is still required in buildings that are base isolated or have energy dissipating devices installed. Design forces can be conservatively determined as outlined below, or more detailed calculations per ASCE 7-22 can be made to establish whether lower design forces could be used. Restraint design and flexibility provided for elements spanning across the isolation interface (e.g., piping or stairs) needs to consider expected differential displacements.

C.5.2.1 Determine the Horizontal Equipment Seismic Design Force (LFRD)

The LRFD horizontal seismic design force, which should be applied independently in two orthogonal directions, is determined using the general equation:

$$F_p = 0.4S_{DS}I_pW_p\left[\frac{H_f}{R_\mu}\right]\left[\frac{C_{AR}}{R_{po}}\right]$$

With

$$0.3S_{DS}I_pW_p < F_p < 1.6S_{DS}I_pW_p$$

Where:

F_p = the horizontal seismic design force (LRFD, see above) centered at the component's center of gravity (C.G.) and distributed relative to component's mass distribution.

S_{DS} = the site (soil) adjusted, 5% damped, design spectral response acceleration at a short (0.2 second) period, expressed as a portion of the gravitational acceleration (g).

I_p = component seismic importance factor. The code-required value varies from 1.0 ("normal" equipment) to 1.5 (essential, life-safety, or hazardous material-containing equipment). Since a higher value increases the likelihood that equipment anchorage will be adequate, consider using an importance factor greater than 1.0 (e.g., 1.25 or 1.5) for valuable or important equipment (e.g., storage racks protected by in-rack sprinklers) even if not code-required.

W_p = component operating weight.

H_f = the factor for force amplification as a function of structure height (1.0 for components at grade, and varies with height for other levels). Where the approximate fundamental period of the supporting building or nonbuilding structure is unknown, $H_f = 1 + 2.5*(z/h)$.

z = the height above the base of the structure to the point of attachment of the component/equipment.

h = the average roof height of structure with respect to the base.

Rμ = the structure ductility reduction factor (1.0 for components at grade, and 1.3 for those above grade when the seismic force resisting system [SFRS] of the building or nonbuilding structure is not specified). For known SFRS, calculate using the formula as per ASCE 7-22.

C_{AR} = the component resonance ductility factor from ASCE 7-22 that converts the peak floor or ground acceleration into peak component acceleration. (Varies from 1.0 to 1.8 for equipment supported at grade, and from 1.00 to 2.20 for equipment supported above grade, based on ASCE 7-22 tables).

R_{po} = the component strength factor. (Varies from 1.3 to 2.0 based on ASCE 7-22 tables; 1.5 can be considered for rigid equipment.)

C.5.2.2 Determine the Vertical Equipment Seismic Design Force (LFRD)

The horizontal lateral force determined in section C.5.2.1 should be combined with a vertical earthquake design force (E_v) of $0.2 \cdot S_{DS} \cdot W_p$ (LRFD), per the load combinations specified in ASCE 7-22.

C.5.2.3 Determine Total Equipment Restraint Design Forces (LRFD or ASD)

Design forces for restraints (e.g., anchors or braces) are determined using appropriate LRFD or ASD load combinations. Note that the horizontal earthquake force can come from any direction and the design should be based on the direction that results in the most conservative restraint.

Assuming that there is no live load or snow load (i.e., the only forces on the equipment are from the dead load of the item and the earthquake forces) anchors are designed for the load combinations given below. Note that the LRFD horizontal (F_p) and vertical (E_v) seismic design forces are multiplied by 0.7 to determine equivalent approximate ASD seismic design forces. Also note that for anchorage, Combination B (LRFD) and Combination D (ASD) likely control since they will result in the maximum tension on the anchor that will need to be combined with the shear in the anchor.

<u>LRFD Combinations</u>:

[Combination A] $(1.2 + 0.2 \cdot S_{DS}) \cdot W_p + \Omega_o \cdot F_p$

[Combination B] $(0.9 - 0.2 \cdot S_{DS}) \cdot W_p + \Omega_o \cdot F_p$

<u>ASD Combinations</u>:

[Combination C] $(1.0 + 0.14 \cdot S_{DS}) \cdot W_p + 0.7 \cdot \Omega_o \cdot F_p$

[Combination D] $(0.6 - 0.14 \cdot S_{DS}) \cdot W_p + 0.7 \cdot \Omega_o \cdot F_p$

Where:

S_{DS}, W_p and F_p are as defined previously.

Ω_o = overstrength factor used to amplify prescribed seismic forces that varies from 1.0 to 2.5. Where anchors do not attach to concrete or masonry, take the factor as 1.0. For anchors to concrete or masonry that are deemed non-ductile (e.g., most post-installed anchors), the overstrength factor (from ASCE 7 tables) is generally 2.0 or 1.75 for LRFD load combinations (and 2.0/1.2 = 1.67 or 1.75/1.2 = 1.46 for ASD load combinations). Rather than applying the Ω_o factor to the loads, an equivalent method when all loads on the anchors result from earthquake would be to reduce capacities of post-installed by the Ω_o factor (as done in Table C.5.4.2).

ASCE 7-02 (a previous edition of ASCE 7) set a criterion that post-installed concrete anchors should have an embedment of at least eight times the bolt diameter (D_b) to be considered “deep” anchors that act in a ductile manner. Later editions of ASCE 7 have modified this criterion, such that the embedment depth and other features necessary for the anchor to be considered ductile require prequalification in accordance with American Concrete Institute (ACI) Standard 355.2, *Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete*. Most post-installed concrete (or masonry) anchors would likely not meet the ACI 355.2 requirements for ductility, so the overstrength factor should generally be applied where anchorage is made to concrete or masonry.

Some components may list capacities based on Load and Resistance Factor Design (LRFD), while others may list capacities based on Allowable Stress Design (ASD). It is important to understand how capacities are reported and use the load combinations appropriate for that listing.

Post-installed concrete anchor capacities given in this data sheet are based on ASD.

Typical calculations of design forces (LRFD) for various equipment as per ASCE 7-22 are provided below:

Example 1: Typical rigid mounted, floor-supported equipment:

a) C_{AR} = 1.0, R_{po} = 1.5 and Ω_o = 2.0 (LRFD) or 1.67 (ASD) for floor-supported storage cabinets and for substantial items constructed of high-deformability materials (for example transformers, generators, batteries, manufacturing and process machinery, wet-side HVAC, chillers and boilers).

b) C_{AR} = 1.4, R_{po} = 2.0 and Ω_o = 2.0 (LRFD) or 1.67 (ASD) for equipment constructed of sheet metal framing (for example motor control centers, switch gear, air-side HVAC, air handlers and air conditioning units).

The above two sets of values result in nearly the same required seismic design forces. With the above values, F_p (LRFD) is approximated as indicated below. (Note that for design of non-ductile anchorage to masonry or concrete, the values below must be doubled, or allowable anchor capacities must be halved; because Ω_o = 2.0),

- At grade level, $F_p = 0.30 \cdot S_{DS} \cdot I_p \cdot W_p$
- Above grade and up to 0.5h, $F_p = 0.45 \cdot S_{DS} \cdot I_p \cdot W_p$, where h is the building height
- At the roof, $F_p = 0.75 \cdot S_{DS} \cdot I_p \cdot W_p$
- For other levels, F_p is determined by linear interpolation

Example 2: Typical vibration-isolated equipment with springs:

a) C_{AR} = 1.8, R_{po} = 1.3 and Ω_o = 2.0 for “at-grade”

b) C_{AR} = 2.2, R_{po} = 1.3 and Ω_o = 2.0 for “above grade”

With the above values, Fp (LRFD) is approximated as indicated below. (Note that for design of non-ductile anchorage to masonry or concrete, the values below must be doubled, or allowable anchor capacities must be halved; because Ω_o = 2.0),

- $F_p = 0.6 \cdot S_{DS} \cdot I_p \cdot W_p$ at or below the base of the building
- $F_p = 1.60 \cdot S_{DS} \cdot I_p \cdot W_p$ at the roof
- F_p is determined by linear interpolation between grade and the roof

Note for the above two examples, the F_p (LRFD) values would be multiplied by the 0.7 load factor to get the ASD loads. If attachment is to concrete or masonry using non-ductile anchors, the resulting ASD loads would then be multiplied by the ASD Ω_o of 1.67 (or the allowable anchor capacities in Table C.5.4.2 would be divided by 1.67).

Example 3: Raised access floors:

a) C_{AR} = 1.0, R_{po} = 2.0 and Ω_o = 2.0 (LRFD) or 1.67 (ASD) for special access floors* (See C.5.5.5 for requirements.)

b) C_{AR} = 2.2 at or below grade and 2.8 for above grade, R_{po} = 1.5 and Ω_o = 1.5 (LRFD) or 1.5/1.2 = 1.25 (ASD) for all other raised access floors

* Special access floors have positive connections (i.e., mechanical fasteners, welds or concrete anchors) from floor stringers to pedestals, pedestals to floors and at ends of braces. Additionally, floor stringers must be adequate for the axial force, and bracing and pedestals must be structural or mechanical shapes produced to specifications that require minimum mechanical properties (e.g., not electrical tubing).

C.5.3 Single-Row and Double-Row Storage Racks

Storage racks and their anchorage can be designed using ASCE 7-22 directly. However, racks can also be designed, with some modifications (e.g., to use an overstrength factor for anchorage) required by ASCE 7-22, per ANSI/RMI MH16.1-2021 (American National Standards Institute, *Design, Testing and Utilization of Industrial Steel Storage Racks).* Additional information can be found in FEMA 460, *Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public* (September 2005).

The discussion below is mainly focused on determining appropriate rack anchorage, not design of the racks (e.g., members, welds) themselves. Additionally, evaluation of deflections is beyond the scope of this data sheet. The registered engineer responsible for the rack structural design must determine these deflections (amplifying those calculated using design forces as necessary) and provide clearance adequate to prevent pounding, etc., based on appropriate analysis. ASCE 7-16 states that the assumed total relative displacement for steel storage racks should not be less than 5% of the structural height above the base unless justified by analysis. ASCE 7-22 states that the assumed total relative displacement for steel storage racks should not be less than 5% of the structural height above the base, unless justified by analysis.

C.5.3.1 Racks in Upper Floors: Horizontal Design Forces

Racks located above grade (located on upper floors) in a building are designed per ASCE 7-22 and ANSI/RMI MH16.1-2021. They should meet the force and displacement requirements of the ASCE 7-22 for nonbuilding structures supported by other structures, including the force and displacement effects caused by amplifications of upper-story motions. The horizontal forces are determined using a_p = 2.5, R_p = 4.0 in the cross-aisle (transverse) direction or R_p = 6.0 in the down-aisle (longitudinal) direction, per ANSI/RMI MH16.1-2021 and the equation for F_p previously discussed for equipment anchorage. This condition will not be discussed further. For more information see ASCE 7 and ANSI/RMI MH16.1.

