Chapter 14 Commentary

NONBUILDING STRUCTURE DESIGN REQUIREMENTS

14.1 GENERAL

14.1.1 Scope. Requirements concerning nonbuilding structures were originally added to the 1994 *Provisions* by the 1991-94 *Provisions* Update Committee (PUC) at the request of the BSSC Board of Direction to provide building officials with needed guidance. In recognition of the complexity, nuances, and importance of nonbuilding structures, the BSSC Board established 1994-97 PUC Technical Subcommittee 13 (TS13), Nonbuilding Structures, in 1995. The duties of TS13 were to review the 1994 *Provisions* and *Commentary* and recommend changes for the 1997 Edition. The subcommittee comprised individuals possessing considerable expertise concerning various specialized nonbuilding structures and representing a wide variety of industries concerned with nonbuilding structures.

Building codes traditionally have been perceived as minimum standards of care for the design of nonbuilding structures and building code compliance of these structures is required by building officials in many jurisdictions. However, requirements in the industry standards are often at odds with building code requirements. In some cases, the industry standards need to be altered while in other cases the building codes need to be modified. Registered design professionals are not always aware of the numerous accepted standards within an industry and may not know whether the accepted standards are adequate. It is hoped that Chapter 14 of the *Provisions* appropriately bridges the gap between building codes and existing industry standards.

One of the goals of TS13 was to review and list appropriate industry standards to serve as a resource. These standards had to be included in the appendix. The subcommittee also has attempted to provide an appropriate link so that the accepted industry standards can be used with the seismic ground motions established in the *Provisions*. It should be noted that some nonbuilding structures are very similar to a building and can be designed employing sections of the *Provisions* directly whereas other nonbuilding structures require special analysis unique to the particular type of nonbuilding structure.

The ultimate goal of TS13 was to provide guidance to develop requirements consistent with the intent of the *Provisions* while allowing the use of accepted industry standards. Some of the referenced standards are consensus documents while others are not.

One good example of the dilemma posed by the conflicts between the *Provisions* and accepted design practice for nonbuilding structures involves steel multilegged water towers. Historically, such towers have performed well when properly designed in accordance with American Water Works Association (AWWA) standards, but these standards differ from the *Provisions* that tension-only rods are required and the connection forces are not amplified. However, industry practice requires upset rods that are preloaded at the time of installation, and the towers tend to perform well in earthquake areas.

In an effort to provide the appropriate interface between the *Provisions* requirements for building structures, nonstructural components, and nonbuilding structures; TS13 recommended that nonbuilding structure requirements be placed in a separate chapter. The PUC agreed with this change. The 1997 *Provisions* Chapter 14 now provides registered design professionals responsible for designing nonbuilding structures with a single point of reference.

Note that building structures, vehicular and railroad bridges, electric power substation equipment, overhead power line support structures, buried pipelines and conduits, tunnels, lifeline systems, nuclear power plants, and dams are excluded from the scope of the nonbuilding structure requirements. The excluded structures are covered by other well established design criteria (e.g., electric power substation equipment, power line support structures, vehicular and railroad bridges), are not under the jurisdiction of local building officials (e.g., nuclear power plants, and dams), or require technical considerations beyond the scope of the *Provisions* (e.g., piers and wharves, buried pipelines and conduits, tunnels, and lifeline systems). Since many components of lifeline systems can be designed in accordance with the *Provisions*, the following information is provided to clarify why lifeline systems are excluded from the scope of the *Provisions*.

Seismic design for a lifeline system will typically require consideration of factors that are unique to or particularly important to that specific system. Seismic design requirements for lifeline systems will typically differ from those for buildings individual structural components for the following reasons:

1. **Physical characteristics.** A building consists of structural and non-structural components within a single site, whereas lifeline systems consist of networks of multiple and spatially distributed linked components (primarily non-building structures and equipment, and possibly some buildings as well.)
2. **Stakeholders.** The stakeholders in the continued operation of a building after an earthquake are a relatively small group of building owners, tenants, and insurers. Lifeline systems provide essential services to a community (e.g., electric power,, communications, transportation, natural gas, water, wastewater, and liquid fuel). Therefore, stakeholders in the seismic performance of such systems are the businesses and residents of the region served by the system, business clients/vendors outside of the region whose continued operation will be impacted by the conditions of the businesses/residents within the region, and the lifeline system's owners and insurers.
3. **Performance.**. Acceptable seismic performance of a building is typically measured by whether life safety of building occupants has been adequately protected (in accordance with minimum building code design provisions.) In addition, for those relatively few buildings for which performance based design has been considered, acceptable seismic performance will also be measured by how well post-earthquake functionality and return-to-service requirements of the building tenants have been met.

   The ability of a lifeline system to maintain an acceptable level of service after an earthquake will depend, not only on the seismic performance of its various spatially dispersed components, but also on the redundancy and service capacity of these components (e.g., number of lanes within roadway elements). To the extent that a lifeline system is comprised of redundant components of sufficient service capacity, it can maintain an acceptable level of service to a community even if some of the redundant components are damaged during the earthquake. In addition, except for certain transportation structures (e.g., bridges and tunnels), earthquake damage to the lifeline system components generally do not result in direct life-safety consequences. Therefore, acceptable seismic performance for a lifeline system is typically based on: (a) whether the system provides an adequate level of service to its users after an earthquake; (b) whether economic losses related to direct damage, lost revenue from an inoperable system, and liability exposure are within tolerable limits; and (c) whether any adverse political, legal, social, administrative, or environmental consequences are experienced. For these reasons, acceptable seismic performance requirements for lifeline systems are best established through interaction with the appropriate stakeholders, including the lifeline agency, its customers or users, and appropriate regulatory interests.

The definition of what constitutes a component of a lifeline system is often complicated. Components of utility lifeline systems are typically identical to components that might be found in industrial or commercial applications. A good example of this overlap are aboveground storage tanks that are common in large industrial or manufacturing facilities as well as water and liquid hydrocarbon transportation systems. Because of this similarity, a clear definition is needed to determine when design in accordance with the recommended approach for lifeline systems should be give preference over requirements in the *Provisions.* Three criteria are considered for determining whether the design of a particular nonbuilding structure can be treated as a component of a lifeline system.

