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Ministry of Petroleum

International Institute of Earthquake
Engineering and Seismology

Seismic Design Regulations

Oil industry facilities and structures

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Iranian Seismic Design Code for Petroleum Facilities and Structures

In the name of God

International Institute of Earthquake Engineering and Seismology

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National Iranian Oil Company
Research and Technology Management

Seismic Design Regulations for the Oil Industry Facilities and Structures

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Table of Contents

Chapter 1
Purpose

1.1. Purpose

The purpose of this Regulations is to provide minimum criteria for the seismic design of structures, facilities and equipments of the oil industry, based on the Scope, Sec. 1.2, such that in low and moderate earthquakes, the possibility of disrupting the efficiency and operation of the facilities, and in strong earthquakes, their possibility of being damaged is minimized. By following the provisions of this Regulations, it is expected that the behavior of the structure in weak earthquakes is such that behavior of the structural members remains within the elastic range, and in moderate to strong earthquakes, depending on the importance of structure, the amount of damage, is controlled and limited.

1.2. Scope

The scope of the Regulations includes the seismic design of buildings, non-building structures, industrial equipments and non-structural components in the oil industry, which are mentioned in separate chapters. In this Regulations, three seismic hazard levels are defined in accordance with Chapter 3, and depending on the type and importance of a system, one or two hazard levels may be used for its design. This Regulations is not for the seismic evaluation of existing facilities and structures.

1.3. Design basis

The seismic design criteria of this Regulations are based on the force-based method, with the drift ratios and lateral displacements of the members and components being checked with the allowable values at the end of the design process. This procedure is deemed to be sufficient for the purposes of f this Regulations (Sec. 1.1). The designer is allowed to use other accepted methods, such as performance-based design method, outlined in valid regulations and standards, provided that the provisions of this Regulations are satisfied.

1.4. Structure of the Regulations

This Regulation is arranged in 16 chapters. In chapters 1 to 6, the general criteria of seismic design including: General, Load Combinations, Seismic

Hazard, Analysis Methods, and Soil-Structure Interaction are presented. In chapter 7, general and common seismic provisions and criteria related to all types of non-building structures are presented. In chapter 8, the seismic design criteria of various types of non-structural components includingmechanical and electrical equipments, and architectural components are presented. Chapters 9 and 10 refer to the seismic design of structures in which a seismic isolation system or a damping system is installed. Chapters 11 to 14 and Chapter 16, respectively, deal with the specific provisions and regulations for the seismic design of some specific non-building structures, such as Chimneys, Tanks, Pipelines, Centralized Networks of Pipes and Offshore Structures. Finally, in chapter 15, the method for loading structures due to tsunami is described.

1.5. System of units

In this Regulations, most of the presented relationships are dimensionless, in order to be compatible with any valid unit system. However, in general, the accepted system of units in this Regulations is the SI System and its related units.

Chapter 2
Combinations of Loads

2.1. General

Buildings and other structures shall be designed using the provisions of either strength or allowable stress design. Both design methodologies should use the load combinations of Sec. 2.2. Where elements of a structure are designed by a particular material standard or specification, they shall be designed exclusively by either strength or allowable stress design.
Along with the requirements of this chapter, any load combination requirement for any specific building, non-building or non-structural element in its special chapter should also be considered.

2.1.1. Symbols

A	:	Design acceleration level
A_k	:	Load or load effect arising from extraordinary event, A
B	:	Response factor
D	:	Dead load
$D_{1,2}$	:	Seismic Design Category
E_h	:	Horizontal seismic effect
E_v	:	Vertical seismic effect
E_{mh}	:	Effect of horizontal seismic forces, including overstrength
F	:	Load caused by fluids with well-defined pressures and maximum heights other than those caused by groundwater pressure
H	:	Load due to lateral earth pressure (including lateral earth pressure from fixed or moving surcharge loads), ground water pressure, or pressure of bulk materials.
L	:	Live load
L_r	:	Roof live load
Q_E	:	Effect of horizontal seismic forces
R	:	Rain load
S	:	Snow load
S_{av}	:	Design vertical response spectral acceleration
S_{DS}	:	Design response spectral acceleration parameter at short periods (0.2s) and 5% damping ratio
T	:	Cumulative effect of self-straining forces and effects arising from contraction or expansion resulting from environmental or operational temperature changes, shrinkage, moisture changes, creep in component

materials, movement caused by differential settlement, or combinations thereof

W : Wind load

ρ : Redundancy factor

Ω_0 : Overstrength factor

2.2. Combinations of loads

The load combinations for allowable stress design and for strength design are provided in sections 2.2.1 and 2.2.2 respectively.

2.2.1. Load combinations for allowable stress design

2.2.1.1. Basic Combinations

Loads listed here shall be considered to act in the following combinations; whichever produces the most unfavorable effect on the building, foundation, or structural member shall be considered. Increases in allowable stress shall not be used with the loads or load combinations given in this regulations unless it can be demonstrated that such an increase is justified by structural behavior caused by rate or duration of load.

$$D \quad (2.1)$$

$$D + L \quad (2.2)$$

$$D + (L_r \text{ or } 0.75S \text{ or } R) \quad (2.3)$$

$$D + 0.75L + 0.75(L_r \text{ or } 0.7S \text{ or } R) \quad (2.4)$$

$$D + 0.6W \quad (2.5)$$

$$D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } 0.7S \text{ or } R) \quad (2.6)$$

$$0.6D + 0.6W \quad (2.7)$$

$$D + 0.7E_v + 0.7\,(E_h + E_{mh}) \quad (2.8)$$

$$D + 0.525E_v + 0.525\,(E_h \text{ or } E_{mh}) + 0.75L + 0.1S \quad (2.9)$$

$$0.6D - 0.7E_v + 0.7\ (E_h \text{ or } E_{mh}) \qquad 2.10$$

where:

D: Dead load

L: Live load

L_r: Roof live load

E_h: Horizontal seismic effect

E_{mh}: Effect of horizontal seismic forces, including overstrength

E_v: Vertical seismic effect

S: Snow load

W: Wind load

F: Load caused by fluids with well-defined pressures and maximum heights (see Exception 1)

H: Load due to lateral earth pressure, ground water pressure, or pressure of bulk materials (see Exception 2)

R: Rain load

Exceptions:

1. Where fluid loads, F, are present, they shall be included in combinations 2.8 through 2.10, with the same factor as that used for dead load, D.
2. Where loads, H, are present, they shall be included as follows:

1. Where the effect of H adds to the principal load effect, include H with a load factor of 1.0.
2. Where the effect of H resists the principal load effect, include H with a load factor of 0.6 where the load H is permanent, or a load factor of 0 for all other conditions.
3. If needed, the effects of seismic loading should also be included as shown in 2.2.4.
4. If the requirements of wind load are considered based on the Iranian National Building Regulations, Clause 6, the wind load should be multiplied by a factor of 1.6.

2.2.1.2. Load combinations including self-straining forces and effects

Where the structural effects of T are expected to adversely affect structural safety or performance, T shall be considered in combination with other loads. Where the maximum effect of load, T, is unlikely to occur simultaneously with the maximum effects of other variable loads, it shall be permitted to reduce the magnitude of T considered in combination with these other loads. The fraction of T considered in combination with other loads shall not be less than 0.75

D + T	2.11
D + 0.75(T + L)	2.12

2.2.2. Load combinations for strength design

2.2.2.1. Basic Combinations

Structures, components, and foundations shall be designed such that their design strength equals or exceeds the effects of the factored loads in the following combinations.

1.4D	2.13
1.2D + 1.6L + (0.5Lr or 0.3S or 0.5R)	2.14
1.2D + (1.6Lr or 1.0S or 1.6R) + (L or 0.5W)	2.15
1.2D + W +L + (0.5Lr or 0.3S or 0.5R)	2.16
0.9D + W	2.17
1.2D + E_v + (E_h or E_{mh}) + L + 0.15S	2.18
2.19. 0.9D – E_v + (E_h or E_{mh})	2.19

The load factor on L in combinations 2.15, 2.16 and 2.18 is permitted to equal 0.5 for all occupancies in which Lr is less than or equal to 5.0 kN/m^2, with the exception of garages or areas occupied as places of public assembly.

Where fluid loads, F, are present, they shall be included with the same load factor as dead load, D, in combinations 2.13 through 2.19 except 2.17.

Where the effect of H adds to the principal load effect, include H with a load factor of 1.6. Where the effect of H resists the principal load effect, include H with a load factor of 0.9if the load H is permanent, and a load factor of 0 for all other conditions.

Each relevant strength limit state shall be investigated.

Exception:

If the requirements of wind load are considered based on the Iranian National Building Regulations, Clause 6, the wind load should be multiplied by a factor of 1.6.

2.2.2.2. Load combinations including self-straining forces and effects

Where the structural effects of T are expected to adversely affect structural safety or performance, T shall be considered in combination with other loads. The load factor on T shall be established considering the uncertainty associated with the likely magnitude of the structural forces and effects, the probability that the maximum effect of T will occur simultaneously with other applied loadings, and the potential adverse consequences if the effect of T is greater than assumed. The load factor on T shall not have a value less than 1.0.

$$1.2D + 1.2T + 0.5L \tag{2.20}$$

$$1.2D + 1.0T + 1.6L \tag{2.21}$$

2.2.2.3. Load combinations for non-specified loads

Where approved by the Authority Having Jurisdiction, the registered design professional is permitted to determine the combined load effect for strength design using a method that is consistent with the method on which the load combination requirements in Section 2.2.2.1 are based. Such a method must be probability-based and must be accompanied by documentation regarding the analysis and collection of supporting data that are acceptable to the Authority Having Jurisdiction.

2.2.3. Seismic load effects and combinations

All members of the structure, including those not part of the seismic force-resisting system, shall be designed using the seismic load effects of Section 2.2.3 unless otherwise exempted by this regulation.

2.2.3.1. Horizontal Seismic Load Effect

The horizontal seismic load effect, E_h, shall be determined in accordance with Equation (2.22) as follows:

$$E_h = \rho Q_E \qquad 2.22$$

where:

ρ: Redundancy factor, as defined in Section 4.7
Q_E: Effect of horizontal seismic forces. Such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.

2.2.3.2. Vertical Seismic Load Effect

The vertical seismic load effect, E_v, shall be determined in accordance with Equation (2.23) as follows:

$$E_v = 0.2 S_{DS} D \qquad 2.23$$

where:
S_{DS}: Design response spectral acceleration parameter at short periods (0.2s) and 5% damping ratio
D: Effect of dead load.

Where the option to incorporate the effects of vertical seismic ground motion is required elsewhere in this regulations, the vertical seismic load effect, E_v, shall be determined in accordance with Equation (2.24) as follows:

$$E_v = 0.3 S_{av} D \qquad 2.24$$

where:

S_{av}: Design vertical response spectral acceleration
D: Effect of dead load

2.2.3.3. Combination of Horizontal and Vertical Seismic Load Effects

Where determining demands on the soil–structure interface of foundations, the vertical seismic load effect, E_v, is permitted to be taken as zero in Equations 2.10 and 2.19.

2.2.4. Seismic Load Effects including overstrength

Structural elements supporting discontinuous walls or frames of structures that have horizontal irregularity Type E of Table 4.1 or discontinuity in Lateral Strength vertical irregularity of Table 4.2 shall be designed to resist the seismic load effects, including overstrength. The connections of such discontinuous walls or frames to the supporting members shall be adequate to transmit the forces for which the discontinuous walls or frames were required to be designed.

Foundations and other elements used to provide overturning resistance at the base of cantilever column elements shall be designed to resist the seismic load effects, including overstrength.

$$D + 0.7E_v + 0.7\Omega_0 Q_\mathrm{E} \qquad 2.25$$

$$\mathrm{D} + 0.75(0.7E_v) + 0.75(0.7\ \Omega_0 Q_\mathrm{E}) + 0.75L + 0.1S \qquad 2.26$$

$$0.6D - 0.7E_v + 0.7\Omega_0 Q_\mathrm{E} \qquad 2.27$$

Exceptions:

1. Where fluid loads, F, are present, they shall be included in combinations 2.25, 2.26, and 2.27, with the same factor as that used for dead load, D.
2. Where loads H are present, they shall be included as follows:

1. Where the effect of H adds to the primary variable load effect, include H with a load factor of 1.0.
2. Where the effect of H resists the primary variable load effect, include H with a load factor of 0.6 if the load H is permanent and a load factor of 0 for all other conditions.

Where allowable stress design methodologies are used with the seismic load effect defined in Section 2.2.4, allowable stresses are permitted to be determined using an allowable stress increase factor of 1.2

$$1.2D + E_v + \Omega_0 Q_\mathrm{E} + L + 0.15S \qquad 2.28$$

$$0.9D - E_v + \Omega_0 Q_\mathrm{E} \qquad 2.29$$

2.2.5. Minimum Upward Force for Horizontal Cantilevers for Seismic Design

Horizontal cantilever structural members shall be designed for a supplemental basic load combination consisting of a minimum net upward force of 0.2 times the dead load.

2.3. Load combinations for extraordinary events

2.3.1. Applicability

Where required by the owner or applicable regulations, strength and stability shall be checked to ensure that structures are capable of withstanding the effects of extraordinary (i.e., low probability) events, such as fires, explosions, and vehicular impact without disproportionate collapse.

2.3.2. Load Combinations

2.3.2.1. Capacity

For checking the capacity of a structure or structural element to withstand the effect of an extraordinary event, the following gravity load combination shall be considered:

$$1.2D + A_k + 0.5L + 0.15S \qquad 2.30$$

$$0.9D + A_k + 0.5L + 0.15S \qquad 2.31$$

in which A_k is the load or load effect resulting from the extraordinary event, A.

2.3.2.2. Residual Capacity

For checking the residual load carrying capacity of a structure or structural element following the occurrence of a damaging event, selected load-bearing elements identified by the registered design professional shall be notionally removed, and the capacity of the damaged structure shall be evaluated using the following gravity load combination:

$$1.2D + 0.5L + 0.2\,(L_r \text{ or } 0.7S \text{ or } R) \qquad 2.32$$

$$0.9D + 0.5L + 0.2\,(L_r \text{ or } 0.7S \text{ or } R) \qquad 2.33$$

2.3.3. Stability Requirements

Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of second-order effects is permitted.

Chapter 3
Hazard Analysis

3.1. General

This Regulation specifies one or two hazard levels for the design of oil industry structures and facilities, depending on the type of structure. The considered hazard levels are presented in Section 3.5. The effect of aleatory and epistemic uncertainties (in seismic source models, ground motion models and earthquake recurrence laws) should be comprehensively considered when developing the spectra resulting from each hazard level. The site-specific study is performed in two ways: probabilistic and deterministic, which complement each other in estimating the hazard of strong ground motion.

3.1.1. Symbols

The symbols used in this chapter are listed below along with their definitions:

b-value	:	Regional parameter depending on the seismicity of the site
C_R	:	Risk coefficient
C_V	:	(Vertical) Coefficient for vertical spectrum
F_a	:	Site coefficient in the constant acceleration region of the spectrum
F_v	:	Site coefficient in the constant velocity region of the spectrum
F_{PGA}	:	Site coefficient for peak ground acceleration (PGA)
$f_M(m)$	:	Probability distribution function for earthquake magnitude
$f_R(r)$	:	Probability distribution function for distance
IM	:	Intensity measure of earthquake event
$IM_{R,10\%}$	:	Intensity measure level equivalent to the 0.1-quantile or 10% collapse probability read from the fragility curve adjusted for the target risk
IM_{2475}	:	A ground motion level which corresponds to a 2475-years seismic hazard level
M_W	:	Moment magnitude of earthquake event
m_{max}	:	Maximum moment of earthquake event
m_{min}	:	Minimum moment of earthquake event
PE	:	Annual probability of exceedance
PGA	:	Peak Ground Acceleration
PGV	:	Peak Ground Velocity

R	:	Source to site distance
R_{max}	:	The maximum distance from the site to a point on the fault, corresponding to the definition of the distance in the employed attenuation relationship
R_{min}	:	The minimum distance from the site to a point on the fault, corresponding to the definition of the distance in the employed attenuation relationship
S_a	:	Spectral acceleration (in unit g) with a damping ratio of 5%
S_{D1}	:	5%-damped spectral acceleration parameter (in unit g) corresponding to the design earthquake at the long period (1 seconds)
S_{DS}	:	5%-damped spectral acceleration parameter (in unit g) corresponding to the design earthquake at the short period (0.2 seconds)
S_{M1}	:	5%-damped spectral acceleration parameter (in unit g) corresponding to the rare earthquake at the long period (1.0 seconds) adjusted for site class effects
S_{MS}	:	5%-damped spectral acceleration parameter (in unit g) corresponding to the rare earthquake at the short period (0.2 seconds) adjusted for site class effects
S_1	:	Spectral acceleration parameter (in unit g) corresponding to a rare earthquake at a period of 1 sec on bedrock, resulting from a site-specific study, with a damping ratio of 5% and multiplied by the coefficient of maximum direction response
S_{aMv}	:	Vertical response spectral acceleration at a rare earthquake hazard level
S_s	:	Spectral acceleration parameter (in unit g) corresponding to a rare earthquake at a period of 0.2 sec on bedrock, resulting from a site-specific study, with a damping ratio of 5% and multiplied by the coefficient of maximum direction response
T	:	The return period of the considered hazard level, or, The fundamental vibration period of the structure
t	:	The structure's lifetime
x	:	A given level of intensity measure
ε	:	The difference of the ground motion from the logarithmic mean of the ground motion model given by the number of standard deviation
λ_{annual}	:	The annual rate of earthquake occurrence exceeding a given intensity measure, whose value is equal to 1/T.
$\bar{\lambda}_{annual}$	:	The weighted average of the λ_{annual} results

μ : The number of earthquake occurrences according to the catalog of the region in the desired time interval (between m_{min} and m_{max}) divided by the corresponding time interval

σ_{IM} : Standard deviation of intensity measure levels

3.2. Definitions

Earthquake hazard: The safety threat of structures caused by different levels of earthquake parameters.

Earthquake hazard analysis: The process of determining the influencing factors of different levels of earthquake motions in the construction and performing the necessary calculations in order to estimate the parameters needed in the analysis and design of the structure (such as peak ground acceleration and peak ground velocity and response spectral values).

Earthquake hazard level (motion): The level of earthquake ground motion with a certain probability of exceedance in the desired site in a certain period of time, which is determined based on the characteristics of regional seismicity.

Design spectrum: The spectrum used to design the structure at the desired earthquake hazard level.

Risk-based design spectrum: A spectrum expected to have a 1% probability of collapse during the lifetime of 50 years for a conventional structure (Occupancy and Risk Group II) based on which it is designed.

Ground motion model (GMM, attenuation relationship): A mathematical model compatible with the seismic characteristics of the region, which calculates the expected values of strong ground motion parameters, and also dispersion around it, as a function of fault type, source to site distance, site soil properties, and others.

Uniform hazard spectrum: A spectrum whose values have the same probability of exceedance in a certain period of time.

Probabilistic seismic hazard analysis (PSHA): The process of calculating the probability of exceeding the desired intensity measure (such as peak ground acceleration and peak ground velocity and response spectral values) from a certain value in a certain period of time (the lifetime of the structure), resulting from seismic activity of all seismic sources in the region by considering all uncertainties in the framework of the probability theory.

Deterministic seismic hazard analysis: It is a process during which the intensity measure of the selected scenario, with a specified magnitude and distance resulting from the seismic disaggregation process,) are determined according to the type of the construction site.

Sensitivity analysis: Determining the effectiveness of the hazard analysis results subjected to changes in parameters, inputs and models used.

Seismic disaggregation: Calculation of the percentage of participation of possible scenarios of different seismic sources in the probability of exceeding the intensity measure.

Site-specific study: All studies and investigations carried out in order to analyze the earthquake hazard and to investigate the other related hazards caused by earthquakes in the construction site, which include field studies and statistical calculations.

Uncertainty: Uncertainty resulting from lack of knowledge, defects in models or available data.

Active fault: A fault on which an earthquake occurred in the Holocene period (from about 10 thousand years ago until now) or a fault that had seismic activity in the second half of the Quaternary (approximately 1 million years ago until now) and its slip rate is more than 1 mm per year.

Direction of maximum response (direction of maximum loading): The direction in which the maximum response of a one-degree-of-freedom system occurs under horizontal earthquake excitation. It is obvious that this direction is different in the general case, for each different vibration period.

Epsilon (ε): Is a vibration period-dependent parameter that is used to evaluate the intensity of ground motion in an arbitrary scenario in comparison with a specific ground motion model (reduction ratio) in the same scenario. Epsilon represents the amount of deviation of the ground motion from the logarithmic mean of the model of the ground motion (GMM) and expresses the number of standard deviations compared to the logarithmic mean of the same model.

Bedrock: Site Class I, in the Iranian Standard No. 2800, with an average shear wave velocity in the first thirty meters of the ground depth (V_{s30}) equal to or more than 750 m/s.

3.3. Applicability

A site-specific study is mandatory for all oil industry facilities. The results of this study include documents related to earthquakes in the region and fault slip rates, seismic catalog, selection of ground motion models, seismic disaggregation and selection of appropriate acceleration time-history for the site response analysis and structural time history analysis, soil profile model and its analysis method, sensitivity analysis and the used logic tree and other related matters should be presented in the form of a technical report.

Note 1: Site-specific studies are not required for structures of the Occupancy and Risk Group III (as defined in Table 4.3) located on Site Classes I and II (as defined in Standard No. 2800) and structures of the Occupancy and Risk Group IV, which can be addressed by the Standard No. 2800 design spectrum. If site-specific studies have been completed for the said structures, the results of these studies can also be used.

3.4. Categories of the Site-Specific Studies

In this regulation, depending on the sensitivity of the structures and the seismicity of the region, the level of seismic hazard analysis studies will be different. Also, in order to evaluate the results of these studies and their compliance with the requirements of this chapter and to control how to interact with various uncertainties, it is necessary to evaluate the documents and reports by a group of experts whose expertise has been approved by the employer, within the framework of laws, regulations and notifications of the

Oil Ministry. The level and manner of this evaluation depends on the importance of the desired facilities and also the level of seismicity of the region. In determining the level of study and evaluation for a set of facilities, the most critical Occupancy and Risk Group is the criterion. In general, two types of study levels are defined in this Regulation as follows.

3.4.1. Category A

This level of study is sufficient for all the facilities of the oil industry, except for those mentioned in study Category B.

3.4.2. Category B

For the structures of the Occupancy and Risk Group I (according to Table 4.3) in areas with high and very high seismic activity (based on the hazard zonation map of Standard No. 2800), as well as the structures of this Occupancy and Risk Group in areas with low and medium seismic activity located in the near-fault zone, according to the provisions of Section 3.11, this level of studies is mandatory. In case these facilities are located in an area of active faults, it is also mandatory to perform probabilistic fault displacement hazard analysis (see Section 3.13) according to valid international references.

3.5. Earthquake hazard levels

All the structures and equipments of the oil industry should be designed according to the relevant chapters, for the design spectrum, such as one or two hazard levels from the hazard levels listed in Sections 3.5.1 to 3.5.3, or specific hazard levels defined therein. How to calculate the relevant design spectra is mentioned in Section 3.8. The basis for calculating earthquake return periods at all three levels in this regulation is based on the acceptance of Poisson's distribution of earthquake occurrence.

Note 2: The response spectrum results obtained from most of GMMs should be converted to the maximum load or response direction (on the horizontal plane) using appropriate coefficients. If the GMMs are based on the geometric mean of two horizontal components, the amplification factor for the transformation of the direction at 1-second vibration-period and more is assumed to be equal to 1.3. The direct conversion coefficient for spectral accelerations at vibration-period intervals of 0.2 seconds and less (including PGA) should be equal to one. Linear interpolation should be used for intermediate values.

3.5.1. First hazard level (operational earthquake)

The operational hazard level is the intensity level of ground motions which, if occuring during lifetime of the structure, behavior of the main structural members expectedly remain in the elastic range. The spectrum resulting from this hazard level is called the operational spectrum. This spectrum is prepared for 2% damping. According to the type of structure, the probability of exceedance and return period of the earthquake is different. For essential buildings (Chapter 4), pipelines (Chapter 13) and offshore structures (Chapter 14), the exceedance probability of the operational earthquake for every 50 years is 70%, 50% and 20%, respectively. These values are equivalent to the approximate return periods of 40, 75 and 200 years, respectively. It is not mandatory to use operational earthquakes for other structures.

3.5.2. The second hazard level (design spectrum)

The second hazard level (design spectrum) is used for design of all structures and facilities of the oil industry. In the case of the pipeline, the values related to the probability of exceedance and the design return period are determined considering the Occupancy and Risk Group according to Chapter 13. This spectrum is generally prepared for a damping ratio of 5%. If the type of building requires a different damping ratio, it can be the basis for preparation of the spectrum. In order to obtain the acceleration spectrum related to other damping values, the damping spectrum of 5% is corrected by applying the D-scale coefficient (see Eq. 14.1).

In this regulation, parameters corresponding to the second hazard level (design earthquake), is obtained by conducting a site-specific analysis, estimating *the seismic parameters* of the third hazard level (*rare* earthquake) and multiplying these values by 2/3, with considering the coefficient of the maximum response (see Section 3.8). For offshore structures, the design return period should be taken into account according to the provisions of Chapter 14.

In cases where it is not necessary to conduct a site-specific study in accordance to Section 3.3, the design spectrum of Standard No. 2800 can be used for the design earthquake.

Note 3: The lower limit of the design spectrum is equal to 80% of the design spectrum of the latest edition of Standard No. 2800, and its upper limit is equal to two-thirds of the spectrum resulting from the deterministic hazard analysis (Section 3.7.3). The response spectrum in Site Classes described in Note 8 should not be less than 80% of the response spectrum of that structure in Standard No. 2800 for the Site Class IV at any spectral periods.

3.5.3. The third hazard level (Rare earthquake)

This level of hazard corresponds to the occurrence of a very large ground shaking intensity (a rare or maximum considered earthquake) and represents the most severe level of ground motion used in this regulation with a very low level of probability of occurrence during the structure's lifetime. The return period of this ground motion intensity is about 2475 years.

Using the risk-based pseudo-acceleration spectrum is also allowed to define this hazard level. This spectrum is calculated on the basis of a collapse probability of 1% during a 50 years lifetime for an ordinary building (equivalent to the annual collapse probability of 2×10^{-4}) (Section 3.9). The latter probability is called the target risk or the base risk. The target risk criteria for designing offshore platforms are mentioned in Chapter 14 (Table 2.14).

In any case, the upper limit of the third hazard level is the spectrum resulting from the deterministic hazard analysis (Paragraph 3.7.3).

The third hazard level is used for the design or control of the seismic isolation system (Chapter 9), structures equipped with dampers (Chapter 10), offshore structures (Chapter 14) and structures of the Occupancy and Risk Group I (according to Table 3.4).

3.6. Selection of the ground motion model

Selecting the acceptable predictive relationship to estimate the characteristics of ground shaking intensity compatible with the tectonic earthquake conditions of the site is of particular importance due to having large uncertainties. The relationships should be prepared, selected or modified based on the data of the study area, the earthquake magnitude (M_w) and the fault style. The GMMs selected or developed should be consistent with the minimum and maximum limits of the magnitude and the various distances defined from seismic sources.

The selected GMMs should satisfactorily cover the expected range of acceleration data associated with the Iranian plateau.

The cases of the GMM for different regions of Iran are as follows:

- Shallow crustal zones
- Interface subduction zones
- In-slab subduction zones

In case of lack of information on local earthquakes, it is possible to use simulation-based GMMs for estimating the characteristics of ground motion, in which the available data is increased by using valid simulation methods.

In the seismic hazard analysis of the Categories A and B (see Section 3.4) to estimate the acceleration spectrum, it is necessary to use in the logic tree, respectively, at least three and five predictive equations with the above characteristics, which are ranked according to valid methods. At least two relationships should be selected from the global or regional GMMs and one relationship from the local GMMs specific to the Iranian Plateau, whose validity in estimating the characteristics of the ground motion in the Iranian Plateau has been proven. It is necessary to use "suitable statistical methods" along with "reference to valid technical literature" (for Category B hazard analysis, Section 3.4.2), and "reference to valid technical literature" for Category A hazard analysis (Section 3.4.1) to select the acceptable ground motion models.

It is not allowed to use models with the following conditions:

- Models that are not compatible with the type of the seismic zone under study.

-Models that have not been published in valid international scientific media.

-Models that are outdated or replaced by new versions.

-Models with the fitting method used in their development not including intra-event and inter-event errors.

- Models that do not consider the nonlinear soil effect, for the case of the sites located on the ground category IV.

3.7. Hazard analysis procedure

3.7.1. General requirements

In the site-specific study, it is necessary to examine the tectonic earthquake characteristics, geology and slip rate of faults, seismicity of the area, expected earthquake occurrence rate, maximum magnitude of active faults and the site class. The minimum radius of this study's area is 150 km for the Iranian plateau and 300 km in the subduction zone of the Makran region.

The site-specific study and estimation of the seismic parameters can be carried out using one of the following three procedures:
Procedure 1) Use of GMMs to estimate the ground motion at the bedrock and applying the site response coefficients (see Table 3.1)
Procedure 2) Use of acceptable ground motion models on the ground surface in a direct manner. The procedure is not allowed for the Site Class IV.
Procedure 3) Use of GMMs at the bedrock and site response analysis (to convert input motion from the bedrock to the ground surface)

Note 4: In hazard analysis of Category B (Section 3.4.2), if site response analysis (Procedure 3 above) is employed, the uncertainties of the soil layers' characteristics shall be quantified with verified procedures available in the technical literature.

In the study, it should be taken care of the characteristics of the near source events (according to the definitions and criteria of Section 3.11), the nonlinear behavior of the soil and the liquefaction possibility in prone areas. The pseudo-acceleration spectrum should be prepared according to Section 3.8. In addition, where necessary, the uniform hazard spectrum of pseudo-acceleration, velocity or displacement for the required hazard levels or the values of the maximum ground acceleration (PGA), velocity (PGV), and displacement (PGD) parameters should be provided.

It is necessary to compile all the data of historical and instrumental earthquakes that occurred in the study area in the form of a catalog. This catalog should be modified by removing foreshocks and aftershocks and by taking care of classifying time intervals with different levels of completeness. It is necessary to determine the levels of completeness of the catalog and its time intervals, using appropriate statistical methods.

It is recommended that in the removal of aftershocks and foreshocks, the uncertainty caused by the use of different removal methods is considered in the framework of the logic tree. This is mandatory in the Category B hazard analysis (Section 3.4.2).

It is necessary to collect the following information for all earthquakes as much as possible:

- Event time
- Epicenter with acceptable accuracy
- Estimated magnitude (for historical data) and recorded one (for instrumental data). These values should be converted into moment magnitude values (M_w) using the appropriate practical relationships corresponding to the Iranian plateau.
- Earthquake depth (for instrumental data)
- Fault style
- Estimated uncertainty in each case

Site-specific procedure is performed by probabilistic and deterministic methods according to Sections 3.7.2 and 3.7.3.

3.7.2. Probabilistic hazard analysis

The steps of the procedure based on the most common algorithm are as follows:

1- Delineate the seismic sources that affect the strong ground motion at the site. Both linear (if the fault trace is clear) and areal (if the rupture is scattered) seismic sources are allowed as long as the criteria of Section 3.7.5 are met. It is necessary to use a non-uniform probabilistic distribution for the source-to-site distance.

2- Determining the probability density function for the moment magnitude, depending on the case, relationships such as double truncated Gutenberg-Richter, the characteristic model or combined functions can be used.
3- Calculating the intensity measure level, x, (S_a, PGA, PGV, etc.) according to Equation 3.1:

$$\lambda_{annual} = \mu \int_{m_{\min}}^{m_{\max}} \int_{R_{\min}}^{R_{\max}} [1 - \Phi(\frac{\ln x - \overline{\ln IM_{m,r}}}{\sigma_{\ln IM}})] f_M(m) f_R(r) dr dm \qquad 3.1$$

m_{max}: The maximum earthquake magnitude obtained by a reliable method for each source.
m_{min}: The minimum earthquake magnitude. This parameter is generally selected between 4 and 5.
λ_{annual}: Annual occurrence rate of the selected intensity measure, equal to $\frac{1}{T}$.
T: Return period of the hazard level of interest,
μ: Annual occurrence rate according to the region's catalog (between m_{min} and m_{max})
$[1 - \Phi(\frac{\ln x - \overline{\ln IM_{m,r}}}{\sigma_{IM}})]$: Probability of intensity measure exceeding an x value assuming the standard Normal distribution.

$f_R(r)$: Probability density function for source-to-site distance
$\overline{\ln IM_{m,r}}$: The logarithmic average value of the intensity measure at the site of interest, which is obtained from the ground motion model (attenuation relationship).
σ_{IM}: The standard deviation of the logarithmic intensity measure levels associated with the attenuation relationship.
$R_{\max}$: The maximum distance from the site to a point on the faults, corresponding to the definition of distance in the employed attenuation relationship.
$R_{\min}$: The minimum distance from the site to a point on the faults, corresponding to the definition of distance in the employed attenuation relationship.

Note 5: In Eq. 3.1, a unique value of m_{max} is defined for each seismic source.

This step is repeated for different intensity measure levels, x, from low to high values, to obtain the desired IM hazard curve.

Note 6: Due to the epistemic uncertainty present in description and determination of seismic sources, estimation of their seismic parameters, and ground motion models, it is necessary to handle different assumptions in the framework of the logic tree based on valid methods and considering regional conditions. The final result ($\bar{\lambda}_{annual}$) should be presented using the weighted average of the logic tree branches and also at confidence levels associated with the 15% and 85% percentiles.

Computing the exceedance probability for the final result of the logic tree, this is obtained by assuming the Poissonian temporal model from Eq. 3.2.

$$PE = 1 - e^{-t\bar{\lambda}_{annual}} \qquad 3.2$$

Where t is the service duration or structure's lifetime.

5- Calculation of the uniform hazard pseudo-acceleration spectrum (if necessary, velocity and transient displacement peak values) at the ground surface (Procedure 2 in Sec. 3.7.1) or the bedrock at the site (Procedures 1 and 3 in Sec. 3.7.1) for the desired return period using the compatible ground motion prediction equations.

6- Calculation of the final spectrum shape in the form of a design spectrum according to the criteria of Section 3.8 (for the second and third hazard levels).

In the above steps, performing the sensitivity analysis is also recommended. In this case, it is necessary to determine the range of model and parameter variations based on valid international and local methods.

3.7.3. Deterministic hazard analysis

The deterministic spectral acceleration at each response period is estimated from the 84th percentile of the maximum considered earthquake (using the attenuation relationships employed in the probabilistic analysis) for the maximum response direction. The values presented in Note 2 can be used to determine the conversion coefficients to the maximum response. The magnitude and distance of the desired deterministic earthquake is specified from the "disaggregation of the hazard analysis of the site" at the 2475-years return period. All seismic scenarios on the known faults of the region with at least 10% relative participation in the seismic hazard analysis are included in the calculation of the deterministic results, and the maximum quantities of the ground motion values among them should be chosen.

It is necessary to select at least 3 and 5 equations for ground motion models compatible with the site, respectively, to estimate the acceleration spectrum in the hazard analysis of the Categories A and B (introduced in Sec. 3.4). Results will be presented for the weighted average of these relationships.

Regarding the site effects, in addition to directly using the appropriate attenuation relationships to estimate the response on the ground surface, it is also possible to use site response analysis or the coefficients in Table 3.1 to convert the ground motion at the bedrock to the ground surface.

In any case, if the peak value of the spectral response resulting from the deterministic earthquake is less than $1.5F_a$ (in g unit), it is necessary to multiply all spectral ordinates by the same coefficient in such a way that the response spectrum peak is equal to $1.5F_a$. For the Site Classes 1 to 3, the value of F_a should be determined from Table 3.1 assuming the value of S_s = 1.5. For the Site Class 4, F_a should be equal to unity.

If the spectral response peak value resulting from the probabilistic hazard analysis corresponding to the return period of 2475 years is less than $1.2F_a$ (in g unit), there is no need to calculate the ground motion resulting from the deterministic earthquake.

3.7.4. Disaggregation of seismic hazard

Disaggregation of seismic hazard in terms of magnitude and distance (and epsilon if required) is an essential part of any site-specific hazard analysis. It is necessary to use the linear source model to perform this analysis if there are both area and linear seismicity models. It is necessary to provide the mean (and if needed, median and mode) values of magnitude, distance and epsilon resulting from the disaggregation procedure.

3.7.5. Definition of Seismicity Source models

The seismicity of the study area is defined by three types of areal, linear and background sources.

In this definition, it is necessary to take into account the minimum and maximum depth of the seismogenic layer and the fault slope, according to the regional regime of the seismic source. In the hazard analysis of Category B (Section 3.4.2), it is necessary to consider the epistemic uncertainty in the expression of the geometrical configuration and seismicity parameters of the seismic sources.

It is not allowed to use a uniform probabilistic distribution to calculate the effects of the distance of different fault segments to the site in the hazard integral.

It is not permissible to use circular areas with heterogeneous seismic characteristics to calculate the seismic parameters and attribute it to its inside sources.

It is necessary to consider the completeness periods of the events catalog when determining its seismicity parameters. Also, the temporal variation of the completeness level should be taken into account in determining the seismicity parameters.

The seismicity model including the magnitude frequency distribution of areal source is based on a double truncated Gutenberg-Richter law.

For a linear source, double truncated Gutenberg-Richter and Characteristic earthquake models with appropriate weight in the logic tree can be used.

Due to the fact that the data attributed to a fault may not be enough to calculate the slope of the Gutenberg-Richter line (or the b-value), the value of this coefficient associated with the homogeneous areal seismic zone that includes this fault can be used.

For the Makran seismotectonic zone, according to three types of shallow crustal earthquakes, interface zones and in-slab subduction zones, separate seismic sources should be defined.

3.8. Acceleration Design Spectrum

3.8.1. Uniform Hazard Design Spectrum

The design spectrum is calculated by conducting a PSHA for ground surface or by using a site response analysis (see the Procedures 2 or 3 of Section 3.7.1). It is represented by Eq. (3.3):

$$S_a = \frac{2}{3} S_{aM} \qquad 3.3$$

in which S_{aM} is the 5%-damped spectral acceleration parameter (in g unit) corresponding to a rare earthquake (i.e. the third hazard level, Section 3.9), which is multiplied by the conversion factor due to the maximum direction response (see Note 2). It is not necessary for this value to exceed the deterministic spectrum at any period. Moreover, the ordinates of the design spectrum should not be less than 80% of their counterpart in the design spectrum of Standard No. 2800 at any period. If site response analysis is used, the resulting spectrum should be properly smoothed for design purposes.

If required, the spectral acceleration of the site-specific design spectrum at the short period (S_{DS}) should be considered equal to 90% of the maximum spectral acceleration of the site-specific spectrum at the response periods between 0.2 to 0.5 seconds.

Also, the spectral acceleration of the site-specific design spectrum at the long period (S_{D1}) is equal to the largest value between the spectral accelerations of the site-specific spectrum at the period of 1 sec and $0.9T.S_a$ in the range of $1 \le T \le 5$ sec for the site with $\bar{V}_{s30} \le 450$ m/s and the range $1 \le T \le 2$ sec for the site with $\bar{V}_{s30} > 450$ m/s.

To determine the seismic base shear coefficient by the equivalent lateral force method of chapter 4, the S_{D1} value should be set from the equivalent S_a value at the period T.

In addition, S_{MS} and S_{M1} are 5%-damped spectral acceleration parameters (in unit g) at the short period and 1-second period, respectively, adjusted for the ground surface and the maximum directional response. They correspond to a rare earthquake (i.e. the third hazard level), which are 1.5 times the above design ordinates.

If the velocity spectrum is required, its ordinates can be obtained from the following relationship:

$$S_v = \frac{S_a}{(2\pi / T)} \qquad 3.4$$

3.8.2. Standard Design spectrum

In accordance with Section 3.7.1.1, the maximum direction design spectrum is developed as indicated in Fig. 3.2. In this figure, the spectral acceleration, S_a, should be taken as given in Eqs. 3.5 to 3.9.

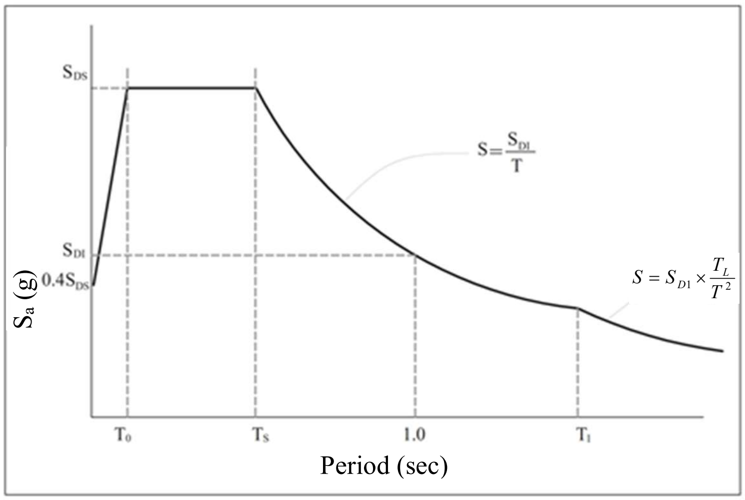


Figure 3.1. Standard Design Spectrum where the Site Class coefficients (Table 3.1) are used.

$$S_a = 0.4S_{DS} \qquad T = 0 \qquad 3.5$$

$$S_a = S_{DS}(0.4 + 0.6\frac{T}{T_o}) \qquad 0 \le T \le T_o \qquad 3.6$$

$$S_a = S_{DS} \qquad T_o \le T \le T_S \qquad 3.7$$

$$S_a = S_{D1} \times \frac{1}{T} \qquad T_L \ge T > T_S \qquad 3.8$$

$$S_a = S_{D1} \times \frac{T_L}{T^2} \qquad T > T_L \qquad 3.9$$

S_{DS} and S_{D1} are design, 5% -damped, spectral acceleration parameters (in unit g) at the short period (0.2 seconds) and a period of 1.0 seconds, respectively, adjusted for the ground surface and the maximum directional response. They should be determined by multiplying the third hazard level, which is resulted from PSHA, by a factor of 2/3.

For offshore structures, the design return period, the damping ratio, and other requirements are extracted from Chapter 14.

T_L represents the period of the second corner of the spectrum, whose value is independent of the site conditions, S_{DS} or S_{D1} values and is equal to 6.0 seconds. On the coasts of the subduction zone of Makran, the value of this period is equal to 16 seconds. For the Persian Gulf region, the value of the period of the second corner is 0.4 seconds.

If the velocity spectrum is required, its ordinates can be obtained from Eq. 3.4.

Values of the spectral acceleration at the required periods are first estimated on the bedrock, and they are then converted to values similar to those on the soil of the site with Eqs. 3.10 and 3.11.

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.9 F_a S_s) \qquad 3.10$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} F_v S_1 \qquad 3.11$$

where:

S_1 : 5%-damped spectral acceleration parameter (in unit g) corresponding to the rare earthquake at a period of 1 sec on the bedrock, which is defined as the medium having a shear wave velocity greater than 750 m/s, resulting from a site-specific study with a damping ratio of 5% and multiplied by the coefficient of maximum direction response.

S_{MS} and S_{M1} are the 5%-damped spectral acceleration parameters (in unit g) corresponding to the rare earthquake (third hazard level) at the short period (0.2 seconds) and a period of 1.0 seconds, respectively, adjusted for the ground surface and the maximum directional response. It is not necessary that their values exceed the deterministic spectrum at any period. Also, the ordinates of the design spectrum should not be less than 80% of their counterparts in the spectrum of Standard No. 2800 at any period. If required, the maximum considered earthquake spectrum should be 1.5 times the design spectrum of this regulation.

The values of the site coefficients in the constant acceleration domain of the spectrum, F_a, and the constant velocity domain, F_v, in accordance with the site class, are obtained from Table 3.1. In the case of estimating the PGA on the bedrock, the ground surface acceleration can be obtained using the following equation:

$$PGA_{surface} = F_{PGA}.PGA$$

The PGA site coefficients are presented in Table 3.1.

Besides, T_0 and T_S are site-dependent periods defined by Eqs. 3.12 and 3.13:

$$T_o = 0.2\frac{S_{D1}}{S_{DS}} \quad 3.12$$

$$T_S = \frac{S_{D1}}{S_{DS}} \quad 3.13$$

Note 7: Instead of using the site coefficients provided in Table 3.1, a site-specific study is recommended in accordance with the Procedures 2 or 3 in Section 3.7.1 for the Site Class II and the Procedure 3 for the Site Class IV, for structures on the Site Class IV with S_s greater than 1.0, and for structures on the Site Classes III and IV with S_s greater than 0.2.

Note 8: Where any of the following conditions exists, a site response analysis in accordance with the Procedure 3 of Section 3.7.1 shall be performed.

1- Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. For structures with fundamental periods equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site coefficient is permitted to be determined in accordance with the Procedure 2 of Section 3.7.1 or the corresponding values of F_a and F_v determined from Table 3.1.
2- Peats and/or highly organic clays with a thickness of more than 3 m
3- Very high plasticity clays with PI > 75more than 7.5 m thick
4- Very thick soft or medium stiff clays, i.e. more than 40 m thick with s_u < 50 kPa, where s_u is the undrained shear strength.

Table 3.1. Site coefficients to convert the spectral response from bedrock to ground surface for the third hazard level

F_a as a function of Site Class and S_s value						
Site Class**	**Spectral acceleration parameter at a period of 0.2 sec on bedrock, S_s**					
	$S_s \leq 0.25$	$S_s \leq 0.5$	$S_s \leq 0.75$	$S_s \leq 1.0$	$S_s \leq 1.25$	$S_s \leq 1.5$
I	1.0	1.0	1.0	1.0	1.0	1.0
II	1.3	1.3	1.2	1.1*	1.0*	1.0*
III	1.6	1.4	1.2	1.1	1.0	1.0
*IV	2.4	1.7	1.3	1.3*	1.2*	1.2*
F_v as a function of Site Class and S_1 value						
Site Class	**Spectral acceleration parameter at a period of 1.0 sec on bedrock, S_1**					
	$S_1 \leq 0.10$	$S_1 \leq 0.20$	$S_1 \leq 0.30$	$S_1 \leq 0.40$	$S_1 = 0.50$	$S_1 \geq 0.60$
I	1.0	1.0	1.0	1.0	1.0	1.0
II	1.5	1.5	1.5	1.5	1.5	1.4
III	2.4	2.5*	2.5*	2.5*	2.5*	2.5*
IV	4.2	4.0*	4.0*	4.0*	4.0*	4.0*
F_{PGA} as a function of Site Class and PGA value						
Site Class	PGA≤0.10	PGA=0.20	PGA=0.30	PGA=0.40	PGA=0.50	PGA≥0.60
I	1.0	1.0	1.0	1.0	1.0	1.0
II	1.3	1.2	1.2	1.1*	1.0*	1.0*
III	1.6	1.4	1.3	1.1	1.0	1.0
IV	2.4	1.9	1.6	1.4	1.2	1.1

* See Note 7.

** Site Class is defined in accordance with Standard No. 2800. Use straight-line interpolation for intermediate values of S_1, S_s, and PGA.

3.9. Risk-targeted Spectrum

The use of the risk-targeted spectrum to define the third hazard level (and consequently in the preparation of the design spectrum in Section 3.8) is allowed, and it can also be required at the discretion of the employer. In preparing this spectrum, the collapse risk at the site and the base Importance Group (Occupancy and Risk Group II) should be equal to the target risk or the base risk. The target risk in this Regulations is a probability of 1% for exceeding the collapse limit-state during a 50-years period.

Value of the ground motion parameter at the third hazard level, which is adjusted based on the target risk, is determined by iterating the risk integral, i.e.convolution of the hazard curve with the assumed lognormal fragility curve with a standard deviation of 0.6, to achieve the target risk. Finally, the risk-targeted response is represented by $IM_{R,10\%}$, i.e. the ground motion parameter equivalent to the 0.1-quantile or 10% collapse probability read from the fragility curve adjusted for the target risk.
Risk-targeted calculations should be performed at all periods necessary to provide a uniform risk spectrum (see Section 3.8.1) or two periods of 0.2 and 1 seconds to provide the standard spectrum (see Section 3.8.1). For convenience, the value of risk-targeted coefficients can be calculated only at the short-period (0.2 sec), C_{RS}, and 1-second period, C_{R1}. Then, interpolation can be used for the intermediate values.. For periods greater than 1 sec, C_{R1} can be used and for periods less than 0.2 seconds, use of C_{RS} is permissible.
The risk-targeted coefficient, known as the risk coefficient, C_R is considered equal to the ratio of the risk-targeted spectral acceleration to spectral acceleration with a return period of 2475 years:

$$C_R = \frac{IM_{R,10\%}}{IM_{2475}} \tag{3.14}$$

3.10. General requirements for site response analysis

3.10.1. Acceleration time-history selection

If there is a need for a site-specific investigation of the site effects according to the Procedure 3 of Section 3.7.1, at least 7 horizontal acceleration time-histories shall be selected. These acceleration time-histories must be related to recorded or simulated earthquakes that are compatible with the controlling earthquake of the desired level (see Section 3.5) in terms of magnitude and distance from the fault. Then, all the selected acceleration time-histories should be scaled in such a way that their response spectrum on average matches the bedrock-response spectrum of the desired hazard level within a period interval between the maximum and minimum significant natural periods of the structure. In the Makran subduction zone, for each type of shallow crustal interface and in-slab earthquakes, at least 7 related acceleration time-histories should be used to analyze the response of the structure.

3.10.2. Calculation of the time history of the ground motion

For each of the selected acceleration time histories, the time history of the input earthquake must be applied to the soil profile as the outcrop motion. Using state-of-the-practice methods capable of modeling the soil dynamic behavior subject to the strong ground shaking in a non-linear or equivalent linear manner, the dynamic site response analysis is performed to obtain the time history of earthquake vibration on the ground surface.

3.10.3. Estimation of Ground Surface Response Spectrum

The steps for estimating the response spectrum on the ground surface are as follows:

A) The response spectrum of all the calculated acceleration time-histories of the ground surface with a 5% damping is obtained. Then their average spectrum is established.

B) For individual acceleration time histories, ratio of the response spectrum of the ground surface to the response spectrum of the input motion at 5% damping is calculated. Their average is then established as the transfer function of the soil deposit. Result of the product of the average spectrum by the spectrum of the desired hazard level at the seismic bedrock, which is computed in period-by-period multiplications, is obtained.

C) The smoothed response spectrum of the desired hazard level on the ground surface should be drawn based on the average spectrum of step A in such a way that it is not less than the response spectrum obtained from step B. It should cover the sensitivity of the site response to numerical model uncertainties, including the uncertainty in material parameters, geometrical configuration and input motion.

3.10.4. Site Numerical Model Development

Based on the problem conditions, the geometry of the site numerical model should be considered as one-dimensional, two-dimensional or three-dimensional. In order to introduce the material characteristics of soil deposits, an acceptable behavior model should be selected for the small strains (linear behavior range) and large strains (nonlinear or equivalent linear behavior model) of soils. For this purpose, shear wave velocity in the range of small strains should be obtained by geophysical measurement at the project site or measurement on similar soils around the site.

The characteristics of shear modulus and damping ratio at large strains should either be extracted from the related laboratory tests of soil dynamics or be adapted from technical literature on similar soils. It should be noted that the uncertainties in the soil characteristics should also be properly considered.

When it is practically not possible to model all layers of the soil deposits extended to the bedrock due to the presence of very deep soil profiles, the material of this layer (in the numerical model) can be considered as the Site Class II. In this case, the response spectrum and acceleration time-history related to the desired hazard level at the seismic bedrock should be converted from the bedrock input to the new site type by applying site coefficients, such as the values mentioned in Table 3.1.

3.11. Special requirements for near field effects

Sites that meet the following conditions are considered as near-source sites:

A) A site with a distance of less than 15 km from the horizontal projection of known active faults with the ability to cause a rupture with a magnitude equal to or greater than 7.

B) A site with a distance of less than 10 km from the horizontal projection of known active faults with the ability to cause a rupture with a magnitude equal to or greater than 6.

In the above criteria, the horizontal projection of parts of the fault located at depths of 10 km or more should not be included (see Figure 3.2).

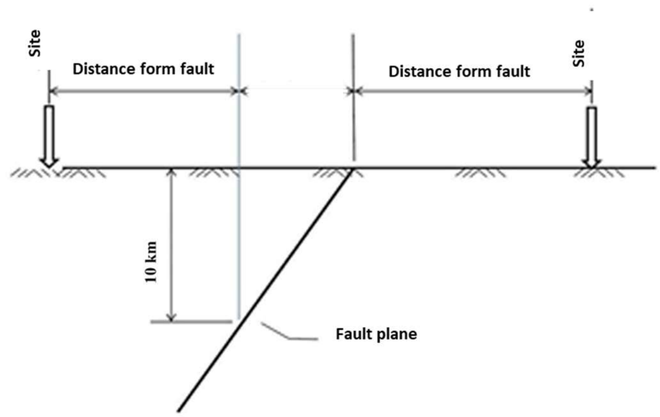


Figure 3.2. Illustrations of geometrical distance definitions to investigate near-field effects

In near-source sites, it is necessary to consider the effects of a pulse with a long period in seismic hazard analysis. Moreover, if necessary, the permanent ground displacement resulting from fault rupture should also be considered in the selected acceleration time history.

If there is a fault near the site, this seismic source:

A) Should be modeled as a finite source with three-dimensional geometrical configuration (length, width and fault slope) at a proper depth.

B) Should be included in occurrence return period calculations with its slip rate, i.e. its earthquake magnitude probability density should be estimated.

Note 9: In the hazard analysis of a Category A site (see Section 3.4), if there is no available data, a sensitivity analysis can be performed with the slip rate of the faults located in the similar seismotectonic enviroments.

3.11.1. Directivity effects in probabilistic seismic hazard analysis

In order to take into account the effect of directivity-based long period pulse in probabilistic hazard analysis, the following approaches or a combination of them can be applied, based on valid technical documents:

A. Statistical approach:

In this method, the hazard analysis is first performed, followed by the disaggregation for the peak ground velocity, or if not available, for the 1-sec spectral acceleration. Number of the pulse-like acceleration time-histories is specified using Eq. (3.15) or similar available relationships in the technical literature. Then, the average response spectrum of the pulse-type and non-pulse-type time-histories is multiplied as an increasing factor by the site-specific spectrum without near-field effects.

The selected acceleration time-histories should be compatible with the scenario resulting from the disaggregation procedure. The minimum number of acceleration time-histories to perform statistical analysis is 11. In the following equation, P is the ratio of the number of pulse records to the total number of records.

$$P = \frac{1}{1 + exp[-3.87 + 1.04 \times R^{0.5} + 15.99 \times (\in +3)^{-2}]} \qquad 3.15$$

where R,distance from the fault in km, and epsilon are the outputs of seismic disaggregation in the return period of interest.

B. Employment of PSHA framework with the modified GMMs:

The synthetic (simulation-based) results can be used as GMM for a source near the site. In developing the simulated scenario, the event uncertainties associated with the location of hypocenter, focal depth, directivity effect, rupture propagation velocity and pattern on the fault, and the mechanical properties of the wave propagation medium should be considered.

Besides, GMMs with the ability to account for directivity at near-fault sites may be utilized.

3.12. Vertical ground motion spectrum

The vertical component design spectrum can be obtained by either site-specific studies or modifying the horizontal spectrum. If required, e.g. for hazard analysis of Category B in Section 3.4.2 or different types of soil from the quadruplet types of ground introduced in Standard No. 2800, the vertical spectrum should be estimated with a site-specific study and valid methods

available in the technical literature. The proposed vertical spectrum should include the local site effects and be properly smoothed.
In the absence of site-specific studies to determine this spectrum, the design vertical spectrum can be considered equal to 2/3 of the value of S_{aMv} as follows:

For vertical periods less than 0.05 sec:

$$S_{aMv} = 10C_v S_{MS} T_v + 0.3C_v S_{MS} \tag{3.16}$$

For vertical periods equal to or greater than 0.05 sec and less than 0.15 sec,

$$S_{aMv} = 0.8C_v S_{MS} \tag{3.17}$$

For vertical periods equal to or greater than 0.15 sec and less than 2.0 sec,

$$S_{aMv} = 0.8C_v S_{MS} (\frac{0.15}{T_v})^{0.75} \tag{3.18}$$

For vertical periods equal to or greater than 2.0 sec, the vertical spectrum ordinate is half of the horizontal spectral ordinate.
T_v is the period of vertical oscillation, and value of C_v is obtained from Table 3.2 according to the Site Class. The vertical spectrum ordinate at any period should not be less than half of the horizontal spectrum. If required, the rare earthquake vertical spectrum can be taken as 1.5 times the design vertical spectrum.

Table 3.2. Values of the Site Coefficient, C_v, for the Vertical Spectrum

C_v as a function of the Site Class and S_s					
Site Class	Spectral acceleration parameter at a period of 0.2 sec on the bedrock, S_s				
	$S_S \le 0.2$	$S_S = 0.3$	$S_S = 0.6$	$S_S = 1.0$	$S_S \ge 2.0$
I	0.7	0.8	0.95	1.0	1.10
II	0.7	0.8	1.0	1.10	1.30
III	0.7	0.85	1.05	1.20	1.40
IV	0.7	0.9	1.1	1.30	1.50

In case of using practical relationships to convert the horizontal to the vertical spectrum, the average horizontal spectral ordinates (before converting to the maximum direction response) should be used in accordance with the existing practical models.

In any case, the proposed vertical spectrum should not be less than 80% of the spectrum obtained from the relations of this Regulation.

If the vertical component of the acceleration time history is required, the vertical components of each of the acceleration time histories must also be modified in such a way that their average spectrum in the required period interval, defined in the Chapters 4, 9 & 10, is not less than the rare earthquake vertical spectrum of this Regulations.

3.13. Estimating ground displacement at the fault rupture site

Method 1: In order to estimate the permanent ground displacement caused by surface faulting, detailed site-specific studies by seismotectonic experts are required in order to estimate the dimensional characteristics of the fault and its seismic potential (magnitude-frequency model) and to diagnose the faulting mechanism in the fault under study. The total probability theorem is then used to estimate the amount of permanent surface displacement of the ground at the fault location in different return periods by combining valid practical magnitude-displacement relationships with the magnitude-frequency relationship of the desired fault. If there are deep soil deposits, the analysis of fault rupture propagation in the soil deposit by the probabilistic methods presented in Chapter 5 should also be considered.

Method 2: As a simplified method, and assuming that seismotectonic experts have not ruled out the possibility of surface faulting, the peak permanent surface displacement at the fault location can be calculated using empirical scaling relationships for the average moment magnitude resulting from the disaggregation procedure (see Section 3.7.4) in the desired return period. The value of this desired displacement, Δ_f, for each of the strike-slip, normal, reverse, or thrust mechanisms, as well as the faults with little information and those undetectable, is presented as follows:

For a strike-slip fault:

3. 19 $$\text{Log}\,(\Delta_{fs})=-4.032+0.558M_w$$

For a normal fault:

$$\mathrm{Log}\,(\Delta_{fn})=-4.967+0.693M_w \qquad 3.20$$

For a reverse or thrust fault:

$$\mathrm{Log}\,(\Delta_{fr})=-3.156+0.451M_w \qquad 3.21$$

, and for the fault with little information, the average values corresponding to the reverse and strike-slip faults should be used:

$$\Delta_{fb}=(\Delta_{fs}+\Delta_{fr})/2 \qquad 3.22$$

where M_w is the magnitude of the dominant earthquake resulting from disaggregation of the hazard analysis in the desired return period for PGA, and Δ_{fs}, Δ_{fn}, Δ_{fr} and Δ_{fb} are the peak permanent surface displacements for the rupture of the fault with strike-slip mechanism, normal, reverse and the fault with low information, respectively. The resulting displacement must be multiplied by the importance factor of the desired member (see Table 13.1).

Chapter 4
Analysis Methods

4.1. General

This chapter provides the requirements for analysis. Section 4.10 provides the requirements for the equivalent lateral force procedure. The linear dynamic method is explained in Section 4.11. In Section 4.12 the requirements for time history analysis are explained.
All structures, along with the requirements of this chapter, should also satisfy the requirements of the Iranian Standard 2800.

4.1.2. Symbols

A	:	Design acceleration level
A_B	:	Section area at base
A_{si}	:	Section area of the ith shear wall in the direction under consideration
A_x	:	Torsional amplification factor
B	:	Response factor
C_d	:	Deflection amplification factor
C_{dx}	:	Deflection amplification factor in the X direction
C_{dy}	:	Deflection amplification factor in the Y direction
E	:	Effect of the horizontal and vertical earthquake-induced forces
F_i	:	Portion of the seismic base shear, V, induced at level i
F_{px}	:	Diaphragm seismic design force at level x
F_x	:	Portion of the seismic base shear, V, induced at level x
h_{sx}	:	Story height below level x($= h_x - h_{x-1}$)
h_{wall}	:	Height of a shear wall
h_x	:	Height above the base to level x
I	:	Importance factor
k_a	:	Coefficient of amplification factor for diaphragm flexibility
L_{wp}	:	Length of wall pier
L_{wall}	:	Length of a shear wall
M_t	:	Torsional moment resulting from eccentricity between locations of the center of mass and the center of rigidity
M_{ta}	:	Accidental torsional moment
N	:	Number of stories above the base
R_u	:	Response modification coefficient
R_{uX}	:	Response modification coefficient in the X direction
R_{uY}	:	Response modification coefficient in the Y direction

S_1	:	MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 s
S_a	:	5% damped design spectral response acceleration parameter at any period
S_{D1}	:	Design, 5% damped, spectral response acceleration parameter at a period of 1 s
S_{DS}	:	Design, 5% damped, spectral response acceleration parameter at short periods
T_s	:	S_{D1}/S_{DS}
T_L	:	Long-period transition period(s)
TIR	:	Torsional irregularity ratio
T_{lower}	:	Period of vibration at which 90% of the actual mass has been recovered in each of the two orthogonal directions of response
T_{upper}	:	Larger of the two orthogonal fundamental periods of vibration
V_{EX}	:	Maximum absolute value of elastic base shear computed in the X direction among all three analyses performed in that direction
V_{EY}	:	Maximum absolute value of elastic base shear computed in the Y direction among all three analyses performed in that direction
V_{IX}	:	Inelastic base shear in the X direction
V_{IY}	:	Inelastic base shear in the Y direction
W	:	Effective seismic weight of the building
W_i	:	Portion of W that is located at or assigned to level i
W_{px}	:	Weight tributary to the diaphragm at level x
W_x	:	Portion of W located at or assigned to level x
x	:	Level under consideration; x = 1 designates the first level above the base
β	:	Ratio of shear demand to shear capacity for the story between levels x and x−1
Δ_a	:	Allowable story drift
Δ_{max}	:	Maximum story drift at the building edge subjected to lateral forces
Δ_{ave}	:	Average of the story drifts at the two opposing edges of the building
δ_{DE}	:	Design earthquake displacement
δ_{di}	:	Displacement due to diaphragm deformation corresponding to the design earthquake
δ_e	:	Elastic displacement computed under design earthquake forces
δ_{max}	:	Maximum displacement at level x considering torsion

δ_{ave}	:	Average of displacements at the extreme points of the structure at level x
δ_M	:	Maximum inelastic response displacement considering torsion
δ_{MT}	:	Total separation distance between adjacent structures on the same property
δ_{M1}	:	Maximum inelastic response displacement of building 1
δ_{M2}	:	Maximum inelastic response displacement of building 2
η_x	:	Force scale factor in the X direction
η_y	:	Force scale factor in the Y direction
θ_i	:	Stability coefficient for P-delta effects
θ_{max}	:	Maximum stability coefficient for P-delta effects
ρ	:	Redundancy factor based on the extent of structural redundancy present in a building
Ω_0	:	Overstrength factor

4.2. Irregular and Regular Classification

Structures shall be classified as having a structural irregularity based on the criteria in this section. Such classification shall be based on their structural configurations. Vertical and horizontal structural irregularities are listed in Tables 4.1 and 4.2, respectively.
Structures having “Discontinuity in Lateral Strength–Extreme Weak Story Irregularity” shall not be permitted for all seismic design categories. Structures assigned to seismic design category D_1 and D_2 that have “Stiffness–Extreme Soft Story Irregularity”, “Discontinuity in Lateral Strength–Weak Story Irregularity” and “Torsional Irregularity with TIR>1.4” also shall not be permitted.

Exception1. Vertical structural “Geometric, Weight (Mass) and Stiffness (Soft Story and Extreme Soft Story) irregularities” in Table 4.1 do not apply where no design story drift ratio is greater than 130% of the story drift ratio of the next story above. For this exception, the following need not be considered:
(a) Torsional effects,
(b) The design story-drift ratio relationship of the top two stories of the structure.

Exception2. Vertical structural irregularities of "Stiffness–Soft Story Irregularity" and "Stiffness–Extreme Soft Story Irregularity" in Table 4.1 are not required to be considered for one or two-story buildings.

For structures having structural irregularity, the design forces shall be increased by 25% at each diaphragm level where the irregularity occurs for the following elements of the seismic force-resisting system:

1. Connections of diaphragms to vertical elements and to collectors,
2. Collectors and their connections, including connections to vertical elements of the seismic force-resisting system.

Table 4.1. Vertical Structural Irregularities

Irregularity Type and Description
a. Stiffness–Soft Story Irregularity: Where there is a story in which the lateral stiffness is less than 70% of that in the story above, or; where there are at least three stories above, less than 80% of the average stiffness of the three stories above.
b. Stiffness–Extreme Soft Story Irregularity: Where there is a story in which the lateral stiffness is less than 60% of that in the story above, or; where there are at least three stories above, less than 70% of the average stiffness of the three stories above.
c. Weight (Mass) Irregularity: Where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
d. Vertical Geometric Irregularity: Where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.
e. In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity: Where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements.
f. Discontinuity in Lateral Strength–Weak Story Irregularity: Where the story lateral strength is less than that in the story above. The story lateral strength is the total lateral strength of all seismic force-resisting system elements resisting the story shear for the direction under consideration.
g. Discontinuity in Lateral Strength–Extreme Weak Story Irregularity:

Where the story lateral strength is less than 65% of that in the story above. The story lateral strength is the total lateral strength of all seismic force-resisting system elements resisting the story shear for the direction under consideration.

Table 4.2. Horizontal Structural Irregularities

Irregularity Type and Description
a. Torsional Irregularity: Torsional irregularity, defined to exist where either: • More than 75% of any story's lateral strength below the diaphragm is provided at or on one side of the center of mass, or • The Torsional Irregularity Ratio (TIR) exceeds 1.2: $TIR = \frac{\Delta_{max}}{\Delta_{ave}}$, where Δ_{max} is the maximum story drift at the building's edge subjected to lateral forces using the equivalent lateral force with the application of accidental torsion and $A_x = 1.0$; and Δ_{ave} is the average of the story drifts at the two opposing edges of the building determined using the same loading and diaphragm rigidity as applied for the determination of Δ_{max}. The story lateral strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.
b. Reentrant Corner Irregularity: Reentrant corner irregularity, defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 20% of the plan dimension of the structure in the given direction.
c. Diaphragm Discontinuity Irregularity: Diaphragm discontinuity irregularity, defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cutout or open area greater than 25% of the gross enclosed diaphragm area, or a change in the effective diaphragm stiffness of more than 50% from one story to the next.
d. Out-of-Plane Offset Irregularity: Out-of-plane offset irregularity, defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.
e. Nonparallel System Irregularity: Nonparallel system irregularity, defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.

4.3. Seismic Importance Factor and Risk Category

Buildings and other structures shall be classified based on the risk to human life, health, and welfare associated with their damage or failure by nature of their occupancy or use, according to Table 4.3, for the purposes of applying flood, wind, tornado, snow, earthquake, and ice provisions.

Table 4.3. Risk Category of structures and Seismic Importance Factor

Risk Category	**Seismic Importance Factor (*I*)**
Risk category I including: a. Buildings and other structures required to maintain their functionality such as: -Control rooms -Mechanical equipment that are essential for the risk category I structures. -Power Stations. -Telecommunication towers. b. Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released.	1.5
Risk category II including: a. Buildings and other structures, the failure of which could pose a substantial risk to human life b. Buildings and other structures not included in Risk Category I, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure c. Buildings and other structures not included in Risk Category I (including, but not limited to, facilities that manufacture, process,	1.25

handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released	
Risk category III including: All buildings and other structures except those listed in Risk Categories I, II, and IV	1.0
Risk category IV including: a. Buildings and other structures that represent low risk to human life in the event of failure b. Temporary structures with a service life less than two years.	1.0

Where operational access to a Risk Category I structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Risk Category I structures. Where operational access is less than 3m from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the Risk Category I structure.

Note: In cases that there are some ambiguities in assigning the importance factor or risk category, the consultant should suggest a rational value for consideration and approval by the authority.

Base shear in service earthquake, V_{ser} in equivalent lateral load procedure is calculated from Equation 4.1.

$$V_{ser} = S_{a_{ser}} I\, W \qquad 4.1$$

where $S_{a_{ser}}$ is the service earthquake spectral acceleration (g) from site Specified Ground Motion Hazard Analysis and W is effective seismic weight of the structure.

4.4. Seismic Design Category

Structures shall be assigned a seismic design category in accordance with Table 4.4.

-Seismic design category 1 (D1): Risk Category I structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.6.

-Seismic design category 2 (D2): Risk Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.6.

-Seismic design category 3 (D3): All structures except those listed in seismic design category 1 or 2.

Note: Where $S_{DS} \geq 0.75$, the seismic design categories D_1 and D_2 shall be assigned to the Risk Category I and II structures, respectively.
The structural systems used shall be in accordance with the structural system limitations and the limits on structural height, contained in Table 4.5.

Table 4.4. Seismic design category based on response acceleration parameters

S_{DS}		S_1		*Seismic design category*
> 0.75	≤ 0.75	≥ 0.6	< 0.6	
D_1	D_3	D_1	D_3	I
D_2	D_3	D_2	D_3	II
D_3	D_3	D_2	D_3	III
D_3	D_3	D_2	D_3	IV

4.5. Structural Systems and Seismic Factors

4.5.1. Structural System Selection, Seismic Parameters and Limitations

The basic lateral and vertical seismic force resisting system shall conform to one of the types indicated in Table 4.5 for building structures and Table 7.1 or 7.2 for nonstructural components. The appropriate response modification coefficient, R_u; overstrength factor, Ω_0; and deflection amplification factor, C_d, indicated in Table 4.5 shall be used in determining the base shear, element design forces, and design story drift. Each selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the system as set forth in the applicable reference document listed in Table 4.5 and the additional requirements set forth in commentary. Use of seismic force resisting systems not contained in Table 4.5 shall be permitted contingent on submittal to and approval by the Authority Having Jurisdiction and independent structural design review of an accompanying set of design criteria and substantiating analytical and test data.

4.5.2. Combinations of Framing Systems in Different Directions

Different seismic force-resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R_u, C_d, and Ω_0 coefficients shall apply to each system, including the structural system limitations contained in Table 4.5.

4.5.3. Combinations of Framing Systems in the Same Direction

4.5.3.1. Vertical Combinations of Framing Systems in the Same Direction

Where a structure has a vertical combination in the same direction, the following requirements shall apply.

a. The period can be assumed as the weighted combination of periods of the systems.
b. The period should be derived from a model consisting the two systems without restraining their degrees of freedom.
c. Where the lower system has a lower response modification coefficient, R_u, the design coefficients (R_u, Ω_0, and C_d) for the upper system are permitted to be used to calculate the forces and drifts of the upper system. For the design of the lower system, the design coefficients (R_u, Ω_0, and C_d) for the lower system shall be used. Forces transferred from the upper system to the lower system shall be increased by multiplying by the ratio of the higher response modification coefficient to the lower response modification coefficient.
d. Where the upper system has a lower response modification coefficient, the design coefficients (R_u, Ω_0, and C_d) for the upper system shall be used for both systems.

Exception: Rooftop structures not exceeding two stories in height and 10% of the total structure weight, are exempted from this section.

4.5.3.2. Two-Stage Analysis Procedure for Vertical Combinations of Systems

A two-stage equivalent lateral force procedure is permitted to be used for structures that have a flexible upper portion above a rigid lower portion, provided the design of the structure complies with all of the following:

(a) Stiffness of the lower portion shall be at least 10 times the stiffness of the upper portion.

(b) Period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure supported at the transition plane from the upper to the lower portion.

(c) The upper portion shall be designed as a separate structure using the appropriate values of R_u and ρ.

(d) The lower portion shall be designed as a separate structure using the appropriate values of R_u and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion, where the effects of the horizontal seismic load, E_h, are amplified by the ratio of the R_u/ρ of the upper portion over R_u/ρ of the lower portion. This ratio shall not be less than 1.0.

(e) The upper portion is analyzed with the equivalent lateral force or modal response spectrum procedure, and the lower portion is analyzed with the equivalent lateral force procedure.

(f) The structural height of the upper portion shall not exceed the height limits of Table 4.5 for the seismic force resisting system used, where the height is measured from the base of the upper portion.

(g) Where out-of-plane offset horizontal irregularity or in-plane discontinuity in vertical lateral force-resisting element vertical irregularity exists at the transition from the upper to the lower portion, the reactions from the upper portion shall be amplified by the overstrength factor in addition to amplification required by Item 2.

4.5.3.3. Horizontal Combinations of Framing Systems in the Same Direction

The value of the response modification coefficient, R, used for design in the direction under consideration shall not be greater than the least value of R for any of the systems used in that direction. The deflection amplification factor,

C_d, and the overstrength factor, Ω_0, shall be consistent with R required in that direction.

Table 4.5 Design Coefficients and Factors for Seismic Force-Resisting Systems

	Seismic Force-Resisting System	R_u	Ω_0[1]	C_d	Structural Height Limitations (m)		
					D_1	D_2	D_3
A.	**BEARING WALL SYSTEMS**						
1	Special reinforced concrete shear walls	5	2.5	5	30	50	50
2	Ordinary reinforced concrete shear walls	Not Permitted					
3	Intermediate precast shear walls	4	2.5	4	12	12	12
4	Ordinary precast shear walls	Not Permitted					
5	Special reinforced masonry shear walls	5	2.5	3.5	30	50	50
6	Intermediate reinforced masonry shear walls	Not Permitted					
7	Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	Not Permitted					
8	Light-frame (cold-formed steel) wall systems using flat strap bracing	4	2	3.5	20	20	20
9	Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	6.5	3	4	20	20	20
10	Reinforced concrete ductile coupled walls	8	2.5	8	30	50	50
11	Shotcrete shear wall	3	2	3	NP[2]	10.5	15
B.	**BUILDING FRAME SYSTEMS**						
1	Steel eccentrically braced frames[3]	8	2	4	30	50	50
2	Steel special concentrically braced frames	6	2	5	30	50	50
3	Steel ordinary concentrically braced frames[4]	3.25	2	3.25	NP[2]	10	10
4	Steel buckling-restrained braced frames	8	2.5	5	30	50	50
5	Special reinforced concrete shear walls	6	2.5	5	30	50	50
6	Ordinary reinforced concrete shear walls	Not Permitted					
7	Intermediate precast shear walls	5	2.5	4.5	12	12	12
8	Steel and concrete composite eccentrically braced frames	8	2.5	4	30	50	50
9	Steel and concrete composite special concentrically braced frames	5	2	4.5	30	50	50
10	Steel and concrete composite ordinary braced frames	Not Permitted					
11	Steel special plate shear walls	7	2	6	30	50	50
12	Steel and concrete composite special shear walls	6	2.5	5	30	50	50
13	Steel and concrete composite ordinary shear walls	Not Permitted					
14	Special reinforced masonry shear walls	5.5	2.5	4	30	50	50

15	Intermediate reinforced masonry shear walls	Not Permitted					
16	Light-frame walls with shear panels of all other materials	2.5	2.5	2.5	NP[2]	NP[2]	10
17	Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	7	2.5	4.5	20	20	20
C.	**MOMENT-RESISTING FRAME SYSTEMS**						
1	Steel special moment frames	8	3	5.5	NL[2]	NL[2]	NL[2]
2	Steel intermediate moment frames	4.5	3	4	NP[2]	NP[2]	10
3	Steel ordinary moment frames	Not Permitted					
4	Steel special truss moment frames	7	3	5.5	NP[2]	30	50
5	Special reinforced concrete moment frames	8	3	5.5	NL[2]	NL[2]	NL[2]
6	Intermediate reinforced concrete moment frames	Not Permitted					
7	Ordinary reinforced concrete moment frames	Not Permitted					
8	Steel and concrete composite special moment frames	8	3	5.5	NL[2]	NL[2]	NL[2]
9	Steel and concrete composite intermediate moment frames	Not Permitted					
D.	**DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES**						
1	Steel eccentrically braced frames	8	2.5	4	NL[2]	NL[2]	NL[2]
2	Steel special concentrically braced frames	7	2.5	5.5	NL[2]	NL[2]	NL[2]
3	Steel buckling-restrained braced frames	8	2.5	5	NL[2]	NL[2]	NL[2]
4	Special reinforced concrete shear walls	7	2.5	5.5	NL[2]	NL[2]	NL[2]
5	Ordinary reinforced concrete shear walls	Not Permitted					
6	Steel and concrete composite eccentrically braced frames	8	2.5	4	NL[2]	NL[2]	NL[2]
7	Steel and concrete composite special concentrically braced frames	6	2.5	5	NL[2]	NL[2]	NL[2]
8	Steel special plate shear walls	8	2.5	6.5	NL[2]	NL[2]	NL[2]
9	Steel and concrete composite plate shear walls	7.5	2.5	6	NL[2]	NL2	NL[2]
10	Steel and concrete composite special shear walls	7	2.5	6	NL[2]	NL[2]	NL[2]
11	Steel and concrete composite ordinary shear walls	Not Permitted					
12	Special reinforced masonry shear walls	5.5	3	5	NL[2]	NL[2]	NL[2]
13	Intermediate reinforced masonry shear walls	Not Permitted					
E.	**DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES**						
1	Steel special concentrically braced frames	6	2.5	5	NP[2]	NP[2]	10
2	Special reinforced concrete shear walls	6.5	2.5	5	30	30	50
3	Steel and concrete composite special concentrically braced frames	5.5	2.5	4.5	NP[2]	30	50

4	Steel and concrete composite ordinary braced frames	Not Permitted					
F.	**CANTILEVERED COLUMN SYSTEMS[5] DETAILED TO CONFORM TO THE REQUIREMENTS FOR:**						
1	Special reinforced concrete moment frames	2.5	2.5	2.5	10	10	10
2	Steel special cantilever column systems	2.5	2.5	2.5	10	10	10
3	Steel ordinary moment frames	Not Permitted					
4	Intermediate reinforced concrete moment frames	Not Permitted					
G.	**STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS**	Not Permitted					

[1] Where the tabulated value of the overstrength factor, Ω_0, is greater than or equal to 2.5, Ω_0 is permitted to be reduced by subtracting the value of 0.5 for structures with flexible diaphragms.
[2] NP = Not Permitted, and NL = Not Limited
[3] Steel ordinary concentrically braced frames are permitted in single-story buildings up to a structural height of 20 m where the dead load of the roof does not exceed 1 kN/m^2 and in penthouse structures.

[4] In cantilever column systems the required axial strength of individual cantilever column elements, considering only the load combinations that include seismic load effects, shall not exceed 15% of the available axial strength, including slenderness effects. Foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall be designed to resist the seismic load effects, including overstrength of Section 2.2.4.

4.6 New Structural Systems

Any new and advanced structural system, other than those of Table 4.5, may also be considered. However, for those first a certificate from related organizations should be obtained by the advice of the Authority Having Jurisdiction.

4.7 Redundancy Factor, ρ

For structures in the direction of interest, ρ shall be taken as 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0.

Condition 1: For each story where the story shear is greater than 35% of the base shear in the direction of interest, the following conditions shall be met with the notional removal of any lateral force resisting element or connection as indicated in Table 4.6

(a) There are at least two bays of seismic force-resisting framing on each side of the center of mass.

(b) The reduction in lateral strength of the story in the direction of interest does not exceed 35%.

(c) The resulting system with consideration of removal of the element does not have a Type 1 Horizontal Irregularity with a TIR > 1.4.

Condition 2: The structure does not have any horizontal irregularities as defined in Table 4.2 in the direction of interest at all levels, and the seismic force-resisting systems shall consist of at least two bays of seismic force-resisting perimeter framing on each side of the structure at each story resisting more than 35% of the base shear. The number of bays of shear walls shall be as defined in Table 4.6.

Exception: The value of ρ is permitted to equal 1.0 for the following:

1. Drift, Δ_x, calculation
2. P-delta effects;
3. Design of nonstructural components;
4. Design of nonbuilding structures not similar to buildings in accordance with Section 7.3;
5. Design of collector elements, splices, and their connections for which the seismic load effects including overstrength of Section 2.2.4 are used;
6. Design of members or connections where the seismic load effects, including overstrength of Section 2.2.4, are required for design;
7. Diaphragm seismic design forces determined in accordance with Section 4.13;

8. Structures with damping systems designed in accordance with Chapter 10; and
9. Design of structural walls for out-of-plane forces, including their anchorage.

Table 4.6 Requirements for each story resisting more than 35% of the base shear

Lateral Force-Resisting Element	Condition
Braced frames, light-frame walls with flat strap bracing	Removal of an individual brace, or connection thereto
Moment frames	Loss of moment resistance at the beam-to-column connections at both ends of a single beam
Shear walls or wall piers with a height-to-length ratio greater than 1.0	Removal of a shear wall bay, or wall pier, with a height-to-length ratio greater than 1.0, or connections thereto. The shear wall and wall pier height-to-length ratios are determined as shown in Figure 4.1. A shear wall bay is defined as the length of the wall divided by the story height (rounded down).
Cantilever columns	Loss of moment resistance at the base of any single cantilever column
Other structural systems	Without condition

4.8. Direction of Loading

The directions of application of seismic forces used in the design shall be those that produce the most critical load effects. In lieu of an analysis that finds the critical direction of loading for each element of the seismic force resisting system, it is permitted to satisfy this requirement using one of the two methods defined in the following:

First Method: The design seismic forces shall be applied independently in each of the two orthogonal directions.
Second Method: The design seismic forces shall be applied in two orthogonal directions simultaneously using one of the two following approaches. Seismic force resisting system member and foundation strength and drift requirements shall be satisfied for all combinations.

(a) For a structure analyzed using the equivalent lateral force analysis procedure of Section 4.10 or the modal response spectrum analysis procedure of Section 4.10, the effects of 100% of the seismic forces for one direction shall be combined with the effects of 30% of the forces for the perpendicular direction. Combinations of force effects shall be computed such that each axis has the 100% factor in positive and negative directions.

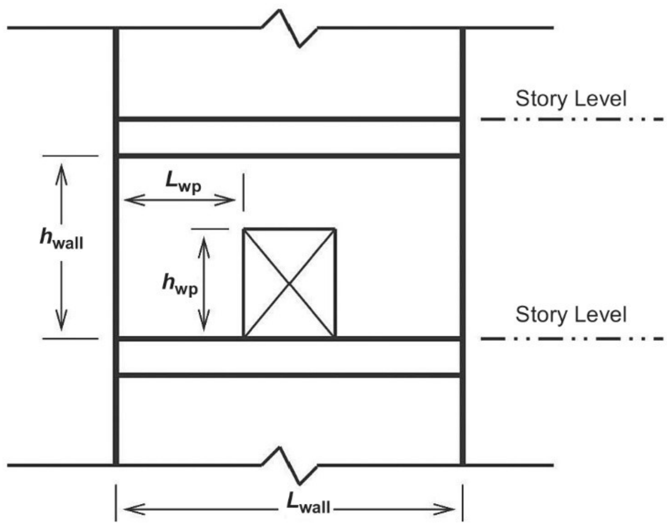


Figure 4.1. Shear wall and wall pier height-to-length ratio determination

h_{wall} = Height of shear wall;
h_{wp} = Height of wall pier;
L_{wall} = Length of shear wall;
L_{wp} =Length of wall pier;
h_{wall}/L_{wall} = Shear wall height-to- length ratio;
h_{wp}/L_{wp} = Wall pier height-to-length ratio.

(b) For a structure analyzed using the linear response history procedure of Section 4.12.2 or the nonlinear response history procedure of Section 4.12.3, orthogonal pairs of ground motion shall be used concurrently.

The second method shall be permitted if one or more of the following conditions exist:

1. A column that forms part of two or more intersecting seismic force-resisting systems and is subjected to axial load due to seismic forces acting

along either principal plan axes, exceeding 20% of the axial design strength of the column.

2. The structure has a torsional (Type a of Table 4.2) and nonparallel system (Type f of Table 4.2) horizontal irregularity.

4.9. Modeling Criteria

4.9.1. Structural Modeling

A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or P-delta effects. The model shall include the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure. When modal response spectrum or response history analysis is performed, a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis at each level of the structure shall be used.

