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 was composed of 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 or if the accepted standards are adequate. It is hoped that the 1997 *Provisions* requirements for nonbuilding *structures* appropriately bridge the gap between building code and existing industry standards.

One of TS13's goals 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 are steel multilegged water towers. Historically, such towers have performed well when properly designed per American Water Works Association (AWWA) standards, but these standards differ from the *Provisions* because 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 *Provision's* 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, nuclear power plants, and dams are excluded from the scope of the nonbuilding structure requirements. The excluded structures are covered either by other sections of the *Provisions* or by other well established design criteria (vehicular and railroad bridges, nuclear power plants, and dams).

14.2 REFERENCES:

American Concrete Institute, (ACI):

ANSI/ACI 349-90 *Code Requirements for Nuclear Safety Related Structures* - Appendix B, 1990. (ACI 349)

ACI 350-99, *Environmental Concrete Concrete Structures*, 1999. (ACI 350)

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

ASCE American Society of Civil Engineers (ASCE), New York:

Petrochemical Energy Committee Task Report, "Guidelines for Seismic Evaluation and Design of Petrochemical Facilities", ASCE publication, 1997. (ASCE Guidelines for Seismic Evaluation and Design of Petrochemical Facilities)

Guidelines for the Seismic Design of Oil and Gas Pipeline Systems, New York, NY, 1984 (ASCE *Guidelines for the Seismic Design of Oil and Gas Pipeline Systems*).

Gaylord and Gaylord, Design of Steel Bins for Storage of Bulk Solids, Prentice Hall, 1984. (Gaylord and Gaylord 1984)

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

Miller, C. D., Meier, S. W., Czaska, W. J., *Effects of Internal Pressure on Axial Compressive*

Strength of Cylinders and Cones, Structural Stability Research Council Annual Technical Meeting, June 1997. (Miller 1997)

NFPA National Fire Protection Association

Standard, ANSI/NFPA 30-1996, *Flammable and Combustible Liquids Code*, 1996. (NFPA 30)

Standard, ANSI/NFPA 58-1995, *Storage and Handling of Liquefied Petroleum Gas*. (NFPA 58)

Standard, ANSI/NFPA 59-1998, *Storage and Handling of Liquefied Petroleum Gases at Utility Gas Plants*. (NFPA 59)

Standard, ANSI/NFPA 59A-1996, *Production, Storage and Handling of Liquefied Natural Gas (LNG)*. (NFPA 59A)

RMI Rack Manufacturers Institute

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

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

Wozniak, R. S. and Mitchell, W. W, *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 1978)

Zick, L.P., *Stresses in Large Horizontal Cylindrical Pressure Vessels on Two Saddle Supports*, Steel Plate Engineering Data, Vol 1and2, American Iron and Steel Institute, Dec 1992. (Zick 1992)

14.4 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*.

14.5 STRUCTURAL DESIGN REQUIREMENTS:

14.5.1 Design Basis: The subcommittee wanted to employ the new seismic ground motion maps and the new methodology for establishing seismic design and detailing contained in the 1997 *Provisions*.

14.5.1.1 Seismic Factors: Table 14.2.1.1 has been formulated to be consistent with the *Provisions*. The values listed here are generally lower than the values for buildings. Lower values are assigned in recognition of the structural performance of nonbuilding structures as opposed to building structures. Nonbuilding structures tend to be lightly damped, less redundant, and more given to performance failure when the structure exhibits nonlinear performance.

14.5.1.2 Importance Factors and Seismic Use Groups Classifications: The Importance Factors and Seismic Use Group classifications assigned nonbuilding structures vary from those assigned 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 (i.e., 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 (i.e., fire fighting, potable water), potentially harmful or hazardous to the environment or general health of the public, biologically lethal or toxic, or explosive or flammable (threat of consequential or secondary damage).

If not covered by the authority having jurisdiction, Table 14.5.1.2 may be used to select the importance factor *(I)*. The value shall be determined by the largest value from the approved Standards, or largest value selected from Table 14.2.1.2. It should be noted that an entire facility need not be restricted to use only one single value of important factor. For further details, refer to

ASCE *Guidelines for Seismic Evaluation and Design of Petrochemical Facilities (*ASCE, 1997). Also, Use of Secondary Containment System, when designed in accordance with an acceptable National Standards, could be considered as an effective means to contain hazardous substance hence reduce the level of H selection.

The specific definition of material hazard and what constitutes a hazard is currently being developed in the 2000 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 other purposes other than that defined in this provision. This include use of higher important factor in order to compensate the use of Site Specific Response Spectra.