C.5.3.2 Racks Located at Grade: Horizontal Design Forces

Storage racks are most often located at grade and are, therefore, designed as independent, non-building structures. Using ANSI/RMI MH16.1-2021, the general form of the design equation (LRFD) for horizontal forces on racks at grade is:

$$V = C_s I_e W_s \quad \text{and} \quad C_s = \frac{S_{DS}}{R} \quad \text{(maximum)}$$

In general, minimum $C_s = 0.044\, S_{DS} \geq 0.01$

But when $S_1 \geq 0.6g$, the value of $C_s \geq 0.8.S_1/R$ (per ASCE 7-22 coefficient).

Where:

V = the total design horizontal force or shear (LRFD) at the base. (Multiply by 0.7 to get ASD forces; see load combination section)

C_S = the seismic response coefficient

W_S = the total gravity load on the rack

S_{DS} = as defined previously

I_e = as defined previously (1.5 if the rack stores hazardous material, racks open to public or racks in essential facility; 1.0 for all other structures)

R = response modification coefficient (4.0 in the cross-aisle braced [transverse] direction and 6.0 in the down-aisle moment frame [longitudinal] direction, per ANSI/RMI MH16.1-2021)

S_{D1} = the site (soil) adjusted, 5% damped, design spectral response acceleration for a period of one second, expressed as a portion of the gravitational acceleration (g)

S_1 = the mapped, 5% damped, spectral response acceleration, assuming a soil at the boundary of Site Classes B and C, for a period of one second, expressed as a portion of the gravitational acceleration (g)

The total design lateral force (V) is vertically distributed along the height of the rack based on the formula:

Force at level “X” (F_x):

$$(F_x) = C_{vx} \cdot V \quad \text{and} \quad C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{(maximum)}$$

Where:

w_x, w_i = the weight at level x or level i

h_x, h_i = the height (above the base of the rack) of level x or level i

k = an exponent (per ASCE 7-22) related to the fundamental period of the rack (T) that equals 1.0 for T ≤ 0.5 seconds, 2.0 for T > 2.5 seconds and is determined by linear interpolation if “T” is between these points.

Where the weight at each rack level is equal and the fundamental period of the rack is 0.5 seconds or less, the force distribution on the rack will be roughly an inverted triangular shape. In ANSI/RMI MH16.1-2021, the exponent “k” is always taken as 1.0.

Steel storage racks are designed for two cases as per ASCE 7-22:

A. Weight of the rack plus every storage level loaded to 67% of its rated load capacity.

B. Weight of the rack plus the highest storage level only, loaded to 100% of its rated load capacity.

Most racks are light in weight compared to their rated load capacity; neglecting the weight of the rack will not typically introduce large inaccuracies in anchorage requirements.

C.5.3.3 Determine Total Storage Rack Restraint Design Forces (LRFD or ASD)

Design forces for rack anchorage are determined using appropriate LRFD or ASD load combinations. Note that the horizontal earthquake force can come from any direction and the design should be based on the direction (cross-aisle [transverse] or down-aisle [longitudinal]) that results in the most conservative anchorage. Forces are assumed to be applied in either the transverse or longitudinal direction; they are not applied in both directions simultaneously.

The load combinations in Section C.5.2.3 can also be used for storage rack anchorage. Replace W_p and F_p in the Section C.5.2.3 load combinations with W and V, respectively, determined for the rack in the previous section.

ASCE 7-22 modifies ANSI/RMI MH16.1-2021, requiring the overstrength factor (Ω_o) for storage rack anchorage be taken as 2.0 (LRFD) or 1.67 (ASD) for anchors to concrete or masonry that are deemed non-ductile (e.g., most post-installed anchors). For ductile concrete or masonry anchors, and for other anchors (e.g., bolts to steel) it is suggested that the overstrength factor be taken as 1.3. This is based on the likelihood that the rack design forces will be required to be increased by a redundancy factor (ρ) of 1.3, and this increased force should also be used to determine anchorage. If analysis of the racks shows that a redundancy factor of 1.0 is appropriate, then Ω_o = 1.0 can be used instead.

C.5.4 Anchorage to Concrete

Note: see also Data Sheet 2-8, *Earthquake Protection for Water-Based Fire Protection Systems*, Section 3.1.9, for additional information on anchoring to concrete, especially when using post-installed concrete anchors.

Because cracks will form in concrete during an earthquake, anchors that are cast into the concrete during construction (i.e., cast-in-place [CIP] anchors) or installed after the concrete is set (i.e., post-installed anchors) must be designed and/or tested to determine their resistance for the cracked concrete condition. Anchors must be designed and/or listed for use in regions of moderate to high seismic risk (e.g., ASCE 7 Seismic

Design Category [SDC] C-F; or UBC Zones 2B, 3 and 4). Anchors that are adequate only in ASCE 7 SDC A and B; or UBC Zones 0, 1, and 2A are not acceptable.

Anchors that are not qualified for cracked concrete and seismic applications should not be used in FM 50-year through 500-year earthquake zones.

C.5.4.1 Cast-In-Place (CIP) Anchors and Inserts

The capacity of relatively simple CIP anchors or inserts (e.g., headed bolts or bolt groups) can be determined using calculations in accordance with ASCE 7 and ACI 318, *Building Code Requirements for Structural Concrete* (or international equivalent). In regions of moderate and high seismic risk (e.g., SDC C through F in ASCE 7), ACI 318 requires the assumption of cracked concrete. Additionally, if the strength of the steel bolt or yielding of the attachment does not control the design, the LRFD anchor capacity is then limited to 40% of the strength of the anchor calculated assuming concrete failure.

Proprietary CIP inserts exist (e.g., cold-formed steel) for which determining the capacity is not straightforward. In these cases allowable capacities are typically determined using calculations (e.g., per ACI 318) in combination with testing. An example of a standard used to qualify specialty inserts is the International Code Council Evaluation Services (ICC-ES) standard AC446 - *Acceptance Criteria for Headed Cast-in Specialty Inserts in Concrete*.

C.5.4.2 Post-Installed Concrete Anchors

Several different types of post-installed concrete anchors are available, and can be used to resist earthquake forces only when they have been tested and prequalified for use in cracked concrete in seismic zones. Anchors that could be included are expansion or wedge, sleeve, shell or drop-in, screws and threaded head screws, undercut and adhesive.

Powder-driven fasteners (see glossary) are also post-installed anchors, similar in shape to a nail, propelled into steel or concrete elements. These do not have high allowable capacities, have typically performed poorly in past earthquakes, and are not appropriate to use in seismic zones).

Some general considerations for post-installed concrete anchors include the following:

A. Not all post-installed concrete anchors are appropriate for use in resisting seismic forces. Testing the anchors to establish their capacities when installed under ideal conditions, their reliability if installed poorly, and their performance under service load conditions (such as cyclical seismic forces) is necessary. In the United States, tests per American Concrete Institute (ACI) Standard 355.2, *Qualification of Post-Installed Mechanical Anchors in Concrete*, and Standard 355.4, *Qualification of Post-Installed Adhesive Anchors in Concrete*, are used to establish parameters for anchor design. ICC-ES Acceptance Criteria AC193 (mechanical anchors) and AC308 (adhesive anchors) are largely equivalent to the ACI standards. When used in FM 50-year through 500-year earthquake zones, allowable tension and shear capacities should be determined in cracked concrete using the seismic tests given in the ACI or ICC-ES criteria. Anchors should be listed as adequate for use in regions of moderate to high seismic risk (e.g., ASCE 7 SDC C through F). Anchors are not acceptable if listed only for use in ASCE 7 SDC A and B. ACI 318 is then used along with information such as concrete strength, embedment depths, etc. to determine the capacity of an anchor or group of anchors. In areas outside the United States, equivalent local building code standards may be used.

B. All manufacturers have specific requirements regarding their anchors that must be followed during design and installation.

C. In order to confirm a quality installation and meet building code requirements, anchors must typically be installed with special inspection per the manufacturers' requirements; for other bolts special inspection is optional. Since the quality of installation is critical to performance, it is strongly suggested that installation of all post-installed anchors be inspected. Further, it is suggested that, as part of the inspection process, the torque be verified for 5% of the installed anchors (minimum of 4 per day). Tension testing these anchors to twice the design load is an acceptable alternative.

D. Before installing anchors in precast/prestressed planks or post-tensioned concrete slabs, make a proper non-destructive evaluation to locate cables, tendons, and/or anchorages so they will not be damaged. Locate embedded items (e.g., conduit, etc.) prior to anchor installation. Avoid cutting reinforcing steel, particularly in elevated slabs or beams.

E. Do not use adhesive anchors in areas with conditions that adversely affect their capacity (e.g., ambient temperatures above 100°F [38°C], substantial radiation, or chemical exposure). Do not use adhesive anchors in overhead installations supporting gravity loads.

F. A common type of post-installed concrete anchor is an expansion or wedge anchor. Allowable forces for concrete expansion or wedge anchors often are reported for a service load (i.e., ASD) condition. There is a great deal of variability in embedment depths, capacities, methods of combining shear and tension forces to determine adequacy of these anchors, etc. The following conditions, though not universal, are fairly representative for concrete expansion or wedge anchors:

- Anchor bolt capacities depend on embedment depth, and typically several embedment depths are listed for each anchor. It is strongly recommended that a minimum embedment depth of 6 to 7 times the bolt diameter (see Table C.5.4.2) be used to increase the likelihood that the anchor will behave in a ductile manner as discussed above.
- Currently, determination of allowable shear (V_A) and tension loads (T_A) is based on a very complex methodology (e.g., such as that found in ACI 318) and depends on multiple variables (concrete strength, depth of embedment, edge distance, anchor spacing, placement of steel reinforcement, etc.). It is therefore difficult to provide generic capacities for post-installed anchors. The reader is directed to ACI 318 if a complete understanding of the analysis is needed.
- Use of an earthquake increase factor on calculated non-earthquake capacities for tension (designated as T_{EI}) and shear (designated as V_{EI}) was common in the past, but typically is no longer allowed.
- The anchor stress ratio (S_R) usually is determined through an interaction equation accounting for the actual tension and shear forces on the anchor. In the past, the form of the interaction equation typically was: $(T_{actual} / [T_{TE} \cdot T_A])^Y + (V_{actual} / [T_{TE} \cdot T_A])^Y \leq 1.0$. The value of the exponent “Y” varied from 1 to 2, but commonly was 5/3 for expansion/wedge anchors. Currently, ACI 318 requires the use of a linear interaction formula of the form $(T_{actual}/T_A) + (V_{actual}/V_A) \leq 1.2$ (with the additional restriction that neither $[T_{actual}/T_A]$ nor $[V_{actual}/V_A]$ can exceed 1.0). An S_R greater than that allowed indicates an overstress condition.
- Anchors need to be installed an adequate distance from concrete edges. This distance varies based on the anchor, concrete type, embedment, reinforcement, etc. Use a minimum edge distance of 12 times the bolt diameter unless otherwise allowed by the manufacturer and calculations are provided to verify adequacy.
- Requirements for spacing between anchors also vary depending on the manufacturer, concrete strength, etc. Use a minimum spacing of 8 times the bolt diameter unless otherwise allowed by the manufacturer and calculations are provided to verify adequacy.
- When multiple anchors are installed near each other, the tension and shear allowed for the group of anchors will be less than the capacity of a single anchor multiplied by the number of anchors used. There can be significant variation in the reduction factor, but for very close spacing the group reduction factor on T_A or V_A could be significant. For an effective group reduction factor of 0.5, if the capacity of one anchor = T_A, the capacity of two anchors may be as little as $(0.5) \cdot (2) \cdot T_A$ or the same as a single anchor. The exact reduction for a group of anchors must be calculated based on the configuration.
- The thickness of concrete typically is required to be approximately 1.5 times the depth of embedment of the anchor.