In addition, the model shall comply with the following:

1. Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections.
2. For steel moment frame systems, the contribution of panel zone deformations to displacement and drift shall be included.

Structures that have horizontal structural irregularity Type 1, 4, or 5 of Table 12.3.1, shall be analyzed using a 3D representation. Where a 3D model is used, a minimum of three degrees of freedom consisting of translation in two orthogonal plan directions and rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms have not been classified as rigid or flexible in accordance with Section 4.13, the model shall include representation of the diaphragm's stiffness characteristics and, when dynamic analysis is performed, sufficient degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

Exception: Analysis using a 3D representation is not required for structures with flexible diaphragms that have Type 4 horizontal structural irregularities.

4.9.2. P-Δ Effects

P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects, are not required to be considered where the stability coefficient, θ, as determined by equation 4-18 is equal to or less than 0.10.
Where the stability coefficient, θ, is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $1.0/(1-\theta)$. Where θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

4.9.3. Infilled Frames and Interaction Effects

Moment-resisting frames that are enclosed or adjoined by elements that are more rigid and not considered to be part of the seismic force-resisting system, shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force-resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at structural deformations corresponding to the design story drift (Δ). In addition, the effects of these elements shall be considered where determining whether a structure has one or more of the irregularities defined in Table 4.1.

4.9.4. Effective Seismic Weight

The effective seismic weight, W, of a structure shall include the dead load above the base and other loads above the base mentioned in Table 4.7.

4.10 Equivalent Lateral Force (ELF) Procedure

4.10.1. General

The equivalent force procedure can be used for one- or two-story structures in risk categories III or IV, or structures with LSF (Light Steel Frame), or regular structures with a height less than 50m from the base.

4.10.2 Seismic Base Shear

The seismic base shear, V_u, in a given direction shall be determined in accordance with equation 4.2:

$$V_u = \frac{S_a}{R_u/I} W \qquad 4.2$$

where

S_a: Design spectral response acceleration parameter (g) with 5% damping ratio defined in Chapter 3.

I: Importance factor determined in accordance with Table 4.3.

W: Effective seismic weight of a structure including the dead load and other loads above the base (Section 4.17). Where provision for partitions is made, the actual partition weight must not be considered less than 50 daN/m^2.

Table 4.7 Percentage of participation of the live or snow loads in calculation of the seismic lateral force

Type of live or snow load	Percentage of participation of live or snow loads
Flat roofs in areas with heavy snow	20
Flat roofs in other areas	-
Residential building, office, hotel, public garages, hospital, school, stores and crowded buildings	20
Library and storage	Minimum of 40
Fluids and bulk material	100

R_u: Response modification factor in Table 4.5;

The effective seismic weight, W, shall include the following items:

1. In areas used for storage, a minimum of 25% of the floor live load shall be included.
2. Partition loads in accordance with the Iranian National Building Regulations, Clause 6.
3. Total operating weight of permanent equipments.
4. 20% of the uniform design snow or live flat load, (in accordance with the Iranian National Building Regulations, Clause 6), whichever is greater.

4.10.2.1. Minimum Base Shear

The minimum value of V_u for structural buildings shall not be less than Eq. 4.3.

$$V_{min} = 0.044 S_{Ds} W I \geq 0.01 W \qquad 4.3$$

In addition, for structures located where $S_1 \geq 0.6$, V_u shall not be less than Eq. 4.4.

$$V_{min} = (0.5 S_1 W)/(R_u/I) \qquad 4.4$$

4.10.2.2. Maximum Base Shear

The maximum value of V_u for structural buildings need not be larger than Eqs. 4.5 and 4.6.

For $T \leq T_L$,

$$V_{max} = \frac{S_{D1}}{T\left(\frac{R_u}{I}\right)} W \qquad 4.5$$

and for $T > T_L$,

$$V_{max} = \frac{S_{D1} T_L}{T^2\left(\frac{R_u}{I}\right)} W \qquad 4.6$$

where:

I: Importance factor determined in accordance with Table 4.3.

R_u: Response modification factor in Table 4.5;

S_{D1}: Design spectral response acceleration parameter at a period of 1.0 s,

The value of T_L shall be extracted from Chapter 3.

4.10.3. Period Determination

Period is derived from:

$$T = C_t H^x \qquad 4.7$$

where:

H: Structural height as defined in Table 4.5 (m)

x: Height exponent determined from Table 4.8

C_t: Factor of period determined from Table 4.8

The fundamental period, T, shall not exceed the product of the coefficient for the upper limit on the calculated period, C_{Tu}, from Table 4.9.
It is permitted to determine the approximate fundamental period, T, in seconds, from Eq. 4.8 for structures not exceeding 12 stories above the base where the seismic force-resisting system consists entirely of concrete or steel moment-resisting frames and the average story height is at least 3 m.

$$T = 0.1\, N \tag{4.8}$$

where N is the number of stories above the base.

Table 4.8 Values of the approximate period parameters C_t and x

Structure type	C_t	x
Steel moment-resisting frames	0.072	0.8
Concrete moment-resisting frames	0.047	0.9
Steel eccentrically braced frames	0.073	0.75
Steel buckling-restrained braced frames	0.073	0.75
All other structural systems	0.049	0.75

Table 4.9 Upper limit coefficient of approximate period

C_{Tu}	S_{D1}
1.4	≥ 0.3
1.5	0.2
1.6	0.15
1.7	≤ 0.1

Note 1:
If the infill restrains the frame lateral displacement, $x = 0.75$ and $C_t = 0.049$.

Note 2: The approximate fundamental period, T, in seconds, for masonry or concrete shear wall structures is permitted to be determined from Eq.:

$$T = \frac{0.00058}{\sqrt{C_w}} H \tag{4.9}$$

C_w is calculated from Eq.4.10:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{m} \frac{A_{si}}{\left[1 + 0.83 \left(\frac{H}{D_i}\right)^2\right]} \qquad 4.10$$

where:
A_B: Area of the base of structure (m^2)
m: Number of shear walls in the building effective in resisting lateral forces in the direction under consideration
A_{si}: Web area of shear wall i
D_i: Length of shear wall i

4.10.4. Vertical Distribution of Seismic Forces

The lateral seismic force, induced at level x shall be determined from Eqs. 4.11 and 4.12:

$$F_x = C_{vx} V_u \qquad 4.11$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad 4.12$$

where:
C_{vx}: Vertical distribution factor;
w_i, w_x: Portion of the total effective seismic weight of the structure (W) located or assigned to level i or x;
h_i, h_x: Height (m) from the base to level i or x;
n: Number of building levels
k: Exponent related to the structure period determined from Eq. 4.13.

$$k = \begin{cases} 1 & T \leq 0.5 \\ 0.5T + 0.75 & 0.5 < T < 2.5 \\ 2 & T \geq 2.5 \end{cases} \qquad 4.13$$

Location of the base shall be determined in accordance with Section 17.4

4.10.5. Horizontal Distribution of Forces

The seismic design story shear in any story between level x and x-1, V_x, shall be determined from Eq. 4.14:

$$V_x = \sum_{i=x}^{n} F_i \tag{4.14}$$

where:
F_i: Portion of the seismic base shear, V, induced at level i.
The seismic design story shear, V_x, shall be distributed to the various vertical elements of the seismic force-resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

4.10.6. Inherent Torsion

For diaphragms that are not flexible, distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t, resulting from eccentricity between the locations of the center of mass and the center of rigidity. For flexible diaphragms, distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

4.10.6.1. Accidental Torsion

Accidental torsional moments, M_{ta}, shall be determined using an assumed relocation of the center of mass each way from its actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces. Where earthquake forces are applied concurrently in two orthogonal directions, the required 5% move of the center of mass need not be applied in both of the orthogonal directions simultaneously but shall be applied in the direction that produces the greater effect for each element considered.
Accidental torsion shall be included in the analysis and design of structures assigned to Seismic Design Category 1. For structure assigned to Seismic Design Category 2 and 3, accidental torsion shall be applied if torsional horizontal structural irregularity exists.

4.10.6.2. Amplification of Accidental Torsional Moment

A structure with rigid diaphragm and torsional horizontal structural irregularity as defined in Table 4.2, shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor, A_x, as determined from Eq. 15.4:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{ave}}\right)^2 \qquad 4.15$$

Where:

δ_{max}: Maximum displacement (mm) at level x, computed assuming $A_x = 1$

δ_{ave}: Average of the displacements (mm) at the extreme points of the structure at level x, computed assuming $A_x = 1$ (see Fig. 4.2)

The torsional amplification factor, A_x, shall not be less than 1 and is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

4.10.7. Overturning

The structure shall be designed to resist overturning effects caused by the seismic forces.

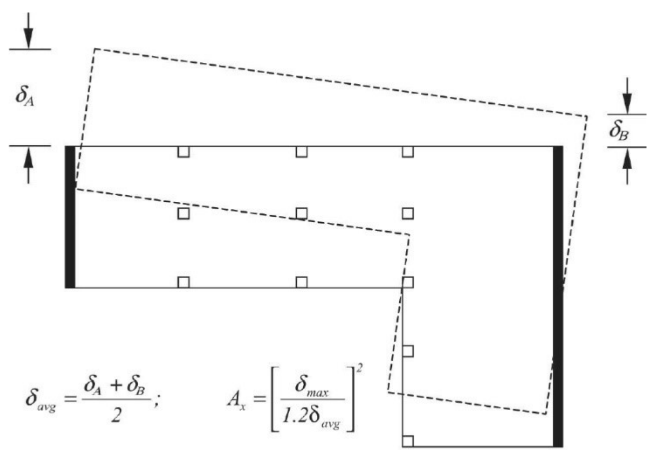


Fig. 4.2. Torsional amplification factor, A_x

4.10.8 Story Drift

4.10.8.1. Design Displacement and Drift Determination

The design displacement, δ_{DE}, shall be determined at the location of an element or component using Eq. 4.16.

$$\delta_{DE} = \frac{C_d \delta_e}{I} + \delta_{di} \qquad 4.16$$

where:

C_d: Deflection amplification factor in Table 4.5.

δ_e: Elastic displacement computed under design earthquake forces, including the effects of accidental torsion and torsional amplification as applicable;

I: Importance factor determined in accordance with Table 4.3;

δ_{di}: Displacement due to diaphragm deformation corresponding to the design earthquake, including diaphragm forces mentioned in Section 4.13.

The design drift, Δ, shall be computed as the difference of the design displacements, δ_{DE}, as determined in accordance with Equation 4.17, at the centers of mass at the top and bottom of the story under consideration.

$$\Delta_x = \delta_{DEx} - \delta_{DEx-1} \qquad 4.17$$

δ_{DEx}: Design displacement at level x

For structures that have horizontal torsional irregularity in accordance with Table 4.2, the design drift, shall be computed as the largest difference of the design displacements of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure, including the effects of diaphragm rotation.

The elastic analysis of the seismic force-resisting system for computing displacement and drift shall be made using the prescribed seismic design forces using a load factor of 1.0

Exception: For determining displacements and drifts, it is permitted to determine the elastic displacements, δ_e, using seismic design forces based on the computed fundamental period of the structure without the upper limit, C_uT_a, specified in Section 4.10.3. In addition, Eq. 4.3 for the minimum base shear need not be considered for drift calculation.

4.10.9. P-Delta Effects

P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered where the stability coefficient, θ, as determined by Eq. 4.18 is equal to or less than 0.10. Where the stability coefficient, θ, is greater than

0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. The stability coefficient, θ, shall not exceed θ_{max} in Eq. 4.19.

$$\theta_i = \frac{P_i \Delta_i}{V_i h_i} \quad 4.18$$

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \quad 4.19$$

where:

P_i: Total vertical design load at and above level i; when computing P_i, no individual load factor needs exceed 1.0;

V_i: Story stiffness at level i;

Δ_i: Elastic story drift between levels x and x − 1

h_i: Story height below level x

β: Ratio of shear demand to design shear capacity for the story between levels x and x − 1. The value of β is permitted to be conservatively taken as 1.0, and shall not be taken less than1.25/Ω_0 in Eq. 4.19.

C_d: Deflection amplification factor in Table 4.5.

4.11. Linear Dynamic Analysis

4.11.1. General

All structures designed in accordance with this section shall be analyzed using a three-dimensional representation.

Where the diaphragms have not been classified as rigid in accordance with Section 4.13, the model shall include representation of the diaphragm's stiffness characteristics and additional dynamic degrees of freedom as required to account for the participation of the diaphragm in the structure's dynamic response. The P-delta effects shall be determined in accordance with Section 4.11.9. A soil–structure interaction reduction is permitted where determined using Chapter 6.

4.11.2. Modal Response Parameters

The value for each related design parameter of interest, including story drifts, support forces, and individual member forces for each mode of response, shall be computed using the properties of each mode and the response spectra divided by the quantity R_u /I. The value for displacement and drift quantities shall be multiplied by the quantity C_d /I.

4.11.3. Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of 100% of the structure's mass. For this purpose, it shall be permitted to represent all modes with periods less than 0.05 s in a single rigid body mode that has a period of 0.05 s.

Alternatively, the analysis shall be permitted to include a minimum number of modes to obtain a combined modal mass participation of at least 90% of the actual mass in each orthogonal horizontal direction of response considered in the model.

4.11.4. Combined Response Parameters

The value for each parameter of interest calculated for the various modes shall be combined using two following methods:

- Square root of the sum of the squares (SRSS) method
- Complete quadratic combination (CQC) method. This method shall be used for each of the modal values where closely spaced modes have significant cross-correlation of translational and torsional response.

4.11.5. Scaling of Responses

Where the combined response for the modal base shear, V_t , is less than 100% of the calculated base shear, V, using the equivalent lateral force procedure, the forces shall be multiplied by V/V_t.

4.11.6. Torsion

Distribution of horizontal shear shall be in accordance with Section 4.10.6. The effects of accidental torsion shall be accounted for by applying a static accidental torsional moment, M_{ta}, determined in accordance with Section 4.10.6.1, to the mathematical model, and combining the results with the scaled design values computed in accordance with Section 4.11.5.

For structures without a torsional horizontal irregularity, the effects of accidental torsion may be included in the dynamic analysis model in lieu of applying M_{ta}. When the effects of accidental torsion are included in the dynamic analysis model and TIR $\leq$ 1.6, amplification of torsion in accordance with Section 4.10.6.2 is not required. If TIR > 1.6, accidental torsion shall be added as a static load case in accordance with Section 4.10.6.2.

4.11.7. Overturning

The structure shall be designed to resist overturning effects in accordance with section 4.10.7.

4.11.8. Story Drift

Story drift determined in accordance with Section 4.11.5 shall not exceed the allowable maximum displacements determined in Section 4.15.

4.11.9. P-Delta Effects

The P-delta effects shall be determined in accordance with Section 4.10.9. Displacement and drifts shall be determined in accordance with Section 4.10.8.

4.11.10. Soil–Structure Interaction Effects

A soil–structure interaction reduction is permitted where determined using Chapter 6 or other generally accepted procedures approved by the Authority Having Jurisdiction.

4.11.11. Structural Modeling

A mathematical model of the structure shall be constructed in accordance with Section 4.9, except that all structures designed in accordance with this section shall be analyzed using a three-dimensional representation. Where the diaphragms have not been classified as rigid in accordance with Section 4.13, the model shall include representation of the diaphragm's stiffness characteristics and additional dynamic degrees of freedom as required to account for the participation of the diaphragm in the structure's dynamic response.

4.12. Time-History Analysis

4.12.1. General

Time-history analysis can be linear based on 4.12.2 or nonlinear based on 4.12.3

4.12.2. Linear Response History Analysis

Linear response history analysis shall consist of an analysis of a linear mathematical model of the structure to determine its response through methods of numerical integration, to suites of spectrally matched acceleration histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the requirements of this section.

4.12.2.1. Procedure for Spectrum Matching

Each component of ground motion shall be spectrally matched over the period range $0.8T_{lower}$ to $1.2T_{upper}$. Over the same period range and in each direction of response, the average of the 5% damped pseudo acceleration ordinates computed using the spectrum-matched records shall not fall above or below the target spectrum by more than 10% in each direction of response.

4.12.2.2. Modification of Response for Design

4.12.2.2.1. Determination of Maximum Elastic and Inelastic Base Shear

For each ground motion analyzed, a maximum elastic base shear, designated as V_{EX} and V_{EY} in the X and Y directions, respectively, shall be determined. The mathematical model used for computing the maximum elastic base shear shall not include accidental torsion. For each ground motion analyzed, a maximum inelastic base shear, designated as V_{IX} and V_{IY} in the X and Y directions, respectively, shall be determined as follows:

$$V_{IX} = \frac{V_{EX}}{(R_{uX}/I)} \qquad \text{4.20-a}$$

$$V_{IXY} = \frac{V_{EY}}{(R_{uY}/I)} \qquad \text{4.20-b}$$

where I, is the importance factor and R_{uX} and R_{uY} are the response modification coefficients for the X and Y directions, respectively.

4.12.2.2.2. Determination of Base Shear Scale Factor

Design base shears, V_X and V_Y, shall be computed in the X and Y directions, respectively, in accordance with Section 4.12.2.2.1. For each ground motion analyzed, base shear scale factors in each direction of response shall be determined as follows:

$$\eta_X = \frac{V_X}{V_{IX}} \geq 1 \qquad 4.21\text{-a}$$

$$\eta_Y = \frac{V_{XY}}{V_{IY}} \geq 1 \qquad 4.21\text{-b}$$

4.12.2.2.3. Determination of the Combined Force Response

For each direction of response and for each ground motion analyzed, the combined force response shall be determined as follows:

(a) The combined force response in the X direction shall be determined as $I.\eta_X / R_{uX}$ times the computed elastic response in the X direction using the mathematical model with accidental torsion, where required, plus $I.\eta_Y/R_{uY}$ times the computed elastic response in the Y direction using the mathematical model without accidental torsion.

(b) The combined force response in the Y direction shall be determined as $I.\eta_Y / R_{uY}$ times the computed elastic response in the Y direction using the mathematical model with accidental torsion, where required, plus $I.\eta_X / R_{uX}$ times the computed elastic response in the X direction using the mathematical model without accidental torsion.

4.12.2.2.4. Determination of the Combined Displacement Response

Response modification factors C_{dX} and C_{dY} shall be assigned in the X and Y directions, respectively. For each direction of response and for each ground motion analyzed, the combined displacement responses shall be determined as follows:

(a) The combined displacement response in the X direction shall be determined as $C_{dX}\eta_X / R_{uX}$ times the computed elastic response in the X direction using the mathematical model with the accidental torsion, where required, plus $C_{dY}\eta_Y / R_{uY}$ times the computed elastic response in the Y direction using the mathematical model without the accidental torsion.

(b) The combined displacement response in the Y direction shall be determined as $C_{dY}\eta_Y / R_{uY}$ times the computed elastic response in the Y

direction using the mathematical model with the accidental torsion, where required, plus $C_{dX}\eta_X / R_{uX}$ times the computed elastic response in the X direction using the mathematical model without accidental torsion.

Note 1: Where the design base shear in the given direction is not controlled by Eq. (12.8.7), the factors η_X or η_Y, as applicable, are permitted to be taken as 1.0 for the purpose of determining the combined displacements.

4.12.2.2.5. Enveloping of Force Response Quantities

Design force response quantities shall be taken as the envelope of the combined force response quantities computed in both orthogonal directions and for all ground motions considered. Where force interaction effects are considered, demand to capacity ratios are permitted to be enveloped in lieu of individual force quantities.

4.12.2.2.6. Enveloping the Displacement Response Quantities

Story drift quantities shall be determined for each ground motion analyzed and in each direction of response using the combined displacement responses defined in Section 4.12.2.2.4. For the purpose of complying with the drift limits specified in Section 4.15, the envelope of story drifts computed in both orthogonal directions and for all ground motions analyzed shall be used.

4.12.2.3. Number of Modes to Include in Modal Response History Analysis

Where the modal response history analysis procedure is used, number of modes to include in the analysis shall be in accordance with Section 4.11.3.

4.12.2.4. Damping

Linear viscous damping shall not exceed 5% critical for any mode with a vibration period greater than or equal to T_{lower}.

4.12.2.5 Accidental Torsion

Accidental torsion, where required by Section 4.10.6.1, shall be included by offsetting the center of mass in each direction (i.e., plus or minus) from its expected location by a distance equal to 5% of the horizontal dimension of the structure at the given floor measured perpendicular to the direction of loading. Amplification of accidental torsion in accordance with Section 4.10.6.2 is not required.

4.12.2.6. P-Delta Effects

The mathematical model shall include P-delta effects. Limits on the stability coefficient, θ, shall be satisfied in accordance with Section 4.10.9.

4.12.2.7. Overturning

Overturning should be checked based on 4.10.7.

4.12.3. Nonlinear Response History Analysis

4.12.3.1. General Requirements

Nonlinear response history analysis shall include the effects of horizontal motion. Nonlinear response history analysis shall explicitly include the effects of vertical response where vertical elements of the gravity force-resisting system are discontinuous.

4.12.3.2. Design Spectral Parameters

A suite of not less than 11 ground motions shall be selected for each target spectrum. Design element forces, nonlinear displacements and story drifts shall be taken as the mean value of the response parameter of interest obtained from the suite of analyses.

4.12.3.3. P-Delta Effects

The mathematical model shall include the P-delta effects.

4.12.3.4. Torsion

Inherent eccentricity resulting from any offset in the centers of mass and stiffness at each level shall be accounted for in the analysis. In addition, where a horizontal torsional structural irregularly exists, as defined in Table 4.2, accidental eccentricity consisting of an assumed displacement of the center of mass each way from its actual location by a distance equal to 5% of the diaphragm dimension of the structure parallel to the direction of mass shift, shall be considered. The required 5% displacement of the center of mass need not be applied in both orthogonal directions at the same time.

4.12.3.5. Design Review

An independent structural design review shall be performed in accordance with the requirements of this section. Reviewer(s) shall consist of one or more individuals acceptable to the Authority Having Jurisdiction and possessing knowledge of nonlinear time history analysis and behavior of structural systems subjected to cyclic loading.

4.12.3.6. Story Drift

The story drift for all building heights shall not exceed 1.25 times the limits of Table 4.10.

4.12.3.7. Overturning

The structure shall be designed to resist overturning effects in accordance with Section 4.10.7.

4.13 Diaphragms, Chords, and Collectors

The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force-resisting system.

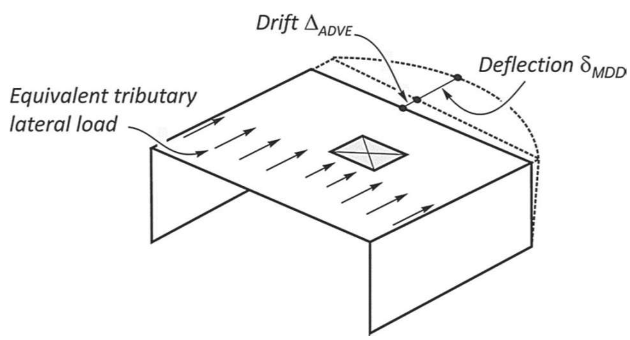


Figure 4.3. Flexible diaphragm

Diaphragms of concrete slabs or concrete-filled metal deck with span-to-depth ratios of 3 or less in structures that do not have a Horizontal Structural Irregularity are permitted to be idealized as rigid.

4.13.1. Design Diaphragms

Diaphragms shall be designed for both the shear and the bending stresses resulting from design forces. At diaphragm discontinuities, such as openings and reentrant corners, design shall ensure that the dissipation or transfer of edge (chord) forces, combined with other forces in the diaphragm, is within the shear and tension capacity of the diaphragm.

Floor and roof diaphragms shall be designed to resist in-plane seismic design forces from the structural analysis but shall not be less than that determined in accordance with Eq. (4.22) as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad 4.22$$

The force determined from Eq. (4.23) shall not be less than:

$$F_{px} = 0.2 S_{DS} I w_{px} \qquad 4.23$$

$$F_{px} = 0.4 S_{DS} I w_{px} \qquad 4.24$$

F_i: Design force applied to level i;

w_{px}: Weight tributary to the diaphragm at level x.

Diaphragms shall be designed for the inertial forces determined from Eq.(4.22) through (4.24) and for applicable transfer forces resisted by the diaphragm between vertical seismic force-resisting elements. For structures that have an Out-of-Plane Offset horizontal structural irregularity in Table 4.2, the transfer forces between horizontally offset vertical seismic force resisting elements shall be increased by the overstrength factor before being added to the diaphragm inertial forces

4.13.2. Collector Design

Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing resistance to those forces.

Collector elements and their connections, including connections to vertical elements, shall be designed to resist the maximum of the following:

a) Forces calculated using the seismic load effects including overstrength in Section 2.2.4 in which the seismic forces are determined by the equivalent lateral force procedure of Section 4.10 or the modal response spectrum analysis procedure of Section 4.11;

(b) Forces calculated using the seismic load effects including overstrength in Section 2.2.4 in which the seismic forces are determined by Eq. (4.22); and,

(c) Forces calculated using the load combinations of Section 2.2.3 in which the seismic forces are determined by Eq. (4.23).

4.13.3. Increase in Forces Caused by Irregularities

For structures having a horizontal structural irregularity of Types a to e in Table 4.1 or a vertical structural irregularity of Type c in Table 4.2, the design forces determined using Section 4.13.1 shall be increased by 25% at each diaphragm level where the irregularity occurs for the following elements of the seismic force-resisting system:

1. Connections of diaphragms to vertical elements and to collectors,
2. Collectors and their connections, including connections to vertical elements of the seismic force-resisting system.

Exception: Forces calculated using the seismic load effects including overstrength in Section 2.2.4, need not be increased.

4.14. Structural Walls

4.14.1. Design for Out-of-Plane Forces

Structural walls shall be designed for a force F_p normal to their surface equal to $0.4S_{DS}I$ times the weight of the structural wall with a minimum force of 10% of the weight of the structural wall.

4.14.2. Anchorage of Structural Walls and Transfer of Design Forces into Diaphragms or Other Supporting Structural Elements

The anchorage of structural walls to supporting construction shall provide a direct connection capable of resisting the following force:

$$F_P = 0.4S_{Ds}k_a I W_P \tag{4.25}$$

in which:

$$k_a = 1 + L_f/30 \tag{4.26}$$

where:

F_P: Design force in the individual anchors;

S_{DS}: Design spectral response acceleration parameter at short periods per Chapter 3
I: Importance factor determined in accordance with Section 4.3;
K_a: Amplification factor for diaphragm flexibility;
L_f: Span, in meter, of a flexible diaphragm that provides the lateral support for the wall; span is measured between the vertical elements that provide lateral support to the diaphragm in the direction considered (use zero for rigid diaphragms); and,
W_P: Weight of the wall tributary to the anchor.
F_p shall not be taken as less than the larger of $0.2k_aIW_p$ and 0.24 kN/m^2 times the area of the wall tributary to the anchor, but k_a need not be taken as larger than 2.0, and need not be taken as larger than 1.0 when the connection is not at a flexible diaphragm.
Where the anchorage is not located at the roof and all diaphragms are not flexible, the value from Eq. (4.25) is permitted to be multiplied by the factor $(1+2z/h)/3$, where z is the height of the anchor above the base of the structure, and h is the height of the roof above the base; however, F_p shall not be less than required by Section 4.14.1 with a minimum anchorage force of $F_p =0.2W_p$ but not less than 0.24 kN/m^2 times the area of the wall tributary to the anchor.
Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.2 m. Interconnection of structural wall elements and connections to supporting framing systems shall have sufficient strength, rotational capacity, and ductility to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

4.15. Drift and Deformation

The design drift ratio at level x, Δ_x/h_{sx}, as determined in Sections 4.10.8 and 4.11.8, shall not exceed the allowable value as obtained from Table 4.10 for any story. h_{sx} is the story height below level x.

Exception 1: For seismic force-resisting systems solely comprising moment frames, the design story drift, Δ, shall not exceed Δ_a/ρ for any story. ρ shall be determined in accordance with Section 4.7.

Exception 2: There shall be no drift limit for single-story structures in which the interior walls, partitions, and ceilings have been designed to accommodate story drifts associated with the design displacement. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are constructed such that moment transfer between shear walls (or, coupling) is negligible.

Table 4.10. Allowable Drift ratio, Δ_a/h_{sx}

Structure	**Risk Category**		
	I, II	**III**	**IV**
Structures, other than masonry shear wall structures, four stories or less above the base with interior walls, partitions, and ceilings that have been designed to accommodate the drifts associated with the design displacements	0.015	0.020	0.025
Masonry cantilever shear wall structures	0.010	0.010	0.010
Other masonry shear wall structures	0.007	0.007	0.007
All other structures	0.010	0.015	0.020

4.15.1. Additional Requirements for Computing Displacement and Drift

For determining displacements and drifts, it is permitted to determine the elastic displacements, δ_e, using seismic design forces based on the computed fundamental period of the structure without the upper limit, C_uT_a, specified in Section 4.10.3. Equation 4.3 need not be considered for computing drift.

Exception 1: For structures that have torsional horizontal irregularity, the design story drift shall be computed as the largest difference of the design displacements of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure, including the effects of diaphragm rotation.

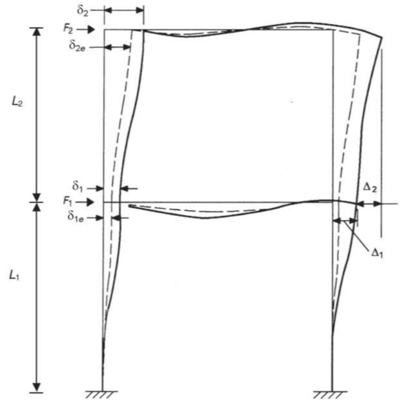


Figure 4.4. Story drift determination

4.15.2. Structural Separation

Adjacent structures on the same property shall be separated by at least δ_{MT}, determined by Eq. 4.27.

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \qquad 4.27$$

Separations shall allow for the design displacements, δ_M, as determined in accordance with Eq. 4.28.

$$\delta_M = \frac{C_d \delta'_{max}}{I} \qquad 4.28$$

where:

δ'_{max}: Maximum elastic displacement computed under design earthquake forces, including the effects of accidental torsion and torsional amplification as applicable

The structure shall be set back from the property line by at least the displacement δ_M of that structure.

4.16. Reduction of Foundation Overturning

Overturning effects at the soil–foundation interface are permitted to be reduced by 25% for foundations of structures that satisfy both of the following conditions:

(a) The structure is designed in accordance with the equivalent lateral force analysis as set forth in Section 4.10,

(b) The structure is not an inverted pendulum or cantilevered column type system.

4.17. Base Level

Base level is the level at which the horizontal seismic ground motions are considered to be imparted to the structure, with the following considerations:

a) For typical buildings on level sites without a basement where vertical elements are supported at various elevations on the top of footings, pile caps, and perimeter foundation walls, the base is generally established as the lowest elevation of the tops of elements supporting the vertical elements of the seismic force-resisting system.
b) For a building with a basement located on a level site, it is often appropriate to locate the base at the floor closest to grade.
c) For a floor level above grade to be considered the base, it generally should not be above grade more than one-half the height of the basement story.
d) Principles for the two-stage equivalent analysis must be satisfied.
e) For the base to be located at a floor level above grade, stiff foundation walls on all sides of the building should extend to the underside of the elevated level considered the base.

4.17.1. Elements that are not part of the lateral resisting system

All the elements that are not part of the lateral resisting system of the structure, should be designed in such a way that can accommodate the lateral displacements of the structure.

Chapter 5
Earthquake Geotechnical Hazards

5.1. Introduction

The site geotechnical hazards during an earthquake include surface faulting, liquefaction, and landslides, where their investigation, analysis approach, and ways to reduce their effects have been discussed in Sections 5.2 to 5.4. These risks should be considered in relation to the desired hazard level. Due to the reliance of the geotechnical engineering profession on numerous laboratory and field tests as well as significant changes in soil properties, in this chapter, more reliance is placed on engineering judgment and interpretation of observations and test results. On the other hand, engineering experience assumes a more fundamental role. Therefore, it is necessary for the contents of each section from 5.2 to 5.4 to be used by a geotechnical expert familiar with earthquake engineering issues and experienced in each section.

Topics related to geological engineering studies and geotechnical investigation, such as engineering geological maps, soil layering, evaluating the engineering characteristics, bedrock level, underground water conditions and other site specifications have not been mentioned in this regulation. In these topics, it is necessary to refer to authoritative sources such as the 7th clause of the Iranian National Building Regulations. One of the important things that should be taken into account in the geotechnical investigation phase is that an earthquake may cause surface earth settlement in limited or large areas. This settlement can be due to compaction in loose granular soils or land extracted from sea. Also, an earthquake may cause the collapse of underground spaces such as mines, tunnels, subterranean canals, caves and karst holes. As a result, a significant settlement will occur in a large area. Using reliable references, the aforementioned settlements should be evaluated based on the information obtained in the geotechnical investigation stage of the land and related to the desired hazard level.

5.1.1. Symbols

The symbols used in this chapter are listed below along with their definitions in alphabetical order.

$CRR_{7.5}$: Cyclic shear stress ratio of soil

CSR : Cyclic shear stress ratio of soil caused by earthquake

FS_L	:	liquefaction safety factor
F_{PGA}	:	Site coefficient for peak ground acceleration (PGA)
g	:	Earth's gravitational acceleration
K_σ	:	Correction factor for overburden pressure
MSF	:	Magnitude scale coefficient
M_w	:	Earthquake magnitude
$N_{1,60}$	:	Modified SPT number for overburden
PGA	:	Maximum acceleration of bedrock
$PGA_{surface}$	:	Maximum acceleration of the ground surface
q_{c1N}	:	CPT normalized tip resistance
r_d	:	Depth reduction factor
V_S	:	Shear wave velocity of the soil layer
V_{s1}	:	Normalized shear wave velocity of the soil layer
σ_{v0}	:	Total stress in the middle of the soil layer
$\sigma_{v0}^{'}$	:	Effective stress in the middle of the soil layer
τ_{ave}	:	Equivalent average stress due to earthquake
$\tau_{\max}$	:	Maximum shear stress of the soil layer

5.2. Surface fault rupture

5.2.1. Introduction

In previous earthquakes, in some cases, faulting has led to very large displacements on the earth's surface and has caused damage to engineering structures exposed to faulting. In this section, investigation of this hazard event isperformed in the form of an analysis approach and engineering measures to reduce the hazard.

5.2.2. Analysis approach

Evaluation of the potential of fault rupture hazard in a construction site should be done in the framework of the following general steps:

1. Considerations related to the type and importance of the structure:
   - The structures are classified into types of concentrated and linear structures. For centralized structures with seismic design categories D_1, D_2, and D_3,according to Chapter 4, it is necessary to prepare a report of geotechnical studies including all hazards of permanent ground movement, including fault rupture, by the relevant expert. Structures of seismic design groups D_1 and D_2, except for pipelines, should not be built in a place where a known active fault , defined in Chapter 3, causes the ground surface to break at the structure's location. If there are problems for locating such structures and there is no other option, a qualified expert team including seismic, geotechnical and structural consultants with expertise related to the design of structures should approve the design process.
   - For designing the tanks of the use and hazard group I and II in near-fault areas, conducting a special study for the effect of fault rupture is necessary. For buried pipelines, it is recommended for the pipe route to be selected in such a way that it does not pass through an active fault where possible. If the pipe route is unavoidable through an active fault, the pipe will collapse due to the design permanent displacement of the faultIn such a case, value of the fault displacement and direction of the slide relative to the pipeline route shall be determined. Moreover, the pipelines should be laid in relatively straight routes and extreme changes in the longitudinal and elevation direction of the pipeline should be avoided. As much as possible, pipelines should be built without bends, elbows and flanges that connect the pipeline to the ground.
2. Estimating the displacement associated with the fault rupture at the bedrock, according to Chapter 3
3. Calculation of displacement in the alluvium layers in the free-field condition and in the presence of footing with suitable methods including validated numerical models or if possible using physical modeling

4. Determining the safe range of setback construction according to the exact position of the fault in its depth and slope, the characteristics of alluvium and considering its uncertainties and its effect on the path of the fault, as well as the effect of the stiffness of the foundation and its overburden on the propagation path of the surface rupture.

5.2.3. Engineering measures to reduce the effects of fault rupture

Since it is not possible to prevent the faulting phenomenon at the foundation, it is necessary to reduce the resulting hazards. These hazards should be investigated separately for two categories of existing and new buildings. In this regard, the following points are important:

- Selecting the type of foundation and paying attention to the depth of the fault, asavoiding to choose deep foundations or surface foundations, per case.
- Selection of suitable structural systems to adapt to the expected displacements and forces.
- Using geotechnical measures, including the deviation of the fault path, or spreading the shear band, or minimizing the hazards of surface faulting.

Regarding the pipes, using special connections with high rotation capability, lightening the pipe and converting the local failure into a beam failure in which the pipe function does not stop.

5.3. Liquefaction

Soil liquefaction is a phenomenon in which saturated cohesionless soil loses a substantial amount of strength due to pore pressure generation resulting from earthquake strong ground shaking. Liquefaction can lead to reduction of foundation-bearing capacity, general or non-uniform settlement, lateral spreading, and flow failure. This phenomenon is investigated in three steps including (1) the requirements of liquefaction assessment, (2) the evaluation of liquefaction potential, and (3) liquefaction effects. If the design criteria of structures, pipelines, and geo-structures are not provided, the destructive effects of liquefaction should be controlled by using soil improvement methods.

5.3.1. Seismic level

Evaluation of liquefaction potential is not required for peak ground surface accelerations, $PGA_{surface}$, less than 0.1g.

Note: Where loose to very loose saturated sands are within the subsurface soil profile and PGA is less than 0.1g, evaluation of the liquefaction potential according to Section 5.3.2 should also be considered if conditions of liquefaction susceptibility mentioned in Section 5.3.1.2 are met. The soil density can be estimated based on valid available correlations with field tests.

5.3.1.2. Liquefaction susceptibility

The liquefaction assessment is required unless one of the following conditions is met:

Ground Water Level: The past, current, and future groundwater level is more than 25m from the existing ground surface or project base level, whichever is deeper.

Depth: Depth of the soil layer is more than 25m from the existing ground surface or project level, whichever is deeper.

Soil Characteristics: If one of the following conditions is met for the soil layers, it is not required for further investigation to evaluate liquefaction:

- In soils containing less than 5% of fine content, if the SPT test is performed, the corrected SPT value corrected for overburden pressure, should be more than 30 ($N_{1,60} > 30$)or if the CPT test is performed, the dimensionless normalized tip resistance should be more than 160 ($q_{c1N} > 160$).

Note: For gravels or gravel-containing soils, the values of SPT and CPT tests should preferably not be used, or if used, the values should be modified.

- The normalized shear wave velocity of each layer should be more than 250 m/s.
- In case of encountering fine-content soils and if the liquefaction potential is not present based on the latest valid criteria for the liquefaction of fine-content soils.

5.3.2. Liquefaction potential assessment

The liquefaction potential of the suspicious soil layers is evaluated using the simplified shear stress method with deterministic and probabilistic approaches. For this purpose, it is necessary to investigate the liquefaction potential of soil layers up to a depth of 25 meters from the ground surface. The peak ground surface acceleration, $PGA_{sureface}$, and the average magnitude, M_w, are used in the simplified shear.

Note: In order to perform the site response analysis for estimations of the peak ground surface acceleration or for stress-strain numerical modeling, to be used for evaluation of the liquefaction effects according to Section 5.3.3, it is necessary to employ 7 pairs of acceleration records, as selected per Chapter 3, for analysis.

The liquefaction potential is calculated by the simplified shear stress method using the liquefaction safety factor, FS_L, which is obtained by Eq. 5.1:

$$FS_L = (\frac{CRR_{7.5}}{CSR})MSF.K_\sigma \qquad 5.1$$

The cyclic stress ratio (CSR) is estimated by:

$$CSR = \frac{\tau_{ave}}{\sigma'_{v0}} = 0.65\frac{PGA}{g}\frac{\sigma_{v0}}{\sigma'_{v0}}r_d \qquad 5.2$$

where $\frac{\tau_{ave}}{\sigma'_{v0}}$ is the representative earthquake induced cyclic shearing stress divided by the initial (i.e. pre-earthquake) effective overburden stress, PGA is the peak ground surface acceleration in units of g, $\frac{\sigma_{v0}}{\sigma'_{v0}}$ is the ratio of the initial total overburden stress to the initial effective overburden stress, and r_d is the soil flexibility factor.

Note: If numerical analyses are carried out for the site response analysis, the values of τ_{ave} in the middle of each layer are calculated using Eq. 5.3:

$$\tau_{ave} = 0.65\tau_{max} \qquad 5.3$$

where τ_{max} is the maximum shear stress obtained from the dynamic shear stress history in the middle of each layer.

A magnitude scaling factor (MSF) should be used for correcting for magnitudes different from 7.5, according to valid references.

The $\boldsymbol{K_\sigma}$factor may be calculated using a valid reference.

For determining the cyclic resistance ratio, $CRR_{7.5}$, simplified methods can be usedbased on the results of field tests including standard penetration test (SPT), cone penetration test (CPT) and shear wave velocity (V_s) test.

5.3.2.1. Deterministic method

In the deterministic approach, the decision criterion is based on the liquefaction safety factor, according to Section 5.3.2. The liquefaction safety factor is calculated for each soil layer , separately. Occurrence of liquefaction is certain for $FS_L<1$ and the residual resistance of soil should be considered for calculations of liquefaction effects. Generation of pore water pressure is likely for $1<FS_L<1.5$ and the effect of excess pore water pressure should be considered on buildings, according to Section 5.3.3, and on foundation bearing capacity. Assessment of liquefaction effects is not required when $FS_L<1.5$.

5.3.2.1. Probabilistic method

The probabilistic methods can provide a more appropriate judgment of liquefaction conditions compared with the deterministic methods in some cases because of the uncertainties of soil parameters, available empirical relationships as well as seismic parameters. Probability of liquefaction, PL can be calculated using probability equations presented in authoritative references.

Note: In important structures or in situations where it is difficult to make a decision regarding the occurrence of liquefaction and its effects, it is recommended to use more accurate and complementary methods of liquefaction assessment such as cyclic tests and physical modeling.

5.3.3. Liquefaction effects

Variation of pore water pressure during liquefaction can lead to settlement, lateral spreading, flow failure and reduction of foundation bearing capacity. Each of these effects should be studied separately using numerical, analytical or semi-analytical methods on liquefiable sites, based on Section 5.3.2, and its results should be considered in the relevant calculations.

Note: In case the construction of a highly important structures on a liquefiable soil is unavoidable, it is necessary to use additional studies such as geotechnical tests and numerical simulations by advanced numerical models, which are based on appropriate models for the cyclic behavior of soils.Effects of liquefaction on the structure should be more carefully investigated and a decision should be made accordingly.

5.3.3.1. Ground surface settlement

The liquefaction process can lead to ground settlements with its amount being in many cases more than the allowable value. The liquefaction-induced settlement can be calculated using the volumetric strain methods for free-field sites. If the structure is placed on such a ground, value of the settlement should be estimated by combining the volumetric settlement caused by the overburden and the shear settlement caused by shear stresses from the building. This settlement can be estimated based on empirical models and numerical methods.

5.3.3.2. Foundation bearing capacity

Soil resistance decreasing during liquefaction can cause a significant displacement and as a result, failures can be resulted from a decrease in the bearing capacity. For this reason, it is not recommended to use single or strip foundations on liquefiable soils, unless the soil is improved or the load of the structure is transferred to deeper layers.

Note: In the design of shallow and deep foundations, the shear resistance reduction coefficients due to liquefaction should be considered.

5.3.3.3. Lateral spreading and flow failure

Liquefaction can lead to lateral spreading or flow failure in sloping or free-field grounds. It is necessary to consider both of these phenomena for the design of coastal walls, caisson quay walls, and other cases. Lateral spreading and flow failure can be calculated using experimental, analytical, or numerical methods such as the sliding block method and the empirical methods. The pseudo-static methods can be used by consideration of soil resistance reduction coefficients accounting for the pore water pressure buildup, or by numerical procedures.

Note: It is necessary to calculate the reduced soil resistance by the available methods for estimation of seismic sliding displacement (e.g., sliding block methods).

5.3.3.4. Soil improvement methods

It is necessary to prevent the liquefaction effects on structures by soil improvement methods in liquefiable soils. Improvement methods include methods that prevent soil liquefaction or transfer the loads of the superstructure to non-liquefiable layers (without soil improvement). Each of these methods can be selected based on the conditions of the construction project.

5.4. Landslide

5.4.1. Introduction

A landslide is defined as the down-slope mass movement of earth resulting from any cause. Landslides and slope instability have occurred in many large earthquakes. Depending on the geometry of the earth, the geotechnical properties of the soil and the characteristics of the earthquake, this phenomenon may occur in different shapes and dimensions in an area.

In this section, landslide hazard assessment requirements at a site will be described in two parts: analysis approach and methods of mitigation of the hazard. All contents of this section shall be used with the approval of a geotechnical engineer familiar with earthquake engineering issues and the term "expert" in the landslide section refers to such an expert. In addition, it should be noted that the estimation of the risk of slipping by an "expert" is based on experience and engineering judgment, and for this reason, in this section, they are presented in a qualitative format.

5.4.2. Analysis Approach

Landslide risk susceptibility and potential assessment at a site is performed in the following general steps:

1- The first stage of assessment begins with small-scale studies. First, the historical earthquakes, existing information, geological, geomorphological and zonation maps, if available, and also the seismicity of the area are studied. Then, on a larger scale, air photos are investigated and site visits are performed. Finally, if the possibility of a landslide is detected by the expert, the instability is evaluated and analyzed based on the data of geotechnical studies and according to what will follow.

2- If the landslide hazard assessment at the site requires large-scale investigation, slope stability analysis based on geotechnical studies is performed. In this case, the subsurface soil layering information must be carefully specified so that the slope is modeled with correct input data. Determining the subsurface layering as accurately as possible is one of the most important requirements for correctly assessing the landslide potential and correctly estimating the resulting displacement. Subsoil layers are determined using geological information, field and laboratory tests on samples taken from boreholes, geophysical tests, and the use of other field evidence.

3- Pseudo-static analyses with limit state methods shall be used to determine slope stability if the soils are not susceptible to liquefaction or otherwise expected to lose shear strength during deformation. If the slope safety factor is less than one, the slope displacement is determined based on a valid sliding block method and performance of the slope and structures on it are evaluated based on the calculated displacement. Judgments about permissible permanent displacement and acceptable performance of the slope depend on the sensitivity and importance of structures and equipment installed or designed and is made by the expert.

4- If soils are susceptible to liquefaction or otherwise expected to lose shear strength during deformation, dynamic analysis based on effective stress shall be performed to determine slope stability. The phenomenon of liquefaction often occurs as a reduction in bearing capacity, total or differential settlement, lateral expansion and flow rupture. Evaluation of this phenomenon is carried out in three stages, including the need to investigate the potential and effects of liquefaction, and if necessary, its destructive effects on structures and pipelines should be controlled with improvement methods. Assessment of liquefaction potential is discussed in Section 3.5.

5- In rock slopes, in addition to the above, it is necessary to determine the joints system at the site to assess the possibility of sliding and rockfall.
6- In checking the instability of a slope that is under the load caused by a structure such as facilities or pipes, all instability modes should be checked. These modes include slope instability and instability caused by insufficient bearing capacity of the foundation of the structure located on the slope. In the latter case, according to the type of loading of the slope by the structure, the slope and the structure are modeled in two-dimensional or three-dimensional form according to the expert's judgment.

5.4.3. Mitigation of the hazard

Mitigation of landslide hazard shall be accomplished through modification of the structure, foundation, soil conditions, or other by other approved approaches.

In order to construct important structures or install important and sensitive equipments on top or downstream of slopes and in case of insufficient geotechnical information, it is necessary that the parameters such as displacement of different points of the slope and changes in water pressure be monitored.

Chapter 6
Soil-Structure Interaction

6.1. Introduction

In this chapter, the impact of the soil-structure interaction phenomenon in determining the seismic demand of the target structure, which is generally defined for systems with flexible foundations, is considered through three main seismic analysis methods.

First, in Section 6.2.1, regulations and limitations are accompanied with the equivalent static analysis method of the structure, using analysis of the spring-dashpot model that replaces soil beneath the foundation, and contributes to the reduction of the base shear. The base shear is mainly influenced by the first mode of vibration. The requirements of this method, are presented based on the equivalent damping and period of the soil-structure system.

Second, in Section 6.2.2, the linear dynamic analysis including the modified response spectrum or the site-specific spectrum corresponding to the soil-structure system is presented. In the case of using the site-specific spectrum, application of analytical relations to consider the radiation damping of the foundation described in Section 6.3 is allowed.

Third, the nonlinear dynamic analysis method, which requires a time domain soil-structure model, is considered in Section 6.2.3. Along with this method, applying the effects of embedment and the incoherence wave field impinging the foundation in the form of modification of the input excitation through the modified site-specific spectrum (corresponding to the soil-structure system) according to section 4.6 and also scaling the acceleration time histories are allowed.

Also, the considerations and limitations of accounting for the effects of the fluid-soil-structure phenomenon, which appears abundantly in fluid storage tanks, are presented in Section 5.6.

6.1.1. Scope

Under the requirements of this section, it is allowed to apply the effects of the soil-structure interaction phenomenon for estimation of seismic demands. For the purpose of this Regulations, both upper bound (by applying a factor of 2) and lower bound (by applying a factor of 0.5) estimates for the soil and foundation stiffness shall be considered. The case that results in a smaller

reduction or greater amplification in response parameters, shall be considered as the design state.
In general, the instructions of this section can be accompanied with all three analysis methods, including the equivalent lateral force method, the linear dynamic method, and the nonlinear dynamic method under the following considerations:

- Effect of the wave passage phenomenon following the embedment effect for embedded foundations and the effects of incoherency of the wave field shall not be taken into account when using the equivalent lateral force method and the linear dynamic analysis method.
- The nonlinear dynamic method using time history analysis can be associated with the effects of the soil-structure interaction phenomenon according to Section 6.2.3 only for structures located on site classes weaker than type II.

Modeling the effect of soil-structure interaction is required for building structures if the value of $(\frac{h}{v_s T})$ is less than 0.1. In this regard, v_s is the average shear wave velocity of site; T is the fundamental period of the structure assuming a fixed base in and h is the effective height of the structure, which is calculated as the height of the center of mass of the structure from the foundation level. Height of the center of structure's mass from the foundation level can be considered equal to the height of the structure in one-story structures, and for other structures, it can be considered as two-thirds of the total height of the structure from the foundation level.

6.1.2. Definitions

The following definitions apply to the provisions of this Chapter.

Base slab averaging: A part of the kinematic interaction that changes the foundation input motion due to incoherency of the wave field over the base area.

Foundation input motion: The changed motion of the free field due to the kinematic interaction, with the total response of the structure and foundation is determined under its effect.

Free-field motion: The excitation experienced by the site in an intact state, that is, before any earthworks or the presence of the structure and foundation mass, and even without applying the rigidity of the structure and foundation.

Inertial SSI: The dynamic interaction between the structure-foundation and its surrounding soil, which is investigated under the effect of applying the foundation input motion.

Kinematic SSI: The influence of foundation rigidity and geometry on the free-field motion and transferring it into input motion to the foundation.

Radiation damping: The damping produced within the soil-structure system due to the propagation of waves towards the half-space caused by the different vibration of the foundation compared to the free-field motion.

Soil damping: The hysteretic (material) damping of the soil.

6.1.3. Symbols

The signs and abbreviations used in the relationships and sections of this chapter are listed in alphabetical order in the following:

a_0	:	Dimensionless frequency of the soil-structure system
B	:	Half of the smaller dimension of the foundation plan.
B_{SSI}	:	Modification factor of the design earthquake response spectrum, the response spectrum of maximum considered earthquakes or a site-specific spectrum for damping ratio other than 5% (Eq. 6.2.3).
b_e	:	The effective size of the foundation (Eq. 6.40).
$\tilde{C}_s$	:	Coefficient of the seismic response assuming the flexibility of the underlying medium at the foundation-soil interface (Section. 6.2.1).
D_s	:	Thickness of a soft (flexible) soil layer located on a stiff layer.
e	:	Embedment depth.
G_{rd}	:	The effective shear modulus used in determining the radiation damping approximated based on ($\boldsymbol{G_{ord}}$) defined in Table 6.2.
$G_{0,rd}$	:	Average shear modulus of soil used in determining the radiation damping over a depth of B or r_f below the base of the foundation at the small strains according to Eq. 6.1.
h^*	:	Effective height of the structure.

K_y , K_r	:	Translational foundation stiffness.
K_{xx} , K_{rr}	:	Rotational foundation stiffness.
L	:	Half of the larger dimension of the foundation plan.
M^*	:	Effective modal mass of the structure in the fundamental mode of vibration in the direction under consideration.
R_u	:	Response modification factor (Table 4.5).
RRS_{bsa}	:	Site-specific response spectral modification factor for base-slab averaging (Eq. 6.37).
RRS_e	:	Site-specific response spectral modification factor for foundation embedment (Eq. 6.37).
r_f	:	Radius of a circular foundation.
$\tilde{S}_a$	:	Response spectral acceleration including the effects of soil-structure interaction phenomenon (Eq. 6.6).
T	:	Fundamental period of the structure without considering the degrees of freedom for the soil-foundation interface (i.e., the fixed base case) calculated using the relationships of chapter 4. In this case, the empirical value of the structural period, T, as well as the limit of the maximum value of $C_{Tu}T$ shall not be applied.
$\tilde{T}$	:	Fundamental period of the structure with considering the degrees of freedom for the soil-foundation interface (i.e., the flexible base case) calculated in accordance with Section 6.1.1. In this case, the empirical value of the structural period, T, as well as the limit of the maximum value of $C_{Tu}T$ shall not be applied.
$(\tilde{T}/T)_{eff}$	:	Effective period lengthening that depends on the expected ductility demand, μ (Eq. 6.8).
T_y , T_r	:	Fundamental translational period of the SSI system (Eqs. 6.19 and 6.29).
T_{xx} , T_{rr}	:	Fundamental rotational period of the SSI system (Eqs. 6.20 and 6.30).
$\tilde{V}$	:	Base shear corresponding to the soil–structure system
V	:	Base shear corresponding to the fixed based structure.
$V_{:}\tilde{V}_i$	:	Base shear corresponding to the soil-structure system determined through modal response spectrum analysis
$v_{s,rd}$	:	Average effective shear wave velocity used in determining the radiation damping over a depth of B or r_f below the level of the

foundation base according to $\boldsymbol{v_{so,rd}}$ and Table 6.1 or site-specific studies.

$v_{so,rd}$: Average low-strain shear wave velocity used in determining the radiation damping over a depth of B or r_f below the level of the foundation (Eq. 6.1)

$$v_{so,rd} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \tag{6.1}$$

where:

$\boldsymbol{d_i}$: Thickness of each layer down to a depth of 30 m.

$\boldsymbol{v_{si}}$: Shear wave velocity ($\frac{\mathrm{m}}{\mathrm{s}}$)

and $\sum_{i=1}^{n} d_i = 30 \quad m$

$v_{s,e}$: Average effective shear wave velocity corresponding to the site-soil conditions used in determining the embedment effects and is taken as average of the shear wave velocity over the embedment depth of the foundation ($\boldsymbol{v_{so,e}}$) according to Table 6.1 or site-specific studies, which shall not be taken less than 200 m/sec.

$v_{so,e}$: Average low-strain shear wave velocity used in determining the embedment effect over the embedment depth of the foundation (Eq. 6.1)

$\bar{W}$: Weight caused by the modal mass in the fundamental mode of vibration, or alternatively, the effective seismic weight according to Section 4.2.10.

α : Reduction factor for the base shear due to foundation damping within soil-structure system.

α_{xx} , α_{rr} : Dimensionless factor, function of dimensionless frequency (Eqs. 6.13 and 6.24)

β : The effective viscous damping ratio of the structureassumed to be 5% by default, unless another value is justified by reasonable analysis.

β_0 : Effective viscous damping ratio of the soil-structure system (Eq. 6.7).

β_f : Effective viscous damping ratio related to foundation-soil interaction (Eq. 6.9).

β_{rd} , β_{rd}' : Effective radiation damping ratios related to foundation-soil interaction (Eqs. 6.21, 6.31 and 6.5).

β_s	:	Effective hysteretic damping ratio in accordance with Section 6.3.5.
β_y , β_r	:	Translational foundation damping coefficients (Eqs. 6.17 and 6.27).
β_{xx} و β_{rr}	:	Rotational foundation damping coefficients (Eqs. 6.18 and 6.28).
γ	:	Average unit weight of the soil over a depth of B below the foundation.
μ	:	Expected ductility demand.
ν	:	Poisson's ratio, assumed to be 0.3 for sand and 0.45 for clay.
Ψ	:	Dimensionless factor; function of the Poisson's ratio (Eqs. 6.12 and 6.23).

6.2. Seismic Demand in Soil-Structure Systems

6.2.1. Equivalent lateral force analysis

To account for the effects of soil-structure interaction, it should be noted that inclusion of kinematic interaction effects with the equivalent lateral force method is not permitted. In this method, the base shear can be reduced according to the following relationship:

$$\tilde{V} = V - \Delta V \qquad 6.2$$

$$\Delta V = \left[c_s - \frac{\tilde{c}_s}{B_{SSI}}\right]\bar{\mathrm{W}} \leq 0.3V \qquad 6.3$$

where:

$$B_{SSI} = \frac{4}{[5.6 - \ln(100\beta_0)]} \leq \begin{cases} 1.4 & R_u \leq 3 \\ 1.7 - R_u/10 \quad for & 3 < R_u < 6 \\ 1.1 & R_u \geq 6 \end{cases} \qquad 6.4$$

in which:

$\tilde{V}$: Base shear of the structure in the soil-structure system.

V: Base shear of the fixed base structure.

R_u: Response modification factor (Table 4.5).

c_s: Seismic response coefficient in accordance with Section 4.10.2, assuming no soil flexibility, equal to $(\frac{S_a}{R_u.I})$

$\tilde{c}_s$: Seismic response coefficient determined in accordance with Section 4.2.10, assuming flexibility of the structural base at the foundation–soil interface in accordance with Section 6.1.1, using $\tilde{T}$ as the fundamental period of the structure in lieu of the period T.
$\overline{\mathrm{W}}$: Weight attributed to the modal mass in the fundamental mode of vibration, or alternatively, the effective seismic weight according to Section 4.2.10.
β_0: Effective viscous damping ratio of the soil-structure system (Eq. 6.7).

6.2.2. Linear dynamic analysis

Consideration of kinematic interaction effects as presented in Section 6.5 or other references is not permitted for the linear dynamic analysis method. To account for the effects of SSI, a linear dynamic analysis is permitted to be performed in accordance with Sections 4.11 and 4.12, using both the design response spectrum (second level earthquake) and response spectrum of third level earthquake (MCE$_R$) or the modified site-specific response spectrum in accordance with the provisions of the soil-structure system and Sections 6.2.2.1 and 6.2.2.2. The resulting response spectral acceleration shall be divided by $(R_u.I)$, where I is prescribed in Section 4.3 and presented in Table 4.3. It is also obvious that the mathematical model used for linear dynamic analysis shall include the flexibility of the foundation and the soil beneath it according to Section 6.1.1.
Scaling of lateral forces within the modal analysis shall be done according to Section 4.12.2 including replacing the base shear V with $\tilde{V}$ and the modal base shear V_t with $\tilde{V}_t$ such that the total base shear for the soil-structure system is not less than %70 of the base shear calculated for the structure with a fixed base, i.e.:

$$\tilde{\mathbf{V}}_{\mathbf{t}} \geq \mathbf{0.7V} \qquad \mathbf{6.5}$$

6.2.2.1. Modified design response spectrum

The design response spectrum that includes the effects of soil-structure interaction for using in the modal analysis method shall be developed according to the following equations:

$$\begin{cases} \tilde{S}_a = \left[\left(\frac{5}{B_{SSI}} - 2\right) \times \frac{T}{T_S} + 0.4\right] \times S_{DS} & 0 < T < T_0 \\ \tilde{S}_a = S_{DS}/B_{SSI} \qquad \qquad for & T_0 \leq T < T_s \\ \tilde{S}_a = S_{D1}/(B_{SSI}T) & T_s < T \leq T_L \\ \tilde{S}_a = S_{D1}T_L/(B_{SSI}T^2) & T_L < T \end{cases} \qquad 6.6$$

where S_{D1}, S_{DS}, T_S, T_0 and T_L are determined according to Section 3.8.

6.2.2.2. Modified site-specific response spectrum

The site-specific response spectrum including the effects of soil-structure interaction may be developed according to the requirements of Chapter 3 by modifying for the effective viscous damping ratio, β_0 of the soil-structure system as defined in Section 6.3.

6.2.3. Nonlinear time history analysis

Consideration of interaction effects within a nonlinear time history analysis in accordance with Section 4.12.3 shall be permitted provided that scaled acceleration time histories are used. The acceleration time histories shall be scaled to the site-specific response spectrum modified for kinematic interaction effects according to Section 6.5.The used mathematical model shall include consideration of the foundation and soil flexibility according to Section 6.1.1 as well as the foundation damping explicitly according to section 6.3. It is also allowed to consider the effects of kinematic interaction including the requirements of Section 6.5 in order to determine the site-specific response spectrum.

The site-specific response spectrum shall be developed in accordance with the requirements of Chapter 3 of this regulation in addition to the following additional requirements:

- The spectrum is permitted to be adjusted for kinematic interaction effects by multiplying the spectral acceleration values for each period by the corresponding response spectrum ratios for embedment (RRS_e) and base slab averaging (RRS_{bsa}) according to Section 6.5. Moreover, the above effects can be directly incorporated into development of the site-specific response spectrum.

- In case of embedded structures in the ground, it is allowed to develop a site-specific response spectrum at the level of the foundation embedment depth instead of the ground level. In this case, the value of (RRS_e) shall be taken as 1.0.
- The modified site-specific response spectrum, which includes the effects of the kinematic interaction, shall not be less than 80% of the S_a values at any period corresponding to the site-specific response spectrum according to Section 3.8.
- The site-specific response spectrum modified for the kinematic interaction effects, shall not be less than 70% of the S_a values at any period corresponding to the design response spectrum and maximum considered response spectrum according to Section 3.8.

It is permitted to include the kinematic interaction effects within the equivalent lateral force or the linear dynamic procedures conducted per Section 4.12.2 using the site-specific response spectrum modified for the kinematic soil–structure interaction, subject to the limitations herein. Where the foundation damping is incorporated within a nonlinear model, procedures of the equivalent lateral force and the linear dynamic methods, in conjunction with Section 4.12.2, shall be based on the provisions of Sections 6.2.1 and 6.2.2, respectively.

6.3. Foundation Damping Effects

6.3.1. Requirements for determining the foundation damping

It is permitted to include foundation damping in the mathematical model of the structure by using the hysteretic and radiation dampings. The methods presented in this section are permitted to be used in conjunction with applying the modifications associated with the equivalent lateral force (Section 6.2.1) and linear dynamic methods (Section 6.2.2) unless one of the following conditions exists:

1. A foundation system consisting of discrete footings that are not interconnected and are spaced less than the larger dimension of the supported lateral force-resisting element in the direction under consideration.

2. A foundation system consisting of, or including, deep foundations such as piles or piers.
3. A foundation system consisting of structural mats interconnected by concrete slabs that are characterized as flexible in accordance with Section 4.13 or are not continuously connected to grade beams or other foundation elements.

6.3.2. Effective damping

The foundation damping effects shall be represented by providing the effective damping ratio, β_0, which is calculated according to Eq. 6.7:

$$\boldsymbol{\beta_0 = \beta_f + \frac{\beta}{\left(\tilde{T}/T\right)^2} \leq 0.20} \qquad 6.7$$

where:

β_f: Effective viscous damping ratio due to soil-foundation interaction;

β: The effective viscous damping ratio of the structure, which may be taken 5% unless other values are recommended.

$\left(\tilde{T}/T\right)_{\mathrm{eff}}$: The effective period lengthening ratio which is calculated from Eq. 6.8:

$$\boldsymbol{\left(\frac{\tilde{T}}{T}\right)_{\mathrm{eff}} = \left\{1 + \frac{1}{\mu}\left[\left(\frac{\tilde{T}}{T}\right)^2 - 1\right]\right\}^{0.5}} \qquad 6.8$$

where μ is the expected ductility demand. For the methods of equivalent lateral force and modal response spectral analysis, value of μ is equal to the result of dividing the maximum base shear by the capacity of elastic base shear; alternatively, it is permitted to be taken as the result of dividing the response modification factor, R_u by the overstrength factor, Ω_0 per Table 4.5. For the time-history analysis method, the ductility demand is calculated by dividing the maximum displacement by the yield displacement of the structure at the highest level of the structure above the ground.

Foundation damping, β_f is caused by hysteretic and radiation dampings and is determined using Eq. 6.9:

$$\beta_f = \left[\frac{(\tilde{T}/T)^2 - 1}{(\tilde{T}/T)^2}\right]\beta_s + \beta_{rd} \qquad 6.9$$

where β_s is the soil hysteretic damping ratio determined in accordance with Section 6.3.5, and β_{rd} is the radiation damping ratio calculated per Sections 6.3.3 or 6.3.4.

Note: If a site consists of a soft stratum of thickness D_s underlined by a layer with a shear wave velocity at least twice that of the surface layer and $\frac{4D_S}{V_S\tilde{T}} < 1$, then the damping ratio β_{rd} should be replaced by $\beta_{rd}{}'$, calculated as follows:

$$\beta_{rd}{}' = \left(\frac{4D_S}{V_S\tilde{T}}\right)^2 . \beta_{rd} \qquad 6.10$$

6.3.3. Radiation damping for rectangular foundations

Radiation damping effects for structures with a rectangular foundation shall be represented by the effective damping ratio of the soil-structure system, β_{rd}, determined in accordance with Eq. 6.11:

$$\beta_{rd} = \frac{1}{(\tilde{T}/T_y)^2}\beta_y + \frac{1}{(\tilde{T}/T_{xx})^2}\beta_{xx} \qquad 6.11$$

$$\beta_y = \left[\frac{4(L/B)}{(K_y/G_{rd}B)}\right]\left[\frac{a_0}{2}\right] \qquad 6.12$$

$$\beta_{xx} = \left[\frac{(4\psi/3)(L/B)a_0^2}{\left(\frac{K_{xx}}{G_{rd}B^3}\right)\left[(2.2 - \frac{0.4}{(L/B)^3}) + a_0^2\right]}\right]\left[\frac{a_0}{2\alpha_{xx}}\right] \qquad 6.13$$

$$T_y = 2\pi\sqrt{\frac{M^*}{K_y}} \qquad 6.14$$

$$T_{xx} = 2\pi\sqrt{\frac{M^*(h^*)^2}{\alpha_{xx}.K_{xx}}} \qquad 6.15$$

$$a_0 = \frac{2\pi B}{\tilde{T} v_{s,rd}} \quad 6.16$$

$$\psi = \sqrt{\frac{2(1-\nu)}{(1-2\nu)}} \leq 2.5 \quad 6.17$$

$$\alpha_{xx} = 1.0 - \left[\frac{\left(0.55 + 0.01\sqrt{(L/B)-1}\right) a_0^2}{\left(2.4 - \frac{0.4}{(L/B)^3}\right) + a_0^2}\right] \quad 6.18$$

$$G_{0,rd} = \gamma {v_{so,rd}}^2/g \quad 6.19$$

$$K_y = \frac{G_{rd} B}{2-\nu}\left[6.8\left(\frac{L}{B}\right)^{0.65} + 0.8\left(\frac{L}{B}\right) + 1.6\right] \quad 6.20$$

$$K_{xx} = \frac{G_{rd} B^3}{1-\nu}\left[3.2\left(\frac{L}{B}\right) + 0.8\right] \quad 6.21$$

where:

M^*: Effective modal mass of the structure in the fundamental mode of vibration in the direction under consideration.

h^*: Effective height of the structure.

L: Half of the larger dimension of the foundation plan.

$v_{s,rd}$: Average effective shear wave velocity which is used in determining the radiation damping over a depth of B or r_f below the level of foundation of the structure according to $v_{so,rd}$ and Table 6.1 or site-specific studies.

$v_{so,rd}$: Average low-strain shear wave velocity used in determining the radiation damping over a depth of B or r_f below the level of the foundation (Eq. 6.1).

G_{rd}: The effective shear modulus used in determining the radiation damping approximated based on $G_{0,rd}$) according to Table 6.2.

$G_{0,rd}$: Average shear modulus of the soil used in determining the radiation damping over a depth of B or r_f below the level of foundation at small strains according to Eq. 6.1.

γ: Average unit weight of the soil over a depth of B below the base of the structure.

ν: Poisson's ratiothat may be assumed to be 0.3 for sand and 0.45 for clay.

Table 6.1. Effective Shear Wave Velocity Ratio ($\frac{v_{s,rd}}{v_{so,rd}}$ or $\frac{v_{s,e}}{v_{so,e}}$)

Effective Peak Acceleration $(S_{DS}/2.5)^{a,c}$				
$S_{DS}/2.5 \geq 0.8$	$S_{DS}/2.5 = 0.4$	$S_{DS}/2.5 = 0.1$	$S_{DS}/2.5 = 0$	***Site Class***
1.00	1.00	1.00	1.00	**I**
0.77	0.87	0.97	1.00	**II**
0.32	0.71	0.95	1.00	**III**
b	0.22	0.77	1.00	**IV**

Table 6.2. Effective Shear Modulus Ratio ($\frac{G_{rd}}{G_{0,rd}}$)

Effective Peak Acceleration $(S_{DS}/2.5)^{a,c}$				
$S_{DS}/2.5 \geq 0.8$	$S_{DS}/2.5 = 0.4$	$S_{DS}/2.5 = 0.1$	$S_{DS}/2.5 = 0$	***Site Class***
1.00	1.00	1.00	1.00	**I**
0.60	0.75	0.95	1.00	**II**
0.10	0.50	0.90	1.00	**III**
b	0.05	0.60	1.00	**IV**

[a] Use straight-line interpolation for intermediate values of $S_{DS}/2.5$.

[b] In this case and in the four situations mentioned in Note 8 of Chapter 3, site-specific geotechnical identification and dynamic analysis of the site response shall be performed.

[c] Use of this table for the response spectrum of the maximum considered earthquake is permitted according to Section 3.8, and in this case, $S_{DS}/2.5$ shall be replaced with $S_{MS}/2.5$. Use of this table for the site-specific response spectrum corresponding to the maximum considered earthquake is permitted according to Chapter 3. In this case, $S_{DS}/2.5$ should be replaced by the spectral acceleration of the maximum considered earthquake corresponding to the natural period of 0.01 seconds.

6.3.4. Radiation damping for circular foundations

Radiation damping effects for structures with a circular foundation plan shall be represented by the effective damping ratio of the soil–structure system, β_{rd} determined in accordance with Equation 6.22:

$$\beta_{rd} = \frac{1}{\left(\tilde{T}/T_r\right)^2}\beta_r + \frac{1}{\left(\tilde{T}/T_{rr}\right)^2}\beta_{rr} \quad 6.22$$

$$\beta_r = \left[\frac{\pi}{(K_r/G_{rd}r_f)}\right]\left[\frac{a_0}{2}\right] \tag{6.23}$$

$$\beta_{rr} = \left[\frac{(\pi\psi/4)a_0^2}{\left(\frac{K_{rr}}{G_{rd}{r_f}^3}\right)[2+a_0^2]}\right]\left[\frac{a_0}{2\alpha_{rr}}\right] \tag{6.24}$$

$$T_r = 2\pi\sqrt{\frac{M^*}{K_r}} \tag{6.25}$$

$$T_{rr} = 2\pi\sqrt{\frac{M^*(h^*)^2}{\alpha_{rr}.K_{rr}}} \tag{6.26}$$

$$a_0 = \frac{2\pi r_f}{\tilde{T}v_{s,rd}} \tag{6.27}$$

$$\psi = \sqrt{\frac{2(1-\nu)}{(1-2\nu)}} \le 2.5 \tag{6.28}$$

$$\alpha_{rr} = 1.0 - \left[\frac{0.35\,a_0^2}{1+a_0^2}\right] \tag{6.29}$$

$$K_r = \frac{8\,G_{rd}r_f}{2-\nu} \tag{6.30}$$

$$K_{rr} = \frac{8\,G_{rd}{r_f}^3}{3(1-\nu)} \tag{6.31}$$

where:

r_f: Radius of the foundation.

$v_{s,rd}$: Average effective shear wave velocity used in determining the radiation damping over a depth of B or r_f below the level of the foundation according to $v_{so,rd}$ and Table 6.1 or site-specific studies.

$v_{so,rd}$: Average low-strain shear wave velocity used in determining the radiation damping over a depth of B or r_f below the level of the foundation (Eq. 6.1).

γ: Average unit weight of the soil over a depth of B below the base of the structure.

G_{rd}: The effective shear modulus used in determining the radiation damping approximated based on $G_{o,rd}$ according to Table 6.2.

$G_{0,rd}$: Average shear modulus of the soil used in determining the radiation damping over a depth of B or r_f below the level of the foundation at small strains according to Eq. 6.1.

6.3.5. Soil damping

Effects of soil hysteretic damping shall be represented by the effective soil hysteretic damping ratio, β_s, determined based on a site-specific study or according to Table 6.3.3:

Table 6.3. Soil Hysteretic Damping Ratio (β_s)

Effective Peak Acceleration $(S_{DS}/2.5)^{a,c}$				
$S_{DS}/2.5 \geq 0.8$	$S_{DS}/2.5 = 0.4$	$S_{DS}/2.5 = 0.1$	$S_{DS}/2.5 = 0$	***Site Class***
0.05	0.03	0.01	0.01	**II**
0.15	0.07	0.02	0.01	**III**
[b]	0.20	0.05	0.01	**IV**

[a] Use straight-line interpolation for intermediate values of $S_{DS}/2.5$.

[b] In this case and in the four situations mentioned in Note 8 of Chapter 3, site-specific geotechnical identification and dynamic analysis of the site response shall be performed.

6.4. Flexible Foundation Effects

To model the foundation flexibility using equivalent springs, first the relative rigidity of the structure-foundation system compared to the bearing soil shall be determined according to Sections 6.4.1 and 6.4.2. Then, stiffness of the equivalent springs in each direction shall be calculated according to Section 6.4.3.

6.4.1. Determining the rigidity of single and mat foundations

Single and mat foundations are assumed to be rigid relative to the underlying soil if Eq. 6.32 is established:

$$4k_{sv}\sum_{m=1}^{5}\sum_{n=1}^{5}\frac{sin^2\left[\frac{m\pi}{2}\right]sin^2\left[\frac{n\pi}{2}\right]}{\pi^4 D_f\left[\frac{m^2}{L^2}+\frac{n^2}{B^2}\right]^2+k_{sv}} < 0.03 \qquad 6.32.a$$

where:

$$D_f=\frac{E_f t^3}{12(1-\nu_f)^2}, k_{sv}=\frac{1.3G}{B(1-\nu)} \qquad 6.32.b$$

and E_f and ν_f are the elastic modulus and Poisson's ratio for the foundation material (i.e. concrete), t is thickness of the foundation, ν is the Poisson's ratio of the soil, and B and L are dimensions of the foundation (or its equivalent rectangle). If the base is a mat foundation, B and L are calculated for the area shared by each column. In the case of mat foundations, it is necessary that the entire foundation meets the rigid foundation criterion in order to be considered rigid.

6.4.2. Determining the rigidity of strip foundation

Strip foundations are assumed to be rigid relative to the bearing soil , if Eq. (6.33) is satisfied:

$$\frac{E_f I_f}{{L_f}^4} > \frac{2}{3}k_{sv}B \qquad 6.33$$

where I_f and L_f are the moment of inertia of the total un-cracked cross-section of the foundation over the axis perpendicular to the investigating direction and length of the portion of foundation assigned to each column, respectively. k_{sv} is the base reaction factor.

6.4.3. Stiffness of equivalent springs

For the case of rigid foundations, stiffness of the equivalent springs should be calculated using Sections 6.4.3.1 or 6.4.3.2. Otherwise, stiffness of the springs shall be determined using Section 6.4.3.3. Section 6.4.3.1 shall be used if the foundation is not included in the soil-structure model, but for integrated modeling of the foundation and supper-structure, Sections 6.4.3.2 and 6.4.3.3 shall be considered.

6.4.3.1. Rigid foundation - discrete method

In this case, in the building structure model, springs are placed at the center of the base of each column or wall corresponding to each degree of freedom according to Figure 6.1. In part b of this figure, x-axis is the local axis parallel to the foundation length. Stiffness of the spring corresponding to the degree of freedom j, $K_{j,emb}$, taking into account the embedment of the foundation, shall be obtained from Equ. (6.34):

$$K_{j,emb} = K_{j,sur}\beta_j \qquad 6.34$$

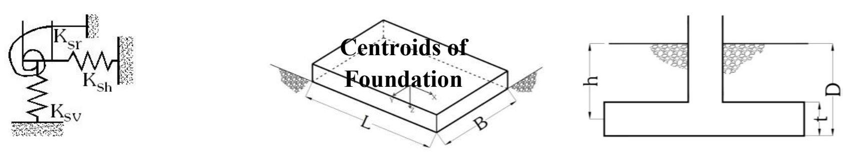
Where:

$j = x, y, z$ for the translational degrees of freedom,

$j = xx, yy, zz$ for the rotational degrees of freedom,

$K_{j,sur}$ is the spring stiffness for the surface foundation,

β_j is the stiffness modification factor for the embedded foundation, and $K_{j,sur}$ and β_j are calculated using Eqs. (6.35) and (6.36).



(a) (b) Note: $L \geq B$ (c)

Figure 6.1. Equivalent foundation springs and parameters of Eqs. 6.35 and 6.36

$$
\begin{aligned}
K_{x,sur} &= \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65}+1.2\right]\\
K_{y,sur} &= \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65}+0.4\frac{L}{B}+0.8\right]\\
K_{z,sur} &= \frac{GB}{1-\nu}\left[1.55\left(\frac{L}{B}\right)^{0.75}+0.8\right]\\
K_{xx,sur} &= \frac{GB^3}{1-\nu}\left[0.4\frac{L}{B}+0.1\right]\\
K_{yy,sur} &= \frac{GB^3}{1-\nu}\left[0.47\left(\frac{L}{B}\right)^{2.4}+0.034\right]\\
K_{zz,sur} &= GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45}+0.51\right]
\end{aligned}
\qquad 6.35
$$

$$
\begin{aligned}
\beta_x &= \left[1+0.21\sqrt{\frac{D}{B}}\right]\left[1+1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]\\
\beta_y &= \left[1+0.21\sqrt{\frac{D}{L}}\right]\left[1+1.6\left(\frac{hd(B+L)}{LB^2}\right)^{0.4}\right]\\
\beta_z &= \left[1+\frac{1}{21}\frac{D}{B}\left(2+2.6\frac{B}{L}\right)\right]\left[1+0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]\\
\beta_{xx} &= 1+2.5\frac{d}{B}\left[1+\frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]\\
\beta_{yy} &= 1+1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5+3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]\\
\beta_{zz} &= 1+2.6\left(1+\frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}
\end{aligned}
\qquad 6.36
$$

where *d*, is the effective foundation-side wall height in contact with the surrounding soil. Rest of the parameters are shown in Fig. 6.1.

6.4.3.2. Rigid foundation - integrated method

In this case, it is considered that the concrete foundation is modeled, along with the structure in a single model. Two orthogonal horizontal springs, one vertical spring and two rotational springs about the local horizontal axes of the foundation according to Fig. 6.1, shall be defined at each node. The stiffness values of the horizontal and vertical springs are obtained from the product of the horizontal and vertical stiffness and ratio of the assigned area of that node with respect to the total area of the foundation footprint according to Eqs. (6.34) to (6.36). In the sites representing classifications I-III, the foundation can be assumed to be constrained in the horizontal directions and useof springs along the two horizontal directions can be ignored. Stiffness of the rotational springs at each node, $k_{xx,emb}$ and $k_{yy,emb}$, are calculated for instance for $k_{xx,emb}$ as follows.

$k_{xx,emb}$ is equal to the product of $(K_{xx,emb} - k_{z,emb} \sum l_i^2)$ and ratio of the assigned area of a node to the total plan area of the foundation. Here, $k_{z,emb}$ is the stiffness of the vertical springs at each node of the (embedded) foundation and l_i is the distance of the (i^{th}) node from the centroid of the foundation area. $k_{yy,emb}$ can be determined in a similar way.

6.4.3.3. Flexible foundation

The flexible foundation shall be modeled along with the structure in a single model. Two orthogonal horizontal springs and one vertical spring should be defined at each node. The stiffness values of the horizontal springs are obtained from the product of the total horizontal stiffness of the foundation by the ratio of the assignedarea of each node to the total area of the foundation, according to Eqs. (6.34) to (6.36). The stiffness values of the vertical springs, k_{sv} are obtained by the product of the assigned area of each node and the subgrade reaction modulus (Eq. 6.32b). In the sites representing the classifications I-III, the foundation can be assumed to be constrained in the horizontal direction and use of springs in horizontal directions can be ignored.

6.5. Base Slab Averaging and Embedment (Kinematic) SSI Effects

In order to consider the effects of kinematic interaction, the spectral response is permitted to be modified by the factor RRS_{bsa} accounting for the incoherency of the wave field effects. Moreover, a correction can be done using the factor RRS_e for the depth of embedment. Both corrections can be

done by multiplying the mentioned factors with the spectral acceleration values corresponding to each period. The above spectrum modification factors are determined according to Sections 6.5.1 and 6.5.2, respectively. Modification of the response spectrum for kinematic interaction effects are permitted only for use with the nonlinear time history analysis according to Section 4.12.3, applying the site-specific response spectrum developed in accordance with the considerations of Chapter 3 and subject to the limitations in Sections 6.2.3, 6.5.1 and 6.5.2.
In no case, the product of $RRS_e \times RRS_{bsa}$ shall take values less than 0.7.

6.5.1. Base slab averaging

Applying the base slab averaging effect using the method presented in this section s is permitted for the following cases:

- All structures located on site classes weaker than type II.
- Structures resting on mat foundations.
- Structures with their foundation consisting of elements interconnected with concrete slabs or continuously connected with grade beams or other foundation elements of sufficient lateral stiffness, suchin a manner not to be characterized as flexible under the requirements of Section 4.13.

The modification factor for base slab averaging, RRS_{bsa}, shall be determined using Eq. 6.37 to adjust the spectrum in each period of interest.

$$RRS_{bsa} = 0.25 + 0.75 \times \left\{\frac{1}{b_0^2}\left[1 - \left(\exp\left(-2b_0^2\right)\right) \times B_{bsa}\right]\right\}^{\frac{1}{2}} \qquad 6.37$$

where:

$$B_{bsa} = \begin{cases} 1 + b_0^2 + b_0^4 + \dfrac{b_0^6}{2} + \dfrac{b_0^8}{4} + \dfrac{b_0^{10}}{12} & b_0 \le 1 \\ [\exp(2b_0^2)] \times [\dfrac{1}{\sqrt{\pi} b_0}\left(1 - \dfrac{1}{16 b_0^2}\right)] & 1 < b_0 \end{cases} \qquad 6.38$$

And:

$$b_0 = 0.0023 \times \left(\frac{b_e}{T}\right) \qquad 6.39$$

where:

b_e is the effective foundation size, as:

$$b_e = \sqrt{A_{base}} \leq 80\,m \qquad 6.40$$

6.5.2. Embedment

The response spectrum shall be developed based on a site-specific study at the depth of the base of the structure. Alternatively, modification for embedment is permitted using the procedure of this section.
The modification factor for embedment, RRS_e, shall be determined using Eq. 6.41 at each period required for analysis.

$$RRS_e = 0.25 + 0.75 \times \cos\left(\frac{2\pi e}{T.v_{s,e}}\right) \qquad 6.41$$

where:
e is the foundation embedment depth in meters. The depth shall not be taken as greater than neither 6 m nor the bottom level of the base slab diaphragm. A minimum of 75% of the foundation footprint shall be present at the embedment depth.The foundation embedment depth for structures located on sloping sites shall be taken to be equal to the shallowest embedment.
$v_{s,e}$: Average effective shear wave velocity corresponding to the site-soil conditions, which is used in determining the embedment effects and is taken as average of the shear wave velocity over the embedment depth of the foundation ($v_{so,e}$) according to Table 6.1 or site-specific studies, not to be taken smaller than 200 m/sec.
$v_{so,e}$**:** Average low-strain shear wave velocity used in determining the embedment effect over the embedment depth of the foundation (Eq. 6.1).
T**:** Response spectrum period of interest, which shall not be taken as less than 0.20 s when used for calculating the kinematic interaction effects.