Following are examples demonstrating 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 14.5.1.2 Importance Factor (*I*) and Seismic Use Group Classification for Nonbuilding Structures

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

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.

Hazard — The contents are not hazardous, therefore use H - I.

Function — The water storage tank is not a designated ancillary structure for post–earthquake recovery, nor serves as emergency back-up facilities for a Seismic Use Group III structure, therefor use F - 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 from a slab on grade.

TABLE 14.5.1.2 Importance Factor (*I*) and Seismic Use Group Classification for Nonbuilding Structures

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

Address each of the issues in the matrix:

Seismic Use Group — Neither the structure nor the contents are critical, therefor use Seismic Use Group I.

Hazard — The contents are not hazardous, however its use could cause a substantial public hazard during earthquake, – subject to local Authority's jurisdiction it is H-II.

Function — The storage rack is not used for earthquake recovery, nor is it required for emergency back-up, therefore use F - I.

Within the steel storage rack section in the *Provisions* there exists a link back to Sec. 6.9 and to Sec. 6.1.5 requiring an I_p or I of 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 14.5.1.2 Importance Factor (*I*) and Seismic Use Group Classification for Nonbuilding Structures

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

Address each of the issues in the matrix:

Seismic Use Group — The office building is Seismic Use Group I.

Hazard — The content and its use are not hazardous to the public, therefore use H - I.

Function — The water tank is required to provide water for fire fighting, however since the Building is not a Seismic Use Group III structures, the water is not used for post earthquake recovery, nor is it required for emergency back-up, therefor use F - I.

Use an importance factor of 1.0 for this water structure.

Example 4:

A petro-chemical storage tank is to be constructed within a refinery tank farm near a populated City neighborhood. Impoundment dike is provided to control liquid spills.

Table 14.5.1.2
Importance Factor (*I*) and Seismic Use Group Classification for Nonbuilding Structures

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

Address each of the issues in the matrix.:

Seismic Use Group — The LNG tank is Seismic Use Group III.

Hazard — The contents constitute a sufficient quantities of high explosive and is near a city neighborhood, despite the diking, it is considered hazardous to the public under earthquake, therefore use H - III.

Function — The tank is not required to provide post-earthquake recovery, nor is used for emergency back-up for Seismic Use Group III structures therefore use F - I.

Use an importance factor of 1.5 for this structure.

14.5.2 Rigid Nonbuilding Structures: The equation included in the 1994 *Provisions* did not agree with the formulas contained in the 1994 *Uniform Building Code* (*UBC*). The Seismic Design Procedure Group recommended using the S_{DS} factor and eliminating the C_a factor. The appropriate changes are incorporated in the 1997 *Provisions*.

14.5.4 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 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 in for specific types of nonbuilding structures may contain more accurate methods for period determination.

14.6 NONBUILDING STRUCTURES SIMILAR TO BUILDINGS: This general class of nonbuilding structures exhibits behavior similar to that of building structure; however, function and performance are different. The *Provisions* were used as the primary basis for design with industry-driven exceptions, modifications, and additions.

14.6.2 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 meet the force requirements of Sec. 5.4. Single column pipe racks that resist lateral loads should be designed as inverted pendulums. See ASCE "Guidelines for the Seismic Evaluation and design of Petrochemical Facilities (1997).

14.6.3 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 and to distinguish between the methods employed to design storage racks supported at grade (as treated in Sec. 14.3.3 Steel Storage Racks, Nonbuilding Structures) from those supported above grade (as treated in Sect. 6.1 Architectural, Mechanical, and Electrical Components Seismic Design Requirements). This will help clarify and coordinate the multiple references to rack structures in these *Provisions* and the different means by which rack structures are analyzed and designed.

The RMI for many years has been working with the various committees of the model code organizations and of the Building Seismic Safety Council and its Technical Sub-Committees to create seismic design provisions particularly applicable to steel storage rack structures. The new 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 new RMI Specification, now including detailed seismic provisions, is seen to be self-sufficient. The 1997 Edition of the RMI Specification is presently undergoing the ANSI canvassing process.

The changes proposed here are compatible and coordinated with the changes recently approved, in March 1999, by the IBC Structural Committee for inclusion in the IBC 2000.

14.6.4 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. For this reason, the exception to height limits in Sec. 14.5.1 was required.

14.6.6 Piers and Wharves: Although previous editions of the *Provisions* did not include a specific section on piers and wharves, the inclusion of these structures was deemed necessary to properly account for the effect of hydrodynamic and liquefaction effects unique to these types of structures.