1. **Spatial distribution.** As noted above, lifeline systems are typically spatially-distributed systems that provide services considered essential to community activities and include electric power, communications, water, waste-water, natural gas, liquid fuel, and transportation systems. Fixed facilities, such as power plants, compressor stations, metering stations, are typically treated as nodes

of a lifeline system and are designed in accordance with these Provisions.

2. **Definition by legal boundary.** Portions of utility lifeline systems upstream of the point defining the legal boundary for ownership and responsibility for maintenance and repair shall be considered as part of a lifeline system. The physical elements of transportation lifeline systems not excluded in the *Provisions* and owned and maintained by a transportation agency are also considered part of a lifeline system.

   Defining lifeline system components by a legal boundary is most appropriate for utility systems that deliver electric power, natural gas, electric power, wastewater and some telecommunication services. Existing regulatory provisions commonly specify a specific interface between the portions of these systems that is under the control of the service provider and the portions of the system under control of the building or facility owner. For electric power, natural gas, and water systems, this boundary is typically the customer's side of the meter. The other typical boundary is the property line. Those components under control of the service provider can be considered as part of a lifeline system.

   It is common for the design and maintenance of physical elements of transportation lifeline systems to fall under the jurisdiction of a governmental or government-regulated entity. Two common examples include state highway departments and port authorities. In such cases, the definition of a lifeline system by legal boundary for these situations is defined by the jurisdiction of these agencies.

3. **Definition by expertise.** Historically, the primary audience of the *Provisions* has been the structural engineering community and building code organizations seeking to modify their seismic provisions. As a result of this focus, the *Provisions* are best suited for the seismic design and performance of individual structures. Since most new construction for lifeline systems address adding components to existing systems, rational design approaches should consider the overall system performance in design of new components and the benefits of improved seismic performance in comparison with the performance of the system for other natural and other hazards, such as man-made threats. The geographically diverse nature of lifeline systems often requires that earthquake hazards be defined by one or more scenario events instead of the probabilistic ground motion hazards defined in the *Provisions*. These additional considerations often require special expertise in addition to that of the structural engineering profession that is dominant audience for the *Provisions*.

14.1.2 References

American Concrete Institute (ACI):

ACI 350, *Environmental Engineering Concrete Structures*, 2001.

ACI 350.3 *Seismic Design of Liquid-Containing Concrete Structures,* 2001.

American Society of Civil Engineers (ASCE):

Guidelines for Seismic Evaluation and Design of Petrochemical Facilities, 1997.

Housner, G.W. *Earthquake Pressures in Fluid Containers*, California Institute of Technology, 1954.

Miller, C. D., S. W. Meier, and W. J. Czaska, *Effects of Internal Pressure on Axial Compressive Strength of Cylinders and Cones,* Structural Stability Research Council Annual Technical Meeting, June 1997.

Rack Manufacturers Institute:

Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks, 1997 (Reaffirmed 2002).

Troitsky, M.S., *Tubular Steel Structures*, 1990. (Troitsky)

Wozniak, R. S., and W. W. Mitchell, *Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks*, 1978 Proceedings—Refining Dept., Vol. 57, American Petroleum Institute, Washington, D.C., May 9, 1978. (Wozniak)

14.1.2.2 Other references

While not cited directly in the Provisions or Commentary, the user may find these other references related to nonbuilding structures helpful.

ACI 307, *Standard Practice for the Design and Construction of Cast-In-Place Reinforced Concrete Chimneys*, 1995.

ANSI K61.1 *Safety Requirements for the Storage and Handling of Anhydrous Ammonia*, American National Standards Institute, 1999.

API 2510 *Design and Construction of Liquefied Petroleum Gas Installation*, American Petroleum Institute, 19952001.

ASCE Petro *Design of Secondary Containment in Petrochemical Facilities* (Petrochemical Energy Committee Task Report), American Society of Civil Engineers, 1997.

ASME B31.8 *Gas Transmission and Distribution Piping Systems*, American Society of Mechanical Engineers, 1995.

ASME B96.1 *Welded Aluminum-Alloy Storage Tanks*, American Society of Mechanical Engineers, 1993.

ASME STS-1 *Steel Stacks*, American Society of Mechanical Engineers, 2001.

ASTM F 1159 *Standard Practice for the Design and Manufacture of Amusement Rides and Devices* (ASTM F 1159-97a), American Society for Testing and Materials, 1997.

ASTM C 1298 *Standard Guide for Design and Construction of Brick Liners for Industrial Chimneys* (ASTM C 1298-95), American Society for Testing and Materials, 1995.

DOT 49CFR193 *Liquefied Natural Gas Facilities: Federal Safety Standards* (Title 49CFR Part 193), U.S. Department of Transportation, 2000.

NFPA 30 *Flammable and Combustible Liquids Code*, National Fire Protection Association, 12000.

NFPA 58 *Storage and Handling of Liquefied Petroleum Gas*, National Fire Protection Association, 2001.

NFPA 59 *Storage and Handling of Liquefied Petroleum Gases at Utility Gas Plants*, National Fire Protection Association, 2001.

NFPA 59A *Production, Storage and Handling of Liquefied Natural Gas (LNG)*, National Fire Protection Association, 2001.

NCEL R-939 Ebeling, R. M., and Morrison, E. E., *The Seismic Design of Waterfront Retaining Structures*, Naval Civil Engineering Laboratory, 1993.

NAVFAC DM-25.1 *Piers and Wharves,* U.S. Naval Facilities Engineering Command, 1987.

TM 5-809-10 *Seismic Design for Buildings*, U.S. Army Corps of Engineers, 1992, Chapter 13 only.

14.1.5 Nonbuilding structures supported by other structures. This section has been developed to provide an appropriate link between the requirements for nonbuilding structures and those for inclusion in the rest of the *Provisions*—especially the requirements for architectural, mechanical, and electrical components.

14.2 GENERAL DESIGN REQUIREMENTS

14.2.1 Seismic use groups and importance factors. The Importance Factors and Seismic Use Group classifications assigned to nonbuilding structures vary from those assigned to building structures.

Buildings are designed to protect occupants inside the structure whereas nonbuilding structures are not normally "occupied" in the same sense as buildings, but need to be designed in a special manner because they pose a different sort of risk in regard to public safety (that is, they may contain very hazardous compounds or be essential components in critical lifeline systems). For example, tanks and vessels may contain materials that are essential for lifeline functions following a seismic event (such as fire-fighting or potable water), potentially harmful or hazardous to the environment or general health of the public, biologically lethal or toxic, or explosive or flammable (posing a threat of consequential or secondary damage).