6.6. Fluid-Soil-Structure Interaction for OilStorage Tanks

In the seismic analysis of reservoirs and large petroleum storage tanks, considering the effects of fluid-soil-structure interaction improves the accuracy of the analysis. The simplest method for modeling the soil beneath the foundation is to consider the effects of soil stiffness with linear springs, where related stiffness values are determined using the subgrade modulus. A more accurate solution is to consider the effect of soil damping and fluid-structure interaction within the calculations. The simplified method of this section to apply the effects of soil-fluid-structure interaction (FSSI) is based on separation into two problems: soil-structure interaction (SSI) and structure-fluid interaction (FSI). In this case, the rocking degree of freedom of the foundation is neglected.

Generally, a two-mass model can be considered for the fluid. The first is the impulsive part supposed to be rigid and to move with the structure. The second is the convective part that can oscillate out-of-phase to the tank Use of such lumped masses leads to modal frequencies related to impulsive and convective modes, respectively. A comprehensive description of such a two-mass model is represented in Chapter 12.

Chapter 7
Nonbuilding Structures

7.1. General

7.1.1. Scope and general requirements

The requirements of this chapter concern structures with industrial application that their main goal is providing shelter to equipment or machinery and the role of their occupants with a minimum presentation is only to maintain or inspect the performance of facilities and machinery or their usage process.

Nonbuilding structures including stacks, tanks, pipelines and offshore structures, in addition to be governed by the specific seismic requirements of chapters 11 to 14, shall satisfy the relevant specifications of sections 7.3 and 7.4.

Nonbuilding structures that are covered by this chapter consist of two categories including building-like and non-building-like structures. Building-like nonbuilding structures comprise of load bearing systems similar to the systems listed in Table 4.5. Examples of such structures are pipe racks, supporting frames of equipment, and steel storage racks. Non-Building-like nonbuilding structures are extensively utilized in industrial complexes and rely on load bearing systems that are not similar to those used in buildings. Horizontal storage tanks, cooling towers, pumps, and heat exchangers are among this category supported on ground or another structure.

In this chapter, general requirements for design and analysis of nonbuilding structures are presented in Sec. 7.2 and the specific regulations for each of the two categories of building-like and non-building-like structures are given in sections 7.3 and 7.4, respectively. Design of foundation of these structures shall be carried out based on the requirements of chapter 15 of the 9th Clause (of the National Building Regulations) and for the machine foundations under dynamic loading on the basis of valid references.

7.1.1.1. Definitions

This chapter has no definitions.

7.1.1.2. Symbols

Notations and abbreviations used in the relations and sections of this chapter are listed alphabetically in the following:

C_d	:	Displacement amplification coefficient
C_s	:	Seismic coefficient, Base shear factor
$D_{1,2,3}$	:	Seismic design categories 1, 2, 3
g	:	Acceleration of gravity
I_e	:	Importance factor
k	:	Exponent in the function for distribution of the lateral force along height
R	:	Response reduction (or behavior) factor
S_1	:	Spectral acceleration parameter (g) corresponding to the rare earthquake (third hazard level) at of 1 sec on the bedrock from the site specific analysis
S_{DS}	:	Spectral acceleration parameter (g) at small periods (0.2 sec) corresponding to the design earthquake for 5% damping ratio
T	:	Natural period
V	:	Base shear
W	:	Effective seismic weight
w_i	:	Effective seismic weight of story, level, the period or part i
δ_i	:	Lateral displacement for natural period calculation
ρ	:	Redundancy factor
Ω_0	:	Overstrength factor

7.1.2. Seismic design

Design of nonbuilding structures shall include providing for sufficient stiffness, strength and ductility based on the requirements of this chapter for resisting the ground motion consequences. Where design specifications are not asserted or no reference for them are mentioned in this chapter, such as the case of piers and wharves or some electrical components used in the oil industry, use can be made of other valid references. In any reference document where the design requirements are based on allowable stresses, the seismic design forces shall be determined based on this chapter and shall be combined with the other required loads as mentioned in Sec. 2.2.1. The resulting forces shall be used along with the allowable stresses for design.

7.1.3. Selection of the structural analysis method

The structural analysis method for building-like nonbuilding structures shall be selected based on Sec. 4.10 to 4.12. Non-building-like structures shall be analyzed using the equivalent lateral force procedure of Sec. 4.10, or the dynamic spectrum procedure based on Sec. 4.11, or the nonlinear dynamic procedure of Sec. 4.12, or the specific procedure mentioned in the reference document, if applicable.

7.2. General design requirements

7.2.1. System selection and design parameters

Nonbuilding structures having specific seismic design criteria in reference documents, shall be designed using the reference documents on the condition that the design spectrum is consistent with the requirements of Sec. 3.8 and the seismic design parameters of this section is used for their design. Otherwise, the nonbuilding structure shall be designed for seismic resistance based on Sec. 7.3 or 7.4, whichever applicable, and the following additional requirements and exceptions.

For building-like and non-building-like structures, the seismic load bearing system shall be selected from within one of the items listed in Table 7.1 and Table 7.2, respectively, under the system and height limitations corresponding to each seismic design category mentioned in the tables. For the systems not mentioned in Table 7.1, Table 4.5 can be utilized. Appropriate relevant tabular values of R, Ω_0, and C_d shall be used for determination of base shear based on Eq. 4.2, and member design forces and relative displacements. Design and detailing requirements shall be followed according to the sections referred to in the tables.

Base shear of the rigid nonbuilding structures is calculated according to Sec. 7.2.6.

Table 7.1. Seismic design parameters of building-like structures

					Limitations of structural system and height, h_n (m)[a]		
					Seismic design category		
Type of nonbuilding structure	Detailing requirements	R_u	Ω_0	C_d	D_3[b]	D_2[b]	D_1[c]
Building frame systems with:							
Special steel concentrically braced frames	Chapter 3 of the 10th Clause	6	2	5	50	50	30
Ordinary steel concentrically braced frames:	Chapter 3 of the 10th Clause	3.25	2	3.25	10[d]	10[d]	NP[d]
with increase of the allowable height	Chapter 3 of the 10th Clause	2.5	2	2.5	50	50	30
with no height limit	Chapters 1 and 2 of the 10th Clause	1.5	1	1.5	NL	NL	NL
Moment frame systems with:							
Special steel moment frames	Chapter 3 of the 10th Clause	8	3	5.5	NL	NL	NL
Special concrete moment frames	The 9th Clause including chapter 20	8	3	5.5	NL	NL	NL
Intermediate steel moment frames:	Chapter 3 of the 10th Clause	4.5	3	4	10[e,f]	NP[e,f]	NP[e,f]
with increase of the allowable height	Chapter 3 of the 10th Clause	2.5	2	2.5	50	50	30
with no height limit	Chapter 3 of the 10th Clause	1.5	1	1.5	NL	NL	NL
Intermediate concrete moment frames:							
with increase of the allowable height	The 9th Clause including chapter 20	3	2	2.5	15	15	15
with no height limit	The 9th Clause including chapter 20	0.8	1	1	NL	NL	NL
Ordinary steel moment frames[g]:	Chapter 3 of the 10th Clause	3.5	3	3	NP[e,f]	NP[e,f]	NP[e,f]
with increase of the allowable height	Chapter 3 of the 10th Clause	2.5	2	2.5	30	30	NP[e,f]
with no height limit	Chapters 1 and 2 of the 10th Clause	1	1	1	NL	NL	NL
Ordinary concrete moment frames:							

with increase of the allowable height	The 9th Clause excluding chapter 20	0.8	1	1	15	15	15
Steel storage racks	Sec. 7.3.2	4	2	۳/۵	NL	NL	NL
Cantilever steel storage racks of hot-rolled steel with:							
Ordinary moment frame[h]	Sec. 7.3.2 and chapter 3 of the 10th Clause	2.5	2	2.5	NL	NL	NL
Ordinary braced frame[h]	Sec. 7.3.2 and chapter 3 of the 10th Clause	3.25	2	3.25	NL	NL	NL
Cantilever steel storage racks of cold-formed steel with[i]:							
Ordinary moment frame[h]	Sec. 7.3.2 and Document 612 of the Planning and Budgetary Organization	1	1	1	NL	NL	NL

[a] NL= No limit; NP= Not permitted

[b] Refer to Sec. 4.5 for description of seismic resistant systems limited to the maximum height limit, h_n, of 75 m.

[c] Refer to Sec. 4.5 for description of seismic resistant systems limited to the maximum height limit, h_n, of 50 m or less.

[d] Ordinary steel braced frames and precast concrete frames are permitted to be used for the support system of pipes limited to the maximum height of 20 m.

[e] Ordinary and intermediate steel moment frames are permitted to be used for the support system of pipes limited to the maximum height of 20 m where in-situ moment connections are of the bolted end-plate type.

[f] Ordinary and intermediate steel moment frames having connections other than in-situ bolted end-plate moment connections, are permitted to be used for the support system of pipes limited to the maximum height of 11 m.

[g] Ordinary steel moment frames are permitted to be used for single story structures in seismic design categories D_2 and D_3 with no height limit except of support systems of pipe racks, where the roof dead load and the dead load of exterior walls at elevations over 10 m, including equipment attached to the roof or walls and columns and their supported equipment, are less than 1 kN/m^2. To determine the equivalent distributed load of equipment, area of walls or the corresponding supported roof shall not be taken larger than 60 m^2. Use of such frames is permitted up to the height of 10m where the roof dead load, including the attached equipment, is less than 1.7 kN/m^2 and the dead load of exterior walls, including the attached equipment and columns and their supported equipment, is less than 1 kN/m^2.

[h] Connection of column to its base shall be designed for the lesser of M_n of column and the factored moment at the column base using the overstrength factor for the seismic moment portion.

[i] Cold-formed sections satisfying Tables 10.2.2.1 to 10.2.2.4 of the 10th Clause, whichever applicable, are permitted to be designed based on Chapter 3 of the 10th Clause.

Table 7.2. Seismic design parameters of non-building-like structures

					Limitations of structural system and height, h_n (m)[a,b]		
					Seismic design category		
Type of nonbuilding structure	Detailing requirements[c]	R_u	Ω_0	C_d	D_3	D_2	D_1
Elevated tanks, vessels, bins, or hoppers supported by:							
Symmetrically braced integral legs (not similar to buildings)	Sec. 7.4.5	3	2[d]	2.5	50	30	30
Unbraced integral legs or asymmetrically braced integral legs (not similar to buildings)	Sec. 7.4.5	2	2[d]	2.5	30	18	18
Horizontal, saddle-supported welded steel vessels	Sec. 7.4.9	3	2[d]	2.5	NL	NL	NL
Flat-bottom ground-supported tanks:	Sec. 7.4.5						
Steel or fiber-reinforced plastic:							
Mechanically anchored		3	2[d]	2.5	NL	NL	NL
Self-anchored		2.5	2[d]	2	NL	NL	NL
Reinforced or prestressed concrete with:							
Reinforced nonsliding base		2	2[d]	2	NL	NL	NL
Anchored flexible base		3.25	2[d]	2	NL	NL	NL
Unanchored and unconstrained flexible base		1.5	1.5[d]	1.5	NL	NL	NL
Other cases		1.5	1.5[d]	1.5	NL	NL	NL
Cast-in-place concrete silos that have walls continuous to the foundation	Sec. 7.4.2	3	1.75	3	NL	NL	NL
Reinforced concrete tabletop structure (not similar to buildings)	Sec. 7.4.4	1.5	1.5	1.5	21	21	21
	Sec. 7.4.4.2	2	2	2	21	21	21
	Sec. 7.4.4.3	2.5	2	2	21	21	21

supporting elevated tanks,	Sec. 7.4.4.4	4	2	2.5	21	21	21
Reinforced masonry structures not similar to buildings detailed as intermediate reinforced masonry shear walls	The 8th Clause[e]	3	2	2.5	15	15	15
Concrete chimneys and stacks	Sec. 7.4.2 and chapter 12	2	1.5	2	NL	NL	NL
Steel chimneys and stacks	Sec. 7.4.2 and chapter 12	2	2	2	NL	NL	NL
All steel and reinforced concrete distributed mass cantilever structures not otherwise covered herein, including stacks, chimneys, silos, skirt-supported vertical vessels; single-pedestal or skirt-supported:	Sec. 7.4.2						
Welded steel	Sec. 7.4.2	2	2[d]	2	NL	NL	NL
Welded steel with special detailing[f]	Sec. 7.4.2	3	2[d]	2	NL	NL	NL
Prestressedorreinforced concrete	Sec. 7.4.2	2	2[d]	2	NL	NL	NL
Prestressed or reinforced concrete with special detailing	Sec. 7.4.2 and the 9th Clause	3	2[d]	2	NL	NL	NL
Trussed towers (freestanding or guyed), guyed stacks, and guyed chimneys	Sec. 7.4.2	3	2	2.5	NL	NL	NL
Steel tubular support structures for onshore wind turbine generator systems		1.5	1.5	1.5	NL	NL	NL
Cooling towers of:							
Concrete or steel		3.5	3.75	3	NL	NL	NL
Wood frames		3.5	3	3	NL	15	15
Telecommunication towers as:							
Steel truss		3	1.5	3	NL	NL	NL
Steel pole		1.5	1.5	1.5	NL	NL	NL
Wood pole		1.5	1.5	1.5	NL	NL	NL
Concrete pole		1.5	1.5	1.5	NL	NL	NL
Steel frame		3	1.5	1.5	NL	NL	NL
Wood frame		1.5	1.5	1.5	NL	NL	NL
Concrete frame		2	1.5	1.5	NL	NL	NL

Inverted-pendulum-type structures (except elevated tanks, vessels, bins, and hoppers)	Table 4.5	2	2	2	NL	NL	NL
Ground-supported cantilever walls or fences	Sec. 7.4.3	1.25	2	2.5	NL	NL	NL
Signs and billboards		3	1.75	3	NL	NL	NL
Steel lighting system support pole structures		1.5	1.5	1.5	NL	NL	NL
Wharves with:							
Verticalprestressedconcrete piles		2		2	NL	NL	NL
Vertical steel piles		2		2	NL	NL	NL
Battered piles		1		1	NL	NL	NL
All other self-supporting structures, tanks, or vessels not covered above or by reference standards that are not similar to buildings		1.25	2	2.5	15	15	15

[a] NL= No limit; NP= Not permitted

[b] To determine the height limitation, height of structure shall be considered from the base level to the top of the frame composing the main seismic load bearing system of structure.

[c] Where no section number or reference is mentioned in the column labeled “Detailing requirements”, there is no specific requirement for the item mentioned at the corresponding row.

[d] Refer to Sec. 7.4.2 for application of Ω_0 to tanks and vessels.

[e] Having details corresponding to a complete vertical load carrying frame.

[f] Requirements of chapter 12 for each function and risk category shall apply.

[g] The slenderness ratio of the corresponding columns shall satisfy $L/r \leq 22$.

7.2.2. Minimum base shear

For building-like structures, the minimum base shear shall be calculated using Eq. 4.10.2.1. For no-building-like structures, Eq. 4.3 is replaced by the following equation:

$$V_{min} = 0.044 S_{DS} I W \geq 0.03W \tag{7.1}$$

For tanks and vessels, if a larger minimum base shear is mentioned in the reference document, requirement of the reference document shall apply.

Moreover, for non-building-like structures located where $S_1 \geq 0.6$, the minimum value required by Eq. 4.4 shall be replaced with the following equation:

$$C_s = 0.8S_1 \,/\, (R/I) \tag{7.2}$$

For tanks and vessels, the coefficient 0.8 in Eq. 7.2 may be replace by 0.5.

In equations 7.1 and 7.2, I is the importance factor determined based on Sec. 7.2.5.

7.2.3. Distribution of the lateral forces in elevation

Distribution of the lateral forces in elevation of the nonbuilding structures shall be determined using the requirements of Sec. 4.10.4 or 4.11, or the reference document, whichever apply.

7.2.4. Considerations of torsion

Where in the analysis, position and value of the mass of various parts of the structural system and its contents and any supported structure or nonstructural component (including staircase, piping, etc.) contributing to the mass or stiffness or both of the main structural system are accounted for, the accidental torsion mentioned in Sec. 4.10.6.1 can be neglected:

a. Rigid nonbuilding structures (refer to Sec. 7.2.6);

b. Non-building-like structures designed using an R-factor not larger than 3.5;

c. Building-like structures designed using an R-factor not larger than 3.5, satisfying at least one of two following conditions:

(1) The rigidity center of each diaphragm is located at a distance not farther than 5% of the diaphragm dimension in both main directions, from the center of mass of the diaphragm;

(2) The structure is not categorized as having the horizontal torsional irregularity defined in Sec. 4.2 and contains at least two axes of lateral resistance along each of the main orthogonal directions. At least one of the two axes of lateral resistance shall be at a distance from the mass center equal to or larger than 20% of the plan dimension along that direction at each side of the center of mass.

In addition, structures designed based on this section shall be modelled three dimensionally according to Sec. 4.9.

7.2.5. Importance factor

The importance factor, I, and the function and risk category of the nonbuilding structures are determined based on the relative hazard of their contents and functionality. Value of I shall be equal to the larger of the values mentioned in the reference document, if available, and the corresponding value in Table 4.3.

Where according to Sec. 7.2.12 two or more of nonbuilding structures are connected by structural or nonstructural components and the structures are analyzed concurrently, the largest importance factor of the structures shall be used for the entire connected structures. For adjacent structures that are not connected, if it can be shown that the non-desirable function of a structure under earthquake results in non-desirable function of the adjacent structures as well, the largest importance factor of the structures shall be used for the entire adjacent structures.

For support structures of components and for the components supported by such structures, the largest importance factor of the supported components and the support structure shall be used for design of the support structure and all of the components it carries.

7.2.6. Rigid nonbinding structures

Non-building-like structures with their fundamental period, T, being less than 0.06 sec and their anchors, shall be designed for the base shear determined by the following equation:

$$V = 0.3S_{DS}WI \tag{7.3}$$

in which:

V= Design base shear of the rigid nonbuilding structure;

S_{DS}= Design spectral acceleration at small periods determined based on Sec. 8.3;

W= Effective seismic weight according to Sec. 7.2.7;

I= Importance factor determined based on Sec. 7.2.5.

The base shear should be distributed in elevation according to Sec. 7.2.3.

7.2.7. Effective seismic weight

The effective seismic weight, W, of the nonbuilding structure equals its dead load plus a portion of the live load according to Sec. 4.9.4. In determination of W weight of all of the common contents during service shall be included for tanks, vessels, bins, hoppers, and pipes. In addition, where the snow-and-ice load is equal to or larger than one-third of other loads contributing to W, it shall be included for determination of W. Except of very important structures, it is not required for the snow-and-ice load to be assumed to be larger than the sum of other loads contributing to W.

7.2.8. Period

The fundamental period, T, shall be determined based on the structural and deformation characteristics of the resisting elements using an analysis procedure according to Sec. 4.10.3. Alternatively, the fundamental period can be calculated using the following equation:

$$\boldsymbol{T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} w_i\delta_i^2}{g\sum_{i=1}^{n} w_i\delta_i}}} \qquad \textbf{7.4}$$

in which w_i is the weight of different parts of the structure considered to be concentrated at a specific location and δ_i is the displacement of the location of w_i, on the assumption that for making the displacement δ_i to happen, the structure is rotated at 90 degrees. Equations 4.7 to 4.9 should not be used for calculation of the fundamental period of nonbuilding structures.

7.2.9. Relative and lateral displacements

Relative displacement between different levels of structure under the design lateral forces shall be determined including the $P - \Delta$ effect. The value of such a displacement shall be checked with the limitations set by the industrial consultant, if any, after being multiplied by C_d / I_e. The drift limitations of Sec. 4.15 are not required to satisfy if using experimental or analytical data it can be shown that by violating the mentioned requirements, no negative effects apply on the stability of structure and its attached components and elements such as staircases and piping.

7.2.10. The P-Δ effect

For nonbuilding structures, the P-Δ shall be considered based on Sec. 4.10.9.
7.2.11. The vertical component and load combinations
For deign of tanks, vessels, suspended structures and structures possessing horizontal cantilevers, the vertical component of the design earthquake shall be considered based on Sec. 3.12 and 4.8. In addition, requirements of Sec. 12.1.8.2 or 12.2.4.6, whichever applicable, shall apply for the tanks. For suspended structures and structures possessing horizontal cantilevers, if the period of vertical motion is not calculated, the design vertical acceleration, S_{av}, shall be taken as equal to the maximum ordinate of the design vertical spectral acceleration based on Sec. 3.12. If the period of vertical motion is determined, S_{av} can be calculated based on the design vertical spectrum of that section using the determined period. The response modification factor, R_u, shall be taken as unity for use in the analysis under the vertical component of the design earthquake, except for determining the hydrodynamic hoop force under the vertical design earthquake in circular tanks that shall be conducted according to Sec. 12.1.8.2 or 12.2.4.6, whichever applicable.
For combining the horizontal and vertical components, the horizontal component can be rotated in the horizontal plane from 0 to 180°, each time at 15°, and its effects are combined with those of the vertical component. In this combination, once the 100% coefficient for the horizontal component and the 30% coefficient for the vertical component, and once the reverse shall be considered. Alternatively, the following simpler load combinations may be used to find the most critical case:

1. 100% of the actions under earthquake in a main horizontal direction combined with 30% of those corresponding to the orthogonal horizontal direction and 30% of the actions under the vertical component of earthquake. For finding the most critical effect at each member section, once the plus sign and once the minus sign should be taken in each term of the combination.
2. 100% of the actions under the vertical component of earthquake combined with 30% of those under earthquake in a main horizontal direction and 30% of the actions under earthquake in the orthogonal horizontal direction, using once the plus and once the minus sign in each term of the

combination. For checking the overall stability (overturning and sliding) of structure, this load combination is not needed.

7.2.12. Structures mutually connected by nonstructural elements

Structures that are connected to each other through nonstructural elements, shall be modelled together and analyzed concurrently, except of the following cases in which separate analysis of each structure is permitted:

1) Ratio of the fundamental periods of the two adjacent structures in the direction of earthquake, calculated by assuming the structures are not connected and by accounting for the tributary weight of the nonstructural element for each structure, is larger than 0.9 and smaller than 1.1;
2) Ratio of the fundamental periods of the two adjacent structures in the direction of earthquake, calculated by assuming the structures are not connected and by accounting for the tributary weight of the nonstructural element for each structure, is larger than 0.8 and smaller than 1.2 and ratio of their effective seismic weight, calculated under the same assumptions, falls in the same range;
3) Ratio of the axial stiffness of the connecting nonstructural element to the lateral stiffness of each connected structure in the direction of ground motion is smaller than 0.2.
4) The connecting nonstructural element is flexibly connected and the connections can sustain sum of the lateral displacements of the connected structures at the level of the element, or the nonstructural element is connected to the adjacent structures using sliding connections designed for the same total displacement.

The connecting nonstructural element shall be consistent with the requirements of chapter 8.

7.2.13. Non-building-like structure on a support structure

Where a non-building-like structure rests on another structure and itself is not part of a seismic load carrying system, the following requirements shall apply:

1) When weight of the resting structure is smaller than 20% of the total effective seismic weight, including the effective weight of the resting and

support structures, the resting structure is considered as a nonstructural element and it shall follow the requirements of chapter 8. The support structure shall be based on the requirements of chapter 4 or this chapter, whichever apply, and solely weight (and not stiffness) of the resting structure is taken to determine its effective seismic weight.

Note. Where ratio of the fundamental period of the resting structure to that of the support structure (which for it weight of the resting structure is also considered) is larger than 2, for design of the resting structure according to Sec. 8, it can be considered as attaching to a rigid base by ignoring acceleration amplification.

2) Where weight of the resting structure is equal to or larger than 20% of the total effective seismic weight, including the effective weight of the resting and support structures, the resting and support structures shall be modelled, analyzed, and designed together. The response modification factor of the combined system equals the smaller value between the corresponding values for the resting and support structures.

Note. Where ratio of the fundamental period of the resting structure to that of the support structure (which for it weight of the resting structure is also considered) is larger than 2 or smaller than 0.5, mutual modelling of the resting and support structures can be abandoned. In the former case the resting structure is assumed to be bearing on ground and it is designed based on the requirements of this chapter. In the latter case too, the resting structure shall be designed based on the requirements of this chapter but its design acceleration shall be determined including amplification according to chapter 8. In both cases, the support structure shall be designed only by including the weight (not stiffness) of the resting structure.

7.2.14. Anchors

Design of anchors to concrete shall be carried out using requirements of chapter 18 of the 9th Clause. Anchoring to masonry shall be consistent with the requirements of the 8th Clause or other valid references. Design of anchors to masonry shall be conducted such that its governing behavior is similar to tensile or shear yielding of a ductile steel component. In lieu of that, anchor design may be accomplished using the amplified seismic load combinations of Sec. 2.2.4. Replacement of a ductile grade anchor with one with lower ductility during construction is only admissible by abiding by the above requirement.

7.2.15. Foundations on liquefiable ground

If the local soil is liquefiable, surface foundations should be prohibited except when such a foundation is designed along with the structure and its attached systems for the case of soil strength degradation and lateral development under the total and differential settlements due to the maximum considered earthquake.

7.3. Specific requirements for building-like structures

7.3.1. Pipe racks

Piperacks not supported by other structures shall be designed based on Sec. 4.10 or 4.11 and by considering chapter 13. The lateral displacement of frame and possibility of impact by the pipes shall be checked by using the actual displacements calculated by multiplying the displacements determined using the above sections by C_d/I_e. Requirements of chapter 13 should also be satisfied for design of the pipe connections. The positive effect of the frictional resistance due to weight of the pipes shall not be used in such analysis.

At points where pipes resting on another line of pipe racks merge to the piping on the considered pipe racks, or where direction of the considered pipe racks is varied due to change of the path of pipes, effects of three dimensional nature of the seismic response shall be considered appropriately in design of the supporting frames.

7.3.2. Steel storage racks

The effective seismic weight for design of storage racks shall be taken equal to weight of the structure plus the stored goods for each of the two following cases to determine the critical case: a) Weight of the goods on all of the rows when each row is loaded at two-thirds of its weight capacity, and b) weight of the goods only at the highest row when this row is loaded at 100% of its weight capacity.

Distribution of seismic forces in elevation shall be based on Sec. 4.10.4 and the lateral force shall be applied at the mass center of each level considering the elevation of the mass center of each row of the racks. In this calculation, the k exponent mentioned in Sec. 4.10.4 may be taken as unity.
The structural system of the storage rack shall resist its lateral seismic displacements. To prevent impact and cross interaction, width of the specified perimeter of the racks where no obstacle including pipes, mechanical channels, cables, and alike should not be present, shall be at least 5% of the total height of the structure of the pipe rack from the base level.

7.3.3. Support structure of tanks and vessels

Where the support structure is integrated with the tank or vessel, it shall be designed according to chapter 12.
Where the support structure is not integrated with the tank or vessel, requirements of Sec. 7.2.13 shall be followed for design of the support structure. Distribution of the vertical responses of the tank or vessel due to its weight and overturning moment on the support structure shall be determined considering the relative stiffness of the tank and structural members of the support structure. If the tank or vessel rests on a grid of beams, the vertical response shall be amplified at 20% for design of the beams and their attachments, in order to include the effect of the non-uniform support and sloshing of the liquid contained in the tank. In determining the clear distance for the tank or vessel to prevent impact to the peripheral structure, the total lateral displacement of the tank and the support structure including the correction factor $C_d \,/\, I_e$ on the elastic displacements, shall be used.

7.4. Specific requirements for non-building-like structures

In determining the seismic loads on non-building-like structures the indeterminacy factor ρ of Sec. 4.7 may be taken as equal to unity.

7.4.1. Earth retaining walls

This section pertains to all earth retaining structures. The lateral earth pressure due to ground motions in earthquake shall be determined based on chapter 5. The function and risk category shall be decided based on adjacency condition of the retaining structure with other structures. If failure of the earth retaining structure affects the adjacent structure, the function and risk category shall not be taken lesser than that of the adjacent structure. The earth retaining wall may designed under seismic loads as sliding or non-sliding, where a limited slide is allowed in the former case. Cantilever walls shall be designed as sliding.

7.4.2. Stacks and their support structures

Stacks should be designed against earthquake based on the requirements of chapter 11. In addition, in the seismic design of covered stacks, interaction of the stack and its cover shall be considered. A minimum distance between the cover and stack surface equal to the relative elastic displacement multiplied by C_d shall be considered.
The peripheral support structure of stack, if present, shall be designed using the seismic parameters of Table 7.1 when it is consistent with a braced frame. It shall be designed using the seismic parameters of the truss tower in Table 7.2 otherwise.

7.4.3. Cantilever walls or fences supported on ground

Cantilever walls or fences supported on ground being at least 1.8 m in height shall be designed against earthquake. Use of plain concrete, autoclaved aerated concrete (AAC) and masonry is not allowed for these elements.

7.4.4. Tabletop reinforced concrete structures for rotating components and tanks and vessels of the process line

7.4.4.1. Introduction

Slab-column frames may be used as support structures conditioned at observing the following:
a) The bending capacity of each column under all load combinations is less than two-thirds of the bending capacity of the slab and two-thirds of the bending capacity of its foundation,
b) Thickness of the slab is not less than 0.9 m, and,
c) Design of the reinforced concrete members satisfy the requirements of the 9th Clause, except its chapter 20.

Each of the R_u factors of Table 7.2 may be used for seismic design of tabletop structures, conditioned at observing the following requirements.

7.4.4.2. Design of tabletop structures with R_u=2

When designing a tabletop structure with R_u=2, the structure shall be supported by at least 6 columns and details of its components shall be based on Sec. 9.20.3 of the 9^{th} Clause.

7.4.4.3. Design of tabletop structures with R_u=2.5

For design of tabletop structures with R_u=2.5, details required by the 9^{th} Clause for ordinary moment frames shall be satisfied and observing the requirements of Sec. 7.4.4.1 of this chapter is not needed. If the total area of the vertical reinforcement in all columns (or each column) is not over 2% of the gross concrete section of the columns (or each column), Sec. 3.20.9 of the 9^{th} Clause may be considered to be satisfied. If the existing reinforcement is more than the mentioned value and the bending capacity of column is larger than that of slab (or beam, if existing), the horizontal shear force for designing the slab-column connection shall be calculated using the bending capacity of slab (or beam, if existing).

7.4.4.4. Design of tabletop structures with R_u=4

In design of tabletop structures with R_u=4, details required by the 9^{th} Clause for intermediate moment frames should be observed and satisfying the requirements of Sec. 7.4.4.1 of this chapter is not needed.

7.4.5 Tanks

7.4.5.1. General

This section applies to all containers of liquid, gas, and granular solids. Tanks can be of reinforced concrete, prestressed concrete, steel, aluminum, FRP, or plastic. Tanks supported at elevations higher than the base level, shall also satisfy the requirements of Sec. 7.2.13. Base shear and overturning moment of the plastic tanks shall be calculated based on the requirements of Sec. 12.2. In the next section special requirements of spherical tanks are presented. Other design requirements of the tanks are mentioned in chapter 12.

7.4.5.2. Spherical tanks

Requirements of Sec. 12.2.1, 12.2.2 and 12.2.4 on the general aspects, internal components, refractory lining, and effective seismic mass should also be satisfied for the spherical tanks.

For spherical tanks resting on column and cross braces:

a) Requirements of Sec. 12.4. shall apply.

b) Stiffening effect of the brace pre-tensioning shall be considered in determination of the natural period.

c) Shell of the sphere shall be designed against local buckling at its connection to each column.

d) The minimum strength of the connection of each brace to liquid storage tanks equals the yield strength of the brace and to gas storage tanks is equal to the value corresponding to the load combination including amplified seismic effects. Direct connection of the brace to shell of the tank shall be prohibited.

For skirt-supported spherical tanks:

a) Requirements of Sec. 12.4.6 shall apply.

b) Local buckling of the skirt under the membrane compressive force due to weight and overturning moment of the tank shall be prevented in design.

c) Openings of the skirt (including inspection gates, pipe ways, etc.) shall be designed such that strength of the skirt around the opening remains unreduced.

7.4.5.3. Secondary tanks

Secondary tank systems such as dikes and spillways shall be designed for the effects of the maximum considered earthquake in the empty condition and for the effects of the design earthquake in the full condition, as well as for all of the hydrodynamic forces determined in Sec. 12.2.4.7.

7.4.6. Hydraulic structures submerged in liquid

Submerged hydraulic structures include separation walls and wave barriers located inside the tanks and vessels containing liquid. Such structures are exposed to liquids at their both sides to the same height at the normal condition of their service. They come under one-sided extra hydrodynamic forces only at the time of earthquake.

Structures exposed to hydrodynamic pressures due to earthquake shall be designed for forces originated by the rigid and sloshing parts of liquid and the inertia of their structural components. The sloshing height shall be determined and checked with the freeboard of the structural wall. The interior components such as baffles or walls, columns, and other bearing elements of the roof, shall be designed for the effects of unbalanced forces and sloshing.

7.4.7. Boilers and pressure vessels

7.4.7.1. General

In design of boilers and pressure vessels, requirements of Sec. 7.2.13 and in case, Sec. 7.2.12 and chapter 12 shall be observed. Maximum strengths of materials are according to Table 7.3.

Table 7.3. Maximum material strengths for seismic design of boilers and pressure vessels

Material	Minimum F_u/F_y ratio	Maximum ratio of material strength (%)	Maximum strength ratio for threaded material[a] (%)
Ductile (such as steel, aluminum, copper)	1.33[b]	90[c]	70[c]
Semi-ductile	1.2[d]	70[c]	50[c]
Nonductile (such as cast-iron, ceramic, fiberglass)	Not applicable	25[e]	20[e]
Notes: [a] Connections threaded to the vessel or support [b] Minimum 20% elongation based on the material characteristics standard [c] Respect to the minimum characteristic yield strength [d] Minimum 15% elongation based on the material characteristics standard e Respect to the minimum characteristic tensile strength			

7.4.7.2. Interior elements and refractory lining

Interior elements of the pressure vessels including refractory lining, cyclones, trays, etc. shall be designed to resist forces determined in this chapter. The alternative approach is designing the interior elements for a strength smaller than that of the pressure vessel such that it fails before damaging the vessel, provided consequences of its failure do not place the pressure vessel at risk. In boilers and pressure vessels containing liquids, if the possibility exists that failure of the interior elements degrades functionality of the pressure vessel, effect of seismic sloshing on the interior elements shall be investigated.

7.4.7.3. Effective weight

If occurrence of sloshing is possible per Sec. 12.2.4.2, its effect shall be considered for calculating the effective weight of the stored materials. Variation of the unit weight of the material under pressure or temperature shall be taken into account.

7.4.7.4. Supports and attachments

The following requirements shall be met for the attachments and supports of the boilers and pressure vessels:

a) Attachments and supports that transfer seismic loads, shall be constructed using ductile material suitable for the intended function and the environmental conditions.

b) Design of the anchors shall be accomplished according to Sec. 7.2.14.

c) Design of the attachments and supports shall be conducted such that they remain ductile under all load amplitudes and reversing seismic displacements.

d) For the attachments, the potential effect of nonuniformity of the vertical bearing reactions, difference of the relative stiffness of support members, dissimilarity of the construction details, nonuniform shimming at the supports, and irregularity of the supports, on the pressure vessel and the support structure shall be considered.

7.4.8. Horizontal storage vessels on saddle supports

7.4.8.1. Effective mass

Effect of variation of the unit weight of the stored materials shall be considered in determining the effective mass. For calculating the supports, saddles and anchor bolts and in checking the foundation against seismic overturning, the material inside the vessel shall be considered as a rigid mass located at the geometrical center of mass.

7.4.8.2. Design of the vessel

In the absence of a more accurate analysis method:

a) For calculating the natural period and the overturning moment, the horizontal vessel with its length being 6 times its diameter or more, may be assumed as a beam on simple saddle supports.

b) For the pressure vessels with their length being less than 6 times their diameter, effect of deep beam shear deformations shall be considered for determining the vibration period and stress distribution.

c) Effect of local bending and buckling of the vessel shell at the location of the saddle supports caused by the seismic load shall be considered. Accounting for the effect of the internal pressure on increasing the buckling capacity of the shell is prohibited.

d) If the vessel is simultaneously used for storing gas and liquid, design of the vessel and its support structure shall be accomplished once by including the gas pressure and once without that (assuming emergent gas discharge).

Chapter 8

Nonstructural Components

8.1. General

8.1.1. Scope

In this chapter, seismic design requirements of nonstructural components of the Petroleum Facilities and Structures are presented. Although these elements are not modelled along with the supporting structure, they are susceptible to considerable earthquake forces. In the cases where weight of a nonstructural component is equal to or larger than 20% of the accumulative effective seismic weight of the nonstructural elements and the supporting structure, W, determination of the seismic forces and design of the components shall be performed according to Chapter 7.

Examples of mechanical and electrical components in the Petroleum Facilities and Structures include: pump, piping, duct, escalator, conveyor, chimney, antenna, crane, computer, monitor, transformer, emergency electricity system, communication system, fire protection system (including alarming, detectors, sensors and their piping), boilers, thermal convertors, rotating machinery and tanks. Examples of the architectural components include: stairways, parapets, short and tall partitions, prefabricated cladding, boards and signs, lighting system and ceilings.

This chapter provides the forces relevant to anchors of the nonstructural components and their stability.

In the oil industry facilities, the internal forces needed for design of each facility are provided by the corresponding vendor. In design of these facilities, in addition to the internal forces, the external forces including the seismic and wind loads and thermal effects shall be considered. Besides, satisfaction of the requirements and load combinations of Chapter 2 of this document is also necessary. Seismic design of nonstructural components is conducted according to their sensitivity as of acceleration sensitive or acceleration and displacement sensitive, based on the definitions of Sec. 8.2.7. On this basis, in design of the nonstructural components and their connections, the required capacity for the component and its anchor shall be provided under the seismic demands of Sec. 8.3. In addition, the specific requirements of the architectural elements per Sec. 8.6 and for the mechanical and electrical elements per Sec. 8.7 shall be followed in design of the components. For design of the elements prone to relative

displacements, sufficient capacity shall be provided per Sec. 8.3.4 where special bearings are utilized for vibration control of facilities during service, it is necessary to determine the capacity of these bearings against earthquake induced forces according to the requirements of this chapter. When the nonstructural component is located outside a building, design of their foundation shall be according to Chapter 5. For calculation of the design forces of nonstructural components, combination of the various types of actions during the service period such as thermal effects and working pressure or vibration of the machinery shall be considered.
In this regulation, the nonstructural components are meant to be elements having the following characteristics:

1. Components that are within a structure or the structure is their support.
2. Components that are outside the structure (except the cases belonging to nonbuilding structures) and are permanently connected to the mechanical and electrical systems.
3. Components that are part of the egress system of a structure.

Exception:

- Where ratio of the fundamental period of the nonstructural component and its connection (to a structure) to that of the support structure (including the concentrated mass of the nonstructural components) is less than 0.5 or larger than 2, the support structure may be designed according to the requirements of the Chapter 7.
- The design requirements and special limitations for stacks, tanks and pipes that based on their weight ratio are not covered by this chapter, are provided respectively in Chapters 11, 12 and 13.
- The following nonstructural components are exempted from the requirements of this chapter:

1. Mechanical and electrical components belonging to the seismic design category D_1 having an importance factor of 1.
2. Mechanical and electrical components in structures belonging to the seismic design categories D_2 and D_3 subjected to the following conditions:

a. The importance factor of the component is 1.
b. Connection of the component to ducts, pipes and alike is flexible and one of the following conditions exists:

b.1. Weight of the component is 1800 N or less and its center of mass is at most 1.2 m above the level the element is resting on.
b.2. Weight of the component is 90 N at most, or in distributed systems is 70 N/m or less.

8.1.2. Definitions

The following definitions are used in this chapter:
Nonstructural component: Nonstructural components (including mechanical, electrical and architectural components) are the elements connected to a structure that are not part of the main load bearing structural system.
Design earthquake: the earthquake with its intensity being two-thirds of the risk-based maximum considered earthquake (MCE_R).
Designated seismic system: The category of nonstructural component which shall be designed based on this chapter and the importance factor, I_p, of the element is larger than unity.
Design drift: The relative displacement of a story under the design earthquake at a certain point in the plan (center of mass or the perimeter).
Story drift: The horizontal displacement at the top of the story relative to its floor at the points on the same vertical axis.
Story drift ratio: The story drift divided by the story height h_{sx}.
Support of the component: Structural members, a set of members or the members produced in factory including braces, frames, pier, holders or saddles carrying gravity and service loads over to the structure.
Support and dick structures of facilities: A set of members for factory produced members except of the integrated supports including but not limited to moment frames, braced frames, the decks composed of steel members, piers being more than 0.6 m tall or walls that bear one or several nonstructural components.
Support of the distribution system: Members that provide vertical or horizontal seismic resistance, including but not limited to holders, braces, pipe racks and pipe bearing sets.
Flexible joints: The type of connections between the facilities where rotational or translational movement is permitted without performance degradation. Universal and accordion joints and flexible steel hoses are examples of flexible connections.

Friction clips: A tool dependent on friction for resisting loads in one or more directions. In such clipses, friction is produced mechanically not with the help of gravity.
Ground level: A reference horizontal level representing the finished level of ground at its contact point with the building at the location of all exterior walls.
Modular prefabricated mechanical and electrical systems: A set of prefabricated mechanical and electrical elements totally or partially confined.
Nonbuilding structure: A structure other than a building according to one of the types introduced in Chapter 7 and constructed based on the limitations defined in the same chapter.
Level above ground: A level at which the floor or roof at all of their points on the perimeter of the structure are located more than1.8 m above the ground level.
Design strength: The nominal strength multiplied by the strength reduction factor φ.
Nominal strength: Strength of a member or section calculated based on the requirements and assumptions of the strengths design method in this regulation (or another reference) excluding the strength reduction factor.
Support: Members, a set of members or members produced in a factory including braces, frames, piers, holders, saddles or inclined piers and their joints transferring the forces between nonstructural components and their attachments to the structure.
Flexible element: The nonstructural component having a fundamental period larger than 0.06 sec.
Rigid element: The nonstructural component with its fundamental period being equal to or smaller than 0.06 sec.
Rugged element: A nonstructural component preserving its continuous service after the design or larger earthquakes, based on observations in previous earthquakes, or the past seismic tests. Categorizing a nonstructural component as rugged, shall be based on comparison of a specific component having similar strength and stiffness. AC motors and compressors are examples of rugged components.

8.1.3 Symbols

Symbols and abbreviation used in the equations and sections of this chapter, are listed followingly in the alphabetical order:

a	:	Response coefficient.
a_i	:	Maximum acceleration at the level of installation of the nonstructural component i.
a_h	:	Force amplification factor as a function of the structure's height.
b_p	:	Width of rectangular glass.
C_m	:	Modified amplification factor of nonstructural component.
C_p	:	Base shear amplification factor of nonstructural component.
C_1	:	Average clear distance of the vertical edge of glass with its frame.
C_2	:	Average clear distance of the horizontal edge of glass with its frame.
d	:	Diameter of bar
D_c	:	Relative displacement of a structure between top and bottom of glass resulting in contact of the glass and its frame.
D_p	:	Relative displacement between the contact points of nonstructural component to structure.
D_{pl}	:	Design relative seismic displacement.
$D_{1,2,3}$	:	Seismic design categories of structure, D_1, D_2 and D_3
E_v	:	Actions of the design vertical earthquake.
F_p	:	Design seismic force.
F_{pi}	:	Force applied at the mass center of the ith part of nonstructural component.
g	:	Acceleration of gravity.
h	:	Average height of roof from the base level.
h_i	:	Height of the ith level from the base level.
h_p	:	Height of rectangular glass.
h_{av}	:	Average height of the connection points of nonstructural component to structure from the base level.
I	:	Importance factor of structure
I_p	:	Importance factor of nonstructural component.
k_p	:	Collective stiffness of component and its connection to structure
l_i	:	Distance between the mass center of the ith part of nonstructural component and the connection point.
n	:	Number of levels, stories, concentrated masses or masses of nonstructural component.

n'	:	Number of parts of nonstructural component.
R_{po}	:	Strength factor of component.
R, R_u	:	Response modification factor of the support structure
R_b	:	Reduction factor due to the ductility of structure.
S_a	:	Spectral acceleration of structure at its fundamental period relative to g.
S_{DS}	:	Spectral acceleration parameter (relative to g) corresponding to small periods (0.2 sec) in the design earthquake, for 0.05 damping ratio.
T	:	Period of structure.
T_p	:	Period of component.
T_a	:	Fundamental period of support structure.
V_p	:	Base shear or total shear existing at the bearing of nonstructural component.
W	:	Effective weight of structure
w_i	:	Effective weight of story, level or part i.
w_p	:	Weight of component in service.
w_{pi}	:	Weight of part i of nonstructural component.
z	:	Height of anchor point relative to the base level.
Δ_a	:	Allowable relative lateral displacement of the story.
Δ_f	:	Allowable relative seismic displacement in cladding walls and interior partitions having glasses.
δ_j	:	Design lateral displacement at jth levels.
μ_p	:	Ductility parameter of component
μ_{eq}	:	Equivalent ductility factor.
μ_p	:	Ductility factor of nonstructural component.
ρ	:	Redundancy factor.
Ω_0	:	Over strength factor.

8.1.4. Seismic design categories

For the purposes of this chapter, the seismic design category of nonstructural component is set to be identical to that of their corresponding structure.

If the nonstructural component is simultaneously connected to two adjacent structures, the higher seismic design category shall be used for the component. For determination of the seismic design category of the structure, Chapter 4 of this regulation shall be followed.

8.1.5. Importance factor of nonstructural component

The Importance factor of nonstructural component, I_p, equals 1.5 for the following components and 1 otherwise:

- Components that, to maintain life safety, their operation should continue after earthquake without interruption, such as fire protection facilities, automatic safety valves and escape stairways.
- Components that contain hazardous, poisonous, inflammable or explosive material possible to harm people or environment.
- Components located in a structure belonging to the Function and Risk Category IV or are connected to such structures through a bearing, or components necessary to maintain continuous operation of essential facilities.

8.1.6. Prefabricated modular mechanical and electrical systems

Prefabricated mechanical and electrical modules being 1.8 m tall and taller not pre-certified according to the requirements of this chapter and modules that contain or brace the mechanical and electrical components, such as cabins, shall be design according to Chapter 7. nonstructural components existing or braced in modular systems shall be designed based on this chapter.

8.1.7. Specific requirements of manufacturer

Where the specific requirements of manufacturer are based on allowable stress or allowable strength, design earthquake forces shall be combined with other loads according to Sec. 2.2.8 and while abiding by the requirements of Chapter 2, they shall be compared with the allowable values mentioned in the specific requirements. In any case, design details shall be followed according to the specific requirements at the responsibility of the manufacturer and they shall be based on the seismicity of the site. In addition, determining the minimum design forces due to the design earthquake, study of the interaction of nonstructural component, checking the displacements and design of bearing anchors are necessary. Where the reference design document is based on the allowable stresses, it is necessary to multiply the earthquake force of Sections 8.3.1, 8.3.2 or 8.3.3 whichever applicable, by

0.7 and used along with the effects of dead, live and service loads in the design of nonstructural component.

8.1.8. Application of the requirements of nonstructural components for nonbinding structures

Nonbuilding structures (including storage racks and tanks) supported by other structures shall be designed according to Chapter 7. If it shall be necessary anywhere in Chapter 7 to calculate the seismic forces using Chapter 8 and the corresponding values of a and R_{p0} are not mentioned in table 8.2 or 8.3, the $\frac{a}{R_{p0}}$ expression in Eq. 8.1 shall be taken equal to $2.5/R_u$. The parameter R_u in this phrase is determined using Table 7.1.

8.2. General design requirements

8.2.1. Construction documents

In design of nonstructural components or their supports and connections, the design requirements mentioned in Table 8.1 shall be specified by an accredited design professional in construction documents for use by the owner, responsible officials, contractors and inspectors.

8.2.2. Load combinations

For design of nonstructural components, their support and connections as covered by this chapter, effects of the load combinations mentioned in Chapter 2 shall be considered according to the type of the component where the mechanical or electrical component rests on seismic isolators, requirements of Chapter 10 shall be followed.

Table 8.1. Requirements applicable to architectural, electrical and mechanical components and their supports and connections

Nonstructural component (component, connection, support)	General design requirements (Sec. 8.1)	Requirements of force and displacement (Sec. 8.3)	Anchor requirements (Sec. 8.5)	Requirements of the architectural components (Sec. 8.6)	Requirements of mechanical and electrical components (Sec. 8.7)
Architectural components and connections, support of architectural components	*	*	*	*	
Mechanical and electrical components	*	*	*		*
Sopport and connections of mechanical and electrical components	*	*	*		*

* Requirements applicable to the mentioned component.

8.2.3. Exclusive requirements of unconventional systems

When the following systems are used, their performance shall be certified by test. In the tests, the necessary capacity as purposed by this regulation for the definite seismic levels shall be shown to be available using valid experimental methods. If no standard procedure exists for the specified test, the testing procedure shall be introduced by the design engineer based on the loading condition and performance expectation of the system.

1. systems with unconventional usage in the oil facilities and industries of the country.
2. systems without seismic check requirements.
3. systems designed and produced based on the regulations developed in the non-seismic countries.

8.2.4. Special seismic certificate for specific nonstructural components

For nonstructural components belonging to the seismic design categories D_2 and D_3, except of those exempted in Sec. 8.7.3, special design certificate shall be issued as described below. Special seismic design certificate exhibits the expected capacity and performance level of the component based on the seismic loading at the specified levels and shall be issued by the manufacturer or accredited laboratory approved by the client.

1. Active components or those that shall be active after the design earthquake: Continuity of service of such components after the design earthquake shall be certified by the manufacturer. Issuance of this certificate shall be based only on performing a shake table test or empirical observations consistent with the design earthquake based on Sec. 8.2.6, unless it can be shown that the corresponding component is stronger than similar certified components. Active components are the ones having rotative or moving parts or connecting to the energy current including electricity.
2. Components containing hazardous materials with their importance factor, I_p, being equal to 1.5: Retention of the continents of these components during and after earthquake shall be certified by the manufacturer. Issuance of such a certificate shall be based on the shake table test or analysis or empirical observations consistent with the design earthquake.
3. Non-active components not containing hazardous material: Issuance of seismic certificate for these components may be performed solely based on analysis. In the design procedure based on Sec. 8.3.1, the $\frac{a}{R_{po}}$ coefficient shall be taken equal to 2.5. If the period of the component, T_p, is equal to or less than 0.06 sec, this coefficient may be taken as equal to unity. Besides, for components located above the ground level, value of R_b shall be taken equal to 1.3.

8.2.5. Successive failure

For nonstructural components having an importance factor of 1.5 and connecting to an essential structure, internal performance and physical relations between different components or their supports and effects of each one on the others shall be investigated, such that failure of other elements does not impair the performance of the considered nonstructural component. Where results of analysis or test have not necessitated another limitation, the clear distance for the sprays of the sprinkler system shall not be less than the values mentioned in Sec. 8.2.5.1.

8.2.5.1. Clear distance between facilities, distribution systems, bearing and sprays of the sprinkler system

The clear distance between adjacent sprays or branches of the sprinkler system shall be 7.5 cm at minimum in the following cases:

1. Stationary systems including their bearings and anchors.
2. Other distribution systems including their bearings and anchors.

Exception: For sprinklers installed using flexible hoses, it is not required to provide the minimum distance mentioned in this section.

8.2.6. Capacity determination based on test or empirical observations

For determining the seismic capacity of nonstructural component and its connections, in lieu of the relations of relevant documents, results of tests on the nonstructural component or similar specimens can be utilized. The test procedure shall be approved by the official in charge beforehand. In any case, the capacity from test shall not be smaller than the seismic demands determined based on Sec. 8.3.1 to 8.3.4.

8.2.7. Characteristics of the seismic response of nonstructural component

The nonstructural components are categorized based on their responses in the two perpendicular horizontal main directions as follows:

Sensitive to acceleration: Nonstructural components anchored or established in stories exposed to damage due to the inertial force or acceleration, shall be categorized as sensitive to acceleration.

Sensitive to acceleration-deformation: Nonstructural components exposed to damage due to acceleration or the inertial force and concurrently due to deformation, shall be categorized as sensitive to acceleration- deformation.

Design of nonstructural components considering their sensitivity to acceleration or acceleration- deformation shall be based on one of the two following procedures:

1. Design of nonstructural component to resist acceleration at the level where it is connected to the main structure. Such nonstructural components shall be designed using Sec. 8.3.
2. Design of nonstructural components for resisting acceleration or the expected lateral displacement. Such nonstructural components shall be designed based on Sec. 8.3.

8.3. Seismic demands of nonstructural components

In this section, three methods are presented for determining the seismic forces applied on nonstructural components. The “equivalent lateral force method” (Sec. 8.3.1) is used for nonstructural components belonging to the seismic design categories D_1 and D_2 (according to Sec. 8.1.4). The “comprehensive interaction method” (Sec. 8.3.2) is necessary for nonstructural components belonging to the seismic design category D_3 and the “nonlinear dynamic analysis method” (Sec. 8.3.4) may be used for all nonstructural components. When using the “comprehensive interaction method”, it is necessary for the sum of the calculated forces on the component not to be smaller than that of the equivalent lateral load method. Otherwise, it is required that the lateral forces of the comprehensive interaction method to be scaled consistently such that their sum is at least equal to the lateral load in the equivalent lateral load method.

8.3.1. The equivalent lateral load method

The horizontal seismic design force, F_p, of the component shall be distributed according to the mass of the component. The redundancy factor, ρ, may be taken as equal to unity. Direction of the force F_p shall be such that it results in the most critical effect on the component, bearings and its connections. If the component has definite main directions, alternatively 100% of the force F_p may be applied in one main direction and 30% of it may be applied concurrently along the orthogonal direction, such that the most severe seismic demands develop in the component.

The horizontal seismic design force is calculated by Eq. (8.1):

$$F_p = 0.4 S_{DS} I_P W_P \left[\frac{a_h}{R_b}\right]\left[\frac{a}{R_{po}}\right] \qquad 8.1$$

It is not necessary that F_p is taken to be larger than the value calculated by Eq. (8.2). In addition, this force shall not be smaller than the value determined by Eq. (8.3)

$$F_P = 1.6 S_{DS} I_P W_P \qquad 8.2$$

$$F_P = 0.3 S_{DS} I_P W_P \qquad 8.3$$

In these equations:
F_p: seismic design force
S_{DS}: Design spectral acceleration parameter at small periods according to the definition of Chapter 3.
I_p: Importance factor of the component according to Sec. 8.1.3.
w_p: Weight of the component in service equal to the maximum weight during service.
a_h: Force amplification factor as a function of the structure's height according to Sec. 8.3.1.1.
R_b: Reduction factor due to the ductility of structure based on Sec. 8.3.1.2.
a: Response factor of the component based on Sec. 8.3.1.3.
R_{po}: Strength factor of the component according to Sec. 8.3.1.4.

8.3.1.1. Height amplification factor, a_h

The height amplification factor for nonstructural component in which the bearing level is the same as or lower than the ground level, is equal to 1. Otherwise, this coefficient may be calculated using Eq. 8.4 or conservatively, using Eq. 8.5.

$$a_h = 1 + a_1\left(\frac{z}{h}\right) + a_2\left(\frac{z}{h}\right)^{10} \qquad 8.4$$

$$a_h = 1 + 2.5\left(\frac{z}{h}\right) \qquad 8.5$$

$a_1 = 1/T_a \leq 2.5$

$a_2 = [1 - (0.4/T_a)^2] \geq 0$

where:

z: Height of the connection point of the component from the base level. In cases where the component is below the base level, z shall be taken as equal to zero. Value of Z/h is not necessary to be assumed to be larger than 1,

h: the average height of the roof relative the base level, and,

T_a: Fundamental period of the support structure equal to the smaller value for lateral vibration along the two main horizontal directions of the structure. Alternatively, the smaller empirical period along the mentioned two directions according to Chapter 4 or 7 may be used. For structures composed of a combination of seismic load bearing systems, the system producing the smallest period T_a shall be considered.

Where the seismic load carrying system is unknown, T_a, may be calculated using Eq. 4.7 by the parameters of period for "all other structural systems".

8.3.1.2 Reduction factor due to ductility of structure, R_b

The reduction factor due to ductility of structure, R_b, is calculated using Eq. 8.6:

$$R_b = [\frac{1.1\,R}{I_e.\Omega_0}]^{\frac{1}{2}} \geq 1.3 \tag{8.6}$$

where:

I_e: Importance factor of the support structure of component based on Chapter 4.

R_u: Response modification factor of the support structure from Table 4.5 or 7.2, and,

Ω_0: Overstrength factor of the support structure of the component according to Table 4.5 or 7.2.

For nonstructural components located at or below the base level, value of R_b shall be taken as unity. If the seismic load bearing system is not included in Table 4.5 or 7.2, R_b for the components located above the ground level shall be taken 1.3, unless the seismic design parameters for the seismic load bearing system are approved by the jurisdiction authority.

Where the lateral load bearing system of the support structure is composed of different systems over height, the smallest value of R_b between the mentioned systems shall be selected. For nonbuilding structures, if according to Table 7.2 there exist various seismic design parameters for a specific system based on its height, value of R_b may be selected using R_u and Ω_0 values for the case of "with increase of the allowable height".

8.3.1.3. Response coefficient of the component, a

The response coefficients of nonstructural component for the architectural components are chosen using Table 8.2 and for the mechanical and electrical components using Table 8.3. In these tables values of the response coefficient of the component, a, for two position in height are presented according to the definition mentioned below the tables.

For the mechanical and electrical components placed on a deck on the support structure, the response coefficient of the component shall be retrieved from the corresponding rows of Table 8.3. For the structure of deck, requirements of the lateral load bearing systems mentioned in Chapters 4 and 7 and Sections 8.7 and 8.7.5 to 8.7.7 shall be followed.

When calculating the response coefficient for distribution systems in Table 8.3, two groups of values, one for the nonstructural component (such as pipe, duct and canal) and the other for its supports are provided.

For the supports of this system, requirements of Sections 8.7.7 and 8.7.8 shall also be satisfied.

8.3.1.4. Strength coefficients of nonstructural component, R_{po}

Values of the strength coefficient of nonstructural component, R_{po}, are given in Tables 8.2 and 8.3.

8.3.1.5. Vertical earthquake force

Nonstructural component along with its supports and connections shall be designed concurrently for the effect of the vertical earthquake force, E_v, determined in Chapter 3 in addition to the horizontal force F_p.

Use of this force for design of the roof panels or access floors is not required.

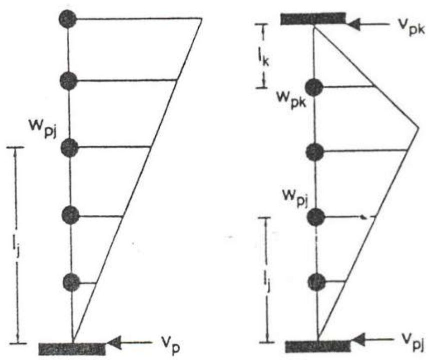
8.3.1.6. Non-seismic forces

When non-seismic forces applied on nonstructural component are larger than F_p, while such forces shall be used in place of F_p for the component design, the detailing requirements and limitations mentioned in this chapter shall be also satisfied.

8.3.2. The comprehensive interaction method

In the simplified interaction method, distribution of mass of the nonstructural component and its flexibility are considered as shown in Fig. 8.1. In this method, the internal forces shall be calculated based on the distribution of lateral forces applied on the component according to Sec. 8.3.3.1 and used for its design.

It is also required for nonstructural component to be checked for the relative seismic displacements based on Sec. 8.3.4, along with the lateral displacements due to other loads.



(a) (b)

Fig. 8.1. Representation of the assumed first mode shape of nonstructural component connected at one or two points.

8.3.2.1. Equivalent lateral force

Distribution of the lateral force is calculated using Eq. 8.7:

$$F_{pi} = \frac{w_{pi} l_i}{\sum_{i=1}^{n'} w_{pi} l_i} V_p \qquad 8.7$$

in which:

F_{pi}: The force applied on the mass center of the ith part of the component. Discretization of the component is performed based on its mass distribution.

w_{pi}: Weight of part i of the component (according to Fig. 8.1)

l_i: Distance from the mass center of part i to the connection point where only one connection point exists (Fig. 8.1a), or distance between the mass center of part i and the lower or upper connection point where two connection points exist (Fig. 8.1b). In the latter case, after determining location of the maximum

lateral displacement of component under a lateral force consistent with the weight of different parts, for the points above the considered location, l_i is equal to the distance of part i from the upper connection point and for the other parts, l_i is equal to the distance of part i from the lower connection point (Fig. 8.1b).
n': Number of the masses of the component considered to be concentrated as Fig. 8.1, and,
V_p: Base shear or sum of the values of the shear forces existing at the component supports calculated using Eq. 8.8.

$$V_p = \frac{S_a}{\lambda / I_p} C_p W_p \qquad 8.8$$

where:
S_a: Spectral acceleration of structure at its fundamental period respective to g (acceleration of gravity) according to Chapter 3,
w_p: Weight of the component in service,
C_p: Amplification factor according to Eq. 8.12 following the requirements of Sec. 8.3.2.2, and,
λ : Parameter calculated using Eq. 8.9:

$$\lambda = \begin{cases} \mu_{eq} T \geq 0.5 \\ \sqrt{2\mu_{eq} - 1} 0.5 > T \geq 0.125 \\ 1 + \frac{33T-1}{25T}(\sqrt{2\mu_{eq} - 1} - 1) 0.125 > T \geq 0.03 \\ 1 T < 0.03 \end{cases} \qquad 8.9$$

where:
T: Fundamental period of structure, and,
μ_{eq}: Equivalent ductility factor determined using Eq. 8.10:

$$\mu_{eq} = \left[\frac{1}{n+n'}\left(\frac{n}{\mu} + \frac{n'}{\mu_p}\right)\right]^{-1} \qquad 8.10$$

in which:
μ: Ductility factor of the support structure according to Eq. 8.11,
μ_p: Ductility factor of the component, Ω_0 =1 and period of the component, T_p, is calculated using Eq. 8.21,
n: Number of stories of the structure, and,
n': Number of parts of nonstructural component as of Fig. 8.1.

$$\mu = \begin{cases} \frac{R_u}{\Omega_0} T \geq 0.5 \\ 0.5\left[(\frac{R_u}{\Omega_0})^2 + 1)\right] \quad T < 0.5 \end{cases} \quad 8.11$$

in which R_u and Ω_0 are respectively the response modification and overstrength factors of the support structure.
The amplification factor C_p in Eq. 8.8 is as follows:

$$C_p = \frac{1}{2\sqrt{\left|\frac{W_p}{W} - \frac{0.0025}{\varphi_0^2}\right|}} \leq 12.5\varphi_0 \quad 8.12$$

where:
W: Effective weight of the structure, and,
φ_0: A variable calculated using Eq. 8.13.

$$\varphi_0 = \frac{Wh_{av}}{\sum_{i=1}^{n} w_i h_i} \quad 8.13$$

in which:
h_i: Height of level i of the structure from the base level,
h_{av}: Average of heights of connection points of the nonstructural component to the structure from the base level, and,
w_i: Effective seismic weight of level i of the structure.
For nonstructural components having more than two connection points, for every part between two adjacent connection points, this method can be used. Limitations of Eqs. 8.2 and 8.3 apply to the sum of the forces F_{pi}.

8.3.2.2. Modified amplification factor

If period of component, T_p, is already calculated or known, instead of the amplification factor C_p according to Eq. 8.12 that is a conservative relation determined at resonance, a modified amplification factor C_m shown in Fig. 8.2 may be used. Parameter b in Fig. 8.2 is calculated using Eq. 8.14.

$$b = \frac{1}{2}\varphi_0\sqrt{W_p/W} \quad 8.14$$

8.3.3. Nonlinear dynamic analysis

In lieu of the forces determined based on Sec. 8.3.1, the nonlinear dynamic analysis method of Chapter 4 may be used for calculation of the design seismic force of nonstructural component.

In such an analysis, if the nonstructural component is not explicitly modeled along with the structure, the seismic design force, F_p, shall be determined using Eq. 8.15:

$$F_p = I_p W_p a_i \left[\frac{a}{R_{po}}\right] \tag{8.15}$$

In the above equation, a_i is the maximum acceleration at the level of installation of the nonstructural component i calculated using the nonlinear dynamic analysis under a suit of at least 7 spectrally consistent earthquake records. Value of a_i shall be the average of the maximum values of acceleration at the installation level of the component from the analyses. The upper and the lower bounds of the force F_p from Eqs. 8.2 and 8.3 shall be followed.

If the mechanical or electrical component is installed on a seismic isolation device, satisfaction of the requirements of Chapter 10 is required.

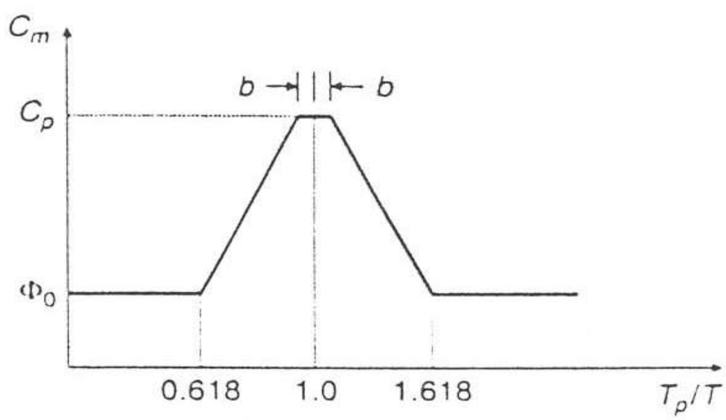


Fig. 8.2 variation of the modified amplification factor with the period ratio.

Table 8.2. Response and behavior factors of architectural components.

Architectural components	a		R_{p0}	Ω_{0p}
	*	**		
Nonstructural walls and interior partitions				
Light frames equal to or shorter than 3 m	**1.0**	**1.0**	**1.5**	**2.0**
Light frames taller than 3 m	1.4	1.4	1.5	**2.0**
Building consisting of reinforced masonry walls	1.0	1.0	1.5	**2.0**
Other walls and partitions	2.2	2.8	1.5	**1.5**
Cantilever members				
Cantilever not-braced or braced to structure below their mass center, such as parapets and	**1.8**	**2.2**	**1.5**	**1.75**
Cantilever braced to structure above its mass center, such as parapets, chimneys and exterior nonstructural walls	1.0	1.0	1.5	**2.0**
Exterior nonstructural wall elements and their connections				
Walls and interior connections of panels	**1.0**	**1.0**	**1.5**	**2.0**
Connection fasteners	2.2	2.8	1.5	**1.0**
Veneers having members and attachments with low deformation capability	1.0	1.0	1.5	**2.0**
Penthouse (when separate from the main frame)				
Lateral load bearing system having a behavior factor larger than 6	-	1.4	2.0	**2.0**
Lateral load bearing system having a behavior factor between 4 and 6	-	2.2	2.0	**1.75**
Lateral load bearing system having a behavior factor lower than 4	-	2.8	2.0	**1.5**
Other systems	-	2.8	1.5	**1.5**
Ceiling	1.0	1.0	1.5	**2.0**
Cabinets installed on the story floors (such as libraries) taller than 1.8 m	1.0	1.0	1.5	**2.0**

Laboratory equipment	1.0	1.0	1.5	**2.0**

Access floor				
Special access floor	**1.0**	**1.0**	**2.0**	**2.0**
Other access floors	2.2	2.8	1.5	**1.75**
Appendages and ornaments	1.8	2.2	1.5	**1.75**
Signs and billboards	1.8	2.2	1.5	**1.75**
Other rigid elements	1.0	1.0	1.5	**2.0**

Other flexible elements				
With members and connections highly deformable	**1.4**	**1.4**	**1.5**	**2.0**
With members and connections intermediately deformable	1.8	2.2	1.5	**1.75**
With members and connections lowly deformable	2.2	2.8	1.5	**1.5**
Stairways and access paths not part of the seismic load bearing system	1.0	1.0	1.5	**2.0**
Connections of stairways and access paths	1.8	2.2	1.5	**1.75**

Table 8.3. Response and behavior coefficients for mechanical and electrical components

Mechanical and electrical components	a		R_{p0}	Ω_{0p}
	*	**		
Air-side HVACR, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing	**1.4**	**1.4**	**2.0**	**2.0**
Wet-side HVACR, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials	**1.0**	**1.0**	**1.5**	**2.0**
Air coolers (fin fans), air-cooled heat exchangers, condensing units, dry coolers, remote radiators, and other mechanical components elevated on integral structural steel or sheet metal supports	**1.8**	**2.2**	**1.5**	**1.75**
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 7	**1.0**	**1.0**	**1.5**	**2.0**
Skirt-supported pressure vessels not within the scope of Chapter 7	**1.8**	**2.2**	**1.5**	**1.75**
Elevator and escalator components	**1.0**	**1.0**	**1.5**	**2.0**
Generators, batteries, inverters, motors, transformers, and other electrical components constructed of high deformability materials	**1.0**	**1.0**	**1.5**	**2.0**
Motor control centers, panel boards, switch gear, instrumentation cabinets, and other components constructed of sheet metal framing	**1.4**	**1.4**	**2.0**	**2.0**
Communication equipment, computers, instrumentation, and controls	**1.0**	**1.0**	**1.5**	**2.0**
Roof-mounted stacks, cooling and electrical towers laterally braced below their center of mass	**1.8**	**2.2**	**1.5**	**1.75**
Roof-mounted stacks, cooling and electrical towers laterally braced above their center of mass	**1.0**	**1.0**	**1.5**	**2.0**
Lighting fixtures	**1.0**	**1.0**	**1.5**	**2.0**
Other mechanical or electrical components	**1.0**	**1.0**	**1.5**	**2.0**
Manufacturing or process conveyors (non-personnel)	**1.8**	**2.2**	**1.5**	**1.75**
Vibration-isolated components and systems [a]				
Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter stops	**1.8**	**2.2**	**1.3**	**1.75**
Spring-isolated components and systems and vibration-isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops	**1.8**	**2.2**	**1.3**	**1.75**
Internally isolated components and systems	**1.8**	**2.2**	**1.3**	**1.75**
Suspended vibration-isolated equipment, including in-line duct devices and suspended internally isolated components	**1.8**	**2.2**	**1.3**	**1.75**
Equipment support structures and platforms				
Support structures and platforms where $T_p/T_a < 0.2$, or $T_p \leq 0.06$ s	**-**	**1.0**	**1.5**	**2.0**
Seismic force-resisting systems with $R > 3$	**1.4**	**1.4**	**1.5**	**2.0**

Seismic force-resisting systems with R ≤ 3	**1.8**	**2.2**	**1.5**	**1.75**
Other systems	**2.2**	**2.8**	**1.5**	**1.5**

Distribution system supports				
Tension-only and cable bracing, cold-formed steel rigid bracing, hot-rolled steel bracing and other rigid bracing	**1.0**	**1.0**	**1.5**	**2.0**
Lateral resistance provided by rods in flexure	**1.8**	**2.2**	**1.5**	**1.75**
Vertical cantilever supports such as pipe tees and moment frames above and supported by a floor or roof	**1.8**	**2.2**	**1.5**	**1.75**

Distribution systems				
Piping in accordance with ASME B31 (2001, 2002, 2008, 2010), including in-line components with joints made by welding or brazing	**1.0**	**1.0**	**3.0**	**2.0**
Piping in accordance with ASME B31, including in-line components, constructed of high- or limited deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings	**1.0**	**1.0**	**2.0**	**2.0**
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high deformability materials, with joints made by welding or brazing	**1.0**	**1.0**	**2.0**	**2.0**
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings	**1.8**	**2.2**	**2.0**	**1.75**
Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics	**1.8**	**2.2**	**1.5**	**1.75**
Duct systems, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing	**1.0**	**1.0**	**2.0**	**2.0**
Duct systems, including in-line components, constructed of high- or limited-deformability materials, with joints made by means other than welding or brazing	**1.0**	**1.0**	**1.5**	**2.0**
Duct systems, including in-line components, constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics	**1.8**	**2.2**	**1.5**	**1.75**
Electrical conduit, cable trays, and raceways	**1.0**	**1.0**	**1.5**	**2.0**
Bus ducts	**1.0**	**1.0**	**1.5**	**2.0**
Plumbing	**1.0**	**1.0**	**1.5**	**2.0**
Pneumatic tube transport systems	**1.0**	**1.0**	**1.5**	**2.0**

[a] Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$ if the nominal clearance (air gap) between the equipment support frame and restraint is greater than 0.25 in. (6 mm). If the nominal clearance specified on the construction

documents is not greater than 0.25 in. (6 mm), the design force is permitted to be taken as F_p.

[b] Overstrength factor as required for anchorage to concrete and masonry.

*Supported at or below ground level

** Supported above ground level on a structure

8.3.4. Relative displacement

Nonstructural components covered by this chapter shall be checked for strength and stability under the design seismic relative displacement, D_{PI}, along with the displacements due to other loads. The design relative displacement, D_{PI}, is calculated using Eq. 8.16:

$$D_{pI} = D_P \times I_P \tag{8.16}$$

in which D_P is the relative displacement between the connection points of nonstructural component to structure, determined using Eqs. 8.17- 8.20.
Effects of the relative seismic displacements shall be appropriately accounted for in combination with the displacements due to other loads.

8.3.4.1. Relative displacement within a structure

Where the nonstructural component is connected at several points along height to the structure, its relative displacement, D_P, in a certain direction between each two adjacent connection levels j and j+1 to the structure, is calculated using Eq. 8.17:

$$D_p = \delta_{j+1} - \delta_j \tag{8.17}$$

In place of Eq. 8.17, D_P can also be determined using Eq. 8.18:

$$D_P = (h_{j+1} - h_j)\frac{\Delta_a}{h_s} \tag{8.18}$$

where:

δ_j and δ_{j+1}: Lateral displacement of structure at levels j and $j+1$ based on Chapter 4,

h_j and h_{j+1}: Height of two successive connection points of the component to the structure relative to the base level,

Δ_a : Allowable relative displacement of story according to chapter 4, and,

h_s : Floor to floor height of the story where the nonstructural component is connected.

The above values of the relative displacement shall be considered also for design of a nonstructural component that passes the seismic isolation level.

8.3.4.2. Relative displacement between structures

For nonstructural components connected to two A and B structures respectively at levels i and j, relative displacement in a certain direction between two connection points of the nonstructural component to the mentioned two systems, D_P, is calculated using Eq. 8.19:

$$D_P = |\delta_i|_A + |\delta_j|_B \tag{8.19}$$

in which δ_i and δ_j are design lateral displacements of the two structures at the levels i and j.

In lieu of using Eq. 8.19, D_P, may be calculated using Eq. 8.20:

$$D_P = \left(\frac{h_i \Delta_a}{h_s}\right)_A + \left(\frac{h_j \Delta_a}{h_s}\right)_B \tag{8.20}$$

The above values of the relative displacements shall also be considered for design of a nonstructural component crossing the separation joint of the structure.

8.4. Period of nonstructural component

Period of nonstructural component including supports and its connection to structure is calculated using Eq. 8.21, on the condition that it is possible to model the component, supports and its connections as a mass-spring single degree of freedom system,

$$T_P = 2\pi \sqrt{\frac{W_P}{K_P \cdot g}} \tag{8.21}$$

where:

T_P : Fundamental period of the component,

W_P : Service weight of the component,

g: Acceleration of gravity,

K_P: Lateral stiffness of nonstructural component at its mass center with accounting for flexibility of the supports and its connections.
Alternatively, the fundamental period, T_P, may be determined using results of tests or substantive and verifiable analysis.

8.5. Anchor of nonstructural component

8.5.1. General requirements of anchor

If the nonstructural component is connected to its support using short expansive anchors, short chemical anchor, or in-situ anchors with low flexibility, the behavior factor R_P in Eqs. 8.1 and 8.7 is set at 1.5 at most. Anchors embedded in concrete or reinforced masonry shall be able to resist the minimum force between the capacity of connection, 1.3 times the calculated force of the connection and the maximum force that can be transferred from the component to the connection. For determining the force of the connection anchor, mounting conditions including effects of eccentricity of forces shall also be considered. Distribution of force on the connection anchors shall be based on the connection stiffness and redistribution of force through the anchors after their yielding.
Nonstructural components shall be anchored to their support structure according to the requirements of Chapter 18 of the 9th (anchoring to concrete) and 8th clauses of the National Building Regulations and other relevant technical regulations.
Connection of component to structure shall not be done with using the frictional resistance due to gravity. Between component and structure, a continuous path of force transfer having sufficient stiffness and strength shall be provided. When such forces govern the members or structural connections in the load transfer path, such members and connections shall be designed for the mentioned forces.
Use of power-actuated anchors to concrete or steel to transfer tension and for masonry structure is only admissible when a seismic resistance certificate based on Sec. 8.2.3 is available. Use of such anchors at the supports of sound proof panels or lay-in panels of ceilings or long elements when the service load of their attachments to concrete is not larger than 0.4 kN and to steel not larger than 1.1 kN, is permissible.

Frictional clips shall not be used to transfer any force in addition to that of earthquake. Use of C-shaped clips is allowed for beams or wide-flange members for hangers, provided to prevent loosening of threaded connections, additional ties or lock nuts are utilized.
Anchoring to masonry shall be conducted such that the capacity of connection of nonstructural component to the anchor does not exceed the yield capacity of the anchor. Otherwise, design force of the anchor shall be taken to be at least 2.5 times the factored force transferred by the connection.

8.5.2. Anchor design requirements

For design of the connections, the force F_P and the relative displacement mentioned in Sec. 8.3 shall be used. Design of connections of nonstructural component to concrete and masonry shall be accomplished based on the requirements of Chapter 18 of the 9th (anchoring to concrete) and 8th clauses of the National Building Regulations and other relevant technical regulations.
Where based on the mentioned regulations the overstrength factor shall be used, the Ω_{0P} values mentioned in Tables 8.2 and 8.3 apply.

8.6. Special requirements for architectural components

8.6.1. General

Architectural components along with their supports and connections, in addition to the requirements of the preceding sections, shall satisfy the requirements of this section.

8.6.2. Special design requirements for suspended components

Components suspended by chain or hanged are such as suspended ceilings, electrical cable trays, ceiling chandelier, ceiling fans and alike. For these components, provided the following conditions are met, satisfaction of the requirements of Sec. 8.3 for force and displacement is not required:

1. The design force of these components is considered to be 1.4 times their service weight pointing downward and simultaneously, a lateral force

equal to 1.4 times their service weight is applied in a horizontal direction having the worst influence.
2. Interaction with the structure and the adjacent components is considered based on Sec. 8.2.5.
3. Connection of the architectural element to the structure has the ability to rotate 360° degrees in the horizontal plane.

8.6.3. Vertical deformation due to rotation of connections

If it is possible for the connections of the cantilever structural members to rotate, the resulting vertical deformation shall be considered in design of the architectural component.

8.6.4. Exterior nonstructural wall and its connections

Exterior nonstructural wall connected to or enveloping the structure, shall be designed such that it resists the relative displacements mentioned in Sec. 8.3.4 and the thermal deformations. Such a component shall be anchored directly or through mechanical connections able to transfer tension to structure, according to the following requirements:

1. Connections of the component shall be able to resist the relative displacement mentioned in Sec. 8.3.4 or 15 mm, whichever is larger.
2. Flexibility of connection to endorse the relative displacement of story and movement of wall in its plane shall be provided by one of the various methods including use of oval hole, flexible (bending) or sliding steel elements, provided that:
    - Connected member has sufficient ductility and rotational capacity to prevent brittle failure at the welding zone.
    - All connection tools such as bolts, nails, anchors, welds and the connection body are designed for their share of the F_P force applied at the mass center of the wall based on Sec. 8.3.

If anchoring is provided using steel straps embedded in concrete or masonry, such straps shall be connected to the reinforcement or be hooked to them and in general they are effectively able to transfer force to the reinforcement or it is confirmed that the preceding failure mechanism is not pull-out of the anchor. When connection of wall to the supports is through the threaded bars

and oval holes, ratio of the clear length to the diameter of bar shall not be lower than 4. Size of the oval holes shall be selected such that it makes no obstacle for the total relative displacement of story along the plane of wall.
When the wall connection resists the relative displacement of story through bending of threaded bars, Eq. 8.22 shall be satisfied:

$$\left(L/d\right)/D_{PI} \geq 0.24 \qquad 8.22$$

in which:
L: Clear length of bar (mm),
D_{PI}: Design seismic displacement (mm), and,
d: Diameter of bar (mm).

8.6.5. Out-of-plane bending

For flat elements, out-of-plane deformation and bending under the forces determined in Sec. 8.3 shall be evaluated. Such values shall not exceed capacity of the components.

8.6.6. False ceiling

False ceilings having areas lower than 15 m^2 are not required to be analyzed for seismic loads provided they are appropriately connected and laterally anchored to the roof and their adjacent wall. In addition, false ceilings consisting of gypsum boards located at the same level that are connected to their perimeter using bolts or nails and confined between walls or horizontal frames anchored to the above roof, are not required to be analyzed for seismic loads.
Weight of the false ceiling, W_P, is equal to the sum of the weights of the roof grid, its segments, lighting systems connected to it and other elements kept by the roof. Such a value shall be considered to be less than 0.2 kN/m^2. The seismic force F_P shall be transferred safely to the structure using connections of the false ceiling. Instead of placing large holes in the vicinity of the sprinklers of the fire extinguishing system of roof, this system may be designed with the whole false ceiling as an integrated unit.

For the sound proof panels or lay-in panels of the false ceiling, in addition to the manufacturer requirements, compliance with the following items is mandatory:

1. Width of the flange of the peripheral angle or channel of the panels shall not be less than 50 mm. When additional peripheral clipses are utilized, their application shall be verified by test. Along both horizontal orthogonal directions, one end of the ceiling mesh shall be attached to the peripheral angle or channel and the other end is distanced clearly at 20 mm in the horizontal plane from the wall such that it can slide freely on the peripheral member.
2. In ceilings being more than 230 m^2 in area, separation joint or full height partitions that cross the false ceiling and divide it into smaller segments each one being smaller than 230 m^2 and having an aspect ratio less than 4 in plane shall be used, except when exact analysis of the false ceiling shows that the clear distance provided on the peripheral members is sufficient for free movement of the ceiling. Each of the divided segments shall satisfy conditions of item 1.

8.6.7. Access floor

The effective weight of the access floor, W_P, is equal to the sum of weights of the flooring system, 100% of the weight of its connected components, and 25% of the weight of the components mounted on it but are not attached to the access floor. The seismic force F_P shall be transferred from the level on the access floor to the support structure. Effect of overturning of the components connected to the access floor shall also be included in its design. Ability of the sliding connections on the pedestals to resist the effects of overturning shall be evaluated. To assess overturning of each pedestal, the maximum concurrent vertical load shall not exceed the share of W_P assigned to the pedestal.

8.6.7.1. Special access floor

An access floor is called special if the following requirements are met in its design:

1. Their connections provided for resisting earthquake forces, including mechanical ties and anchoring to concrete based on Sec. 8.5, are welded or bearing,
2. For resisting and transferring the seismic force, power-actuated fasteners, glue, or friction due to gravity only, is not used,
3. For design of braces of the access floor, effect of buckling of compressional members is considered.
4. Bracing members and pedestals are made of standard structural or mechanical sections having known mechanical properties. Using electrical conduits for this purpose is not allowed.
5. Sub-floor piers designed for axial forces due to earthquake and connected to support pedestals are utilized.

8.6.8. Partitions

Design of partitions shall be based on the requirement of Chapter 6 of Standard 2800. Partitions connected to the roof and any partition taller than 1.8 m shall be braced to the structure in the direction normal to its plane. Such a bracing shall be independent from the lateral bracing of the false ceiling. Distance between the braces shall be such that the value of the horizontal movement of the top of the partition is limited based on Chapters 4 and 7 for the false ceilings or other requirements of this chapter for other roofs.

Requirements of this section are not required to be met for partitions satisfying all of the following conditions:

1. Partitions with their height shorter than or equal to 2.7 m,
2. Partition where the weight of its unit length is not more than 50 daN multiplied by the partition height,
3. Partition with the seismic horizontal load on its surface being not more than 25 daN/m^2 .

8.6.9. Glass cladding and walls

Glass in cladding walls and interior partitions should be designed and installed such that it can resist the relative displacements mentioned in this section without breaking or fall out. If glass is glued to its frame, specific requirements of specialty standards such as ASTM-C1087 shall also apply.

The seismic relative displacement in cladding glass walls and interior glass partitions shall satisfy Eq. 8.23:

$$\Delta_f \geq \max(1.25 \times I \cdot D_P \ , 15 \text{ mm}) \qquad 8.23$$

in which:

Δ_f : Seismic relative displacement at which failure of the glass in cladding walls and interior partitions occurs, which can be determined using the recommendations of AAMA 501.6 or by accomplishing structural analysis.

D_P: Design relative displacement according to Sec. 8.3.4 that shall be calculated between the top and the bottom of the glass element.