14.7 NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS: This general class of nonbuilding structures exhibits behavior markedly different from that of building structures. Most of these types of structures have industry standards that address their unique structural performance and behavior. The new elements of the *1997 Provisions* regarding ground motion required that a prudent link to the industry standards be developed.

14.7.1 General:

14.7.2 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 on how to design 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*.

14.7.3 Tanks and Vessels:

14.7.3.1 General: 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 and 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:

I. The dynamic modeling of the structure and its contents. When a liquid-filled tank is subjected to a 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 liquid 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.

II. 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.

III. 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 14.7.3 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.

More recently, ACI Committee 350 has drafted a document, ACI 350.3, titled *"ACI Practice for the Seismic Design of Liquid-Containing Structures"*. This document, which covers all types of concrete tanks (prestressed and non-prestressed, circular and rectilinear), is currently being revised to conform with the seismic risk guidelines of NEHRP 1997 and IBC 2000. This ACI *"Practice"* will serve as a practical, "how-to" - and yet rigorous - guide to supplement Chapter 21 (*"Special Provisions for Seismic Design"*) of ACI 350.

14.7.3.2 Design Basis: Two important tasks of TS-13 are to (a) partially expand the Provision's coverage of nonbuilding structures; and (b) provide comprehensive cross-references to all the applicable industry standards. This endeavor will hopefully 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 will probably necessitate certain adjustments on the part of current industry standards to minimize existing inconsistencies among them. At the same time, however, this process must take cognizance of the fact that structures designed and built over the years in accordance with these standards have performed well in earthquakes of varying severity.

The most important inconsistencies among current standards that need to be addressed in any standardization/update process relate primarily to differences in the derivation of the terms that make up the traditional base shear equation :

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

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

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

  On the other hand, "Soil Profile Coefficient" S does vary from one standard to another. In some standards these two terms are combined.

- I: 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: 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 C 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 (2 percent - 7 percent).

Where a site-specific response spectrum is available, calculation of 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 (>2.5 seconds). Since site-specific spectra are usually constructed for high damping values (3 percent - 7 percent); and since the site-specific spectral profile may not be well-defined in the high-period range, an equation for C_c applicable to 0.5 percent damping ratio is necessary in order to calculate the convective component of the seismic force.

- R: 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 structural redundancy of the structure, it is also influenced by serviceability considerations, particularly in the case of liquid-containing structures.

In NEHRP 1997 and IBC 2000, the base shear equation for most structures has been reduced to $V = C_s W$, where the Seismic Response Coefficient C_s replaces the product $\frac{ZSC}{R_w}$. C_s is determined from the Design Spectral Response Accelerations S_{DS} or S_{D1} (at short periods, or at 1 second period respectively) which, in turn, are obtained from the mapped MCE (Maximum Considered Earthquake) spectral accelerations S_s and S_1 obtained from the new 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, each designed to provide a means of properly applying the design criteria of a particular industry standard with the latest NEHRP practices. 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 industrial standards. Defining the detailed resistance and allowable stresses of the structural elements for each industrial structure has also been left to the approved standard except in instances where additional information has led to additional requirements.

14.7.3.3 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 that the rods stretch and do not fail prematurely in the threaded portion of the connection, or the connection of the rod to the column fail prior to yielding of the rod.

14.7.3.4 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. Contrary to the design methods used by many piping designers, which impart mechanical loading to the tank shell, piping systems in seismic areas should be designed in such a manner as to impose negligible mechanical loads on the tank connection for the values shown in Table 14.4.3.1.2.

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 are out of phase

14.7.3.5 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 unanchored tanks.

Premature failure of anchor bolts has been observed when the bolt and attachment are not properly aligned (i.e 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

14.7.3.6 Ground-Supported Storage Tanks for Liquids:

14.7.3.6.1 General: 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 350.3provide 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 defined by equation (14.7.3.7.1) of these Provisions.

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

This equation utilizes a spectral response coefficient $S_{ac} = \frac{1.5S_{D1}}{T_c}$ for $T_c < 4.0$ sec., and

$S_{ac} = \frac{6S_{D1}}{T_c^{\,2}}$ for $T_c > 4.0$ sec. The first definition of S_a represents the constant-velocity region of the response spectra and the second the constant-displacement region of the response spectrum at 0.5 percent damping. In practical terms, the latter is the most commonly used definition since most tanks have a fundamental period of liquid oscillation (sloshing wave period) greater than 2.5 sec., and, most commonly, 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" A.S. Veletsos).

14.7.3.6.1.1 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 is defined in the industry standards is only the portion of the moment that is transferred to the shell. It is important the design professional realize that this 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 1978 or TID 7024 for further information.