If not covered by the authority having jurisdiction, Table 14.2-1 may be used to select the importance factor *(I)*. The value shall be determined by taking the larger of the value from the approved Standard or the value selected from Table 14.2-1. It should be noted that a single value of importance factor may not apply to an entire facility. For further details, refer to ASCE Petro. The use of a secondary containment system, when designed in accordance with an acceptable National Standard, could be considered as an effective means to contain hazardous substances and thus reduce the hazard classification.

The specific definition of material hazard and what constitutes a hazard is being developed in the *International Building Code* process. The hazards will be predicated on the quantity and type of hazardous material.

The importance factor is not intended for use in making economic evaluations regarding the level of damage, probabilities of occurrence, or cost to repair the structure. These economic decisions should be made by the owner and other interested parties (insurers, financiers, etc.). Nor it is intended for use for purposes other than that defined in this provision.

Examples are presented below demonstrate how this table may be applied.

Example 1. A water storage tank used to provide pressurized potable water for a process within a chemical plant where the tank is located away from personnel working within the facility.

Table 14.2-1 Seismic Use Groups and Importance Factors for Nonbuilding Structures

Seismic Use Group	**I**	**II**	**III**
Function	F-I	F-II	F-III
Hazard	H-I	H-II	H-III
Importance Factor	I = 1.0	I = 1.25	I = 1.5

Address each of the issues implied in the matrix:

- Seismic Use Group: Neither the structure nor the contents are critical, therefore use Seismic Use Group I.
- Function: The water storage tank is neither a designated ancillary structure for post-earthquake recovery, nor identified as an emergency back-up facilities for a Seismic Use Group III structure, therefore use F-I.
- Hazard: The contents are not hazardous, therefore use H-I.
- This tank has an importance factor of 1.0.

Example 2. A steel storage rack is located in a retail store in which the customers have direct access to the aisles. Merchandise is stored on the upper racks. The rack is supported by a slab on grade.

Table 14.2-1 Seismic Use Groups and Importance Factors for Nonbuilding Structures

Seismic Use Group	**I**	**II**	**III**
Function	F-I	F-II	F-III
Hazard	H-I	H-II	H-III
Importance Factor	I = 1.0	I = 1.25	I = 1.5

Address each of the issues in the matrix:

- Seismic Use Group: Neither the structure nor the contents are critical, therefore use Seismic Use Group I.
- Function: The storage rack is neither used for post-earthquake recovery, nor required for emergency back-up, therefore use F-I.
- Hazard: The contents are not hazardous. However, its use could cause a substantial public hazard during an earthquake. Subject to the local authority's jurisdiction it is H-II.
- According to Sec. 14.3.5.2 the importance factor for storage racks in occupancies open to the general public must be taken as 1.5.
- Use an importance factor of 1.5 for this structure.

Example 3. A water tank is located within an office building complex to supply the fire sprinkler system.

Table 14.2-1 Seismic Use Groups and Importance Factors for Nonbuilding Structures

Seismic Use Group	**I**	**II**	**III**
Function	F-I	F-II	F-III
Hazard	H-I	H-II	H-III
Importance Factor	I = 1.0	I = 1.25	I = 1.5

Address each of the issues in the matrix:

- Seismic Use Group: The office building is assigned to Seismic Use Group I.
- Function: The water tank is required to provide water for fire fighting. However since the building is not a Seismic Use Group III structure, the water is used neither for post-earthquake recovery, nor for emergency back-up, so use F-I.
- Hazard: The content and its use are not hazardous to the public, therefore use H-I.
- Use an importance factor of 1.0 for this water structure.

Example 4. A petrochemical storage tank is to be constructed within a refinery tank farm near a populated city neighborhood. An impoundment dike is provided to control liquid spills.

Table 14.2-1 Seismic Use Groups and Importance Factors for Nonbuilding Structures

Seismic Use Group	**I**	**II**	**III**
Function	F-I	F-II	F-III
Hazard	H-I	H-II	H-III
Importance Factor	I = 1.0	I = 1.25	I = 1.5

Address each of the issues in the matrix:

- Seismic Use Group: The LNG tank is assigned to Seismic Use Group III.
- Function: The tank is neither required to provide post-earthquake recovery nor used for emergency back-up for a Seismic Use Group III structure, so use F-I.
- Hazard: The tank contains a substantial quantity of high explosive and is near a city neighborhood. Despite the diking, it is considered hazardous to the public in the event of an earthquake, so use H-III.
- Use an importance factor of 1.5 for this structure.

14.2.3 Design basis. The design basis for nonbuilding structures is based on either adopted references, approved standards, or these *Provisions*. It is intended that the *Provisions* applicable to buildings apply to nonbuilding structures, unless specifically noted in this Chapter.

14.2.4 Seismic force-resisting system selection and limitations. Nonbuilding structures similar to buildings may be designed in accordance with either Table 4.3-1 or Table 14.2-2, including referenced design and detailing requirements. For convenience, Table 4.3-1 requirements are repeated in Table 14.2-2.

Table 14.2-2 of the 2000 NEHRP Provisions for nonbuilding structures similar to buildings prescribed R, Ω_o, and C_d values to be taken from Table 4.3-1, but prescribed less restrictive height limitations than those prescribed in Table 4.3-1. This inconsistency has been corrected. Nonbuilding structures similar to buildings which use the same R, Ω_o, and C_d values as buildings now have the same height limits, restrictions and footnote exceptions as buildings. The only difference is that the footnote exceptions for buildings apply to metal building like systems while the exceptions for nonbuilding structures apply to pipe racks. In addition, selected nonbuilding structures similar to buildings have prescribed an option where both lower R values and less restrictive height limitations are specified. This option permits selected types of nonbuilding structures which have performed well in past earthquakes to be constructed with less restrictions in Seismic Design Categories D, E and F provided seismic detailing is used and design force levels are considerably higher. It should be noted that revised provisions are considerably more restrictive than those prescribed in Table 4.3-1.

Nonbuilding structures not similar to buildings should be designed in accordance Table 14.2-3 requirements, including referenced design and detailing requirements.

Nonbuilding structures not referenced in either Table 14.2-2, Table 14.2-3, or Table 4.3-1 may be designed in accordance with an adopted reference, including its design and detailing requirements.