I: Importance factor of structure according to Chapter 4.

Fulfilling the requirements of this section is not required for the following cases:

1. Sufficient clear distance exists between the glass and its frame such that glass does not touch the frame at least up to a relative displacement of $1.25\ D_P$. The relative displacement between the top and bottom of glass that results in touch of glass and frame, D_c, is calculated for a rectangular wall glass using Eq. 8.24:

$$D_c = 2c_1(1 + \frac{h_p c_2}{b_p c_1}) \geq 1.25 D_{PI}$$

in which:

h_p : Height of the rectangular glass,

b_p : Width of the rectangular glass,

c_1: Average of the clear distances of the two vertical edges of glass from its frame,

c_2 : Average of the clear distances of the two horizontal edges of glass from its frame.

2. Integral thermoset glass in Function and Risk Category Ⅱ to Ⅳ mounted at an elevation up to 3 m from the adjacent pedestrian floor.
3. Laminated thermoset glass with its interior layers being not thinner than 0.75 mm and braced within a lay-in wall frame. To brace the glass, a waterproof elastic band shall be inserted between the glass edges and its frame where it is in touch with the band at least on a width of 15 mm. Use of other approved bracing systems is also allowable.

8.6.10. Stairways and escape ramps

For seismic design of stairways and escape ramps two approaches are allowed. In the first approach, such elements are considered to be part of the main structure and are modeled and analyzed along with it. In this case, stairway and escape ramps are not counted as nonstructural elements. In the second approach, using appropriate connections, such elements are protected as much as possible from being loaded by the main structure.

When the second approach is followed, the stairways and escape ramps shall be designed such that they can accommodate the seismic relative displacement D_{PI} (Sec. 8.3.4) and diaphragm deformations. Such elements shall be connected to the main structure using appropriate connections having the following characteristics:

1. Sliding joints accommodated with slit or loose holes, sliding bearings having bumpers or restrainers and joints in which displacement is provided for with deformation of steel elements. Such connections shall be designed for D_{PI} but not for a value less than 15 mm. The design shall be accomplished such that the nonstructural component does not lose the support needed for bearing its weight and/or occurrence of impact between the component and the main structure is prevented.
2. Use of sliding bearings is not allowed without bumpers or restrainers.
3. Special metallic bearings shall be designed for the rotation associated with the relative seismic displacement equal to $1.5D_{PI}$ not less than 25 mm. Strength of such metal bearings shall not be limited to bolt shear, weld failure, or other brittle failure modes.

8.6.11. Stair roof and roof-top structures

Structures mounted on the roof not as an extension of the main structural system, shall be designed based on this section. The horizontal seismic design force, F_p , shall be calculated using Sec. 8.3 and the coefficients listed in Table 8.2.

It the roof-top structure is developed as an extension of the main structural system, it shall be analyzed and designed along with other parts of the structure according to Chapter 4.

8.6.11.1 The seismic load bearing systems for roof-top structures

The seismic load bearing system of the roof-top structures shall resemble one of the systems introduced in Tables 4.5 or 7.1. In the utilized structural system, the limitations mentioned in these tables shall be observed. When checking the height limitation, height of the roof-top structure shall be determined from the roof level to the average height of the roof of the structure mounted on the main roof.

When the roof-top structure has a load bearing system belonging to the "other systems" category in the above tables, its height shall not be over 8.5 m.

8.7. Special requirements for mechanical and electrical components

8.7.1. General

In seismic design of the mechanical and electrical components and their supports, the requirements of this section shall be observed. Design of the connections of such components to their supports shall be performed following the requirement of Sec. 8.5 and 8.7.5.

If the mechanical or electrical element is mounted on vibration isolators in the vertical direction (for the service condition of the element), it is necessary to use bumpers in both horizontal directions in the vicinity of the element. If the clear distance between the nonstructural component or its support and the bumper is larger than 6 mm, design force of the nonstructural component shall be increased to $2F_P$. Otherwise, the design force may be assumed to be F_P. In addition, in cases that overturning of the component is possible, vertical restrainers shall be used at the support. Sleeve of the vibration isolator and displacement restrainers shall be of ductile material. To lower the effect of pounding between the bumper and component, bearing pads produced by viscoelastic or similar material having an appropriate thickness may be used.

For light-weight lighting systems, illuminating sign guides and roof fans not connected to pipes or canals and suspended by a means from the structure, provided the following conditions are met, observing the requirements of Sec. 8.7.4 is not necessary:

1. The nonstructural component is designed simultaneously for a horizontal and a vertical force being equal to 1.4 times the component weight each. The horizontal force shall be applied in a direction that makes the most critical condition.
2. Effects of seismic interaction are considered according to Sec. 8.3.3.
3. Connection of the component to the structure is able to rotate 360 degrees in the horizontal plane. For seismic design of mechanical and electrical components, dynamic effects of their elements, contents, and connections shall be noted.

8.7.2. Mechanical components

For the mechanical components having an I_P larger than 1, in addition to observe the requirements of force and relative displacement of Sec. 8.3, considerations shall be provided to eliminate earthquake-induced pounding or to lower its effects. Omitting possibility of pounding for the components made of nonductile material or material that their ductility is reduced at the service condition (such as sub-zero temperatures) is mandatory.

Besides, loading due to other elements connected to such components or the loads due to un-equal displacements between their bearing points shall be considered. For components accommodated with seismic isolation, the relative displacement between the two opposite sides of the seismic isolation system shall be considered in the design.

8.7.3. Issuing seismic certificate for cooling, heating and air ventilation (HVACR) system

Issuance of the seismic certificate for the component of the cooling, heating and air ventilation system that comply with the requirements of chapter 1-10 of the standard ANSI/AHRI-1270-1271, is conducted under the following conditions:

1. For active components, i.e. the components having mechanical motions or containing electric current during service, issuing the certificate is solely allowed based on the shake table test or empirical data,
2. For non-active components, issuing of the certificate based on an analytical study is also permissible. For analysis of such components,

the horizontal seismic design force, F_P, shall be calculated based on Sec. 8.3.1 by considering the ratio a/R_{po} as 2.5. Value of R_b for non-active components located above the base level shall be taken as 1.3. If period of nonstructural component, T_P, is equal to or less than 0.06 sec, a/R_{po} can be considered as unity.

3. Rugged nonstructural component, i.e. those with desirable behavior in the past earthquakes such as machines, turbines, electrical motors, compressors, and pumps; are considered to be approved and do not need a certificate.

8.7.4. Electrical components

For electrical components with their I_P to be larger than 1, in addition to observing the force and relative displacement requirements of Sec. 8.3, the following additional requirement shall also be satisfied:

1. Consideration shall be taken to prevent pounding between adjacent components due to earthquake,
2. In design of the components with their connected utility lines being also connected to another structure (other than the support structure of the component), it is necessary to account for the load applied by such lines to the component,
3. Batteries mounted on racks, shall be braced using strap restrainers to prevent their fall. To prevent damage incurred to the battery boxes, it is necessary to place separators between braces and batteries. Racks shall have sufficient lateral capacity,
4. Interior circuits of dry-type transformers shall have positive connections to their supports in the transformer,
5. Control boxes, computer components and other facilities able to move on rails, shall be provided with stabilizer brakes.
6. Electrical boxes shall follow relevant valid standards. Detailing of the connection of electrical box to the support structure or wall shall be designed to resist earthquake forces and it shall not be left solely to be accomplished by the contractor.

7. If the pertinent properties are not provided by the manufacturer, seismic design of the connections of attachments of the nonstructural components weighting more than 45 kg shall be conducted.
8. Canals, cables trays or other similar components in the electricity distribution system simultaneously connected to structures possible to move relative to each other or are connected to the two opposite sides of seismic isolators, shall be able to resist the relative seismic displacements mentioned in Sec. 8.3.4.

8.7.5. Supports of components

For design of supports of mechanical and electrical components, in addition to observing the force and displacement requirement of Sec. 8.3, the following additional requirements shall also be satisfied:

1. For the supports provided along with the component by the manufacturer, documents shall be presented showing seismic sufficiency of the support through tests performed by the manufacturer. Otherwise, such supports shall be evaluated under the forces and displacements mentioned in this chapter.
2. For components with $I_P = 1.5$, local effects of stress distribution in connection elements shall be investigated.
3. For the components in which the connection elements are manufactured integrally with the component, if the documents and technical specifications confirm adequacy of such elements under earthquake, except of the components with $I_P = 1.5$, it is not needed to re-check them based on the requirements of this chapter.
4. If the considered component contains more than one support point, the relative displacements between such points shall be considered in the component design.
5. Materials used for connections of the component and their elements, shall be consistent with the service conditions of the component, for instance working out low temperatures.
6. Seismic support should be constructed such that it is permanently connected to the component.
7. Necessary strengthening such as using stiffeners and especial conical washers in bolted connections on steel plates shall be conducted for

transferring seismic forces from the component to the structure. If the considered component is provided with seismic certificate, connection elements shall be installed using the certificate based on the manufacturer considerations. If the component does not have a seismic certificate, the mentioned strengthening elements shall be designed.

8. When the component is mounted on a vibration isolator, restrainers in the two horizontal directions shall be provided to restrain them against large displacements. The restrainer shall be also used in the vertical direction if it is necessary to prevent overturning. To lower the pounding effects between the component and the displacement restrainers, viscoelastic pads shall be utilized. Container of the isolators and the restrainers shall be constructed using ductile material.

8.7.9 Support and deck structures of components

Deck and structures of the component supports shall be designed based on this section. The horizontal seismic force, F_P, shall be determined based on Sec. 8.3.1 to 8.3.3 using the parameters listed in Table 8.3. The load bearing systems of decks and support structures shall comply with one of the systems introduced in Table 8.3 and their limitations. It is necessary for the design and constructional details of the mentioned seismic load bearing system to be conducted based on the requirements established in the references or documents of that system.

If $T_P / T_a < 0.2$ or $T_P \leq 0.06$ sec, deck and support system of the components mounted on a building or a non-building structure may be designed considering $a = 1$, $R_{Po} = 1.5$, and $\Omega_o = 2$. When estimating value of T_P for the deck or support structure of the components, effects of mass and stiffness of the supported components shall also be considered.

8.7.7. Supports of the distribution system

Supports of the distribution system are assigned a reduction factor in Table 8.3 due to their shape based on the type of the support system. Vertical and lateral supports of the distribution systems, such as trapeze suspended support systems, shall be designed for seismic forces and relative displacements introduced in Sec. 8.3. Except of the cases mentioned in the following, distribution systems shall be braced for resisting the vertical, lateral and longitudinal seismic forces. The seismic force share of the supports of the distribution system and trapeze suspended support systems shall be determined based in their tributary weight of the distribution system. Share of each support from the weight of the distribution system shall include weight of in-route connections and components.

8.7.8. Distribution systems

In electrical and mechanical distribution systems, in addition to observing the force and relative displacement requirements of Sec. 8.3, the following additional requirements shall also be followed.
In cases where the electrical and mechanical distribution systems rest on common supports, it is necessary to use the strictest of the following requirements for their design.

8.7.9 Electrical distribution systems

Cable conduits smaller than 65 mm in dimension are not required to be designed for forces and displacements of Sec. 8.3. When $I_P = 1.5$, at cross points of the separation joints, appropriate considerations shall be provided to resist the seismic relative displacements.
Cable conduits larger than 65 mm in diameter and connected to terminals, electrical boxes, or other components under seismic displacement, D_{PI}, shall be provided with flexible connections or be designed for the forces and displacements mentioned in Sec. 8.3.

8.7.10 Canals

Air ventilation and other canals shall be designed for the forces and displacements mentioned in Sec. 8.3.
Canals not containing poisonous substances or inflammatory gases and are not utilized for smoke control, provided all of the following conditions are met, are not required to be designed for earthquake:

a) Necessary arrangements to prevent from pounding to other canals or mechanical components are in place or sufficient protection of such components if pounding occurs is provided,
b) A positive connection is provided between the distribution system and the structure,

c) Section area of canal is less than 0.56 m^2,
d) Weight of canal is equal to or less than 300 N/m.

Components located in the path of canal and their weight is larger than 330 N; such as fans, thermal convertors, and moisturizers; shall be laterally braced independent from canal. Such braces shall be designed for the forces mentioned in Sec. 8.3. For design of such braces, weight of the attachments of the canal system, if present, shall be included. If weight of the components located in the path of the canal is less than 330 N and such components do not have independent lateral braces, they shall be connected to the canals using positive mechanical anchors.

Pipes and canals connected to such components shall have sufficient flexibility to resist the seismic relative displacements mentioned in Sec. 8.3.4.

8.7.11 Pipes

For seismic design of pipes, requirements of Chapter 13 shall be followed.

8.7.12 Utility lines

Between the adjacent structures or parts of a structure able to move independently, utility lines shall have sufficient flexibility to resist the relative displacements mentioned in Sec. 8.3.4. Possibility of failure of the utility lines in structures belonging to the Function and Risk Category IV shall be investigated. Vulnerability of underground utility lines and their egress point at the ground surface in the soil type 4 (according to Standard 2800) and a site with $S_{DS} \geq 0.33$ shall be accurately studied.

8.7.13 Elevators and escalators

The support structure of elevators and escalators and their accessories shall satisfy the force and displacement requirements of Sec. 8.3.

Elevators with their service velocity being larger than 45 $m/\min$ shall be provided with seismic switches. The seismic switch shall comply with the requirements of ASME A17.1. When it is not possible to install a seismic switch in the vicinity of a column, the switch shall have two sensors along the two horizontal orthogonal directions and its threshold activation acceleration for the case of its mounting around the base level is set on 0.2 g

and in other cases on 0.5 g. For the facilities where turning the elevator off results in endangering life safety of individuals, use of elevator after activation of the seismic switch is only when permissible that:

1. Velocity of elevator is not more than its service speed.
2. Before embarkation, the elevator is moved downward and then upward for once to make sure it is safe for use.

In addition, at the top and bottom of the elevator cabin and its balancing weight, protector plates shall be installed.

8.4.14 Other electrical and mechanical components

For design of mechanical components containing hazardous substances and those with $F_P = 1.5$ and boilers and pressure tanks using the allowable stress method, the following mechanical properties shall be used:

1. For the components made of ductile materials, such as steel, aluminum or copper, 90% of the minimum characteristic yield stress,
2. For the components made of ductile materials with threaded joints, 70% of the minimum characteristic yield stress,
3. For the component made of nonductile materials such as cast iron, ceramic, or plastic, 10% of the minimum characteristic tensile strength of the material,
4. For the components made of nonductile materials with threaded joints, 80% of the minimum characteristic tensile strength of the material.

Chapter 9
Seismically Isolated Structures

9.1. Introduction

Every isolated structure and every part of it shall be designed and constructed in accordance with the requirements of this section and the applicable requirements of this regulation.

9.1.1. Definitions

The following definitions are applicable only to seismically isolated structures, the subject of the provisions of this Cahpter, and are in addition to the definitions presented in Chapters 2, 3, and 4.

Base level. The first floor of the isolated structure that is placed just above the isolation level.

Displacement restraint system. A set of structural elements that limits lateral displacement of seismically isolated structures during the MCE_R.

Effective damping. The equivalent viscous damping value corresponds to the energy dissipated during cyclic response of the isolation system.

Effective stiffness. The value of lateral force created in the isolation system, or in any of its elements, divided by its corresponding lateral displacement.

Isolation lnterface. The boundary between the upper part of the structure, which is isolated, and the lower part of the structure, which moves rigidly with the ground.

Isolation system. A set of structural elements that includes all the individual isolation devices, all the structural elements that transfer force between elements of the isolation system, and all the connections to other structural elements. In addition, the isolation system includes the wind restraint system and energy dissipation devices. If in order to comply with the requirements of this section displacement restraint systems and devices are used, these items are also considered part of the isolation system.

Isolator device. A structural member of the isolation system that is flexible in the lateral direction and rigid in the vertical direction and allows the large lateral deformations under seismic design loads. An isolator device may be used either as part of, or in addition to, the structural weight supporting system.

Maximum displacement. The maximum lateral displacement used for design of the isolation system. This quantity does not include additional displacement caused by actual and accidental torsion.

The maximum displacement shall be determined separately using upper bound and lower bound properties.

Scragging. Reduction effect in stiffness characteristics during cyclic loading or operation of rubber products, including elastomeric isolators. Part of this deterioration will be recovered over time.

Story drift. The difference between the instantaneous displacement values of the ceiling and the floor in one story.

Total maximum displacement. The total maximum lateral displacement, including the additional displacement caused by actual and accidental torsion, which is used to verify the stability of the isolation system or its elements, design of structure separations, and vertical load testing of isolator unit prototypes. The total maximum displacement shall be determined separately using upper bound and lower bound properties.

Wind restraint system. A set of structural elements that provides restraint to the seismically isolated structure against wind loads. The wind restraint system may be either an integral part of isolation devices or a separate device.

9.1.2. Symbols

Symbols presented in this section are applicable only to seismically isolated structures, the subject of provisions of this section, and are in addition to the definitions presented in Chapters 2, 3, 4.

b	:	The shortest dimension of the structure's plan (m) measured perpendicular to *d*.
B_M	:	Numerical coefficient presented in Table 9.1 corresponding to the effective damping ratio of the isolation system, β_M
C_{vx}	:	Distribution coefficient in height
d	:	The longest dimension of the structure's plan (m) perpendicular to *b*.
D_M	:	Maximum displacement (m) at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 9.5.
D'_M	:	Maximum displacement (m) at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Eq. 9.16.

D_{TM} : Total maximum displacement (m) of an isolation system element, including the translational displacement of the center of rigidity and the displacement component due to torsion in the direction under consideration, as prescribed by Eq. 9.7.

e : Actual eccentricity (m) between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity (m) which is 5% of the largest dimension of the structure's plan in the direction perpendicular to the direction of the seismic force.

E_{loop} : Energy dissipated (kN-m) in an isolator device during a complete loading cycle over the test displacement range between Δ+ and Δ-, which is taken as the closed area of the force-deflection curve loop.

F^+ : Maximum positive force (kN) in an isolator device during a single cycle of prototype testing at the displacement amplitude of Δ+.

F^- : Maximum negative force (kN) in an isolator device during a single cycle of prototype testing at the displacement amplitude of Δ-.

F_x : Lateral seismic force (kN) at level x, as prescribed by Eq. 9.13.

h_i, h_1, h_x : Height (m) above the isolation interface corresponding to level *i*, *l*, or *x*.

h_{sx} : Height of story below level *x*

k_{eff} : Effective stiffness (kN/m) of an isolation device, as prescribed by Eq. 9.17.

k_M : Effective stiffness (kN/m) of the isolation system in the horizontal direction under consideration.

L : Live load effect in Chapter 9.

N : Number of isolator devices.

P_T : Ratio of the effective translational period of the isolation system to its effective torsional period obtained from dynamic analysis or as prescribed by Eq. 9.8, but need not be considered less than 1.0.

r_I : Radius of gyration (m) of the isolation system

R_I	:	Numerical coefficient related to the type of seismic resistant system above the isolation system
T_{fb}	:	Fundamental period (seconds) of the structure above the isolation interface, determined by assuming fixed-base conditions for the structure.
T_M	:	Effective period (seconds) of the seismicslly isolated structure at the displacement D_M in the direction under consideration as prescribed by Eq. 9.7.
V_b	:	Total lateral seismic design force or shear (kN) acting on elements of the isolation system or elements below the isolation system, as prescribed by Eq. 9.9.
V_s	:	Total lateral seismic design force or shear (kN) acting on elements above the base level, as prescribed by Eq. 9.9 and the limits of Section 9.5.4.3.
V_{st}	:	Total unreduced lateral seismic design force or shear (kN) acting on elements above the base level, as prescribed by Eq. 9.11.
W	:	Effective seismic weight (kN) of the structure above the isolation interface, as defined by Section 4.9.4.
W_s	:	Effective seismic weight (kN) of the structure above the isolation interface, as defined by Section 4.9.4, excluding the effective seismic weight (kN) of the base level.
w_i, w_l, w_x	:	Portion of the effective seismic weight (kN) assigned to level i, l, or x.
x_i, y_i	:	Horizontal distances (m) from the center of mass to the i^{th} isolator device in the two horizontal axes of the isolation system.
y	:	Distance (m) between the center of rigidity of the isolation system and the element under consideration, measured in the direction perpendicular to the direction of seismic loading under consideration.
β_{eff}	:	Effective damping ratio of the isolation system, as prescribed by Eq. 9.18.
β_M	:	Effective damping ratio of the isolation system at the displacement D_M, as prescribed by Eq. 9.4.
$\Delta+$	:	Maximum positive displacement (m) of an isolator device during each cycle of prototype testing,

Δ-	:	Maximum negative displacement (m) of an isolator device during each cycle of prototype testing.
λ_{max}	:	Property modification factor for calculating the maximum value of the isolator characteristic of interest, in order to include all the factors that cause changes in the isolator characteristics as defined in Section 9.2.8.4.
λ_{min}	:	Property modification factor for calculating the minimum value of the isolator characteristic of interest, in order to include all the factors that cause changes in the isolator characteristics as defined in Section 9.2.8.4.
$\lambda_{(ae,max)}$	:	Property modification factor for calculating the maximum value of the isolator characteristic of interest, to consider the effects of aging and environmental conditions, as defined in Section 9.2.8.4.
$\lambda_{(ae,min)}$	:	Property modification factor for calculating the minimum value of the isolator characteristic of interest, to consider the effects of aging and environmental conditions, as defined in Section 9.2.8.4.
$\lambda_{(spec,max)}$	:	Property modification factor for calculating the maximum value of the isolator characteristic of interest, to consider the effects of permissible manufacturing variation on the average properties of a group of same-sized isolators, as defined in Section 9.2.8.4.
$\lambda_{(spec,min)}$	:	Property modification factor for calculating the minimum value of the isolator characteristic of interest, to consider the effects of permissible manufacturing variation on the average properties of a group of same-sized isolators, as defined in Section 9.2.8.4.
$\lambda_{(test,max)}$	:	Property modification factor for calculating the maximum value of the isolator characteristic of interest, to consider the effects of heating, rate of loading, and scragging, as defined in Section 9.2.8.4.
$\lambda_{(test,min)}$	:	Property modification factor for calculating the minimum value of the isolator characteristic of interest, to consider the effects of heating, rate of loading, and scragging, as defined in Section 9.2.8.4.

$\sum E_M$	:	Total energy dissipated (kN-m) in the isolation system during a complete response cycle at displacement, D_M.
$\sum \lvert F^{+}{}_{D} \rvert_{max}$	:	Sum of the maximum absolute value of force (kN) on all isolator devices, at a positive displacement equal to D_M.
$\sum \lvert F^{-}{}_{D} \rvert_{max}$	:	Sum of the maximum absolute value of force (kN) on all isolator devices, at a negative displacement equal to D_M.

9.2. General Design Requirements

9.2.1. Importance Factor

All parts of the structure, including the structure above the isolation system, are assigned a risk category in accordance with Table 4.3. The importance factor, I_e, for a seismically isolated structure, regardless of the assigned risk category, is considered equal to 1.0.

9.2.2. Configuration

If the horizontal configuration of an isolated structure above the isolation system in accordance with Table 4.2(a), has a Torsional Irregularity Ratio (TIR) greater than 1.4, or if it has a vertical structural irregularity of type A, B, C or D, in accordance with Table 4.1, it should be considered to have a structural irregularity, as defined in Table 12.3 2.

Each isolated structure is considered to have a structural irregularity, If its structural horizontal configuration above the isolation system in accordance with Table 4.2(a), has a Torsional Irregularity Ratio (TIR) greater than 1.4, or if it has a vertical structural irregularity of type A, B, C or D.

9.2.3. Redundancy Factor

Based on requirements of Section 4.7, a redundancy factor, ρ, is assigned to the structure above the isolation system. For seismically isolated structures that do not have structural irregularity as defined in Section 9.2.2, the redundancy factor, ρ, may be considered to be equal to 1.0.

9.2.4. Isolation system

9.2.4.1. Environmental Conditions

In addition to the requirements for vertical and lateral loads caused by wind and earthquake, the isolation system shall have the necessary characteristics to deal with other environmental conditions, including aging effects, creep, fatigue, operating temperature, and exposure to moisture or harmful substances.

9.2.4.2. Wind Load

Isolated structures shall be able to withstand design wind loads at all levels above the isolation interface. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to the story drifts of the structure above the isolation interface, in accordance with Section 9.5.6.

9.2.4.3. Fire Resistance

The resistance of the isolation system against fire shall be provided at least equal to degree of protection as the fire resistance required for the columns, walls, or other gravity-bearing elements in the same area of the structure.

9.2.4.4. Lateral Restoring Force

The configuration of the isolation system shall be such that, for both upper bound and lower bound isolation system characteristics, it can produce a restoring lateral force at the corresponding maximum displacement that is at least 0.025W greater than the lateral force to 50% of the corresponding maximum displacement.

9.2.4.5. Displacement Restraints

Configuration of the isolation system, shall not be such that an obstacle limits lateral displacement caused by Risk-Targeted Maximum Considered Earthquake (MCE_R) to a value less than the total maximum displacement, D_{TM}, unless all the following criteria are considered in design of the seismically isolated structure:

1. Response of the structure to MCE_R shall be determined in accordance with the dynamic analysis requirements of Section 6.9, taking into account the nonlinear characteristics of the isolation system and the structure above the isolation system.
2. The ultimate capacity of the isolation system and strauctural elements placed below the isolation system shall exceed the strength and displacement required for the MCE_R response.
3. Stability and ductility demand for the structure above the isolation system shall be checked and ensured to MCE_R.
4. The displacement restraint shall not be activated before the isolation system reaches the value of $0.6D_{TM}$.

9.2.4.6. Stability

Each element of the isolation system shall be designed in such a way that to be stable under the design vertical load, at a horizontal displacement equal to D_{TM}. The design vertical load shall be determined for the maximum vertical load using load combination 2 of Section 9.2.7.1, and for the minimum vertical load using load combination 3 of Section 9.2.7.1.

9.2.4.7. Overturning

The safety factor for overall structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. For overturning calculations, seismic forces shall be based on MCE_R ground motions, and W shall be used for the vertical restoring force.

Local uplift is not allowed in any elements, unless it is shown that the resulting deflections do not cause overstress or instability of the isolating devices or other elements of the structure.

9.2.4.8. Inspection and Replacement

All of the following items shall be considered as part of the long-term inspection and replacement program:

1. For inspection and replacement, access to all components of the isolation system shall be provided.

2. Before the issuance of the certificate of occupancy for the seismically isolated structure, a complete set of final inspections by Registered Design Professional (RDP) shall be performed on the isolated parts of the structure and the components that cross the isolation interface. These observations shall confirm the conditions of free and unhindered movement of the structure up to the total maximum displacement, and for the components cross the isolation interface, the possibility of accomodating this movement until the maximum total displacement shall be provided.
3. A monitoring, inspection and maintenance plan of the isolation system shall be provided for the seismically isolated structures by RDP who is responsible for the design of the isolation system.
4. Making any structural change, repair or retrofitting at the isolation system interface, including the components that cross the isolation interface, shall be performed under supervision of a RDP.

9.2.4.9. Quality control

A quality control testing program for isolation devices shall be developed by the RDP who is responsible for the design of the structure, incorporating the requirements of production testing of Section 9.8.5.

9.2.5. Structural System

9.2.5.1. Horizontal Distribution of Force

A horizontal diaphragm or other structural elements shall provid continuity of the elements above the isolation interface shall have adequate strength and ductility to transfer forces from one part of the structure to the other parts.

9.2.5.2. Minimum Building Separations

Minimum separations between the isolated structure and retaining walls or other permanent obstacles around the building, shall not be less than the total maximum displacement.

9.2.5.3. Nonbuilding Structures

Seismic design and construction of nonbuilding structures shall be done in accordance with the requirements of Chapter 7 using design forces and displacements determined in accordance with Sections 9.5 or 9.6.

9.2.5.4. Steel Ordinary Concentrically Braced Frames

Steel ordinary concentrically braced frames may be used in seismically isolated structures as the seismic force-resisting system assign to Seismic Design Category D_1 up to a maximum height of 50 m, provided that all the following design requirements are met:

1. The value of R_I, as defined in Section 9.5.4, is equal to 1.0.
2. The total maximum displacement (D_{TM}), as defined in Eq. 9.7 shall be increased by a factor of 1.2.

9.2.5.5. Connections of the isolation system

Connections of the moment-resisting frame of structural steel elements of the seismic isolation system, which is located below the base level, are permitted to conform to the requirements of ordinary steel moment frames.

9.2.6. Structural Elements and Nonstructural Components

Parts of an isolated structure, permanent nonstructural components and their attachments, and the appendages to permanent equipment supported by a structure shall be desiged to resist seismic forces and displacements in accordance with this section and the related requirements of Chapter 8.

9.2.6.1. Components at or above the Isolation lnterface

Elements of seismically isolated structures and nonstructural components, or parts thereof, which are placed at or above the isolation interface, shall be designed to resist a total lateral seismic force equal to the maximum dynamic response of the elements or components under consideration, obtained from a response history analysis.

Exception: Elements of the seismically isolated structures and non-structural components or parts thereof, which are designed to resist seismic forces and displacements, as defined in Chapter 4 and 8 as appropriate, does not need to comply with the provision of this section.

9.2.6.2. Components Crossing the Isolation lnterface

Elements of seismically isolated structures and nonstructural components, or parts thereof, which cross the isolation interface, shall be designed to withstand the total maximum displacement, and be able to accommodate in long term, any permanent residual displacement.

9.2.6.3. Components below the Isolation lnterface

Elements of seismically isolated structures and nonstructural components, or parts thereof, which are placed below the isolation interface, shall be designed and constructed in accordance with the requirements of Chapter 4 and Chapter 8.

9.2.7. Seismic Load Effects and Load Combinations

All elements of isolated structure, including those that are not part of the seismic force-resisting system, shall be designed using the seismic load effects of Section 2.2.3 and the additional load combinations of Section 9.2.7.1 for the design of the isolation system and for testing of prototype isolator devices.

9.2.7.1. Vertical load combination for isolator device

The average, maximum and minimum vertical load for each type of isolation device shall be determined through the application of horizontal seismic forces, Q_E, caused by MCE_R ground motions and the proper vertical load combinations as follows:

1. Average vertical load: load corresponding to $1.0D + 0.5L$, where D is the dead load, and L is the live load as defined by the Standard 2800;

2. Maximum vertical load: load corresponding to $1.2D + E_V + E_h + L + 0.15S$, where S is the snow load, E_h is the horizontal seismic load, and $E_v = 0.12\ S_{MS}D$ is the effect of the vertical seismic load, in which S_{MS} is the value of the spectral acceleration in MCE_R (in g-sec) for a damping ratio of 5% at short period of 0.2 sec.
3. Minimum vertical load: load corresponding to $0.9D - E_V + E_h$, where the effect of vertical seismic load is considered as defined in item 2.

9.2.8. Isolation system characteristics

9.2.8.1. Types of the Isolation System Components

All components of the isolation system shall be classified and grouped in accordance with common type and size of the isolator device, and common type and size of the damping device that complement, if these devices are also part of the isolation system.

9.2.8.2. Nominal characteristics of the isolator device

The nominal characteristics of the isolator device shall be determined based on the average characteristics in three cycles of prototype testing specified by item 2 in Section 9.8.2.2. Variation in characteristics of the isolator device with respect to vertical load may be established by a single deformation cycle, by averaging the characteristics determined using three vertical load combinations, defined in Section 9.2.7.1, at each required displacement, if requirements of Section 9.8.2.2 should be considered.

Exception: If the measured values of effective stiffness and effective damping for the isolator device under the load combination 1 of Section 9.2.7.1 differs by less than 15% from the average of the measured values for the three vertical load combinations of Section 9.2.7.1, then nominal design characteristics may be determined only based on the load combination 1 of Section 9.2.7.1.

9.2.8.3. Bounding Properties of isolation system components

Bounding properties of isolation system components shall be determined for all types of isolation system components. Bounding properties shall include changing in all characteristics of the following components:

1. Evaluation of changing in characteristics of the prototype isolator device through prototype testing, item 2 of Section 9.8.2.2, caused by necessary changes in vertical test load, rate of test loading or velocity effect, effects of temperature increase during cyclic motion, history of loading, scragging (temporary degradation of mechanical properties over successive cycles), and other potential sources of change, measured by prototype testing.
2. Permissible tolerances in production characteristics, which provided for the acceptability of production isolator devices based on the requirements of Section 9.8.5, and,
3. Including aging and environmental effects, including creep, fatigue, contamination, operating temperature and duration of exposure to that temperature and wear during the life of the structure.

9.2.8.4. Property Modification Factors

Maximum and minimum property modification (λ) factors shall be used to account for the variation of the nominal design parameters for each type of isolator device, to consider the effects of heating caused by cyclic dynamic motion, loading speed, scragging and recovery, change in production bearing characteristics, temperature, aging, environmental exposure, and contamination. When the specific information of the quality test by the manufacturer, in accordance with Section 9.8, is approved by RDP, these data can be used to determine the property modification factors, and there is no need to apply the maximum and minimum limits of Eq. 9.1 and Eq. 9.2. When the specific quality test information in accordance with Section 9.8 is not approved by RDP, the maximum and minimum limits of Eq. 9.1 and Eq. 9.2 shall apply.

Propert modification (λ) factors, shall be determined for each type of isolation device, and when applied to nominal design parameters, in the range of displacement required from $\pm 0.5D_M$ up to and including the maximum displacement, $\pm D_M$, shall envelope the hysteretic response of the structure. Property modification factors for environmental conditions may be obtained from data in which the requirements of similarity in Section 9.8.2.7 are not necessarily met.

For each type of isolator device, the maximum property modification factor, λ_{max}, and the minimum property modification factor, λ_{min}, shall be determined in accordance with Eq. 9.1 and 9.2, respectively:

$$\lambda_{max} = \left(1 + \left(0.75 \times \left(\lambda_{(ae,max)} - 1\right)\right)\right) \times \lambda_{(test,max)} \times \lambda_{(spec,max)} \geq 1.8 \qquad 9.1$$

$$\lambda_{min} = \left(1 - \left(0.75 \times \left(1 - \lambda_{(ae,min)}\right)\right)\right) \times \lambda_{(test,min)} \times \lambda_{(spec,min)} \leq 0.6 \qquad 9.2$$

where $\lambda_{(ae,max)}$ and $\lambda_{(ae,min)}$ are property modification factors for calculating the maximum and minimum values of the isolator characteristic of interest respectively, to consider the effects of aging and environmental conditions; $\lambda_{(test,max)}$ and $\lambda_{(test,min)}$ are property modification factors for calculating the maximum and minimum values of the isolator characteristic of interest respectively, to consider the effects of heating, rate of loading, and scragging; $\lambda_{(spec,max)}$ and $\lambda_{(spec,min)}$ are property modification factors for calculating the maximum and minimum values of the isolator characteristic of interest respectively, to consider the effects of permissible manufacturing variation on the average properties of a group of same-sized isolators;

Exception: If the prototype isolator testing is performed on a full-scale specimen that meets the dynamic test conditions of Section 9.8.2.3, then the values of the property modification factors shall be determined based on the test data, and there is no need to apply the upper and lower limits of Eq. 9.1 and Eq. 9.2.

9.2.8.5. Upper and Lower Bounds of Force-Deflection Behavior of Isolation System Components

For each type of isolation system components, a mathematical model of upper bound force-deflection (cyclic) behavior shall be developed. Upper bound force-deflection behavior of isolation system components, which are basically hysteretic devices (e.g., isolation devices) shall be modeled by using the maximum values of the isolation characteristics determined by the property modification factors in Section 9.2.8.4. Upper bound force-deflection behavior of isolation system components that are essentially viscous devices (e.g., supplementary viscous dampers) shall be modeled in accordance with the requirements of Chapter 10 for such devices.

For each type of isolation system components, a mathematical model of lower bound force-deflection (cyclic) behavior shall be developed. Lower bound force-deflection behavior of isolation system components, which are basically hysteretic devices (e.g., isolation devices) shall be modeled by using the minimum values of the isolation characteristics determined by the property modification factors in Section 9.2.8.4. Lower bound force-deflection behavior of isolation system components that are essentially viscous devices (e.g., supplementary viscous dampers) shall be modeled in accordance with the requirements of Chapter 10 for such devices.

9.2.8.6. Characteristics of Isolation System at Maximum Displacements

The effective stiffness, k_M, of the isolation system at the maximum displacement, D_M, shall be determined considering both upper bound and lower bound force-deflection behavior of individual isolation devices, as follows:

$$k_M = \frac{\sum|F_M^+| + \sum|F_M^-|}{2D_M} \qquad 9.3$$

The effective damping, β_M, of the isolation system at the maximum displacement, D_M, shall be determined considering both upper bound and lower bound force-deflection behavior of individual isolation devices, as follows:

$$\beta_M = \frac{\sum E_M}{2\pi k_M D_M^2} \qquad 9.4$$

where $\sum E_M$ is the total energy dissipated (kN-m) in the isolation system during a complete cycle of response at the maximum displacement, D_M; $\sum F^+_M$ is the sum of the absolute value of the force (kN) at a positive displacement equal to D_M for all isolator devices; and $\sum F^-_M$ is the sum of the absolute value of the force (kN) at a negative displacement equal to D_M for all isolator devices.

9.2.8.7. Upper and Lower Bounds of Isolation System Characteristics at Maximum Displacement

The analysis of the isolation system and structure shall be perform separately for upper bound and lower bound properties, and the governing case for each response parameter of interest shall be used for design. In addition, the analysis shall comply with all of the following:

1. For the equivalent lateral force procedure, and for determining the minimum forces and displacements for dynamic analysis, the following variables shall be determined independently for the upper bound and lower bound isolation system characteristics: k_M and β_M per Section 9.2.8.6 (Eq. 9.3 and Eq. 9.4), D_M per Section 9.5.3.1 (Eq. 9.5), T_M per Section 9.5.3.2 (Eq. 9.6), D_{TM} per Section 9.5.3.3 (Eq. .7), V_b per Section 9.5.4.1 (Eq. 9.9), and V_s and V_{st} per Section 9.5.4.2 (Eqs. 9.10 and 9.11).
2. Limitations on V_s, determined in Section 9.5.4.3, shall be evaluated indepedently for both upper bound and lower bound isolation system characteristics, and the most adverse requirement shall govern.
3. For the equivalent lateral force procedure and in order to determine the minimum story shear forces for response spectrum analysis, the load distribution at height of the structure from Section 9.5.5 shall be determined separately for upper bound and lower bound isolation system characteristics. This evaluation

requires independent calculation of F_1, F_x, C_{vx} and k per Eq. 9.12 through Eq. 9.15, respectively.

9.3. Seismic hazard

9.3.1. Spectral Response Acceleration Parameters and Response Spectrum

The MCE_R spectral response acceleration parameters (S_{MS} and S_{M1}) and The MCE_R spectrum shall be determined in accordance with the requirements of chapters 2 and 3.

9.3.2. Ground motions for response history analysis

In cases where response history analysis in accordance with Section 9.6.3.4 is used to design seismically isolated structure, the provisions of Section 3.10 shall be applied except that in liue of the requirements of Section 4.12.2, the period range shall be determined in accordance with the following:
Range of the period, shall be determined corresponding to the vibration periods that effectively contribute to lateral dynamic response of the structure. The upper bound of this period range shall be greater than or equal to $1.25T_M$, which is determined using lower bound isolation system characteristics. The lower bound of this period range shall be determined that includes at least the number of modes required to achieve 90% mass participation in each principal horizontal direction and shall not exceed T_{fb}. When the vertical response is considered in the analysis, the lower bound period of the period range used to correct the vertical components of the ground motion need not be taken as less than either of the following two values: 0.1 sec, or the lowest period at which an effective mass contribution occurs in the vertical direction.

9.4. Analysis Procedure Selection

Seismically isolated structures, except those defined in Section 9.4.1, shall be designed using the dynamic procedures of Section 9.6. In cases where supplementary viscous dampers are used, the response history analysis procedures of Section 9.4.2.2 shall be used.

9.4.1. Equivalent Lateral Force procedure

The equivalent lateral force procedure of Section 9.5 is allowed to be used for design of a seismically isolated structure, provided that all of the following conditions are met. These requirements shall be evaluated separately for upper bound and lower bound isolation system characteristics, and the more restrictive requirement shall be govern.

1. The structure is located on a Site Class type I, type II, or type III.
2. The effective period of the isolated structure at the maximum displacement, D_M, is less than, or equal to 5.0 sec.
3. The structure above the isolation interface has four stories or less, or structural height measured from the base level is less than 20 m.

   Exception: If there is no tension or uplift on the isolators, these limits may be exceeded.
4. The effective damping of the isolation system at the maximum displacement, D_M, is less than or equal to 30%.
5. The effective period of the isolated structure, T_M, which is determined by a rational modal analysis, is greater than three times the elastic period of the structure above the isolation system assuming a fixed base.
6. The structure above the isolation system does not have any structural irregularity, as defined in Section 9.2.2.
7. The isolation system meets all of the following criteria:

   a) The effective stiffness of the isolation system at the maximum displacement is greater than one-third of the effective stiffness at 20% of the maximum displacement.
   b) The isolation system has the ability to produce the restoring force, as specified in Section 9.2.4.4.
   c) The isolation system does not limit maximum displacement resulting from earthquake to a value less than the total maximum displacement, D_{TM}.

9.4.2. Dynamic Procedures

The dynamic procedures of Section 9.6 may be used as specified in this section.

9.4.2.1. Response Spectrum Analysis Procedure

It is permitted to use the response spectrum analysis procedure to design a seismically isolated structure only if the structure, site and isolation system meet items 1, 2, 3, 4 and 6 of the criteria of Section 9.4.1.

9.4.2.2. Response History Analysis Procedure

The response history analysis procedure may be used to design any seismically isolated structure. This procedure shall be used for design of all seismically isolated structures that do not comply with the criteria of Section 9.4.2.1.

9.5. Equivalent Lateral Force Procedure

9.5.1. General

In cases where the equivalent lateral force procedure is used to design seismically isolated structures, the requirements of this section shall be applied.

9.5.2. Deformation Characteristics of the Isolation System

Minimum lateral earthquake design displacements and forces on seismically isolated structures, shall be determined based on the deformation characteristics of the isolation system. If a wind restraint system is used to comply with the requirements of this Regulation, its effects shall be included in the deformation characteristics of the isolation system. The deformation characteristics of the isolation system shall be determined based on proper prototype tests that are performed in accordance with Section 9.8, and the property modification factors shall be specified in accordance with Section 9.2.8.4.

The analysis of the isolation system and structure shall be performed separately for upper bound and lower bound isolation characteristics, and the more critical conditions governing each response parameter of interest shall be used for design.

9.5.3. Minimum Lateral Displacements Required for Design

The isolation system shall be designed and constructed in such a way that, in the most critical direction of horizontal response, it has a minimum deflection at the maximum displacement, D_M, determined using upper bound and lower bound isolation characteristics, determined as follows:

$$D_M = \frac{gS_{M1}T_M}{4\pi^2 B_M} \quad 9.5$$

where g is the acceleration of gravity; S_{M1} (g-sec) is the spectral acceleration at MCE_R for 5% damping ratio at the period of 1 sec, as determined in Chapter 3; T_M (sec) is the effective period of the seismically isolated structure at the displacement, D_M, in the direction of interest, as prescribed by Eq. 9.6; and, B_M is a numerical coefficient presented in Table 9.1 for the effective damping ratio of the isolation system β_M at the displacement, D_M.

9.5.3.2. Effective Period at the Maximum Displacement

The effective period of the isolated structure, T_M, at the maximum displacement, D_M, shall be determined using upper bound and lower bound deformation characteristics of the isolation system as follows:

$$T_M = 2\pi\sqrt{\frac{W}{K_M g}} \quad 9.6$$

where W is the effective seismic weight of the structure above the isolation interface, as defined in Section 4.9.4; k_M is the effective stiffness of the isolation system (kN/m), at the maximum displacement D_M, as prescribed in Eq. 9.3; and, g is the acceleration of gravity.

Table 9.1 Damping Factor, B_M

Effective Damping ratio, β_M (percentage of critical) [1, 2]	B_M Factor
≤ 2	0.8
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
≥ 50	2.0

[1] The damping factor is based on the effective damping ratio of the isolation system determined based on the requirements of Section 9.2.8.6.

[2] The damping factor, for other effective damping values than those given in Table, shall be determined by using a linear interpolation.

9.5.3.3. Total Maximum Displacement

The total maximum displacement, D_{TM}, of elements of the isolation system shall include additional displacement caused by actual and accidental torsion, determined based on the spatial distribution of the lateral stiffness of the isolation system and the most inappropriate position of eccentric mass. For elements of an isolation system, the total maximum displacement, D_{TM}, shall not be considered less than the value prescribed as follows:

$$D_{TM} = D_M \left[1 + \left(\frac{y}{P_T^2} \right) \frac{12e}{b^2 + d^2} \right] \tag{9.7}$$

where D_M is the displacement at the center of rigidity of the isolation system in the direction of interest, as prescribed by Eq. 9.5; y is the distance between the centers of rigidity of the isolation system and the element of interest measured perpendicular to the direction of the seismic load under investigation; e is the actual eccentricity measured in plan, between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, which is equal to 5% of the longest plan dimension of the structure perpendicular to the direction of force of interesting; b is the shortest plan dimension of the structure measured perpendicular to d; d is the longest plan dimension of the structure; and, P_T is the ratio of the effective translational period of the isolation system to the effective torsional period of the isolation system determined through dynamic analysis or as prescribed by Eq. 9.8, but need not be considered less than 1.0.

$$P_T = \frac{1}{r_1}\sqrt{\frac{\sum_N^{i=1}(x_i^2 + y_i^2)}{N}} \qquad 9.8$$

where x_i, y_i are horizontal distances from the center of mass to the i^{th} isolator device along both horizontal axes of the isolation system; N is the number of isolator devices; and r_I is the radius of gyration of the isolation system.

The total maximum displacement, D_{TM}, shall not be taken as less than $1.15D_M$.

9.5.4. Minimum Lateral Forces Required for Design

9.5.4.1 Isolation System and Structural Elements below the Base Level

The isolation system, the foundation, and all structural elements placed below the level of the foundation shall be designed and constructed to withstand a minimum lateral seismic force, V_b, using all of the requirements for a nonisolated structure, calculated using both upper bound and lower bound isolation system characteristics, as follows:

$$V_b = K_m D_M \qquad 9.9$$

where k_M (kN/m), is the effective stiffness of the isolation system at the displacement D_M, as prescribed by Eq. 9.3; and, D_M (m) is the maximum displacement at the center of rigidity of the isolation system in the direction of interest, as prescribed by Eq. 9.5.

V_b shall not be considered less than the maximum force created in the isolation system at any displacement up to and including the maximum displacement, D_M, as defined in Section 9.5.3.

Overturning loads on elements of the isolation system, the foundation, and structural elements placed below the base level, caused by the lateral seismic force V_b, shall be determined based on the force distribution in height in accordance with Section 9.5.5, except that the unreduced lateral seismic design force, V_{st}, shall be used in lieu of V_s, in Eq. 9.13.

9.5.4.2. Structural Elements Located above the Base Level

The structure above the base level shall be designed and constructed using all of the requirements for a nonisolated structure, for a minimum shear force, V_s. This force is determined using both upper bound and lower bound isolation system characteristics, as follows:

$$V_s = \frac{V_{st}}{R_1} \qquad 9.10$$

where R_I is a numerical coefficient corresponding to the type of seismic force resisting system located on the top of the isolation system; and, V_{st} is the total unreduced lateral seismic design force or shear on elements above the base level, as prescribed by Eq. 9.11.

The R_I factor shall be determined based on the type of the seismic force resisting system used for the structure above the base level in the direction of interest as three-eighths of the R value presented in Table 4.5. It should not be considered greater than 2.0 or smaller than 1.0.

Exception: It may be used the R_I value greater than 2.0, provided that the strength of the structure above the base level in the direction of interest, which is determined by nonlinear static analysis at a roof displacement corresponding to a maximum story drift, is not less than $1.1V_b$. The maximum story drift is lesser of the MCE_R drift or $0.015h_{sx}$.

The total unreduced lateral seismic force or shear on elements above the base level shall be determined using upper bound and lower bound isolation system characteristics, as follows:

$$V_{st} = V_b \left(\frac{W_s}{W}\right)^{(1-2.5\beta m)} \qquad 9.11$$

where W (kN), is the effective seismic weight of the structure above the isolation interface, as defined in Section 4.9.4; and, W_s (kN), is the effective seismic weight of the structure above the isolation interface, as defined in Section 4.9.4, excluding the effective seismic weight (kN) of the base level.

When the average distance from the top of the isolator to the underside of the base level above the isolators is greater than 90 cm, the effective seismic weight W_s in Eq. 9.11 shall be considered equal to W.

Exception: Isolation systems whose hysteretic behavior is associated with a sudden change in transition from pre-yield/slip to post-yield/slip behavior, the exponent expression $(1–2.5\beta_M)$ in Eq. 9.11, shall be replaced by $(1–3.5\beta_M)$.

9.5.4.3. Limits on V_s

The value of V_s shall not be less than any of the following:

1. The lateral seismic force required by Section 4.10.4 for a fixed-base structure of the same effective seismic weight, W_s, and a period equal to the period of the isolation system. T_M, which is determined by the upper bound properties;
2. The base shear corresponding to the factored wind load; and,
3. The lateral seismic force, V_{st}, determined using Eq. 9.11, where V_b is equivalent to the force required for full activation of the isolation system. The value of V_b is the greater of the upper bound properties or
   a) 1.5 times the nominal properties for the yield level of a softening system
   b) The ultimate capacity of a restraint system against the wind,
   c) The frictional force of the initiation of motion in a sliding system, or,
   d) Force at zero displacement of a sliding system following dynamic motion during a complete cycle to D_M.

9.5.5. Vertical Distribution of Force

The lateral seismic force V_s, determined using upper bound and lower bound isolation system characteristics, shall be distributed over the height of the structure above the base level, using the following equations:

$$F_1 = \frac{(V_b - V_{st})}{R_1} \qquad 9.12$$

$$F_x = C_{vx} V_s \qquad 9.13$$

$$C_{vx} = \frac{w_x\, h_x^k}{\sum_{i=2}^{n} w_i h_i^k} \qquad 9.14$$

$$k = 14\beta_M T_{fb} \qquad 9.15$$

where F_1 (kN), is lateral seismic force induced at level 1, i.e. base level; F_x (kN), is lateral seismic force induced at level x level, where x is greater than 1; C_{vx} is vertical distribution factor; V_s is the total lateral seismic design force or shear on elements above the base level, as prescribed by Eq. 9.10, with the limits of Section 9.5.4.3; w_i w_x are portions of W_s that is located at or assigned to level i or x; h_i h_x are heights above the isolation interface of level i or x; and, T_{fb} is the fundamental period of the structure above the isolation interface determined using a rational modal analysis, assuming fixed-base conditions.

Exception: Instead of using Eq. 9.10 and Eq. 13.9, the lateral seismic force F_x, may be determined by the average value of the force at level x in the direction of interest, using the results of a simplified stick model of the building and a lumped representation of the isolation system using response history analysis scaled to V_b/R_I at the base level.

9.5.6. Drift Limits

The maximum story drift for the structure above the isolation system shall not exceed the value of $0.015h_{sx}$. The story drift shall be determined using Eq. 4.16, in which C_d for the isolated structure is replaced by R_I, as defined in Section 9.5.4.2.

9.6. Dynamic Analysis Procedures

9.6.1. General

In cases where dynamic analysis is used for the design of seismically isolated structures, the requirements of this section shall be applied.

9.6.2. Modeling

The mathematical models for the isolated structure, including the isolation system, the seismic force resisting system, and other structural elements, shall be in accordance with Section 4.9 and also the requirements of Sections 9.6.2.1 and 9.6.2.2.

9.6.2.1. Isolation System

The isolation system shall be modeled in accordance with Section 9.2.8, using deformation characteristics. The lateral Displacements and forces shall be determined separately for upper bound and lower bound isolation system characteristics, as defined in Section 9.2.8.5. The isolation system shall be modeled with sufficient detail to take into account all of the following:

1. Spatial distribution of isolation devices;
2. Translation in both horizontal directions, and torsion of the structure above the isolation interface considering the most detrimental location of eccentric mass;
3. Overturning and uplift forces on each isolator devices; and,
4. Effects of vertical load, bilateral load, and/or the rate of loading if the force-deflection characteristics of the isolation system depend on one or more of these features.

The total maximum displacement, D_{TM}, at the isolation system level shall be determined using a model of the isolated structure in which the force-deflection characteristics of nonlinear elements of the isolation system and the seismic force resisting system are included.

9.6.2.2. Isolated Structure

A linear elastic model for the isolated structure may be used to calculate the maximum displacement of each floor, and design forces and displacements in elements of the seismic force resisting system, provided that all elements of the structur and the seismic force resisting system above the isolation system remain fully elastic.

The design lateral force for the seismic load resisting systems with fully elastic elements including, but not limited to, regular structural systems, shall not be less than 100% of V_s as defined in sections 9.5.4.2 and 9.5.4.3.

The analysis of the structure and isolation system shall be done separately for upper bound and lower limit characteristics, and the more critical conditions governing each response parameter of interest shall be used for design.

9.6.3. Description of Procedures

9.6.3.1. General

Response spectrum analysis shall be performed in accordance with Section 4.11 and the requirements of Section 9.6.3.3. Response history analysis shall be performed in accordance with the requirements of Section 9.6.3.4.

9.6.3.2. MCE$_R$, Ground Motions

The MCE$_R$ ground motions mentioned in Section 9.3 shall be used to determine the lateral forces and displacements of the isolated structure, the total maximum displacement of the isolation system, and the forces in the isolator devices, isolator device connections, and supporting framing immediately above and below the isolator devices, required to resist isolator P-Δ demands.

9.6.3.3. Response Spectrum Analysis Procedure

Response spectrum analysis shall be performed using a modal damping ratio for the fundamental mode in the direction of interest, which shall not greater than the effective damping ratio of the isolation system or

30% of critical damping, whichever is less. The values of the modal damping ratio for higher modes shall be selected in such a way that are suitable for response spectrum analysis of the structure above the isolation system with the assumption of a fixed base.

To determine the total maximum displacement in response spectrum analysis, the analytical model should be subjected to simultaneous excitation of 100% of the ground motion in the critical horizontal direction, and 30% of the ground motion in the perpendicular horizontal direction. The maximum displacement of the isolation system shall be determined as the resultant vector sum of the two orthogonal displacements.

9.6.3.4. Response History Analysis Procedure

Response history analysis shall be performed for a set of ground motion pairs selected and scaled in accordance with Section 9.3.2. Each pair of ground motion components, while the most disadvantageous location of eccentric mass is considered, shall be simultaneously applied to the analytical model. The maximum displacement of the isolation system shall be determined as the resultant vector sum of the two orthogonal displacements at each time step.

The response parameters of interest shall be determined for each ground motion used for the response history analysis, and the average value of the response parameters of interest shall be used for design.

9.6.3.4.1. Accidental Mass Eccentricity. Torsional response due to asymmetry in mass and stiffness shall be included in the analysis. In addition, accidental eccentricity, including moving the center of mass from the computed position by an amount of 5% of the diaphragm dimension, shall be considered separately in each of two orthogonal directions at the level under consideration.

The effects of accidental eccentricity, may be accounted for by using amplification factors of force, story drift, and deformation determined from an analysis, in which only the computed center of mass is used, provided that these factors produce results that bound all the mass-eccentric cases.

9.6.4. Minimum Lateral Displacements and Forces

9.6.4.1. Isolation System and Structural Elements below the Base Level

The isolation system, foundation, and all structural elements placed below the base level, shall be designed using all of the requirements for a nonisolated structure and the forces obtained from the dynamic analysis without applying the reduction factor, but the design lateral force shall not be considered less than 90% of V_b determined by Eq. 9.9.

The total maximum displacement of the isolation system shall not be considered less than 80% of D_{TM}, as prescribed in Section 9.5.3.3, except that D'_M may be used in lieu of D_M, where:

$$D'_M = \frac{D_M}{\sqrt{1 + (T/T_M)^2}} \tag{9.16}$$

in which D_M (m), is the maximum displacement at the center of rigidity of the isolation system in the direction of interest, as defined in Eq. 9.5; T (sec), is the period of the elastic and fixed-base structure above the isolation system, as determined by Section 4.10.3, including the coefficient C_u, if the approximate period formulas are used; and, T_M (sec), is the effective period of the seismically isolated structure, at the displacement D_M in the direction of interest, in accordance with Eq. 9.6.

9.6.4.2. Structural Elements above the Base Level

In accordance with the specific procedure and limitations of this section, the structural elements placed above the base level shall be designed using the requirements for a nonisolated structure and the forces obtained from the dynamic analysis reduced by the factor R_I, as determined in accordance with Section 9.5.4.2,.

For response spectrum analysis, the design shear at each story shall not be less than the shear of the story resulting from application of the forces determined with Eq. 9.13, where the value of V_b is equal to the base shear obtained from the response spectrum analysis in the desired direction.

For response history analysis of regular structures, value of V_b shall not be less than 80% of the value determined in accordance with Section 9.5.4.1, and the value of V_s shall not be less than 100% of the limits specified by Section 9.5.4.3.

For response history analysis of irregular structures, the value of V_b shall not be less than 100% of the value determined in accordance with Section 9.5.4.1, and the value of Vs shall not be less than 100% of the limits specified by Sections 9.5 and 4.3.

9.6.4.3. Scaling of Results

If the factored lateral shear force applied on structural elements determined using either the response spectrum or response history procedure, is less than the minimum values determined in Sections 9.6.4.1 and 9.6.4.2, all design parameters shall be increased proportionally.

9.6.4.4. Drift Limits

Maximum story drift corresponding to the design lateral force, including displacement caused by vertical deformation of the isolation system, shall comply with either of the following limits:

1. Where response spectrum analysis is used, the maximum value of story drift of the structure above the isolation system shall not exceed $0.015h_{sx}$.
2. here response history analysis is used, based on the nonlinear force-deflection characteristics of elements of the seismic force resisting system, the maximum story drift of the structure above the isolation system shall not exceed $0.020h_{sx}$.

Story drift shall be determined using Equation 4.16, where C_d for the isolated structure is equal to R_I, as defined in Section 9.5.4.2.

If the story drift ratio exceeds the value $(0.010/R_I)$, the secondary effects of the maximum lateral displacement of the structure above the isolation system in combination with the gravity forces shall be investigated.

9.7. Design Review

Design review of the isolation system and related test programs shall be independently performed by one or more professionals possessing knowledge of the following items, at least one of them is a RDP. Design review of the isolation system shall include, but not be limited to, all the following:

1. Design criteria of the project, including site-specific spectra of the building and ground motion histories.
2. Preliminary design, including selection of the isolation devices, determining the maximum displacement, the total maximum displacement, and the lateral force level.
3. Review of qualification data and proper property modification factors for the manufacturer and isolation device selected.
4. Prototype testing program (Section 9.8.2).
5. Final design of the entire structural system and all supporting analyses, including modeling of the isolators for response history analysis, if performed.
6. Production testing program of the isolator device (Section 9.8.5.).

9.8. Testing

9.8.1. General

The deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated structures shall be determined before construction, based on the tests of selected specimens of the components, in accordance with the items stated in this section. If a wind restraint system is used in the design, the components of the isolation system to be tested shall include the wind restraint system.

The tests specified in this section are for establishing and validating the test characteristics of the isolator devices and isolation system, which are used to determine design characteristics of the isolation system in accordance with Section 9.2.8.

9.8.1.1. Qualification Tests

Isolation device manufacturers shall submit, the results of qualification tests, analysis of test data, and additional scientific studies that are allowed to be used to quantify the effects of heating caused by cyclic dynamic motion, loading rate, scragging, variability and uncertainty in production bearing characteristics, temperature, aging, environmental exposure, and contamination. The qualification testing shall be applicable for all types of components, models, materials, and sizes to be used in the construction. The qualification testing shall have been performed on the components manufactured by the same manufacturer that provides the components used in the construction. In the case of using scaled specimens in the qualification testing, principles of scaling and similarity shall be used to interpret the data.

9.8.2. Prototype Tests

Prototype tests shall be performed separately on two full-size specimens (or a set of specimens, as appropriate) of each predominant type and size of isolator device of the isolation system. If the wind restraint system is used during design, the test specimens shall include such a wind restraint system.

The tested specimens shall not be used for construction unless they are accepted by the RDP responsible for the design of the structure.

9.8.2.1. Record

For each cycle of each test, the force-deflection behavior of the test specimen shall be recorded.

9.8.2.2. Sequence and Cycles

Each of the following test sequence shall be performed for the prescribed number of cycles at a vertical load equal to the average dead load plus one-half the effects due to live load on all isolator devices of the same type and size. Before performing these tests, the following set of production tests specified in Section 9.8.5 shall be performed on each isolator:

1. Twenty complete reversed cycles of loading at a lateral force corresponding to the wind design force.
2. The sequence of either item (a) or item (b) mentioned below shall be performed:
   a) Three complete reversed cycles of loading at each of the following increments of the displacement: $0.25D_M$, $0.5D_M$, $0.67D_M$, and $1.0D_M$, where D_M is determined in accordance with Section 9.5.3.1 or Section 9.6, as appropriate.
   b) The following sequence shall be performed dynamically at the effective period T_M: Continuous loading of one complete reversed cycle at each of the following increments of the maximum displacement: $1.0D_M$, $0.67D_M$, $0.5D_M$, and $0.25D_M$, followed by continuous loading of one complete reversed cycle at $0.25D_M$, $0.5D_M$, $0.67D_M$, and $1.0D_M$. A rest interval is allowed between these two loading sequences.
3. Three complete reversed cycles of loading at the maximum displacement, $1.0D_M$.
4. The sequence of either item (a) or item (b) mentioned below shall be performed:
   a) $30S_{M1}/(S_{MS}B_M)$, but not less than 10 times, continuous complete reversed cycles of loading at $0.75D_M$.
   b) Performing the test of item (a) dynamically at the effective period, T_M. This test may consist of separate sets of multiple cycles of loading, provided that the number of continuous cycles of loading of each set is not less than five continuous cycles.

If an isolator device is also an element of the vertical load carrying system, then item 3 of the sequence of cyclic tests specified above shall be performed for the two additional vertical load cases specified in Section 9.2.7.1. The amount of load increment resulting from the overturning effect of the earthquake, Q_E, shall be equal to or greater than the peak earthquake vertical force response corresponding to the test displacement being evaluated. In these tests, the combined vertical load shall be considered based on the typical or average downward force on all isolator devices of the same type and size. The values of axial load and displacement for each test shall be the greater of the values determined by analysis using upper bound and lower bound isolation system characteristics, in accordance with Section 9.2.8.5. The effective period, T_M, shall be the lower of two periods determined by analysis using upper bound and lower bound values.

9.8.2.3. Dynamic Testing

Tests specified in Section 9.8.2.2 shall be performed dynamically at the effective period, T_M, which is the lower value obtained using upper bound and lower bound properties.

If the prototype testing is dynamically performed on the isolators of the same size, in which the requirements of Section 9.8.2.7 are meet, and the testing was carried out at similar loads where effects of velocity, amplitude of displacement, and the heating effects are considered, there will not need to perform the dynamic testing. The prior dynamic prototype test data shall be used to determine factors that adjust three-cycle average values of k_d and E_{loop} to take into account the difference in test velocity and heating effects, to determine $\lambda_{(test,\ min)}$ and $\lambda_{(test,max)}$.

Only if full-scale testing is not possible, reduced-scale prototype specimens may be used to quantify rate-dependent properties of isolators. The reduced-scale prototype specimens shall be of the same type and material, and shall be manufactured with the same processes and quality as full-scale prototypes, and shall be tested at a frequency that reflects the loading rate of the full-scale prototype.

9.8.2.4. Units Dependent on Bilateral Load

If the force-deflection characteristics of isolator devices indicate bilateral load dependence, the tests specified in Section 9.8.2.2 and Section 9.8.2.3 shall be added to include bilateral load at the increments of the maximum displacement, D_M, as follows: 0.25 and 1.0, 0.5 and 1.0, 0.67 and 1.0 and 0.1 and 0.1.

If specimens with reduced scale are used to quantify dependency to bilateral load properties, the requirements of Section 9.8.2.7 shall be met. Reduced scale specimens shall be of the same type and material and be produced with the same processes and quality as full-scale prototypes.

If the effective stiffness of an isolator device under bilateral loading is different by more than 15% different from its effective stiffness under unilateral loading, the force-deflection characteristics of that isolator device shall be considered to be dependent on the bilateral load.

9.8.2.5. Maximum and Minimum Vertical Load

Isolation devices that transmit vertical load shall be subjected to one fully reversed cycle of loading at the total maximum displacement, D_{TM}, with each of the maximum and minimum downward vertical loads specified in Section 9.2.7.1, which can be applied on any isolator of the same type and size. Axial load and displacement values for each test shall be the greater values determined through analysis using upper bound and lower bound isolation system characteristics, calculated in accordance with Section 9.2.8.5.

Exception: Instead of performing one test using envelope values, it is acceptable to perform two tests, each for the combination of vertical load and horizontal displacement using the upper bound and lower bound values of isolation system characteristics, respectively, calculated in accordance with Section 9.2.8.5.

9.8.2.6. Sacrificial Wind-Restraint Systems

If a restraint system is to be used against the wind, its ultimate capacity shall be determined through testing.

9.8.2.7. Testing Similar Units

If an isolation device is comparable to another isolation device that has been tested, there is no need to conduct prototype tests for that isolation device if all the following criteria are met:

1. Dimension design of the isolator device is not more than 15% larger nor more than 30% smaller than the previously tested prototype;
2. The design of the isolator is of the same type and material;
3. The energy dissipated per cycle, E_{loop}, of the designed isolator device, is not less than 85% of the value for the previously tested isolator device;

4. The design is made by the same manufacturer using the same or more rigorous documented manufacturing and quality control procedures;
5. For elastomeric-type isolators, the design shall not be subject to a greater shear strain nor greater vertical stress than that values of the previously tested prototype; and
6. For sliding-type isolators, the design shall not be subject to a greater vertical stress or sliding velocity than thAT values of the previously tested prototype with similar sliding material.

Exemption from prototype testing mentioned above shall be approved by independent design review process, as specified in Section 9.7.

When the results of tests of similar isolator devices are used to establish dynamic characteristics based on Section 9.8.2.3, in addition to the items 2 to 4 mentioned above, the following criteria shall also be satisfied:

7. The similar device shall be tested at a frequency that represents design full-scale loading rate in accordance with principles of scaling and similarity.
8. The length scale coefficient of reduced scale specimens shall not be greater than two.

9.8.3. Determination of Force-Deflection Characteristics

The force-deflection characteristics of an isolator device shall be determined based on the cyclic load tests of prototype isolators in accordance with Section 9.8.2.

As required, the effective stiffness of an isolator device, k_{eff}, should be determined for each loading cycle as follows:

$$k_{eff} = \frac{|F^{+}| + |F^{-}|}{|\Delta^{+}| + |\Delta^{-}|} \qquad 9.17$$

where F^{+} and F^{-} are respectively the positive and negative forces at the maximum positive displacement Δ^{+} and the minimum negative displacement Δ^{-}.

As required, the effective damping, β_{eff}, of an isolator device shall be determined for each load cycle as follows:

$$\beta_{eff} = \frac{2}{\pi} \frac{E_{loop}}{k_{eff}(|\Delta^{+}| + |\Delta^{-}|)^{2}} \qquad 9.18$$

in which the energy dissipated per cycle of loading, E_{loop}, and the effective stiffness, k_{eff}, shall be determined based on peak test displacements of Δ^{+} and Δ^{-}.

As required, the postyield stiffness, k_d, of each isolator device at each cycle of loading shall be determined using the following assumptions:

1. It shall be assumed that a test loop has bilinear hysteretic characteristics with values of k_1, k_d, f_o, f_y, k_{eff}, and E_{loop} as shown in Fig. 9.1.

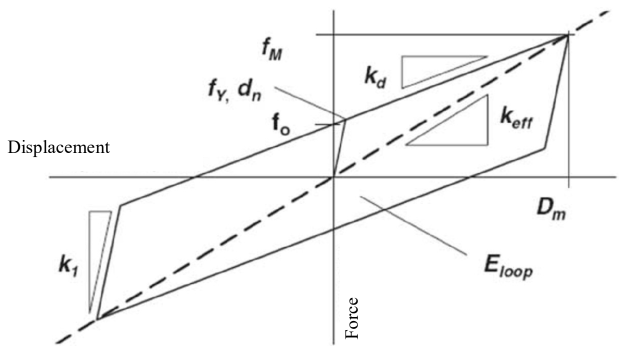


Fig. 9.1 Nominal characteristics of the isolator bilinear force-deflection model.

2. The determined loop shall have the same values of effective stiffness, k_{eff}, and energy dissipated per cycle of loading, E_{loop}, as the test loop.
3. The assumed value of k_1 shall be a visual fit to the elastic stiffness of the isolator device during unloading, immediately after D_M.

It may be used different methods for fitting the loop, for example, for k_d fitting a straight line directly to the hysteresis curve and continuing it to D_M and then determining k_1 to match E_{loop}.

9.8.4. Test Specimen Adequacy

If all of the following conditions are met, the performance of the test specimens shall be considered adequate:

1. The force-deflection diagrams for all tests specified in Section 9.8.2 have a positive incremental force-resisting capacity.
2. The average postyield stiffness, k_d, and energy dissipated per cycle, E_{loop}, for the three cycles of test specified in item 3 of Section 9.8.2.2 shall satisfy the following: the vertical load equal to the average dead load in addition to one-half of the live load effects, including the effects of heating and loading rate, in accordance with Section 9.2.8.3. The mentioned quantities shall be within the range of nominal design values defined based on the allowed range for each individual isolator, and is typically ±5% greater than the range of $\lambda_{(spec,min)}$ and $\lambda_{(spec,max)}$ for the average of all isolators.
3. For each increment of test displacements $0.67D_M$ and $1.0D_M$, specified in item 2 and item 3 of Section 9.8.2.2, and for each vertical load case specified in Section 9.8.2.2, the value of the postyield stiffness, k_d, at each cycles of test at the same displacement shall be within the range defined by the nominal value of potyield stiffness multiplied by the coefficients $\lambda_{(test,min)}$ and $\lambda_{(test,max)}$.
4. For each specimen, there is no greater than 20% in the initial effective stiffness over the cycles of test specified in item 4 of Section 9.8.2.2.
5. For each test specimen, the postyield stiffness, k_d, and energy dissipated per cycle, E_{loop}, for each cycle of test of item 4 (a) in Section 9.8.2.2, shall be within the range of the nominal design values, defined by $\lambda_{(test,min)}$ and $\lambda_{(test,max)}$.
6. For each specimen, there is no greater than 20% decrease in the initial effective damping over the cycles of test specified in item 4 of Section 9.8.2.2.
7. All specimens of vertical load-carrying elements of the isolation system remain stable where tested, in accordance with Section 9.8.2.5.

Exception: RDP is allowed to set the limit range of item 3, item 4, and item 6 to consider the property variation factors of Section 9.2.8.4, which is used for design of the isolation system.

9.8.5. Production Tests

RDP shall establish a test program for the isolation devices used in the construction of the structure. The test program shall confirm the consistency of the measured values of nominal characteristics of the isolator device by testing 100% of the isolators in combined compression and shear, at a displacement value not less than two-thirds of the maximum displacement, D_M, determined using lower bound characteristics.

The average results of all tests shall be within the range of values defined by $\lambda_{(spec,max)}$ and $\lambda_{(spec,min)}$ established in Section 9.2.8.4. It is allowed to use a different range of values for individual isolator devices and for the average value of all isolator devices of a certain type, provided that differences in the ranges of values are accounting for in the design of each element of the isolation system, as prescribed in Section 9.2.8.

Chapter 10
Structures with Damping Systems

10.1. General

Every structure with a damping system and its components shall be designed and constructed in accordance with the requirements of this Regulation and requested modifications of Chapter 10. In cases, the damper is installed across the isolation interface of a seismically isolated structure, the values of displacements, velocities and accelerations shall be determined in accordance with chapter 9 of this Regulation.

10.1.1. Definitions

The following definitions are applicable only to the structures with damping systems, the subject of the provisions of this Chapter, and are in addition to the definitions presented in Chapters 2, 3, and 4.

Damping device: A flexible structural element of the damping system that dissipates energy due to relative motion of each end of the device. Damping device includes all pins, bolts, gusset plates, brace extensions, and other components required to connect damping device to the other elements of the structure. Damping devices are classified as displacement-dependent or velocity-dependent, or a combination thereof, and may be configured for linear or nonlinear operation.

Damping system: A set of structural elements that includes all the individual damping devices, all structural elements or bracing required to transfer force from damping devices to the base of the structure and the structural elements required to transfer forces from damping devices to the seismic force-resisting system.

Displacement-dependent damping device: The force response of a displacement-dependent damping device is primarily a function of the relative displacement between each end of the device. The response is essentially independent of the relative velocity between each of the device and/or the excitation frequency.

Force-controlled elements: Elements whose acceptable inelastic deformation capacity is associated with a significant reduction in strength.

Velocity-dependent damping device: The force-deflection relation for a velocity-dependent damping device is basically a function of the relative velocity between the two ends of the device, and it may also be a function of the relative displacement between the two ends.

10.1.2. Symbols

Symbols presented in this section are applicable only to structures with damping system, the subject of provisions of this Chapter, and are in addition to the symbols presented in Chapters 2, 3, and 4.

B_{1D}	:	Numerical coefficient, specified in Table 10.1, for effective damping ratio equal to β_{mD} (m=1) and period of structure equal to T_{1D}.
B_{1E}	:	Numerical coefficient, specified in Table 10.1, for effective damping ratio equal to $\beta_I+\beta_{V1}$ and period of structure equal to T_1.
B_{1M}	:	Numerical coefficient, specified in Table 10.1, for effective damping ratio equal to β_{mM} (m=1) and period of structure equal to T_{1M}.
B_{mD}	:	Numerical coefficient, specified in Table 10.1, for effective damping ratio equal to β_{mD} and period of structure equal to T_m.
B_{mM}	:	Numerical coefficient, specified in Table 10.1, for effective damping ratio equal to β_{mM} and period of structure equal to T_m.
B_R	:	Numerical coefficient, specified in Table 10.1 for effective damping ratio equal to β_R and period of structure equal to T_R.
B_{V+I}	:	Numerical coefficient, specified in Table 10.1, for effective damping ratio equal to the sum of the viscous damping in the fundamental mode of vibration of the structure in the direction under consideration, β_{Vm} (m=1), plus inherent damping, β_I, and period of structure equal to T_1.
C_{mFD}	:	Force coefficient, specified in Table 10.2

C_{mFV}	:	Force coefficient, specified in Table 10.3
C_{s1}	:	Seismic response coefficient of the fundamental mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.4 or 10.7.2.2.4 (m=1).
C_{sm}	:	Seismic response coefficient of the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.4 (m=1) or Section 10.7.1.2.6 (m>1).
C_{SR}	:	Seismic response coefficient of the residual mode of vibration of the structure in the direction under consideration, Section 10.7.2.2.8.
D_{1D}	:	Fundamental mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 10.7.2.3.2.
D_{1M}	:	Fundamental mode MCE_R displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 10.7.2.3.5.
D_{mD}	:	Design displacement at the center of rigidity of the roof level of the structure due to the m^{th} mode of vibration in the direction under consideration, Section 10.7.1.3.2.
D_{mM}	:	MCE_R displacement at the center of rigidity of the roof level of the structure due to the m^{th} mode of vibration in the direction under consideration, Section 10.7.1.3.5.
D_{RD}	:	Residual mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 10.7.2.3.2.
D_{RM}	:	Residual mode MCE_R displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 10.7.2.3.5.
D_Y	:	Displacement at the center of rigidity of the roof level of the structure at the effective yield point of the seismic force-resisting system, Section 10.7.3.3.
E_{loop}	:	Area of one load-displacement hysteresis loop, Section 10.6.2.5.
f_i	:	Lateral force at level i of the structure distributed approximately in accordance with Sections 4.10.4 and 10.7.2.2.3.
F_{i1}	:	Inertial force at level i (or concentrated mass i) in the fundamental mode of the structure in the direction under consideration, Section 10.7.2.2.9.

F_{im}	:	Inertial force at level *i* (or concentrated mass *i*) in the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.7.
F_{iR}	:	Inertial force at level *i* (or concentrated mass *i*) in the residual mode of vibration of the structure in the direction under consideration, Section 10.7.2.2.9.
h_i	:	Height of level *i* from the base level, Section 10.7.2.2.3.
h_n	:	Height of building structure, Section 10.7.2.2.3.
I_e	:	Seismic importance factor determined in accordance with Section 4.3.
q_H	:	Hysteresis loop adjustment factor, as prescribed by Section 10.7.3.2.2.1.
Q_{DSD}	:	Force in an element of the damping system required to resist design seismic forces of displacement-dependent damping devices, Section 10.7.4.5.
Q_E	:	Seismic design force in each element of the damping system, Section 10.7.4.5.
Q_{mDSV}	:	Force in an element of the damping system required to resist design seismic forces of velocity-dependent damping devices, due to the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.4.5.
Q_{mSFRS}	:	Force in an element of the seismic force-resisting system equal to the design seismic force due to the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.4.5.
S_{D1}	:	Design spectral acceleration parameter (g) corresponding to 1 sec period, for 5% damping ratio.
S_{DS}	:	Design spectral acceleration parameter (g) corresponding to short period (0.2 sec), for 5% damping ratio.
S_{M1}	:	MCE_R spectral acceleration parameter (g) corresponding to 1 sec period, for 5% damping ratio.
S_{MS}	:	MCE_R spectral acceleration parameter (g) corresponding to short period (0.2 sec), for 5% damping ratio.
T_0	:	$0.2(S_{D1}/S_{DS})$, Section 10.7.3.1.
T_1	:	Fundamental period in the direction under consideration.
T_{1D}	:	Effective period (sec) of the fundamental mode of vibration of the structure at the design displacement in the direction under consideration, as prescribed by Sections 10.7.1.2.5 or 10.7.2.2.5

T_{1M}	:	Effective period (sec) of the fundamental mode of vibration of the structure at the MCE_R displacement in the direction under consideration, as prescribed by Sections 10.7.1.2.5 or 10.7.2.2.5.
T_m		Period (sec) of the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.6.
T_R		Period (sec) of the residual mode of vibration of the structure in the direction under consideration, Section 10.7.2.2.7.
T_S		(S_{D1}/S_{DS}), Sections 10.7.1.2.4, 10.7.1.2.6, 10.7.2.2.4, 10.7.3.2.2.1, and 10.7.3.4.
V		Seismic base shear in the direction under consideration, Section 10.2.1.1.
V_1		Design value of the seismic base shear force of the fundamental mode of vibration of the structure in the direction under consideration, as prescribed by Sections 10.7.2.2.1 or 10.7.2.2.2.
V_m		Design value of the seismic base shear of the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.2.
V_{min}	:	Minimum allowable value of base shear permitted for design of the seismic force-resisting system of the structure in the direction under consideration, Section 10.2.1.1.
V_R	:	Design value of the seismic base shear of the residual mode of vibration of the structure in the direction under consideration, as prescribed by Section 10.7.2.2.6.
w_i	:	Effective seismic weight of the i^{th} floor of the structure, Section 18.7.1.2.2.
$\overline{W_1}$	:	Effective fundamental mode seismic weight, as determined by Eq. 10.6b (m=1).
$\overline{W_m}$	:	Effective seismic weight of the m^{th} mode of vibration of the structure, Section 10.7.1.2.2.
W_m	:	Maximum strain energy of the m^{th} mode of vibration of the structure in the direction under consideration at modal displacements, δ_{im}, Section 10.7.3.2.2.1
W_{mj}	:	Work done by the j^{th} damping device in one fully cycle of dynamic response corresponding to the m^{th} mode of vibration of the structure in the direction under consideration at modal displacements, δ_{im}, Section 10.7.3.2.2.1.
$\overline{W_R}$	:	Effective residual mode seismic weight, as prescribed by Eq. 10.34.
α	:	Velocity exponent relating damping device force to damping device velocity.

β_{HD} : Part of effective damping ratio of the structure in the direction under consideration due to postyield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand, μ_D, Section 10.7.3.2.2.

β_{HM} : Part of effective damping ratio of the structure in the direction under consideration due to postyield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand, μ_M, Section 10.7.3.2.2.

β_I : Part of effective damping ratio of the structure due to the inherent energy dissipation by elements of the structure, at or just below the effective yield displacement of the seismic force-resisting system, Section 10.7.3.2.1.

β_{mD} : Total effective damping of the m^{th} mode of vibration of the structure in the direction under consideration at the design displacement, Section 10.7.3.2.

β_{mM} : Total effective damping of the m^{th} mode of vibration of the structure in the direction under consideration at the MCE_R displacement, Section 10.7.3.2.

β_R : Total effective damping in the residual mode of vibration of the structure in the direction under consideration, as determined in accordance with Section 10.7.3.2 (using μ_D=1.0 and μ_M=1.0).

β_{Vm} : Part of effective damping ratio of the m^{th} mode of vibration of the structure in the direction under consideration due to viscous energy dissipation by the damping system, at or just below the effective yield displacement of the seismic force-resisting system, Section 10.7.3.2.3.

Γ_1 : Participation factor of the fundamental mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.3 or 10.7.2.2.3 (m=1).

Γ_m : Participation factor of the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.3.

Γ_R : Participation factor of the residual mode of vibration of the structure in the direction under consideration, Section 10.7.2.2.7

δ_i : Elastic deflection of level i of the structure due to applied lateral force, f_i, Section 10.7.2.2.3.

δ_{i1D} : Fundamental mode design deflection of level i at the center of rigidity of the structure in the direction under consideration, Section 10.7.2.3.1.

δ_{iD} : Total design deflection of level *i* at the center of rigidity of the structure in the direction under consideration, Section 10.7.2.3.

δ_{iM} : Total MCE_R deflection of level *i* at the center of rigidity of the structure in the direction under consideration, Section 10.7.2.3.

δ_{im} : Deflection of level *i* in the m^{th} mode of vibration at the center of rigidity of the structure in the direction under consideration, Section 10.7.3.2.3.

δ_{imD} : Design deflection of level *i* in the m^{th} mode of vibration at the center of rigidity of the structure in the direction under consideration, Section 10.7.1.3.1.

δ_{iRD} : Residual mode design deflection of level *i* at the center of rigidity of the structure in the direction under consideration, Section 10.7.2.3.1.

Δ_{1D} : Design story drift due to the fundamental mode of vibration of the structure in the direction under consideration, Section 10.7.2.3.3.

Δ_D : Total design story drift of the structure in the direction under consideration, Section 10.7.2.3.3.

Δ_M : Total MCE_R story drift of the structure in the direction under consideration, Section 10.7.2.3.

Δ_{mD} : Design story drift due to the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.1.3.3.

Δ_{RD} : Design story drift due to the residual mode of vibration of the structure in the direction under consideration, Section 10.7.2.3.3.

$\lambda_{(ae,max)}$: Factor considering possible variation in damper properties above the nominal values, due to aging and environmental effects, which is determine by multiplying all the individual aging and environmental effects, Section 10.2.4.5.

$\lambda_{(ae,min)}$: Factor considering possible variation in damper properties below the nominal values, due to aging and environmental effects, which is determine by multiplying all the individual aging and environmental effects, Section 10.2.4.5.

λ_{max} : Factor considering all possible variations in damper properties above the nominal values, Section 10.2.4.5.

λ_{min} : Factor considering all possible variations in damper properties below the nominal values, Section 10.2.4.5.

$\lambda_{(spec,max)}$: Factor considering permissible variations in production damper nominal properties, above the values assumed in design, as determined by RDP, Section 10.2.4.5.

$\lambda_{(spec,min)}$	:	Factor considering permissible variations in production damper nominal properties, below the values assumed in design, as determined by RDP, Section 10.2.4.5.
$\lambda_{(test,max)}$	:	Factor considering possible variations in damper properties above the nominal values obtained from the prototype tests, Section 10.2.4.5.
$\lambda_{(test,min)}$	:	Factor considering possible variations in damper properties below the nominal values obtained from the prototype tests, Section 10.2.4.5.
μ	:	Effective ductility demand on the seismic force-resisting system in the direction under consideration.
μ_D	:	Effective ductility demand on the seismic force-resisting system in the direction under consideration due to the design earthquake ground motions, Section 10.7.3.3.
μ_M	:	Effective ductility demand on the seismic force-resisting system in the direction under consideration due to the MCE_R earthquake ground motions, Section 10.7.3.3.
μ_{max}	:	Maximum allowable effective ductility demand on the seismic force-resisting system in the direction under consideration due to the design earthquake ground motions, Section 10.7.3.4.
φ_{i1}	:	Displacement amplitude at level i of the fundamental mode of vibration of the structure in the direction under consideration, normalized to unity at the roof level, Section 10.7.2.2.3.
φ_{im}	:	Displacement amplitude at level i of the m^{th} mode of vibration of the structure in the direction under consideration, normalized to unity at the roof level, Section 10.7.1.2.2.
φ_{iR}	:	Displacement amplitude at level i of the residual mode of vibration of the structure in the direction under consideration, scaled to unity at the roof level, Section 10.7.2.2.7.
∇_{1D}	:	Design story velocity due to the fundamental mode of vibration of the structure in the direction under consideration, Section 10.7.2.3.4
∇_D	:	Total design story velocity of the structure in the direction under consideration, Section 10.7.1.3.4.
∇_M	:	Total MCE_R story velocity of the structure in the direction under consideration, Section 10.7.2.3.
∇_{mD}		Design story velocity due to the m^{th} mode of vibration of the structure in the direction under consideration, Section 10.7.1.3.4.
∇_{RD}	:	Design story velocity due to the residual mode of vibration of the structure in the direction under consideration, Section 10.7.2.3.4.