Again, concrete expansion anchors vary in their allowable capacity, but representative average values for single expansion or wedge anchors are given in Table C.5.4.2.

Table C.5.4.2. Approximate ASD Capacities vs. Earthquake Loading for a Single Post-Installed Concrete Expansion or Wedge Anchor in 3000 psi (20.7 MPa) Normal Weight Concrete[1]

Nominal Bolt Diameter in. (mm)	*Minimum Nominal Embedment[2] in. (mm)*	*Tension Allowable[3] Lb (kN)*	*Shear Allowable[4] Lb (kN)*
¼ (6)	1-3/4 (44)	200 (0.89)	250 (1.1)
3/8 (10)	2-1/4 (57)	450 (2.0)	500 (2.22)
½ (12)	3-1/2 (89)	600 (2.67)	1100 (4.89)
5/8 (16)	3-3/4 (95)	900 (4.0)	1500 (6.67)
¾ (20)	4-1/2 (114)	1300 (5.78)	2200 (9.79)

Notes:
1. Table values are for earthquake loading, using Allowable Stress Design (ASD) assuming cracked concrete, for single anchors determined based on averages across several manufacturers. The table ASD values are based on 0.42 times the normally allowed LRFD capacity, and so already account for the 0.7 LRFD to ASD load factor adjustment and the 1/1.67 = 0.6 adjustment for the ASD overstrength factor (Ω_0) - see Section C.5.2.3 and Data Sheet 2-8, Section 3.1.9 for more information. Where attachments are made to light weight concrete (e.g., light weight concrete-filled metal deck) use 60% of table values.
2. Manufacturers typically supply bolts in many lengths; choose bolts with a nominal embedment (i.e., embedment before the anchor is set) at least equal to that shown in the table (nominal embedment values should typically be 6 to 7 times the bolt diameter [$6D_b$ to $7D_b$]). Use anchor distance from concrete edges of at least $12{\cdot}D_b$ For anchor groups (anchors spaced closer than $18{\cdot}D_b$) use an anchor spacing of at least $8{\cdot}D_b$ and use 75% of the capacity given in the table (i.e., for a two-anchor connection, take the capacity), as [0.75] ·[2]·[allowable from table]). Analysis for a specific installation may result in capacities, embedments, edge distances, and / or spacings that vary from those assumed for this table.
3. Tension allowable assumes a high-quality installation (e.g., achieved by requiring special inspection or by verifying installations by testing a percentage of installed anchors). If concrete can be demonstrated to remain uncracked, values in the table for tension may be increased by a factor of 1.4; no other earthquake increase factor is applicable.
4. Shear allowable assumes a distance from concrete edges of at least $12{\cdot}D_b$. There is no increase allowed when concrete can be demonstrated to remain uncracked.

C.5.5 Generic Equipment Anchorage

Damage to equipment from earthquakes can be minimized by anchoring them to the structure to prevent toppling, sliding and falling. Most properly restrained (i.e., anchored or braced) equipment has sustained relatively little damage during strong ground shaking.

The following documents from the United States Federal Emergency Management Agency (FEMA) show generic anchorage techniques. These documents are available for download at www.fema.gov

- FEMA E74 (December 2012), *Reducing the Risks of Nonstructural Earthquake Damage*
- FEMA 412 (December 2002), *Installing Seismic Restraints for Mechanical Equipment*
- FEMA 413 (January 2004), *Installing Seismic Restraints for Electrical Equipment*
- FEMA 414 (January 2004), *Installing Seismic Restraints for Duct and Pipe*

The equipment can be anchored to a building or structure in different ways, such as rigid floor mounted/pad mounted, wall mounted, vibration isolated, suspended and placed on raised floors. Raised floor bracing and anchorage details are also provided in this section.

C.5.5.1 Rigid Floor Mounted/Pad Mounted Equipment Anchorage

Equipment can be attached rigidly to the floor directly through the existing base plate, by using additional steel structural beams, by using bumpers or mounted on a raised floor. Different ways of attaching equipment rigidly to the structural floor are shown in Figures C.5.5.1A through C.5.5.1G.

When the equipment is rigidly fixed and not expected to topple, bumpers can be used to restrain the base from moving horizontally. An example of equipment installed with elastomeric bumpers is shown in Figure C.5.5.1D and a photo of equipment restrained with bumpers is shown in Figure C.5.5.1E. Two examples of single direction bumpers are shown in Figure C.5.5.1F.

Anchorage of equipment can also be performed using a base plate with anchors as shown in Figure C.5.5.1G.

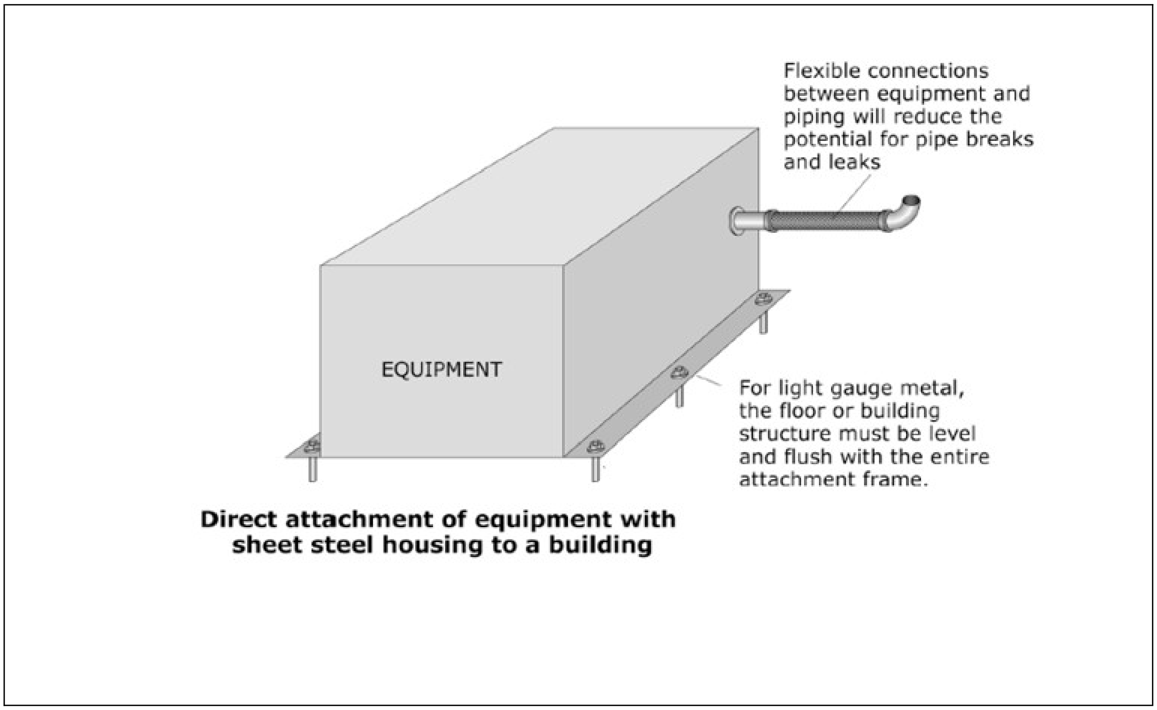

Fig. C.5.5.1A Example of equipment attached directly through existing base plate to structural floor (Figure 6.4.1.1-6 from FEMA E-74)

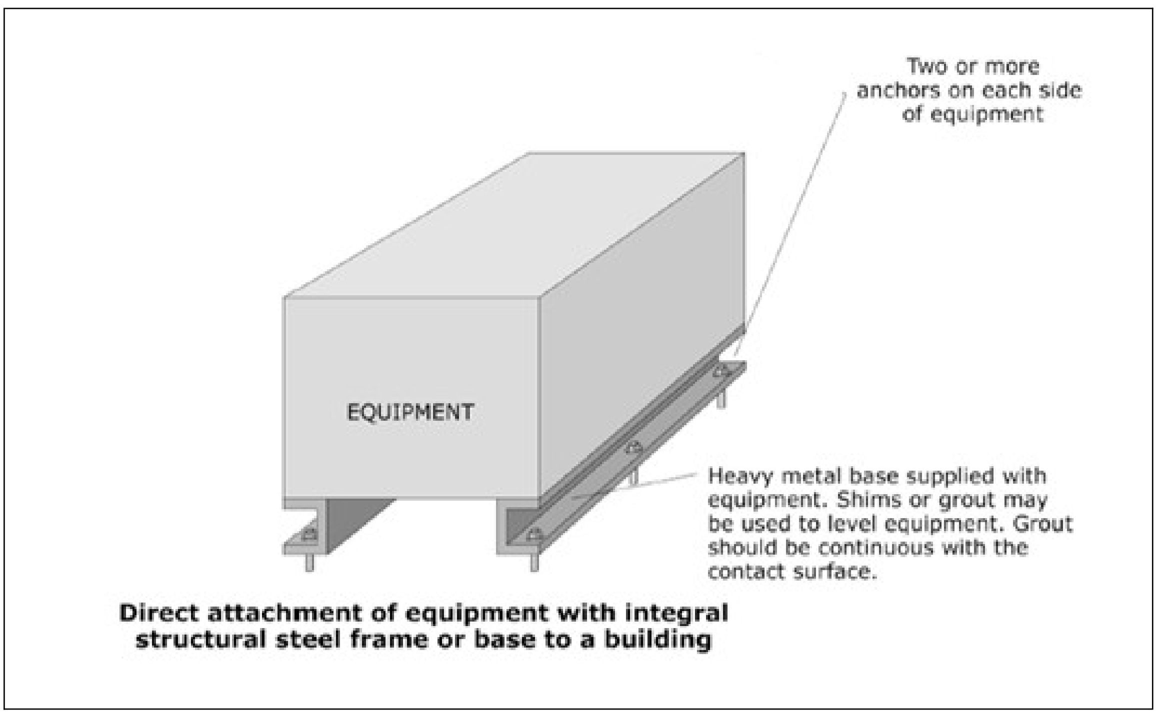

Fig. C.5.5.1B. Example of equipment welded/bolted to steel structural beams which are anchored to structural floor (Figure 6.4.1.1-6 from FEMA E-74)