14.7.3.6.1.2 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, not catastrophic failure, even this limited damage can be prevented or significantly mitigated when the following aspects are considered:

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

A minimum freeboard of $0.7\delta_s$ is recommended for economic considerations but 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 14.7.3.7.1.2 is based on a median response spectrum rather than on the one standard deviation response spectra found in TID 7024. It is also based on the seismic design event as defined by the *Provisions*. Estimates for the sloshing height contained in national standards are based on the one standard deviation spectra applied at a working stress level. Users of the *Provisions* may estimate slosh heights different from those recommended in the national standards.

14.7.3.6.1.5 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. 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 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.

14.7.3.6.1.6 Local Shear Transfer: The lateral seismic shear from the roof to the shell and the shell to the base is resisted by a combination of membrane shear and the 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 shell, which have a greater radial shear stiffness, the shear transfer may be shared. The user is referred to Commentary of ACI 350 for further discussion.

14.7.3.6.1.7 Pressure Stability: Internal pressure may increase the critical buckling capacity of a shell. Provisions 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.

14.7.3.6.1.8 Shell Support: Anchored steel tanks should be shimmed and grouted to provide proper support for the shell and reduce impact on the anchor bolt under reversible loads. The high bearing pressures on the toe of the tank shell may cause inelastic deformations in compressible material (i.e. fiberboard) that creates 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.

14.7.3.6.1.9 Repairs, Alterations, and Modifications: 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, wasterwater, chemical, etc) as it relates to repairs, modifications or relocation that effects the pressure boundary or lateral force resisting system of the tank or vessel.

14.7.3.7 Water and Water Treatment Tanks and Vessels:

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

1. The high-frequency amplified response to seismic motion of the tank shell, roof, and impulsive mass (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 (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 sec 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.

14.7.3.7.2 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.

14.7.3.8.3 Reinforced and Prestressed Concrete: Given T_I, the natural period of tank shell plus the confined (impulsive) liquid; and T_C (or T_W), the first-mode sloshing wave period (as defined in 14.7.3.7.1 of these Provisions),

(a) For $T_I < T_o$, and $T_I > T_s$, the term $\frac{ZICi}{Ri}$ in the base shear and overturning moment equations of AWWA D110-95 and D115-95; and the term $\frac{ZISG_i}{R_i}$ in the base shear and overturning moment equations of draft ACI 350.3 are both replaced by $\frac{S_a I}{1.4R}$

(b) For $T_o \le T_I \le T_s$, $\frac{ZICi}{Ri}$ and $\frac{ZISG_i}{R_i}$ are replaced by $\frac{S_{DS} I}{1.4R}$

(c) For all values of T_C, $\frac{ZIC_c}{R_c}$ and $\frac{ZISC_c}{R_c}$ are replaced by $\frac{6S_{D1}I}{T_c^{\,2}}...or...\frac{6S_{DS}I}{T_c^{\,2}}T_s$

where T, S_a, S_1, and S_{DS} are defined in Sec. 4.1.2.6, and T_c.

14.7.3.8 Petrochemical and Industrial Tanks and Vessels Storing Liquids

14.7.3.8.1 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 impingment 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 top with top wind girder stiffeners designed per API 650 has been good.

14.7.3.10 Elevated Tanks and Vessels for Liquids and Granular Materials:

14.7.3.10.4 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.

14.7.3.10.5 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 beyond 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 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, Sa, 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 (i.e. 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.

14.7.3.10.7 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.

As these structures began in the market place, the design–build firms developed proprietary standards and methods for their structures. The Steel Plate Fabricators Association developed a guideline specification for this style tank in the early 1990's. After debate, an AWWA Committee was formed in 1992 to prepare a standard for composite elevated water tanks. Also in 1992, the American Concrete Institute Committee 371 began work on a recommended practice for the design and construction of concrete-pedestal water towers (ACI 371R), which was first published in 1998. ACI 371R focused on the application of loads to the structure, and on the design and construction aspects of concrete *components* and foundations. Design and construction requirements for the steel tank were included by reference to national standards. The draft AWWA D170 uses applicable portions of ACI 371R and AWWA D100 as resources, and expands upon them to provide a comprehensive design and construction document for composite elevated water tanks.

There is limited experience with the seismic performance of this type of tank compared to the other styles of elevated water storage tanks built in the US for the past several decades. This style of tank was initially marketed and built in Canada and the southwest US (primarily in Texas), primarily in regions of low seismicity. While this style of tank has spread to cover much of the eastern US, none have been located in an area where a significant seismic event has occurred. The design rules in the Provisions are based on present day design procedures and engineering principles used by the design-builders, the ACI 371 recommended practice, and the draft AWWA standard. All of these methods are at present unproven.