Table 10.1. Damping coefficients β_{V+1}، β_{1D}، β_{1E}، β_R، β_{1M}، β_{mD}، β_{mM}
(where period of the structure ≥ T_0***)***

Effective Damping ratio, β (Percentage of critical)	β_{V+1}، β_{1D}، β_{1E}، β_R، β_{1M}، β_{mD}، β_{mM} (where period of the structure ≥ T_0)
≤ 2	0.8
5	1.0
10	1.2
20	1.5
30	1.8
40	2.1
50	2.4
60	2.7
70	3.0
80	3.3
90	3.6
≥ 100	4.0

10.2. General Design Requirements

10.2.1. System Requirements

Design of the structure shall consider the main requirements for the seismic force-resisting system and the damping system, as defined in the following sections. The seismic force-resisting system shall have the required strength to confront the forces defined in Section 10.2.1.1. The combination of the seismic force-resisting system and the damping system may be used to confront the drift requirement.

10.2.1.1. Seismic Force-Resisting System

Structures with a damping system shall have a seismic force-resisting system in each lateral direction that corresponds to one of the types of systems provided in Standard 2800.

The design of the seismic force-resisting system in each direction shall fulfill the requirements of the minimum base shear, as defined in this Section, and the following requirements:

a) If the nonlinear response history procedure (Section 10.3) is used, the requirements of Section 10.4;

b) If either the response spectrum procedure (Section 10.7.1), or the equivalent lateral force procedure (Section 10.7.2) is used, the requirements of Section 10.7.4.

The base shear used for seismic design of the seismic force-resisting system shall not be less than $V_{\min}$, which is the larger value obtained from Eq. 10.1 and Eq. 10.2:

$$V_{\min} = \frac{V}{B_{V+1}} \qquad 10.1$$

$$V_{\min} = 0.75V \qquad 10.2$$

In which V is the seismic base shear in the direction under consideration according to Section 4.10, and B_{V+I} is the numerical coefficient provided in Table 10.1, for effective damping ratio equal to the sum of viscous damping in the fundamental mode of vibration of the structure in the direction under consideration, β_{Vm} (m=1), plus inherent damping ratio, β_I, and period of structure equal to T_I.

Exception: If either of the following conditions exist, the seismic base shear used for seismic design of the seismic force-resisting system shall not be less than $1.0V$:

1. Configuration of the damping system on each floor level, has less than two damping devices to resist torsion, in the direction under consideration.
2. The seismic force-resisting system has Torsional Irregularity Ratio, *TIR*, greater than 1.4, or vertical irregularity Type (b) (Table 4.1).

10.2.1.2. Damping System

Damping devices and other components required to connect damping devices to the other elements of the structure shall be designed to remain elastic for MCE_R loads. Other elements of the damping system may be allowed to have inelastic response at MCE_R, if it is shown through analysis or test that inelastic response of these elements would not have negative effects on performance of the damping system. If either the response spectrum procedure (Section

10.7.1) or the equivalent lateral force procedure (Section 10.7.2) is used, the inelastic response shall be limited, in accordance with the requirements of Section 10.7.4.6.
Force-controlled elements of the damping system shall be designed for seismic forces that are increased by 20% from those corresponding to average MCE_R response.

10.2.2 Seismic Hazard

10.2.2.1. Spectral Response Acceleration Parameters and Response Spectrum

Spectral response acceleration parametrs for the design earthquake amd MCE_R (S_{DS}, S_{D1}, S_{MS} and S_{M1}), as well as the spectra of the design earthquake and MCE_R shall be determined in accordance with the requirements of Chapter 3.

10.2.2.2. Ground Motions for Response History Analysis

In cases where the response history analysis (Section 10.3) is used to design structures with damping systems, the provisions of Sections 10.3 and 4.12.2 shall apply, with the difference that for determining the period range in Section 4.12.2, nominal properties of damping devices at the MCE_R per Section 10.2.4.4 shall be assumed. Design earthquake ground motions shall be considered as two-thirds MCE_R ground motions.

10.2.3 Procedure Selection

Structures with a damping system provided for seismic resistance shall be analyzed and designed using the nonlinear response history procedure of Section 10.3.
Exception: It shall be permitted to analyze and design the structures, using the response spectrum procedure of Section 10.7.1, subject to the limits of Section 10.2.3.1, or the equivalent lateral force procedure of Section 10.7.2, subject to the limits of Section 10.2.3.2.

10.2.3.1. Response Spectrum Procedure

The response spectrum procedure of Section 10.7.1 is allowed to be used for analysis and design, provided that all of the following conditions are met:

1. Configuration of the damping devices in each pricipal direction in each story is such that the damping system has at least two damping devices to resist torsion.
2. The total effective damping ratio of the fundamental mode of vibration, β_{mD} (m=1), of the structure is not greater than 35% of critical in the direction under consideration.

10.2.3.2. Equivalent Lateral Force Procedure

The equivalent lateral force procedure of Section 10.7.2 is allowed to be used for analysis and design, provided that all the following conditions are met:

1. Configuration of the damping devices in each pricipal direction in each story is such that the damping system has at least two damping devices to resist torsion.
2. The total effective damping ratio of the fundamental mode of vibration, β_{mD} (m=1), of the structure is not greater than 35% of critical in the direction under consideration.
3. The seismic force-resisting system does not have horizontal irregularity type (a) or (b) in accordance with Table 4.1, or vertical irregularity type (a) or (b) in accordance with Table 4.2.
4. Floor diaphragm are rigid, as defined in Section 4.13.
5. The height of the structure above the base level does not exceed 30 m.

10.2.4 Damping System

10.2.4.1 Device Design

The design, construction and installation of damping devices shall be based on response to MCE_R ground motions and also checking all of the following items:

1. Low-cycle, large-displacement degradation due to seismic loads.

2. High-cycle, small-displacement degradation due to wind, thermal or other cyclic loads.
3. Forces or displacements due to gravity loads.
4. Adhesion of device parts due to corrosion or abrasion, biodegradation, moisture, or chemical exposure.
5. Exposure to environmental conditions, including but not limited to, temperature, moisture, humidity, radiation (e.g., ultraviolet light), and reactive or corrosive substances (e.g., saltwater).

Devices with biometallic interfaces, where there is a possibility of cold welding of the sliding inteface, shall not be used in a damping system.

Damping devices subject to low-cycle fatigue shall be able to withstand wind forces without slip, movement, or inelastic cycling.

Damping devices shall be designed and constructed that the range of thermal conditions, device wear, manufacturing tolerances, and other effects that may cause changing device characteristics during the design life of the device, has been considered in accordance with Section 10.2.4.4. Ambient temperature shall be the normal in-service temprature of the damping device. The design temperature range shall cover the annual minimum and maximum in-service temperatures of the damping device.

10.2.4.2. Multiaxis Movement

Connection points of damping devices shall have sufficient flexibility to adapt to simultaneous longitudinal, lateral, and vertical displacements of the damping system.

10.2.4.3. Inspection and Periodic Testing

Necessary access for inspection and removal of all damping devices shall be provided. The registered design professional (RDP) who is responsible for design of the structure shall stablish an inspection, maintenance and testing plan for each type of damping device to ensure that they consistently perform their intended function without any issues,and work reliably for their entire intended lifespan. The level of inspection and testing shall be set based on in-service history of damping devices and the possibility of change in characteristics during the life of the devices.

10.2.4.4. Nominal Design Properties

Nominal design characteristics for energy dissipation devices shall be established from either project-specific prototype test data or previous prototype tests conducted on devices of a similar type and size. The nominal design characteristics shall be determined by Section 10.6.2.4 (2), based on data obtained from prototype tests specified in Section 10.6.2.2 (2). These nominal design characteristics shall be modified by using property variation or lambda (λ) factors, as specified in Section 10.2.4.5.

10.2.4.5. Maximum and Minimum Damper Properties

To consider variation of the nominal design parameters of each damping device type for the effects of heating caused by cyclic dynamic motion, loading rate, duration of seismic and wind loading, variability and uncertainty in the characteristics of the production device, operating temperature, aging, environment exposure, and contamination, the maximum and minimum property modification (λ) factors are used. In order to develop the property modification factors, it is necessary to use data specific to manufacturer qualification test, in accordance with Section 10.6.1.1 and prototype test data, in accordance with Section 10.6.2.

Maximum and minimum property modification (λ) factors for each device, shall be established, in accordance following equations RDP and used in analysis and design to account for the variation from nominal properties.

$$\lambda_{max} = \left(1 + \left(0.75 \times \left(\lambda_{(ae,max)} - 1\right)\right)\right) \times \lambda_{(test,max)} \times \lambda_{(spec,max)} \geq 1.2 \qquad \text{10.3a}$$

$$\lambda_{min} = \left(1 - \left(0.75 \times \left(1 - \lambda_{(ae,min)}\right)\right)\right) \times \lambda_{(test,min)} \times \lambda_{(spec,min)} \leq 0.85 \qquad \text{10.3b}$$

where, $\lambda_{(ae,max)}$ is the factor considering possible variation in damper properties above the nominal values, due to aging and environmental effects, which is determine by multiplying all the individual aging and environmental effects; $\lambda_{(ae,min)}$ is the factor considering possible variation in damper properties below the nominal values, due to aging and environmental effects, which is determine by multiplying all the individual aging and environmental effects; $\lambda_{(test,max)}$ is the factor considering possible variations in damper properties above the nominal values obtained from the prototype tests; $\lambda_{(test,min)}$ is the factor considering possible variations in damper properties below the nominal values obtained from the prototype tests; $\lambda_{(spec,max)}$ is the factor considering permissible variations in production damper nominal properties, above the values assumed in design, as determined by RDP; and $\lambda_{(spec,min)}$ is the factor considering permissible variations in production damper nominal properties, below the values assumed in design, as determined by RDP.

Exception: If test data is carefully checked and accepted by the RDP, the use of values smaller than 1.2 for λ_{max} and values greater than 0.85 for λ_{min} is allowed.

Maximum and minimum analysis and design properties for each device, shall be determined for each modeling parameter as follows:

$$\text{maximum design property} = \lambda_{max} \times \text{nominal design property} \tag{10.4a}$$

$$\text{minimum design property} = \lambda_{min} \times \text{nominal design property} \tag{10.4b}$$

A maximum and minimum analysis and design property shall be established for each modeling parameter as necessary for the selected method of analysis. For the maximum analysis and design case, maximum velocity coefficients, stiffness, strength, and energy dissipation shall be considered together, and for the minimum analysis and design case, minimum velocity coefficients, strength, stiffness, and energy dissipation shall be considered together.

It is necessary to determine maximum and minimum properties separately for loads and displacements corresponding to the design conditions and the MCE_R conditions.

10.2.4.6. Damping System Redundancy

If less than four energy dissipation devices are installed on any story of a building in either principal directions, or less than two devices are placed on both sides of the center of rigidity of any story in either principal direction, all energy dissipation devices shall be capable of sustaining displacements equal to 130% of the maximum calculated displacement in the device under MCE_R. A velocity-dependent device shall be capable of sustaining the force and displacement corresponding to a velocity equal to 130% of the maximum calculated velocity for that device under MCE_R.

10.3. Nonlinear Response History Procedure

The stiffness and damping properties of the damping devices used in the analytical models shall be based on, or confirmed based on, testing of the damping devices, as specified in Section 10.6. As required, in order to account for dependence of the device on frequency, amplitude and duration of seismic loading, it is necessary to clearly model the nonlinear characteristics of force-velocity-displacement of damping devices.

A nonlinear response history analysis shall be used an appropriate mathematical models described in this section, for the seismic force-resisting system and the damping system. Such a model shall explicitly consider the nonlinear hysteretic behavior of all elements and connections undergoing inelastic behavior in a manner consistent with available laboratory test data. Test data shall not be extrapolated beyond tested displacement levels. If the results of the analysis show that there is possibility of degradation in the stiffness or strength of the elements, the hysteretic models shall include these effects.

Exception: By using strength reduction factor, φ=1.0, if the determined force in an element of the seismic force-resisting system or damping system does not exceed 1.5 times its expected strength, the element is permitted to be modeled as linear.

Inherent damping ratio of the structure shall not be considered greater than 3% of critical, unless test data at, or just below, the effective yield displacement of the seismic force-resisting system support higher values.

Analysis shall be performed at both the design earthquake and at the MCE_R earthquake levels. The design earthquake analysis need not consider the effects of accidental eccentricity. Results from the design earthquake are used to design the seismic force-resisting system. Results from the MCE_R analysis are used to design the damping system.

10.3.1. Damping Device Modeling

Mathematical models of displacement-dependent damping devices shall consider the hysteretic behavior of the devices, corresponding to the test data, and take into account all the effective changes in stiffness, strength, and shape of the hysteretic loop. Mathematical models of velocity-dependent damping devices shall consider the velocity coefficient corresponding to the test data. If properties of damping device change with time and/or temperature, this behavior shall be modeled explicitly. The flexible elements of damping devices that connect the damping units to the structure shall be included in the model.

Exception: If it is expected that during the response history analysis, the characteristics of the damping devices will change, the dynamic response is permitted to be enveloped by the maximum and minimum characteristics of the device, as prescribed by Section 10.2.4.5. All these limit cases resulting from variable device characteristics shall satisfy the same conditions as if the time-dependent behavior of the devices were explicitly modeled.

10.3.2. Accidental Mass Eccentricity

Inherent eccentricity due to asymmetry in mass and stiffness shall be considered in the MCE_R analysis. In addition, during the analysis, accidental eccentricity in each of two orthogonal directions at the level of each diaphragm shall be separately considered, including moving the center of mass from determined location by a value equal to 5% of the diaphragm dimension.

Exception: The effects of accidental eccentricity may be considered by determining the magnification factors on forces, drifts, and deformations. These factors allow the results obtained from an analysis with only the configuration of the computated center-of-mass to be scaled to limit the results of all the states of mass eccentricity.

10.3.3. Response Parameters

Maximum values of each response parameters of interest shall be determined for each ground motion used for the response history analysis. Response parameters shall include the forces, displacements, and velocities (in the case of velocity-dependent devices) in each individual damping device. The average value of a response parameter of interest across the suit of design earthquake or MCE_R ground motions, may be used for design.

10.4. Seismic Load Conditions and Acceptance Criteria for Nonlinear Response History Procedure

For the nonlinear response history procedure of Section 10.3, the seismic force-resisting system, damping system, loading conditions, and acceptance criteria for response parameters of interest shall comply with the requirements of the following subsections.

10.4.1. Seismic Force-Resisting System

The seismic force-resisting system shall meet the strength requirements of Section 4.5 using both of the following items:

1. Seismic base shear, V_{min}, as provided in Section 10.2.1.1, and,
2. The demands resulting from the nonlinear response history analysis under design earthquake.

The story drifts shall be determined using the combined model of the seismic force-resisting system and the damping system under the MCE_R ground motions. Accidental eccentricity shall be included.

The story drifts in MCE_R earthquake shall not exceed 3%, not the drift limits in Table 10.4 multiplied by the smaller of $1.5R/C_d$ or 1.9. Values of C_d and R for the building framing under consideration, shall be taken from Table 4.5.

10.4.2. Damping System

The damping devices and their connections shall have a capacity and size to resist the forces, displacements and velocities caused by the MCE_R ground motions. Force-controlled elements of the damping system shall be designed for seismic forces that are increased by 20% from the average response forces corresponding to the MCE_R ground motions.

10.4.3. Combinations of Load Effects

The effects of gravity loads and seismic forces on the damping system shall be combined with the effects of horizontal seismic forces Q_E in accordance with Section 2.2.3, except that Q_E shall be determined under the MCE_R earthquake. When load combinations including the live load are used, it is permitted to use a load factor of 25% on live load for nonlinear response history analysis. The redundancy factor, ρ, shall be equal to 1.0 in all cases, and the seismic load effect, including overstrength of Section 2.2.4, need not apply to the design of the damping system.

10.4.4. Acceptance Criteria for the Response Parameters of Interest

Components of the damping system shall be evaluated by the strength design criteria of this Regulations using the seismic forced and seismic loading conditions determined from the nonlinear response history analyses under the MCE_R earthquake and strength reduction factor, φ=1.0.

10.5. Design Review

An independent design review of the damping system and the program of related tests shall be performed by one or more individual experts who possessing necessary knowledge of the following items: at least one reviewer shall be a RDP. Damping system design review shall include, but need not be limited to, all of the following:

1. Project design criteria including site-specific spectra and history of ground motions;
2. Preliminary design of the seismic force-resisting system and the damping system, including selection of the devices and their design parameters;
3. Review of manufacturer test data and property modification factors for the selected manufacturer and device;
4. Prototype testing program (Section 10.6.2);
5. Final design of the entire structural system and supporting analyses, including modeling of the damping devices for response history analysis, if performed; and,
6. Production test program of damping device (Section 10.6.3).

10.6 Testing

10.6.1 General

The force-velocity-displacement Relationships and damping characteristics, as mentioned the damping device nominal design properties in Section 10.2.4.4, shall be confirmed by the tests conducted in accordance with Section 10.6.2, or shall be based on prior tests of devices meeting the similarity requirements of Section 10.6.2.3.

The prototype tests specified in Section 10.6.2 shall be conducted to comply with the force-velocity-displacement characteristics of the damping devices assumed for analysis and design, as well as to demonstrate the robustness of individual devices under seismic excitation. These tests shall be conducted prior to production of devices for construction.

The test requirements for production are specified in Section 10.6.3. Nominal properties of the device, determined from the prototype testing shall comply with the acceptance criteria using $\lambda_{(spec,max)}$ and $\lambda_{(spec,min)}$ from Section 10.2.4.5. These criteria shall take into account possible variations in material properties.

The nominal characteristics of the device determined from the production testing of Section 10.6.3, shall comply with the acceptance criteria using $\lambda_{(spec,\ max)}$ and $\lambda_{(spec,\ min)}$ from Section 10.2.4.5.

The manufacture and quality control procedures shall be identical for all prototype and production devices. These procedures shall be approved by the RDP before the manufacture of prototype devices.

10.6.1.1. Qualification Tests

In order to obtain approval by the RDP, manufacturers of damping device shall submit the results of qualification tests, analysis of test data, and supporting studies used to quantify the heating effects due to cyclic dynamic motion, loading rate, duration of seismic and wind loading, variability and uncertainty in the characteristics of the production device, operating temperature, aging, environmental exposure, and contamination. The qualification testing shall be appropriate and feasible for the component types, materials, and force-velocity-displacement response to be used in the proposed manufacturing process.

10.6.2. Prototype Tests

The following tests shall be performed separately on two full-scale damping devices of each type and size used in the design process, in the order listed below.

Representative sizes of each type of device are allowed to be used for prototype testing, provided that both of the following conditions are met:

1. Manufacturing and quality control procedures are identical for each type and size of device used in the structure.
2. Prototype testing of representative sizes is approved by the RDP who is responsible for the design of the structure.

Test specimens shall not be used for construction, unless they are approved by RDP who is responsible for design of the structure and meet the requirements related to the prototype and production tests.

10.6.2.1. Data Recording

The force-deflection relationship for each cycle of each test shall be electronically recorded.

10.6.2.2. Sequence and Cycles of Testing

For all test sequences below, each damping device shall be subjected to gravity load and effects of thermal environments that are representative of the device installed conditions. In order to perform the seismic test, the displacement in the devices determined for the MCE_R ground motions, and called the maximum device displacement, shall be used.

Before carrying out the sequence of prototype tests, defined in this section, a production test in accordance with Section 10.3.6, shall be performed, and data obtained from this test shall serve as a baseline for comparison with subsequent tests on production dampers.

1. Each damping device shall be subjected to the number of cycles expected in the design windstorm, but not less than 2000 continuous full reversed cycles of wind load. Wind load shall be at amplitudes expected in the design windstorm, and shall be applied at a frequency equal to the inverse of the fundamental period of the structure, $1/T_1$.

   Using alternative loading protocols, representative of the design windstorm, is permitted such that dividing the total wind displacement into its expected static, pseudostatic, and dynamic components.

   Exception: If damping devices are not subject to wind-induced forces or displacements, or if the design wind force is less than the device yield or slip force, these tests need not be performed.
2. Each damping device shall reach the ambient temperature and be loaded with the following fully reversed, sinusoidal cycles at a frequency equal to $0.67/T_1$.
   a. Ten fully reversed cycles at the displacement in the energy-dissipation device, corresponding to 0.33 times the MCE_R device displacement;
   b. Five fully reversed cycles at the displacement in the energy-dissipation device, corresponding to 0.67 times the MCE_R device displacement;

c. Three fully reversed cycles at the displacement in the energy-dissipation device, corresponding to 1.0 times the MCE_R device displacement; and,
d. If in the test (c), the force produced in the energy dissipation device is less than the MCE_R, test (c) shall be repeated at a frequency that produce a force equal to, or greater than, the MCE_R force from analysis.

3. In cases where the properties of the damping device vary with operating temperature, the tests of Section 10.6.2.2, items 2(a) to 2(d), shall be performed on at least one device, at a minimum of two additional temperatures (minimum and maximum), such that it covers the entire design temperature range.

   Exception: Use of alternative procedures for testing damping devices are permitted, provided that all of the following conditions are met:

   a) Alternative testing procedures are equivalent to the cyclic testing requirements of this section;
   b) Alternative procedures include the dependence of the damping device response on ambient temperature, loading frequency, and temperature rise during testing;
   c) Alternative procedures shall be approved by the RDP who is responsible for the design of the structure.

4. If the force-deformation properties of the damping device at any displacement less than, or equal to, the maximum device displacement, change by more than 15% when changing the testing frequency from $0.67/T_1$ to $2.5/T_1$, then the previous tests (2(a)) through 2(c)) shall also be performed at frequencies equal to $1/T_1$ and $2.5/T_1$.

   Exception: When full-scale dynamic testing is not possible due to limitations of the test equipment, it is permitted to use reduced-scale prototypes to qualify the rate-dependent characteristics of damping devices, provided scaling principles and similitude are used in the design of the reduced-scale devices and test protocol.

10.6.2.3. Testing Similar Devices

Prototype tests need not be conducted on a particular damping device, if there is a previously prototype tested unit that meets all the following conditions:

1. It has dimensional specifications, internal construction, and static and dynamic internal pressures (if any) similar to the subject damping device; and,
2. It is of the same type and materials as the subject damping device; and,
3. It was made using the same documented manufacturing and quality control procedures as for the subject damping device; and,
4. It was tested under the same maximum displacement range and forces to those required of the subject damping device.

10.6.2.4. Determination of Force-Velocity-Displacement Characteristics

The force-velocity-displacement characteristics of the prototype damping device shall be based on the cyclic displacement tests specified in Section 10.6.2.2, and all the following requirements:

1. The maximum force and minimum force at zero displacement, the maximum force and minimum force at maximum device displacement, and the hysteresis loop area (E_{loop}) shall be determined for each deformation cycle. As required, the effective stiffness of a damping device shall be determined for each deformation cycle using Equation 9.17.
2. Damping device nominal test characteristics for analysis and design shall be determined based on the average value for the first three cycles of test at a given displacement. For each cycle in each test, the Lambda (λ_{test}) factors related to cyclic effects shall be established by comparison of nominal characteristics and the characteristics of each cycle.
3. When full-scale prototype test data are available, Lambda (λ) factors for velocity and temperature shall be determined along with the fsctors for cyclic effects. In cases where these or similar effects are obtained from separate tests, the Lambda factors shall

be obtained from comparison between the properties determined under prototype test conditions with corresponding properties determined under the range of test conditions applicable to the property variation parameter.

10.6.2.5. Device Adequacy

If all the conditions listed below are met, the performance of a damping device prototype shall be considered adequate. The limits mentioned in the following cases can be increased up to 15% by RDP, who is responsible for the design of the structure, provided that it is shown using analysis that increase of this limit does not have a deleterious effect on the response of the structure.

10.6.2.5.1. Displacement-Dependent Damping Devices

The performance of the prototype displacement-dependent damping devices should be considered appropriate if all the following conditions, based on tests specified in Section 10.6.2.2, are met:

1. For Test 1, there is no signs of damage, including leakage, yielding, or breakage.
2. For a damping device, the maximum force and minimum force at zero displacement for any one cycle of Tests 2, 3 and 4, does not differ by more than 15% from the average maximum and minimum forces at zero displacement, as determined from all cycles in that test at a certain frequency and temperature.
3. For a damping device, the maximum force and minimum force at maximum device displacement for any one cycle of Tests 2, 3 and 4, does not differ by more than 15% from the average maximum and minimum forces at maximum device displacement, as determined from all cycles in that test at a certain frequency and temperature.
4. The area of hysteresis loop, E_{loop}, of a damping device for any one cycle of Tests 2, 3 and 4, does not differ by more than 15% from the average area of hysteresis loop, E_{loop}, as determined from all cycles in that test at a certain frequency and temperature.
5. The average maximum and minimum forces at zero displacement and maximum displacement, and the average area of hysteresis

loop, E_{loop}, obtained in each test through the sequence of Tests 2, 3 and 4, shall not differ by more than 15% from the target values specified by the RDP who is responsible for the design of the structure.

6. The average maximum and minimum forces at zero displacement and maximum displacement, and the average area of hysteresis loop, E_{loop}, as determined for test 2(c), shall be within the limits specified by the RDP, as described by the nominal characteristics and the Lambda factors for the specification tolerance, $\lambda_{(spec,max)}$ and $\lambda_{(spec,min)}$ from Section 10.2.4.5.
7. The test lambda factors for damping units, determined in accordance with Section 10.6.2.4, shall not exceed the values specified by the RDP, in accordance with Section 10.2.4.5.

10.6.2.5.2. Velocity-Dependent Damping Devices

The performance of the prototype velocity-dependent damping devices shall be considered appropriate if all the following conditions, based on tests specified in Section 10.6.2.2, are met:

1. For Test 1, there is no signs of damage, including leakage, yielding, or breakage.
2. For velocity-dependent damping devices with stiffness, the effective stiffness of a damping device for any cycle of Tests 2, 3 and 4, does not differ by more than 15% from the average effective stiffness, as determined from all cycles in that test at a certain frequency and temperature.
3. For a damping device, the maximum force and minimum force at zero displacement for any cycle of Tests 2, 3 and 4, does not differ by more than 15% from the average maximum and minimum forces at zero displacement, as determined from all cycles in that test at a certain frequency and temperature.
4. The area of hysteresis loop, E_{loop}, of a damping device for any one cycle of Tests 2, 3 and 4, does not differ by more than 15% from the average area of hysteresis loop, E_{loop}, as determined from all cycles in that test at a certain frequency and temperature.

5. The average maximum and minimum forces at zero displacement, effective stiffness (only for damping devices with stiffness), and average area of the hysteretic loop, E_{loop}, determined for Test 2(c) shall be within the range specified by the RDP, as described based on the nominal characteristics and the Lambda factor for specification tolerance, $\lambda_{(spec,max)}$ and $\lambda_{(spec,min)}$ from Section 10.2.4.5.
6. The test lambda factors for damping units, determined in accordance with Section 10.6.2.4, shall not exceed the values specified by the RDP, in accordance with Section 10.2.4.5.

10.6.3. Production Tests

Damping devices, before installation in a building, shall be tested in accordance with the requirements of this section.

A test prpgram for the production damping devices shall be stablished by the RDP. The nominal properties shall be validated by the test program, by testing all the devices for three cycles at 0.67 times the MCE_R displacement at a frequency equal to $0.67/T_1$. The measured values of the nominal properties shall be within the ranges provided in the project specifications. These ranges shall agree with the specification tolerances on nominal design characteristics established in Section 10.2.4.5.

Exception: If it can be shown in other ways that the characteristics of the damping devices meet the requirements of the project specifications, there is no need to perform this test program. In such cases, the RDP shall provide an alternative program to ensure the quality of the installed damping devices. This alternative program shall include production testing of at least one device of each type and size, unless project-specific prototype tests have been performed on that identical device type and size. Devices in which inelastic action is observed, or are damaged in some way during this test, shall not be used in construction.

10.7 Alternate Procedures and Corresponding Acceptance Criteria

Structures that are analyzed by the response spectrum procedure shall meet the requirements of Sections 10.7.1, 10.7.3, and 10.7.4. Structures analyzed by the equivalent lateral force procedure shall meet the requirements of Sections 10.7.2, 10.7.3, and 10.7.4.

10.7.1. Response Spectrum Procedure

In cases where the response spectrum procedure is used to analyze a structure with a damping system, the requirements of this section shall apply.

10.7.1.1. Modeling

A mathematical model that represents the spatial distribution of mass, stiffness and damping all over the structure shall be developed for the seismic force-resisting system and the damping system. The model and analysis shall conform the requirements of Section 4.11 for the seismic force-resisting system, and to the requirements of this section for the damping system. The stiffness and damping properties of the damping devices used in the analytical models shall either be based on, or verified by, testing of the damping devices, as specified in Section 10.6.

The elastic stiffness of the damping system elements, other than damping devices, shall be explicitly defined in the model. Stiffness of damping devices shall be modeled depending on type of damping device as follows:

1. For displacement-dependent damping devices: Displacement-dependent damping devices shall be modeled with an effective stiffness that corresponds to damping device force at the response displacement of interest (e.g., design story drift). Alternatively, the stiffness of hysteretic and friction devices is allowed to be removed from response spectrum analysis, provided that design forces created in displacement-dependent damping devices, Q_{DSD},

are applied to the analytical model as external loads (Section 10.7.4.5).

2. For velocity-dependent damping devices: Velocity-dependent damping devices that have a stiffness component (e.g., viscoelastic damping devices) shall be modeled with an effective stiffness corresponding to the amplitude and frequency of interest.

10.7.1.2. Seismic Force-Resisting System

10.7.1.2.1 Seismic Base Shear

The seismic base shear, V, of the structure in a specific direction, shall be determined as the combination of modal components, V_m, subject to the constraints of Eq. 10.5:

$$V \geq V_{\min} \tag{10.5}$$

The seismic base shear, V, of the structure shall be determined by the square root of the sum of the squares (SRSS) method or complete quadratic combination (CQC) of modal base shear components, V_m.

10.7.1.2.2 Modal Base Shear

Modal base shear corresponding to the m^{th} mode of vibration, V_m, of the structure in the direction under consideration, shall be determined as follows:

$$V_m = C_{Sm} \bar{W}_m \tag{10.6a}$$

$$\bar{W}_m = \frac{\left(\sum_{n}^{i=1} w_i \phi_{im} \right)^2}{\sum_{n}^{i=1} w_i \phi_{im}^2} \tag{10.6b}$$

where C_{Sm} is the seismic response coefficient of the m^{th} mode of vibration of the structure in the direction under consideration, as determined from Section 10.7.1.2.4 (m=1) or Section 10.7.1.2.6 (m>1); $\bar{W}_m$ is the effective seismic weight of the m^{th} mode of vibration of the structure; and ϕ_{im} is the displacement amplitude at level i of the structure in the m^{th} mode of vibration in the direction under consideration, scaled to unity at the roof level.

10.7.1.2.3. Modal Participation Factor

The modal participation factor of the mth mode of vibration, Γ_m, of the structure in the direction under consideration, is determined as follows:

$$\Gamma_m = \frac{\bar{W}_m}{\sum_{n}^{i=1} w_i \phi_{im}} \quad 10.7$$

10.7.1.2.4 Fundamental Mode Seismic Response Coefficient

Seismic response coefficient of the fundamental mode (m=1), C_{S1}, in the direction under consideration, shall be determined as follows:

For $T_{1D} < T_S$,

$$C_{S1} = \left(\frac{R}{C_d}\right)\frac{S_{DS}}{\Omega_0 B_{1D}} \quad 10.8$$

For $T_{1D} \geq T_S$,

$$C_{S1} = \left(\frac{R}{C_d}\right)\frac{S_{D1}}{T_{1D}(\Omega_0 B_{1D})} \quad 10.9$$

where Ts = (S_{D1}/S_{DS}); S_{DS} is the design spectral acceleration parameter (g) corresponding to short period (0.2 sec), for 5% damping ratio; S_{D1} is the design spectral acceleration parameter (g) corresponding to period 1 sec, for 5% damping ratio; B_{1D} is the numerical coefficient, specified in Table 10.1, for effective damping ratio equal to βmD (m=1) and period of structure equal to T_{1D}.

10.7.1.2.5. Effective Fundamental Mode Period Determination

The effective period of the fundamental mode (m=1) at the design earthquake ground motion, T_{1D}, and at the MCE_R ground motion, T_{1M}, shall be based on either explicit consideration of the post-yield force deflection characteristics of the structure, or determined as follows:

$$T_{1D} = T_1\sqrt{\mu_D} \quad 10.\ 10$$

$$T_{1M} = T_1\sqrt{\mu_M} \qquad 10.11$$

where μ_D is the effective ductility demand on the seismic force-resisting system in the direction under consideration due to the design earthquake ground motions, Section 10.7.3.3; and μ_M is the effective ductility demand on the seismic force-resisting system in the direction under consideration due to the MCE_R ground motions, Section 10.7.3.3.

10.7.1.2.6. Higher Mode Seismic Response Coefficient

The seismic response coefficient, C_{Sm}, of the higher mode (m>1) corresponding to the mth mode of vibration of the structure in the direction under consideration, shall be determined as follows:

For $T_m<T_S$:

$$C_{Sm} = \left(\frac{R}{C_d}\right)\frac{S_{DS}}{\Omega_0 B_{mD}} \qquad 10.12$$

For $T_m \geq T_S$:

$$C_{Sm} = \left(\frac{R}{C_d}\right)\frac{S_{D1}}{T_m(\Omega_0 B_{mD})} \qquad 10.13$$

where T_m (sec) is the period of the m^{th} mode of vibration of the structure in the direction under consideration; and B_{mD} is the numerical coefficient, specified in Table 10.1, for the total effective damping ratio of the m^{th} mode of vibration of the structure in the direction under consideration at the design displacement, Section 10.7.3.2.

10.7.1.2.7. Design Lateral Force

Design lateral force at level i of the structure caused by the vibrations at the mth mode, F_{im}, in the direction under consideration shall be determined as follows:

$$F_{im} = w_i \phi_{im} \frac{\Gamma_m}{\bar{W}_m} V_m \qquad 10.14$$

Design forces in elements of the seismic force-resisting system shall be determined by combining the modal design forces by the SRSS or CQC method.

10.7.1.3. Damping System

Design forces in damping devices and other elements of the damping system shall be determined based on the response parameters of the floor deflection, story drift, and story velocity which are stated in the following sections.
Displacements and velocities used to determin maximum forces in damping devices at each story shall account for the angle of orientation of each device from the horizontal and consider the effects of increased response due to torsion required for design of the seismic force-resisting system.
Floor deflections at level i, δ_{iD} and δ_{iM}, story drifts, Δ_D and Δ_M, and story velocities, ∇_D and ∇_M, shall be determined for both the design earthquake ground motions and the MCE_R ground motions, respectively, in accordance with this section.

10.7.1.3.1. Design Earthquake Floor Deflection

The deflection of structure caused by the design earthquake ground motions at level i of the structure in the m^{th} mode of vibration, δ_{imD}, in the direction under consideration shall be determined as follows:

$$\delta_{imD} = D_{mD}\phi_{im} \tag{10.15}$$

The total design deflection at each floor of the structure, shall be determined by combining the modal design earthquake deflections by the SRSS or CQC method.

10.7.1.3.2. Design Earthquake Roof Displacement

Fundamental (m=1) and higher mode (m>1) roof displacements due to the design earthquake ground motions, D_{1D} and D_{mD}, of the structure in the direction under consideration, shall be determined as follows:
for m=1,

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{DS}T_{1D}^2}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{DS}T_1^2}{B_{1E}}, \quad T_{1D} < T_S \tag{10.16a}$$

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{D1}T_{1D}}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{D1}T_1}{B_{1E}}, \quad T_{1D} \geq T_S \qquad 10.16b$$

For m>1:

$$D_{mD} = \left(\frac{g}{4\pi^2}\right)\Gamma_m \frac{S_{D1}T_m}{B_{mD}} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_m \frac{S_{DS}T_m^2}{B_{mD}} \qquad 10.17$$

10.7.1.3.3. Design Earthquake Story Drift

Design story drift in the fundamental mode, Δ_{1D}, and higher modes, Δ_{mD} (m>1), of the structure in the direction under consideration, shall be determined in accordance with Section 4.10.8 using the modal roof displacements of Section 10.7.1.3.2.

The total design story drift, Δ_D, shall be determined by combining the modal design earthquake drifts by the SRSS or CQC method.

10.7.1.3.4. Design Earthquake Story Velocity

Design story velocity in the fundamental mode, ∇_{1D}, and higher modes, ∇_{mD} (m>1), of the structure in the direction under consideration, shall be determined as follows:

$$\text{For } m=1, \quad \nabla_{1D} = 2\pi \frac{\Delta_{1D}}{T_{1D}} \qquad 10.18$$

$$\text{For } m>1, \quad \nabla_{mD} = 2\pi \frac{\Delta_{mD}}{T_m} \qquad 10.19$$

The total design story velocity, ∇_D, shall be determined by combining modal design velocities by the SRSS or CQC method.

10.7.1.3.5. MCE$_R$ Response

Total maximum floor deflection at level i, MCE$_R$ story drift values, MCE$_R$ story velocity values are specified respectively based on Sections 10.7.1.3.1, 10.7.1.3.3 and 10.7.1.3.4, exception that design roof displacement shall be replaced by MCE$_R$ roof displacement. MCE$_R$ roof displacement of the structure in the direction under consideration shall be determined as follows:

For m=1,

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_{1M}^2}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_1^2}{B_{1E}}, \quad T_{1M} < T_S \qquad 10.20a$$

$$D_{1M}=\left(\frac{g}{4\pi^2}\right)\Gamma_1\frac{S_{M1}T_{1M}}{B_{1M}}\geq\left(\frac{g}{4\pi^2}\right)\Gamma_1\frac{S_{M1}T_1}{B_{1E}},\quad T_{1M}\geq T_S \tag{10.20b}$$

For $m > 1$,

$$D_{mM}=\left(\frac{g}{4\pi^2}\right)\Gamma_m\frac{S_{M1}T_m}{B_{mM}}\leq\left(\frac{g}{4\pi^2}\right)\Gamma_m\frac{S_{MS}T_m^2}{B_{mM}} \tag{10.21}$$

where B_{mM} is the numerical coefficient presented in Table 10.1, for the effective damping raio of β_{mM}, and period of structure equal to T_m.

10.7.2. Equivalent Lateral Force Procedure

Where the equivalent lateral force procedure is used to design a structure with a damping system, the requirements of this section shall apply.

10.7.2.1. Modeling

Elements of the seismic force-resisting system shall be modeled in a manner consistent with the requirements of Section 4.9. For purposes of analysis, the base of the structure shall be considered as a fixed-base.

Elements of the damping system shall be modeled as required to determine design forces transferred from damping devices to both the ground and the seismic force-resisting system. The effective stiffness of velocity-dependent damping devices shall be modeled.

Damping devices need not be explicitly modeled provided that calculated effective damping in accordance with the procedures of Section 10.7.4 and used to modify response, as required in Sections 10.7.2.2 and 10.7.2.3.

The stiffness and damping characteristics of the damping devices used in the analytical models shall be based on, or verified by, testing of the damping devices, as specified in Section 10.6.

10.7.2.2 Seismic Force-Resisting System

10.7.2.2.1. Seismic Base Shear

The seismic base shear, V, of the seismic force-resisting system in a specific direction, shall be determined by combining the two modal components, V_1 and V_R, in accordance with Eq. 10.22:

$$V = \sqrt{V_1^2 + V_R^2} \geq V_{\min} \tag{10.22}$$

where V_1 is the seismic design value of the seismic base shear force of the fundamental mode of the structure in the direction under consideration, as determined in Section 10.7.2.2.2; V_R is the design value of the seismic base shear of the residual mode of vibration of the structure in the direction under consideration, as determined in Section 10.7.2.2.6; and, V_{min} is the minimum allowable value of the base shear that may be used for design of the seismic force-resisting system of the structure in the direction under consideration, as determined in Section 10.2.1.1.

10.7.2.2.2. Fundamental Mode Base Shear

The fundamental mode base shear, V_1, shall be determined in accordance with Eq. 10.23:

$$V_1 = C_{S1} \overline{W}_1 \tag{10.23}$$

where C_{S1} is the fundamental mode seismic response coefficient of the structure in the direction under consideration, as determind in Section 10.7.2.2.4; and, $\overline{W_1}$ is the effective fundamental seismic weight, including portions of the live load, as determined by Eq. 10.6b (m=1).

10.7.2.2.3. Fundamental Mode Properties

The fundamental mode shape, φ_{i1}, and participation factor, Γ_1, shall be determined by either dynamic analysis using the elastic structural properties and deformational characteristics of the resisting elements or using Eqs. 10.24 and 10.25:

$$\phi_{i1} = \frac{h_i}{h_n} \tag{10.24}$$

$$\Gamma_1 = \frac{\overline{W}_1}{\sum_{i=1}^{n} w_i \phi_{i1}} \tag{10.25}$$

where h_i is the height of level i from the base level; h_n is the structural height; and, w_i is the portion of the total effective seismic weight, W, located at or assigned to level i.

The fundamental period, T_1, shall be determined either from dynamic analysis using the elastic structural properties and deformational characteristics of the resisting elements, or using Eq. 10.26 as follows:

$$T_1 = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} f_i \delta_i}} \tag{10.26}$$

where f_i is the lateral force at level i of the structure distributed in accordance with Sections 4.10.4; and, δ_i is the elastic deflection at level i of the structure caused by applied lateral force, f_i.

10.7.2.2.4. Fundamental Mode Seismic Response Coefficient

The fundamental mode seismic response coefficient, C_{S1}, shall be determined using Eq. 10.27 or 10.28 as follows:

For $T_{1D} < T_S$,

$$C_{S1} = \left(\frac{R}{C_d}\right) \frac{S_{DS}}{\Omega_0 B_{1D}} \tag{10.27}$$

For $T_{1D} \geq T_S$,

$$C_{S1} = \left(\frac{R}{C_d}\right) \frac{S_{D1}}{T_{1D}(\Omega_0 B_{1D})} \tag{10.28}$$

where S_{DS} is the design spectral response acceleration parameter corresponding to the short period (0.2 sec) for 5% damping ratio; S_{D1} is the design spectral response acceleration parameter corresponding to the 1 sec period, for 5% damping ratio; and, B_{1D} is the numerical coefficient, specified in Table 10.1, for the effective damping ratio corresponding to β_{mD} for the fundamental mode when period of the structure is equal to T_{1D}.

10.7.2.2.5. Effective Fundamental Mode Period Determination

The effective fundamental mode period at the design earthquake, T_{1D}, and at the MCE_R, T_{1M}, shall be determined based on explicit consideration of postyield force deflection characteristics of the structure or shall be determined using Eqs. 10.29 and 10.30 as follows:

$$T_{1D} = T_1\sqrt{\mu_D} \tag{10.29}$$

$$T_{1M} = T_1\sqrt{\mu_M} \tag{10.30}$$

10.7.2.2.6. Residual Mode Base Shear

The residual mode base shear, V_R, shall be determined in accordance with Eq. 10.31:

$$V_R = C_{SR}\overline{W}_R \tag{10.31}$$

where C_{SR} is the residual mode seismic response coefficient of vibration of the structure in the direction under consideration, Section 10.7.2.2.8; and, $\overline{W_R}$= is the eEffective residual mode seismic weight of the structure, as determined by Eq. 10.34.

10.7.2.2.7. Residual Mode Properties

The residual mode shape, φ_{iR}, participation factor, Γ_R, effective seismic weight, $\overline{W_R}$, and effective period, T_R, shall be determined using Eqs. 10.32 to 10.35, as follows:

$$\phi_{iR} = \frac{1-\Gamma_1\phi_{i1}}{1-\Gamma_1} \tag{10.32}$$

$$\Gamma_R = 1-\Gamma_1 \tag{10.33}$$

$$\overline{W}_R = W-\overline{W}_1 \tag{10.34}$$

$$T_R = 0.4T_1 \tag{10.35}$$

10.7.2.2.8. Residual Mode Seismic Response Coefficient

The residual mode seismic response coefficient, C_{SR}, shall be determined in accordance with Eq. 10.36, as follows:

$$C_{SR} = \left(\frac{R}{C_d}\right)\frac{S_{DS}}{\Omega_0 B_R} \qquad 10.36$$

where B_R is the numerical coefficient presented in Table 10.1, for the effective damping ratio β_R, and the period of the structure equal to T_R.

10.7.2.2.9. Design Lateral Force

The design lateral force in elements of the seismic force-resisting system at level i due to the fundamental mode response, F_{i1}, and residual mode response, F_{iR}, of the structure in the direction under consideration in accordance with Eqs. 10.37 and 10.38, as follows:

$$F_{i1} = w_i \phi_{i1} \frac{\Gamma_1}{\bar{W}_1} V_1 \qquad 10.37$$

$$F_{iR} = w_i \phi_{iR} \frac{\Gamma_R}{\bar{W}_R} V_R \qquad 10.38$$

Design forces in elements of the seismic force-resisting system shall be determined by combining the forces caused by the fundamental mode and the residual mode of vibration using the SRSS method.

10.7.2.3. Damping System

Design forces in damping devices and other elements of the damping system shall be determined based on the response parametrs of the floor deflection, story drift, and story velocity, which are stated in the following sections.

Displacements and velocities used to determin maximum forces in damping devices at each story, shall account for the angle of orientation of each device from the horizonal and consider the effects of increased response due to torsion required for design of the force-resisting system.

Floor deflections at level i, δ_{iD} and δ_{iM}, story drifts, Δ_D and Δ_M, and story velocities, ∇_D and ∇_M, shall be respectively determined for both the design earthquake ground motions and the MCE_R ground motions, in accordance with the following sections.

10.7.2.3.1. Design Earthquake Floor Deflection

The total design floor deflection at each floor of the structure in the direction under consideration shall be determined by combining the fundamental and residual mode deflections using the SRSS method. The fundamental and residual mode floor deflections due to the design earthquake ground motions, δ_{i1D} and δ_{iRD}, at the center of rigidity of level i of the structure in the direction under consideration shall be determined using Eqs. 10.39 and 10.40 as follows:

$$\delta_{i1D} = D_{1D}\phi_{i1} \tag{10.39}$$

$$\delta_{iRD} = D_{RD}\phi_{iR} \tag{10.40}$$

where D_{1D} is the fundamental mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 10.7.2.3.2; and, D_{RD} is the residual mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 10.7.2.3.2.

10.7.2.3.2. Design Earthquake Roof Displacement

The fundamental and residual mode displacements caused by the design earthquake ground motions, D_{1D} and D_{1R}, at the center of rigidity of the roof level of the structure in the direction under consideration shall be determined using Eqs. 10.41 and 10.42 as follws:

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{DS}T_{1D}^2}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{DS}T_1^2}{B_{1E}}, \quad T_{1D} < T_S \tag{10.41a}$$

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{D1}T_{1D}}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{D1}T_1}{B_{1E}}, \quad T_{1D} \geq T_S \tag{10.41b}$$

$$D_{RD} = \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{D1}T_R}{B_R} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{DS}T_R^2}{B_R} \tag{10.42}$$

10.7.2.3.3. Design Earthquake Story Drift

The design story drifts, Δ_D, in the direction under consideration shall be determined using Eq. 10.43 as follows:

$$\Delta_D = \sqrt{\Delta_{1D}^2 + \Delta_{RD}^2} \tag{10.43}$$

where Δ_{1D} is the fundamental mode design story drift of the structure in the direction under consideration; and, Δ_{RD} is the residual mode design story drift of the structure in the direction under consideration.

Modal design story drifts, Δ_{1D} and Δ_{RD}, shall be determined by calculating the difference between the deflections of the top and bottom of the story under consideration using the floor deflections of Section 10.7.2.3.1.

10.7.2.3.4. Design Earthquake Story Velocity

The design velocity of the floor, ∇_D, in the direction under consideration shall be determined in accordance with Eqs. 10.44 to 10.46 as follows:

$$\nabla_D = \sqrt{\nabla_{1D}^2 + \nabla_{RD}^2} \tag{10.44}$$

$$\nabla_{1D} = 2\pi \frac{\Delta_{1D}}{T_{1D}} \tag{10.45}$$

$$\nabla_{RD} = 2\pi \frac{\Delta_{RD}}{T_R} \tag{10.46}$$

where ∇_{1D} is the fundamental mode design story velocity of the structure in the direction under consideration; and, ∇_{RD} is the residual mode design story velocity of the structure in the direction under consideration.

10.7.2.3.5. MCE$_R$ Response

The total modal MCE$_R$ floor deflection at level *i*, maximum story drifts, and maximum story velocities shall be respectively determined using the equations in Sections 10.7.2.3.1, 10.7.2.3.3, and 10.7.2.3.4, except that design roof displacements shall be replaced by MCE$_R$ roof displacements. The MCE$_R$ roof displacements shall be determined in accordance with Eqs. 10.47 and 10.48 as follows:

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_{1M}^2}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_1^2}{B_{1E}}, \quad T_{1M} < T_S \tag{10.47a}$$

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_{1M}}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_1}{B_{1E}}, \quad T_{1M} \geq T_S \tag{10.47b}$$

$$D_{RM} = \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{M1}T_R}{B_R} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{MS}T_R^2}{B_R} \tag{10.48}$$

where S_{M1} is the MCE_R spectral response acceleration parameter corresponding to 1 sec period, for 5% damping ratio; S_{MS} is the MCE_R spectral response acceleration parameter corresponding to short period (0.2 sec), for 5% damping ratio; and, B_{1M} is the numerical coefficient, specified in Table 10.1, for effective damping ratio corresponding to β_{mM} (m=1) period of structure equal to T_{1M}.

10.7.3. Damped Response Modification

Response of the structure shall be modified for the effects of the damping system, as required in Sections 10.7.1 and 10.7.2.

10.7.3.1. Damping Coefficient

In cases where the period of the structure is greater than or equal to T_0, the damping coefficient shall be in accordance with the values specified in Table 10.1. In cases where period of the structure is less than T_0, the damping coefficient shall be linearly interpolated between a value 1.0 for the period of zero second for all values of effective damping, and the value of the damping coefficient corresponding to the period T_0 from Table 10.1.

10.7.3.2. Effective Damping

The effective damping at the design displacement, β_{mD}, and at the MCE_R displacement, β_{mM}, related to the m^{th} mode of vibration of the structure in the direction under consideration, shall be determined by using Eqs. 10.49 and 10.50 as follows:

$$\beta_{mD} = \beta_I + \beta_{Vm}\sqrt{\mu_D} + \beta_{HD} \tag{10.49}$$

$$\beta_{mM} = \beta_I + \beta_{Vm}\sqrt{\mu_M} + \beta_{HM} \tag{10.50}$$

where β_{HD} is part of effective damping ratio of the structure in the direction under consideration due to postyield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand, μ_D, Section 10.7.3.2.2; β_{HM} is part of effective damping ratio of the structure in the direction under consideration due to post-yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at the effective ductility demand, μ_M, Section 10.7.3.2.2; β_I is part of effective damping ratio of the structure due to the inherent energy dissipation by elements of the structure, at or just below the effective yield displacement of the seismic force-resisting system, Section 10.7.3.2.1; β_{Vm} is part of effective damping ratio of the m^{th} mode of vibration of the structure in the direction under consideration due to viscous dissipation of energy by the damping system, at or just below the effective yield displacement of the seismic force-resisting system, Section 10.7.3.2.3; μ_D is the effective ductility demand on the seismic force-resisting system in the direction under consideration due to the design earthquake ground motions, Section 10.7.3.3; and, μ_M is the effective ductility demand on the seismic force-resisting system in the direction under consideration due to the MCE_R earthquake ground motions, Section 10.7.3.3.

The effective ductility demand of higher modes of vibration in the direction under consideration shall be taken as 1.0, unless analysis or test data support other values.

10.7.3.2.1. Inherent Damping

The inherent damping ratio, β_I shall be determined based on the type of materials, configuration, and dynamic behavior of the structure and non-structural components at, or just below yield of the seismic force-resisting system. Inherent damping ratio of all modes of vibration shall be taken as not greater than 3% of critical, unless analysis or test data support other values.

10.7.3.2.2. Hysteretic Damping

The hysteretic damping ratio of the seismic force-resisting system and elements of the damping system shall be determined based on test or analysis, or using Eqs. 10.51 and 10.52 as follows:

$$\beta_{HD} = q_H (0.64 - \beta_I)\left(1 - \frac{1}{\mu_D}\right) \tag{10.51}$$

$$\beta_{HM} = q_H (0.64 - \beta_I)\left(1 - \frac{1}{\mu_M}\right) \tag{10.52}$$

where q_H is the hysteresis loop adjustment factor, in accordance with the definition of Section 10.7.3.2.2.1; μ_D is the effective ductility demand on the seismic force-resisting system in the direction under consideration due to the design earthquake ground motions, Section 10.7.3.3; and, μ_M is the effective ductility demand on the seismic force-resisting system in the direction under consideration due to the MCE_R earthquake ground motions, Section 10.7.3.3.

The hysteretic damping ratio of higher modes of vibration in the direction under consideration shall be taken as zero, unless analysis or test data support other values.

10.7.3.2.2.1. Hysteresis Loop Adjustment Factor

The calculation of hysteretic damping ratio of the seismic force-resisting system and elements of the damping system, shall consider the effects of pinching and other effects that reduce the area of the hysteresis loop during repeated cycles of earthquake demand. Unless analysis or test data support other values, the fraction of the total area of hysteretic loop of the seismic force-resisting system used for design shall be considered as equal to the factor q_H, in accordance with Eq. 10.53 as follows:

$$q_H = 0.67 \frac{T_S}{T_1} \tag{10.53}$$

where $T_S = S_{D1}/S_{DS}$, and, T_1 is the fundamental period of the structure in the direction under consideration.

The value of q_H shall not be taken as greater than 1.0 and need not be taken as less than 0.5.

10.7.3.2.3. Viscous Damping

Viscous damping ratio of the m^{th} mode of vibration, β_{Vm}, shall be determined using Eqs. 10.54 and 10.55 as follows:

$$\beta_{Vm} = \frac{\sum_j W_{mj}}{4\pi W_m} \tag{10.54}$$

$$W_m = \frac{1}{2}\sum_j F_{im}\delta_{im} \tag{10.55}$$

where W_{mj} is the work done by the j^{th} damping device in one complete cycle of dynamic response corresponding to the m^{th} mode of vibration of the structure in the direction under consideration at modal displacements, δ_{im}, Section 10.7.3.2.2.1; W_m is the maximum strain energy in the m^{th} mode of vibration of the structure in the direction under consideration at the modal displacements δ_{im}, Section 10.7.3.2.2.1; F_{im} is the m^{th} mode inertial force at the level i, Section 10.7.1.2.7; and δ_{im} is the deflection of level i in the m^{th} mode of vibration at the center of rigidity of the structure in the direction under consideration, Section 10.7.3.2.3.

Viscous damping ratio of displacement-dependent damping devices shall be based on a response amplitude equal to the effective yield displacement of the structure.

The calculation of the work done by individual damping devices shall consider orientation and participation of each device with respect to the mode of vibration of interest. As required, to account for the flexibility of elements, including pins, bolts, gusset plates, brace extensions, and other components that connect damping devices to other elements of the structure, the work done by individual damping devices shall be reduced.

10.7.3.3. Effective Ductility Demand

The effective ductility demand on the seismic force-resisting system due to design earthquake ground motions, μ_D, and due to the MCE$_R$ ground motions, μ_M, shall be determined by using Eqs. 10.56 to 10.58 as follows:

$$\mu_D = \frac{D_{1D}}{D_Y} \geq 1.0 \tag{10.56}$$

$$\mu_M = \frac{D_{1M}}{D_Y} \geq 1.0 \tag{10.57}$$

$$D_Y = \left(\frac{g}{4\pi^2}\right)\left(\frac{\Omega_0 C_d}{R}\right)\Gamma_1 C_{S1} T_1^2 \tag{10.58}$$

where D_{1D} is the fundamental design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 10.7.1.3.2 or 10.7.2.3.2; D_{1M} is the fundamental mode displacement at the center of rigidity of the roof level of the structure caused by the MCE_R earthquake in the direction under consideration, Section 10.7.2.3.5; D_Y is the displacement at the center of rigidity of the roof level of the structure at effective yield point of the seismic force-resisting system; R is the response modification coefficient from Table 4.5; C_d is the deflection amplification factor from Table 4.5; Ω_0 is the overstrength factor from Table 4.5; Γ_1 is the participation factor of the fundamental mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.3 or 10.7.2.2.3 (m=1); C_{S1} is the seismic response coefficient of the fundamental mode of vibration of the structure in the direction under consideration, Section 10.7.1.2.4 or 10.7.2.2.4 (m=1); and, T_1 is the period of the fundamental mode of vibration on the structure in the direction under consideration.

The design ductility demand, μ_D, shall not exceed the maximum value of the effective ductility demand, μ_{max}, provided in Section 10.7.3.4.

Exception: Use of nonlinear modeling mentioned in Section 10.3, is permitted to develope a force-displacement (pushover) curve of the seismic force-resisting system. It is allowed to use this curve instead of the effective yield displacement, D_Y, of Eq. 10.58, to determine the effective ductility demand due to the design earthquake ground motions, μ_D, in Eq. 10.56, and due to the MCE_R ground motions, μ_M, in Eq. 10.57. In this case, the value of (R/C_d)

shall be taken as 1.0 in Eqs. 10.8, 10.9, 10.12, and 10.13.

10.7.3.4. Maximum Effective Ductility Demand

To determine the hysteresis loop adjustment factor, hysteretic damping ratio, and other parameters, the maximum value of effective ductility demand, μ_{max}, shall be determined using Eqs. 10.59 and 10.60 as follows:

For $T_{1D} \leq T_S$,

$$\mu_{max} = 0.5[(R/(\Omega_0 I_e))^2 + 1] \qquad 10.59$$

For $T_{1D} \geq T_S$,

$$\mu_{max} = R/(\Omega_0 I_e) \qquad 10.60$$

where I_e is the seismic importance factor determined in accordance with Section 4.3; and, T_{1D} is the effective period of the fundamental mode of vibration of the structure at the design displacement in the direction under consideration.

For $T_1 < T_S < T_{1D}$, μ_{max} is determined by linear interpolation between the values of Eqs. 10.59 and 10.60.

10.7.4. Seismic Load Conditions and Acceptance Criteria for RSA and ELF Procedures

Design forces and displacements determined in accordance with the response spectrum procedure of Section 10.7.1, or the equivalent lateral force procedure of Section 10.7.2, shall be checked using the strength design criteria of this Regulations and the seismic loading conditions of of Section 10.7.4.3.

The sismic force-resisting system, damping system, seismic loading conditions, and acceptance criteria shall comply with the following subsections.

10.7.4.1. Seismic Force-Resisting System

The seismic force-resisting system shall meet the requirements of Section 4.5 using seismic base shear and design forces determined in accordance with Section 10.7.1.2 or 10.7.2.2.

The design story drift, Δ_D, as determined in either Section 10.7.1.3.3 or 10.7.2.3.3, shall not exceed (R/C_d) times the allowable story drift, as obtained from Table 4.10, including the torsion effects as required in Section 4.15.

10.7.4.2. Damping System

The damping system shall meet the requirements of Section 4.5 for seismic design forces, and seismic loading conditions determined in accordance with Section 10.7.4.3. Force-controlled elements of the damping system shall be designed for seismic forces that are increased by 20% from those values corresponding to average MCE_R response.

10.7.4.3. Combination of Load Effects

The effects of gravity loads and seismic forces on the damping system and its components shall be combined in accordance with Section 2.2.3, using the effect of horizontal seismic forces, Q_E, determined in accordance with Section 10.7.4.5. The redundancy factor, ρ, shall be taken as equal to 1.0 in all cases, and for the design of the damping system, there is no need to apply the seismic load effect including overstrength of Section 2.2.4.

10.7.4.4. Modal Damping System Design Forces

Modal damping system design forces shall be determined based on the type of damping devices and modal design story displacements and velocities determined in accordance with Sections 10.7.1.3 or 10.7.2.3.

Modal design story displacements and velocities shall be increased as required to envelope the total design story displacements and velocities of the floor determined in accordance with Section 10.3, where peak response is required to be confirmed br response history analysis.

For displacement-dependent damping devices, the design seismic force shall be determined based on the maximum force produced in the device at displacements up to, and including, the design story drift, Δ_D.

For velocity-dependent damping devices, the design seismic force in each mode of vibration shall be determined based on the maximum force produced in the device at velocities up to, and including, the design story velocity for the mode of interest.

Displacements and velocities that are used to determine the design forces in damping devices at each story shall incorporate the angle of orientation of the damping device from the horizontal and consider the effects of increased floor response due to torsional motions.

10.7.4.5. Seismic Load Conditions and Combination of Modal Responses

Seismic design force, Q_E, in each element of the damping system shall be considered equal to the maximum force obtained from the folloeing three loading conditions:

1. Maximum displacement stage: Seismic design force at the maximum displacement stage shall be determined in accordance with Eq. 10.61 as follows:

$$Q_E = \Omega_0 \sqrt{\sum_m (Q_{mSFRS})^2} \pm Q_{DSD} \tag{10.61}$$

where Q_{mSFRS} is the force in an element of the seismic force-resisting system equal to the seismic design force of the m^{th} mode of vibration of the structure in the direction under consideration; and, Q_{DSD} is the force in an element of the damping system required to resist seismic design forces of displacement-dependent damping devices.

Seismic forces in elements of the damping system, Q_{DSD}, shall be determined by applying design forces of displacement-dependent damping devices on the damping system as pseudostatic forces. Seismic design forces of displacement-dependent damping devices shall be applied in both positive and negative directions at peak displacement of the structure.

2. Maximum velocity stage: Seismic design force at the maximum velocity stage shall be determined in accordance with Eq. 10.62 as follows:

$$Q_E = \sqrt{\sum_m (Q_{mDSV})^2} \qquad 10.62$$

where Q_{mDSV} is the force in an element of the damping system required to resist seismic design forces of velocity-dependent damping devices, caused by the m^{th} mode of vibration of the structure in the direction under consideration.

Modal seismic design forces in elements of damping system, Q_{mDSV}, shall be determined by applying modal design forces of velocity-dependent devices on the nondeformed damping system as pseudostatic forces. Modal seismic design forces shall be applied in directions that are consistent with the deformed shape of the mode of interest. Horizontal restraint forces shall be applied at each floor level i, of the nondeformed damping system, concurrent with the design forces in velocity-dependent damping devices, such that the horizontal displacement at each level of the structure is zero. The restraint forces at each floor level i, shall be proportional to and applied at the location of each mass point.

3. Maximum acceleration stage: Seismic design force at the maximum acceleration stage shall be determined in accordance with Eq. 10.63 as follows:

$$Q_E = \sqrt{\sum_m (C_{mFD}\Omega_0 Q_{mSFRS} + C_{mFV} Q_{mDSV})^2 \pm Q_{DSD}} \qquad 10.63$$

The force coefficients, C_{mFD} and C_{mFV}, shall be determined from Tables 10.2 and 10.3, respectively, using effective damping ratio values determined in accordance with the following requirements:

For fundamental mode response (m=1) in the direction under consideration, the coefficients C_{1FD} and C_{1FV} shall be determined based on the velocity exponent, α, which relates device force to the damping device velocity. The effective fundamental mode damping shall be considered equal to the total effective damping ratio of the fundamental mode minus the hysteretic component of damping ratio (β_{1D}-β_{HD} or β_{1M}-β_{HM}) at the response level of interest ($\mu = \mu_D$ or $\mu = \mu_M$).

For higher modes (m>1) or residual mode response in the direction under consideration, the coefficients C_{mFD} and C_{mFV} shall be determined based on a value of α equal to 1.0. The effective modal damping ratio shall be considered equal to the total effective damping ratio of the mode of interest (β_{mD} or β_{mM}). To determine the coefficient C_{mFD}, the ductility demand is considered equal to the ductility value of the fundamental mode ($\mu = \mu_D$ or $\mu = \mu_M$).

Table 10.2 Force Coefficient,[a, b] C_{mFD}

Effective Damping	$\mu \le 1.0$[c]				$C_{mFD} = 1.0$
	$\alpha \le 0.25$	$\alpha = 0.5$	$\alpha = 0.75$	$\alpha \ge 1.0$	
≤ 0.5	1.00	1.00	1.00	1.00	$\mu \ge 1.0$
0.1	1.00	1.00	1.00	1.00	$\mu \ge 1.0$
0.2	1.00	0.95	0.94	0.93	$\mu \ge 1.1$
0.3	1.00	0.92	0.88	0.86	$\mu \ge 1.2$
0.4	1.00	0.88	0.81	0.78	$\mu \ge 1.3$
0.5	1.00	0.84	0.73	0.71	$\mu \ge 1.4$
0.6	1.00	0.79	0.64	0.64	$\mu \ge 1.6$
0.7	1.00	0.75	0.55	0.58	$\mu \ge 1.7$
0.8	1.00	0.70	0.50	0.53	$\mu \ge 1.9$
0.9	1.00	0.66	0.50	0.50	$\mu \ge 2.1$
≥ 1.0	1.00	0.62	0.50	0.50	$\mu \ge 2.2$

[a] Unless analysis or test data support other values, the force coefficient C_{mFD} for viscoelastic systems shall be taken as 1.0.

[b] Interpolation shall be used for intermediate values of velocity exponent, α, and ductility demand, μ.

[c] C_{mFD} shall be taken as equal to 1.0 for values of ductility demand, μ, greater than or equal to the values shown.

Table 10.3 Force Coefficient,[a, b] C_{mFV}

Effective Damping	$\alpha \le 0.25$	$\alpha = 0.5$	$\alpha = 0.75$	$\alpha \ge 1.0$
≤ 0.5	1.00	0.35	0.20	0.10
0.1	1.00	0.44	0.31	0.20
0.2	1.00	0.56	0.46	0.37
0.3	1.00	0.64	0.58	0.51
0.4	1.00	0.70	0.69	0.62
0.5	1.00	0.75	0.77	0.71
0.6	1.00	0.80	0.84	0.77
0.7	1.00	0.83	0.90	0.81
0.8	1.00	0.90	0.94	0.90
0.9	1.00	1.00	1.00	1.00
≥ 1.0	1.00	1.00	1.00	1.00

[a] Unless analysis or test data support other values, the force coefficient C_{mFV} for viscoelastic systems shall be taken as 1.0.

[b] Interpolation shall be used for intermediate values of velocity exponent, α.

10.7.4.6. Inelastic Response Limits

Element Forces of the damping system may exceed the required strength limits for design loads, provided that it is shown by analysis or test that all of the following conditions are met:

1. Inelastic response does not adversely affect the function of damping system.

Element forces determined in accordance with Section 10.7.4.5, using a value of 1.0 for Ω_0, do not exceed the strength required to satisfy the load combinations of Section 2.

Chapter 11
Chimney, Stack and flare

11.1. General considerations

In this chapter, the criteria for seismic analysis and design of chimneys and stacks and other similar structures, such as flares, are presented.
Chimneys, stacks and flares are statically divided into two types: self-supported, and restrained. Chimneys, stacks and flares whose weight of the shell and inner covering from the foundation is less than 20% of the total weight of the structure (the weight of the chimney or stack plus the weight of its supporting structure), are included in the group of non-structural components and must be designed based on the criteria stated in the Chapter 8.
Chimney, stack and flare must be designed for the second hazard level, Section 2.5.2.

11.1.1. Definitions

The inner lining of the chimney or stack: it is a protective layer against high temperatures, erosion and corrosion for the chimney or stack shell, which does not have a structural function.

Chimney or stack shell: It is said to be the structural part of the chimney or stack that supports all applied loads such as dead, live, earthquake and wind.

Chimney or stack: It is a duct with a circular or square section that leads combustion products to the outside environment. Chimneys or stacks are made of reinforced concrete, steel or other suitable materials and have two main parts: shell and inner lining.

Flare: It is a stack that is used in facilities and oil wells, refineries, petrochemicals, etc. to burn gases.

11.1.2. Symbols

A	The cross section of the chimney, stack or flare shell at the level of foundation (m^2)
C_d	Displacement magnification factor according to Table 7.2

C_T	The correction coefficient of the period which is proportional to the slenderness of the chimney, stack or flare according to Table 11.1
C_u	Seismic coefficient according to Eq. 11.2
C_v	The shear force modification factor which is proportional to the slenderness of the chimney or flare according to Table 11.1
D	Dead load
$D_{1,2,3}$	Allowable height of the structure for seismic design groups (m) according to Table 7.2
D_m	Bending moment distribution coefficient for the distance z from the top of the chimney, stack or flare according to Table 11.3
D_s	Spectrum conversion factor with 5% damping ratio
D_v	Shear force distribution coefficient for the distance z from the top of the chimney or flare according to Table 11.3
E_h	Seismic load
E_s	Elastic modulus of the shell material (N/m^2)
I	Importance factor of chimney, stack or flare
L	Distance from the top of the cone to the top level of the chimney or flare with an incomplete cone shape (m)
M_z	Bending moment at the distance z from the top of the chimney or stack
R_u	Response modification factor of the structure according to Table 7.2
S_1	Spectral acceleration parameter (g)
S_a	Spectral acceleration (g)
T_n	The main period of the chimney or flare
T	The effects of chimney or stack operation temperature
V_u	Base shear
V_z	Shear force at a distance z from the top of the chimney or stack
W_i	The concentrated weight of the i-th element in the chimney or stack
W_t	The total weight of the chimney, stack or flare including the weight of the structure and cover from the base level
a	Distance from the top of the cone to the top level of the chimney, stack or flare with an incomplete cone shape (m)
g	Acceleration of gravity (m/s^2)
h	Chimney, stack or flare height from the base level (m)

h_g Height of the center of mass of the chimney, stack or flare from the base level

k Shape correction factor for chimneys, stacks or flares with an incomplete cone shape according to Table 11.2

n: Number of elements with concentrated mass in chimney, stack or flare modelling

r_e Radius of gyration of the section of the chimney, stack or flare shell at the foundation level (m)

$y_{\max}$ Maximum lateral elastic displacement at the highest point of the chimney, stack or flare (m)

δ_i Lateral static displacement of the i-th element under the effect of the concentrated weight of the elements

η Structural damping ratio (in percent)

Γ_δ Coefficient for calculating the chimney, stack or flare period

Ω_0 Overstrength Factor

11.2 Modeling

The analytical model of chimney, stack and flare shall be accurate enough such that changes in the mass and stiffness of the shell and internal lining of the chimney as well as the footing conditions are included in the model. In modeling, the height of the chimney or stack shall be divided into at least 10 segments and at least in 3 elements along each half wave of the highest mode used in the modal analysis. Due to changes in mass, stiffness, or support conditions of the inner lining, more elements may be required. The stiffness characteristics of the chimney or stack structures, such as concrete, should be modeled based on the uncracked section.

Considering interaction of the structure and soil, in addition to the cases that are considered mandatory according to Chapter 5, if a chimney, stack or flare with a large height (height more than five times the maximum diameter of the base) is built on a soft soil (e.g., soil, type IV), consideration of soil-structure interaction is mandatory, due to significant increase in secondary effects.

If the internal lining of the chimney or stack relies on the shell at any point of the chimney or stack, the modeling shall include the shell and the internal lining and the effect of the interaction between the shell and the lining shall be included in the model. If the mass of the inner cover is small compared to the structural shell of the chimney, stack or flare, the effect of the inner cover can be modeled as concentrated masses at the lateral connection points with the chimney or stack shell or flare, or the inner cover can be modeled as a beam element, such that the relevant elements are properly connected to the vertical and lateral supports along the height of the chimney.

If the chimney, stack or flare has a lateral restraining system, as either a cable or a supporting frame, effects of the side restraining system should be considered in the modeling, analysis and design of the structure.

In chimney or concrete chimney-like structures, the P-Δ effect due to vertical loads and lateral seismic displacements shall be considered.

11.3. Analysis methods

In this chapter, three methods, including the equivalent lateral load analysis method (or the simplified method), the spectral analysis method, and the time-history analysis procedure, are presented for the seismic analysis of chimney, stack or flare. In the case of using another valid reference that allows static or dynamic inelastic methods, the design criteria of that reference must be used and the design must be approved by a competent independent expert group. The equivalent lateral load method can only be used for the initial estimation of the structural characteristics of chimneys, stacks, or flares whose mass and stiffness are almost uniformly distributed in height. Use of one of the dynamic analysis methods is mandatory.

In chimneys, stacks and flares with a circular cross-section, the horizontal component of the earthquake can be applied alone and in one direction, but in chimneys, stacks and flares with a non-circular cross-section, both horizontal components of the ground motion shall be applied concurrently. In the spectral analysis, the SRSS criterion shall be used to combine the maximum responses along two perpendicular horizontal axes.

It is not necessary to consider the effect of the vertical component of the earthquake in the analysis and design of chimneys, stacks and flares. If the vertical component is considered, the value of the earthquake coefficient of the vertical component can be considered as 2/3 of the seismic coefficient of the horizontal component.

In chimneys, stacks and flares with a height of more than 80 m located in areas with high and very high seismicity, it is recommended to consider the rotational component of the earthquake in the analysis. In this case, there is no need to consider the effect of the vertical component of the earthquake and the rotational component of the earthquake at the same time in the combination of loads.

11.3.1. Equivalent lateral load method (simplified method)

11.3.1.1. Base shear

The value of base shear V_u is calculated from Eq. 11.1:

$$V_u = C_v C_u W_t \qquad 11.1$$

where:

C_v : Shear force correction factor according to chimney, stack or flare slenderness according to Table 11.1

C_u : Seismic coefficient according to Eq. 11.2

W_t : The total weight of the chimney, stack or flare including the weight of the structure and lining from the base level (N)

$$C_u = \frac{S_a I}{R_u} \qquad 11.2$$

where:

S_a : Spectral acceleration (in g) according to Chapter 3

I : Importance factor of the chimney, stack or flare, which is obtained according to the type of structural function from Table 3.4, unless larger values are provided in the specific criteria provided by the manufacturer.

R_u : Structure response modification factor according to Table 7.2.

Seismic coefficient C_u shall not be less than the value provided by Eq. 7.5-a. In a chimney, stack or flare located in an area with $S_1 \geq 0.6$, the seismic coefficient shall not be less than the values provided byEq. 7.5-b.

The main period of the chimney or cylindrical flare on a rigidbase can be calculated from Eq. 11.3 or 11.4, with Eq. 11.3 being more accurate. If exists, the actual measured period of a structure on similar soil and foundation conditions is acceptable.

$$T_n = k.C_T \sqrt{\frac{W_t.h}{E_s.A.g}} \qquad 11.3$$

where:

h : Height of the chimney, stack or flare from the base level (m)

E_s : Modulus of elasticity of the shell material (N/m^2)

A : Cross section area of the chimney, stack or flare shell at the level of foundation (m^2)

g : Acceleration of gravity (m/s^2)

r_e : Radius of gyration of the chimney, stack or flare shell section at the level of the foundation (m)

C_T : Correction coefficient of the period according to the slenderness of the chimney, stack or flare according to Table 11.1

k : Shape correction factor for chimney, stack or flare with incomplete cone shape according to Table 11.2

a : Distance from the top of the cone to the top level of the chimney, stack or flare with an incomplete cone shape (m)

L : Distance from the top of the cone to the top level of the chimney, stack or flare with an incomplete cone shape (m)

Table 11.1: The coefficients C_T and C_v

h/r_e	5	10	15	20	25	30	35	40	45	Greater than 50
C_T	14.4	21.2	29.6	38.4	47.2	56.0	65.0	73.8	82.8	(h/r_e) 1.8
C_v	1.02	1.12	1.19	1.25	1.30	1.35	1.39	1.43	1.47	1.50

Table 11.2: The modifying factor k

a/L	0.4	0.6	0.8	1.0
k	0.72	0.84	0.95	1.0

$$T_n = \frac{2\pi}{\sqrt{g.\Gamma_\delta}} \qquad 11.4.a$$

$$\Gamma_\delta = \frac{\sum_{i=1}^{n} W_i.\delta_i}{\sum_{i=1}^{n} W_i.\delta_i^2} \qquad 11.4.b$$

where:

W_i: The concentrated weight of the i-th element, which is applied as a horizontal force

δ_i : Lateral static displacement of the i-th element under the effect of the concentrated weight of the elements

n : Number of elements with concentrated mass (the minimum number of the concentrated masses is 10)

g : Acceleration of gravity (m/s^2)

11.3.2.1 Shear force and bending moment distribution

Distribution of the shear force and bending moment in the height at the distance z from the top of the chimney, stack or flare is obtained from Eqs. 11.5 and 11.6, respectively.

$$V_z = C_v C_u W_t D_v \qquad 11.5$$

$$M_z = C_u W_t h_g D_m \qquad 11.6$$

where:

D_v : Shear force distribution coefficient for the distance z from the top of the chimney, stack or flare according to Table 11.3

D_m : Bending moment distribution coefficient for the distance z from the top of the chimney, stack or flare according to Table 11.3

h_g : The height of the center of mass of the chimney, stack or flare from the base level

Table 11.3: Values of D_m and D_v

Foundation Condition	D_m	D_v
Fixed	$0.4\left(\frac{z}{h}\right)^{0.5}+0.6\left(\frac{z}{h}\right)^{4}$	$1.1\left(\frac{z}{h}\right)^{0.5}+0.75\left(\frac{z}{h}\right)+0.9\left(\frac{z}{h}\right)^{4}\leq 1.0$
Pile Foundation	$0.5\left(\frac{z}{h}\right)^{0.5}+0.5\left(\frac{z}{h}\right)^{4}$	$0.66\left(\frac{z}{h}\right)^{0.5}-0.20\left(\frac{z}{h}\right)+0.54\left(\frac{z}{h}\right)^{4}$

11.3.2. Dynamic analysis

Dynamic analysis is performed by two methods of spectral analysis and time-history procedure, following the requirements of Chapter 4 and assuming a suitable damping ratio. The proper damping ratio value for concrete and steel shells are 5% and 2%, respectively.

There is no need to adjust the base shear of the spectral analysis to the base shear of the equivalent static method. To determine the spectrum with the damping ratio η (in percentage), the values of the 5% spectrum shall be multiplied by D_s according to Eq. 11.7.

$$D_s = \frac{-Ln\left(\frac{\eta}{100}\right)}{Ln(20)} \qquad 11.7$$

11.3.2.1. Spectral analysis

Spectral analysis is performed using the criteria of Chapters 3 and 4 of this Regulations.
In order to obtain the dynamic response of the chimney, stack or flare in each of the two main orthogonal directions of the plan, number of vibration modes should be considered in such a way that the sum of the effective masses of the modes is not less than 90% of the total mass of the structure.

11.3.2.2. Time-history analysis

In the case of using the time-history method, the criteria of Section 10.4 must be considered.

11.4. Design criteria

11.4.1. Overturning control

The minimum safety factor against overturning, without considering the load factors, is 1.5 for the chimney or stack shell alone and 2 for the entire chimney or stack considering the foundation.

11.4.2. Displacement control

The maximum lateral elastic displacement at the highest point of the chimney, stack or flare, (in meters), without applying the load factors, shall not be more than the value obtained from Eq. 11.8.

$$y_{\max} = 0.0033\,h \qquad 11.8$$

In calculating the relative displacement of the concrete chimney or stack, it is not necessary to consider the concrete section cracking and it is assumed that the base of the chimney or stack is rigid.
The minimum separation between the chimney, stack or flare with the cover equals the relative lateral displacement of the elements multiplied by C_d.

11.4.3. Load combinations

To design a chimney or flare using the resistance method, the loading combinations including the effect of seismic load, as Eq. 11.9, shall be used:

$$1.2D + 1.2T + E_h \qquad 11.9.a$$

$$0.9D + 1.2T + E_h \qquad 11.9.b$$

To design a chimney, stack or flare using the allowable stress method, the loading combinations including the effect of seismic load, as Eq. 11.10 shall be used. The minimum safety factor in this method for the seismic load combinations is 1.5.

$$D + 0.9T + 0.75\,(0.7E_h) \qquad 11.10.a$$

$$D + 0.7E_h \qquad 11.10.b$$

$$0.6D + 0.9T + 0.7E_h \quad 11.10.c$$

where:

T: the effects of operating temperature of the chimney, stack or flare

D: Dead load

E_h: Seismic load

11.4.4. Detailing and design considerations

In general, dimensions of the chimney, stack or flare are determined based on the operating conditions, environmental issues, location of the nearby structures and the like, and are designed using the codes and relevant standards. In any case, it is necessary to comply with this section as minimum design requirements.

11.4.4.1. Concrete chimney or flare design considerations

Concrete chimneys and flares shall be designed according to the regulations of ACI 307, except that the base shear and the relevant coefficients are determined from Section 11.3.1. Moreover, at openings, the following criteria shall be considered.

More than 50% of the reinforcements shall not be cut in any section. In addition, if the cross-sectional area of the opening is more than 10% of the total cross-sectional area, the sections at the opening shall be designed for vertical, shear and bending forces at the section using the effective section properties and considering the overstrength factor to be equal to 1.5. The range where the overstrength factor is applied is within the opening height and a distance equal to half the width of the opening beyond the opening location. The longitudinal reinforcements shall be anchored in tension beyond the mentioned zone.

The reinforcement detailing requirementsaround the openings are similar to those of the columns, according to the 9th Clause of the National Building Regulations. The mentioned details shall be observed in the transverse direction at least twice the thickness of the shell and in the longitudinal direction at least twice the thickness of the shell at the top and bottom of the opening, provided that it is not less than the length of the additional longitudinal reinforcements.

If there is an opening near the footing in such a way that it is not possible to observe the above details in the lower part of the opening, the reinforcements of the areas on the sides of the opening shall continue inside the footing. Percentage of the longitudinal reinforcements in this range is determined according to the 9^{th} Clause of the National Building Regulations for compression members.

11.4.4.2. Steel stack and flare design considerations

A self-supported steel stack or flare with a height of more than 40 meters needs to increase the diameter at the base level in order to maintain stability. This part is called leg, whose minimum height is one third of the total height of the stack or flare. In a stack or flare without an insulating layer, the outer diameter of the stack or flare at the highest level shall be at least 1/20 of the height of its cylindrical part. In the case of a stack or flare with an insulating layer, the outer diameter of the stack or flare shall be at least 1/25 of the height of the cylindrical part of the stack or flare. The minimum outer diameter of the stack or flare at the base level shall be 1.6 times the outer diameter of the chimney or flare at its highest level.

The thickness of the shell shall be determined based on the actions and lateral displacements of the stack or flare. It is necessary to add the permitted corrosion values according to the relevant standard to the calculated thickness of the shell. Thickness of the shell with the addition of permitted corrosion shall not be less than 6 mm and 1/500 of the outer diameter of the shell at the desired point.

Chapter 12
Tank

12.1. General

This chapter provides minimum requirements for seismic design of tanks. Design earthquake is as defined in Chapter 3. Requirements of this Chapter shall be applied sticking to the general requirements given here.
This section gives statements on the hypotheses, expectations, significant factors and essentials of the seismic analysis and design of tanks prior to dealing with the requirements next.

12.1.1. Definitions

P-Delta effect
The second order effects in structural forces which develop when vertical axial forces act simultaneously with the lateral displacement. This effect might be significant in elevated tanks.

Backing

A piece which is placed behind the weld joint to let the welding proceed effectively and assist in weld quality assurance.

Corrosion allowance

Thickness added to the calculated required thickness of any part of the tank to compensate for expected life-long corrosion.

Annular plate

A ring-shaped strap of plate which constitutes a part of the tank bottom, welded to and supporting the shell plate. Thickness of the annular plate might be different from the rest of the tank bottom.

Allowable stress design

A structural design method where elements are proportioned such that elastic stresses due to nominal loads do not exceed allowable stresses.

Strength design

A structural design method where elements are proportioned such that element internal forces due to factored loads do not exceed its design strength.

Base metal

Metal or alloy which is to be welded or cut.