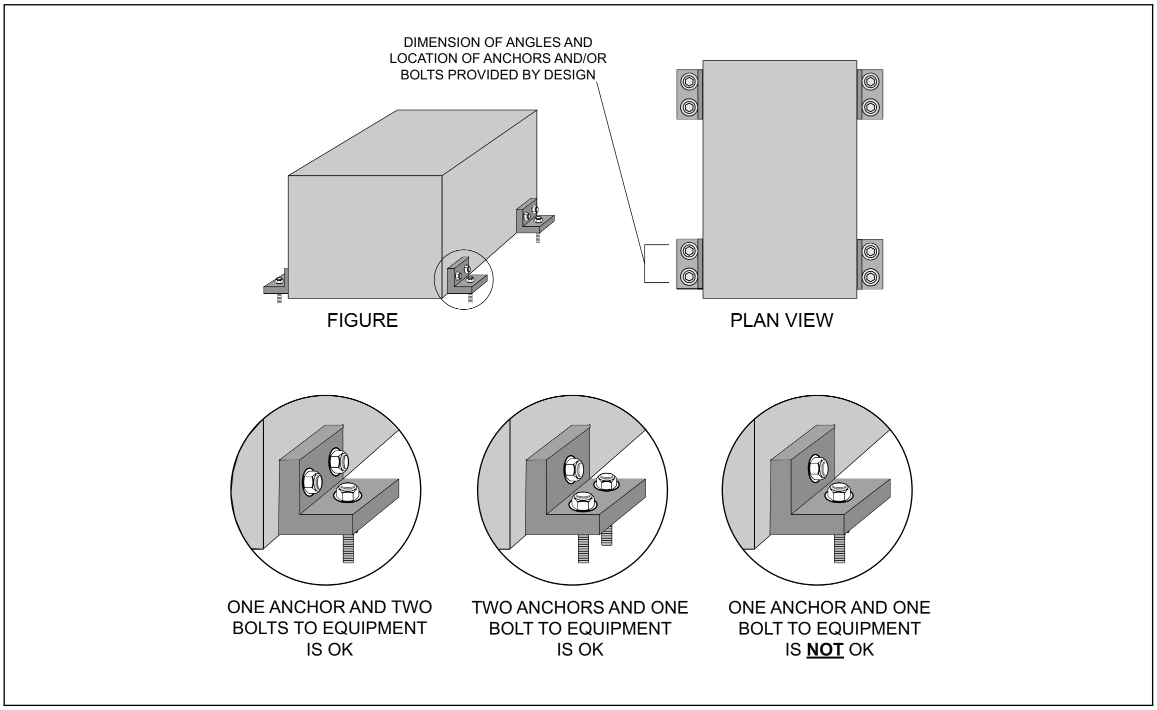

Fig. C.5.5.1C. Example of equipment attached using additional steel structural angles to structural floor (Figure 73 from FEMA 412)

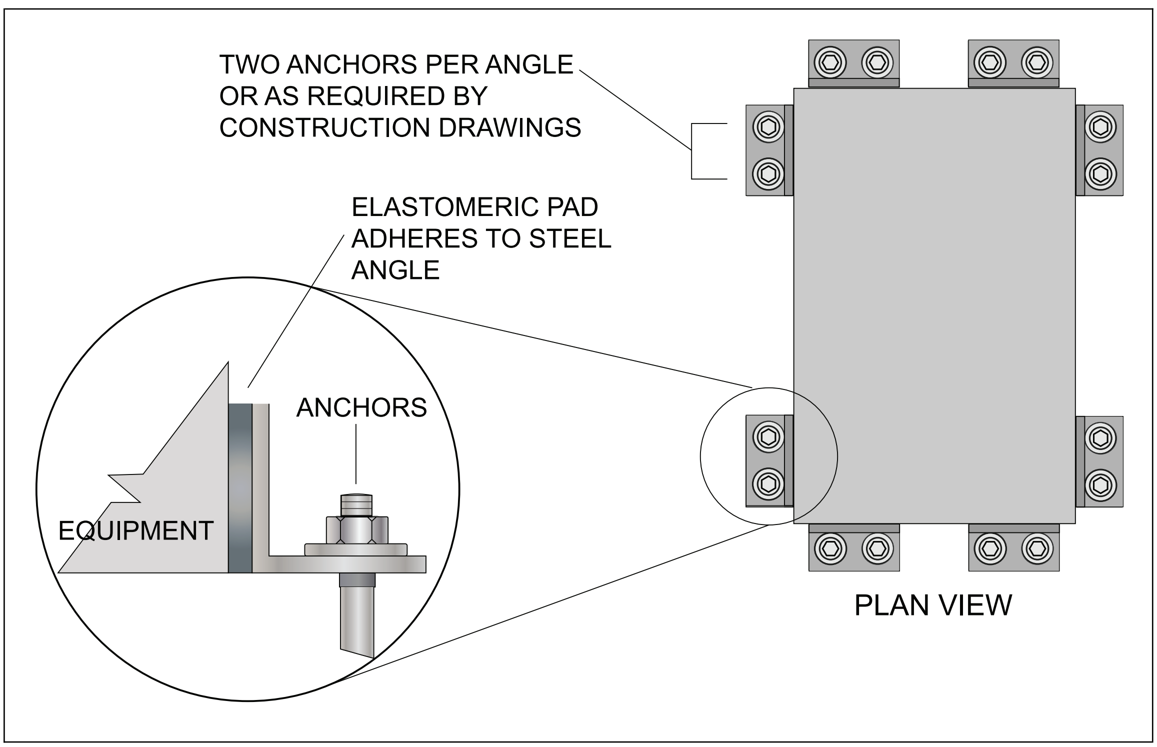

Fig. C.5.5.1D. Schematics of equipment installed with bumpers (Figure 77 from FEMA 412)

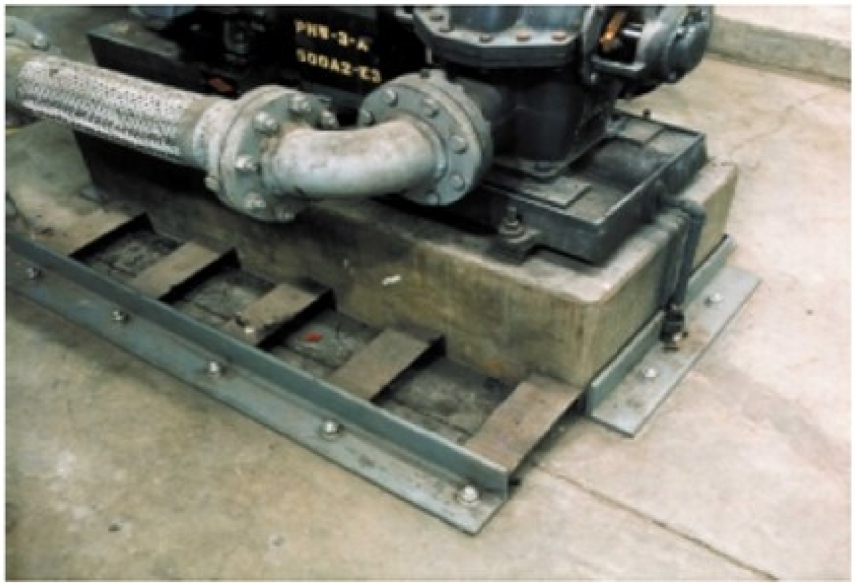

Figure 79: Equipment restrained with bumpers.

Fig. C.5.5.1E. Example of equipment restrained with bumpers (Figure 79 from FEMA 412)

Figure 116: Two examples of bumpers.

Fig. C.5.5.1F. Examples of single direction bumpers (Figure 116 from FEMA 412)

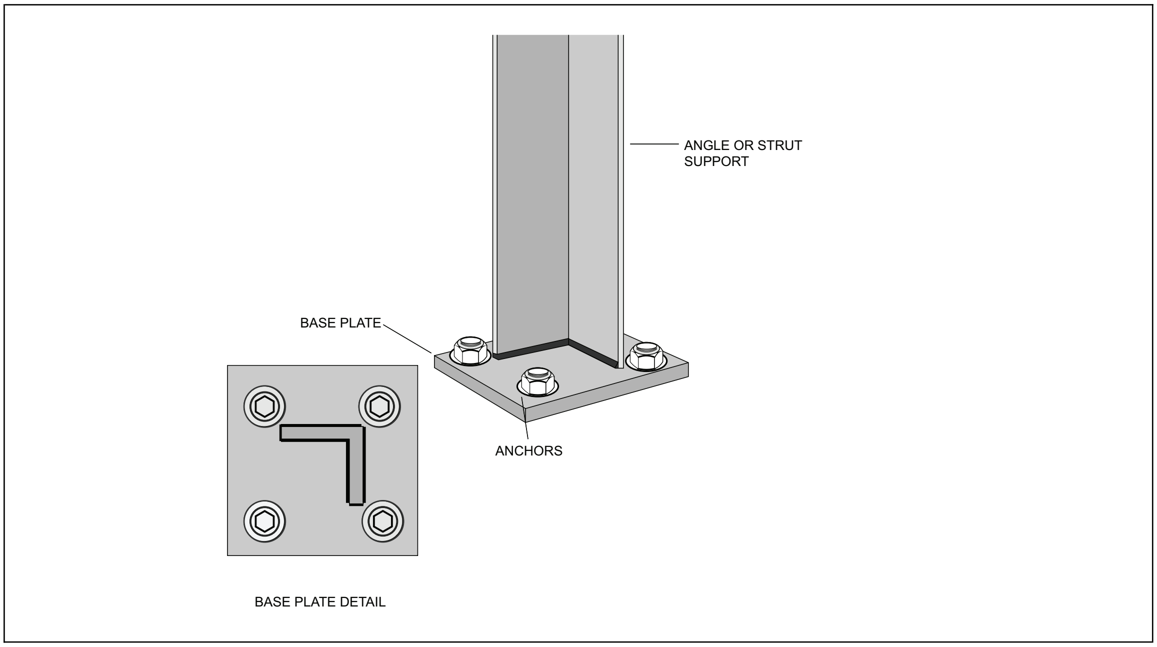

Fig. C.5.5.1G. Example of other equipment anchored to structural floor using base plate (Figure 81 from FEMA 412)

C.5.5.2 Wall Mounted Equipment Anchorage

Equipment can be attached directly to the wall or by using additional structural steel members as shown in Figure C.5.5.2A.