14.7.4 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 such 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, "Multilevel Guyed Stacks" in *Tubular Steel Structures* by M. S. Troitsky does not specifically address seismic loading, it remains an applicable methodology for resolution of seismic forces that can be determined in these*Provisions*.

Appendix to Chapter 14

PREFACE: The following sections were originally intended to be part of the Nonbuilding Structures Chapter of this Commentary. The *Provisions* Update Committee felt that given the complexity of the issues, the varied nature of the resource documents, and the lack of supporting consensus resource documents, time did not allow a sufficient review of the proposed sections required for inclusion into the main body of the chapter.

The Nonbuilding Structures Technical Subcommittee, however, expressed that what is presented herein represents the current industry accepted design practice within the engineering community that specializes in these types of nonbuilding structures.

The *Commentary* sections are included here so that the design community specializing in these nonbuilding structures can have the opportunity to gain a familiarity with the concepts, update their standards, and send comments on this appendix to the BSSC.

It is hoped that the various consensus design standards will be updated to include the design and construction methodology presented in this Appendix. It is also hoped that industry standards that are currently not consensus documents will endeavor to move their standards through the consensus process facilitating building code inclusion.

C14A.1 REFERENCES:

Agrawal P.K. and Kramer J.M., Analysis of Transmission Structures and Substation structures and Equipment for Seismic Loading, Sargent & Lundy Transmission and Substation Conference, December 2, 1976. (Agrawal 1976)

American Society of Civil Engineers (ASCE):
ANSI/ASCE 10-97, *Design of Latticed Transmission Structures*, New York, NY, 1997. (ASCE 10)

ASCE Manual 72, *Tubular Pole Design Standard*, New York, NY, 1991 (ASCE 72).

ASCE Manual 74, *Guidelines for Electrical Transmission Line Structural Loading*, New York, NY, 2000. (ASCE 74).

ASCE 7-95, *Minimum Design Loads for Buildings and Other Structures,* 1995 (ASCE 7).

American Society of Civil Engineers, (ASCE 1997), ASCE Manual 91, *The Design of Guyed Electrical Transmission Structur*es, New York, NY, 1997. (ASCE 97)

Substation Structure Design Guide, New York, NY, 2000. (ASCE 2000)

LI, H-N., Wang, S., Lu, M., and Wang, Q., Aseismic Calculations for Transmission Towers, ASCE Technical Council on Lifeline Earthquake Engineering, Monograph No. 4, August, 1991. (ASCE Li Monograph 4)

Steinhardt, Otto W., Low Cost Seismic Strengthening of Power Systems, Journal of The Technical Councils of ASCE, American Society of Civil Engineers, April 1981. (ASCE Steinhardt 1981)

Ameri, G. G. and McClure, G., Seismic Response to Tall Guyed Telecommunication Towers, Paper No. 1982, Eleventh World Conference on Earthquake Engineering, Elsevier Science Ltd., 1996. (Ameri 1996)

Australian Standards:
Australian Standard 3995, Standard Design of Steel Lattice Towers and Masts, 1994. (AS 3995)

Canadian Standards Association (CSA):
Antennas, Towers, and Masts, 1994, (AS 3995)

Earthquake Engineering Research Institute (EERI):
Li, H-N., Suarez, and Singh M.P., Seismic Effects on High-Voltage Transmission Tower and Cable Systems, EERI Conference, Chicago 1994. (EERI Seismic Effects 1994)

Federal Emergency Management Agency (FEMA):
Earthquake Resistant Construction of Electric Transmission and Telecommunication Facilities Serving the Federal Government Report, FEMA-202, Federal Emergency Management Agency, September 1990. (FEMA 202)

Galvez, C. A., and McClure, G., A Simplified Method for Aseismic Design of Self-Supporting Latticed Telecommunication Towers, Seventh Canadian Conference on Earthquake Engineering, Montreal, 1995. (Galvez 1995)

Institute of Electrical and Electronics Engineers (IEEE):
National Electrical Safety Code, ANSI C2, New Jersey, 1997. (NESC)

IEEE Standard 693, Recommended Practices for Seismic Design of Substations, Power Engineering Society, Piscataway, New Jersey, 1997 (IEEE 693).

IEEE Standard 751, *Trial-Use Design Guide for Wood Transmission Structures*, Power Engineering Society, Piscataway, New Jersey, 1991. (IEEE 751)

Long, L.W., Analysis of Seismic Effects on Transmission Structures, IEEE Paper T 73 326-6, April 1973. (IEEE T 73 326-6).