It is not consistent with the intent of the *Provisions* to take design values from one table or standard and design and/or detailing provisions from another.

14.2.5 Structural analysis procedure selection. Nonbuilding structures that are similar to buildings should be subject to the same analysis procedure limitations as building structures.

Nonbuilding structures that are not similar to buildings should not be subject to these procedure limitations. However, they should be subject to any procedure limitations prescribed in specific adopted references.
For nonbuilding structures supporting flexible system components, such as pipe racks, the supported piping and platforms are generally not regarded as rigid enough to redistribute seismic forces to the supporting frames.

For nonbuilding structures supporting rigid system components, such as steam turbine generators (STG's) and Heat Recovery Steam Generators (HRSG's), the supported equipment, ductwork, and other components (depending on how they are attached to the structure) may be rigid enough to redistribute seismic forces to the supporting frames. Torsional effects may need to be considered in such situations.

14.2.9 Fundamental period. The rational methods for period calculation contained in the *Provisions* were developed for building structures. If the nonbuilding structure has dynamic characteristics similar to those of a building, the difference in period is insignificant. If the nonbuilding structure is not similar to a building structure, other techniques for period calculation will be required. Some of the references for specific types of nonbuilding structures contain more accurate methods for period determination.

Equations 5.2-6, 5.2-7, and 5.2-7 are not recommended because they are not relevant for the commonly encountered nonbuilding structures.

14.3 NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

Nonbuilding structures exhibit behavior similar to that of building structures; however, their function and performance are different. Although the *Provisions* for buildings are used as the primary basis for design, this section identifies appropriate exceptions, modifications, and additions for selected nonbuilding structures similar to buildings.

14.3.1 Electrical power generating facilities. Electrical power plants closely resemble building structures, and their performance in seismic events has been good. For reasons of mechanical performance, lateral drift of the structure must be limited. The lateral bracing system of choice has been the concentrically braced frame. The height limits on braced frames in particular can be an encumbrance to the design of large power generation facilities.

14.3.3 Piers and wharves. Current industry practice recognizes the distinct differences between the two categories of piers and wharves described in the *Provisions*. The piers and wharves with public occupancy, described in paragraph (a) are commonly treated as the "foundation" for buildings or building-like structures, and design is performed using the *Provisions*. The design is likely to be under the jurisdiction of the local building official.

Piers and wharves where occupancy by the general public is not a consideration, as described in paragraph (b), are often treated differently. In many cases, they do not fall under the jurisdiction of building officials, and utilize other design approaches more common to this industry.

Economics plays a major role in the design decisions associated with these structures. These economic decisions may be affected not only by the wishes of the owners, but also by overlapping jurisdictional entities with local, regional, or state interests in commercial development.

In the cases where the Building Officials have jurisdiction, they typically do not have experience analyzing pier and wharf structures. In these instances, they have come to rely on and utilize the other design approaches that are more common in the industry.

Major ports and marine terminals in seismic regions of the world routinely design structures as described in paragraph (b). The design of these often uses a performance-based approach, with criteria and methods that are very different than those used for buildings, as provided in the Provisions.

Design approaches most commonly used are generally consistent with the practices and criteria described in the following documents:

- Working Group No. 34 of the Maritime Navigation Commission (PIANC/MarCom/WG34), 2001, *Seismic Design Guidelines for Port Structures*, A. A. Balkema, Lisse, Netherlands, 2001.
- Ferritto, J., Dickenson, S., Priestley N., Werner, S., Taylor, C., Burke D., Seelig W., and Kelly, S., 1999, *Seismic Criteria for California Marine Oil Terminals,* Vol.1 and Vol.2, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, CA.
- Priestley, N.J.N., Frieder Siebel, Gian Michele Calvi, *Seismic Design and Retrofit of Bridges*, 1996, New York.
- *Seismic Guidelines for Ports*, by the Ports Committee of the Technical Council on Lifeline Earthquake Engineering, ASCE, edited by Stuart D. Werner, Monograph No. 12, March 1998, published by ASCE, Reston, VA.
- *Marine Oil Terminal Engineering and Maintenance Standards*, California State Lands Commission, Marine Facilities Division, May 2002.

These alternative approaches have been developed over a period of many years by working groups within the industry, and consider the historical experience and performance characteristics of these structures that are very different than building structures.

The main emphasis of the performance-based design approach is to provide criteria and methods that depend on the economic importance of a facility. Adherence to the performance criteria in the documents listed above is expected to provide as least as much inherent life-safety, and likely much more, than for buildings designed using the Provisions. However, the philosophy of these criteria is not to provide uniform margins of collapse for all structures. Among the reasons for the higher inherent level of life-safety for these structures are the following:

- These structures have relatively infrequent occupancy, with few working personnel and very low density of personnel. Most of these structures consist primarily of open area, with no enclosed building structures which can collapse onto personnel. Small control buildings on marine oil terminals or similar secondary structures are commonly designed in accordance with the local building code.
- These pier or wharf structures are typically constructed of reinforced concrete, prestressed concrete, and/or steel and are highly redundant due to the large number of piles supporting a single wharf deck unit. Tests done for the Port of Los Angeles at the University of California at San Diego have shown that very high ductilities (10 or more) can be achieved in the design of these structures using practices currently used in California ports.
- Container cranes, loading arms, and other major structures or equipment on the piers or wharves are specifically designed not to collapse in an earthquake. Typically, additional piles and structural members are incorporated into the wharf or pier specifically to support that item.
- Experience has shown that seismic “failure” of wharf structures in zones of strong seismicity is indicated not by collapse, but by economically unrepairable deformations of the piles. The wharf deck generally remains level or slightly tilting but shifted out of position. Complete failure that could cause life-safety concerns has not been known to ever occur historically due to earthquake loading.

- The performance-based criteria of the listed documents include repairability of the structure. This service level is much more stringent than collapse prevention and would provide a greater margin for life-safety.

- Lateral load design of these structures is often governed by other marine loading conditions, such as mooring or berthing.

14.3.4 Pipe racks. Free standing pipe racks supported at or below grade with framing systems that are similar in configuration to building systems should be designed to satisfy the force requirements of Sec. 5.2. Single column pipe racks that resist lateral loads should be designed as inverted pendulums. See ASCE Petro.