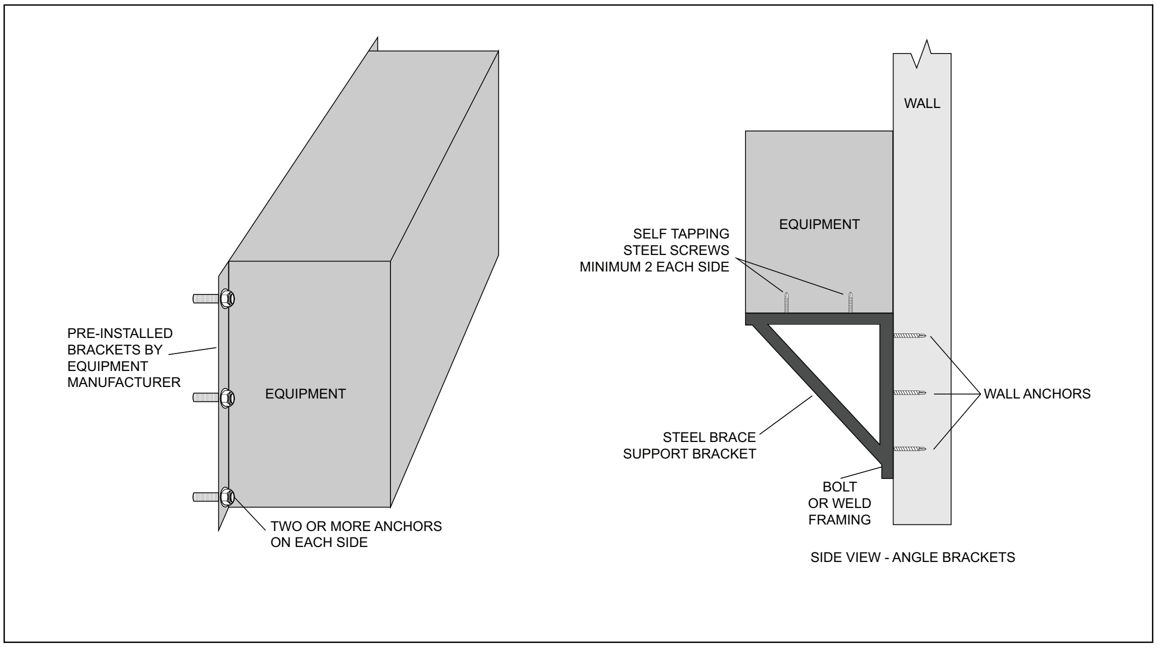

Fig. C.5.5.2A. Example of equipment attached directly to wall or supported by additional steel members (Figures 123 and 124 from FEMA 413)

C.5.5.3 Vibration Isolators for Equipment Anchorage

Vibration isolators prevent the transmission of noise and vibration into the building structure by isolating the base of the equipment. They also reduce the amount of horizontal earthquake force transmitted from the floor to the equipment. Examples of all-direction restrained vibration isolators are shown in Figure C.5.5.3A. Vibration isolators can cause equipment to topple or be displaced when subjected to horizontal shaking and need to be analyzed and designed by a qualified structural engineer.

Figure 114: Four types of restrained springs.

Fig. C.5.5.3A. Four types of all-direction restrained vibration isolators (Figure 114 from FEMA 412)

The movement of equipment supported on vibration isolators is limited by using devices called snubbers. A photo of equipment with open spring isolators restrained with snubbers is shown in Figure C.5.5.3B. Two examples of snubbers are shown in Figure C.5.5.3C. An example of equipment installed with vibration isolators and restrained with snubbers is shown in Figure C.5.5.3D.

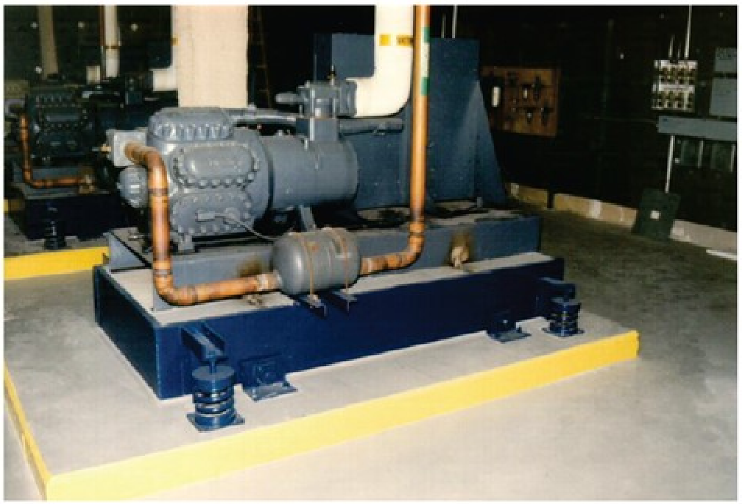

Figure 6.4.1.3-6 Open springs and snubbers used to support equipment (Photo courtesy of Mason Industries).

Fig. C.5.5.3B. Example of equipment restrained with snubbers (Figure 6.4.1.3-6 from FEMA E-74, courtesy of Mason Industries, Inc.)

Figure 115: Two examples of snubbers.

Fig. C.5.5.3C. Examples of all-direction snubbers (Figure 113 from FEMA 412)

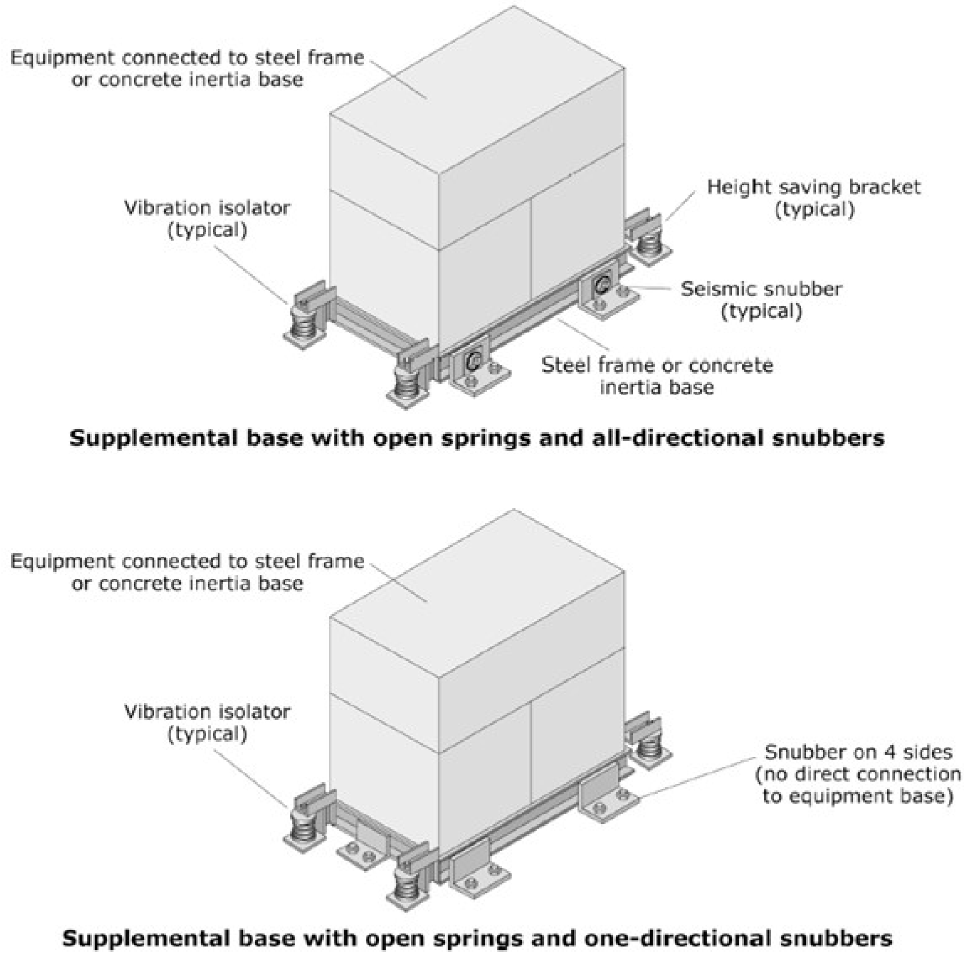

Figure 6.4.1.3-8 HVAC equipment with vibration isolation (ER).

Fig. C.5.5.3D. Example of equipment with vibration isolators restrained with snubbers (Figure 6.4.1.3-8 from FEMA E-74).

C.5.5.4 Equipment Anchorage on Raised Access Floors

Equipment on a raised floor can be attached to the building structure by using a stand rated for the weight of the equipment and laterally braced to withstand seismic loads. Ensure that the equipment is rigidly attached to the stand, and the stand is rigidly bolted to the floor beneath the raised floor as shown in Figures C.5.5.4A through C.5.5.4C.

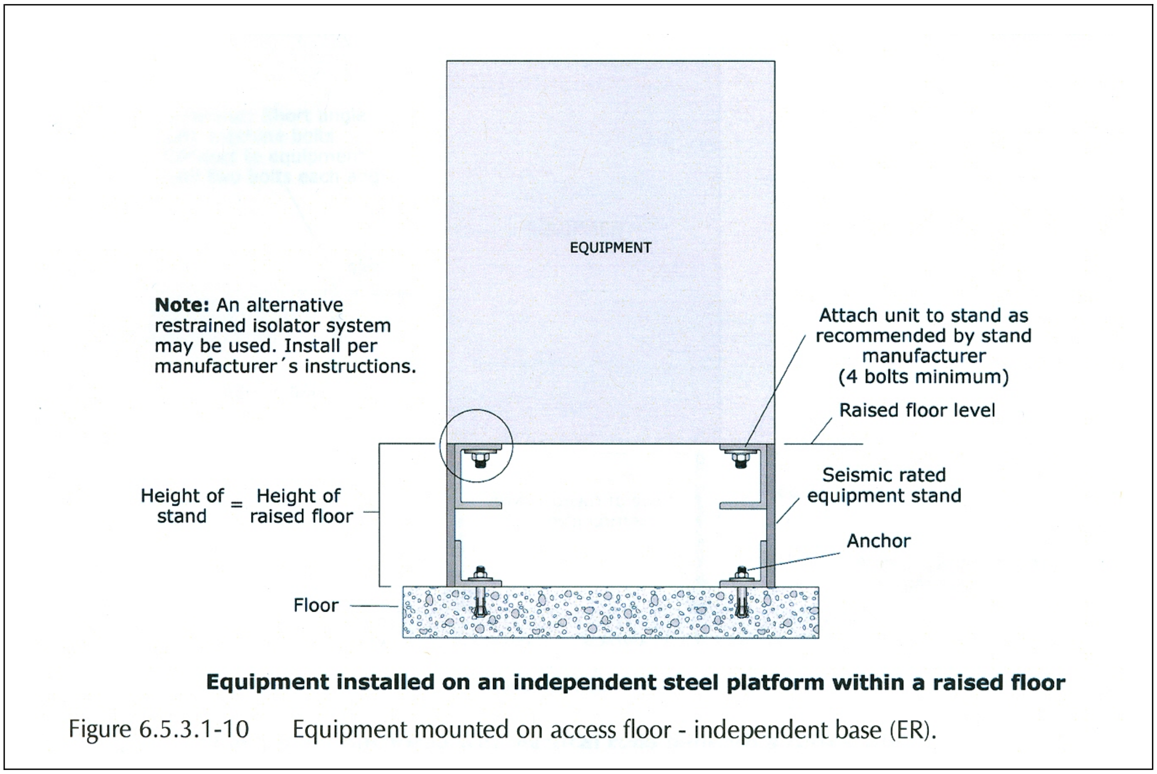

Fig. C.5.5.4A. Example of equipment on raised access floor but supported on structural floor (Figure 6.5.3.1-10 from FEMA E-74)