Lum, W. B., Nielson, N. N., Koyanagi, R., and Chui, A. N. L., Damage Survey of the Kasiki, Hawaii Earthquake of November 16, 1993, Earthquake Spectra, November 1984. (Lum 1984)

Lyver T.D., Mueller W.H., Kempner L, Jr., Response Modification Factor, R_w, for Transmission

Towers, Research Report, Portland State University, Portland, Oregon, 1996. (Lyver 1996)Towers, Research Report, Portland State University, Portland, Oregon, 1996. (Lyver 1996)

National Center for Earthquake Engineering Research (NCEER):
The Hanshin-Awaji Earthquake of January 17, 1995 Performance of Lifelines, National Center for Earthquake Engineering Research, Technical Report NCEER-95-0015, State University of New York at Buffalo, November 3, 1995. (NCEER-95-0015)

Rural Electrical Administration (REA):
Bulletin 1724E-200, *Design Manual for High Voltage Transmission Lines*, 1992 (REA 1724).

Bulletin 65-1, Design Guide for Rural Substations, 1978 (REA 65-1).

Bulletin 160-2, *Mechanical Design Manual for Overhead Distribution Lines*, 1982. (REA 160)

Telecommunications Industry Association (TIA):
TIA/EIA 222F *Structural Standards for Steel Antenna Towers and Antenna Supporting Structures*, 1996. (TIA 222)

C14A.2 ELECTRICAL TRAMSMISSION, SUBSTATION, AND DISTRIBUTION STRUCTURES: The design of electrical transmission, substation wire support, and distribution structures is typically controlled by high wind, ice-wind combinations, and unbalance longitudinal wire loads (Agrawal 1976, ASCE 74, ASCE 72, ASCE 10, ASCE 2000, ASCE 97, REA 65-1, NESC). Distribution structures typically support equipment with low mass and seismic loads do not control their design (REA 160, REA 1724, IEEE 751, NESC). Earthquake performance of these structures has demonstrated that seismic loads can be resisted based on traditional electrical transmission, substation, and distribution wire support structure loading (Steinhardt 1981). These structures may be used in special situations were seismic loads should be considered in their design. The special situations for transmission and substation wire support structures may include site specific low wind velocity and ice load, and no designed unbalance longitudinal wire load. For distribution structures, the number of supported transformers may result in significant seismic load. Seismic lateral loads and design criteria for substation structures should satisfy the requirements of IEEE 693, 1997.

Earthquake-related damage to electrical transmission, substation wire support, and distribution structures typically is caused by large displacements of the foundations due to landslides, ground failure, and liquefaction (FEMA, 1990). These situations have resulted in structural failure or damaged structural members without complete loss of structure function.

The fundamental frequency of these structure types typically ranges from 0.5 to 6 Hz. Single pole type structures have fundamental mode frequencies in the 0.5 to 1.5 Hz range. H-frame structures have fundamental mode frequencies in the 1 to 3 Hz ranges, with the lower frequencies in the direction normal to the plane of the structure and the higher frequencies in plane. Four legged lattice structures have fundamental mode frequencies in the range of 2 to 6 Hz. Lattice tangent structures typically have lower frequencies with the higher frequencies being representative of angle and dead end structures. These frequency ranges can be used to determine if earthquake loading should be a design consideration. If it is determined that earthquake loads are significant then a more detailed evaluation of the structure vibration frequencies and mode shapes should be performed. This can be accomplished using available commercial finite element computer programs. The default viscous damping value to be used in such an analysis should be 2 percent. A higher damping value can be used if determined using sound engineering data.

A minimum importance factor (I) of 1.0 should be used to provide the necessary seismic resistance. An I of 1.0 is required to minimize the loss of function after an earthquake event even though these systems are normally redundant.

The R values shown in Table C14A.3.2 reflect the inelastic reserve strength of the structural systems during an earthquake event. The values presented for these types of structures were determined based on a review of published values established for building structures and nonbuilding structures. An analysis of lattice (truss) type transmission towers dictated R values in the range of 3 to 8 (Lyver 1996). The value of 3 for truss systems shown in Table C14A.3.2 represents the lower bound value of R. In general, the remaining R values shown reflect the earthquake performance of these structural systems and engineering judgment. Other values may be appropriate if determined using sound engineering data.