14.3.5 Steel storage racks. This section is intended to assure comparable results from the use of the RMI Specification, the NEHRP *Provisions*, and the IBC code approaches to rack structural design.

For many years the RMI has been working with the various committees of the model code organizations and with the Building Seismic Safety Council and its Technical Subcommittees to create seismic design provisions particularly applicable to steel storage rack structures. The 1997 RMI Specification is seen to be in concert with the needs, provisions, and design intent of the building codes and those who use and promulgate them, as well as those who engineer, manufacture, install, operate, use, and maintain rack structures. The RMI Specification, now including detailed seismic provisions, is essentially self-sufficient.

The changes proposed here are compatible and coordinated with those in the 2000 *International Building Code*.

14.3.5.2 Importance factor. Until recently, storage racks were primarily installed in low-occupancy warehouses. With the recent proliferation of warehouse-type retail stores, it has been judged necessary to address the relatively greater seismic risk that storage racks may pose to the general public, compared to more conventional retail environments. Under normal operating conditions, retail stores have a far higher occupancy load than an ordinary warehouse of a reasonable size. Failure of a storage rack system in the retail environment is much more likely to cause personal injury than a similar failure in a storage warehouse. Therefore, to provide an appropriate level of additional safety in areas open to the public, Sec 14.3.5.2 now requires that storage racks in occupancies open to the general public be designed with an importance factor equal to 1.50. Storage rack contents, while beyond the scope of the *Provisions*, pose a potentially serious threat to life should they fall from the shelves in an earthquake. Restraints should be provided to prevent the contents of rack shelving open to the general public from falling in strong ground shaking.

14.4 NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding structures not similar to buildings exhibit behavior markedly different from that of building structures. Most of these types of structures have adopted references that address their unique structural performance and behavior. The ground motion in the *Provisions* requires appropriate translation to allow use with industry standards. Such translation is provided in this section.

14.4.2 Earth retaining structures. In order to properly develop and implement methodologies for the design of earth retaining structures, it is essential to know and understand the nature of the applied loads. Concerns have been raised concerning the design of nonyielding walls and yielding walls for bending, overturning, sliding, etc., taking into account the varying soil types, importance, and site seismicity. See Sec. 7.5.1 in the *Commentary*.

14.4.3 Stacks and chimneys. The design of stacks and chimneys to resist natural hazards is generally governed by wind design considerations. The exceptions to this general rule involve locations with high seismicity, stacks and chimneys with large elevated masses, and stacks and chimneys with unusual geometries. It is prudent to evaluate the effect of seismic loads in all but those areas with the lowest seismicity. Although not specifically required, it is recommended that the special seismic details required elsewhere in the *Provisions* be evaluated for applicability to stacks and chimneys.

Guyed steel stacks and chimneys are generally light weight. As a result, the design loads due to natural hazards are generally governed by wind. On occasion, large flares or other elevated masses located near the top may require an in-depth seismic analysis. Although Chapter 6 of Troitsky does not specifically address seismic loading, it remains an applicable methodology for resolution of seismic forces that are defined in these *Provisions*.

14.4.7 Tanks and vessels. Methods of seismic design of tanks, currently adopted by a number of industry standards, have evolved from earlier analytical work by Jacobsen, Housner, Veletsos, Haroun, and others. The procedures used to design flat bottom storage tanks and liquid containers is based on the work of Housner, Wozniak, and Mitchell. The standards for tanks and vessels have specific requirements to safeguard against catastrophic failure of the primary structure based on observed behavior in seismic events since the 1930s. Other methods of analysis using flexible shell models have been proposed but are presently beyond the scope of these *Provisions.*

These methods entail three fundamental steps:

1. The dynamic modeling of the structure and its contents. When a liquid-filled tank is subjected to ground acceleration, the lower portion of the contained liquid, identified as the impulsive component of mass W_I, acts as if it were a solid mass rigidly attached to the tank wall. As this mass accelerates, it exerts a horizontal force, P_I, against the wall that is directly proportional to the maximum acceleration of the tank base. This force is superimposed on the inertia force of the accelerating wall itself, P_w. Under the influence of the same ground acceleration, the upper portion of the contained liquid responds as if it were a solid mass flexibly attached to the tank wall. This portion, which oscillates at its own natural frequency, is identified as the convective component W_c, and exerts a force P_c on the wall. The convective component oscillations are characterized by the phenomenon of sloshing whereby the liquid surface rises above the static level on one side of the tank, and drops below that level on the other.
2. The determination of the frequency of vibration, w_I, of the tank structure and the impulsive component; and the natural frequency of oscillation (sloshing), w_c, of the convective component.
3. The selection of the design response spectrum. The response spectrum may be site-specific or it may be constructed deterministically on the basis of seismic coefficients given in national codes and standards. Once the design response spectrum is constructed, the spectral accelerations corresponding to w_I and w_c are obtained and are used to calculate the dynamic forces P_I, P_w, and P_c.

Detailed guidelines for the seismic design of circular tanks, incorporating these concepts to varying degrees, have been the province of at least four industry standards: AWWA D100 for welded steel tanks (since 1964); API 650 for petroleum storage tanks; AWWA D110 for prestressed, wire-wrapped tanks (since 1986); and AWWA D115 for prestressed concrete tanks stressed with tendons (since 1995). In addition, API 650 and API 620 contain provisions for petroleum, petrochemical, and cryogenic storage tanks. The detail and rigor of analysis employed by these standards have evolved from a semi-static approach in the early editions to a more rigorous approach at the present, reflecting the need to factor in the dynamic properties of these structures.

The requirements in Sec 14.4.7 are intended to link the latest procedures for determining design level seismic loads with the allowable stress design procedures based on the methods in these Provisions. These requirements, which in many cases identify specific substitutions to be made in the design equations of the national standards, will assist users of the *Provisions* in making consistent interpretations.

ACI has published a document, ACI 350.3-01 titled "*Seismic Design of Liquid-Containing Concrete Structures.*" This document, which covers all types of concrete tanks (prestressed and non-prestressed, circular and rectilinear), has provisions made consistent with the seismic guidelines of the 2000 *Provisions*. This ACI document serves as both a practical "how-to" loading reference and a guide to supplement application of Chapter 21 "Special Provisions for Seismic Design" of ACI 318.