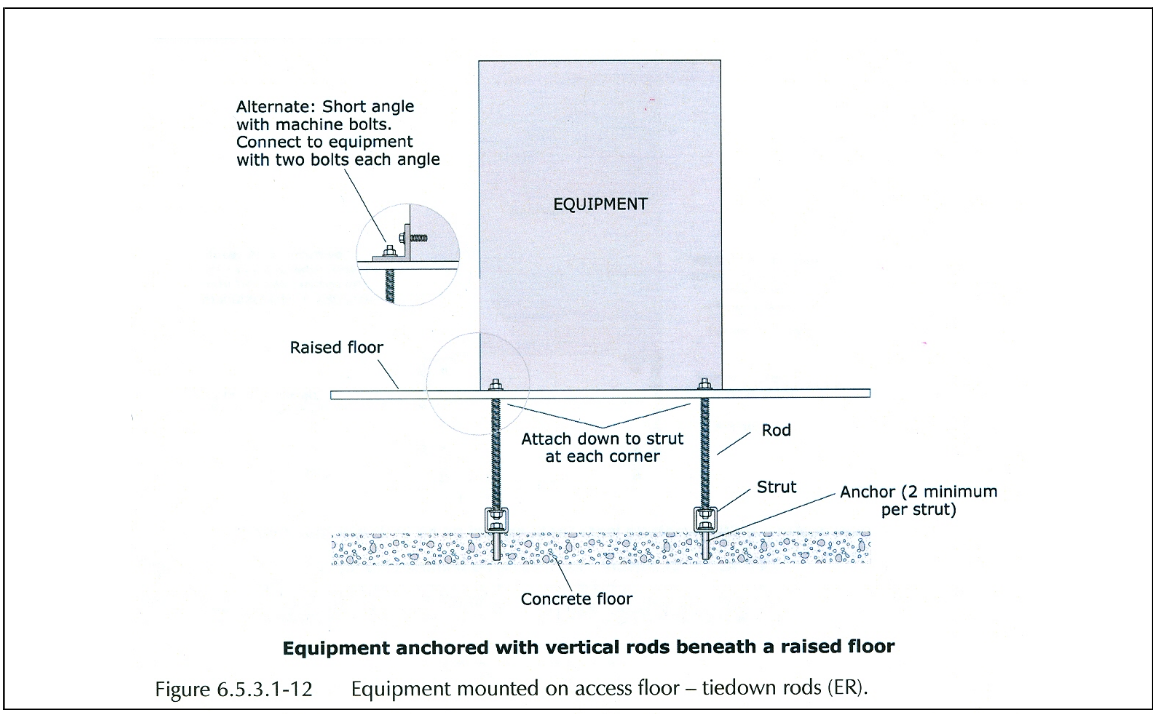

Fig. C.5.5.4B. Example of equipment supported on raised access floor, restrained with vertical rods to the structural floor (Figure 6.5.3.1-12 from FEMA E-74)

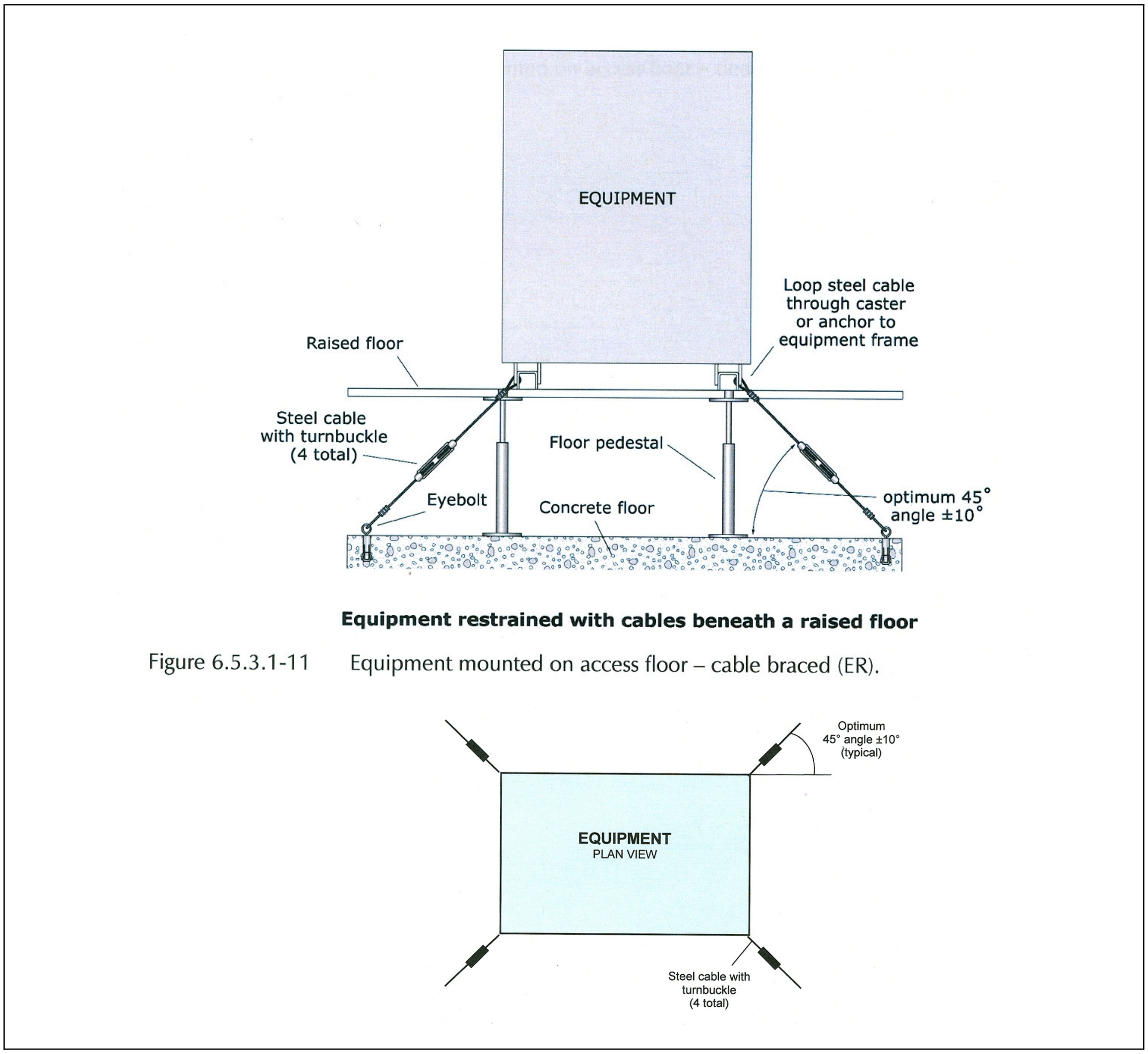

Fig. C.5.5.4C. Example of equipment supported on raised access floor restrained with four cable tethers to the structural floor (Figure 6.5.3.1-11 from FEMA E-74 with additional plan view)

C.5.5.5 Anchorage of Raised Access Floor

The unique considerations for raised access floors are provided in Figure C.5.5.5. Equipment on a raised access floor can be attached to the building structure as explained in Section C.5.5.4.

Special access floors that comply with applicable ASCE 7-22 requirements should be provided in areas of moderate and high seismicity. Special access floors are access floors which have positive connections (i.e., mechanical fasteners, welds or concrete anchors) from floor stringers to pedestals, pedestals to floors and at ends of braces. Additionally, floor stringers must be adequate for the axial force; and bracing and pedestals must be made up of structural steel.

Raised access floors should have the following features:

- Stringers should be positively attached to pedestals (e.g., with screws)
- Individual pedestals should be positively anchored to the structural floor with anchors. Note that pedestals at brace locations may require anchorage with larger capacity
- Bracing should be provided as per ASCE7-22 requirements and should be positively anchored to the floor