The C_d and W values shown in Table C14A.3.2 for these types of *structures* are presented for information only and to be consistent with parameters presented for other facilities covered by the *Provisions*. The C_d value is a factor used to estimate the peak inelastic deflection (d_{inel}) during a seismic event when the elastic displacements (d_{el}) from a static analysis using seismic loads are known ($d_{inel} = d_{el}C_d$). The W values represent a component force factor to provide increased reliability in strength for a critical component (component force times W). The magnitude of this factor is currently specified (when used) by the industry design standards and recommended practices specified in Sec. 14.3.

Traditionally, wire supported mass and dynamic effects have not been included in the evaluation of structural response (Long 1973). Some studies have suggested that for long spans the seismic contribution of the wires should not be neglected (Li 1991, Li 1994). Reasons for neglecting the supported wires are the order of magnitude difference between the wire system natural frequency and that of the supporting structures and the method of connection between these two systems. The spatial distribution of the structural system (varying wire spans, tower location and geometry, and seismic ground motion) also helps mitigate the effects of dynamic coupling. The satisfactory performance of these structures during earthquakes does not justify the additional loading as a result of the wire dynamics. Engineering judgment should be used to determined the inclusion or the significance of the wire mass.

C14A.3: TELECOMMUNICATION TOWERS: This section was placed in the Appendix to Chapter 14 for the following reasons:

1. To provide a starting point for continued development.
2. To stimulate comment and input for development of this section to the end that it will be incorporated in the *Provisions* in the future.
3. It was determined by TS13 and the Provisions Update Committee that it would be premature to incorporate this section into the *Provisions* for the 2000 edition.
4. Accepted industry standards are in the process of incorporating seismic design methodology reflecting

the *Provisions*.

It is not the intent of the Provisions Update Committee to discourage incorporation of this section into a building code or to minimize the importance of this section. Placing this section in the appendix indicates only that this section requires further development.

The design of telecommunication towers is typically controlled by extreme wind, ice and wind combinations, and restrictive deflection (serviceability) limits (TIA 222; CSA 1994). Earthquake performance of these structures has demonstrated that seismic loads can be resisted based on traditional telecommunication loading (Lum, 1983). As a minimum, this requirement should be to determine the significance of seismic loads in the design of the tower. Seismic lateral loads in combination with long-term ice loads should be considered. Recommendations for combined load effects can be found in ASCE 7.

A general industry survey indicated that the seismic performance of these structures to earthquake loading has been acceptable. Reported earthquake damage has been limited to failure of building mounted towers and shifting of mounted antennas resulting in misalignment of the signal path (FEMA,202, Lum, 1983; NCEER, 1995; Steinhardt, 1981).

The fundamental frequency of these structural types typically ranges from 0.5 to 10 Hz. If it is determined that earthquake loads are significant then a more detailed evaluation of the structure's vibration frequencies and mode shapes should be performed. This can be accomplished using available commercial finite element computer programs. The default viscous damping value to be used with such an analysis should be 2 percent. A higher damping value can be used if determined using sound engineering data.

Recent studies (Galvez, 1995) have suggested that a linear lateral force distribution ($k = 1$) is not an accurate representation for self-supporting telecommunication towers. The lateral force distribution being studied accounts for the mass participation of the lowest three flexural modes of vibration of the tower. Until further studies have been completed and a final recommendation is available it is recommended that a linear distribution be used with the *Provisions* when a refined lateral force distribution is required.

The R values shown in Table C14A.3.2 reflect the inelastic reserve strength of the structural systems during an earthquake event. The values presented for these types of structures were determined based on a review of published values established for building structures and nonbuilding structures. Other values may be appropriate if determined using sound engineering data.

The C_d and Ω values shown in Table C14A.3.2 for these types of structures are presented for information only and to be consistent with parameters presented for other facilities covered by the *Provisions*. The C_d value is a factor used to estimate the peak inelastic deflection (d_{inel}) during a seismic event when the elastic displacements (d_{el}) from a static analysis using seismic loads are known ($d_{inel} = d_{el}C_d$). The Ω values represent a component force factor to be used to provide increased reliability in strength for a critical component (component force times Ω). The magnitude of this factor is currently specified (when used) by the industry design standards and recommended practices specified in Sec.14.3.

Guyed towers taller than 66 m should be evaluated using modal analysis procedures. Modeling of a guyed tower must allow for geometric nonlinearities and potential interactions between the mast and the guy wires (Amiri, 1996). The significant earthquake effect will be due to the dynamic interaction between the mast and the guy wires. The analysis of guyed towers can be accomplished using available commercial finite element computer programs.

Reference AS 3995 has an informative appendix that provides guidance on when earthquake design of guyed and self-supporting telecommunication towers may be appropriate. The following information is obtained from this document.