14.4.7.1 Design basis. Two important tasks of TS 13 were (a) to partially expand the coverage of nonbuilding structures in the *Provisions*; and (b) to provide comprehensive cross-references to all the applicable industry standards. It is hoped that this endeavor will bring about a standardization and consistency of design practices for the benefit of both the practicing engineer and the public at large.

In the case of the seismic design of nonbuilding structures, standardization requires adjustments to industry standards to minimize existing inconsistencies among them. However, the standardization process should recognize that structures designed and built over the years in accordance with industry standards have performed well in earthquakes of varying severity.

Of the inconsistencies among industry standards, the ones most important to seismic design relate to the base shear equation. The traditional base shear takes the following form:

$$V = \frac{ZIS}{R_w} CW$$

An examination of those terms as used in the different references reveals the following:

- *ZS*: The "seismic zone coefficient," *Z*, has been rather consistent among all the standards by virtue of the fact that it has traditionally been obtained from the seismic zone designations and maps in the model building codes.

  On the other hand, the "soil profile coefficient," *S*, does vary from one standard to another. In some standards these two terms are combined.

- *I*: The importance factor, *I*, has also varied from one standard to another, but this variation is unavoidable and understandable owing to the multitude of uses and degrees of importance of liquid-containing structures.

- *C*: The coefficient *C* represents the dynamic amplification factor that defines the shape of the design response spectrum for any given maximum ground acceleration. Since coefficient *C* is primarily a function of the frequency of vibration, inconsistencies in its derivation from one standard to another stem from at least two sources: differences in the equations for the determination of the natural frequency of vibration, and differences in the equation for the coefficient itself. (For example, for the shell/impulsive liquid component of lateral force, the steel tank standards use a constant design spectral acceleration (namely, a constant *C*) that is independent of the "impulsive" period *T*.) In addition, the value of *C* will vary depending on the damping ratio assumed for the vibrating structure (usually between 2 percent and 7 percent of critical).

  Where a site-specific response spectrum is available, calculation of the coefficient *C* is not necessary – except in the case of the convective component (coefficient C_c) which is assumed to oscillate with 0.5 percent of critical damping, and whose period of oscillation is usually high (greater than 2.5 sec). Since site-specific spectra are usually constructed for high damping values (3 percent to 7 percent of critical); and since the site-specific spectral profile may not be well-defined in the high-period range, an equation for C_c applicable to a 0.5 percent damping ratio is necessary in order to calculate the convective component of the seismic force.

- R_w: The "response modification factor," R_w, is perhaps the most difficult to quantify, for a number of reasons. While R_w is a compound coefficient that is supposed to reflect the ductility, energy-dissipating capacity, and redundancy of the structure, it is also influenced by serviceability considerations, particularly in the case of liquid-containing structures.

In the *Provisions* the base shear equation for most structures has been reduced to $V = C_sW$, where the seismic response coefficient, C_s, replaces the product $\frac{ZSC}{R_w}$. C_s is determined from the design spectral response acceleration parameters S_{DS} and S_{D1} (at short periods and at a period of 1 sec, respectively) which, in turn, are obtained from the mapped MCE spectral accelerations S_s and S_1 obtained from the seismic maps. As in the case of the prevailing industry standards, where a site-specific response spectrum is available, C_s is replaced by the actual spectral values of that spectrum.

As part of its task, TS 13 has introduced a number of provisions, in the form of bridging equations, each designed to provide a means of properly applying the design criteria of a particular industry standard in the context of these *Provisions*. These provisions are outlined below and are identified with particular types of liquid-containing structures and the corresponding standards. Underlying all these provisions is the understanding that the calculation of the periods of vibration of the impulsive and convective components is left up to the industry standards. Defining the detailed resistance and allowable stresses of the structural elements for each industry structure has also been left to the approved standard except in instances where additional information has led to additional requirements.

It is intended that, as the relevant national standards are updated to conform to these *Provisions*, the "bridging" equations of Sec. 14.4.7.6, 14.4.7.7, and 14.4.7.9 will be eliminated.

14.4.7.2 Strength and ductility. As is the case for building structures, ductility and redundancy in the lateral support systems for tanks and vessels are desirable and necessary for good seismic performance. Tanks and vessels are not highly redundant structural systems and, therefore, ductile materials and well-designed connection details are needed to increase the capacity of the vessel to absorb more energy without failure. The critical performance of many tanks and vessels is governed by shell stability requirements rather than by yielding of the structural elements. For example, contrary to building structures, ductile stretching of the anchor bolts is a desirable energy absorption component when tanks and vessels are anchored. The performance of cross-braced towers is highly dependent on the ability of the horizontal compression struts and connection details to fully develop the tension yielding in the rods. In such cases, it is also important to assure that the rods stretch rather than fail prematurely in the threaded portion of the connection and that the connection of the rod to the column does not fail prior to yielding of the rod.

14.4.7.3 Flexibility of piping attachments. The performance of piping connections under seismic deformations is one of the primary weaknesses observed in recent seismic events. Tank leakage and damage occurs when the piping connections cannot accommodate the movements the tank experiences during the a seismic event. Unlike the connection details used by many piping designers, which connections impart mechanical loading to the tank shell, piping systems in seismic areas should be designed in such a manner as to impose only negligible mechanical loads on the tank connection for the values shown in Table 14.4-1.

In addition, interconnected equipment, walkways, and bridging between multiple tanks must be designed to resist the loads and displacements imposed by seismic forces. Unless multiple tanks are founded on a single rigid foundation, walkways, piping, bridges, and other connecting structures must be designed to allow for the calculated differential movements between connected structures due to seismic loading assuming the tanks and vessels respond out of phase.

14.4.7.4 Anchorage. Many steel tanks can be designed without anchors by using the annular plate procedures given in the national standards. Tanks that must be anchored because of overturning potential could be susceptible to shell tearing if not properly designed. Ideally, the proper anchorage design will provide both a shell attachment and embedment detail that will yield the bolt without tearing the shell or pulling the bolt out the foundation. Properly designed anchored tanks retain greater reserve strength to resist seismic overload than do unanchored tanks.

Premature failure of anchor bolts has been observed where the bolt and attachment are not properly aligned (that is, the anchor nut or washer does not bear evenly on the attachment). Additional bending stresses in threaded areas may cause the anchor to fail before yielding.