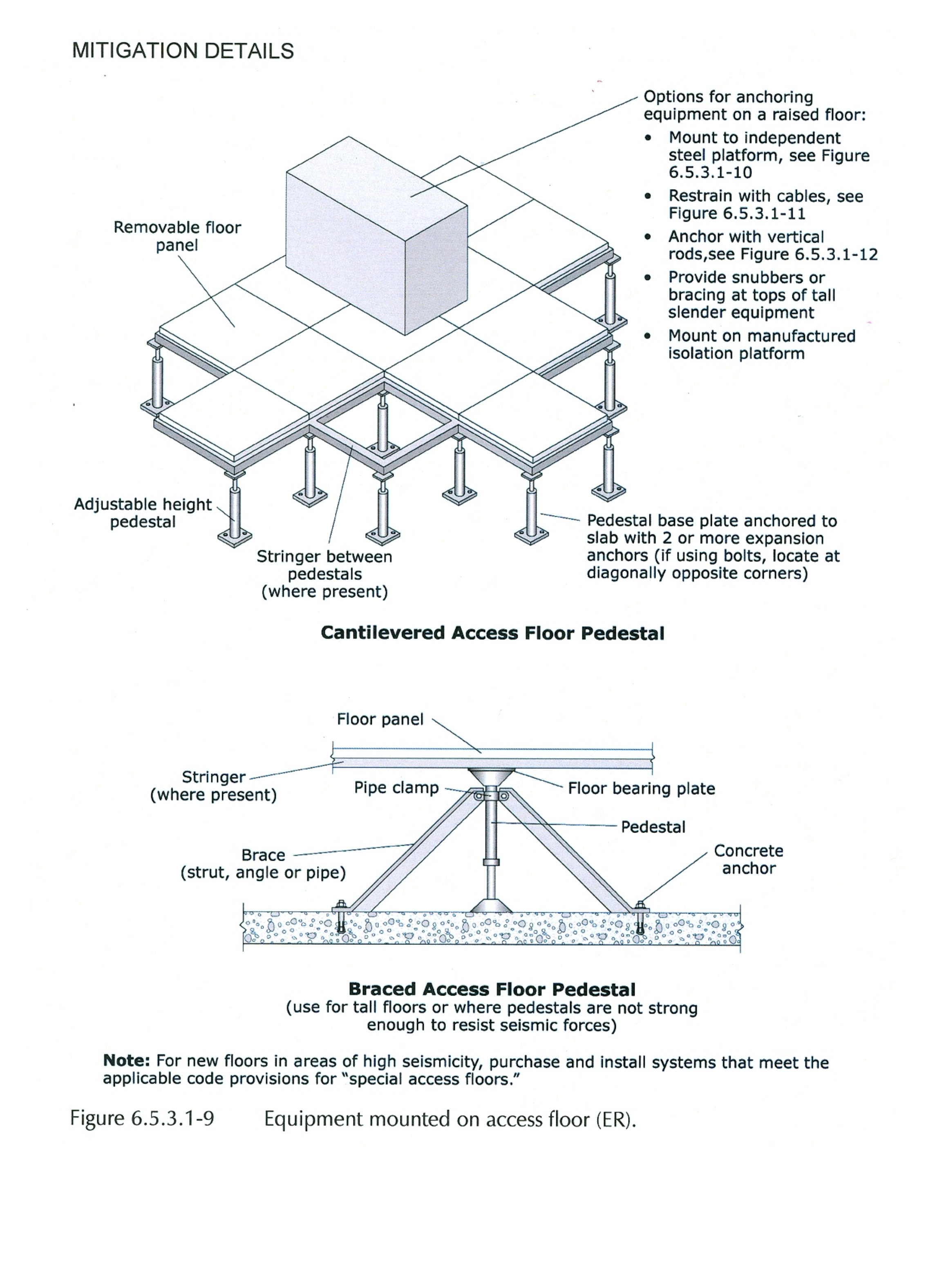

Fig. C.5.5.5 Example raised access floor bracing and other details (Figure 6.5.3.1-9 from FEMA E-74)

C.5.5.6 General Anchors for Equipment

Equipment can be attached to the concrete structure using a variety of anchors as shown in Figure C.5.5.6. The type of anchor for a particular item can be determined by the structural engineer-in-charge of the equipment anchorage design. In regions of moderate and high seismic risk, use anchors which are prequalified for seismic applications.

Where post-installed concrete anchors are to be used in earthquake-exposed areas, they should be determined based on recognized national or international standards. See Section 2.2.2.1.3, 2.2.2.1.4 and Appendix C.5.4 for detailed information and acceptance criteria.

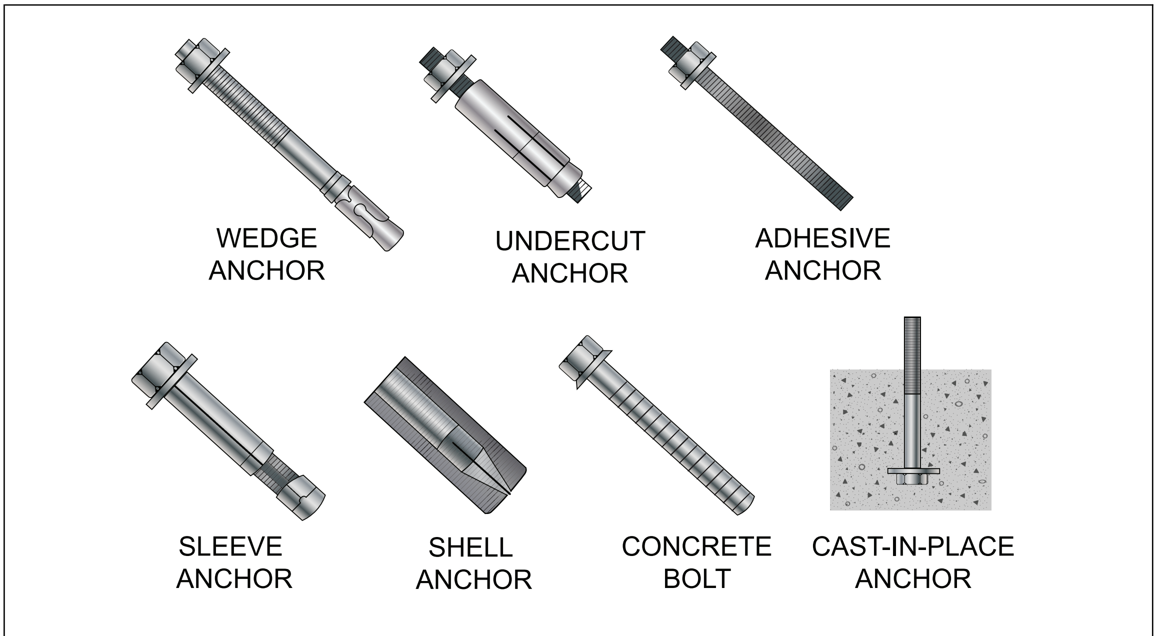

Fig. C.5.5.6. Different types of anchors for anchoring equipment to concrete structures (Figure 127 from FEMA 413)

C.6 FM Worldwide Earthquake Map

C.6.1 Scope

The FM Worldwide Earthquake Map displays zones that convey mean return times of soil-adjusted "damaging" earthquake ground motions (i.e., sufficient to cause non-trivial damage to structures and contents that are not properly designed to resist earthquake forces). The FM map differs from building code maps, which display motions (e.g., spectral accelerations) damaging or otherwise for a uniform soil or rock site condition and a single return period (often approximately 500 years or 2,500 years) or probability of collapse (i.e., in ASCE 7 earthquake hazard is based on a 1% in 50-year probability of collapse). In other words, the FM map displays earthquake risk (showing the mean return times of damaging ground motions at a site) while building code maps are earthquake hazard maps (showing underlying bedrock accelerations, unadjusted for the typically amplifying effects of local soils, for a single return period or probability of collapse). Although they map different parameters, the underlying science of the hazard calculation is the same. Table C.6.1 presents the mean return times of damaging ground motions and the relative risk for FM earthquake zones. The table also shows the likelihood that damaging ground motions will occur at least once for a commonly assumed 50-year facility life (see Section C.6.2 for more information).

Table C.6.1. FM Earthquake Zones

FM Earthquake Zone	*"Damaging" Earthquake Ground Motions* Mean Return Period	Approximate Chance of at Least One Occurrence in 50-Year Facility Life	*Relative Risk*
50-year	0 to 50 years	≥ 63%	Severe
100-year	51 to 100 years	39-63%	High
250-year	101 to 250 years	18-39%	Moderate
500-year	251 to 500 years	10-18%	Moderate
>500-year	>500 years	≤10%	Low

C.6.2 General

The mean recurrence interval (MRI) of an event (e.g., damaging ground motion) is the average number of years between successive events. A MRI of 500 years does not imply that successive events will be exactly 500 years apart. Nor does it imply that there is 100% probability of its occurrence in a 500 year period. (Compare rolling a 6-sided die. There is a one-in-six chance of rolling a "3" [i.e., a "recurrence interval" of 6]. However, in six rolls of the die, it is possible that a "3" will not be rolled and it is also possible that a "3" will be rolled more than once.) The following relationship gives the probability of an event in a given period:

$P = 1\text{-}exp(-t/T)$

where, P is the probability that an event of MRI "T" will occur at least once in a time period t.

The probability of a 500-year event occurring at least once in 50 years is:

$P = 1\text{-}exp(-50/500)$ = 0.0952 (9.5%).

The probability of a 500-year event occurring at least once in 500 years is:

$P = 1\text{-}exp(-500/500)$ = 0.632 (63.2%).

Put another way, there is about a 37% chance that a 500-year event will not occur in a given 500-year period. However, assuming independence of seismic events, there is also about the same probability that a 500-year event will occur at least twice in 500 years (i.e., 0.632 × 0.632 = 0.4 [40%]).

The FM Worldwide Earthquake Map conveys the MRI of damaging ground motions, not the MRI of damaging earthquakes, at the site. To distinguish between the two, refer to Figure C.6.2. In that figure, assume seismic source A and source B produce earthquakes of magnitude 7.5 and 6.7 respectively, at a 500-year MRI, and the region of damaging ground motions is shown by oval areas surrounding these sources. In the non-overlapping (green) region, damaging ground motions occur at a 500-year MRI. In the overlapping (tan) region, the damaging ground motions occur at a 250-year MRI. Further, assume source C produces earthquakes of magnitude 5.7 at a 125-year MRI and the region of damaging ground motions due to this source is contained entirely within the region of damaging ground motions due to source A. The annual chance of damaging ground motions in this region is then: 1/500 (due to source A) + 1/125 (due to source C) = 0.01/year. The MRI of damaging ground motions is 1/0.01 = 100 years (shown by red).

FM Worldwide Earthquake map zones are based on the following methodology:

1. Seismic sources that are likely to rupture again are identified.

2. The magnitude of the maximum earthquake in a seismic source is established from fault dimensions, tectonic considerations, historic earthquake records, or geodetic observations. Geological, historical, and instrumental data are used to establish the magnitude-frequency relations using a Gutenberg-Richter model, or a modified version; this defines the rates of earthquakes of different magnitudes. This model assumes a fault produces earthquakes of different magnitudes at different rates. It produces smaller earthquakes more frequently than larger earthquakes (as discussed in Section C.1.5).

3. The ground motion prediction relationships appropriate for the region are selected. These relations provide the variation of ground motion parameters (e.g., spectral accelerations) as a function of distance from the earthquake, magnitude, type of faulting, and other parameters.

4. The magnitude-frequency and ground motion prediction relations are used to generate the ground motions for uniform site conditions for selected mean return times of 50, 100, 250, and 500 years.