Occupancy

Occupancy of a structure or facility specifies the purpose which it has been built or is used for.

Purchaser

The work owner or his authorized representative, which might be a person or an institution.

Ring-wall

A strip foundation usually made of reinforced concrete where the annular plate rests on and the mechanical anchors are placed into.

Mechanically-anchored tank

A tank which is anchored to the foundation by means of anchor bolts, anchor straps or any other mechanical device.

Self-anchored tank

A tank which is stable against overturning moment without the need to mechanical anchors to resist uplift.

Open-top tank

A tank not equipped with a roof, whether fixed or floating roof, gas-holder cover or dome.

Authority having jurisdiction

An organization, institution or individual who is in charge of the implementation or supervising thereto of the provisions of this standard or other standards to the extent specified herein.

Design specific gravity

The maximum value of the specific gravity of the tank liquid content at the design temperature. The larger value applies where the tank is designed for storage of multiple fluids.

12.1.2. Symbols

Symbols in this chapter together with their units are defined as follows:

A : Design basis acceleration (as ratio of g)

A_c : Convective design spectrum acceleration parameter (as ratio of g)

A_i : Impulsive design spectrum acceleration parameter (as ratio of g)

A_v : Vertical component acceleration parameter = $2/3 \times 0.7 \times S_{DS} = 0.47 S_{DS}$, (as ratio of g)

$C.A.$: Corrosion allowance (mm)

C_d : Lateral displacement amplification factor, 2.0 for self-anchored and 2.5 for mechanically anchored tanks

C_i : Dimensionless coefficient for calculation of impulsive period of vibration of tank system

D : Nominal tank diameter (m)

d_i : Height (or bridth) of the i-th course of tank shell, ordered bottom up (m)

E : Elastic modulus of tank material (MPa)

F : Seismic design equivalent lateral load

F_c : Allowable longitudinal shell membrane compression stress (MPa)

F_{ty} : Minimum specified yield stress of a tank shell course (MPa)

F_y : Minimum specified yield stress of annular plate, of an anchor bolt, or of a bracing member (MPa)

g : Acceleration due to gravity (m/s^2)

G : Design specific gravity (dimensionless)

G_e : Effective specific gravity including vertical seismic effects = $G(1-0.4A_v)$

H : Maximum design product level (m)

h_c : Height from the bottom of the tank shell to the centre of action of lateral seismic force related to the convective liquid force for ring-wall moment (m)

h_{cs} : Height from the bottom of the tank shell to the centre of action of lateral seismic force related to the convective liquid force for the slab moment (m)

h_i : Height from the bottom of the tank shell to the centre of action of the lateral seismic force related to the impulsive liquid force for ring-wall moment (m)

h_{is}	:	Height from the bottom of the tank shell to the centre of action of the lateral seismic force related to the impulsive liquid force for the slab moment (m)
h_r	:	Height from the bottom of the tank shell to the roof and roof appurtenances centre of gravity (m)
h_s	:	Height from the bottom of the tank shell to the shell's centre of gravity (m)
I	:	Importance factor coefficient set by category and occupancy of the tank
J	:	Anchorage ratio
K	:	Coefficient to adjust the spectral acceleration from 5% to 0.5% damping = 1.5 unless otherwise specified. If 0.5%-damped response spectrum is used, then take it 1.0.
L	:	Required minimum width of thickened bottom annular ring measured from the inside of the shell (m)
L_s	:	Selected width of annulus (bottom or thickened annular ring) to provide the resisting force for self-anchorage, measured from the inside of the shell (m)
M_{rw}	:	Ring-wall moment, portion of the total overturning moment that acts at the base of the tank shell perimeter (N-m)
M_s	:	Slab moment, used for slab and pile cap design (N-m)
n	:	Number of courses over the height of the wetted tank shell
N_c	:	Convective hoop membrane force in tank shell (N/mm)
N_h	:	Product hydrostatic membrane force (N/mm)
N_i	:	Impulsive hoop membrane force in tank shell (N/mm)
N_s	:	Total hoop membrane force in tank shell (N/mm)
P_f	:	Overturning bearing force based on the maximum longitudinal shell compression at the base of shell, used in design of the ring-wall foundation (N/m)
Q	:	Scaling factor used to adjust MCE (Third seismic hazard level, Chapter 3) spectral accelerations to the design level = $^2/_3$.
R_v	:	Force reduction factor for vertical response using allowable stress design methods
R_{wc}	:	Force reduction factor for the convective mode using allowable stress design methods
R_{wi}	:	Force reduction factor for the impulsive mode using allowable stress design methods
S_1	:	5% damped, spectral response acceleration parameter at a period of one second for MCE (Third seismic hazard level, Chapter 3) (as ratio of g)

S_a : 5% damped, design spectral response acceleration parameter at any period (as ratio of g)

S_{av} : 5% damped, design spectral response acceleration parameter for vertical component (as ratio of g)

S_{D1} : 5% damped, design spectral response acceleration parameter at one second $= Q\,F_v\,S_1$, (as ratio of g)

S_{DS} : 5% damped, design spectral response acceleration parameter at short periods (T = 0.2 seconds) $= Q\,F_a\,S_S$, (as ratio of g)

S_S : 5% damped, spectral response acceleration parameter at short periods (T = 0.2 seconds) for MCE, (as ratio of g)

t : Thickness of the shell course under consideration for hoop stress calculation, less corrosion allowance (mm)

t_a : Thickness, excluding corrosion allowance, of the bottom annulus under the shell required to provide the resisting force for self-anchorage. The bottom plate of this thickness shall extend radially at least the distance L from the inside of the shell. This term applies for self-anchored tanks only. (mm)

t_b : Thickness of tank bottom plate (mm)

T_c : Natural period of the convective (sloshing) mode of vibration of the liquid (seconds)

T_i : Natural period of the impulsive mode of vibration (seconds)

t_i : Thickness of the *i*-th shell course, ordered bottom up (mm)

T_L : Transition period for longer period ground motion (seconds). Refer to Chapter 3.

t_s : Thickness of the bottom shell course less corrosion allowance (mm)

t_u : Equivalent uniform thickness of tank shell, calculated over the wetted height (mm)

T_v : Natural period of vibration for vertical response (seconds)

V : Total design base shear (N)

V_c : Design base shear due to the convective component of the effective sloshing weight (N)

V_i : Design base shear due to the impulsive component of the effective weight of tank and contents (N)

V_{max} : Maximum local shear at bottom-shell joint, per unit circumferential length (N/m)

w_a : Force resisting uplift in annular region, per unit circumferential length (N/m)

W_c : Effective weight of the convective (sloshing) portion of the liquid (N)

W_{eff} : Effective weight contributing to seismic response (N)

W_f : Weight of the tank bottom (N)

W_{fd} : Total weight of tank foundation (N)

W_g : Weight of soil directly over tank foundation (N)

W_i : Effective weight of the impulsive portion of the liquid (N)

w_{int} : Calculated design uplift load due to design pressure per unit circumferential length (N/m)

W_p : Total weight of the tank contents based on the design specific gravity of the product (N)

W_{pu} : In an uplifting self-anchored tank, weight of a portion of the contents resting on an area of the tank bottom which is still in contact with the ground (a circle of radius r), (N)

W_r : Total weight of fixed tank roof including framing, any permanent attachments and 10% of the roof balanced design snow load (N)

W_{rs} : Roof load acting on the tank shell including 10% of the roof balanced design snow load (N)

w_{rs} : Roof load acting on the shell, including 10% of the roof balanced design snow load, per circumferential length (N/m)

W_s : Total weight of tank shell and appurtenances (N)

W_{su} : In an uplifting self-anchored tank, compressive reaction at the base of the tank shell (N)

W_T : Total weight of tank including shell, roof, framing, bottom, attachments, appurtenances, product, and 10% of the roof balanced snow load (N)

w_t : Tank body and roof weight acting at base of shell, per circumferential length (N/m)

Y : Distance from liquid surface to analysis point (positive down), (m)

Y_i : Distance from centre of the i-th shell course to the free liquid surface, ordered bottom up (m)

y_u : Estimated uplift displacement for self-anchored tank (mm)

Ω_0 : Overstrength factor

γ_L : Unit weight of the liquid (N/m^3)

μ_f : Friction coefficient between the tank bottom and the foundation

ρ : Density of liquid (kg/m^3)

σ_c : Maximum longitudinal shell compression stress (MPa)

σ_h : Product hydrostatic hoop stress in the shell (MPa)

σ_s : Hoop stress in the shell due to impulsive and convective forces of the stored liquid (MPa)

σ_T : Total combined, static and dynamic, hoop stress in the shell (MPa)

12.1.3. Scope

Liquid storage tanks such as steel aboveground cylindrical tanks with a fixed or floating roof, aboveground concrete tanks, steel or reinforced concrete elevated tanks, towers, columns, pressure vessels and spheres, shall be design to resist earthquake according to the provisions of this chapter or the referenced codes. Design of tank ring-wall or granular berm foundations and the secondary containment for spill control shall be based on the requirements specified herein.
Bolted steel tanks for storage of water or petroleum products and tanks for non-fluid storage, e.g. granular material, is beyond the scope of this code.

12.1.4. Performance goals

The general performance objective in the seismic design of tanks according to the provisions of this chapter is to prevent the global collapse of the tank and fatalities due to it. Application of these provisions does not mean that no damage is expected to the tank and its appurtenances in an earthquake event. Further performance objectives are defined for low-temperature fluid storage tanks in section 12.9.7.2.

12.1.5. Significant factors

12.1.5.1. Shape

The effect of tank shape, whether it be circular, rectangular, cylindrical, spherical, etc. shall be considered in the seismic analysis and design. Provisions of this chapter are specified for different tank shapes where required.

12.1.5.2. Wall inclination

Provisions of this chapter are prepared while bearing vertical tank walls in mind. For non-vertical walls or axis of revolution, responsibility lies with the design engineer to estimate any possible effects.

12.1.5.3. Material

Effect of the tank body material shall be considered in the seismic analysis and design.
This chapter deals with the seismic design requirements of steel, reinforced or pre-stressed concrete, composite, corrugated steel, RTP and FRP tanks.

12.1.5.4. Bottom elevation

12.1.5.4.1. Ground-supported tanks

Provisions for the seismic design of ground-supported steel tanks are given in section 12.2. Provisions for concrete tanks are presented in section 12.3.

12.1.5.4.2. Elevated tanks

Seismic design requirements of elevated tanks are given in section 12.4. Requirements for pressure vessels, boilers and spheres are addressed in section 12.5.

12.1.5.4.3. Buried tanks

Requirements specific to buried tanks are given in section 12.9.3.

12.1.5.5. Anchorage

Tank bottom anchorage provisions are given in sections 12.1.11.3, 12.2.5.1 and 12.9.1.

12.1.5.6. Fixed/floating roof

With the exception of floating roof elements, the rest of provisions of this chapter apply to tanks with fixed or floating roof and to open-top tanks. This code does not provide requirements for the seismic design of the floating roof, yet the design engineer is required to calculate the sloshing effects due to seismic excitation in a sound and reliable method which is capable of sufficiently accurate estimation of the seismic response of the floating roof.

12.1.5.7. Liquid level

Effect of operational conditions, such as the tank being packed with liquid or partially-filled, shall be considered in the analysis and design of the tank shell and roof. For fixed-roof tanks, it is the intention of this chapter to specify a minimum freeboard to avoid sloshing waves hitting the roof due to seismic excitation. Otherwise it is required for the effects of sloshing on the roof to be estimated by the design engineer.

12.1.6. Tank categories and importance

The purchaser shall specify category and importance factor of the tank within the order, however it shall not be less than the values specified herein. Otherwise the tank is considered "ordinary". Importance factor is given in Table 12.1 where the tank categories are defined as below:

Critical tanks. Are those tanks which are needed to continue operation immediately after the earthquake, e.g. firewater tanks and hazardous tanks used for production, processing, management, storage, use or elimination of fuels, chemicals, hazardous waste or explosives of large inventory that their release exposes the general public to great danger or causes a critical condition.

Significant tanks. Are those tanks that serve facilities which their failure causes considerable loss of human beings, and tanks used for production, processing, management, storage, use or elimination of fuels, chemicals, hazardous waste or explosives of sufficient inventory such that their release exposes the facility personnel to danger.

Ordinary tanks. Are those tanks not included in the definition of Critical or Significant categories.
For tanks that are used for multiple purposes, the classification corresponding to the higher importance factor shall be taken into account.

Table 12.1 Seismic importance factor for tanks

Tank category	*Importance factor, I*
Ordinary	1
Significant	1.25
Critical	1.5

12.1.7. Damping

Damping ratio shall be taken 0.5 percent for the convective component.
For aboveground cylindrical steel storage tanks, damping ratio shall be taken 5 percent and 0.5 percent for the impulsive and convective modes respectively.

12.1.8. Method of analysis

Seismic forces shall be estimated with the methods specified in this chapter. Where a method has not been specified, methods specified in referenced documents shall be used. Otherwise, it is responsibility of the design engineer to apply a substantiated method according to reliable references of seismic design.

12.1.8.1. Modal combination

Impulsive and convective components shall be combined by the direct sum or the square root of the sum of the squares (SRSS) method where the modal periods are separated.

12.1.8.2. Vertical component

Vertical earthquake forces shall be considered in accordance with the applicable reference document. For tanks and vessels not covered by a reference document, the forces caused by the vertical acceleration shall be defined as follows:

1. Hydrodynamic vertical and lateral forces in non-cylindrical tank walls: The increase in hydrostatic pressures caused by the vertical excitation of the contained liquid shall correspond to an effective increase in unit weight, γ_L, of the stored liquid equal to 0.4 S_{av} γ_L, where S_{av} is taken as the peak of the vertical response spectrum corresponding to the natural period of vibration in vertical direction, as defined in Chapter 3.
2. Hydrodynamic hoop forces in cylindrical tank walls: as defined in section 12.2.4.7.
3. Vertical inertia forces in cylindrical and rectangular tank walls: Vertical inertia forces associated with the vertical acceleration of the structure itself shall be taken as equal to 0.4 S_{av} W, where S_{av} is taken as the peak of the vertical response spectrum defined in Chapter 3.

12.1.8.3. Lateral load distribution

Design of aboveground steel cylindrical tanks in this chapter is based on the allowable stress design (ASD) method and the equivalent lateral force analysis that applies equivalent static lateral forces to a linear mathematical model of the tank based on rigid walls, fixed bases and linear behaviour. Load combinations are given in section 12.1.11.1. This procedure is based on spectral analysis and considers two response modes of the tank and its contents—impulsive and convective. Dynamic analysis is not required nor included within the scope of this Annex. The lateral forces and the overturning moment applied to the shell as a result of the response of the masses to lateral ground motion are determined. Provisions are included to assure stability of the tank shell with respect to overturning and to resist buckling of the tank shell as a result of longitudinal compression.

Dynamic analysis methods incorporating fluid-structure and soil-structure interaction are permitted to be used in lieu of the procedures contained in this Chapter with Purchaser approval and provided the design and construction details are as safe as otherwise provided herein.

Ground-supported, flat-bottom tanks storing liquids shall be designed to resist the seismic forces as follows:

a. For tanks or vessels storing liquids with a diameter or width less than or equal to 1.5 m, the base shear and overturning moment shall be calculated as if the tank and the entire contents act as an impulsive mass. The convective mass shall be set equal to zero. The lateral force distribution shall be according to the reference design standard of the tank.
b. Tanks or vessels storing liquids with a diameter or width greater than 1.5 m shall be designed to consider the hydrodynamic pressures of the liquid in determining the equivalent lateral forces and the lateral force distribution.

It is permissible to use the procedure provided in ACI 350.3-06 for distribution of hydrodynamic and horizontal or vertical inertia forces in the shell of cylindrical or rectangular tanks.

12.1.9. Site-specific seismic study

Where the tank is built on soil of the following conditions, with the exceptions specified, it is required to perform a site-specific seismic study:

- Soil profile that includes soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. For structures that have fundamental periods of vibration equal to, or less than, 0.5 s, site response analysis is not required for liquefiable soils; it is permitted to use the specified design spectrum in such case.
- Soil profile includes peats and/or highly organic clays thickness of soil is more than 3 m.
- Soil profile includes very high plasticity clays with PI > 75 thicker than 7.6 m. Site response analysis is not required for this clay category for Significant and Ordinary tanks in areas where $S_{DS}<0.33$ and $S_{D1}<0.133$ and Critical tanks in areas where $S_{DS}<0.167$ and $S_{D1}<0.067$.
- Soil profile includes soft/medium stiff clays of thickness more than 37 m with s_u < 50 kPa. Site response analysis is not required for this clay category for Significant and Ordinary tanks in areas where $S_{DS}<0.33$ and $S_{D1}<0.133$ and Critical tanks in areas where $S_{DS}<0.167$ and $S_{D1}<0.067$.

A site-specific response spectrum is required for low-pressure storage tanks —as defined in section 12.9.7 of this chapter— located in regions where peak ground acceleration is greater than 0.15g or S_s is greater than 0.3g.

It is recommended to use site specific ground motions where any of the following conditions apply:

- The tank is located within 10 km of a known active fault.
- The structure is designed using base isolation or energy dissipation systems.
- The performance requirements defined by the owner exceed the objectives defined in this Chapter.

The ordinates of the site-specific response spectrum shall in no case be less than 80 % of the ordinates of the design response spectrum defined in this code.

When site-specific design methods are specified, the impulsive and convective seismic parameters shall be defined as follows:

Impulsive spectral acceleration parameter:

$$A_i = 2.5QS_{a0}\left(\frac{I}{R_{wi}}\right) \qquad 12.1$$

Alternatively, using either the impulsive period of the tank system, or assuming the impulsive period = 0.2 sec, it may be determined by this relationship:

$$A_i = QS_a\left(\frac{I}{R_{wi}}\right) \tag{12.2}$$

For flat-bottom tanks with H/D ≤ 0.8, where it is either self-anchored, or a mechanically-anchored tank equipped with traditional anchor bolt and chairs at least 450 mm high that are not otherwise prevented from sliding laterally at least 25 mm, S_a need not exceed 150 %g. This limitation on the upper value of S_a does not apply to low-pressure storage tanks defined in section 12.9.7. Convective spectral acceleration parameter:

$$A_c = KQS_a\left(\frac{I}{R_{wc}}\right) < A_i \tag{12.3}$$

12.1.10. Soil-structure interaction

Taking advantage of the soil-structure interaction, wherever deemed allowable in this chapter, shall comply with the provisions of Chapter 6.

12.1.11. Design procedure

Design of structural elements which contribute to the seismic load-carrying system shall keep to the requirements of this section.

12.1.11.1. Design load combinations

When tanks are designed by the Allowable Stress Design (ASD) method, the basic load combinations for earthquake load E, dead load D, and live load L are:

$$D + L \tag{12.4}$$

$$D + L + E \tag{12.5}$$

In allowable stress design of tanks and their foundations, when earthquake load in combined with other loads, a 33 percent increase in the allowable stress is permitted unless otherwise stipulated.

12.1.11.2. Connections

Connection of seismic load-bearing system, other than the anchor bolts embedded in concrete, shall be designed to meet the requirements of Section 12.2.6.4.

12.1.11.3. Anchor bolts

Anchorage length of the anchor bolts embedded in concrete shall be determined to meet the following requirements:

1. It is permitted to design ground-supported tanks without anchorage provided that they meet the requirements for self-anchored tanks. Tanks supported by structural towers or building structures above the grade, shall be anchored to the supporting structure.
2. For the anchorage of steel tanks and vessels with a diameter or width greater than 1.5 m or a height greater than 3.0 m, if the tank is of category Ordinary or Significant in areas where $S_{DS}<0.33$ and $S_{D1}<0.133$ or category Critical in areas where $S_{DS}<0.167$ and $S_{D1}<0.067$, anchorage shall be designed in accordance with Chapter 17 of ACI 318.
3. Tanks and vessels with a diameter or width greater than 1.5 m or a height greater than 3.0 m, except for the tanks of category Ordinary or Significant in areas where $S_{DS}<0.33$ and $S_{D1}<0.133$ or category Critical in areas where $S_{DS}<0.167$ and $S_{D1}<0.067$, shall meet all of the following requirements:
   a. In anchorage design, the anchor embedment into the concrete shall be designed to develop the steel strength of the anchor in tension in accordance with ACI 318 Section, 17.10.5.3(a), Equation 17.6.1.2, or shall be designed using anchor reinforcement in accordance to develop the steel strength of the anchor in tension per ACI 318, Equation 17.6.1.2.
   b. The minimum gauge length of anchor rod, which is the length over which its elongation can occur, shall be at least eight times the rod diameter.
   c. Post-installed anchors are permitted to be used, provided the anchor embedment into the concrete is sufficient to develop the steel strength of the anchor rod in tension.
   d. Where the special detailing requirements of this section apply, the load combinations that include overstrength do not apply.

For steel tanks and vessels with a diameter or width less than or equal to 1.5 m and a height less than or equal to 3.0 m, anchorage design shall be in accordance with ACI 318.

In addition, the connection of the anchors to the tank or vessel shall be designed to develop the lesser of the strength of the anchor in tension as determined by API 650 or AWWA D100 or forces calculated using the seismic load effects including overstrength in accordance with the requirements of Chapters 2 and 7. The overstrength requirements of Chapter 2 and the overstrength factor values given in Chapter 7 do not apply to the design of walls, including interior walls, of tanks or vessels.

12.1.11.4. Attached Piping and other Equipment

Flexibility of attached piping shall be in accordance to Section 12.2.6.3.

Equipment, piping and walkways or other appurtenances attached to the tank shall be designed to accommodate the displacements imposed by seismic forces.

12.1.11.5. Freeboard

Freeboard requirements shall be in accordance with Section 12.2.6.2. For rectangular tanks, D in the equation given in Section 12.2.6.2 shall be replaced by the longer plan dimension of the tank.

No minimum freeboard is required for open-top tanks where the following conditions are met:

1. Contained fluid is not toxic or explosive and has been approved by the Authority Having Jurisdiction as acceptable for product spill
2. A site-specific product spill prevention, control, and countermeasure plan (SPCC) has been developed and approved by the Authority Having Jurisdiction to properly handle resulting spill. The SPCC shall account for proper site drainage, infiltration, foundation scour, and protection of adjacent facilities from sloshing spill.

12.1.11.6. Sliding resistance

Requirements for sliding resistance of the tank shall be in accordance with Section 12.2.6.6.

12.1.11.7. Local shear transfer

Requirements for local shear transfer shall be in accordance with Section 12.2.6.7.

12.1.11.8. Repair, modification, renovation

Repairs, modifications, or reconstruction (i.e., cut down and re-erect) of a tank or vessel shall conform to industry standard practice in addition to this standard. Tanks that are relocated shall be re-evaluated for the seismic loads at the new site.

12.2. Above-ground steel cylindrical tanks

12.2.1. Period of vibration

The analytical method presented here is based on the natural period of vibration of the structure and its contents.

12.2.1.1. Impulsive mode

The impulsive period of the system of tank and the impulsive liquid is estimated from Eq. 12.6:

$$T_i = \frac{1}{\sqrt{2000}}\left(\frac{C_i H}{\sqrt{\frac{t_u}{D}}}\right)\left(\frac{\sqrt{\rho}}{\sqrt{E}}\right) \qquad 12.6$$

where C_i is a dimensionless coefficient measured from Fig. 12.1. It can alternatively be calculated from Table 12.2. Linear interpolation may be used for H/D values not tabulated.

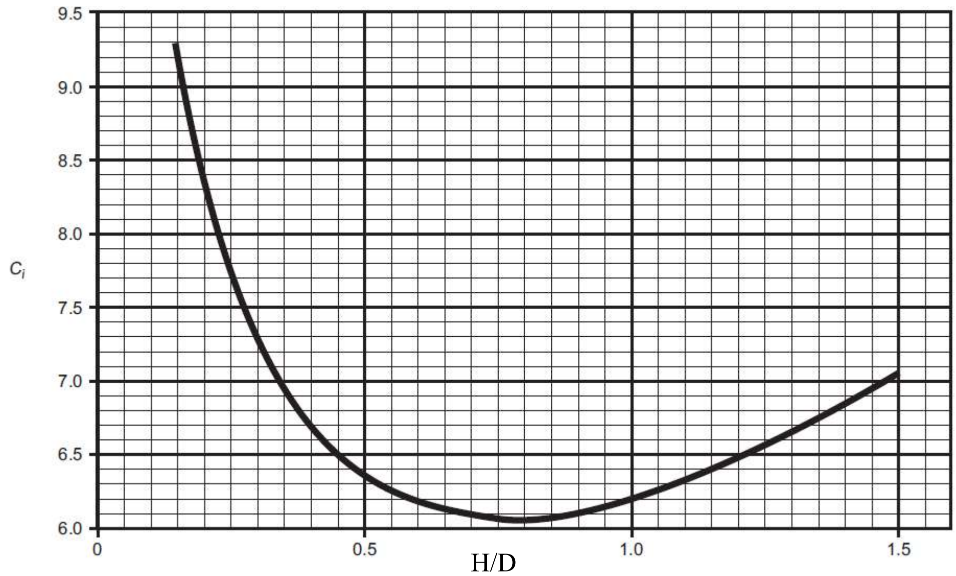


Fig. 12.1: Coefficient C_i* versus *H/D

Equivalent uniform thickness of the tank shell is calculated depending on the material, thickness and longitudinal stress within the tank shell. For uniform shell material, Eq. 12.7 is used to calculate equivalent uniform thickness.

$$t_u = \frac{\sum_{i=1}^{n} t_i d_i Y_i}{\sum_{i=1}^{n} d_i Y_i} \qquad 12.7$$

12.2.1.2. Convective mode

Convective mode period of the liquid is defined as Eq. 12.8:

$$T_c = 1.8\, K_s \sqrt{D} \qquad 12.8$$

K_s is called “sloshing coefficient” and is given in Eq. 12.9:

$$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68\, H}{D}\right)}} \qquad 12.9$$

Table 12.2 Coefficient C_i for calculation of the impulsive period

H/D	0.15	0.25	0.35	0.50	0.75	1.0	1.25	1.5
C_i	9.28	7.74	6.97	6.36	6.06	6.21	6.56	7.03

12.2.2. Spectral response acceleration

Design spectrum for flat-bottomed ground-supported tanks is defined using the following parameters:

12.2.2.1. Impulsive spectral acceleration parameter

Impulsive spectral acceleration parameter A_i is given in Eq. 12.10:

$$A_i = S_{DS}\left(\frac{I}{R_{wi}}\right) = 2.5QF_aS_0\left(\frac{I}{R_{wi}}\right) \quad 12.10$$

And it is required that $A_i \geq 0.007$,
and for $S_1 \geq 0.6$:

$$A_i \geq 0.5S_1\left(\frac{I}{R_{wi}}\right) = 0.625S_0\left(\frac{I}{R_{wi}}\right) \quad 12.11$$

Convective spectral acceleration parameter
Convective spectral acceleration parameter A_c is given in Eq. 12.12:

If $T_c \leq T_L$: $A_c = KS_{D1}\left(\frac{1}{T_c}\right)\left(\frac{I}{R_{wc}}\right) = 2.5KQF_aS_0\left(\frac{T_s}{T_c}\right)\left(\frac{I}{R_{wc}}\right) \leq A_i$ 12.12(a)

If $T_c > T_L$: $A_c = KS_{D1}\left(\frac{T_L}{T_c^{\,2}}\right)\left(\frac{I}{R_{wc}}\right) = 2.5KQF_aS_0\left(\frac{T_sT_L}{T_c^{\,2}}\right)\left(\frac{I}{R_{wc}}\right) \leq A_i$ 12.2(b)

12.2.3. Seismic design factors

12.2.3.1. Seismic coefficient

The equivalent static lateral seismic design force is calculated from the general relationship F = A · Weff where A is the lateral seismic acceleration coefficient and W_{eff} is the effective weight.

12.2.3.2. Response modification factor

When a steel cylindrical tank is designed using the Allowable Design Method (ASD), the response modification factor shall not exceed the values set forward in Table 12.3, depending on the tank anchorage.

Table 12.3 Response modification factor for aboveground cylindrical steel storage tanks

Tank category	R_{wi} (impulsive mode)	R_{wc} (convective mode)
Mechanically-anchored tank	4	2
Self-anchored tank	3.5	2

12.2.3.3. Importance factor

Importance factor is determined from Table 12.1 depending on the tank category and occupancy.

12.2.4. Design

12.2.4.1. Design loads

Ground-supported flat-bottomed tanks shall be designed considering the effective mass and dynamic liquid pressures in determining the equivalent lateral seismic forces and the lateral force distribution. The equivalent lateral load corresponding to the impulsive liquid and the tank roof, bottom and shell, is the product of the impulsive spectral acceleration parameter, A_i, and the weight of each component in accordance with Section 12.2.2.1. The convective lateral load is the product of convective spectral acceleration parameter, A_c, and the weight of the convective liquid as specified in Section 12.2.2.2.

Total design base shear, V, is determined from the square root of sum of the squares (SRSS) of the impulsive and the convective components as given in Eq. 12.13:

$$V = \sqrt{V_i^2 + V_c^2} \qquad 12.13$$

$$V_i = A_i (W_i + W_r + W_f + W_s) \qquad 12.14$$

$$V_c = A_c W_c \qquad 12.15$$

12.2.4.2. Effective weights

Weight of impulsive content, W_i, and convective content, W_c, are calculated as parts of total product weight, W_p as given in Eq. 12.16 and 12.17:

For $\frac{D}{H} \geq \frac{4}{3}$,

$$W_i = \frac{\tanh(0.866\frac{D}{H})}{0.866\frac{D}{H}} W_p \qquad 12.16(a)$$

and for $\frac{D}{H} < \frac{4}{3}$,

$$W_i = \left[1.0 - 0.218\frac{D}{H}\right] W_p \qquad 12.16(b)$$

$$W_c = 0.230\frac{D}{H}\tanh(\frac{3.67H}{D}) W_p \qquad 12.17$$

12.2.4.3. Centre of action for effective lateral loads

In order to determine overturning moments, centre of action of equivalent lateral loads due to the seismic response of the product is defined herein as height from the tank bottom.

For the impulsive mode, the centre of action of the lateral load is assumed to be at the centre of gravity of the shell, roof or any other part of the tank.

12.2.4.4. Centre of action for ring-wall overturning moment

The ring-wall overturning moment, M_{rw}, is the portion of the total overturning moment that acts at the base of the tank shell perimeter. This moment is used to determine loads on a ring-wall foundation, the tank anchorage forces, and to check the longitudinal shell compression.

The heights from the bottom of the tank shell to the centre of action of the lateral seismic forces applied to W_i and W_c, is determined from the Eq. 12.18 and 12.19.

For $\frac{D}{H} \geq \frac{4}{3}$,

$$h_i = 0.375H \qquad 12.18(a)$$

and for $\frac{D}{H} < \frac{4}{3}$,

$$h_i = \left(0.500 - 0.094\frac{D}{H}\right)H \qquad 12.18(b)$$

$$h_c = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \qquad 12.17$$

12.2.4.5. Centre of action for slab overturning moment

The slab overturning moment, M_s, is the total overturning moment acting across the entire tank base cross-section. This moment is used to design slab and pile cap foundations.

The heights from the bottom of the tank shell to the centre of action of the impulsive and convective lateral load for the calculation of slab overturning moment is determined from Eqs. 12.20 and 12.21.

For $\frac{D}{H} \geq \frac{4}{3}$, $$h_{is} = 0.375\left[1.o + 1.333\left(\frac{0.866\frac{D}{H}}{\tanh\left(0.866\frac{D}{H}\right)} - 1.0\right)\right]H \qquad 12.20(a)$$

and for $\frac{D}{H} < \frac{4}{3}$, $$h_{is} = \left(0.500 + 0.060\frac{D}{H}\right)H \qquad 12.20(b)$$

$$h_{cs} = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1.937}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \qquad 12.21$$

12.2.4.6. Vertical seismic component

Vertical acceleration effects shall be considered as acting in both upward and downward directions and combined with lateral acceleration effects by the SRSS method. Vertical acceleration effects for hydrodynamic hoop stresses shall be determined in accordance with Section 12.2.4.7. Vertical acceleration effects need not be combined concurrently for determining loads, forces, and resistance to overturning in the tank shell except as specified in this Chapter. The vertical seismic acceleration parameter, A_v, shall be taken as Eq. 12.22:

$$A_v = 0.47 S_{DS} \qquad 12.22$$

The total vertical seismic force shall be:

$$F_v = \pm A_v W_{eff} \qquad 12.23$$

Vertical seismic effects shall be considered in the following cases when specified:

a. Shell hoop tensile stresses (Section 12.2.4.7);
b. Shell-membrane compression (Section 12.2.5.2.3);
c. Anchorage design (Section 12.2.5.1.2);
d. Fixed roof components design;
e. Sliding resistance (Section 12.2.6.6);
f. Foundation design (Section 12.2.5.3).

12.2.4.7. Dynamic liquid hoop forces

Dynamic liquid hoop forces of impulsive and convective modes, N_i and N_c respectively, shall be determined as follows from Eqs. 12.24 and 12.25:

For $\frac{D}{H} \geq \frac{4}{3}$, $N_i = 8.48 A_i G D H \left[\frac{Y}{H} - 0.5 \left(\frac{Y}{H} \right)^2 \right] \tanh\left(0.866 \frac{D}{H} \right)$ 12.24(a)

For $\frac{D}{H} < \frac{4}{3}$ and $Y < 0.75D$, $N_i = 5.22 A_i G D^2 \left[\frac{Y}{0.75D} - 0.5 \left(\frac{Y}{0.75D} \right)^2 \right]$ 12.24(b)

For $\frac{D}{H} < \frac{4}{3}$ and $Y \geq 0.75D$, $N_i = 2.6 A_i G D^2$ 12.24(c)

For any $\frac{D}{H}$ value, $N_c = \dfrac{1.85 A_c G D^2 \cosh\left[\frac{3.68(H-Y)}{D} \right]}{\cosh\left(\frac{3.68 H}{D} \right)}$ 12.25

Total dynamic liquid hoop force per unit height of the shell shall be calculated from Eq. 12.26:

$$N_s = \sqrt{N_i^2 + N_c^2 + (A_v N_h/2.5)^2} \qquad 12.26$$

Dynamic hoop stress, $\sigma_s = N_s/t$, shall be combined with static hoop stress, $\sigma_h = N_h/t$, as in Eq. 12.27.

$$\sigma_T = \sigma_h \pm \sigma_s \qquad 12.27$$

N_h is the static hoop force per unit height of the shell which might be estimated from the "one-foot method" as $N_h = 0.5\rho g D(Y - 0.3)$ or other analytical methods.

12.2.4.8. Overturning moments

The seismic overturning moment at the base of the tank shell shall be the SRSS summation of the impulsive and convective components multiplied by the respective moment arms to the centre of action of the forces.

Ring-wall Moment, M_{rw}, shall be determined by Eq. 12.28:

$$M_{rw} = \sqrt{\left[A_i(W_i h_i + W_s h_s + W_r h_r)\right]^2 + \left[A_c W_c h_c\right]^2} \qquad 12.28$$

Slab Moment, M_s, shall be determined by Eq. 12.29:

$$M_s = \sqrt{\left[A_i(W_i h_{is} + W_s h_s + W_r h_r)\right]^2 + \left[A_c W_c h_{cs}\right]^2} \qquad 12.29$$

Unless a more rigorous method is used, the overturning moment at the bottom of each shell course shall be defined by linear approximation using the following:

1. If the tank is equipped with a fixed roof, the impulsive shear and overturning moment is applied at the top of the shell.
2. The impulsive shear and overturning moment for each shell course is included based on the weight and centroid of each course.
3. The overturning moment due to the liquid is approximated by a linear variation that is equal to the ring-wall moment, M_{rw} at the base of the shell to zero at the maximum liquid level.

12.2.4.9. Soil-structure interaction

If specified by the Purchaser, the effects of soil-structure interaction may be considered in accordance with Chapter 6. Tanks shall be equipped with a reinforced concrete ring-wall, mat or similar type foundation supported on grade; they shall be mechanically anchored to the foundation and the effective damping factor for the structure-foundation system shall not exceed 20 %.

Soil-structure interaction effects for tanks supported on granular berm or pile type foundation are outside the scope of this Code.

12.2.5. Resistance to design loads

The allowable stress design (ASD) method is utilized for steel storage tanks. Allowable stresses in structural elements may be increased by 33% when the effects of the design earthquake are included unless otherwise specified in this Chapter.

12.2.5.1. Anchorage

Resistance to the design overturning (ring-wall) moment at the base of the shell may be provided by:

- The weight of the tank shell, weight of roof reaction on shell, and by the weight of a portion of the tank contents adjacent to the shell for self-anchored tanks. Tanks are permitted to be designed without anchorage when they meet the requirements for self-anchored tanks listed in Section 12.2.5.1.1.
- Mechanical anchorage devices.

12.2.5.1.1. Self-anchored tanks

For self-anchored tanks, the restoring force against the overturning moment, M_{rw} according to Eq. 12.28, is provided by the weight of the tank shell, weight of roof reaction on shell, and by the weight of a portion of the tank contents adjacent to the shell. Weight of the portion of the contents over the unit length of the shell base perimeter shall be determined as:

$$w_a = 99t_a\sqrt{F_y H G_e} \leq 201.1 G_e H D \qquad 12.30(a)$$

$$G_e = G(1 - 0.4A_v) \qquad 12.30(b)$$

This relationship applies whether or not a thickened bottom annulus is used. If w_a exceeds the limit of 201.1HDG_e, the value of L shall be set to 0.035D and the value of w_a shall be set equal to 201.1HDG_e. A value of L defined as L_s that is less than that determined by the Eq. 12.33 may be used. If a reduced value L_s is used, a reduced value of w_a shall be used as determined below:

$$w_a = 5742\ HG_eL_s \tag{12.31}$$

The tank is self-anchored providing the following conditions are met:

1. The resisting force is adequate for tank stability (i.e. the anchorage ratio, J≤1.54
2. The maximum width of annulus for determining the resisting force does not exceed 3.5% of the tank diameter.
3. The shell compression satisfies requirements of Section 12.2.5.2.
4. The required annulus plate thickness does not exceed the thickness of the bottom shell course.
5. Piping flexibility requirements are satisfied.

12.2.5.1.1.1. Anchorage ratio

The anchorage ratio, J, shall be determined using the Eq. 12.32. The requirements for anchorage ratio are as specified in Table 12.4.

$$J = \frac{M_{rw}}{D^2\left[w_t(1-0.4A_v)+w_a-F_p w_{int}\right]} \tag{12.32(a)}$$

$$w_t = \left[\frac{W_s}{\pi D} + w_{rs}\right] \tag{12.32(b)}$$

w_a is the weight of the portion of the tank contents which is accounted for the self-anchored tanks only.

For self-anchored tanks where the tank bottom is uplifted, the requirements for piping flexibility and tank attachments shall be satisfied as specified in Sections 12.2.6.3 and 12.2.6.5.

Table 12.4: Anchorage ratio criteria

Anchorage ratio, J	***Criteria***
$J \leq 0.785$	No calculated uplift under the design seismic overturning moment. The tank is self-anchored.
$0.785 < J \leq 1.54$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and cannot be self-anchored for the design load. Modify the annular ring if L < 0.035D is not controlling or add mechanical anchorage.

12.2.5.1.1.2. Annular plate requirements

Fig. 12.2 depicts a schematic of the position and thickness of the shell plate, bottom plate and the annular plate. Provided that the following limitations are satisfied, thickness of the annular plate at the base of the shell may be taken equal to or greater than the rest of the tank bottom plate.

Notice: In thickening of the bottom annulus, the intent is not to force a thickening of the lowest shell course thereby inducing an abrupt thickness change in the shell, but rather to impose a limit on the bottom annulus thickness based on the shell design.

1. The thickness, t_a, corresponding to the final w_a shall not exceed the first shell course thickness, less the shell corrosion allowance.
2. Nor shall the thickness, t_a, exceed the actual thickness of the plate under the shell less the corrosion allowance for tank bottom.
3. When the bottom plate under the shell is thicker than the remainder of the tank bottom, the minimum projection L of the supplied thicker annular ring inside the tank wall, shall be the greater of 0.45 m or as determined in Eq. 12.33; however, L need not be greater than 0.035D.

$$L = 0.01723 t_a \sqrt{\frac{F_y}{H G_e}} \qquad 12.33$$

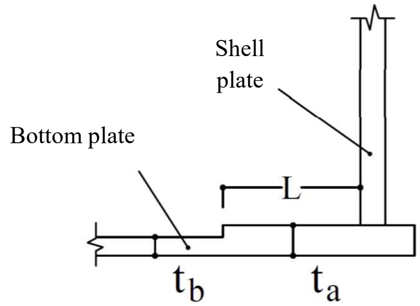


Fig. 12.2: Dimensions of the shell and bottom plates

12.2.5.1.1.3. Annular plate welding requirements

For critical tanks located where S_{DS} = 0.5g or greater, butt-welded annular plates shall be required. Annular plates exceeding 10 mm thickness shall be butt-welded. The weld of the shell to the bottom annular plate shall be checked for the design uplift load.

12.2.5.1.2. Mechanically-anchored tanks

The design of the anchorage and its attachment to the tank shall meet the requirements of API 650 and the amendments specified in IPS-G-ME-100. Hooked anchor bolts (L- or J-shaped embedded bolts) or other anchorage systems based solely on bond or mechanical friction shall not be used when anchors are required for seismic load. Post-installed anchors may be used provided that testing validates their ability to develop yield load in the anchor under cyclic loads in cracked concrete in accordance with the relevant standards.

End-plates may be used to provide the required strength of the anchor bolt or anchor straps.

12.2.5.2. Maximum longitudinal shell membrane compression stress

12.2.5.2.1. Self-anchored tanks

The maximum longitudinal shell compression stress at the bottom of the shell when there is no calculated uplift, J ≤ 0.785, shall be determined by Eq. 12.34(a):

$$\sigma_c = \left[w_t(1 + 0.4A_v) + \frac{4M_{rw}}{\pi D^2}\right]\frac{1}{1000\ s} \qquad 12.34(a)$$

When there is calculated uplift, J > 0.785, it shall be determined by Eq. 12.34(b):

$$\sigma_c = \left[\frac{w_t(1+0.4A_v)+w_a}{0.607-0.18667[J]^{2.3}} - w_a\right]\frac{1}{1000t_s} \qquad 12.34(b)$$

12.2.5.2.2. Mechanically-anchored tanks

The maximum longitudinal shell compression stress at the bottom of the shell for mechanically-anchored tanks shall be determined by Eq. 12.35:

$$\sigma_c = \left[w_t(1+0.4A_v) + \frac{4M_{rw}}{\pi D^2}\right]\frac{1}{1000t_s} \qquad 12.35$$

12.2.5.2.3. Allowable longitudinal shell membrane compression stress

The maximum longitudinal shell compression stress, as determined by Eqs. 12.34 or 12.35, shall be less than the seismic allowable stress, F_C, which is determined by Eq. 12.36 for Allowable Stress Design (Note: these relationships already include the 33% increase for ASD; such increase does not need to be applied separately). These formulas for F_C, consider the effect of internal pressure due to the liquid contents.

When $\frac{GHD^2}{t^2} \geq 44$, $F_c = \frac{83t_s}{D}$ 12.36(a)

When $\frac{GHD^2}{t^2} < 44$, $F_c = \frac{83t_s}{2.5D} + 7.5\sqrt{GH} \leq 0.5F_{ty}$ 12.36(b)

If the thickness of the bottom shell course calculated to resist the seismic overturning moment is greater than the thickness required for hydrostatic pressure, less corrosion allowance, then the calculated thickness of each upper shell course for hydrostatic pressure shall be increased in the same proportion (ratio of the thickness calculated for the overturning moment to the thickness required for the hydrostatic pressure), unless a special analysis is made to determine the seismic overturning moment and corresponding stresses at the bottom of each upper shell course.

12.2.5.3. Foundation

Foundations and footings for mechanically-anchored flat-bottom tanks shall be proportioned to resist peak anchor uplift and the maximum bearing pressure due to overturning. Product and soil load directly over the ring-wall and footing may be used to resist the maximum anchor uplift on the foundation, provided the ring-wall and footing are designed to carry this eccentric loading.

Product load shall not be used to reduce the anchor load.

When vertical seismic accelerations are applicable, the product load directly over the ring-wall and footing:

a. When used to resist the maximum anchor uplift on the foundation, the product pressure shall be multiplied by a factor of $(1 - 0.4A_V)$ and the foundation ring-wall and footing shall be designed to resist the eccentric loads with or without the vertical seismic accelerations.
b. When used to evaluate the bearing (downward) load, the product pressure over the ring-wall shall be multiplied by a factor of $(1 + 0.4A_V)$ and the foundation ring-wall and footing shall be designed to resist the eccentric loads with or without the vertical seismic accelerations.

The overturning stability ratio for mechanically-anchored tank system excluding the vertical seismic effects shall be as defined in Equation Eq. 12.37.

$$\frac{0.5D(W_T + W_{fd} + W_g)}{M_s} \geq 2.0 \qquad 12.37$$

Ring-walls for self-anchored flat-bottom tanks shall be proportioned to resist the maximum longitudinal shell compression force at the base of the shell, P_f, which includes the bearing pressure due to the seismic overturning moment, and the pressure due to gravity, w_t, per unit circumferential length as specified in Eq. 12.38.

$$P_f = \frac{4M_{rw}}{\pi D^2} + w_t(1 + 0.4A_v) \qquad 12.38$$

Requirements for allowable soil pressure shall be checked in accordance with Chapter 2.

12.2.5.4. Hoop stresses

The maximum tensile hoop stress due to earthquake combined with the hydrostatic stress according to Section 12.2.4.7, shall not exceed the allowable hoop tension stress which shall be the lesser of the basic allowable membrane stress for the shell plate material increased by 33%, or $0.9F_y$ times the joint efficiency, where F_y is the lesser of the published minimum yield strength of the shell material or the weld material.

12.2.6. Detailing requirements

12.2.6.1. Shell support

Self-anchored tanks resting on concrete ring-walls or slabs shall have a uniformly supported annulus under the shell. Uniform support shall be provided by one of the following methods:

a. Shimming and grouting the annulus.

b. Using fibre-board or other suitable padding.

c. Using double butt-welded bottom or annular plates resting directly on the foundation. Annular plates or bottom plates under the shell may utilize back-up bar welds if the foundation is notched to prevent the back-up bar from bearing on the foundation.

d. Using closely spaced shims (without structural grout) provided that the localized bearing loads are considered in the tank wall and foundation to prevent local crippling and spalling.

Mechanically-anchored tanks shall be shimmed and grouted.

In either case, the foundation must be supplied to the tolerances required in relevant reference standards in order to provide the required uniform support.

12.2.6.2. Freeboard

Sloshing of the liquid within the tank or vessel shall be considered in determining the freeboard required above the top capacity liquid level, based on the maximum wave height and specifications of the tank. A minimum freeboard shall be provided per Table 12.5. The height of the sloshing wave above the product design height, δ_s, can be estimated by Eq. 12.39:

$\delta_s = 0.42\, DA_f$ 12.39

For ordinary and significant tanks:

When $T_C \le 4$ sec , $A_f = KS_{D1}I\left(\frac{1}{T_C}\right) = 2.5KQF_aS_0I\left(\frac{T_S}{T_C}\right)$ 12.40(a)

When $T_C > 4$ sec , $A_f = KS_{D1}I\left(\frac{4}{T_C^2}\right) = 2.5KQF_aS_0I\left(\frac{4T_C}{T_C^2}\right)$ 12.40(b)

For critical tanks, it is estimated as below where importance factor is taken as unity:

For $T_C \le T_L$, $A_f = KS_{D1}\left(\frac{1}{T_C}\right) = 2.5KQF_aS_0\left(\frac{T_S}{T_C}\right)$ 12.40(c)

For $T_C > T_L$, $A_f = KS_{D1}\left(\frac{4}{T_C^2}\right) = 2.5KQF_aS_0\left(\frac{T_CT_L}{T_C^2}\right)$ 12.40(d)

Table 12.5: Minimum required freeboard

Value of S_{DS}	Ordinary tanks	Significant tanks	Critical tanks
$S_{DS} < 0.33g$	Note (a)	Note (a)	δ_s , Note (c)
$S_{DS} \ge 0.33g$	Note (a)	$0.7\delta_s$, Note (b)	δ_s , Note (c)

Notes:

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

b. A freeboard equal to $0.7\delta_s$ is required unless one of the following alternatives are provided:
   1. Secondary containment is provided to control the product spill.
   2. The roof and tank shell are designed to contain the sloshing liquid.

c. Freeboard equal to the calculated wave height, δ_s, is required unless one of the following alternatives are provided:
   1. Secondary containment is provided to control the product spill.
   2. The roof and tank shell are designed to contain the sloshing liquid.

12.2.6.3. Piping flexibility

Piping systems connected to tanks shall consider the potential movement of the connection points during earthquakes and provide sufficient flexibility to avoid release of the product by failure of the piping system. The piping system and supports shall be designed so as not to impart significant mechanical loading on the attachment to the tank shell. Local loads at piping connections shall be considered in the design of the tank shell. Mechanical devices which add flexibility such as bellows, expansion joints, and other flexible apparatus may be used when they are designed for seismic loads and displacements.

Unless otherwise calculated, piping systems shall provide for the minimum displacements in Table 12.6 at working stress levels (with the 33 % increase for seismic loads) in the piping, supports and tank connection. The piping system and tank connection shall also be designed to tolerate $1.4C_d$ times the working stress displacements given in Table 12.6 without rupture, although permanent deformations and inelastic behaviour in the piping supports and tank shell is permitted. C_d is 2.5 for mechanically-anchored tanks and 2.0 for self-anchored tanks.

For attachment points located above the support or foundation elevation, the displacements in Table 12.6 shall be increased to account for drift of the tank or vessel.

The values given in Table 12.6 do not include the influence of relative movements of the foundation and piping anchorage points due to foundation movements (such as settlement or seismic displacements). The effects of foundation movements shall be included in the design of the piping system.

When $S_{DS} < 0.1$, the values in Table 12.6 may be reduced to 70% of the values shown.

12.2.6.3.1. Tank uplift estimating method

The maximum uplift at the base of the tank shell for a self-anchored tank constructed to the criteria for annular plates may be approximated by Eq. 12.41:

$$y_u = \frac{12.1 F_y L^2}{t_b - C.A.} \qquad 12.41$$

t_b is thickness of the bottom plate.

C.A. = Corrosion Allowance.

12.2.6.4. Connections

Connections and attachments for other lateral force resisting components shall be designed to develop the strength of the component (i.e. minimum published yield strength, F_y in direct tension, or plastic bending moment), or 4 times the calculated element design load.

Table 12.6: Design Displacements for Piping Attachments

Tank condition	ASD design displacement (mm)
Mechanically-anchored tanks	
Upward vertical displacement relative to support or foundation	25
Downward vertical displacement relative to support or foundation	13
Range of horizontal displacement (radial and tangential) relative to support or foundation	13
Self-anchored tanks	
Upward vertical displacement relative to support or foundation:	
Anchorage ratio less than or equal to 0.785	25
Anchorage ratio greater than 0.785	100
Downward vertical displacement relative to support or foundation:	
For tanks with a ring-wall/mat foundation	13
For tanks with a berm foundation	25
Range of horizontal displacement (radial and tangential) relative to support or foundation	50

Penetrations, manholes, and openings in shell components shall be designed to maintain the strength and stability of the shell to carry tensile and compressive membrane shell forces. The bottom connection on a self-anchored flat-bottom tank shall be located inside the shell a sufficient distance to minimize damage by uplift. As a minimum, the distance measured to the edge of the connection reinforcement shall be the width of the calculated self-anchored bottom hold-down (L_{req} in accordance with Section 12.2.5.1.1.2) plus 300 mm.

12.2.6.5. Internal components

The attachments of internal equipment and accessories which are attached to the shell or bottom of the primary liquid- or pressure-retaining tank or vessel, or provide structural support for major components shall be designed for the

lateral loads due to the sloshing liquid in addition to the inertial forces. To estimate the sloshing liquid effects, simplified pressure distribution may be used from relevant references.
Seismic design of roof framing and columns shall be made if specified by the Purchaser. The Purchaser shall specify live loads and amount of vertical acceleration to be used in seismic design of the roof members. Columns shall be designed for lateral liquid inertial loads and acceleration as specified by the Purchaser. Design of such beams and columns shall be based on the requirements set forth in AISC 360.
Internal columns shall be remain stable against lateral loads even if the roof components are not specified to be designed for seismic loads; including tanks that need not be designed for seismic ground motion.

12.2.6.6. Sliding resistance

The transfer of the total lateral shear force between the tank and the subgrade shall be considered.
For self-anchored flat-bottom steel tanks, the overall horizontal seismic shear force shall be resisted by friction between the tank bottom and the foundation or subgrade. The friction coefficient, μ_f, is determined based on testing or reliable reference documents according to the interface between the tank bottom and the foundation or subgrade, however it shall not exceed 0.4. The base shear value calculated by Eq. 12.8 shall not exceed Vs as specified in Eq. 12.42:

$$V_s = \mu(W_s + W_r + W_f + W_P)(1 - 0.4A_v) \qquad 12.42$$

Lower values of the friction coefficient should be used if the interface of the tank bottom and the supporting foundation does not justify the friction value above (e.g., leak detection membrane beneath the bottom with a lower friction factor, smooth bottoms, etc.).
For mechanically-anchored steel tanks, there is requirement for sliding control, even though small movements of approximately 25 mm are possible.
The lateral shear transfer behaviour for special tank configurations (e.g., highly crowned tank bottoms, tanks on grillage) are beyond the scope of this Code.

12.2.6.7. Local shear transfer

Local transfer of the shear from the roof to the shell and the shell of the tank into the base shall be provided. For cylindrical tanks, the peak local tangential shear per unit length, V_{max}, shall be calculated by Eq. 12.43:

$$V_{\max} = \frac{2V}{\pi D} \qquad 12.43$$

Tangential shear in flat-bottom steel tanks shall be transferred through the welded connection to the steel bottom. The shear stress in the weld due to V_{max} shall not exceed 0.8 times the weld or base metal yield stress. This transfer mechanism is deemed acceptable for steel tanks designed in accordance with the provisions of this Code while $S_{DS} < 1.0$g.

12.2.6.8. Connection to adjacent structures

Equipment, piping, and walkways or other appurtenances attached to the tank or adjacent structures shall be designed to accommodate the elastic displacements of the tank imposed by design seismic forces amplified by a factor of 3.0 plus the amplified displacement of the other adjacent structure. When stairways are connected to the shell of the tank, the ends of the stringers of the stairways shall be clear of the ground to prevent local damage to the tank shell in the event of an earthquake.

12.3. Above-ground concrete tanks

For concrete liquid-containing structures, system ductility and energy dissipation under allowable stress design level loads shall not be allowed to be achieved by inelastic deformations to such a degree as to jeopardize the serviceability of the structure. Stiffness degradation and energy dissipation shall be allowed to be obtained either through limited micro-cracking, or by means of lateral force resistance mechanisms that dissipate energy without damaging the structure

12.3.1. Tanks with rigid walls

Seismic analysis and design of reinforced concrete storage tanks, whether aboveground, semi-buried or buried, shall be in accordance with the requirements of Code No. 123, “Design and Analysis of Ground Concrete

Water Reservoirs", First Edition, published by the Iranian Management and Planning Organization, including the following amendments:

a. Design earthquake acceleration spectrum shall be based on the definitions of this Code (Pub. Ref. 038).
b. Importance factor shall be determined according to the requirements of this code (Pub. Ref. 038).

12.3.2. Circular concrete tanks prestressed with wire, strands or tendons

Seismic analysis and design of 12.3.2 Circular concrete tanks prestressed with wire, strands or tendons shall be in accordance with the requirements of Chapter 18 or Appendix G of ACI 350, including the following amendments:

a. Design earthquake acceleration spectrum shall be based on the definitions of this Code (Pub. Ref. 038).
b. Importance factor shall be determined according to the requirements of this code (Pub. Ref. 038).
c. As the design method of prestressed tanks includes both the strength design requirements (under factored loads) and serviceability design requirements (under un-factored loads), it is due to the designer to apply the relevant parameters and relationships properly.

12.4. Elevated tanks

12.4.1. Strength and ductility

Support towers for tanks and vessels, where the support tower is integral with the tank or vessel, with irregular bracing, unbraced panels, asymmetric bracing, or concentrated masses shall be designed using the requirements of Chapter 4 for irregular structures. Support towers using chevron or eccentrically braced framing shall comply with the seismic requirements of this standard. Support towers using tension-only bracing shall be designed such that the full cross section of the tension element can yield during overload conditions.

In support towers for tanks and vessels, where the support tower is integral with the tank or vessel, compression struts that resist the reaction forces from tension braces shall be designed to resist the lesser of the yield load of the

brace, A_gF_y, or forces calculated using the seismic load effects including over-strength factor, Ω_0.
The vessel stiffness relative to the support system (foundation, support tower, skirt, etc.) shall be considered in determining forces in the vessel, the resisting elements, and the connections.

12.4.2. General

This section applies to tanks and vessels that are elevated above grade and where the supporting tower is an integral part of the structure. Tanks and vessels that are supported by another structure are considered mechanical equipment and shall be designed in accordance with the requirements of Chapter 7 (Non-Building Structures).
Elevated tanks shall be designed for the force and displacement requirements.

12.4.3. Effective mass

The design of the supporting tower or pedestal, anchorage, and foundation for seismic overturning shall assume that the stored material is a rigid mass acting at the volumetric centre of gravity. The effects of fluid–structure interaction are permitted to be considered in determining the forces, effective period, and mass centroids of the system if the following requirements are met:

a. The sloshing period, T_c, is greater than $3T$, where T is the natural period of the tank with confined liquid (rigid mass) and supporting structure, and
b. The sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid–structure interaction analysis or testing.

Soil–structure interaction is permitted to be included in determining T, provided the requirements of Chapter 6 (Soil-Structure Interaction) are met.

12.4.4. P-Delta effects

The lateral drift of the elevated tank shall be considered as follows:

a. The design drift, as determined by an elastic analysis, shall be increased by the factor C_d / I_e for evaluating the additional load in the support structure.
b. The base of the tank shall be assumed to be fixed rotationally and laterally.
c. Deflections caused by bending, axial tension, or compression shall be considered. For pedestal tanks with a height-to-diameter ratio less than 5, shear deformations of the pedestal shall be considered.
d. The dead load effects of roof-mounted equipment or platforms shall be included in the analysis.
e. Initial tilt need not be considered in the P-delta analysis if constructed within the specified allowable plumbness tolerances.

12.4.5. Lateral load transfer to support structure

For post-supported tanks and vessels that are cross-braced:

a. The bracing shall be installed in such a manner as to provide uniform resistance to the lateral load (e.g., pre-tensioning or tuning to attain equal sag).
b. The additional load in the brace caused by the eccentricity between the post-to-tank attachment and the line of action of the bracing shall be included.
c. Eccentricity of compression strut line of action (elements that resist the tensile pull from the bracing rods in the seismic force-resisting systems) with their attachment points shall be considered.
d. The connection of the post or leg with the foundation shall be designed to resist both the vertical and lateral resultant from the yield load in the bracing, assuming that the direction of the lateral load is oriented to produce the maximum lateral shear at the post-to-foundation interface. Where multiple rods are connected to the same location, the anchorage shall be designed to resist the concurrent tensile loads in the braces.

12.4.6. Structures sensitive to buckling failure

Where the structural assessment concludes that buckling of the support is the governing primary mode of failure, the following requirements shall be met:

a. The seismic response coefficient for this evaluation shall be in accordance with Chapter 4, with I/R set equal to 1.0. Soil–structure and fluid-structure interactions are permitted to be used in determining the structural response. Vertical or orthogonal combinations need not be considered.
b. The resistance of the structure shall be defined as the critical buckling resistance of the element, that is, a factor of safety set equal to 1.0.

12.4.7. Elevated water tanks

Elevated water tanks shall meet the seismic design requirements of AWWA D100 and the height limitations of Chapter 7 of this Code.

12.4.8. Elevated water tanks on concrete pedestal support

Elevated water tanks on concrete pedestal support shall meet the seismic design requirements of AWWA D107.

12.5. Pressure vessels, boilers and spheres

Refer to Chapter 7 for the requirements of pressure vessels, boilers and spheres.

12.6. Overturning and sliding stability

Overturning and sliding stability requirements are as specified in Sections 12.2.5.3, 12.2.6.6 and 12.8.3.

12.7. Tank foundations

12.7.1. General

It is beyond the scope of this Chapter to specify the general requirements of design and construction of storage tank foundations. The reader shall refer to the relevant reference standards. The foundation shall have enough strength to carry the earthquake loads due to the response of the tank to the ground motion. It shall also provide for the tank stability against overturning and sliding.

Load combinations of Section 12.1.11.1 shall be used for design of the tank foundations. Requirements for design of foundation for steel storage tanks is given in Section 12.2.5.3. Foundations for elevated tanks shall be designed in accordance to the requirements of Chapter 7 (Non-Building Structures).

12.7.2. Foundation loads for steel cylindrical tanks

For steel storage tanks, ring-wall and slab overturning moments due to seismic loading shall be as specified in Section 12.2.4.8.

12.7.3. Sliding resistance in steel flat-bottomed tanks

Refer to Section 12.2.6.6 for the specified requirements.

12.7.4. Granular berm foundation without ring-wall

For low-rise insignificant tanks, where the soil load-bearing capacity is deemed appropriate and soil settlement is not expected to be large, granular berm tank foundations of mixed granular material without a ring-wall, may be used.

12.7.5. Granular berm foundations with ring-wall

Width of the ring-wall shall be designed to restrain the differential settlement between the inside and the circumferential areas beneath the tank.
The ring-wall shall withstand the flexural, torsional and shear forces due to lateral loads.

12.7.6. Mat foundation for aboveground tanks

In grounds of weak soil or high settlement, concrete mat foundations may be used, where piles are used to transfer loads to the underlying stronger layers. Concrete mat foundations may be idealised using the theory of plates on elastic subgrade and the analytical computations may be carried out by the available software. As long as the equivalent lateral load method is used, Eq. 12.29 specifies the flexural moment applied to the concrete mat.

12.8. Secondary containment

12.8.1. The need

Tanks containing toxic, highly toxic (as defined by 29 CFR 1910.1200 Appendix A), or explosive substances are permitted to be classified as category Ordinary (corresponding to an importance factor of 1.0, Section

12.1.6) if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment that a release of the toxic, highly toxic, or explosive substances is not sufficient to pose a threat to the public.

Secondary containment of the toxic, highly toxic, or explosive substances—including, but not limited to, double-wall tank, dike of sufficient size to contain a spill, or other means to contain a release of these substances within the property boundary of the facility and prevent release of harmful quantities of contaminants to the air, soil, groundwater, or surface water—is permitted to be used to mitigate the risk of release. Where secondary containment is provided, it shall be designed for all environmental loads and is not eligible for the above-mentioned reduced classification.

12.8.3. General

Secondary containment systems, such as impoundment dikes and walls, shall meet the requirements of the applicable standards for tanks and vessels.

Secondary containment systems shall be designed to withstand the effects of the maximum considered earthquake ground motion where empty and two-thirds of the maximum considered earthquake ground motion where full, including all hydrodynamic forces.

Where it is determined by the seismic risk assessment or by the Authority Having Jurisdiction that the site may be subject to aftershocks of the same magnitude as the maximum considered motion, secondary containment systems shall be designed to withstand the effects of the maximum considered earthquake ground motion where full, including all hydrodynamic forces.

12.8.3. Freeboard

Sloshing of the liquid within the secondary containment area shall be considered in determining the height of the impoundment. Where the primary containment has not been designed with a reduction in the tank category (i.e., no reduction in Importance Factor, as permitted by Section 12.8.1), no freeboard provision is required. Where the primary containment has been designed for a reduced tank category (i.e., Importance Factor reduced), a minimum freeboard shall be provided as specified by Eq. 12.2.6.2.

For circular impoundment dikes, D shall be taken as the diameter of the impoundment dike. For rectangular impoundment dikes, D shall be taken as the plan dimension of the impoundment dike for the direction under consideration.

12.9. Further requirements

12.9.1. Self-anchored tanks

12.9.1.1. Loads

In self-anchored tanks, distribution of dynamic liquid forces over the tank bottom in terms of shape and values is of significance. Fig. 12.3 displays a schematic distribution of dynamic liquid forces over the tank bottom. The overpressure (the pressure in addition to the hydrostatic pressure inside the tank) is largely due to the convective mode of vibration. Assuming a maximum liquid wave height of d_{max}, the overpressure may be calculated.

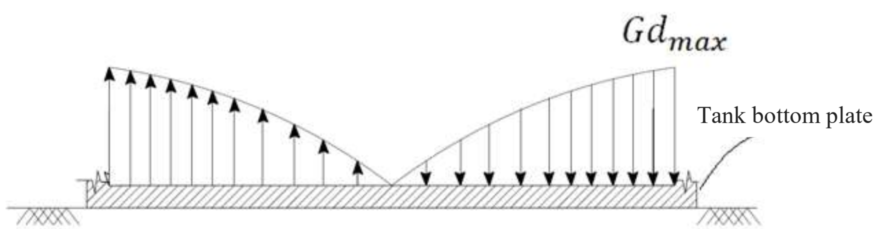


Fig. 12.3: Distribution of dynamic liquid forces on the tank bottom

12.9.1.2. Modelling

Modelling of the self-anchored tanks requires to include the separation between the tank bottom plate and the subgrade. If finite element method is used, compression-only spring elements may be used underneath the bottom plate as a simplified procedure. Such spring elements have a bilinear behaviour with a stiffness proportional to the subgrade stiffness, K_s, in compression while having no stiffness in tension.

12.9.2. Uplift estimation procedure (Non-mandatory)

This section presents methods to estimate uplift of the bottom plate in self-anchored tanks. First, an approximate method is discussed which is suitable to be used in dealing with the flexibility and displacement of piping systems attached to the tank in early stage of the design process. Then an analytical method is provided to calculate uplift, allowable uplift and membrane tensile force in the tank bottom plate due to uplift. Maximum uplift, y_u, and width of the annular plate, L, used in relationships of Section 12.9.2.3.1 are determined based on the maximum values obtained from Sections 12.9.2.1 and 12.9.2.2 below. However, it may be reduced according to the remarks and relationships given in Section 12.9.2.3.1.

12.9.2.1. Approximate method

Elastic uplift of a self-anchored tank bottom, y_u, may be approximately estimated by Eq. 12.44:

$$y_{\mathrm{u}} = \frac{12.1 F_y L^2}{t_b - C.A.} \qquad 12.44$$

$$L = 0.01723 t_a \sqrt{F_y / (H G_e)} \qquad 12.45$$

12.9.2.2. Analytical method

Figs. 12.4 and 12.5 depict the forces applied to bottom of a self-anchored tank following to the uplifting. For circular tanks, maximum uplift, y_u, at the joint of the bottom plate and the tank shell should meet this inequality:

$$0.001 y_{\mathrm{u}} \left(\frac{H}{2} + 0.001 y_{\mathrm{u}}\right) \leq \frac{R^2}{2} \qquad 12.46$$

An important effects of uplifting is the axial force within the tank shell. It also may cause warping in the shell and axial forces within the bottom plate.

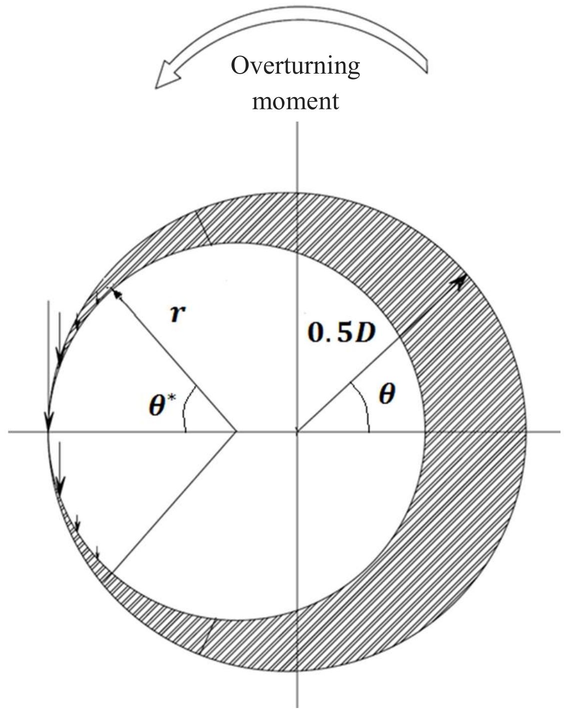


Fig. 12.4: Forces applied to bottom of a self-anchored tank following to the uplifting

If M_{rw} and M_R stand respectively for the overturning and the restoring moments, we have:

$$M_{\mathrm{R}} = 0.5\mathrm{W}_{su}\mathrm{k}D + \mathrm{W}_{pu}(0.5\mathrm{D} - \mathrm{r}) \qquad 12.47$$

W_{pu}: Weight of the contents of the portion of the tank bottom (a circle of radius r) which is still in contact with the ground, N.
$W_{su} = W_P + W_r + W_s - W_a$, Compressive reaction at the base of the tank shell.
θ^*: One half of the angle subtended by an arc of the bottom plate which is in contact with the ground following to the uplift.
0.5kD: Distance between the centre of compressive reaction and the tank centre.
Restoring moment, M_R, is calculated iteratively following these steps:
Step 1 – make a guess for the value of $\tau = \frac{2r}{D}$.

Step 2 – calculate θ^* using Eq. 12.48:

$$\theta^* = \arctan(\frac{\tau}{1-\tau}) \tag{12.48}$$

Step 3 – calculate k using Eq. 12.49:

$$k = \frac{2}{\theta^{*2}}(1-\cos\theta^*) \tag{12.49}$$

Step 4 – calculate M_R using Eq. 12.50:

$$M_R = 0.5DW_T(k\left(1+\frac{W_r+W_s}{W_P}\right)+(1-k)\tau^2-\tau^3) \tag{12.50}$$

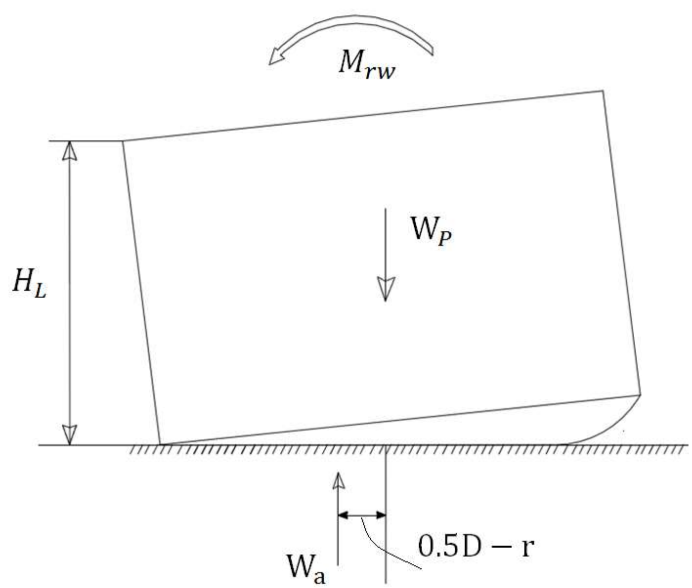


Fig. 12.5: Post-uplifting diagram of forces in a self-anchored tank

Step 5 – repeat steps 1 to 4 until $M_{rw} = M_R$.
Uplift is calculated by Eq. 12.51:

$$y_u = \frac{1}{C}\left[\frac{F_y {t_b}'^2}{6{N_X}'} + \frac{{p_0}'L}{{N_X}'}\left[\frac{L}{2} - (\frac{\bar{E}{t_b}'^3}{12{N_X}'})^{0.5}\right]\right] \tag{12.51}$$

where:

F_y is yield strength of tank bottom plate

$$N_x = t_b f_{rb}$$

$$L = D(1 - \tau)$$

C is foundation stiffness, equal to 1.0 for rigid foundations and 0.5 for flexible foundations.

Note: A monolithic reinforced concrete foundation is considered as rigid and a compacted soil, with or without lean concrete layer, is considered as flexible. For a concrete ring-wall foundation with inside compacted soil fill, a value of 0.5 should be used.

In this equation, f_{rb} is the radial stress within the bottom plate:

$$f_{rb} = \frac{1}{t_b}\left[\frac{2\bar{E}t_b {p_0}^2(0.5D)^2(1-\tau)^2}{3}\right]^{\frac{1}{3}} \quad 12.52$$

where,

t_b: is the thickness of the bottom plate.

$$\bar{E} = \frac{E}{1-\nu^2} \quad 12.53$$

$H \times G = p_0$: is hydrostatic pressure on the bottom plate.

$$p_0' = p_0 * 10^6 \quad 12.54$$

$$N_X' = N_X * 10^3 \quad 12.55$$

$$t_b' = t_b * 10^{-3} \quad 12.56$$

12.9.2.3. Limit on maximum uplift

12.9.2.3.1. Limit on plastic rotation at the base plate of an uplifting tank

To let the membrane action develop within the bottom plate, it is required for a plastic hinge to develop at the joint of the tank shell and the bottom plate. To prevent weld failure at this joint, there needs to limit the rotation of the connection. Observations in past earthquake events have shown such kind of damage. Referring to Fig. 12.6 and taking 0.05 radians as the maximum allowable rotational strain, we can have a maximum rotation of 0.2 radians from Eq. 12.57. The relationship between θ_P and the maximum uplift is defined as Eq. 12.58.

$$\theta_P = \left(\frac{0.05}{\frac{t_b}{2}}\right) 2t_b = 0.2 \text{ Radians} \qquad 12.57$$

$$\theta_P = \left(\frac{2y_u}{1000 \times L} - \frac{y_u}{1000 \times D}\right) \leq 0.20 \qquad 12.58$$

If the plastic rotation exceeds the limit set forth in Eq. 12.58, it should be reduced by increasing the thickness of the bottom plate which lets y_u to decrease for a given value of L. If thickness of the bottom plate calculated as such, exceeds the shell plate thickness, it may be required to make the tank mechanically anchored to prevent uplift.

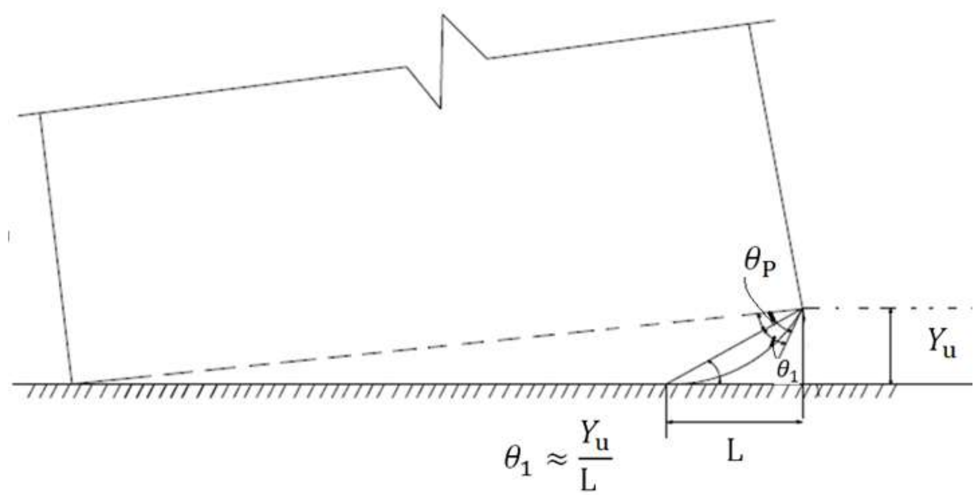


Fig. 12.6: Limiting the plastic rotation at the joint between the shell and bottom plates of an uplifting tank

12.9.2.3.2. Limitations on shell plate thickness

Thickness of the shell plate shall in no case be less than the values specified in the following table. Constructional limitations requires to keep an upper limit of 40 millimetres on the shell plate thickness.

Table 12.7: Minimum tank shell plate thickness

Nominal shell plate thickness (mm)	Nominal tank diameter (m)
5	Less than 15
6	15 to 36
8	36 to 60
10	More than 60

12.9.2.3.3. Limit on maximum uplift

Maximum uplift shall satisfy the following inequality:

$$0.001y_{\mathrm{u}}(\frac{H}{2}+0.001y_{\mathrm{u}}) \leq \frac{(0.5D)^2}{2} \qquad 12.59$$

If the inequality is not satisfied, then it is required to reduce the maximum uplift (i.e., increase the bottom plate thickness).

12.9.3. Buried tanks

Buried or semi-buried tanks are those which are partially or to the full height below natural ground level. This may be required by passive defence provisions. In addition to the general requirements of aboveground tanks, such tanks need to account for the static and dynamic pressure applied to the shell from the surrounding soil.

Buried tank design shall check for two critical loading conditions: an empty tank under lateral earth pressure, and a full tank without pressure from the surrounding soil. The engineering calculations for the latter case (a full tank without pressure from the surrounding soil) is as specified earlier in this Chapter. The following Sections 12.9.3.1 and 12.9.3.2 deal with the forces due to the surrounding soil.

12.9.3.1. Static earth pressure on tank walls

Static earth pressure is proportional to the depth from the ground level as defined in Eq. 12.60. Overburden pressure due to the loads on the surrounding soil should be added to the static earth pressure. In this equation, k_0 is the coefficient of lateral earth pressure at-rest.

$$P_S = k_0 \gamma_s H_e \qquad 12.60$$

12.9.3.2. Dynamic earth pressure on tank walls

Eq. 12.61 gives the minimum resultant dynamic lateral force due to the vibration of the soil surrounding tank walls. k_h, is determined depending on the soil properties and lateral displacement of the tank walls due to seismic motion. It shall not be taken less than $\frac{A}{2}$ for walls satisfying the active earth pressure requirements (in accordance with National Building Code, Part 7, Section 7.5.4.2.2) and less than A for stiff walls.
Distribution of the dynamic pressure over wall height should be in a trapezoidal shape as shown in Fig. 12.7. If there is a uniformly distributed overburden, p'_0, then the resulting dynamic pressure equal to $k_h p'_0$ should be applied uniformly in a rectangular shape over the height of the wall.

$$\Delta P_{OE} = \gamma_s {H_e}^2 k_h \qquad 12.61$$

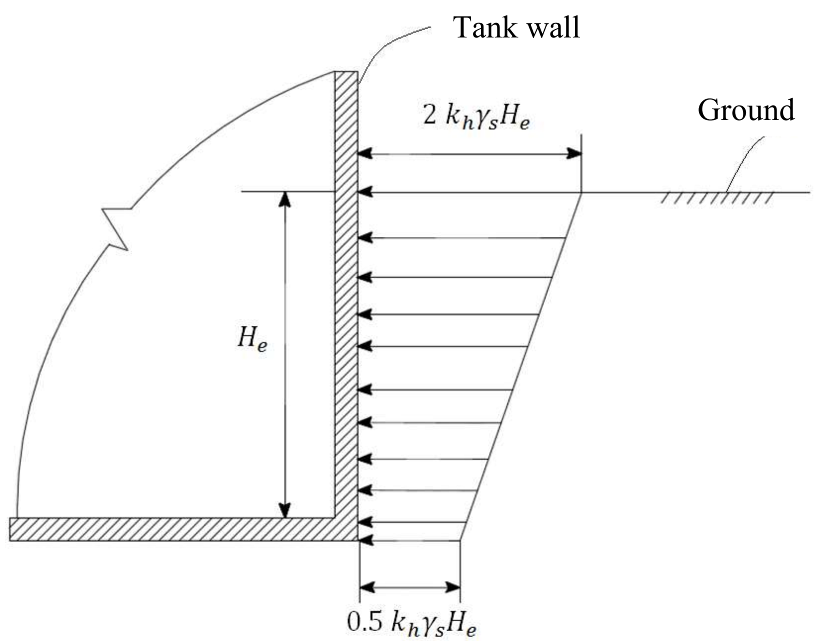


Fig. 12.7: Dynamic earth pressure on tank walls

12.9.4. Base-isolated tanks

With the exception of the requirements specified in Section 12.9.7.14 for low-pressure storage tanks, design of base-isolated tanks is beyond the scope of this Chapter.

12.9.5. Corrugated steel tanks

Corrugated steel water storage tanks, except those assigned to category Ordinary, shall meet the requirements of Sections 12.1.5.5, 12.1.5, 12.1.6, 12.1.8 and the following requirements:

a. Vertical elements shall be added to the tank shell to resist compressive seismic forces.
b. The base shear and overturning moment shall be calculated as if the tank and the entire contents act as an impulsive mass. The convective mass shall be set equal to zero.
c. Tank sliding, relative to the foundation, shall be resisted by embedment of the tank shell in the foundation. Alternatively, the shell is permitted to be confined by a concrete curb designed to transfer the base shear forces from the tank to the foundation.
d. The tank shall be mechanically anchored to the foundation to resist seismic uplift.

12.9.6. Plastic tanks

Refer to Chapter 7 for requirements of plastic tanks.

12.9.7. Low-temperature liquid storage tanks

12.9.7.1. General

Tanks and facilities for the storage of liquefied hydrocarbons and refrigerated liquids shall meet the requirements of this Code. Low-pressure welded steel flat-bottom, ground-supported storage tanks for liquefied hydrocarbon gas (e.g., liquefied petroleum gas or butane) and refrigerated liquids (e.g., ammonia) shall be designed in accordance with the requirements of this Code and API 620.

12.9.7.2. Earthquake levels and performance objectives

Tank systems for low-temperature liquid storage shall be designed for the following three levels of seismic motion.

- **Operating Level Earthquake (OLE).** The tank system shall be designed to continue to operate during and after OBE event.
- **Safe Shutdown Earthquake (SSE).** The tank system shall be designed to provide for no loss of containment capability of the primary container

and it shall be possible to isolate and maintain the tank system during and after SSE event.

- **Aftershock Level Earthquake (ALE).** The ALE earthquake shall be considered only for the seismic design of secondary containment, assuming that the primary container is damaged after the SSE event. The tank system, while subjected to ALE, shall provide for no loss of containment from the secondary container while containing the primary container volume at the maximum normal operating level. The ALE shall be defined as half of the SSE event.

12.9.7.3. Seismic design requirements

Steel storage tanks shall be design in accordance with the following requirements in addition to the requirements of Section 12.2. Concrete storage tanks shall be design in accordance with ACI 376.

If the tank foundation rests on soil (i.e., ground condition category II through IV according to Iranian Standard No. 2800) it is required to perform soil-structure analysis. Refer to Chapter 6.

Tanks supported on a framework elevated above grade are beyond the scope of this Section.

12.9.7.4. Force reduction factors

Tables 12.8 and 12.9 specify maximum allowable reduction factors for low-temperature liquid storage tanks.

The inner and outer tanks may be decoupled for seismic design of the tank and anchorage and assumed to act independently. However, if the inner and outer tanks are supported by a common foundation, the seismic loading of the foundation shall be calculated using the lesser response modification values for either the inner tank or outer tank for both tanks and a dynamic analysis shall be performed to determine the combined effect.

12.9.7.5. Allowable stresses

For storage tanks of low-temperature liquids, design allowable stresses depend on the tank material or alloys used, and shall be specified referring to Annex Q or R of API 620.

When the seismic load effects are combined with those from other design loads, the allowable stresses may be increased by 33%, however they shall not exceed 80% of the specified minimum yield strength for carbon steel; and the allowable tensile stress shall not exceed 90% of the minimum specified yield strength for stainless steel or aluminium.

Table 12.8: Maximum force reduction factors for liquefied gas storage tanks at temperature -198 °C or above, designed in accordance to API 620, Annex Q

Anchorage system	Impulsive mode, R_{wi}	Convective mode, R_{wc}
Inner tank		
Steel (nickel, or stainless)		
Self-anchored	1.5	1.0
Mechanically anchored	1.75	1.0
Aluminium		
Self-anchored	1.25	1.0
Mechanically anchored	1.5	1.0
Outer tank (empty)		
Self-anchored	2.0	Not applicable
Mechanically anchored	2.0	Not applicable
NOTE: The above factors are applied for SSE event. For OLE, the elastic design (i.e, reduction factor = 1.0) is used.		

Table 12.9: Maximum force reduction factors for storage tanks operating between -51 and 5 °C, designed in accordance to API 620, Annex Q

Anchorage system	Impulsive mode, R_{wi}	Convective mode, R_{wc}
Inner tank		
Self-anchored	2.25	1.5
Mechanically anchored	2.5	1.5
Outer tank (empty)		
Self-anchored	2.0	Not applicable
Mechanically anchored	2.0	Not applicable
NOTE: The above factors are applied for SSE event. For OLE, the elastic design (i.e, reduction factor = 1.0) is used.		

12.9.7.6. Annular plate

Width, thickness and fabrication requirements for the annular plate of the inner and outer tanks shall be as specified in Annex Q or R of API 620.

12.9.7.7. Sliding resistance

The tank system, whether self-anchored or mechanically-anchored, shall be configured such that the overall horizontal shear force at the base of the tank does not exceed the friction capacity. Mechanical anchorage shall not be used to resist sliding.