14.4.7.5 Ground-supported storage tanks for liquids

14.4.7.5.1 Seismic forces. The response of ground storage tanks to earthquakes is well documented by Housner, Mitchell and Wozniak, Veletsos, and others. Unlike building structures, the structural response is strongly influenced by the fluid-structure interaction. Fluid-structure interaction forces are categorized as sloshing (convective mass) and rigid (impulsive mass) forces. The proportion of these forces depends on the geometry (height-to-diameter ratio) of the tank. API 650, API 620, AWWA D100, AWWA D110, AWWA D115, and ACI 350.3 provide the necessary data to determine the relative masses and moments for each of these contributions.

The *Provisions* stipulate that these structures shall be designed in accordance with the prevailing approved industry standards, with the exception of the height of the sloshing wave, d_s, which is to be calculated using Eq. 14.4-9 of these *Provisions*.

$$\delta_s = 0.5DIS_{ac}$$

This equation utilizes a spectral response coefficient $S_{ac} = \dfrac{1.5S_{D1}}{T_c}$ for $T_c < 4.0$ sec., and $S_{ac} = \dfrac{6S_{D1}}{T_c^2}$ for $T_c > 4.0$ sec. The first definition of S_a represents the constant-velocity region of the response spectrum and the second the constant-displacement region of the response spectrum, both at 0.5 percent damping. In practical terms, the latter is the more commonly used definition since most tanks have a fundamental period of liquid oscillation (sloshing wave period) greater than 4.0 sec.

Small diameter tanks and vessels are more susceptible to overturning and vertical buckling. As a general rule, the greater the ratio of H/D, the lower the resistance is to vertical buckling. When $H/D > 2$, the overturning begins to approach “rigid mass” behavior (the sloshing mass is small). Large diameter tanks may be governed by additional hydrodynamic hoop stresses in the middle regions of the shell.

The impulsive period (the natural period of the tank components and the impulsive component of the liquid) is typically in the 0.25 to 0.6 second range. Many methods are available for calculating the impulsive period. The Veletsos flexible-shell method is commonly used by many tank designers. (For example, see “Seismic Effects in Flexible Liquid Storage Tanks” by A. S. Veletsos.)

14.4.7.5.2 Distribution of hydrodynamic and inertia forces. Most of the methods contained in the industry standards for tanks define reaction loads at the base of the shell and foundation interface. Many of the standards do not give specific guidance for determining the distribution of the loads on the shell as a function of height. The design professional may find the additional information contained in ACI 350.3 helpful.

The overturning moment at the base of the shell as defined in the industry standards is only the portion of the moment that is transferred to the shell. It is important for the design professional to realize that the total overturning moment must also include the variation in bottom pressure. This is important when designing pile caps, slabs, or other support elements that must resist the total overturning moment. See Wozniak or TID 7024 for further information.

14.4.7.5.3 Freeboard. Performance of ground storage tanks in past earthquakes has indicated that sloshing of the contents can cause leakage and damage to the roof and internal components. While the effect of sloshing often involves only the cost and inconvenience of making repairs, rather than catastrophic failure, even this limited damage can be prevented or significantly mitigated when the following items are considered:

1. Effective masses and hydro-dynamic forces in the container.
2. Impulsive and pressure loads at
    a. Sloshing zone (that is, the upper shell and edge of the roof system),
    b. Internal supports (roof support columns, tray-supports, etc.), and
    c. Equipment (distribution rings, access tubes, pump wells, risers, etc.).
3. Freeboard (which depends on the sloshing wave height).

A minimum freeboard of $0.7\delta_s$ is recommended for economic considerations but is not required.

Tanks and vessels storing biologically or environmentally benign materials do not typically require freeboard to protect the public health and safety. However, providing freeboard in areas of frequent seismic occurrence for vessels normally operated at or near top capacity may lessen damage (and the cost of subsequent repairs) to the roof and upper container.

The estimate given in the Provision Sec. 14.4.7.5.3 is based on the seismic design event as defined by the Provisions. Users of the Provisions may estimate slosh heights different from those recommended in the national standards.

If sloshing is restricted because the freeboard provided is less than the computed sloshing height, δ_s, the sloshing liquid will impinge on the roof in the vicinity of the roof-to-wall joint, subjecting it to a hydrodynamic force. This force may be approximated by considering the sloshing wave as a hypothetical static liquid column having a height, δ_s. The pressure exerted on any point along the roof at a distance y_s above the at-rest surface of the stored liquid, may be assumed equal to the hydrostatic pressure exerted by the hypothetical liquid column at a distance $\delta_s - y_s$ from the top of that column

Another effect of a less-than-full freeboard is that the restricted convective (sloshing) mass "converts" into an impulsive mass thus increasing the impulsive forces. This effect should be taken account in the tank design. Preferably, sufficient freeboard should be provided whenever possible to accommodate the full sloshing height.

14.4.7.5.6 Sliding resistance. Steel ground-supported tanks full of product have not been found to slide off foundations. A few unanchored, empty tanks have moved laterally during earthquake ground shaking. In most cases, these tanks may be returned to their proper locations. Resistance to sliding is obtained from the frictional resistance between the steel bottom and the sand cushion on which bottoms are placed. Because tank bottoms usually are crowned upward toward the tank center and are constructed of overlapping, fillet-welded, individual steel plates (resulting in a rough bottom), it is reasonably conservative to take the ultimate coefficient of friction as 0.70 (U.S. Nuclear Regulatory Commission, 1989, pg. A-50) and, therefore, a value of tan 30^{o} (= 0.577) is used. The vertical weight of the tank and contents as reduced by the component of vertical acceleration provides the net vertical load. An orthogonal combination of vertical and horizontal seismic forces following the procedure in Sec. 5.2 may be used.

14.4.7.5.7 Local shear transfer. The transfer of seismic shear from the roof to the shell and from the shell to the base is accomplished by a combination of membrane shear and radial shear in the wall of the tank. For steel tanks, the radial shear is very small and is usually neglected; thus, the shear is assumed to be carried totally by membrane shear. For concrete walls and shells, which have a greater radial shear stiffness, the shear transfer may be shared. The user is referred to the ACI 350 commentary for further discussion.

14.4.7.5.8 Pressure stability. Internal pressure may increase the critical buckling capacity of a shell. Provision to include pressure stability in determining the buckling resistance of the shell for overturning loads is included in AWWA D100. Recent testing on conical and cylindrical shells with internal pressure yielded a design methodology for resisting permanent loads in addition to temporary wind and seismic loads. See Miller et al., 1997.