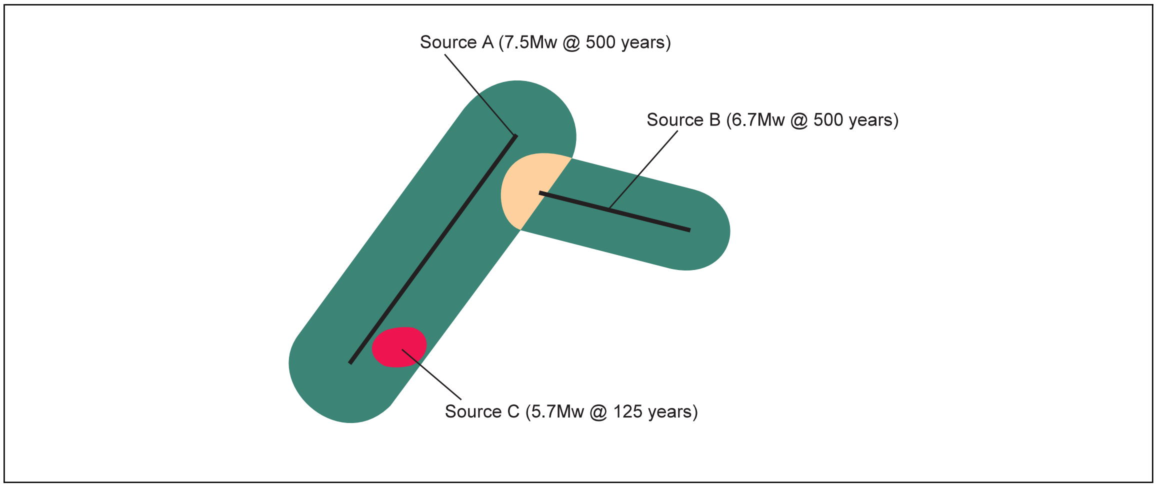

Fig. C.6.2. Mean recurrence interval (MRI) of earthquakes vs. MRI of damaging ground motion

5. The ground motions are adjusted for local site conditions by applying the soil amplification factors. This gives the surface motions at selected return periods. “Local site conditions” used are established based on the best available geologic mapping or other site condition proxies over large areas-they may not be representative of the soils at a particular site.

6. The threshold for non-trivial damage to average weak buildings, derived from a broad range of global building types and also aligned with shake damage thresholds of non-structural components, is applied to the free-surface motions (at various mean return times) to identify regions over which a reasonable amount of damage could occur to structures, to nonstructural systems (e.g., suspended ceilings, raised access floors) or to equipment (e.g., sprinkler piping, storage racks) lacking seismic protection. The outer zone boundary represents the threshold ground shaking that can cause reasonable losses; facilities closer to the center of the zone may experience much higher levels of ground shaking.

For building or equipment design, local building codes, where available, should be followed, with the caveat that the earthquake protection recommendations included in the data sheets should be used where they are more stringent. In the absence of local codes with earthquake design provisions, consult an FM earthquake technical specialist for guidance.

C.6.3 Earthquake Zones

An overview of the FM Worldwide Earthquake Map is shown in Figures C.6.3A and C.6.3B; more detail is available online at https://www.fmglobal.com. The map displays, for engineering purposes, FM earthquake zones for the continents and Oceania. FM earthquake zones convey the 50-year, 100-year, 250-year, 500-year, and > 500-year mean recurrence intervals of damaging ground shaking.

The FM Worldwide Earthquake Map overview provided includes:

Figure C.6.3A. FM Worldwide Earthquake Map – Western Hemisphere (as of January 2021)
Figure C.6.3B. FM Worldwide Earthquake Map – Eastern Hemisphere (as of January 2021)

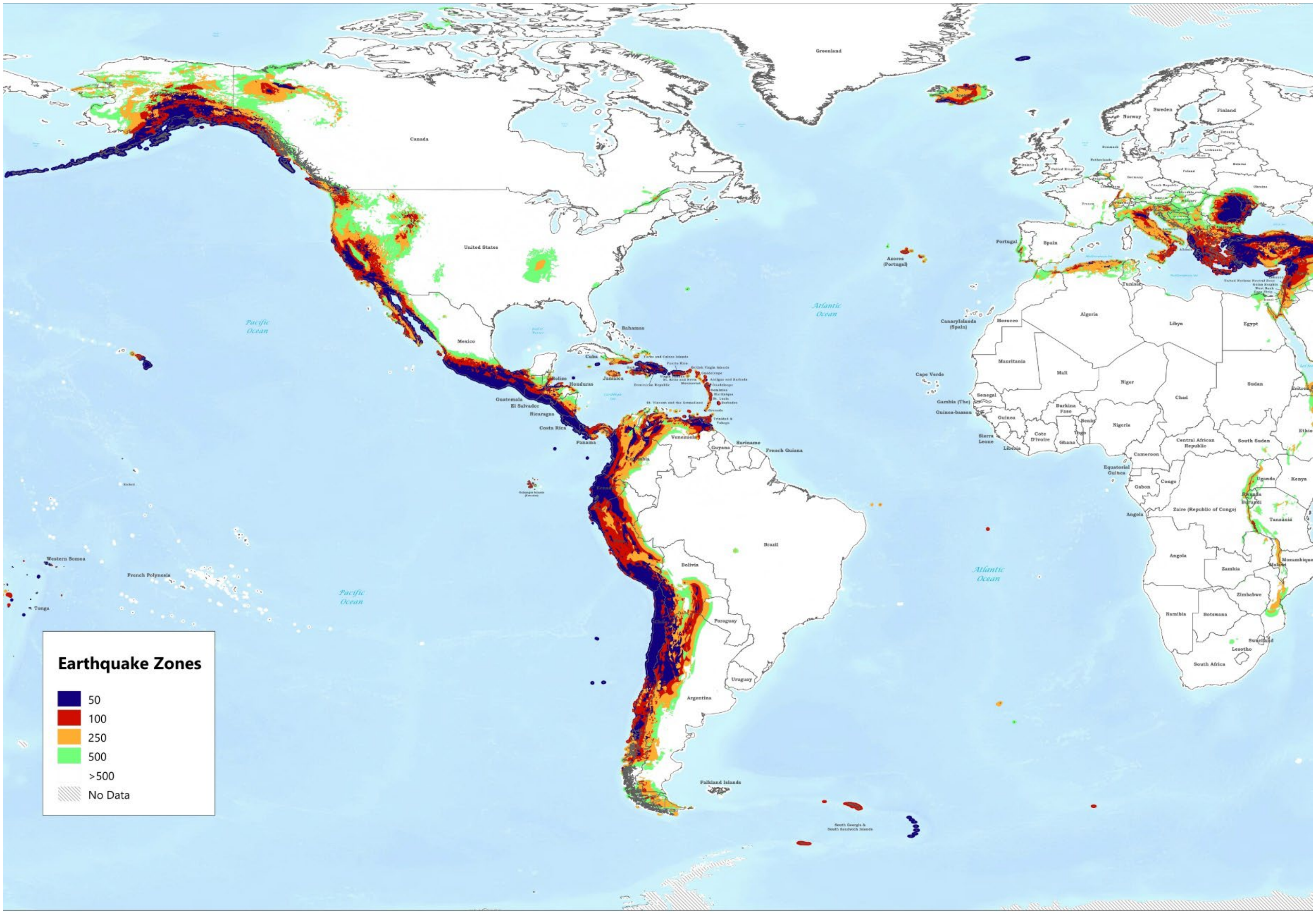

Fig. C.6.3A. FM Global Worldwide Earthquake Map – Western Hemisphere (as of January 2021)

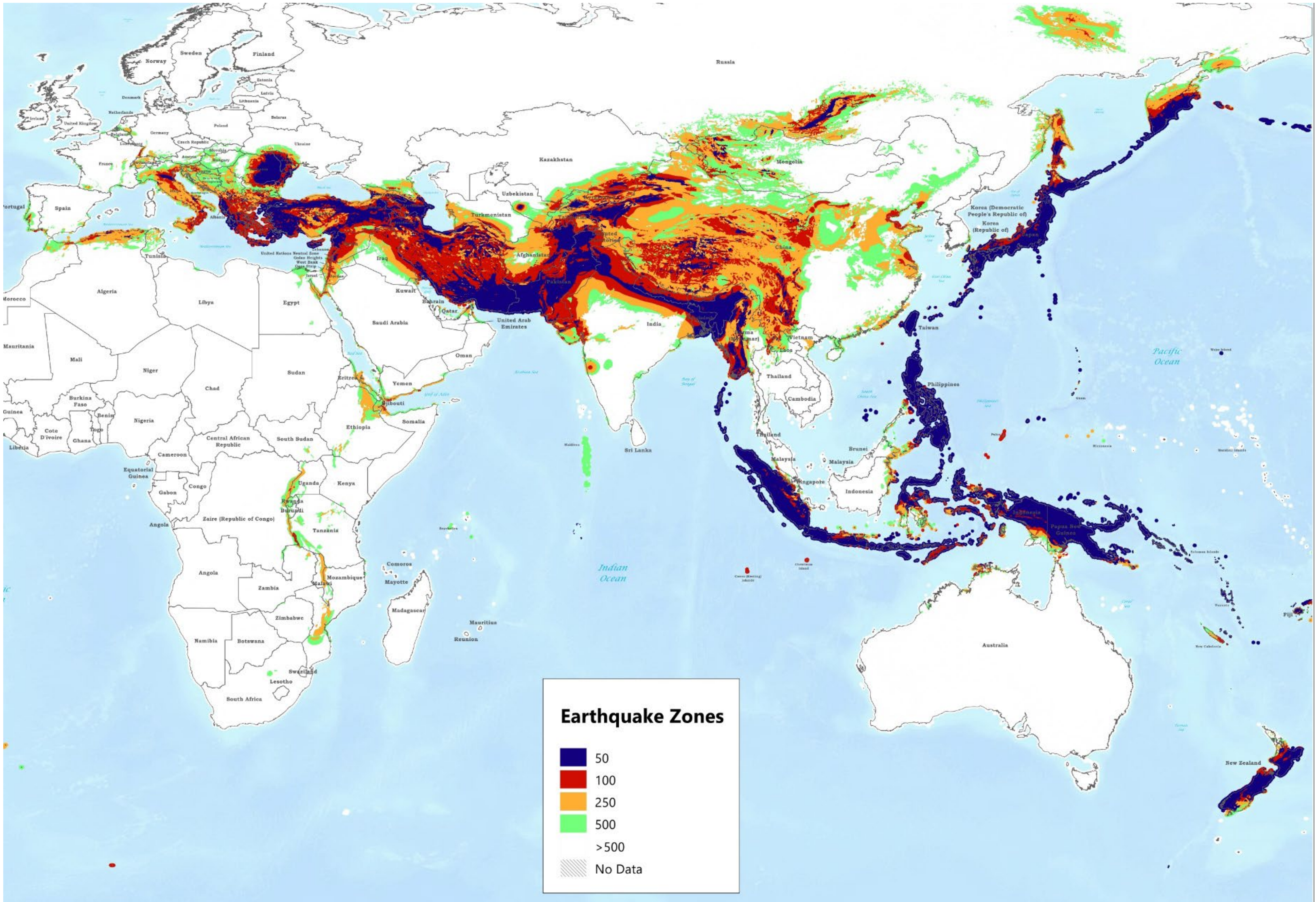

Fig. C.6.3B. FM Global Worldwide Earthquake Map – Eastern Hemisphere (as of January 2021)