12.9.7.8. Insulation load

For tanks designed and constructed with an outer tank containing loose fill insulation in the annular space between the tanks, the insulation weight shall be divided equally to the inner and outer tank wall for seismic lateral loads unless a more rigorous analysis is performed to determine the distribution. The insulation within the annular space shall not be used to calculate resistance to overturning.

Insulation on the roof or suspended deck shall be applied to the tank supporting the load at the point or centre of gravity of attachment and may be used to resist overturning. For single wall tanks with insulation or double wall tanks with the insulation adhered to the plate surface, the additional weight of the insulation shall be included and may be included in the tank weight used to resist overturning.

12.9.7.9. Additional roof loads

When $S_{DS} > 0.33g$, the tank is classified as Critical category, and equipment loads such as pumps, platforms, piping platforms supported directly by the roof exceed 25% of the combined weight of the roof and shell, a dynamic analysis shall be performed to determine the effective roof weight and the seismic response of the roof and parts connected to the roof.

12.9.7.10. Provisions for performance level design

12.9.7.11. Operating level earthquake

Adjustment factors. The OLE design forces shall not be adjusted by an importance factor, or force reduction factor. Nor shall the forces be reduced by the 0.7 multiplier applied to convert contingency level events to ASD methods.

Soil-structure interaction. Soil-structure interaction may be used providing the damping ratio does not exceed 10%.

Allowable stresses. Shall be in accordance with Section 12.9.7.5.

Self-anchored inner tank. The anchorage ratio for a self-anchored inner tank shall not exceed 1.0.

Stability. The overturning ratio shall be equal to or greater than 3.0.

Inner tank freeboard. Freeboard shall be equal to the maximum wave height as specified in Section 12.2.6.2, plus an additional height of not less than 300 mm.

Piping Flexibility. Flexibility of connections and supports shall satisfy the limitations of Table 12.6. A 33% increase in stress is permitted.

Sliding Resistance. The safety factor against sliding shall not be less than 1.5. Anchorage may not be used to resist sliding. If the sliding force exceeds the available friction, the tank shall be re-configured.

Connections with Adjacent Structures. The calculated displacements shall be amplified by 1.25 for OLE.

Bottom and Shell Support. The tank under-bottom insulation shall be designed to resist the combined pressures from the product load, the overturning seismic load and the vertical seismic load. These seismic pressures may be combined by SRSS. The bearing ring under the shell shall be designed to resist the calculated OLE peak compressive force in the tank shell due to overturning, including dead and live loads. A 33% increase in allowable bearing stress is permitted.

12.9.7.12. Safe shutdown earthquake

Adjustment factors. Importance factor shall be taken as 1.0. Force reduction factor shall be as specified in Section 12.9.7.4.

Inner Tank Freeboard. Freeboard shall be equal to the maximum wave height as calculated by the relationship given in Section 12.2.6.2.

Sliding Resistance. The safety factor against sliding shall not be less than 1.0. Anchorage may not be used to resist sliding. If the sliding force exceeds the available friction, the tank shall be re-configured.

Soil-structure interaction. Soil-structure interaction may be used providing the damping ratio does not exceed 20%.

Allowable stresses. Shall be in accordance with Section 12.9.7.5.

12.9.7.13. Aftershock level earthquake

If the outer tank is not designed as a secondary containment (i.e. it serves as vapour barrier and pressure boundary only and is not constructed of material suitable for the inner tank), then no design or evaluation for ALE is required for the inner or outer tank.

If the outer tank is designed as the secondary containment (i.e. constructed of material suitable for the inner tank and designed to contain liquid contents), the outer tank, foundation and anchorage shall be designed for the ALE assuming the inner tank no longer exists and all of the liquid is contained by the outer tank system and the following provisions apply.

Adjustment factors. Importance factor shall be taken as 1.0. Force reduction factor shall be as specified in Section 12.9.7.4.

Soil-structure interaction. Soil-structure interaction may be used.

Allowable stresses. Shall be in accordance with Section 12.9.7.5.

12.9.7.14. Base isolation

Base isolations systems may be used providing:

1. the inner and outer tanks are both isolated on a common foundation to avoid excessive differential displacements between the tanks and connecting internals;

2. the anchorage, internal and external piping, insulation and other attached equipment are designed for the larger differential deformations associated with an isolated system;
3. a site-specific response spectrum is mandatory and includes the long term periods necessary to define the system response;
4. all external piping connections to the isolated system are designed for the calculated displacements for the actual ground motions (i.e., no importance or force reduction factor is applied:
5. the design is peer-reviewed for technical adequacy by an independent party knowledgeable in the design and behaviour of base-isolation systems; or, the design is verified by scaled shake-table tests or 3.dimensional nonlinear analysis applying simultaneous horizontal and vertical time-histories fit to the design OLE and SSE spectra and including the supporting soil and isolators.

Refer to Chapter 9 for the requirements of the base isolation.

Chapter 13
Pipeline Systems

13.1. General notes

The basic design of the pipelines is usually based on the process and mechanical needs such as operating pressure, temperature, fluid type and the like, which is outside the scope of this Regulations. In this chapter, only the load combinations that include seismic loads and the rules related to the control and design of the pipeline for seismic hazards are presented. It is notable that the load combination used in this section differ from the provisions of Chapter 2 as they have been specifically stated for pipelines.

In facing geotechnical hazards, two approaches are permissible. The first approach is a simplified method as will be presented in the following. The second approach relies on the considerations of Chapter 5 that is a more accurate method. In any case, even in the first approach, in qualitative discussions and preventive considerations, it is necessary to pay attention to the criteria raised in chapter 5, as long as it does not conflict with the proposed routines of this chapter.

Pipelines are divided into two types: continuous and segmented. Steel pipelines with welded joints are considered continuous, while segmented pipelines include cast iron pipes with gasket joints, malleable iron pipes with rubber gasket joints, asbestos pipes and similar ones.

The pipeline shall be controlled for all possible seismic hazards, which are discussed in this chapter where the analysis procedure and general design criteria of the pipeline for some general seismic hazards are given. In the case of specific local risks, the seismic evaluation of the pipeline shall be done based on the specific local study reports.

In areas prone to seismic construction risks, appropriate arrangements shallbe made to cut off the fluid flow and quickly replace the damaged parts of the pipe. Also, if possible, it is recommended to choose the route of the pipe in such a way that it does not pass through the active faults or regions susceptible to severe geotechnical hazards.

In Section 13.2, the seismic design categories of the pipeline are specified. Details related to seismic loading and analysis methods are presented for buried pipelines in Section 13.3, above-ground pipelines in Section 13.4, and pipelines bearing on supporting structures in Section 13.5.

Seismic analysis of the pipeline can be done by the equivalent seismic load method, according to the criteria of Sections 13.3.1 and 13.4.1, or more accurately by the dynamic method, according to the criteria of Sections 13.3.2 and 13.4.2.
The content of this chapter refers only to the onshore pipelines and does not include the offshore pipelines.

13.1.1. Definitions

The definitions of some of the keywords used in this chapter are common to those presented in other chapters. Among these keywords, "design earthquake", "peak ground acceleration" and "peak ground velocity", are the ones mentioned in Chapter 3, too. Also, the expressions "shear wave velocity", "liquefaction", "landslide", "lateral expansion", "fault rupture" and "permanent ground displacement" are defined in Chapter 5. Other required definitions are mentioned in the following sections.

13.1.2. Symbols

The symbols and notations used in this chapter are listed in alphabetical order as follows.

A_g	:	The maximum acceleration in the direction perpendicular to the direction of wave propagation caused by the design earthquake, which is considered equal to the maximum design acceleration in the related risk category.
$A_{1\text{-}5}$	:	Coefficients used in calculating the coefficient of horizontal bearing capacity of the soil
A_p	:	The cross-section of the pile or pipe
C	:	Wave propagation velocity
D	:	The nominal diameter of the tank or the outer diameter of the pipe
D'	:	An auxiliary parameter to calculate the allowable compressive strain under seismic waves
D_{min}	:	The smallest inner diameter of the pipe, taking into account the roughness or distortion of the pipe wall
E_p	:	The initial elastic modulus of the pipe
F_b	:	The net upward force per unit length of the pipe
F_{stop}	:	Maximum design friction force

f	:	Friction coefficient between soil and pipe
g	:	Gravity acceleration
H_s	:	Distance from the ground surface to the center of the buried pipe
h_{sp}	:	Height of the soil on the pipe
h_w	:	Water table height above the pipe
I_L	:	Pipe importance factor
i	:	Stress intensification coefficient
k_0	:	Lateral earth pressure coefficient of the soil at rest
L	:	Length of the pipe region with permanent displacement
L_a	:	The unrestrained length of the pipe
L_b	:	Length of the pipe in the buoyancy region
L_{max}	:	Maximum permissible span of the pipe between two lateral and vertical seismic restraints
L_T	:	Recommended value for the distance between pipe seat supports
L_z	:	Length of the permanent displacement region in landslides
L_0	:	Length of the pipe between two connections
M_a	:	Equivalent moment due to relative displacement between restraints
M_i	:	Equivalent moment due to inertial forces
M_w	:	Seismic moment magnitude
N_c	:	Soil bearing capacity coefficient
N_{ch}	:	Coefficient of lateralsoil bearing capacity depending on adhesion
N_{cv}	:	Coefficient of soil bearing capacity related to uplift strains depending on adhesion
N_q	:	Coefficient of soil bearing capacity
N_{qv}	:	Coefficient of soil bearing capacity related to tensile strains depending on internal friction
N_{qh}	:	Coefficient of soil bearing capacity depending on internal friction
N_γ	:	Soil bearing capacity coefficient
n	:	Pipe material constitutive model parameter
n_C	:	Number of the chain connections at the beginning or the end of the mobile mass of soil, which can withstand the permanent displacement of the earth by increasing each connection length.

P_p	:	The maximum internal operational pressure in the pipe
P_u	:	The maximum lateral resistance of the soil per unit length of the pipe
P_v	:	Vertical pressure of soil
Q_d	:	Compressive bearing capacity of soil per unit length of the pipeline
Q_u	:	Tensile bearing capacity of the soil per unit length of the pipeline
R	:	Outer radius of the pipe
r	:	Pipe material constitutive model parameter
S_{DS}	:	Spectral acceleration (in terms of g) related to the short period (0.2 seconds) in the seismic design spectrum with a damping ratio of 5%
S_p	:	Longitudinal stress in the pipe caused by internal pressure
S_r	:	Longitudinal stress caused by temperature change in the pipe
S_s	:	Allowable seismic stress at the temperature of -30 to 40 degrees
S_u	:	Undrained shear strength of soil
T_1	:	Temperature in the pipe during installation
T_2	:	Temperature in the pipe during operation
t	:	Thickness of the pipe wall
t_p	:	Nominal thickness of the pipe wall
t_u	:	The maximum friction force per unit length of the pipe-soil contact surface
V_g	:	Peak ground velocity for each risk category and pipe hazard
V_{g0}	:	Peak ground velocity in the desired location at the second hazard level
W_c	:	Weight of the pipe contents per unit length
W_p	:	Pipe weight per unit length
W_s	:	Total weight of the soil with equivalent volume of the pipe per unit length
x	:	Pipe burial ratio
Z_e	:	Section modulus of the pipe
α_t	:	Linear coefficient of thermal expansion
α_ε	:	Soil strain coeficient
β	:	The intersection angle of the pipeline with the fault

Δ_a	:	Confidence margin for connection deformation
$\Delta_{allowable}$	:	Allowable connection deformation provided by the manufacturer
Δ_{D+L}	:	Deformation of connection caused by gravity loads
Δ_{oper}	:	Maximum operational displacement in the connection
$\Delta_{seismic}$	:	The maximum seismic deformation in the pipeline
$\Delta_{oper+seismic}$	:	The maximum joint deformation caused by the operational loads and earthquake
Δ_p	:	The maximum amount of transverse displacement of the pipe
Δ_{Qu}	:	Displacement at which the uplift strength is mobilized
Δ_{Qd}	:	Displacement at which the compressive strength is mobilized
Δ_t	:	The maximum active soil displacement along the axis of the pipeline
δ	:	Friction angle between soil and pipe
δ_{fax}	:	The fault displacement along the pipeline
$\delta_{fax\text{-}design}$	:	The design fault displacement along the pipeline
δ_{fb}	:	Mean displacement of the fault with unknown type
δ_{fn}	:	Average displacement of the normal fault
δ_{fr}	:	Average displacement of the reverse fault
δ_{fs}	:	Average displacement of the strike-slip fault
δ_{ftr}	:	The transverse fault displacement perpendicular to the pipeline
$\delta_{ftr\text{-}design}$	:	The design transverse fault displacement perpendicular to the pipeline
δ_{fvt}	:	The vertical fault displacement
$\delta_{fvt\text{-}design}$	:	The vertical design fault displacement
δ^l	:	Maximum longitudinal displacement of the ground
δ^l_{design}	:	longitudinal design displacement of the ground
δ^t	:	Maximum permanent lateral displacement of the ground
δ^t_{design}	:	Permanent lateral design displacement of the ground
ε	:	Pipe strain
ε_a	:	Axial pipe strain
$\varepsilon_{allowable}$	:	Allowable pipe strain

ε_b	:	Maximum bending pipe strain
$\varepsilon_{c\text{-}PGD}$	:	Allowable pipe strain under permanent displacements
$\varepsilon_{c\text{-}wave}$	:	Allowable pipe compressive strain under wave propagation
$\varepsilon_{cr\text{-}c}$	:	Pipeline wrinkling threshold strain
ε_{D+L}	:	Pipe strain caused by gravity loads
ε_{oper}	:	Operating strain in the pipeline
ε_p	:	Pipe strain caused by internal pressure
$\varepsilon_{seismic}$	:	Pipe design strain caused by seismic hazards
ε_t	:	Pipe strain caused by temperature changes
ε_u	:	Ultimate tensile strength of pipe material
ε_y	:	Material yield strain
ϕ	:	The angle of internal soil friction
$\overline{\gamma}$	:	Effective unit weight of soil
γ_d	:	Unit weight of dry soil
γ_w	:	Unit weight of water
λ_e	:	The apparent wavelength of the wave field on the ground surface
$\theta_{seismic}$	:	Seismic angular deformation of the connection
σ	:	Stress in the pipe
σ_{bf}	:	Bending stress caused by buoyancy
σ_y	:	Yield stress of pipe material
σ_{yo}	:	Yield stress of pipe material at operating temperature
ψ	:	Angle between the normal fault rupture surface and horizontal surface

13.2. Risk category

The pipeline is divided into four groups in terms of usage and risk.

Group I: Pipeline with essential use, including pipeline containing flammable substances with high pressure or temperature, or pipeline containing toxic substances, pipeline that must maintain its performance during and after an earthquake, such as fire water pipeline, and the pipeline whose breakdown or damage can lead to extensive loss of life or severe impact on the environment.

Group II: Pipeline with important use, including a pipeline whose damage is dangerous for the public. The main distribution pipeline and the pipeline whose destruction will cause a lot of economic losses, such as the oil and gas pipelines with medium pressure, which is a vital supplier of energy but its performance can be interrupted to the extent of partial repairs, is categorized in this group.

Group III: Pipelines with normal use, excluding groups I, II and IV, such as low-pressure oil and gas pipelines.

Group IV: Pipeline with very little importance, in which damage will have an insignificant effect on the safety of life, the environment, and the operation of the facility, and will not require immediate repair.

13.3. Buried Pipelines

Seismic hazards that are directly related to buried pipeline damage can be classified as follows:

- Earthquake wave propagation
- Permanent ground deformations caused by fault rupture, landslide, and liquefaction (including settlement and lateral expansion).

Analysis of the buried pipeline against seismic waves and also under the effect of ground deformation shall be done according to the requirements of this section. The hazard levels for groups III, II and I have the return period corresponding to the design earthquake (the second hazard level according to the criteria of Chapter 3), 975 years and 2475 years, respectively. Also, it is allowed that for all risk categories, the acceleration, velocity and displacement components of the design earthquake are used by applying the importance factor presented in Table 13.1. For group IV, there is no need to consider seismic considerations.

Table13.1: Importance factor (I_L) for different risk categories

Risk category	Wave propagation	Fault rupture	Liquefaction induced permanent displacements	Landslide
I	1.5	2.3	1.5	2.6
II	1.25	1.5	1.35	1.6
III	1	1	1	1

In general, it is recommended to use the capacity of the inelastic behavior of the pipeline, but the critical parts of the pipeline that can cause widespread casualties or severe impact on the environment shall remain elastic.
If the constitutive model of the pipe material is not determined, it is possible to use the relation 13.1 as an estimate:

$$\varepsilon = \frac{\sigma}{E_p}\left[1+\frac{n}{1+r}\left(\frac{\sigma}{\sigma_y}\right)^r\right] \qquad 13.1$$

where:
ε : Strain
σ : Stress
E_p: initial elastic modulus
σ_y : Yield stress of the pipe material
n, r : Parameters of the constitutive model presented for some API 5L standard pipes in Table 13.2.
For other pipe materials, the parameters of the constitutive model can be obtained from the experiment or reliable references.

Table 13.2: Parameters of the constitutive model for steel pipes

Pipe grade	Grade-B	X-42	X-52	X-60	X-70
n	10	15	9	10	5.5
r	100	32	10	12	16.6

Following the application of operational loads, the earthquake load is applied to the pipeline and the generated strains are calculated. The resulting strain shall be less than the allowed value.

The stresses (or strains) resulting from seismic analysis, shall be combined with the stresses (or strains) resulting from internal pressure and temperature change according to the relations 13.2 and 13.3.
The longitudinal stress in the pipe caused by internal pressure is:

$$S_p = \frac{P_p D \nu}{2 t_p} \qquad 13.2$$

P_p: The maximum internal operational pressure in the pipe

D : The outer diameter of the pipe

ν : Poisson's ratio (that for steel can be assumed to be 0.3)

t_p : Nominal thickness of the pipe wall

The longitudinal stress caused by temperature change in the pipe, can be calculated from Eq. 13.3:

$$S_r = E_p \alpha_t (T_2 - T_1) \qquad 13.3$$

where:

α_t: Coefficient of linear thermal expansion

T_1 : Temperature in the pipe during installation

T_2 : Temperature in the pipe during operation

The maximum allowable strain values of buried continuous pipes, corresponding to the pipes that comply with the API-5L standard, are presented in Table 13-3. For other types of pipes, the allowable strains provided by the pipe manufacturer can be used as a criterion after approval by the competent authorities.
The onset strain of the pipe wrinkling is determined from Eq. 13.4:

$$\varepsilon_{cr-c} = 0.175 \frac{t_p}{R} \qquad 13.4$$

where:

R : The outer radius of the pipe

Other parameters presented in Table 13.3 are determined from Eqs.13.5 to 13.7:

$$\varepsilon_{c-PGD} = 0.88 \frac{t}{R} \qquad 13.5$$

$$\varepsilon_{c-wave} = 0.75\left[0.5\frac{t}{D'} - 0.0025 + 3000\left(\frac{P_p D}{2E_p t}\right)^2\right] \quad 13.6$$

$$D' = \frac{D}{1 - \frac{3}{D}(D - D_{\min})} \quad 13.7$$

in which:

t: Pipe wall thickness

$D_{\min}$: The smallest inner diameter of the pipe, taking into account the unevenness or distortion of the pipe wall (Figure 13.1)

Table 13.3: Allowable strain criteria for buried continuous pipelines

Allowable Strain		Pipe Category	Scope of application
Tension	Compression		
2%	For permanent ground displacement: Onset of Wrinkling (ε_{cr-c}) For wave propagation: 50% to 100% of the onset of wrinkling (0.5 to 1 ε_{cr-c})	Ductile Cast Iron Pipe	Continuous Oil and Gas Pipeline
3%		Steel Pipe	
20%		Polyethylene Pipe	
1%		Bends and Tees of pipe	
0.25 ε_u[1] or 5%	For permanent ground displacement: ε_{c-PGD} For wave propagation: ε_{c-wave}	Steel and iron pipe	Continuous Water Pipeline

[1] ε_u is the ultimate tensile strain is the pipe material.

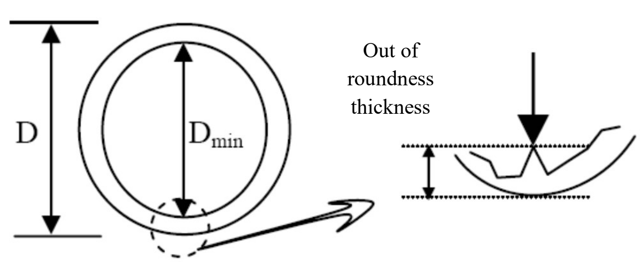


Figure 13.1: Determining D_{min}

The design strain of the continuous pipeline shall be less than the allowable strain, i.e.:

$$\varepsilon_{seismic} + \varepsilon_{oper} \leq \varepsilon_{allowable} \qquad 13.8$$

where:

$\varepsilon_{allowable}$: Allowable pipe strain according to Table 13.3

$\varepsilon_{seismic}$: Pipe design strain caused by seismic hazards

ε_{oper} : The operating strain in the pipeline. equal to $\varepsilon_p + \varepsilon_t + \varepsilon_{D+L}$

ε_p : Pipe strain caused by internal pressure

ε_t : Pipe strain caused by temperature changes

ε_{D+L}: Pipe strain caused by gravity loads

In segmented pipes, the maximum deformation in the location of the connections throughout the pipeline shall be less than its allowable value according to Eq. 13.9:

$$\Delta_{oper+seismic} \leq \Delta_{allowable} - \Delta_a \qquad 13.9$$

where:

$\Delta_{allowable}$: The allowable connection deformation, which is determined and provided by the connection manufacturer.

Δ_a: Safety margin for connection deformation, usually equal to 6 mm

$\Delta_{oper+seismic}$: The maximum connection deformation due to the application of operating load and earthquake

Δ_{oper}: The maximum change of operating location in the connection, equal to $\Delta_p + \Delta_t + \Delta_{D+L}$

Δ_p: Deformation of connection caused by internal pressure

Δ_t: Deformation of connection caused by temperature change

Δ_{D+L}: Deformation of connection caused by gravity loads

In calculating the deformation caused by operating loads, it is necessary to consider the effects of gravity, thermal and pressure loads. The deformation values of the connection in the above relationship are calculated from the product of the strain of the pipeline along the length of the pipe segment.

13.3.1. Analysis for seismic waves using the equivalent seismic load method

It is possible to ignore the bending strain of the pipe resulted from ground curvature, due to its small amount, and consider only the longitudinal axial strain of the pipe as the seismic response of the pipeline against earthquake motion.

In this Regulations, the peak ground velocity and the corresponding hazard is considered as a characteristic of the seismic design. The peak ground velocity can be determined for each risk category and the corresponding return period according to Section 13.2 or obtained from Eq. 13.10:

$$V_g = V_{g0}\, I_L \qquad 13.10$$

where:

V_{g0}: The peak ground velocity at the desired location in the earthquake at the second risk level, which can be calculated according to the criteria of Chapter 3 using the peak ground acceleration related to the earthquake risk level.

I_L: Importance factor according to Table 13.1.

13.3.1.1. Pipe strains due to seismic waves

The axial strain of the continuous pipe caused by seismic waves is estimated using the wave propagation velocity. As a general rule in determining this strain, velocity of the shear wave (S-wave) is used in places whose distance to the epicenter of the earthquake is less than five times its focal depth. Otherwise, velocity of the Rayleigh wave (R-wave) is used.

The maximum longitudinal axial strain that can be created in the pipe due to the propagation of earthquake waves can be estimated from equation 13.11:

$$\varepsilon_{seismic} = \frac{V_g}{\alpha_\varepsilon C} \le \frac{t_u \lambda_e}{4 A_p E_p} \qquad 13.11$$

where:

α_ε: Ground strain coefficient (2 for S-wave and 1 for other waves)

C: Earthquake wave propagation velocity (in the absence of accurate information, the value2 km/seccan be used for the S-wave.)

t_u: The maximum friction force per unit length of the pipe-soil contact surface according to Eq. 13.12

λ_c: The apparent wavelength of the earthquake on the surface of the earth (in the absence of accurate information, the value 1 km can be used).

A_P: cross-sectional area of the pipe

$$t_u = \pi D c \alpha_s + \frac{\pi D}{2} \bar{\gamma} H_s (1 + k_0) \tan \delta \qquad 13.12$$

where:

c: Soil cohesion

α_s: The dimensionless coefficient of soil and pipe cohesion, which is obtained from Eq. 13.13:

$$\alpha_s = 0.608 - 1.23c - \frac{0.274}{1 + 100c^2} + \frac{0.695}{1 + 1000c^3} \qquad 13.13$$

In this regard, the unit of soil cohesion is MPa.

$\bar{\gamma}$: Effective unit weight of soil

H_s: Distance from the ground surface to the center of the buried pipe

k_0: Coefficient of lateral pressure of the soil at rest

δ: The friction angle between the soil and the pipe, which can be assumed to be equal to $f \times \varphi$ where φ is the internal friction angle of the soil and f is the coefficient of friction between soil and pipe. Some suggested values for f are listed in the Table 13.4.

In segmental pipelines, the amount of deformation in the connection is controlled according to Eq. 13.9. This value can be obtained as the sum of displacements caused by the operating and earthquake loads Moreover, rotation at the connection shall be determined from Eq. 13.14:

$$\theta_{seismic} = 1.5 \frac{A_g}{C^2} L_0 \qquad 13.14$$

where:

A_g: The design maximum acceleration in each hazard category

L_0: Length of the pipe between two connections

The value of the allowable connection internal rotation is provided by the manufacturer.

13.3.2. Dynamic analysis

In the dynamic analysis of the pipeline, it is necessary to obtain the required parameters, including parameters of the nonlinear behavior of the soil, the propagation velocity of the waves in the soil and the pipe, and the dominant frequency of the surrounding medium along the pipe route based on reliable methods.

Table 13.4: Coefficient of friction between soil and pipe (f)

Pipe cover material	Coefficient of friction
Concrete	1
Coal Tar	0.9
Rough Steel	0.8
Smooth Steel	0.7
Fusion Bonded Epoxy	0.6
Polyethylene	0.6

13.3.3. Analysis for permanent displacement due to fault rupture

According to Chapter 3, it is necessary to investigate the potential of surface rupture in the faults within the target region. If there is a surface fault rupture potential and the pipline passing through the fault is unavoidable, it is necessary to determine the following:

1- Type of fault (strike-slip, normal, reverse, compound)
2- The degree of fault activity and its seismicity rate
3- The width and extent of the ruptured area of the fault
4- The direction of slip relative to the pipeline route
5- The range of vertical or horizontal displacements, proportional to the level of hazard.

After determining the above, it is strongly recommended to pass the pipeline through the fault in such a way that the movement of the fault does not cause compression in the line. The method of estimating the possible location of the fault is mentioned in Section 3.13.

13.3.3.1. Displacement of the pipeline at intersection with the strike-slip and normal faults

The fault movement components along the pipeline, δ_{fax}, and perpendicular to the pipeline, δ_{ftr}, for a strike-slip fault can be calculated from Eq. 13.15:

$$\delta_{fax} = \delta_{fs}\cos\beta \quad 13.15.a$$

$$\delta_{ftr} = \delta_{fs}\sin\beta \quad 13.15.b$$

where:

β: Angle of intersection of the pipeline with the fault (Fig. 13.2)

The fault movement components along the pipeline, δ_{fax}, and perpendicular to the pipeline, δ_{ftr}, and in vertical direction, δ_{fvt}, for a normal fault can be calculated from Eq. 13.16:

$$\delta_{fax} = \delta_{fn}\cos\psi.\sin\beta \quad 13.16.a$$

$$\delta_{ftr} = \delta_{fn}\cos\psi.\cos\beta \quad 13.16.b$$

$$\delta_{fvt} = \delta_{fn}\sin\psi \quad 13.16.c$$

where:

β: Angle of intersection of the pipeline with the fault (Figs. 13.2 and 13.3)

ψ: Angle between the rupture surface of the normal fault and the horizontal surface (Fig. 13.3)

13.3.3.2. Displacement of the pipeline at the intersection with the reverse fault

Displacement components in a reverse fault can be obtained from normal fault relationships by including the negative sign for the angle of the fault's rupture surface.

13.3.3.3. Displacement of the pipeline at the intersection with the compound fault

In the fault with combined behavior, strike-slip and normal (or reverse) displacements in the longitudinal, transverse and vertical directions imposed on the pipeline axis are added together, considering the corresponding directions.

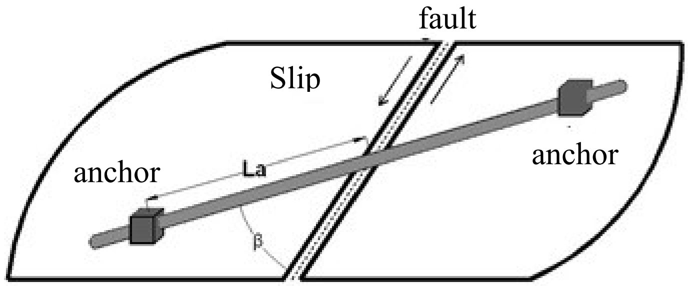


Fig. 13.2: Pipeline crossing strike-slip fault

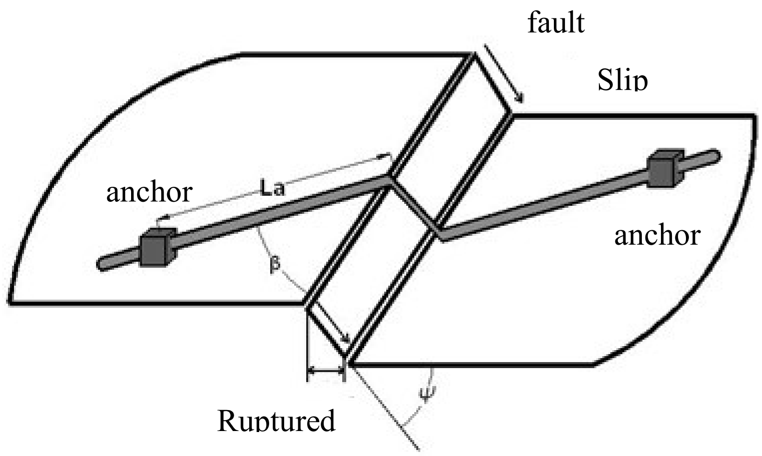


Fig. 13.3: Pipeline crossing normal fault

13.3.3.4. Design displacement of the fault rupture

The design displacement of the fault can be obtained by multiplying the importance factor (Table 13.1) by the probable displacement of the fault according to Eq. 13.17:

$$\delta_{fax-design} = \delta_{fax} I_L \qquad \text{In the pipeline direction} \qquad 13.17.a$$

$$\delta_{ftr-design} = \delta_{ftr} I_L \qquad \text{In the transverse direction} \qquad 13.17.b$$

$$\delta_{fvt-design} = \delta_{fvt} I_L \qquad \text{In the vertical direction} \qquad 13.17.c$$

13.3.3.5. Pipeline strain at the intersection with fault

The average seismic strain of the pipe at the intersection with the strike-slip fault is obtained from Eq. 13.18:

$$\varepsilon_{seismic} = 2\left[\frac{\delta_{fax-design}}{2L_a} + \frac{1}{2}\left(\frac{\delta_{ftr-design}}{2L_a}\right)^2\right] \qquad 13.18$$

where:

L_a: The unrestrained length of the pipe in the area of intersection with the fault, which can be chosen as the lowest of the following two values:

a) If there are no restrictions such as bends, joints, etc. in the area of intersection with the fault, the effective unrestrained length of the pipeline can be determined from Eq. 13.19:

$$L_a = \frac{\pi D t_p E_p \varepsilon_y}{t_u} \qquad 13.19$$

where:

ε_y: Material yield strain

b) If there is any restraint (such as a bend, a change in the thickness of the soil on the pipe, etc.), that place shall be assumed as a restraint point. The length of the pipeline from that point to the fault line is considered as the effective unrestrained length.

The average seismic strain of the pipe at the intersection with the fault, after combining with the operating strain, must meet the condition of the allowable strain (Table 13.3).

Coefficient 2 in Eq. 13.18 is considered as a confidence coefficient due to the uncertainty in this relation. The mentioned relationship is only an approximation for initial estimation. For a more accurate evaluation, it is recommended to use more suitable models with nonlinear analysis.

In order to calculate the average seismic strain of the pipe at the intersection with dip-slip faults, it is suggested to use numerical analysis, such as the finite element method, by applying the displacements obtained from Eq. 13.17 on both sides of the fault.

In segmental pipes, it is assumed that the displacement of the fault is absorbed in the pipe connections on both sides of the fault. The amount of design deformation in pipe connections shall be calculated from Eq. 13.20 and be compared with the values of the allowable displacement provided by the manufacturer.

$$\Delta_{seismic} = \delta_{fax} I_L \qquad 13.20$$

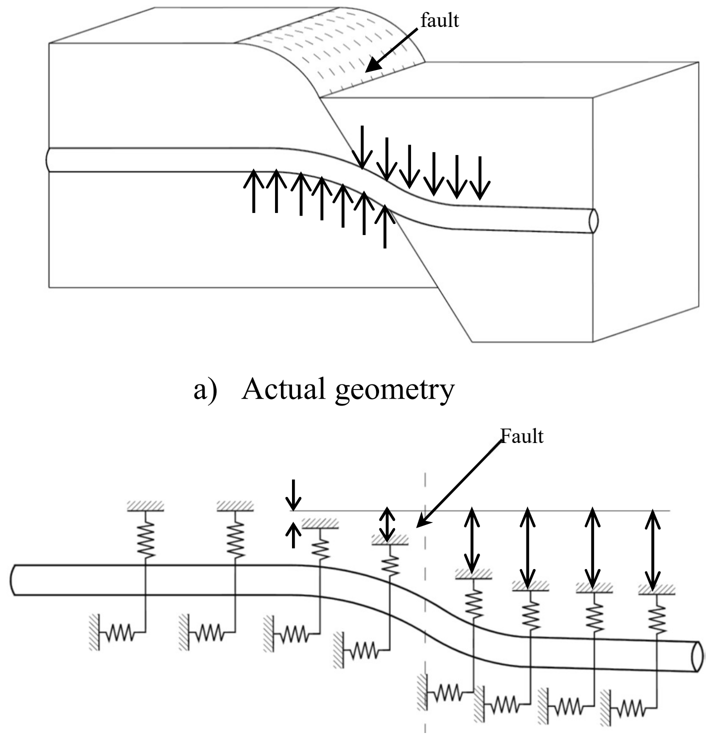
13.3.3.6. Finite element method

In this method, with a suitable software, it is possible to analyze the pipeline by using nonlinear behavior models of the soil and pipe materials and taking into account large deformations (i.e., nonlinear geometry). By applying displacement to any target point of the soil-pipe system as an input, the effect of fault displacement can be considered. The use of such a software requires sufficient knowledge of the nonlinear behavior of soil and structure, as well as the practical aspects of the finite element method.

In order to achieve better results, a sufficient length of the pipe shall be considered on both sides of the fault. To model soil behavior, with the help of reliable references, one can use three-dimensional elements or equivalent nonlinear springs (Figs. 13.4 and 13.5). In Section 13.3.7, soil modeling with equivalent springs is described.

13.3.4. Analysis of the effects of landslides

In the procedure of selecting the route of the pipelines, it is recommended to avoid passage through landslide-prone areas. Figure 13.6 shows the model of the pipe under the effect of landslide. As shown in the figure, the load caused by sliding can be assumed to be uniform in the direction of sliding, and the intensity of this uniform load can be calculated based on the proper equations that take the interaction between the soil and the pipe into account. If the sliding direction is not perpendicular to the pipeline direction, in addition to the lateral component of the soil force, the axial component shall also be considered. The length of the restrained parts of the pipe on both sides of the prone area shall be determined by trial and error. If on one or both sides of the sliding area the pipe is restrained by the support and the distance of the restriction from the region of sliding is not more than the specified length, the effect of the restriction shall also be included in the analysis. Usually, the pipe with an unreinforced cross-section goes beyond the elastic range due to soil sliding. For this purpose, the analysis of the ductile pipe can be performed by the methods appropriate to include plastic hinges in the beams.



a) Actual geometry

b) Simplified modeling

Fig. 13.4: modeling of the pipeline under the effect of fault movement, a) acual geometry and b) simplified modeling

The values of the permanent design displacement resulting from the landslide can be calculated based on the return period of the pipeline hazard category (Section 2.13). In addition, those values can be calculated based on the earthquake level (i.e.,the second risk level according to the criteria of Chapter 3) and the results can be multiplied by the importance factor according to

Table 13.1. The method of determining the strain resulting from landslide in the pipe is presented in Section 13.3.6.

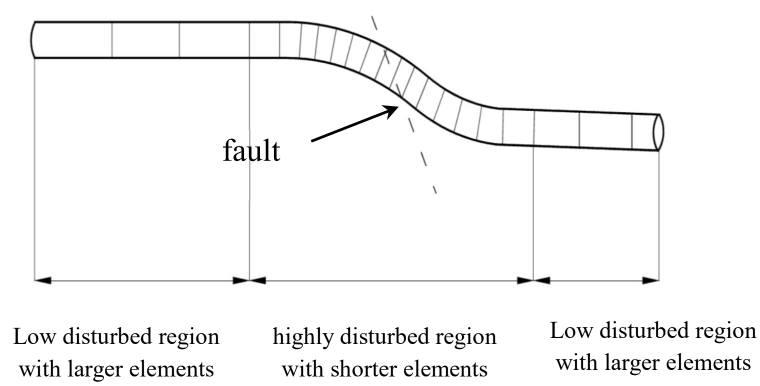


Figure 13.5: Modeling of the pipeline by the finite element method in the crossing-fault region

13.3.5. Analysis of the effects of liquefaction

In selecting the route of pipelines, it is recommended to avoid passing through areas prone to liquefaction. The analysis of the pipeline in areas prone to liquefaction can be done by assuming the pipe as a beam on an elastic medium. In this method, the stiffness of the pipe's surrounding medium can be calculated based on proper equations that take the interaction between the soil and the pipe into account. It is possible to conservatively assume the stiffness of the soil in the zone prone to liquefaction to be zero. The length of the parts on both sides of the prone area, required for analysis, is determined by trial and error. As a first guess, one can start with a length of about 0.25 of the length of the zone prone to liquefaction. The final values of permanent displacement, used in the design, resulting from buoyancy caused by liquefaction can be initially calculated based on the suggestions presented in Chapter 5 and then according to the importance coefficients related to the pipeline risk category (Table 13.2).

When liquefaction occurs in the soil around the pipe, buoyancy forces are applied to the pipeline, against which the pipe must be properly restrained. The net upward force per unit length of the pipeline can be calculated from Eq. 13.21:

$$F_b = W_s - \left[W_p + W_c + \left(P_v - \gamma_w h_w\right)D\right] \qquad 13.21$$

where:

W_s: The total weight of the soil is equivalent to the volume occupied by the pipe per unit length

W_p: Pipe weight per unit length

W_c: Weight of pipe contents per unit length

P_v: Vertical earth pressure according to Eq. 13.22

γ_w: Unit weight of water

For simplicity, the cohesion of the soil to the pipe wall has been ignored in the above calculations.

$$P_v = \gamma_w h_w + \gamma_d h_{sp} - 0.33\gamma_d h_w \qquad 13.22$$

where:

γ_w: Unit weight of dry soil

h_{sp}: Height of the soil over the pipe

The bending stress, caused by buoyancy in a relatively short piece of continuous pipeline in units of Pa, can be calculated from Eq. 13.23. In this equation, F_b is in units of N/m:

$$\sigma_{bf} = \frac{F_b L_b^2}{10 Z_e} \qquad 13.23$$

where:

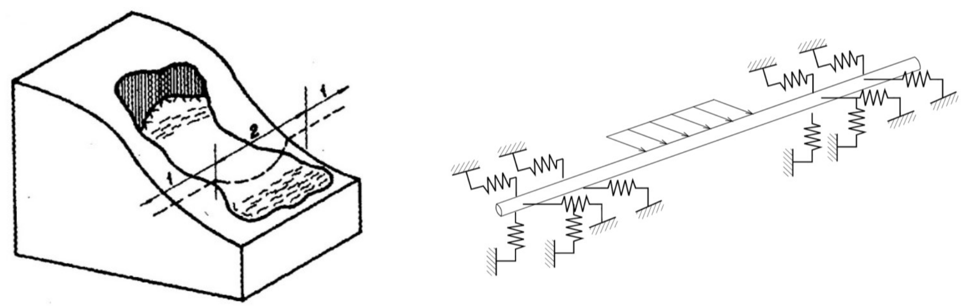
L_b: Length of the pipe in the buoyancy region (m)

Z_e: The elastic section modulus of the pipe (m^3)

The maximum strain, related to the stress caused by bending, can be obtained from Eq. 13.1. After combining with the operating strain according to Eq. 13.8, the maximum strain should be compared with the allowable strain values in Table 13.3.

For longer pieces of pipeline under buoyancy force, resistance against the upward force can be estimated based on the both beam and cable actions in the pipe.

The analysis of the pipeline under the effect of ground permanent displacements due to the lateral spread of the ground caused by liquefaction is given in Section 13.3.6.



a) Actual slide geometry b) Simplified modeling

Fig. 13.6: Modeling of the pipeline under the effect of landslides, a) actual slide geometry and b) simplified modeling

Response of the segmented pipes under the effect of forces caused by the liquefaction phenomenon shall be determined according to the location of the connections and by using the equilibrium equations of forces and bending moments. In these analyses, pipe connections can be considered as moment-free joints and consequently their displacements and rotations can be calculated. Internal displacements of connections can be considered as controlling responses, which shall be less than the allowable limits attributed to the pipe connections.

13.3.6. Analysis for permanent displacements induced by liquefaction and landslides

This section deals with permanent ground displacements due to lateral spread caused by liquefaction and landslides. Part of the pipe prone to lateral spread is ideally supposed to possess an oval shape with the larger diameter of L_z and shorter diameter of W_z. With the help of geotechnical studies, it is necessary to estimate the lateral spread region dimensions along with the maximum longitudinal mass movement, δ^l, known as the permanent displacement. Generally, it is difficult to estimate the appropriate amount of

δ^l and dimensions L_z and W_z. Then, a range of values is usually given for the above parameters and based on that, seismic design controls are carried out. The permanent displacement used in the pipe design in the longitudinal direction, can be obtained from Eq. 13.24:

$$\delta^l_{design} = \delta^l I_L \qquad 13.24$$

In general, for buried pipelines that are affected by the permanent longitudinal displacement of the ground and supposing that this displacement is uniform (that is, the longitudinal movements of the ground are uniform in the entire region affected by the permanent displacement), the following two conditions are assumed:

State 1. The ground displacement amount, δ^l_{design}, is large and the pipe strain can be calculated by the length of the permanent displacement region. In this case, the maximum longitudinal axial strain of the pipe in tension and compression is calculated from Eq. 13.25:

$$\varepsilon_a = \frac{t_u L_z}{2\pi D t_p E_p}\left[1 + \frac{n}{1+r}\left(\frac{t_u L_z}{2\pi D t_p \sigma_y}\right)^r\right] \qquad 13.25$$

where:

n, r: Same parameters introduced in after Eq. 13.1

t_u: The maximum friction force per unit length of the pipe at the contact surface with the soil according to Eq. 13.12

State 2. The length of the region of permanent deformation, L_z, is large and the strain of the pipe is obtained based on the amount of soil deformation. In this case, the maximum strain of the pipe in tension and pressure is calculated from Eq. 13.26:

$$\varepsilon_a = \frac{t_u L_e}{\pi D t_p E_p}\left[1 + \frac{n}{1+r}\left(\frac{t_u L_e}{\pi D t_p \sigma_y}\right)^r\right] \qquad 13.26$$

where:

L_e: Effective length of the pipeline under friction force, which can be calculated from Eq. 13.27:

$$\delta^l_{design} = \frac{t_u L_e^2}{\pi D t_p E_p}\left[1+\left(\frac{2}{2+r}\right)\left(\frac{n}{1+r}\right)\left(\frac{t_u L_e}{\pi D t_p \sigma_y}\right)^r\right] \qquad 13.27$$

The seismic strain of the pipe, $\varepsilon_{\text{seismic}}$, is assumed to be equal to the minimum amount of strains obtained from Eqs. 13.25 and 13.26 for permanent longitudinal displacement of the ground.

To reduce the effects of permanent displacement in continuous pipelines, it is recommended to use expansion joints on both sides of the permanent displacement area.

Like as the procedure described to account for the permanent longitudinal displacement, the range of permanent lateral displacement of the ground, δ^t, along with specific ranges of L_z and W_z shall be tried by conducting a proper seismic design check.

The permanent displacement for design in the lateral direction, can be calculated from Eq. 13.28:

$$\delta^t_{design} = \delta^t I_L \qquad 13.28$$

where:

δ^t: Maximum permanent lateral displacement of the ground

The maximum bending strain in the pipe can be considered conservatively equal to the lowest value obtained from Eq. 13.29:

$$\varepsilon_b = \pm\frac{\pi D \delta^t_{design}}{W_z^2} \qquad 13.29.a$$

$$\varepsilon_b = \pm\frac{P_u W_z^2}{3\pi E_p t_p D^2} \qquad 13.29.b$$

where:

P_u: Maximum lateral resistance of the soil per unit length of the pipe according to Eq. 13.30.

The maximum strain calculated above shall be assumed as the design strain of the pipe, $\varepsilon_{\text{seismic}}$.

$$P_u = S_u N_{ch} D \qquad \text{for clay} \qquad 13.30.a$$

$$P_u = \bar{\gamma} H_s N_{qh} D \qquad \text{for sand} \qquad 13.30.b$$

where:
S_u: Undrained shear strength of soil
N_{ch}: Coefficient of lateral horizontal bearing capacity depending on cohesion according to Eq. 13.31.
N_{qh}: Coefficient of lateral horizontal bearing capacity depending on the internal friction according to Eq. 13.32.

$$N_{ch} = A_1 + A_2 x + \frac{A_3}{(x+1)^2} + \frac{A_4}{(x+1)^3} \le 9 \qquad 13.31$$

$$N_{qh} = A_1 + A_2 x + A_3 x^2 + A_4 x^4 + A_5 x^5 \qquad 13.32$$

in which coefficients A_1 to A_5 are determined from Table 13.5 and $x = H_s / D$. The simple calculation relationships given in the above section can be used to calculate the pipe strain in the initial design. It is recommended to use the finite element analysis method in the design of important pipelines considering the nonlinear behavior of the pipe and soil.
In segmental pipes, the seismic deformation of the pipe is considered equal to the maximum opening in the pipe joint due to the permanent deformation of the ground, and its value is determined from Eq. 13.33:

$$\Delta_{seismic} = \delta^{l}_{design} \qquad 13.33$$

In this equation, the permanent design displacement is considered in the longitudinal direction of the pipe. The deformation of the joint must be less than the allowable deformation provided by the manufacturer.
The adequate number of joints in segmental pipes depends on the permanent ground displacement. In these pipes, at least one joint at the beginning and one joint at the end of the region of permanent earth displacement shall be used. In areas with a low permanent ground displacement, pipes with push-on joints (joints without mechanical stops) can be used.

Table 13.5: The coefficients of Eqs. 13.31 and 13.32 according to the internal friction angle of the soil

Coefficients	ϕ (degrees)	A_1	A_2	A_3	A_4	A_5
N_{ch}	0	6.752	0.065	-11.063	7.119	–
N_{qh}	20	2.399	0.439	-0.03	1.059×10^{-3}	-1.754×10^{-5}
	25	3.332	0.839	-0.09	5.606×10^{-3}	-1.319×10^{-4}
	30	4.565	1.234	-0.089	4.275×10^{-3}	-9.159×10^{-5}
	35	6.816	2.019	-0.146	7.651×10^{-3}	-1.683×10^{-4}
	40	10.959	1.783	0.045	-5.425×10^{-3}	-1.153×10^{-4}
	45	17.658	3.309	0.048	-6.443×10^{-3}	-1.299×10^{-4}

In areas with a large permanent ground displacement, a larger number of chain joints can be used in the vicinity of the head and toe of displacement prone area. Chain joints shall be installed in such a way that at least three joints are located outside this area and at its border. The design displacement of each joint shall be calculated from Eq. 13.34:

$$\Delta_{seismic} = \left[\frac{\delta^{l}_{design}}{L/2}\right] L_0 \qquad 13.34$$

where:

L: The length of the area with permanent ground displacement

The connection mechanical stops used in chain joints must be designed in such a way that they can withstand the maximum design friction force, F_{stop}, according to Eq. 13.35.

$$F_{stop} = 2\left[\frac{n_c + 1}{2}\right] L_0 t_u \qquad 13.35$$

where:

n_c: Number of chain connections at the head or toe of the moving mass of the soil, which by expansion, absorb the permanent movement of the ground.

In no case, it is permissible to take the design friction force, F_{stop}, greater than the yield strength of the pipe.

The design deformation of the segmental pipes at joints, $\Delta_{seismic}$, for the permanent displacement of the ground in the transverse direction can be calculated from the sum of the axial extension and extension due to the rotation of the joints. The amount design deformation can be calculated from Eq. 13.36:

$$\Delta_{seismic} = \frac{\pi^2 L_0 \delta^{t\,2}_{design}}{W^2}\left[\frac{2D}{\delta^t_{design}}\right] \qquad 0.268 \le D/\delta^t_{design} \le 3.73 \qquad 13.36.a$$

$$\Delta_{seismic} = \frac{\pi^2 L_0 \delta^{t\,2}_{design}}{W^2}\left[1+\left(\frac{D}{\delta^t_{design}}\right)^2\right] \qquad \text{Other values of } D/\delta^t_{design} \qquad 13.36.b$$

Deformation of the joint shall be less than the amount of its allowable limit provided by the manufacturer.

13.3.7. Modeling of buried pipeline using equivalent soil spring

For soil modeling and pipe-soil interaction, the pipe can be modeled in a semi-infinite environment of soil directly using numerical methods. Another method of modeling the effect of soil flexibility, which is shown in Figure 13.7, is to use the theory of beam on Winkler inelastic springs (longitudinal, transverse, and vertical springs). With known geotechnical parameters, the characteristics of these springs can be calculated. The nonlinear behavior of these springs is modeled according to Fig. 13.8.

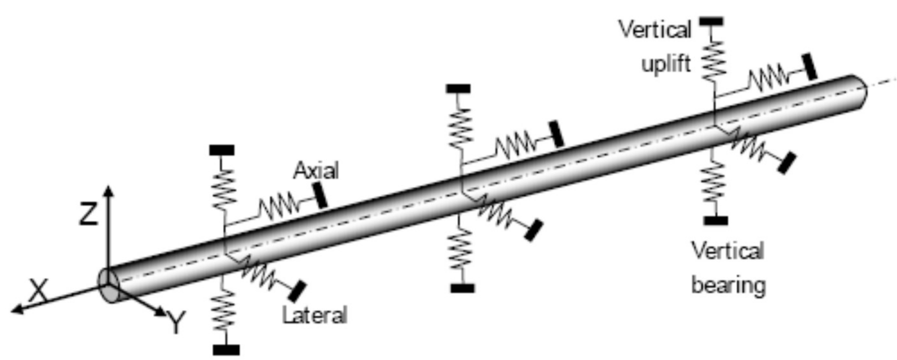


Figure 13.7: Soil substitute springs in axial, lateral and vertical directions

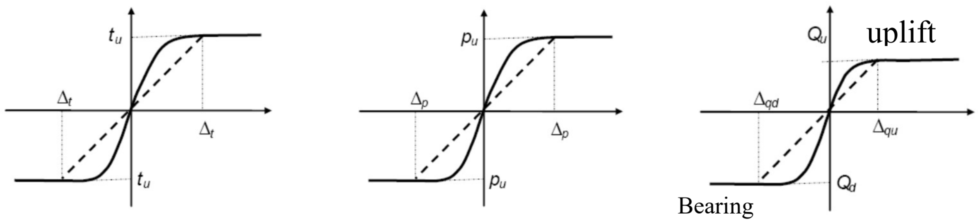


Figure 13.8: Soil substitute springs force-displacement curves, a) axial, b) lateral and c) vertical directions.

Nonlinear static analysis can be used to investigate the behavior of the pipe due to permanent displacement. The desired ground deformation is also applied to the end of the springs (the spring nodes that are not attached to the pipe).

To investigate the behavior of the pipe against wave propagation, the timehistory analysis may be used, by applying the ground motion to the end of the springs and performing dynamic analysis.

The specifications of the springs are determined according to the construction method using properties of the soil around the pipe and rest of the soil. The mentioned equations are for buried pipelines with a burial depth between 0.5 and 2 m. In the case of larger burial depths, modeling shall be done by considering more precise models preferably considering a direct model with 2D or 3D elements for the soil environment. The values shown in Fig. 13.8 are determined according to the soil and pipe characteristics as follows.

13.3.7.1. Axial springs (springs oriented along the pipe)

The specifications of the axial springs are determined according to the specifications of the soil. The maximum axial resistance of the soil per unit length of the pipe is obtained from Eq. 13.12. The yield displacement of the soil spring along the axis of the pipeline depends on the type of soil and can be determined from Table 13.6.

Table 13.6: Yield displacement of the soil spring

Yield displacement, Δ_s (mm)	Soil Type
3	Dense sand
5	Loose sand
8	Stiff clay
10	Soft clay

13.3.7.2. Lateral springs

haracteristics of the lateral springs are determined according to the characteristics of the main body of soil. The maximum lateral resistance of the soil per unit of pipe length is obtained from Eq. 13.37.

$$P_u = N_{ch} cD + N_{qh} \bar{\gamma} H_s D \qquad 13.37$$

where the values of N_{ch} and N_{qh} are determined by Eqs. 13.31 and 13.32.
The maximum value of transverse displacement is determined from Eq. 13.38:

$$\Delta_p = 0.04(H_s + D/2) \leq (0.1 \sim 0.15)D \qquad 13.38$$

13.3.7.3. Vertical springs

Behavior of the vertical springs includes two different parts, one of which is related to the uplift load and the other to the bearing load. For uplift, the specifications of backfill soil are used, and for bearing load, the specifications of the native soil are considered.
The characteristics of soil behavior in the uplift part can be determined from Eqs. 13.39 to 13.42:

$$Q_u = N_{cv} cD + N_{qv} \bar{\gamma} H_s D \qquad 13.39$$

where:
N_{cv}: Load factor related to cohesion at break according to Eq. 13.40
N_{qv}: Load factor related to internal friction at break according to Eq. 13.41

for values $\frac{H_s}{D} \leq 10$ $\quad N_{cv} = 2\frac{H_s}{D} \leq 10$ 13.40

$$N_{qv} = \frac{\phi H_s}{44D} \leq N_q \tag{13.41}$$

N_q: Soil bearing coefficient which can be determined from Eq. 13.42 or Fig. 13.9:

$$N_q = \exp(\pi \tan\phi) \times \tan^2(45 + \phi/2) \tag{13.42}$$

The value of threshold displacement, Δ_{Qu}, at the force, Q_u, is obtained from Eq. 13.43:

$$\Delta_{Qu} = (0.01\sim0.02)H_S \leq 0.1D \quad \text{for sand} \tag{13.43.a}$$
$$\Delta_{Qu} = (0.1\sim0.2)H_S \leq 0.2D \quad \text{for clay} \tag{13.43.b}$$

The characteristics of soil behavior in the vertical bearing can be determined from Eq. 13.44:

$$Q_d = N_c cD + N_q \bar{\gamma} H_s D + N_\gamma \gamma \frac{D^2}{2} \tag{13.44}$$

where:
N_c and N_γ: Soil load coefficients determined from Eqs. 13.45 and 13.46 as well as Fig. 13.9.

$$N_c = [\cot(\phi + 0.001)]\left\{\exp[\pi \tan(\phi + 0.001)]\tan^2\left(45 + \frac{\phi + 0.001}{2}\right) - 1\right\} \tag{13.45}$$

$$N_y = \exp(0.18\phi - 2.5) \tag{13.46}$$

In the cases where the internal friction angle of the soil is equal to zero, the minimum value is used in the above equations. The value of the displacement corresponding to the maximum bearing load, Δ_{Qu}, is equal to $0.1D$ for granular soils and $0.2D$ for cohesive soils.

13.4. Above-ground pipelines

Above-ground pipelines usually have many sliding supports, in addition to the anchor points. In the seismic analysis of the pipelines, it is necessary to accurately model the friction between the pipe and the support and other effective factors.

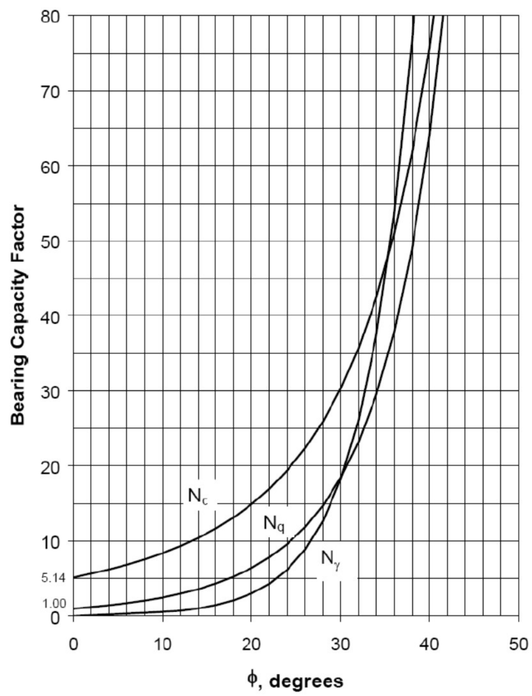


Fig. 13.9: Bearing capacity factors of soils for different soil friction values

The pipeline has expansion joints or special bends, depending on the diameter of the pipe, to withstand deformations caused by temperature changes, which makes the geometry of the structure more complicated. In the seismic design of the above-ground pipeline, it is recommended to provide sliding supports with minimal friction. These supports must have a sufficient width or a suitable limit in the vertical direction along the pipe such that the pipe does not fall from the support due to the lateral movement caused by earthquake. In Section 13.4.6, the quantified design criteria for above-ground pipelines are presented.

13.4.1. The equivalent seismic load method

In the analysis of the above-ground pipeline by the equivalent seismic load method, the maximum relative displacement between adjacent restraining points can be estimated with a site response analysis, and the pipeline can be analyzed as a multi-span beam under the effect of the maximum relative displacement. If the seismic waves incident direction with the pipeline can be estimated, the longitudinal and transverse movements of the restraining points can be used as the basis for calculations. Otherwise, it is necessary to consider the worst case among all combinations of support movement, which creates the uppermost stress in the pipe. This can be done with the help of repeating calculations for different wave incident scenarios. In this method, pipeline analysis in the area of bends and intersections requires special attention.

13.4.2. Dynamic analysis

Dynamic analysis of the above-ground pipelines is similar to that of the buried ones, with the difference that supports or supporting structures can be modeled as equivalent springs or with more detailed structural components. Seismic design can be done based on one of the two methods of combining seismic inputs from different directions:

1. The square root of the sum of the squares of each of the two horizontal components and the vertical component, whichever larger.
2. The square root of the sum of the squares of both horizontal and vertical components at the same time.

Other cases are the same as Section 13.3.2.

13.4.3. Analysis under the effect of fault rupture

To analyze the above-ground pipeline under the effect of fault rupture, the pipe can be considered as a multi-span beam with non-linear spring supports in three orthogonal directions at the location of support points. The number of support points considered in this analysis can be obtained by trial and error. For this purpose, the number of spans inside the fault area (at least one span) and two spans on both sides of the fault area are usually considered first. If the forces resulting from the movement of the fault in the two end supports are such that their springs remain in the elastic state, the analysis has sufficient accuracy. Otherwise, one shall add an opening to the model on each side and repeat the calculations. To estimate the displacement of the fault and the amount of pipe deformation, it is possible to do the same as Section 13.3.3.

13.4.4. Analysis under the effect of landslides

To analyze the above-ground pipeline due to landslides, the method of Section 13.3.4 can be used, with the difference that in this case a multi-span beam with inelastic spring supports in three orthogonal directions at the location of the restraining points is considered.

13.4.5. Analysis under the influence of liquefaction

For this analysis, the method of Section 13.3.5 can be used. To determine the number of openings in the modeling, the same procedure as Section 13.4.3 can be followed.

13.4.6. Seismic design of above-ground pipeline

The seismic design method and the sections used in this method are presented in Table 13.7. The seismic design method depends on the classification of the pipe system (as risk categories I, II or III) besides the intensity of the seismic input and the diameter of the pipe. In all cases, the design can be done by performing the analysis according to Sections 13.4.6.2 and 13.4.6.3.

13.4.6.1. Pipe design by rule

In the cases allowed in Table 13.7, the seismic resistance of the pipe system can be ensured by providing lateral and vertical seismic supports with a maximum distance according to Equation 13.47:

$$L_{\max} = \min \begin{cases} 1.94 L_T / S_{DS}^{0.25} \\ 0.211 L_T (\sigma_{yo} / S_{DS})^{0.5} \end{cases} \qquad 13.47$$

where:

L_{max}: The maximum permissible span length of the pipe between two lateral and vertical seismic restraints

L_T: The recommended value for the distance between gravity supports (Table 8.13)
S_{DS}: Spectral acceleration parameter (in g) in the design earthquake at short periods (0.2 seconds)
σ_{yo}: Yield stress of pipe material at operating temperature in MPa

Table 13.7: Considerations for the design of above-ground pipe and related parts (D_n nominal diameter in mm).

S_{DS} (g)	Risk category III			Risk category I, II	
	$D_n \leq 50$	$50 < D_n < 150$	$D_n \geq 150$	$D_n \leq 50$	$D_n \geq 50$
<0.3	13.4.6.7	13.4.6.7	13.4.6.5 13.4.6.6 13.4.6.7	13.4.6.1 13.4.6.4 13.4.6.5 13.4.6.6 13.4.6.7	13.4.6.1 13.4.6.4 13.4.6.5 13.4.6.6 13.4.6.7
>0.3	13.4.6.7	13.4.6.1 13.4.6.4 13.4.6.5 13.4.6.6 13.4.6.7	13.4.6.1 13.4.6.4 13.4.6.5 13.4.6.6 13.4.6.7	13.4.6.1 13.4.6.4 13.4.6.5 13.4.6.6 13.4.6.7	13.4.6.2 (or 13.4.6.3) 13.4.6.4 13.4.6.5 13.4.6.6 13.4.6.7

In addition, in straight pipes with a length greater than three times the values presented in Table 13.8, a longitudinal restraint shall also be installed. Distance between the horizontal and vertical restraints in pipes that have heavy components (with the total weight of the components 10% more than the weight of the pipeline span) shall be reduced. Equation 13.47 is presented based on limiting the deformation of the middle of the pipe opening to 50 mm and its maximum stress to $0.5\sigma_{yo}$.

Table 13.8: Recommended maximum distance between the pipe supports, l_T

D_n	(in)	1	2	3	4	6	8	12	16	20	24
	(mm)	25	50	75	100	150	200	300	400	500	600
L_T (cm)	Water service	200	300	350	430	520	580	700	820	900	980
	Steam, Gas or Air Service	270	400	460	520	210	640	900	1070	1200	1280

13.4.6.2. Pipe design by the analytical method

In the analytical method, the longitudinal elastic stress calculated in the pipe due to the design earthquake (calculated by the static or dynamic method) must satisfy the conditions of Eq. 13.48:

$$i\sqrt{M_i^2+M_a^2}\Big/Z_e < S_s \tag{13.48}$$

where:
i: stress increase coefficient (based on the pipe design standard such as ASME B31)
M_i: Net moment due to inertial forces
M_a: Net moment due to relative displacement of supports
S_s: Allowable seismic stress at the temperatures of 30 to 40 degrees, which is equal to 110 MPa for carbon and low alloy steel and 130 MPa for austenitic stainless steel.
Therefore, the resultant moment at a point is the square root of sum of the squares of the three moment components (in-plane bending, out-of-plane bending, and torsion) at that point.

13.4.6.3. Alternative design methods

In cases where the conditions of Eq. 13.48 are not satisfied, the pipe system can be designed by more accurate analyses considering the effects of fatigue and inelastic deformations or the ultimate limit analysis.

13.4.6.4. Mechanical connections

In the pipes belonging to the risk categories I and II, displacement and rotation as well as internal forces in mechanical connections must remain within the range defined by the manufacturer.

13.4.6.5. Seismic restraint

Seismic restraint is installed to prevent the pipe from falling from the support. Seismic forces generated in seismic restraints and their connection to the structure and foundation must be calculated by performing static or dynamic analysis. In pipes with a diameter of up to 50 mm, it is allowed to consider the gap between the pipe and the restrain stop equal to the nominal diameter of the pipe, and in larger pipes, a gap equal to 50 mm is allowed. If this gap is included, an impact factor equal to 2 shall be applied to the earthquake force obtained by assuming absence of the gap.

13.4.6.6. Pipeline components

Seismic forces shall be considered as a part of the pipe design procedure along with the operating forces that are introduced at the connection points. The procedure for calculating stresses under combinations of loads including seismic loads for flanges and control valves, is according toSections 16.6.2 and 16.6.3.

13.4.6.7. Interactions

The possible interaction of pipe systems shall be investigated. Significant interactions shall be identified and necessary measures to eliminate or reduce their effects shall be taken through analysis, testing or design modification.

13.5. Pipeline resting on the support structure

The pipeline support structure is designed according to the requirements of Chapter 7. Design of other parts of the pipeline is according to Section 13.4.

Chapter 14
Recommendations for Seismic Design of Fixed Steel Offshore Platforms

14.1. General

In general, fixed steel offshore platforms, are designed in two stages: analysis of in-service condition and analysis of the pre-service conditions (from construction to installation). In the in-service stage, the environmental and operational conditions of the platform installation location such as waves, water depth, soil, wind, currents, and ship impact, earthquake and fatigue are considered in the analysis and design. In the pre-service analysis stage, the different temporary loading conditions that an offshore platform is exposed to during the construction period in the onshore yard, and then during the transportation to the installation location, and finally through the various sequences of the installation are considered in the design. Some of the mentioned temporary loading conditions are the lifting of topside elevations and jacket rows roll-up in the yard, loadout, seafastening, transportation and installation in a specified location on the seabed, jacket launch, jacket floatation, jacket upending, on-bottom stability, pile driving, lifting and installing the topside.

Basically, the design life, configuration of the jacket members and connections and the chemical properties of the structural steel material used in the offshore platforms depend on the environmental conditions of the installation location, and pre-service conditions have no considerable criteria in this regard. Anyway, it is very important to mention that in many cases, especially in the Persian Gulf, the pre-service conditions are governed in the analysis and design.

14.1.1. Definitions

Fixed steel offshore platform: is defined as a platform extending above the water surface and supported at the seabed by means of piling, spread footing(s), or other means with the intended purpose of remaining stationary over an extended period.

Manned platform: refers to a platform that is continuously (or nearly continuously) occupied by persons accommodated and living.

Unmanned platform: refers to a platform that is not normally manned but may include an emergency shelter for temporary employed operators.

Operator: the person, firm, corporation, or other organization employed by the owners to conduct operations.

Dynamic loads: are the loads imposed on the platform due to response to an excitation of a cyclic nature or due to reacting to impulsive loads or impact. Excitation of a platform may be caused by waves, wind, earthquake, or machinery. Impact may be caused by a barge or boat berthing against the platform or by drilling operations.

14.1.2. Symbols

a_R	:	Slope of the seismic hazard curve
C1	:	Index of high consequence of failure
C2	:	Index of medium consequence of failure
C3	:	Index of low consequence of failure
C_c	:	Correction coefficient applied to the spectral acceleration to account for uncertainties not captured in a seismic hazard curve
C_r	:	Seismic reserve capacity factor
D	:	Scaling factor for damping/outer diameter
F_a	:	Site coefficient
F_v	:	Site coefficient
L-1	:	High exposure level
L-2	:	Medium exposure level
L-3	:	Low exposure level
N_{DLE}	:	Scale coefficient of ductility level earthquake
P	:	Probability of exceedance
P_e	:	Annual probability of exceedance
P_f	:	Target annual probability of failure
S1	:	Classification of life safety index for Manned-nonevacuated
S2	:	Classification of life safety index for Manned-evacuated
S3	:	Classification of life safety index for unmanned platform
S_a	:	Spectral acceleration
$S_{a,DLE}(T)$	:	Ductility level earthquake spectral acceleration associated with a single degree of freedom oscillator period T
$S_{a,map}(0.2)$	:	Bedrock spectral acceleration associated with a single degree of freedom oscillator at the period 0.2 second
$S_{a,map}(1.0)$:	:	Bedrock spectral acceleration associated with a single degree of freedom oscillator at the period 1.0 second
$S_{a,site}(T)$	:	Ground level earthquake spectral acceleration associated with a single degree of freedom oscillator period T

$S_{a,SLE}(T)$	:	Resistance level earthquake spectral acceleration associated with a single degree of freedom oscillator period T
SRC	:	Seismic Risk Category
T	:	Natural period of a simple, single degree of freedom oscillator
μ	:	Critical damping

14.1.3. Platforms Exposure Categories

In order to determine the seismic design criteria for an offshore platform according to the API RP 2A, Structures can be categorized by various levels of exposure to determine criteria for the design of new platforms that are appropriate for the intended service of the structure. The levels are determined by consideration of life safety and consequences of failure. Life safety considers the maximum anticipated environmental event that would be expected to occur while personnel
are on the platform. Consequences of failure should consider the anticipated losses to industry and government.

14.1.3.1. Categories for life safety

Categories for life safety are as follows :

S1: Manned – nonevacuated

S2: Manned - evacuated

S3: Unmanned

Also, Categories for consequences of failure are as follows::

C1: High consequence of failure

C2: Medium consequence of failure

C3: Low consequence of failure

The level to be used for platform categorization is the more restrictive level for either life safety or consequence of failure. Platform categorization may be revised over the life of the structure as a result of changes in factors affecting life safety or consequence of failure.

The exposure category should be determined using the matrix provided in Table 14.1.

Table 14.1. Exposure Category Matrix

Life Safety Category	Consequence Category		
	C1: High consequence of failure	C2: Medium consequence of failure	C3: Low consequence of failure
S1: Manned-non-evacuated	L-1	L-1	L-1
S2: Manned-evacuated	L-1	L-2	L-2
S3: Unmanned	L-1	L-2	L-3

14.1.3.1.1. Life safety

The determination of the applicable level for life safety should be based on the following descriptions :

S1 (manned-nonevacuated): The manned-nonevacuated category refers to a platform that is continuously (or nearly continuously) occupied by persons accommodated and living thereon and from which personnel evacuation prior to the design environmental event is either not intended or impractical.

S2 (manned-evacuated): The manned-evacuated category refers to a platform that is normally manned except during a forecast design environmental event.

S3 (unmanned): The unmanned category refers to a platform that is not normally manned or a platform that is not classified as either manned-nonevacuated or manned-evacuated. Platforms in this classification may include emergency shelters. However, platforms with permanent quarters are defined as manned and should be classified as manned-nonevacuated or as manned-evacuated as defined above. An occasionally manned platform may be categorized as unmanned only in certain conditions.

14.1.3.1.2. Consequence of failure

consequences of failure should include consideration of anticipated losses to the owner, the other operators, and the industry in general. The following descriptions of relevant factors serve as a basis for determining the appropriate level for consequence of failure.

C1 (High Consequence): The high consequence of failure category refers to major platforms and/or those platforms that have the potential for well flow of either oil or sour gas in the event of platform failure. In addition, it includes platforms where the shut-in of the oil or sour gas production is not planned or not practical prior to the occurrence of the design event (such as areas with high seismic activity). Platforms that support major oil transport lines and/or storage facilities for intermittent oil shipment are also considered to be in the high consequence category.

C2 (Medium Consequence): The medium consequence of failure category refers to platforms where production would be shut-in during the design event. All wells that could flow on their own in the event of platform failure shall contain fully functional, subsurface safety valves, which are manufactured and tested in accordance with the applicable API specifications. Oil storage is limited to process inventory and “surge” tanks for pipeline transfer.

C3 (Low Consequence): The low consequence of failure category refers to minimal platforms where production would be shut-in during the design event. All wells that could flow on their own in the event of platform failure shall contain fully functional, subsurface safety valves, which are manufactured and tested in accordance with applicable API specifications. These platforms may support production departing from the platform and low volume infield pipelines. Oil storage is limited to process inventory. platforms in this category includes caissons or small well protectors with no more than five well completions on or connected to the platform and no more than two conductors at the platform.

14.2. Basics of the seismic design

Earthquake load, should be imposed on the platform as a separate environmental loading condition (wind, wave, current, dynamic loads of machinery). Environmental loads, with the exception of earthquake load, should be combined in a manner consistent with the probability of their simultaneous occurrence during the loading condition being considered.

Design criteria include all operational requirements and environmental data that could affect the detailed design of the platform.

It should be noted that regardless of the details required for the perfect seismic design of platforms, seismic consideration and calculations should be performed in all stages of design (basic and detailed) based on the latest available information.

14.3. Reference standard for the design of structural members

Latest edition of API RP 2A recommendations can be used for the seismic design of structural members of an offshore platform.

14.4. Earthquakes

Considering that the plateau of Iran is located in seismically Active area, the seismic forces should be considered in platform design. Areas are considered seismically active on the basis of previous records of earthquake activity, both in frequency of occurrence and in magnitude. Seismic activity of an area for purposes of design of offshore structures is rated in terms of possible severity of damage to these structures. However, other seismic hazards shall also be considered in the design and should be addressed by a site-specific seismic hazard study.

Consideration of seismic events for seismically active regions shall include (not limited to) soil liquefaction, sea floor slide, fault movement, investigation of the characteristics of ground motions and the acceptable seismic risk for structures.

Platforms in shallow water that may be subjected to tsunamis shall be investigated for the effects of the resulting forces. Considering the fact that the propagation of seismic waves on the ground is different from the propagation of tsunami waves in the sea, the possibility of the simultaneous occurrence of both phenomena is negligible in offshore platforms.

14.5. Seismic Design Principles

14.5.1. General

This Regulations, presents guidelines for the design of a platform for earthquake ground motion including both strength and ductility requirements. Strength requirements are intended to provide a platform that is adequately sized for strength and stiffness to ensure no significant structural damage for the level of earthquake shaking that has a reasonable likelihood of not being exceeded during the life of the structure. Shutdown of production operations is tolerable and the structure should be inspected subsequent to this level of earthquake occurrence. The ductility requirements are intended to ensure that the platform has sufficient reserve capacity to prevent its collapse during rare intense earthquake motions, although structural damage may occur.

Two levels of earthquake resistance and ductility is defined as follows:

1. Strength level: Earthquake with a severity which the structure should sustain without major damage
2. Ductility level: Earthquake with a very low probability of occurrence during the life of the platform.

Characteristics of the strength and ductility levels, as well as the return period of these levels are determined based on the procedure presented in Section 14.10, which is possible to modify to achieve the desired goals of the clients or meet specific regional requirements.

It should be noted that the recommendations provided in this section and in the following paragraphs are intended to apply to the design of major steel framed structures. Only vibratory ground motion is addressed in this section. Other major concerns such as large soil deformations or instability should be resolved by special studies.

14.6. Preliminary Consideration

14.6.1. Evaluation of Seismic Activity

For seismically active areas it is intended that the intensity and characteristics of seismic ground used for design be determined by a site-specific study (See Chapter 3). Evaluation of the intensity and characteristics of ground motion should consider the active faults within the region, the type of faulting, the maximum magnitude of earthquake that can be generated by each fault, the regional seismic activity rate, the proximity of the site to the potential source faults, the attenuation of the ground motion between these faults and the platform site, and the soil conditions at the site.

Characteristics of ground motion are presented as a smoothed acceleration response spectrum or a set of earthquake acceleration records which are consistent with the design level. As the platforms must follow specific requirements for resistance and ductility, two levels of strength and ductility are considered for design.

14.6.2. Evaluation for Zones of Low Seismic Activity

In areas of low seismic activity, platform design would normally be controlled by storm or other environmental loading rather than earthquake. For areas defined in Table 14.2 as Site Seismic Zone 0, earthquake analysis may be omitted, since the design for environmental loading other than earthquake will provide sufficient resistance against potential effects from seismically active zones. The horizontal ground acceleration in the strength level earthquake, for the regions located in the Persian Gulf and the Sea of Oman, shall not be taken less than 0.060 g.

For areas where the strength level design horizontal ground acceleration is in the range of 0.05g to 0.10g, inclusive, all of the earthquake requirements, except those for deck appurtenances, may be considered satisfied if the strength requirements are met using the ground motion intensity and characteristics of the ductility level (rare, intense) earthquake in lieu of the strength level earthquake. In this event, the deck appurtenances should be designed for the strength level earthquake in accordance with 14.5.1, but the ductility requirements are waived, and tubular joints need be designed for allowable stresses specified in Provided by API RP 2A using the computed joint loads instead of the tensile load or compressive buckling load of the member.

14.7. Strength Requirements

14.7.1. Design Basis

The platform should be designed to resist the inertially induced loads produced by the strength level ground motion determined in accordance with 14.5.1 using dynamic analysis procedures such as response spectrum analysis or time history analysis.

In the Strength level earthquake, the design procedures are basically based on linear elastic methods of structural analysis in which, for example, nonlinear soil-structure interaction is considered to be linearly equivalent. However, if seismic isolation or passive energy dissipation devices are used, nonlinear time history methods should be used.

Structural Modeling

The mass used in the dynamic analysis should consist of the following:

- mass of the platform associated with gravity loading defined in 14.7.3,
- the mass of the fluids enclosed in the structure and the appurtenances, and
- the added mass. The added mass may be estimated as the mass of the displaced water for motion transverse to the longitudinal axis of the individual structural framing and appurtenances. For motions along the longitudinal axis of the structural framing and appurtenances, the added mass may be neglected.

The analytical model should include the three-dimensional distribution of platform stiffness and mass. Asymmetry in platform stiffness or mass distribution may lead to significant torsional response that should be considered.

In computing the dynamic response of braced, pile supported steel structures, a uniform modal damping ratio of 5 % of critical damping should be used for an elastic analysis. Where substantiating data exist, other damping ratios may be used.

Response Analysis

It is intended that the design response should be comparable for any analysis method used. When the response spectrum method is used and one design spectrum is applied equally in both horizontal directions, the complete quadratic combination (CQC) method may be used for combining modal responses and the square root of the sum of the squares (SRSS) may be used for combining the directional responses. If other methods are used for combining modal responses, such as the square root of the sum of the squares, care should be taken not to underestimate corner pile and leg loads. For the response spectrum method, as many modes should be considered as required for an adequate representation of the response. At least two modes having the highest overall response should be included for each of the three principal directions plus significant torsional modes.

Where the time history method is used, the design response should be calculated as the average of the maximum values for each of the time histories considered. If the time history analysis method is used, a minimum of 4 sets of time history records shall be used to capture the randomness in seismic motions. a scale factor of 1.05 shall be applied to the records if less than 7 sets of records are used.

If the time-history analysis method is used, at least 4 pairs of acceleration time histories should be used to consider the random nature of the seismic motion. In addition, if the number of records is less than 7 pairs, they should be multiplied by the scale factor of 1.05.

Earthquake loading should be combined with other simultaneous loadings such as gravity, buoyancy, and hydrostatic pressure. Gravity loading should include the platform dead weight (comprised of the weight of the structure, equipment, appurtenances), actual live loads and 75 percent of the maximum supply and storage loads Actual live load in earthquake analysis should be taken as proportion of live load considered in in-place analysis.

Response Assessment

In the calculation of member stresses, the stresses due to earthquake induced loading should be combined with those due to gravity, hydrostatic pressure, and buoyancy. For the strength requirement, the basic allowable stresses and may be increased by 70 percent. Pile-soil performance and pile design requirements should be determined on the basis of special studies. These studies should consider the design loadings of 14.7.3, installation procedures, earthquake effects on soil properties and characteristics of the soils as appropriate to the axial or lateral capacity algorithm being used. Both the stiffness and capacity of the pile foundation should be addressed in a compatible manner for calculating the axial and lateral response.

14.8. .Ductility Requirement

The intent of these requirements is to ensure that platforms to be located in seismically active areas have adequate reserve capacity to prevent collapse under a ductility level earthquake.

In most cases, it is not economical to design a structure such that the DLE event would be resisted without major nonlinear behavior. Therefore, the ductility design check allows non-linear methods of analysis.in this method, structural elements are allowed to behave plastically, foundation piles are allowed to reach axial capacity or develop plastic behavior. In fact, platform ductility is based upon a combination of reserve strength and ductility expressed by the seismic reserve capacity factor, Cr (the ratio between the ductility and strength level spectral accelerations), as defined in 14.10.4.2.

The Cr factor represents a structure's ability to sustain ground motions due to earthquakes beyond the strength level event. For a platform to use a Cr factor of 2.0 or greater, the structure-foundation system shall be in accordance with the following.

a) The structure has eight or more legs supported by piles.

b) The piles are founded in competent soils that are not susceptible to liquefaction during the strength and the ductility level events.

c) The legs of the structure, including any enclosed piles, meet the requirements of 14.7.4 using twice the design load during the strength level event.

d) The vertical framing transmitting shear forces between horizontal frames arranged such that shear between horizontal frames is carried by braces in both tension and compression. K-bracing should not be used. Where these conditions are not met, including areas such as the portal frame between the jacket and the deck, the structural components should be designed to meet the requirements of Section 14.7.4 using twice the strength level seismic loads.

e) Horizontal members are provided between all adjacent legs at horizontal framing levels in vertical frames. These horizontal members have sufficient strength in compression to support the redistribution of actions resulting from any buckling of adjacent diagonal braces.

f) The slenderness ratio (KL/r) of primary diagonal bracing in vertical frames is limited to no more than 80 and $(F_yD)/(E_t) \leq 0.069$.

g) All non-tubular members at connections in vertical frames are designed as compact sections in accordance with the AISC Specifications or designed to meet the requirements of 14.7.4 using twice the strength level seismic loads.

h) Joints for primary structural members in the structure are all sized to meet the minimum strength requirements given in 14.9.1. This requirement may be relaxed if joint strengths are verified by time history analyses simulating the ductility event.

Structure-foundation systems that do not meet the conditions listed above shall be analyzed to demonstrate their ability to withstand the ductility earthquake without collapsing. Models of the structural and soil elements should include their characteristic degradation of strength and stiffness under extreme load reversals and the interaction of axial forces and bending moments, hydrostatic pressures and local inertial forces, as appropriate. The P-delta effect of loads acting through elastic and inelastic deflections of the structure and foundation should be considered.

14.9 Additional Guidelines

14.9.1 Tubular Joints

Where the strength level design horizontal ground motion is 0.05g or greater, joints for primary structural members should be sized for either the tensile yield load or the compressive buckling load of the members framing into the joint, as appropriate for the ultimate behavior of the structure.

In cases that in accordance with section 14.6.2, strength requirements are checked for the ductility level earthquake, tubular joints need be designed for allowable stresses specified by API RP 2A using the computed joint loads instead of the tensile load or compressive buckling load of the member.

Joint capacity may be determined in accordance with 7.3, with the exception that Equations (7.1) through (7.4) should all have the safety factor (FS) equal to 1.0. See B.7.2 for the influence of chord load and other detailed considerations.

Joint capacity may be determined in accordance with API RP 2A equations by considering the safety factor (FS) set equal to 1.0.

14.9.2 Deck Appurtenances and Equipment

Equipment, piping, and other deck appurtenances should be supported so that induced seismic forces can be resisted and induced displacements can be restrained such that no damage to the equipment, piping, appurtenances, and supporting structure occurs. Equipment should be restrained by means of welded connections, anchor bolts, clamps, lateral bracing, or other appropriate tie-downs. The design of restraints should include both strength considerations as well as their ability to accommodate imposed deflections. Special consideration should be given to the design of restraints for critical piping and equipment whose failure could result in injury to personnel, hazardous material spillage, pollution, or hindrance to emergency response.

Design acceleration levels should include the effects of global platform dynamic response and, if appropriate, local dynamic response of the deck and appurtenance itself. Because of the platform's dynamic response, these design acceleration levels are typically much greater than those commonly associated with the seismic design of similar onshore processing facilities

In general, most types of properly anchored deck appurtenances are sufficiently stiff so that their lateral and vertical responses can be calculated directly from maximum computed deck accelerations, since local dynamic amplification is negligible. Forces on deck equipment that do not meet this "rigid body" criterion should be derived by dynamic analysis using either: 1) uncoupled analysis with deck level floor response spectra or 2) coupled analysis methods.

Appurtenances that typically do not meet the "rigid body" criterion are

1.drilling rigs,
2. flare booms,
3. deck cantilevers,
4. tall vessels,
5. and cranes.

Coupled analyses that properly include the dynamic interactions between the appurtenance and deck result in more accurate and often lower design accelerations than those derived using uncoupled floor response spectra.