14.4.7.5.9 Shell support. Anchored steel tanks should be shimmed and grouted to provide proper support for the shell and to reduce impact on the anchor bolts under reversible loads. The high bearing pressures on the toe of the tank shell may cause inelastic deformations in compressible material (such as fiberboard), creating a gap between the anchor and the attachment. As the load reverses, the bolt is no longer snug and an impact of the attachment on the anchor can occur. Grout is a structural element and should be installed and inspected as if it is an important part of the vertical- and lateral-force-resisting system.

14.4.7.5.10 Repair, alteration, or reconstruction. During their service life, storage tanks are frequently repaired, modified or relocated. Repairs or often related to corrosion, improper operation, or overload from wind or seismic events. Modifications are made for changes in service, updates to safety equipment for changing regulations, installation of additional process piping connections. It is imperative these repairs and modifications are properly designed and implemented to maintain the structural integrity of the tank or vessel for seismic loads as well as the design operating loads.

The petroleum steel tank industry has developed specific guidelines in API 653 that are statutory requirements in some states. It is the intent of TS 13 that the provisions of API 653 also be applied to other liquid storage tanks (water, wastewater, chemical, etc.) as it relates to repairs, modifications or relocation that affects the pressure boundary or lateral force resisting system of the tank or vessel.

14.4.7.6 Water and water treatment structures

14.4.7.6.1 Welded steel. The AWWA design requirements for ground-supported steel water storage structures are based on an allowable stress method that utilizes an effective mass procedure considering two response modes for the tank and its contents:

1. The high-frequency amplified response to seismic motion of the tank shell, roof, and impulsive mass (that portion of liquid content of the tank that moves in unison with the shell), and
2. The low-frequency amplified response of the convective mass (that portion of the liquid contents in the fundamental sloshing mode).

The two-part AWWA equation incorporates the above modes, appropriate damping, site amplification, allowable stress response modification, and zone coefficients. In practice, the typical ground storage tank and impulsive contents will have a natural period, T, of 0.1 to 0.3 sec. The sloshing period typically will be greater than 1 sec (usually 3 to 5 seconds depending on tank geometry). Thus, the substitution in the *Provisions* uses a short- and long-period response as it applies to the appropriate constituent term in the AWWA equations.

14.4.7.6.2 Bolted steel. The AWWA Steel Tank Committee is responsible for the content of both the AWWA D100 and D103 and have established equivalent load and design criteria for earthquake design of welded and bolted steel tanks.

14.4.7.7 Petrochemical and industrial liquids

14.4.7.7.1 Welded steel. The American Petroleum Institute (API) also uses an allowable stress design procedure and the API equation has incorporated an R_w factor into the equations directly.

The most common damage to tanks observed during past earthquakes include:

- Buckling of the tank shell near the base due to excessive axial membrane forces. This buckling damage is usually evident as "elephant foot" buckles a short distance above the base, or as diamond shaped buckles in the lower ring. Buckling of the upper ring has also been observed.
- Damage to the roof due to impingement on the underside of the roof of sloshing liquid with insufficient freeboard.
- Failure of piping or other attachments that are overly restrained..
- Foundation failures.

The performance of floating roofs during earthquakes has been good, with damage usually confined to the rim seals, gage poles, and ladders. Similarly the performance of open tops with top wind girder stiffeners designed per API 650 has been good.

14.4.7.9 Elevated tanks for liquids and granular materials. There are three basic lateral-load resisting systems for elevated water tanks that are defined by their support structure. Multi-leg braced steel tanks (trussed towers), small diameter single-pedestal steel tanks (cantilever columns), and large diameter single-pedestal tanks of steel or concrete construction (load-bearing shear walls). Unbraced multi-leg tanks are not commonly built. Behavior, redundancy, and resistance to overload of these types of tanks are not the same. Multi-leg and small diameter pedestal have higher fundamental periods (typically over 2-sec) than the shear wall type tanks (typically under 2-sec). Lateral load failure mechanism is usually by bracing failure for multi-leg tanks, compression buckling of small diameter steel tanks, compression or shear buckling of large diameter steel tanks, and shear failure of large diameter concrete tanks. In order to utilize the full strength of these structures adequate connection, welding, and reinforcement details must be provided. The R-factor used with elevated tanks is typically less than that for comparable lateral load-resisting systems for other purposes in order to provide a greater margin of safety.

14.4.7.9.3 Transfer of lateral forces into support tower. The lateral transfer of load for tanks and vessels siting on grillage or support beams should consider the relative stiffness of the support beams and the shear transfer at the base of the shell, which is not typically uniform around the base of the tank. In addition, when tanks and vessels are supported on discrete points on grillage or beams, it is common for the vertical loads to vary due to settlements or variations in construction. This variation in load should be considered when analyzing the combined vertical and horizontal loads.

14.4.7.9.4 Evaluation of structures sensitive to buckling failure. Nonbuilding structures that have low or negligible structural redundancy for lateral loads need to be evaluated for a critical level of performance to provide sufficient margin against premature failure. Reserve strength for loads beyond the design loads can be limited. Tanks and vessels supported on shell skirts or pedestals that are governed by buckling are examples of structures that need to be evaluated at this critical condition. Such structures include single pedestal water towers, process vessels, and other single member towers.

The additional evaluation is based on a scaled maximum considered earthquake. This critical earthquake acceleration is defined as the design spectral response acceleration, S_a, which includes site factors. The I/R coefficient is taken as 1.0 for this critical check. The structural capacity of the shell is taken as the critical buckling strength (that is, the factor of safety is 1.0). Vertical or orthogonal earthquake combination need not be made for this critical evaluation since the probability of critical peak values occurring simultaneously is very low.

14.4.7.9.6 Concrete pedestal (composite) tanks. A composite elevated water-storage tank is a structure comprising a welded steel tank for watertight containment, a single pedestal concrete support structure, foundation, and accessories. Lateral load-resisting system is that of a load-bearing concrete shear wall. Seismic provisions in ATC 371R-98 are based on ASCE 7-95, which used NEHRP 1994 as the source document. Seismic provisions in the proposed AWWA standard being prepared by committee D170 are based on ASCE 7-98, which used NEHRP 1997 as the source document.

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