1. Steel lattice and guyed towers are less sensitive to earthquake loads than most other structure types.
2. Self-supporting lattice towers up to 100 m high and having insignificant mass concentrations less than 25 percent of their total mass need not be designed for earthquakes.
3. Self-supporting lattice towers of insignificant mass and over 100 m high or lesser height with significant mass concentrations may experience base shears and base overturning moments approaching those caused by ultimate wind loads.
4. Self-supporting lattice towers and guyed steel masts that are in earthquake design zones should be designed considering the vertical component of ground motion. For very tall guyed towers, some vertical ground motion differentials between the mast base and guy anchorage points may be an important design consideration depending on local seismicity.

C14A.4 BURIED STRUCTURES:

This section was placed in the Appendix to Chapter 14 for the following reasons:

1. To provide a starting point for continued development.
2. To stimulate comment and input for development of this section to the end that it will be incorporated in the *Provisions* in the future.
3. It was determined by TS13 and the Provisions Update Committee that it would be premature to incorporate this section into the *Provisions* for the 2000 edition.
4. Accepted industry standards are in the process of incorporating seismic design methodology reflecting the *Provisions*.

It is not the intent of the Provisions Update Committee to discourage incorporation of this section into a building code or to minimize the importance of this section. Placing this section in the appendix indicates only that this section requires further development.

Seismic forces on buried structures may include forces due to: soil displacement, seismic lateral earth pressure, buoyant forces related to liquefaction, permanent ground displacements from slope instability, lateral spread movement, or fault movement, dynamic ground displacement from dynamic strains from wave propagation. Identification of appropriate seismic loading conditions is dependent upon subsurface soil conditions and the configuration of the buried structure. Conditions related to permanent ground movement can often be avoided by careful site selection for isolated buried structures such as tanks and vaults. Relocation is often impractical for long buried structures such as tunnels and pipelines.

Wave propagation strains are a significant seismic force condition to buried structures if local site conditions can support the propagation of large amplitude seismic waves (e.g., deep surface soil deposits with low shear wave velocities). Wave propagation strains tend to be most pronounced at the junctions of dissimilar buried structures (e.g., a pipeline connecting with a building) or at the interfaces of different geologic materials (e.g., a pipeline passing from rock to soft soil).

Loading conditions related to liquefaction require detailed subsurface information that can assess the potential for liquefaction and, for long buried structures, the length of structure exposed to liquefaction effects. In addition, the assessment of liquefaction requires specifying an earthquake magnitude that is consistent with the definition of ground shaking. It is recommended that one refer to Chapter 7 C*ommentary* for additional guidance in determining liquefaction potential and seismic magnitude. Providing detailed structural design procedures in this area is beyond the scope of this document.

Loading conditions related to lateral spread movement and slope instability can be defined in terms of lateral soil pressures or prescribed ground displacements. In both cases, sufficient subsurface investigation in the vicinity of the buried structure is necessary to estimate the amount of movement, the direction of movement relative to the buried structure, and the portion of the buried structure exposed to the loading conditions. Definition of lateral spread loading conditions requires special geotechnical expertise and specific procedures in this area are beyond the scope of this document.

Defining the loading conditions for fault movement requires specific location of the fault and an estimate of the earthquake magnitude on the fault that is consistent with the ground shaking hazard in the *Provisions*. Identification of the fault location should be based on past earthquake movements, trenching studies, information from boring logs, or other accepted fault identification techniques. Defining fault movement conditions requires special seismological expertise. Additional guidance can be found in the Chapter 7 *Commentary*.

It may not be practically feasible to design a buried structure to resist the effects of permanent ground deformation. Alternative approaches in such cases may include relocation to avoid the condition, ground improvements to reduce the loads, or implementing special procedures or design features to minimize the impact of damage (e.g., remote controlled or automatic isolation valves, that provide the ability to rapidly bypass damage, post-earthquake procedures to expedite repair). The goal of providing procedures or design features as an alternative to designing for the seismic loadings is to change the hazard and function classification of the buried structure such that it is not classified as *Seismic Use Group* II or III.

It is recommended that one refer to Chaper 7 C*ommentary* for additional guidance in determining liquefaction potential, and determining seismic magnitude.

Buried *structures* are subgrade *structures* such as tanks, tunnels, and pipes. Buried *structures* that are designated as *Seismic Use Group* II or III, or are of such a size or length to warrant special seismic design as determined by the registered design professional shall be identified in the geotechnical report.

Buried *structures* shall be designed to resist minimum seismic lateral forces determined from a substantiated analysis using approved procedures. Flexible couplings shall be provided for buried *structures* requiring special seismic considerations where changes in the support system, configuration, or soil condition occur.