Drilling and well servicing structures shall be designed for earthquake loads in accordance with API 4F. It is important that these movable structures and their associated setback and piperack tubulars be tied down or restrained at all times except when the structures are being moved.

Deck-supported structures, and equipment tie-downs, should be designed with a one-third increase in basic allowable stresses, unless the framing pattern, consequences of failure, metallurgy, and/or site-specific ground motion intensities suggest otherwise.

14.10. Seismic design Procedures

14.10.1. General

Two alternative procedures for seismic design are provided. A simplified method may be used where seismic considerations are unlikely to govern the design of a structure, while the detailed method shall be used where seismic considerations have a significant impact on the design. The selection of the appropriate procedure depends on the exposure level of the structure and the expected intensity and characteristics of seismic events. The simplified procedure allows the use of generic seismic maps; while the detailed procedure requires a site-specific seismic hazard study. In all cases, the simplified procedure may be used to perform appraisal and concept screening for a new offshore development. Figure 14.1 presents a flowchart of the selection process and the steps associated with both procedures.

14.10.2. Seismic risk category

The complexity of a seismic action evaluation and the associated design procedure depends on the structure's seismic risk category, SRC, as determined below. By using the acceleration levels provided in the general seismic maps (or special site-specific studies), the seismic zones and subsequently the appropriate seismic design procedure are determined. The selection of the procedure depends on the structure's exposure level as well as the severity of ground motion. The following steps shall be followed to determine the SRC.

A) Determine the site seismic zone: from the worldwide seismic maps, read the value for the 1.0s horizontal spectral acceleration, $S_{a,map}(1.0)$; using this value, determine the site seismic zone from Table 14.2.

Table 14.2. Site seismic zone

$S_{a,map}(1.0)$	<0.03g	0.03 g to 0.10 g	0. 11 g to 0.25 g	0.26 g to 0.45 g	>0.45g
Seismic zone	0	1	2	3	4

B) Determine the structure's exposure level, L1 (most important) or L3 (least important). The L2 exposure level is not applicable in seismic regions because it is not feasible to evacuate the platform prior to a seismic event. The target annual probabilities of failure associated with each exposure level are given in 14.3; these are required in the detailed procedure to determine seismic actions. Other target probabilities may be used in the detailed seismic action procedure if recommended by consultant and approved by client. The simplified seismic action procedure has been calibrated to the target probabilities given in 14.3.

Table 14.3. Target Annual Probability of Failure, P_f

Exposure level	P_f
L1	4×10^{-4}
L3	2.5×10^{-3}

C) Determine the structure's seismic risk category, SRC, based on the exposure level and the site seismic zone the SRC is determined from Table 14.4.

If the design lateral seismic action is smaller than 5 % of the total vertical action comprising the sum of permanent actions plus variable actions minus buoyancy actions, SRC 4 and SRC 3 structures may be recategorized as SRC 2.

Table 14.4. Seismic risk category, SRC

Site seismic zone	Exposure level	
	L3	**L1**
0	SRC 1	SRC 1
1	SRC 2	SRC 3
2	SRC 2	SRC 4
3	SRC 2	SRC 4
4	SRC 3	SRC 4

14.10.3 Seismic Design Requirements

Table 14.5 gives the seismic design requirements for each SRC; these requirements are also shown in Figure 14.1.
In seismically active areas, the designer shall strive to produce a robust and ductile structure, capable of withstanding extreme displacements in excess of normal design values. Where available for a given structure type, architectural and detailing requirements and recommendations for ductile design should be followed for all cases (except SRC 1).

Table 14.5. Seismic design requirements

SRC	Seismic action procedure	Evaluation of seismic activity	Nonlinear ductile level analysis
1	None	None	None
2	Simplified	regional maps	Permitted
3*	Simplified	Site-specific, regional maps	Recommended
	Detailed	Site-specific	Recommended
4	Detailed	Site-specific	Required

* For an SRC 3 structure, a simplified seismic action procedure is in most cases more conservative than a detailed seismic action procedure. For evaluation of seismic activity, results from a site-specific probabilistic seismic hazard analysis (PSHA), are preferred and should be used, if possible. Otherwise regional seismic maps may be used. A detailed seismic action procedure requires results from a PSHA whereas a simplified seismic action procedure may be used in conjunction with either PSHA results or seismic maps (regional maps).

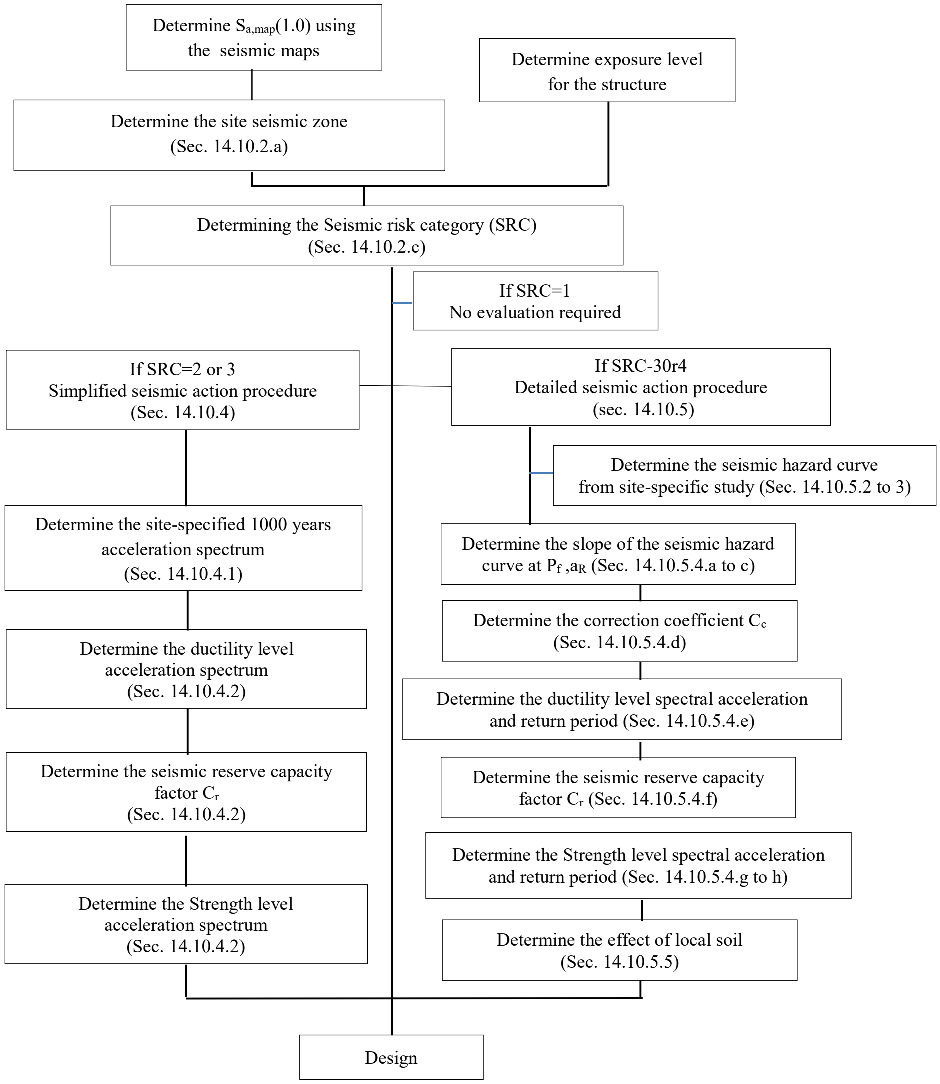


Figure 14.1. Seismic Design Procedures

14.10.4 Simplified Seismic Action Procedure

14.10.4.1 Soil classification and spectral shape

Having obtained the bedrock spectral accelerations at oscillator periods of 0.2 s and 1.0 s, Sa,map(0.2) and Sa,map(1.0), the following steps shall be followed to define the site response spectrum corresponding to a return period of 1000 years:

a) Determine the site class as follows. The site class depends on the seabed soils on which a structure is founded.

The soil type is a function of the average properties of the top 30 m of effective seabed as defined in Standard 2800 in compliance with the additional requirements of Chapter 3.

For deep pile foundations, the site class should consider the 30 m of soil immediately below the seat of pile resistance, which will generally be at different depths for lateral and vertical actions. For deep pile foundations, the seat of resistance would be at the centroidal depth of P-Y resisting forces for lateral and of T-Z for vertical.

b) Determine F_a and F_v as follows.

For deep pile foundations, the site coefficients F_a and F_v are listed in Table 14.6.

Table 14.6. Determination of the site class

Site class	F_a	F_v
I	1.0	0.8
II	1.0	1.0
III	1.0	1.2
IV	1.0	1.8
In sites having a special soil type (see Section 3.8), a site-specific study should be performed for seismic geotechnical investigation and analysis of the dynamic response of the site.		

c) The horizontal acceleration spectrum with a return period of 1000 years is prepared by multiplying the design spectrum (Section 3.8.2) by 1.5.

d) The site vertical spectral acceleration at a period T shall be taken as half the corresponding horizontal spectral acceleration. The vertical spectrum shall not be reduced further due to water depth effects.

e) The acceleration spectra obtained using the preceding steps correspond to 5 % damping. To obtain acceleration spectra corresponding to other damping values, the ordinates may be scaled by applying a correction factor D:

$$D = \frac{\ln(\frac{100}{\eta})}{\ln(20)} \qquad 14.1$$

Where η is the percent of critical damping.

As an alternative to the procedure given in a) to e), uniform hazard spectra (Section 3.8.1) obtained from PSHA with 1000 years return period may be used.

14.10.4.2 Design Seismic spectra

The design seismic acceleration spectra to be applied to the structure shall be determined as follows.
For each oscillator period T, the DLE horizontal and vertical spectral accelerations are obtained from the corresponding values of the site 1000 year spectral acceleration [see 14.10.4.1 c) and d)]:

$$S_{a,\,DLE}(T) = N_{DLE} \times S_{a,site}(T) \qquad 14.2$$

Where the scale factor N_{DLE} is dependent on the structure exposure level and shall be obtained from Table 14.7.

Table 14.7. Scale Factors for DLE Spectra

Exposure level	**Ductility level earthquake scale factor N_{DLE}**
L3	0.85
L1	1.60

The SLE horizontal and vertical spectral accelerations at oscillator period T may be obtained from:

$$S_{a,SLE}(T) = S_{a,DLE}(T)/C_r \qquad 14.3$$

where C_r is a seismic reserve capacity factor for the structural system that considers the static reserve strength and the ability to sustain large non-linear deformations of structure. The C_r factor represents the ratio of spectral acceleration causing catastrophic system failure of the structure, to the SLE spectral acceleration. The value of C_r should be estimated prior to the design of the structure in order to achieve an economic design that will resist damage due to an SLE and is at the same time likely to meet the DLE performance requirements. Values of C_r may be justified by prior detailed assessment of similar structures. Values of C_r for fixed steel structures are specified in Table 14.8. Values of C_r other than those recommended in the standard applicable to the type of offshore structure may be used in design, however such values shall be verified by an DLE analysis.

To avoid return periods for the SLE that are too short, C_r values shall not exceed 2.8 for L1 structures and 2.0 for L3 structures.

Table 14.8. C_r Factors for steel jacket of fixed offshore platforms

Characteristics of structure design	C_r
The recommendations for ductile design in Section 14.8 are followed and a nonlinear static pushover analysis according to this Regulations is performed to verify the global performance of the structure under the ductility level earthquake condition.	Variable up to 2.80, as demonstrated by analysis
The recommendations for ductile design in Section 14.8 are followed, but a nonlinear static pushover analysis to verify the ductility level earthquake performance is not performed.	Variable up to 2.00, as demonstrated by analysis.
The jacket structure has a minimum of three legs and a bracing pattern consisting of leg-to-leg diagonals with horizontals, or X-braces without horizontals. The slenderness ratio (kL/r) of the diagonal braces in vertical frames is limited to no more than 80 and $F_yD/Et \leq 0.069$. For X-bracing in vertical frames the same restrictions apply, where the length L to be used depends on the loading pattern of the X-bracing. A nonlinear analysis to verify the ductility level performance is not performed.	1.40
Where none of the above characterizations apply.	1.10

14.10.5. Detailed method of seismic analysis

14.10.5.1. Site-Specific Seismic Hazard Assessment

The most widely used seismic input parameter for the seismic design and analysis of offshore structures is the design acceleration spectrum. In site-specific studies, the design acceleration spectrum is usually derived from an acceleration spectrum computed from a probabilistic seismic hazard analysis (PSHA) with possible modifications based on local soil conditions. Deterministic seismic hazard analysis may be used to complement the PSHA results. These analyses are described in chapter 3.

14.10.5.2. Seismic Action Procedure

This procedure is based on the results of a PSHA (Chapter 3). The site-specific seismic hazard curve shall have been determined in terms of the annual exceedance probability of a spectral acceleration corresponding to a period that is equal to the dominant modal period of the structure $\overline{S_a}(T_{dom})$. In lieu of more specific information about the dominant modal period of the structure, the seismic hazard curve may be determined for the spectral acceleration at a period of 1.0s, $\overline{S_a}(1.0)$.

The DLE spectral accelerations are determined from the site-specific hazard curve and the target annual probability of failure, Pf, listed in Table 14.3. The specific steps to define the DLE and SLE events are illustrated in Fig. 14.2 and are described in the following steps.

a) Plot the site-specific hazard curve for T = Tdom on a log10-log10 basis, i.e. showing the probability distribution of the parameter [see Fig. 14.2.a].

b) Choose the target annual probability of failure, Pf, as a function of the exposure level as indicated in Table 14.3, and determine the site-specific spectral acceleration at Pf, from Figure 14.2.a.

c) Determine the slope of the seismic hazard curve (aR) in the region close to Pf by drawing a tangent line to the seismic hazard curve at Pf. The slope aR is defined [see Figure 14.2.a] as the ratio of the spectral accelerations corresponding to two probability values, one at either side of Pf, that are one order of magnitude apart. P1 should preferably be close to Pf.

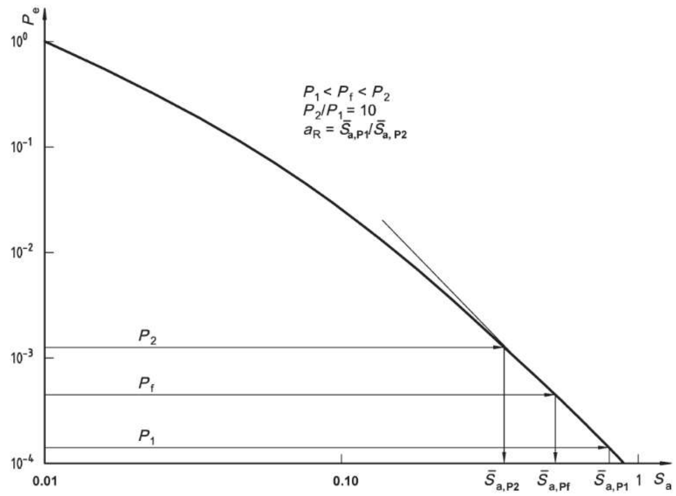
d) From Table 14.9 determine the correction factor, Cc, corresponding to aR. This correction factor captures the uncertainties not reflected in the seismic hazard curve.

e) Determine the DLE spectral acceleration by applying the correction factor Cc to $\overline{S}_{a,Pf}(T_{dom})$ the site-specific spectral acceleration at the required Pf and the structural dominant period Tdom:

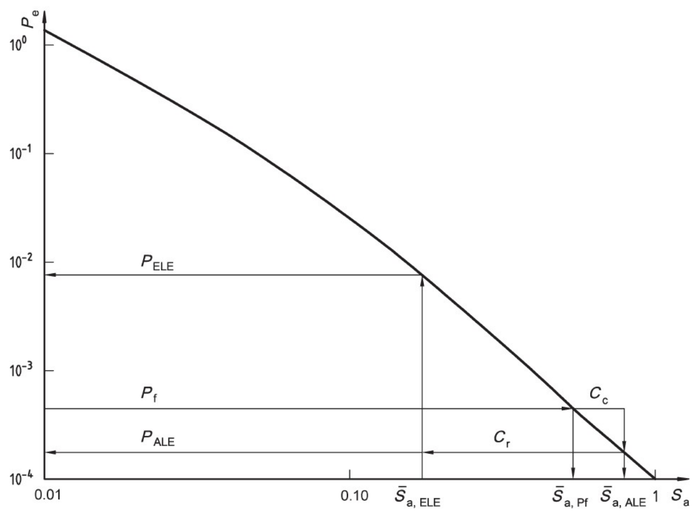
$$\overline{S}_{a,DLE}(T_{dom}) = C_c \times \overline{S}_{a,Pf}(T_{dom}) \qquad 14.4$$

The annual probability of exceedance for the DLE event (P_{DLE}) can then be directly read from the seismic hazard curve, see Figure 14.2.b. The DLE return period is determined from the annual probability of exceedance using Equation (14.5).

$$T_{return} = 1/Pe \qquad 14.5$$



a. Derivation of the slope a_R of the seismic hazard curve for $T=T_{dom}$



b. Derivation of spectral accelerations and probabilities for ductility and strength level earthquakes

Figure 14.2. Typical Seismic Hazard Curve.

Table 14.9. Correction coefficient C_c.

a_R	1.75	2.0	2.5	3.0	3.5
Correction coefficient C_C	1.20	1.15	1.12	1.10	1.10

P_{DLE} is smaller than Pf to accommodate uncertainties in action and resistance evaluations not represented in the seismic hazard curve (as captured in the correction factor Cc).

f) For certain structure types whose reserve strength and ductility characteristics are known, the SLE spectral acceleration is next determined from:

$$\overline{S}_{a,SLE}(T_{dom}) = \frac{\overline{S}_{a,DLE}(T_{dom})}{C_r} \qquad 14.6$$

Values of Cr for fixed steel structures are specified in Table 14.8.

g) The annual probability of exceedance for the SLE event (P_{SLE}) can now be read from the seismic hazard curve, Figure 14.2.b. The SLE return period is determined from the annual probability of exceedance using Equation (14.5).

Having determined DLE and SLE return periods, obtain DLE spectral accelerations and SLE spectral accelerations for other natural periods from the PHSA results.

h) Modifications of DLE and SLE acceleration spectra for local geology and soil conditions shall be addressed by a site response analysis using linear or nonlinear models of the subsoil.

Minimum SLE return periods are given in Table 14.10 to ensure economic viability of a design, as a function of exposure level. If the SLE return period that is obtained from the procedure in this subclause is lower than the corresponding return period listed in Table 14.10, the return period in this table shall be used for $S_{a,SLE}(T)$.

Table 14.10. Minimum return periods of the resistance level earthquake (years).

Importance level	Minimum return period of the resistance level earthquake
L3	50
L1	200

Chapter 15
Tsunami Loads and Effects

15.1. General

In this chapter, the method of estimating loads and effects caused by tsunami on the structure is presented. For this purpose, at first, tsunami risk categories are determined, scope of application and necessary definitions are stated, and then the method of calculation the tsunami flow velocity and depth at the construction site of the desired structure is discussed. In the following, the method of calculating forces (including hydrostatic and hydrodynamic forces) and the effects of this phenomenon on the structure, such as debris impact and scouring, will be discussed.

15.1.1. Tsunami risk categories

Tsunami Risk Categories for buildings and other structures, Based on the type of their application and the amount of damage to other structures and people caused by their damage, shall be the Risk Categories given in Table 4.3 with the following modifications:

1- Tsunami Vertical Evacuation Refuge Structures shall be included in Tsunami Risk Category I
2- The employer is allowed to include critical facilities such as power generation stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities that are not included in in Tsunami Risk Category I, in category II.
3- The following structures do not need to be included in Tsunami Risk Category I and the employer must designate them as Tsunami Risk Category II or III:
   a- Fire and emergency stations.
   b- Police stations that are not required for post-disaster emergency response as a unique critical center.
4- It is recommended that public facilities that are needed to provide services in times of crisis, such as fire stations, emergency services, police stations, parking lots, and locations for road construction machinery and equipment, should not be included in the tsunami plan as much as possible.

15.1.2. Scope of application

The following buildings and other structures located in the tsunami design zone (15.2.4) must be designed for maximum tsunami effects such as hydrostatic and hydrodynamic loads, debris impact loads, settlements and scour effects in accordance with the requirements of this chapter.

a. Buildings and structures of tsunami risk category I

b. Buildings and structures of tsunami risk category II with inundation depth greater than one meter

c. Buildings and structures of Tsunami risk category III with an average height of more than 20 meters and inundation depth greater than one meter in cases where, according to the employer's decision, design for tsunami effects is required.

If Tsunami Risk Category III buildings are one story of any height and do not have mezzanines or any critical equipment or systems, they need not be designed for the tsunami loads and effects specified in this chapter.

The tsunami risk category is determined according to 15.1.1 and the tsunami design zone is determined according to 15.2.4.

The entry of this chapter, which according to the Appendix 1 is water height above grade line on the coastline due to the maximum considered tsunami, has been prepared for Makran region in Iran.

Considering the tsunami load and effects for the included structures according to this section is mandatory, but the use of this chapter is strongly recommended for its application.

15.1.3. Peer review

In case of using the site-specific procedures and also for buildings and structures of tsunami risk category I, peer review is required.

15.1.4. Definitions

With the purpose of uniformity and clarity, a glossary of applied words in this chapter is presented.

LOCAL SCOUR:

Removal of material from a localized portion of land surface, resulting from flow around, over, or under a structure or structural element.

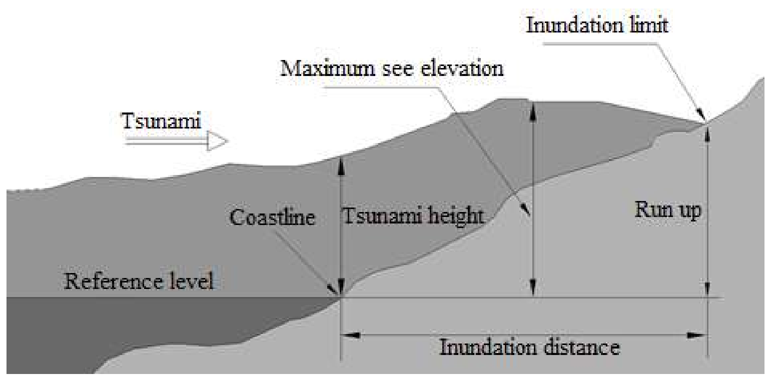
PILE SCOUR:

A special case of enhanced local scour that occurs at a pile, bridge pier, or similar slender structure.

TOE SCOUR:
A special case of enhanced local scour that occurs at the base of a seawall or similar structure on the side directly exposed to the flow. Toe scour can occur whether or not the structure is overtopped.
SUSTAINED FLOW SCOUR:
Enhanced local scour that results from flow acceleration around a structure. The flow acceleration and associated vortices increase the bottom shear stress and scour out a localized depression.

PLUNGING SCOUR:
A special case of enhanced local scour that occurs when the flow passes over a complete or nearly complete obstruction, such as a barrier wall, and drops steeply onto the ground below, scouring out a depression.
CHANNELIZED SCOUR:
Scour that results from broad flow that is diverted to a focused area such as return flow in a preexisting stream channel or alongside a seawall.
LIQUIFACTION SCOUR:
The limiting case of pore pressure softening associated with hydrodynamic flow, where the effective stress drops to zero. In noncohesive soils, the shear stress required to initiate sediment transport also drops to zero during liquefaction scour.
STRUCTURAL COMPONENTS:
A component of a building that provides gravity-load-carrying or lateral-force resistance as part of a continuous load path to foundation, including beams, columns, slabs, braces, walls, wall piers, coupling beams, and connections.
PRIMARY STRUCTURAL COMPONENT
Structural components required to resist tsunami forces and actions and inundated structural components of the gravity-load-carrying system.
SECONDARY STRUCTURAL COMPONENT
A structural component that is not primary.
DESIGNATED NONSTRUCTURAL COMPONENTS AND SYSTEMS:
Nonstructural components and systems that are assigned a component Importance Factor, I_p, equal to 1.5 according to section 4.1.3 of Iranian 2800 standard.

RUNUP:
When the tsunami wave reaches the shore, the water level temporarily rises on the shore and, which is called Runup (Fig. 15.1).
MAXIMUM CONSIDERED TSUNAMI:
A probabilistic tsunami having a 2% probability of being exceeded in a 50-year period or a 2,475-year mean recurrence interval.
DESIGN TSUNAMI PARAMETERS:
The tsunami parameters used for design, consisting of the inundation depths and flow velocities at the site (Fig. 15.1)
CRITICAL FACILITY:
Buildings and structures that are vital for the implementation of crisis management programs and the return of society to normal conditions, such as facilities for power, water, communications, major transportation infrastructure, and medical centers.
CRITICAL EQUIPMENT OR CRITICAL SYSTEMS:
Nonstructural components designated essential for the functionality of the critical facility or essential facility or that are necessary to maintain safe containment of hazardous materials.
INUNDATION ELEVATION:
The elevation of the tsunami water surface (caused by the maximum considered tsunami, including relative sea level change) with respect to the reference sea level (Fig. 15.1).
RUNUP ELEVATION:
Ground elevation at the maximum tsunami inundation limit, including relative sea level change, with respect to the reference sea level (Fig. 15.1)



Figurers 15.1: Inundation depth and inundation distance

REFERENCE SEA LEVEL:

The sea level datum used in energy grade line method and site-specific inundation modeling that in this regulation it is taken to be Mean sea Level (MSL).

RELATIVE SEA LEVEL CHANGE:

The local change in the level of the sea relative to the land, which might be caused by sea rise and/or subsidence of the land.

TSUNAMI AMPLITUDE:

The absolute value of the difference between a particular peak of the tsunami and the undisturbed sea level at the time.

TSUNAMI BREAKAWAY WALL:

Any type of wall subject to flooding that is not required to provide structural support to a building or other structure and that is designed and constructed such that, before the development of the design flow conditions of Inundation Load Case 1(15.3.3.1), the wall will collapse or detach in such a way that (1) it allows substantially free passage of floodwaters and external or internal waterborne debris, including unattached building contents and (2) it does not damage the structure or supporting foundation system.

OPEN STRUCTURE:

A structure in which the portion within the inundation depth has no greater than 20% closure ratio, and in which the closure does not include any tsunami breakaway walls, and which does not have interior partitions or contents that are prevented from passing through and exiting the structure as unimpeded waterborne debris.

TSUNAMI VERTICAL EVACUATION REFUGE STRUCTURE:

A structure designated and designed to serve as a point of refuge to which a portion of the community's population can evacuate above a tsunami when high ground is not available.

NONBUILDING CRITICAL FACILITY STRUCTURE:

Nonbuilding structure whose Tsunami Risk Category is designated as either I or II.

LIFE SAFETY STRUCTURAL PERFORMANCE LEVEL:

The post event damage state is that in which a structure has damaged components but retains a margin against onset of partial or total collapse.

COLLAPSE PREVENTION STRUCTURAL PERFORMANCE LEVEL:
A post event damage state in which a structure has damaged components and continues to support gravity loads but retains little or no margin against collapse.
DAMAGE CONTROL STRUCTURAL PERFORMANCE LEVEL:
The Damage Control performance level is an intermediate structural performance level between Life Safety and Immediate Occupancy.
MOMENTUM FLUX:
The quantity $\rho_s hu2$ for a unit width based on the depth-averaged flow speed u, over the inundation depth h, for equivalent fluid density ρ_s, having the units of force per unit width.
GRADE PLANE:
A horizontal reference plane at the site representing the average elevation of finished ground level adjoining the structure at all exterior walls.
FROUDE NUMBER, Fr:
A dimensionless number defined by $u/\sqrt{gh}$, where u is the flow velocity averaged over the cross section perpendicular to the flow, which is used to quantify the normalized tsunami flow velocity as a function of water depth. Based on the Froude number, the flow can be classified into three types of subcritical, critical, and supercritical.
INUNDATION DEPTH:
The depth of maximum considered tsunami water level, including relative sea level change, with respect to the grade plane at the structure.
GENERAL EROSION:
A general wearing away and erosion of the land surface over a significant portion of the inundation area, excluding localized scour actions.
TSUNAMI RISK CATEGORY:
The Risk Category against tsunami according Section 15.1.1.
INUNDATION LIMIT:
The maximum horizontal inland extent of flooding for the Maximum Considered Tsunami.
BATHYMETRIC PROFILE:
A cross section showing ocean depth plotted as a function of horizontal distance from a reference point (such as a coastline).

TOPOGRAPHIC TRANSECT:
Profile of vertical elevation data versus horizontal distance along a cross section of the terrain, in which the orientation of the cross section is perpendicular or at some specified orientation angle to the shoreline.

TSUNAMI DESIGN ZONE:
An area between the shoreline and the inundation limit, within which structures are analyzed and designed for inundation by the Maximum Considered Tsunami.

PORE PRESSURE SOFTENING:
A mechanism that enhances scour through increased pore-water pressure generated within the ground during rapid tsunami loading and the release of that pressure during drawdown.

CLOSURE RATIO:
Ratio of the area of enclosure, not including glazing and openings, that is inundated to the total projected vertical plane area of the inundated enclosure surface exposed to flow pressure.

TSUNAMI DESIGN ZONE MAP:
The height of the water on the coastline due to the maximum considered tsunami (Appendix 1).

15.1.5. SYMBOLES AND NOTATIONS

A	:	Vertical projected area of an individual element
A_{beam}	:	Vertical projected area of an individual beam element
A_{col}	:	Vertical projected area of an individual column element.
A_{wall}	:	Vertical projected area of an individual column element.
B	:	Overall building width
b	:	Width subject to force
C_{cp}	:	A coefficient for calculating the forces on the pipe due to tsunami, which is equal to 1.5.
C_d	:	Drag coefficient for hydrodynamic force.
C_{dis}	:	Dimensionless flow coefficient.
C_{cx}	:	Proportion of closure coefficient.
C_r	:	Lateral resistance coefficient of the pipe .

C_0 : Orientation coefficient (of debris)

C_l^- : Pipe resistance coefficient against downward vertical force .

C_l^+ : Pipe resistance coefficient against upward vertical force .

C_{2v} : Plunging scour coefficient equals to 2.8.

D : Dead load.

D_p : Pipe diameter

D_s Scour depth

F_d : Drag force on an element or component

F_{dx} Drag force on the building or structure at each level

F_h : Unbalanced hydrostatic lateral force

F_i : Debris impact design force

F_l^- : The downward vertical force per unit length of the pipe due to tsunami.

F_l^+ : The upward vertical force per unit length of the pipe due to tsunami.

F_{pw} : Hydrodynamic force on a perforated wall

F_r : Froude number

F_{rp} : Horizontal force per unit length of the pipe due to tsunami

F_{TSU} : Tsunami load or effect

F_v : Buoyancy force

F_w : Tsunami load on wall or pier

$F_{w\theta}$: Force on a wall oriented at an angle θ to the flow

g : Acceleration caused by gravity

H_{TSU} : Load caused by Tsunami-induced lateral earth pressure under submerged conditions

H_0 : Depth to which a barrier is overtopped above the barrier height

H_B : Barrier height of a levee, seawall, or freestanding retaining wall

h : Tsunami inundation depth above grade plane at the structure

h_0 : The height of the water on the coastline due to the maximum considered tsunami

h_e : Inundation depth

h_{max} : Maximum inundation depth above grade plane at the structure

h_r	:	Residual water height within a building
h_s	:	Height of structural floor slab above grade plane at the structure
h_{sx}	:	The average height of the upper and lower floors of each floor below the tsunami inundation level
I_{tsu}	:	Importance Factor for Tsunami forces
k_s	:	Fluid density factor
L	:	Live load
n	:	Manning's coefficient
P_c	:	Flow pressure on the wall
P_r	:	Residual water hydrostatic pressure
P_s	:	Hydrostatic surcharge pressure caused by tsunami inundation
P_{uw}	:	Equivalent uniform lateral static pressure for hydrodynamic load
q	:	Discharge per unit width over an overtopped structure
S	:	Snow load
u	:	Tsunami flow velocity
U	:	Jet velocity of plunging flow
U_{max}	:	Maximum Tsunami flow velocity at the structure
V_w	:	Displaced water volume
x	:	Distance from the coastline
x_R	:	Inundation limit distance from the coastline
z	:	Ground elevation above mean sea level
α	:	Coefficient specifying the flow type. For supercritical flow, it is equal to 1.3 and otherwise it is equal to one
Δx	:	Horizontal distance between two points
γ_s	:	Fluid weight density
γ_{sw}	:	Seawater weight density
Ω_0	:	Overstrength factor for the lateral-force-resisting system
ρ_s	:	Fluid mass density
ρ_{sw}	:	Seawater mass density
ψ	:	Angle between the plunging jet at the scour hole and the horizontal

15.2. Calculating the tsunami intensity and inundation

In order to calculate the effects of tsunami and design or control the building against these effects, it is necessary to estimate the intensity of tsunami hazard based on the parameters of the maximum inundation depth and the maximum flow velocity at the site of the structure. In this part, the method and relationships required to obtain an acceptable estimate of these parameters (based on the water height at the coastline according to Appendix 1) are presented.

15.2.1. Calculation of the maximum inundation depth and flow velocity at site

The maximum inundation depth and the flow velocity at the desired site can be obtained from two methods of energy grade line and site-specific inundation analysis.

The use of the energy grade line method, which conservatively estimates the water flow parameters at the site, is allowed for structures of tsunami risk category I, II and III.

Site-specific tsunami inundation analysis can also be used for tsunami risk category I, II and III structures, but is not obligatory.

In case of using the site-specific procedures and also for buildings and structures of tsunami risk category I, peer review is required.

In both methods, the flow velocity at the site should not be considered less than 3(m/s). Also, it is not necessary to consider the flow velocity more than the minimum value of two numbers 15(m/s) and $\sqrt{1.5(gh_{\max})}$, where $h_{\max}$ is the maximum inundation depth at site.

The inundation depth at the site obtained from the site-specific analysis, should not be less than eighty percent of the inundation depth obtained from the energy grade line method.

15.2.2. The steps of calculating the maximum inundation depth and the flow velocity at site using the energy grade line method

The basis of this method is based on the conservation of energy and considering the effect of friction to gradually reduce the energy level of the wave caused by the tsunami from the coastline towards the inundation limit. To use the energy method, the topographic transect from the coastline to the site should be determined according to section 15.2.3.

Figure 15.2 shows the parameters of the energy method between the coastline and the inundation limit on the coast. The ground level (z_i) is

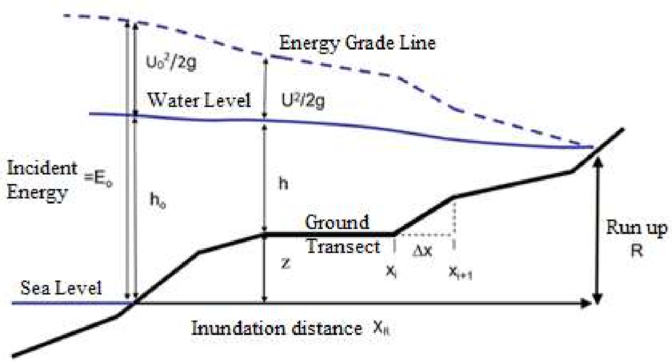


Figure 15.2: Parameters of the energy method between the coastline and the end of the inundation distance (inundation limit)

Determined along the desired profile at points with a maximum horizontal distance of 30 meters. The values of Manning's coefficient can be different for each part. With the height of the water on the coastline (According to the zoning map of Appendix 1 plus the amount tidal elevation), it is possible to proceed from the coastline to the desired point using the step-by-step method of energy and calculate the maximum inundation depth (h_{max}) and the maximum flow velocity (U_{max}) at the site.

The amount of inundation depth (h) (water height from the ground surface) at point $i+1$ along the path of wave advance (from the coastline to the final limit inundation) is obtained from equation 15.1.

$$\left(z+h+\frac{1}{2}F_r^2 h\right)_{i+1}=\left(z+h+\frac{1}{2}F_r^2 h\right)_i-\left(\frac{gF_r^2 n^2 \Delta x}{h^{1/3}}\right)_i \quad 15.1$$

where z is the height of the earth, h is the height of water above the surface of the earth, g is the acceleration caused by gravity, F_r is Froude number (according to equation 15.3) and n is Manning's coefficient, which is determined from table 15.1. In case of doubt in determining the conditions listed in this table, it is necessary to consider the lower value of Manning's coefficient. Δx is the horizontal distance between two points that should be smaller than 30 meters and its amount can vary along the topographic transect. Equation 15.2 is used to calculate the horizontal flow velocity at any point.

$$U=F_r\sqrt{gh} \quad 15.2$$

Also, the Froude number is calculated according to the equation 15.3.

$$F_r=\alpha\left(1-\frac{x}{x_R}\right) \quad 15.3$$

In relation 15.3, x is the horizontal distance from the coastline to the desired point and x_R is the inundation distance based on the energy method. Coefficient α is equal to 1.3 for supercritical flow and otherwise equal to one.

Supercritical water flow for a tsunami occurs when one of the following conditions exists:

1- The dominant slope of the seabed near the coast is 1/100 or more gentle.

2- Shallow reefs or other step-shaped discontinuities are present in the shallow part of the seabed near the shore.

Table 15.1: Manning's Roughness, n, for Energy Grade Line Analysis

Description of Frictional Surface	n
Coastal water nearshore bottom friction	0.025 to 0.03
Open land or field	0.025
Buildings of at least urban density	0.04
All other cases	0.03

15.2.3. Topographic transect for calculations of energy grade line method, loads and effects of tsunami

In the design of structures for tsunami loads and effects, instead of assuming that the extension of the tsunami wave is uniformly perpendicular to the coast, it is assumed that the extension of the incoming current from the perpendicular line along the coastline changes by ±22.5 degrees, so that the center of this changes is located in the geometric center of the structure in the plan.

Therefore, the first step in the calculations of the energy method is to determine the topographic transect between the coastline to the site. For this purpose, it is necessary to consider three topographic transects, according to Figure 15.3, the first one is perpendicular to the average 300 meters of the coastline (150 meters from each side). Two other transects with angles of 22.5 degrees in the clockwise and anti-clockwise directions are considered relative to this transect.

It is necessary to determine points with a maximum horizontal distance of 30 meters on these topographic transects. Then for all three transects, inundation limit and energy grade line calculations are done and inundation depth and flow velocity are calculated at site and the maximum results are considered as the final results.

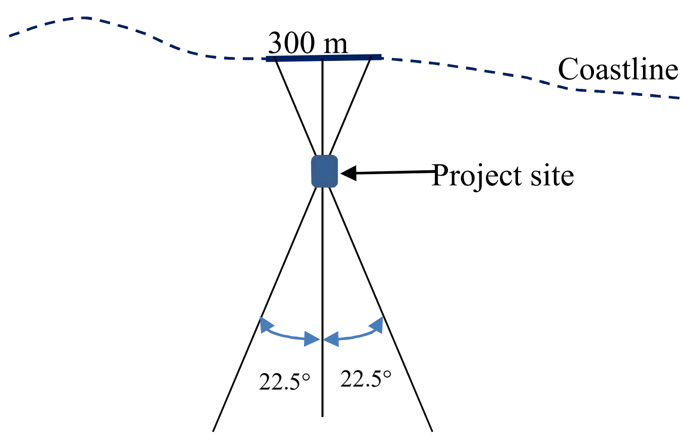


Figure 15.3: Illustration of the position of three topographic transects at desired site in the energy grade line method

15.2.4. Tsunami Design Zone (Inundation limit)

The upper limit for the tsunami design zone is where the height of the ground (z) is 1.5 times the height of the maximum tsunami height in the coastline (according to the tsunami design zone map of Appendix 1 plus the height increase due to the tide). If the desired site is outside this range, there is no need for tsunami calculations, but for structures located in this range, it is necessary to calculate the exact inundation limit.

In order to obtain the inundation limit, it is necessary to do some grid searches and assume all the points as the end of the inundation distance, respectively, in all three topographic transects, and calculate the height of the water at the end of inundation distance.

Finally, the inundation limit of each topographic transect is the point where the inundation height is equal to zero when we assume it as the end of inundation distance.

15.2.5. Special considerations

15.2.5.1. Seismic Subsidence before Tsunami Arrival

The creation of a tsunami due to displacement on the Makran subduction fault, which is located near the coast, in addition to the rise of the sea floor, can cause land subsidence in nearby areas or on the coasts. This subsidence is added to the height of the tsunami wave. In the results of the zoning values of this regulation, the results of the probabilistic analysis of land subsidence in the West Makran coast, in the return period of 2475 years, have been added to the calculated of the probabilistic analysis of the wave height on the coastline.

15.2.5.2. Tide

Sea level rise due to the tide is not included in the tsunami design zone map presented in this regulation and it is necessary, the amount of sea level rise due to tide compared to the mean water level (the difference between the mean high water level and mean sea level) to be added to it in each station.

15.2.5.3. Sea level change

The possible rise of the sea level (due to local or global factors such as the melting of polar ice due to global warming) is not included in the tsunami design zone map presented in this regulation, and any additional effect of this phenomenon should be added. This increase should be equal to the rise of sea water during the project lifecycle of the structure. A project lifecycle of not less than 50 years shall be used, but certain structures can have a project lifecycle longer than this amount.

15.3. Structural design procedures for tsunami effects

The procedure and details of the calculation steps related to the design and control of structures and oil facilities against tsunami are presented in this section. The limit and acceptable criteria in this section are recommended as minimum requirements and can be changed based on the final opinion of the employer and consulting engineer. For this purpose, the suggested performance levels for buildings in each of the 4 tsunami risk categories due to the maximum considered tsunami are presented. For the functional criteria of non-building structures, the criteria of paragraph 15.4.8 must also be observed.

15.3.1. Performance of Tsunami Risk Category I Buildings and Other Structures and Tsunami Risk Category II Critical Facilities (clauses B and C)

Structural components, connections, foundations of buildings and other structures of tsunami risk category I and critical structures of clause b and c of tsunami risk category II that are included in the tsunami design zone should be designed in such a way that meet the minimum structural performance criteria of " Damage Control performance level" and the following requirements:

- Non-structural operational components and equipment in the building that are required for essential functions, as well as the lowest horizontal member of the structure that supports these components and equipment, must be above the inundation elevation of the maximum considered tsunami (at level of 1.3hmax and higher)
- Structural components and connections in occupiable levels, as well as foundations, shall be design to maintain the "Immediate Occupancy Structural Performance" level. It is allowed to operate and live in levels of buildings and supporting structures that are higher than the level of inundation caused by the maximum considered tsunami.

- Tsunami Vertical Evacuation Refuge Structures shall also comply with Section 15.4.6

Note: Tsunami Vertical Evacuation Refuge Structures must be designed for the "Immediate Occupancy Structural Performance" level.

15.3.3. Performance of Tsunami Risk category II and III Buildings and Other Structures.

Structural components, connections, foundations of buildings and other structures of Tsunami Risk Categories II and III that are included in the tsunami design zone must be designed to meet the minimum structural performance criteria of “Collapse preservation structural performance level". The proposed performance objectives structural and non-structural members are summarized in Table 15.2

15.3.3. Structural performance evaluation

In order to ensure the ability of the structure to resist the loads and effects caused by the tsunami, it is necessary to evaluate the strength and stability of the structure based on the load cases define in section 15.3.3.1. It is also necessary to consider the corresponding acceptance criteria based on the expected performance levels. Structural acceptance criteria for this evaluation are given in section 15.3.3.4 and 15.3.3.5.

15.3.3.1. Inundation Load cases

In the tsunami design limit and for designing buildings and structures against tsunami, the following three situations are considered:

- The overall stability of the structure and foundation under the combined effect of the buoyancy and hydrodynamic forces;
- The maximum effects of hydrodynamic forces on the structure (corresponding to the maximum flow velocity and maximum specific momentum flux)
- The effects of the maximum hydrodynamic forces on the structure (corresponding to the maximum inundation depth).

Table 15.2: Mandatory conditions to consider tsunami effects for different tsunami risk categories and corresponding performance levels

Tsunami risk category	Mandatory conditions (section 15.1.2) to check the effect of maximum considered tsunami on tsunami risk categories	Requirements(section 15.3.1, 15.3.2)	
		structural components	Non-structural components and equipment
Tsunami Vertical Evacuation Refuge Structures	Located in tsunami design zone	Immediate occupancy	-
Tsunami Risk Category I Buildings and Other Structures and Tsunami Risk Category II Critical Facilities (clauses B and C)	Located in tsunami design zone	Damage Control	Transfer to a level of the structure that is higher than the level of Inundation
Tsunami Risk category II Buildings and Other Structures.	Located in tsunami design zone with $h_{\max} > 1(m)$	Collapse Prevention	Collapse Prevention
Tsunami Risk category III Buildings and Other Structures.	According to the decision of the employer within the Tsunami design zone and with $h_{\max} > 1(m)$ with an average height of the building higher than the specified height (20 meters)	Collapse Prevention	-

For this purpose, the following three load cases due to inundation should be evaluated as minimum criteria:

Load case 1

Load case 1 requires consideration of the combined effects of buoyancy and lateral hydrodynamic load when the exterior inundation depth reaches the lesser of one story or the height of the top of the first-story windows, provided this does not exceed the maximum inundation depth at the site.

Note: Load Case 1 need not be applied to open structures nor to structures where the soil properties or foundation and structural design prevent detrimental hydrostatic pressurization on the underside of the foundation and lowest structural slab.

Load case 1

In this case, the inundation is considered to be two-thirds of its maximum value and it is assumed that the maximum velocity and maximum specific momentum flux occur in either incoming or receding directions.

Load case 3

In this case, the velocity is considered to be one third of maximum in either incoming or receding directions when the inundation depth is considered to be equal to the maximum value.

15.3.3.2. Tsunami Importance Factors

Importance Factors, I_{tsu}, given in Table 15.3 shall be applied to the tsunami hydrodynamic and impact loads in Sections 15.4.2 and 15.4.3, respectively.

Table 15.3: Tsunami impact factors for hydrodynamic and impact loads

Tsunami risk categories	I_{tsu}
Tsunami Risk Category I, Vertical Evacuation Refuges, and Tsunami Risk Category II Critical Facilities	1.25
Tsunami risk category II	1
Tsunami risk category II	1

15.3.3.3. Load combinations

Principal tsunami forces and effects shall be combined with other specified loads in accordance with the load combinations:

$$0.9D + F_{TSU} + H_{TSU} \tag{15. 4}$$

$$1.2D + F_{TSU} + 0.5L + 0.2S + H_{TSU} \tag{15.5}$$

where

F_{TSU} = tsunami load effect for incoming and receding directions of flow, and

H_{TSU} = load caused by tsunami-induced lateral foundation pressures developed under submerged conditions. Where the net effect of H_{TSU} counteracts the principal load effect, the load factor for H_{TSU} shall be 0.9.

D = Dead load.

L = Live load.

S = Snow load.

15.3.3.4. Acceptance criteria of lateral-force-resisting system

To evaluate the capacity of the structural system at the level of life safety structural performance against the lateral load caused by the maximum considered tsunami, 75% of the required horizontal seismic load can be used, according to the fourth chapter of this regulation. This load includes the system's overstrength factor (Ω_0), according to section 4.5.1 and table 4.5 In order to control the level of immediate occupancy structural performance objectives, it is necessary to explicitly analyze and evaluate the lateral-force-resisting system.

15.3.3.5. Acceptance criteria of structural component based on component design strength

Structural components must be designed for combination of the forces caused by the tsunami on the structural system and any resultant actions caused by the tsunami pressures acting locally on the individual structural components for that direction of flow. The internal forces and system displacements are calculated using a linearly elastic, static analysis.

If the design strength of structural components and structural connections is greater than the loads and effects caused by the maximum considered tsunami (which are calculated based on the combination of loads introduced in section 15.3.3.3) then the expected structural performance level criteria in sections 15.3.1 and 15.3.2, as applicable, can be considered controlled.

The material resistance factor ϕ for the component and behavior under consideration is according to the values introduced in chapters 8, 9 and 10 of the Iranian National Building Regulations (INBR).

15.3.4. Minimum fluid density for tsunami loads

Seawater specific weight density γ_{sw} is considered equal to $10\,KN/m^3$ and its mass density ρ_{sw} is considered equal to $1025 Kg/m^3$. Due to the presence of suspended solids and debris flow-embedded smaller objects in the water, the fluid specific weight density in the tsunami flow will be different. Therefore, the minimum fluid specific weight density γ_s to determine the hydrostatic loads is calculated according to the following equation:

$$\gamma_s = k_s \gamma_{sw} \tag{15.6}$$

The minimum mass density of the fluid, to determine the hydrodynamic loads of the tsunami in order to consider the objects and debris immersed in water, is calculated according to the following:

$$\rho_s = k_s \rho_{sw} \tag{15.7}$$

where k_s, fluid density factor, is considered equal to 1.1.

15.3.5. Minimum closure ratio for load determination

It is necessary to calculate the loads on the building by assuming a minimum closure ratio of 70% of the inundated projected area along the perimeter of the structure, unless the structure is an open structure according to the definition in section 15.1.4.

The load effect caused by the accumulation of debris against or within the open structure should be considered by using a minimum closure ratio of 50% of the inundated projected area along the perimeter of the open structure. For open structures, there is no need to consider load case 1 of section 15.3.3.1.

15.3.6. Minimum number of tsunami flow cycles

At least two tsunami inflow and outflow cycles should be considered in the design. The first one is based on 80% of the maximum inundation depth and the second cycle is considered based on the maximum inundation depth caused by the maximum considered tsunami. Local scour effects caused by the first cycle are estimated based on section 15.4.4.1 at the construction site. These effects are considered as initial conditions of the second cycle.

15.3.7. Seismic effects on the foundations preceding maximum considered tsunami

In parts of the coast that are exposed to tsunami waves caused by earthquakes, it is necessary to design structures and non-structural components preceding coseismic effects. In these areas, the foundation of the buildings should be designed to resist the preceding earthquake ground motion and associated effects in accordance with chapter 5 of this regulation and the Iranian National Building Regulations, including chapters 7 and 9 in related cases.

In the design of the foundation against tsunami, it is necessary to consider the possible changes in the site surface and the characteristics of the local soil due to the occurrence of the tsunami-causing earthquake as initial conditions. The geotechnical studies should include foundation effects in reference to seismic effects preceding the tsunami, taking into account the possible instability of the slopes, liquefaction, total and differential settlement, and surface displacement due to faulting and seismically induced lateral spreading or lateral flow. Also, the requirements of chapter 15.4.4 must also be evaluated.

15.3.8. Physical modeling of tsunami flow, loads and its effects

The physical modeling of tsunami load and its effects on the building can be used as an alternative option for estimating the flow velocity magnification, hydrodynamic forces (section 15.4.2), debris impact loads (section 15.4.3) and foundation design (section 15.4.4) should be used.

15.4. Tsunami loads and effects on structures and industrial facilities

In general, loads and effects of tsunami on industrial structures and facilities are divided into three categories: 1- load caused by fluid (hydrostatic and hydrodynamic), 2- effect of debris impact load, 3- effects of tsunami on the foundation of structures and industrial facilities.

15.4.1. Hydrostatics loads

Hydrostatic loads to be considered in tsunami design include buoyancy, unbalanced lateral hydrostatic force, residual water surcharge on floors and walls, and pressure on foundations. The amount of these loads has a direct relationship with the specific weight density and the inundation depth.

15.4.1.1. Buoyancy

Reduced net weight due by buoyancy should be evaluated for all inundated structural and designated nonstructural elements of the building. For this purpose, the buoyancy force is calculated according to Eq. 15.8.

$$F_v = \gamma_s V_w \tag{15.8}$$

In this equation, V_w is displaced water volume and γ_s is minimum fluid density. This force is shown in a simple form in Figure 15.4.

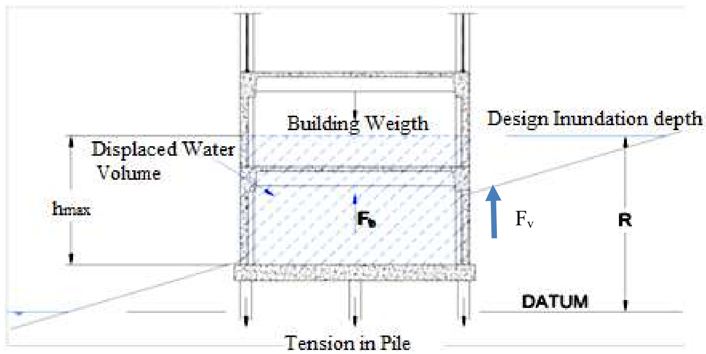


Figure 15.4: Buoyancy force in a structure where the fluid cannot penetrate to

Buoyancy must account for the influence of air trapped beneath floors, such as integral structural slabs, as well as in enclosed areas where the walls are not designed to break away. For windows, except those specifically constructed to withstand high-velocity wind-borne debris impact or blast loading, they may be regarded as openings once the depth of flooding reaches the uppermost part of the windows or the anticipated strength of the glazing, whichever is lower. The calculation of uplift shall encompass the volumetric displacement of foundation components, excluding deep foundations.

15.4.1.2. Unbalanced Lateral Hydrostatic Force

Inundated structural walls, where the openings constitute less than 10% of the total wall area and either have a length exceeding 9 m without neighboring tsunami breakaway walls or possess a two- or three-sided perimeter structural wall configuration irrespective of length, shall be designed to resist an unbalanced hydrostatic lateral force as specified by Eq. 15.9. This force occurs during the Load Case 1 and Load Case 2 inflow cases outlined in Section 15.3.3.1.

$$F_h = P_c A_{wall} = \frac{1}{2}\gamma_s b h_{\max}^2 \qquad 15.9$$

Where

F_h= unbalanced hydrostatic lateral force,

P_c= Flow pressure on the wall,

A_{wall}= vertical projected area of an individual column element,

γ_s= fluid weight density,

b= width subject to force, and

$h_{\max}$= maximum inundation depth above grade plane at the structure

In conditions where the flow surpasses the height of the wall, the value of $h_{\max}$ in Eq. 15.9 is substituted with the actual height of the wall.

15.1.4.3. Residual Water Surcharge Load on Floors and Wall

Horizontal floors located below the maximum inundation depth must be designed to withstand the combination of dead load and an additional residual water surcharge pressure, represented as P_r in accordance with Eq. 15.10. Moreover, structural walls capable of retaining water during drawdown must also be designed to resist residual water hydrostatic pressure.

$$\begin{aligned} P_r &= \gamma_s h_r \\ h_r &= h_{\max} - h_s \end{aligned} \qquad 15.10$$

Here, h_s refers to the elevation of the top of the floor slab. Nevertheless, it is not necessary for h_r to surpass the height of the continuous section of any perimeter structural element at the floor.

15.1.4.4. Hydrostatic Surcharge Pressure on Foundation

The hydrostatic surcharge pressure resulting from tsunami inundation shall be computed as follows:

$$P_s = \gamma_s h_{\max} \tag{15.11}$$

15.4.2. HYDRODYNAMIC LOADS

Hydrodynamic loads can be calculated by two methods: the simplified equivalent uniform lateral static pressure and detailed hydrodynamic lateral forces.

15.4.2.1. Simplified Equivalent Uniform Lateral Static Pressure

It is permissible to consider the combination of unbalanced lateral hydrostatic and hydrodynamic loads by applying an equivalent maximum uniform pressure, denoted as P_{uw}, determined in accordance with Eq. 15.12. This pressure is applied over a distance of 1.3 times the calculated maximum inundation depth, $h_{\max}$, at the specific site, in each flow direction.

$$P_{uw} = 1.25 I_{tsu} \gamma_s h_{\max} \tag{15.12}$$

15.4.2.2. Detailed Hydrodynamic Lateral Forces

15.4.2.2.1. Overall Drag Force on Buildings and Other Structures

The design of the building's lateral-force-resisting system must be capable of withstanding the total drag forces experienced at each level, resulting from either incoming or outgoing flow during Load Case 2, as defined by Eqs. 15.13 and 15.14.

$$F_{dx} = \frac{1}{2} \rho_s I_{tsu} C_d C_{cx} B\left(hu^2\right) \tag{15.13}$$

In the equation provided, the drag coefficient for the building, denoted as C_d and specified in Table 15.4.

Table 15.4: Drag Coefficients for Rectilinear Structures

Width to Inundation Depth[a] Ratio B/h_{sx}	C_d Drag Coefficient
<12	1.25
16	1.3
26	1.4
36	1.5
60	1.75
100	1.8
≥ 120	2

[a] The inundation depth for each of the three specified Load Cases of inundation in Section 15.3.3.1 shall be determined. In the case of intermediate values of the width to inundation depth ratio (B/hs), interpolation should be employed.

Additionally, C_{cx} is determined as follows:

$$C_{cx} = \frac{\sum\left(A_{col} + A_{wall}\right) + 1.5 A_{beam}}{Bh_{sx}} \tag{15.14}$$

A_{col} and A_{wall} are represented the vertical projected areas of individual column and wall elements, respectively. A_{beam} is the combined vertical projected area of the slab edge facing the flow and the deepest beam exposed laterally to the flow. The summation of these column, wall, and beam areas is divided by the overall building wall area, which is calculated as the product of the width B and the average of the story heights, h_{sx}, above and below each level, for each story below the tsunami inundation height. This calculation is performed for each of the three specified Load Cases in Section 15.3.3.1. Any wall, whether structural or nonstructural, that is not designated as a tsunami breakaway wall should be included in the A_{wall} calculation. The value of C_{cx} should not be less than the closure ratio specified in Section 15.3.5, but it is not required to exceed 1.0.

15.4.2.2.2. Drag Force on Components

The lateral hydrodynamic load, as defined by Eq. 15.15, should be exerted as a pressure resultant on the projected inundated height, he, of all structural components and exterior wall assemblies located below the specified inundation depth.

$$F_d = \frac{1}{2}\rho_s I_{tsu} C_d b\left(h_e u^2\right) \tag{15.15}$$

For interior components, the C_d values specified in Table 15.5 should be employed, while b represents the width of the component perpendicular to the flow. In the case of exterior components, a C_d value of 2.0 must be utilized, and the width dimension b should be determined by multiplying the tributary width by the closure ratio value stated in Section 15.3.5. It is important to note that the drag force on individual component elements should not be included in the overall drag force calculation performed in Section 15.4.2.2.1.

15.4.2.2.3. Tsunami Loads on Vertical Structural Components

The force F_w acting on vertical structural components shall be determined by calculating the hydrodynamic drag forces using Eq. 15.16.

$$F_w = \frac{1}{2}\rho_s I_{tsu} C_d b\left(h_e u^2\right) \qquad 15.16$$

In cases where a tsunami bore flows with a Froude number exceeding 1.0 at the specific site, and where individual wall, wall pier, or column components possess a width to inundation depth ratio of 3 or higher, the force F_w shall be computed using Eq. 15.17. This force F_w is to be applied to all vertical structural components that their width exceed 3 times the inundation depth associated with Load Case 2 during inflow, as outlined in Section 15.3.3.1.

$$F_w = \frac{3}{4}\rho_s I_{tsu} C_d b\left(h_e u^2\right)_{bore} \qquad 15.17$$

Table 15.5: Drag Coefficients for Structural Components

Structural Element Section	Drag Coefficient *Cd*
Round column or equilateral polygon with six sides or more	1.2
Rectangular column of at least 2:1 aspect ratio with longer face oriented parallel to flow	1.6
Triangular pointing into flow	1.6
Freestanding wall submerged in flow	1.6
Square or rectangular column with longer face oriented perpendicular to flow	2
Triangular column pointing away from flow	2
Wall or flat plate, normal to flow	2
Diamond-shape column, pointed into the flow (based on face width, not projected width)	2.5
Rectangular beam, normal to flow	2
I, L, and channel shapes	2

15.4.2.2.4. Hydrodynamic Load on Perforated Walls

For walls that feature openings enabling the flow to pass between wall piers, the force exerted on the elements of the perforated wall, denoted as F_{pw}, may be calculated using Eq. 15.18. However, it should be noted that F_{pw} should not be less than the force F_d calculated according to Eq. 15.15:

$$F_{pw} = \left(0.4C_{cx} + 0.6\right)F_w \qquad 15.18$$

15.4.2.2.5. Walls Angled to the Flow

For walls that are positioned at an angle less than 90° relative to the flow directions described in Section 15.3.3.1, the transient lateral load per unit width, denoted as $F_{w\theta}$, must be calculated following Eq. 15.19.

$$F_{w\theta} = F_w \sin^2\theta \qquad 15.19$$

Where θ represents the angle enclosed between the wall and the direction of the flow.

15.4.2.2.6. Loads on Above-Ground Horizontal Pipelines

Horizontal pipelines located above the ground level, which are essential for the operation of Tsunami Risk Category III and IV buildings and other structures, must be designed to withstand the loads specified in Sections 15.4.2.2.6.1 and 15.4.2.2.6.2.

15.4.2.2.6.1. Hydrodynamic Loads on Above-Ground Horizontal Pipelines

Horizontal force per unit length needs to be computed in accordance with:

$$F_{rp} = C_{cp} C_r \rho_s I_{tsu} D_p u^2 \qquad 15.20$$

Where

C_{cp} = 1.5;

C_r = Pipe resistance coefficient as given in Table15.6

ρ_s = Minimum fluid mass density for design hydrodynamic loads;

I_{tsu} = Importance Factor;

D_p = Pipe diameter; and

u = Tsunami flow velocity.

Table 15.6: Pipe Resistance Coefficient, Cr .

Froude number, Fr	Resistance Coefficient, *Cr*
$0 < F_r < 0.25$	3.6
$0.25 < F_r < 1.3$	$4.22 - 2.48\,F_r$
$1.3 < F_r < 2.75$	1

The vertical force per unit length, acting in the upward direction, shall be computed as follows:

$$F_{l+} = C_{cp} C_l^+ \rho_s I_{tsu} D_p u^2 \qquad 15.21$$

where

C_{cp} = 1.5,

C_l^+ = Upward lift coefficient as given in Table 15.7

ρ_s = Minimum fluid mass density for design hydrodynamic loads,

I_{tsu} = Importance Factor,

D_p = Pipe diameter, and

u = Tsunami flow velocity.

Table 15.7: Upward Lift Coefficient, C_l^+ þ for Pipelines

Froude number, *Fr*	Upward Lift Coefficient, C_l^+
$0 < F_r < 0.25$	2.8
$0.25 < F_r < 1.3$	$3.23 - 1.71\,F_r$
$1.3 < F_r < 2.75$	1

The vertical force per unit length, acting in the downward direction, shall be determined as:

$$F_{l-} = C_{cp} C_l^- \rho_s I_{tsu} D_p u^2 \qquad 15.22$$

Where

C_{cp} = 1.5,

C_l^- = Downward lift coefficient as given in Table 15.8

ρ_s = Minimum fluid mass density for design hydrodynamic loads,

I_{tsu} = Importance Factor,

D_p = Pipe diameter, and

u = Tsunami flow velocity.

Table 15.8: Downward Lift Coefficient, C_l^-, for Pipelines.

Froude number, *Fr*	**Downward Lift Coefficient, C_l^-**
$0 < F_r < 0.25$	-2.8
$0.25 < F_r < 1.3$	$2.19 F_r - 3.35$
$1.3 < F_r < 2.75$	-0.5

15.4.2.2.6.2. Debris Impacts on Above-Ground Horizontal Pipelines

Debris impact loads on above-ground horizontal pipelines shall be in accordance with Section 15.4.3.

15.4.2.3. Hydrodynamic Pressures Associated with Slabs

The hydrodynamic pressures on the slabs shall be determined properly.

15.4.3. DEBRIS IMPACT LOADS

The determination of debris impact loads shall be carried out following the guidelines specified in this section. All buildings and other structures that satisfy the criteria outlined in table 15.9 must be designed to withstand the impact caused by floating wood poles, logs, and vehicles, as well as tumbling boulders and concrete debris. It is important to note that these loads do not need to be combined with other tsunami-related loads as determined in other sections of this regulation.

If a site is situated near a port or container yard, the potential for strikes from shipping containers, ships, and barges should be evaluated using appropriate methods documented in valid technical references. Inundation depths and velocities associated with Load Cases 1, 2, and 3, as defined in Section 15.3.3.1, should be considered. It is not necessary to apply impact loads simultaneously to all structural components affected by these potential strikes.

Table 15.9: Mandatory conditions to consider debris impact load in building design

Debris description	All buildings and other structures	Inundation depth limit
Floating wood poles, logs, and vehicles	*	1 m
Tumbling boulders and concrete debris	*	2 m
Shipping containers	*	1 m
Ships and barges	**	3.7 m

* All buildings and other structures described in paragraph 15.1.1

** Tsunami Risk Category I, and Tsunami Risk Category II Critical Facilities in an impact zone

15.4.3.1.Alternative Simplified Debris Impact Static Load

Instead of calculating the detailed debris impact loads caused by floating wood poles, logs, vehicles, tumbling boulders and concrete, and shipping containers, debris impact can be considered by applying the maximum static load given by Eq. 15.23.

$$F_i = 1470\ C_0 I_{tsu} \tag{15.23}$$

where F_i is debris impact design force and C_0 is the orientation coefficient, equal to 0.65.

This force should be applied at critical locations for flexure and shear for all relevant members within the inundation depth corresponding to Load Case 3, as specified in Section 15.3.3.1.

In cases where the site is not located near the port, or it can be shown with more detailed analysis that the site is not in an impact zone for shipping containers, ships, and barges, it is permissible to reduce the simplified debris impact force to 50% of the value specified by Eq. 15.23.

If the detailed debris impact loads calculations are used in determining the impact loads caused by floating wood poles, logs, vehicles, tumbling boulders and concrete, and shipping containers, the impact load should not be considered more than the value determined by Eq. 15.23.

15.4.4. FOUNDATION DESIGN

Tsunami flow affects and damages the foundation of the building in two main ways:

-Erosion and scour
-Pore pressure changes
The conditions of the bed, foundation and structure are different in each of these two cases, and the way to deal with the issue in each case will depend on the characteristics and scope of the effect of the tsunami phenomenon.
Design of structure foundations and tsunami barriers shall provide resistance to the loads and effects of Section 15.4.4.4, shall provide capacity to support the structural load combinations defined in Section 15.3.3.1, and shall accommodate the displacements determined in accordance with Section 15.4.4.4.5. Foundation embedment depth and the capacity of the exposed piles to resist structural loads, including grade beam loads, shall both be determined taking into account the cumulative effects of general erosion and local scour.

15.4.4.1. Scour and Erosion

Tsunami flow can move earth materials around the foundation and create a hole under or around it due to passing with speed or falling over the obstacle objects. This reduces the bearing capacity of the foundation.
Estimating the intensity of scour effects forimportant buildings, shelters and Tsunami Risk Category II Critical Facilities (clauses B and C) is necessary to maintain their performance against tsunami.
Based on the characteristics of scour and the type of fluid flow, three mechanisms of local scour, channelized scour and plunging scour have been observed during the tsunami event.

15.4.4.2. Pore pressure changes

The pore pressure of the soil increases due to the sudden formation of the tsunami wave. This increase in pore pressure reduces the bearing capacity of the foundation.

15.4.4.3. Resistance Factors for Foundation Stability Analyses

To calculate resistant capacities the resistance factor of ϕ shall be assigned in accordance with the geotechnics chapter of this regulation and the Iranian National Building Regulations for use with stability analyses and for potential failures associated with bearing capacity, lateral pressure, internal stability of geotextile and reinforced earth systems, and slope stability, including drawdown conditions. A resistance factor shall also be assigned for the resisting capacities of uplift resisting anchorage elements.

15.4.4.4. Load and Effect Characterization

Foundations and tsunami barriers must be designed to accommodate the following factors: lateral earth pressure, as outlined in chapter 5 of this regulation and chapter 7 of the Iranian National Building Regulations; hydrostatic forces, calculated according to Section 15.4.1; hydrodynamic loads, computed in accordance with Section 15.4.2; and uplift and underseepage forces, determined as per Section 15.4.4.1.

The foundations should be capable of withstanding uplift and overturning caused by hydrostatic, hydrodynamic, and debris loads exerted on the building superstructure due to tsunamis. Additionally, the impact of soil strength reduction, general erosion, and scouring should be taken into account, following the requirements specified in this section. The effects of a minimum of two wave cycles, as outlined in 15.3.6, should be considered for these considerations.

15.4.4.4.1. Uplift and Underseepage Forces

Evaluation of tsunami uplift and underseepage forces should be conducted in accordance with the procedures outlined in this section. The following considerations apply.

1. Uplift and underseepage forces must encompass the three designated inundation Load Cases specified in Section 15.3.3.1.
2. The assessment should account for the Strength loss resulting from scour and other soil-related phenomena such as liquefaction and pore pressure softening. Moreover, the foundation's susceptibility to uplift and underseepage forces should be determined in the following scenarios:
   a. When the soil is expected to be saturated prior to the tsunami.
   b. When soil saturation is predicted to occur during the incoming sequence of tsunami waves.
   c. When the concerned area is anticipated to remain inundated after the tsunami.
3. It is important to note that the uplift resistance should not consider the effect of live load or snow load.

15.4.4.4.2. Loss of Strength

The reduction in shear strength due to pore pressure softening caused by tsunamis must be taken into consideration within a depth of 1.2 times the maximum inundation depth. However, it is not necessary to account for tsunami-induced pore pressure softening in areas where the maximum Froude number is below 0.5.

15.4.4.4.3. General Erosion

The occurrence of general erosion during the runup and drawdown phases of tsunami inundation should be taken into account.

15.4.4.4.4 Scour

In general, to estimate the effects of scouring, physical modeling and estimation of the consequences of this phenomenon is necessary. However, due to the impossibility of performing these measures in all cases, approximate methods such as 15.4.4.4.4.1 and 15.4.4.4.4.2 can be used.
It is important to note that scour evaluation is not required for nonerodible strata, such as rock, that effectively prevent scouring caused by tsunami flow velocities of 9 m/s. Additionally, it is not necessary to assess scour for open structures.

15.4.4.4.4.1. Sustained Flow Scour

The phenomenon of sustained flow scour, which encompasses the effects of continuous flow around structures and includes building corner piles, must be taken into account. The design depth and extent of the area affected by sustained flow scour should be determined through dynamic numerical or physical modeling, or by employing empirical methods found in reputable literature. Alternatively, it is permissible to assess sustained flow scour and the accompanying pore pressure softening by referring to the guidelines provided in Table 15.10.

Table 15.10: Design Scour Depth Caused by Sustained Flow and Pore Pressure Softening

Inundation Depth h	Scour Depth *Da**
< 3m	1.2 h
> 3m	3.7

* Not applicable to scour at sites with intact rock strata.

The local scour depth resulting from sustained flow, as provided in Table 15.10, may be reduced by applying an adjustment factor in areas where the maximum flow Froude number is less than 0.5. This adjustment factor should linearly vary from 0 at the horizontal inundation limit to 1.0 at the point where the Froude number reaches 0.5. For the purpose of analysis, the assumed area limits should encompass the exposed building perimeter and extend on either side of the foundation perimeter. The extension distance should be equal to the scour depth for consolidated or cohesive soils, and three times the scour depth for nonconsolidated or noncohesive soils.

15.4.4.4.4.2. Plunging Scour

The horizontal extent and depth of plunging scour should be determined using dynamic numerical or physical modeling techniques, or empirical methods. The parameters for plunging scour are depicted in Fig. 15.5. If site-specific dynamic modeling and analysis are not available, the plunging scour depth (D_s) can be calculated using Eq. 15.24.

$$D_s = C_{2V}\sqrt{\frac{qU\sin(\psi)}{g}} \qquad 15.24$$

Where

C_{2V} = Dimensionless scour coefficient, permitted to be taken as equal to 2.8;

ψ = Angle between the jet at the scour hole and the horizontal, taken as the lesser value of 75° and the side slope of the overtopped structure on the scoured side, in the absence of other information;

g = Acceleration caused by gravity;

q= Discharge per unit width over the overtopped structure, as illustrated in Fig. 15.5 and calculated in accordance with Eq. 15.25,

U= Jet velocity approaching the scour hole, obtained in accordance with Eq. 15.26, and

C_{dis} = a dimensionless discharge coefficient obtained in accordance with Eq. 15.27.

$$q = C_{dis}\frac{2}{3}\sqrt{2g}\,H_0^{3/2} \qquad 15.25$$

U represents the jet velocity as it approaches the scour hole, which is a result of the vertical drop between the upstream water surface height (h) and any additional elevation difference (d_d) on the side where scouring occurs, as specified by Eq. 15.26.

$$U = \sqrt{2g\left(h + d_{\mathrm{d}}\right)} \qquad 15.26$$

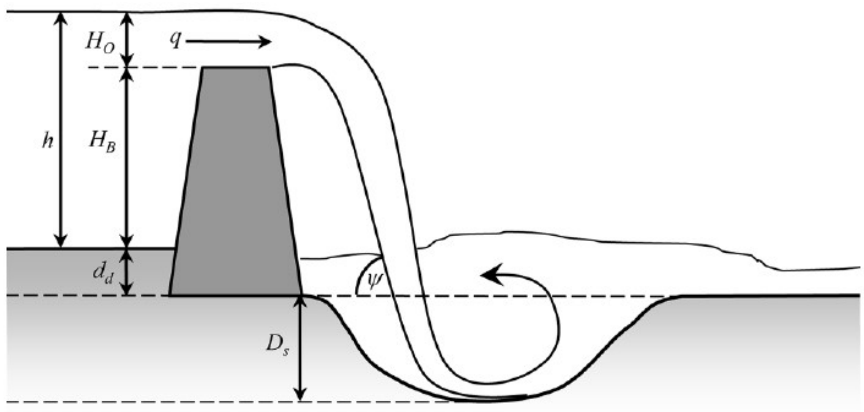


Figure 15.5: Plunging scour parameters

where d_d is the additional elevation difference between the upstream and scouring sides of the structure, as illustrated in Fig. 15.5.
And dimensionless discharge coefficient

$$C_{dis} = 0.611 + 0.08\frac{H_0}{H_B} \qquad 15.27$$

15.4.4.4.4.3. Displacements

The vertical and horizontal displacements of foundation elements and slope displacements must be determined by employing empirical, elastoplastic analytical, or numerical methods found in reputable literature. These methods should be applied while considering the tsunami loads determined in Section 15.4.4.4 together with other applicable geotechnical and foundation loads in accordance with the Iranian National Building Regulations, especially chapter 7.

15.4.5. STRUCTURAL COUNTERMEASURES FOR TSUNAMI LOADING

The structural effects of tsunamis can be mitigated by implementing the following countermeasures, which are permitted.

15.4.5.1. Open Structures

Load Case 1 of Section 15.3.3.1 does not apply to Open Structures. However, the load impact resulting from debris accumulation against or within the Open Structure should be assessed by considering a minimum closure ratio of 50% of the inundated projected area along the perimeter of the Open Structure. This evaluation ensures the effects of debris accumulation are properly considered.

15.4.5.2. Tsunami Barriers

Tsunami barriers utilized as an external perimeter structural countermeasure must be designed in a manner that aligns with the performance objectives of the protected structure. This design approach aims to collectively meet the specified performance criteria. These criteria encompass various aspects such as barrier strength, stability, and protection against slope erosion, prevention of toe scour, geotechnical stability requirements, as well as barrier height and footprint to ensure complete prevention of inundation during the Maximum Considered Tsunami.

In cases where a barrier is designed to be overtopped by the design event or intends to offer only partial impedance to the design event, the protected structure and its foundation must be designed to withstand the residual inundation resulting from the design event. Furthermore, the foundation system treatment requirements outlined in Section 15.4.4 of this chapter should also be applied.

15.4.5.2.1. Site Layout

The spatial limits defining the arrangement of tsunami barriers should adhere to the following guidelines:

1. To provide perimeter protection for the protected structure, the tsunami barrier should be positioned at a distance from the structure.

Any alteration in alignment should have a minimum radius of curvature equal to or greater than half the maximum inundation depth.
2. For overtopping or partial impedance to inundation, at a minimum the barrier limits shall protect the structure from inundation flow based on an approach angle of 22.5 degrees from the shoreline.

15.4.6. TSUNAMI VERTICAL EVACUATION REFUGE STRUCTURES

Tsunami Vertical Evacuation Refuge Structures, which serve as alternative means of evacuation with the approval of competent authorities, must be designed in accordance with the supplementary requirements outlined in this section.

15.4.6.1. Minimum Inundation Elevation and Depth

The tsunami refuge floors should be positioned at a minimum vertical distance that is the greater of 3 m or one-story height above 1.3 times the Maximum Considered Tsunami inundation elevation at the specific site, as depicted in Fig. 15.6. Determination of the Maximum Considered Tsunami inundation elevation should be carried out in compliance with section 15.2.

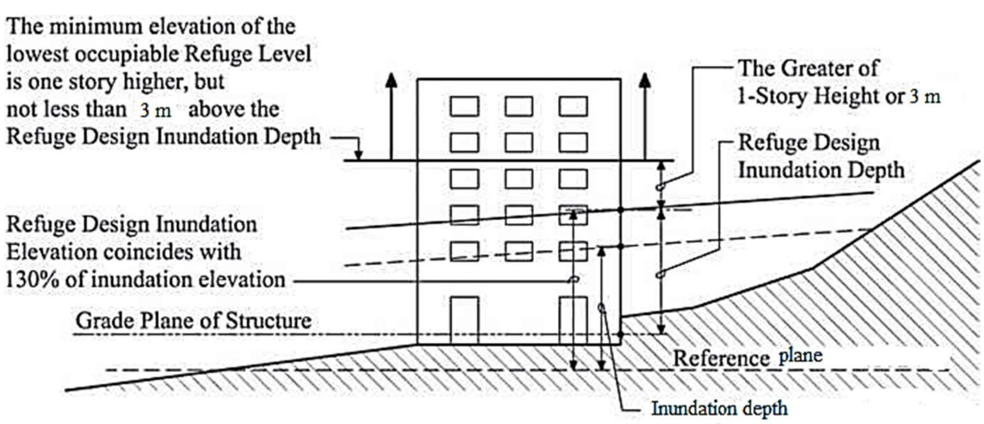


Figure 15.6 Minimum refuge level elevation

15.4.6.2. Refuge Live Load

In any designated evacuation floor area within a tsunami refuge floor level, a Refuge Live Load of 5 kPa, along with other applicable live loads, must be applied.

15.4.6.3. Laydown Impacts

When the maximum inundation depth exceeds 2 m, it is necessary to take into account the potential impact of adjacent pole structures collapsing onto occupied areas of the building.

15.4.6.4. Information on Construction Documents

The construction documents must incorporate the tsunami design criteria and provide information about the occupancy capacity of the tsunami refuge area. The floor plans should clearly indicate all refuge areas within the facility and the corresponding exit routes from each area. Additionally, the construction documents should include the latitude and longitude coordinates of the building for precise location reference.

15.4.7. DESIGNATED NONSTRUCTURAL COMPONENTS AND SYSTEMS

Designated nonstructural components and systems (as defined in 15.1.1) and located within structures in the Tsunami Design Zone must either be protected from the effects of tsunami inundation or positioned above the inundation elevation of the Maximum Considered Tsunami. This positioning ensures that these designated nonstructural components and systems can fulfill their critical function both during and after the occurrence of the Maximum Considered Tsunami.

Alternatively, it is permissible to directly design the designated nonstructural components and systems to withstand tsunami effects, provided that the inundation does not hinder their ability to perform their critical function during and after the Maximum Considered Tsunami.

15.4.8. NONBUILDING TSUNAMI RISK CATEGORY I AND II STRUCTURES

15.4.8.1. Requirements for Tsunami Risk Category I Nonbuilding Structures

Tsunami Risk Category I designated nonstructural systems in nonbuilding structures situated within the Tsunami Design Zone must comply with the following provisions:

(1) They should be protected from the impacts of tsunami inundation.
(2) Alternatively, they should be positioned at a level above 1.3 times the inundation elevation of the Maximum Considered Tsunami in a manner that ensures the Tsunami Risk Category I nonbuilding structure can perform its critical function during and after the Maximum Considered Tsunami.
(3) Another option is to design these systems to withstand the effects of tsunami loads according to the requirements outlined in Section 15.3.1 of this regulation, as well as the specific performance criteria specified in that section.
For tsunami barriers utilized as protection against inundation, the top-of-wall elevation should be equal to or greater than 1.3 times the maximum inundation elevation at the barrier. Additionally, these tsunami barriers must satisfy the requirements in Section 15.4.5.

15.4.8.2. Requirements for Tsunami Risk Category II Nonbuilding Structures

Tsunami Risk Category II nonbuilding structures situated within the Tsunami Design Zone must adhere to the following guidelines:
(1) They should be protected from the impacts of tsunami inundation, or
(2) They should be designed to withstand the effects of tsunami loads as specified in Section 15.3.2 of this regulation, while meeting the specific performance criteria outlined in that section.
For tsunami barriers utilized for protection against inundation, the top-of-wall elevation must be equal to or greater than 1.3 times the maximum inundation elevation at the barrier. Additionally, these tsunami barriers must fulfill the requirements stated in Section 15.4.5.

Appendix 1

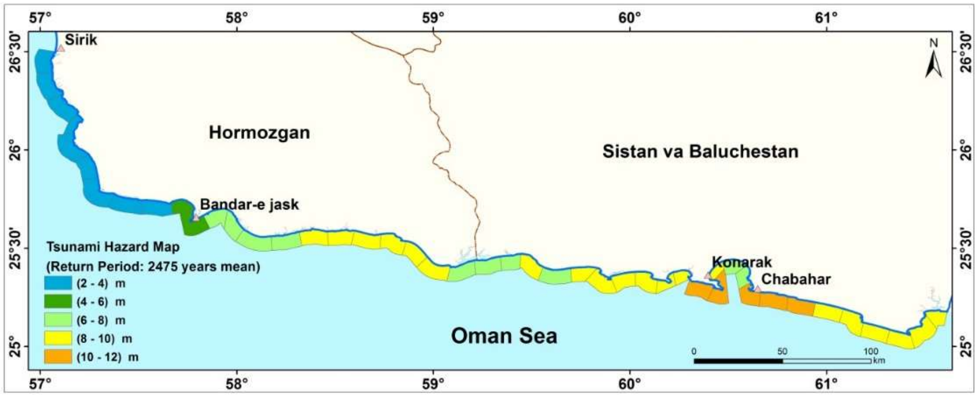


Figure A1: Uniform hazard plans for western Makran corresponding to a return period of 2475 years (Mean level)

Table A1: Wave height values in Makran coastline

Sector number	Coordinates of the beginning of the sector		Coordinates of the end of the sector		Wave height on the coastline (m)
	latitude (degrees)	longitude (degrees)	latitude (degrees)	longitude (degrees)	
1	26.5	57.08	26.39	57.06	2
2	26.39	57.06	26.28	57.1	2
3	26.28	57.1	26.18	57.19	2.5
4	26.18	57.19	26.08	57.18	2.5
5	26.08	57.18	25.97	57.25	2.5
6	25.97	57.25	25.86	57.29	2.5
7	25.86	57.29	25.77	57.37	2.5
8	25.77	57.37	25.74	57.51	2.5
9	25.74	57.51	25.74	57.66	3.5
10	25.74	57.66	25.69	57.78	5
11	25.69	57.78	25.66	57.82	5
12	25.66	57.82	25.69	57.95	5.5
13	25.69	57.95	25.59	58.03	6
14	25.59	58.03	25.56	58.17	7
15	25.56	58.17	25.58	58.31	7.5
16	25.58	58.31	25.58	58.45	8
17	25.58	58.45	25.59	58.59	8.5
18	25.59	58.59	25.57	58.73	8
19	25.57	58.73	25.53	58.86	8
20	25.53	58.86	25.45	58.96	8
21	25.45	58.96	25.41	59.08	8
22	25.41	59.08	25.43	59.2	7.5
23	25.43	59.2	25.45	59.33	7.5
24	25.45	59.33	25.47	59.44	7.5
25	25.47	59.44	25.4	59.55	8

Table A1 (Continued): Wave height values in Makran coastline

Sector number	Coordinates of the beginning of the sector		Coordinates of the end of the sector		Wave height on the coastline (m)
	latitude (degrees)	longitude (degrees)	latitude (degrees)	longitude (degrees)	
26	25.4	59.55	25.39	59.69	7.5
27	25.39	59.69	25.39	59.83	8.5
28	25.39	59.83	25.35	59.9	8.5
29	25.35	59.9	25.38	60.02	9.5
30	25.38	60.02	25.34	60.14	9
31	25.34	60.14	25.37	60.21	8.5
32	25.37	60.21	25.34	60.3	9.5
33	25.34	60.3	25.3	60.42	10
34	25.3	60.42	25.33	60.42	10
35	25.33	60.42	25.43	60.46	9
36	25.43	60.46	25.41	60.59	6
37	25.41	60.59	25.32	60.61	6.5
38	25.32	60.61	25.27	60.66	10
39	25.27	60.66	25.25	60.8	10.5
40	25.25	60.8	25.22	60.94	10.5
41	25.22	60.94	25.19	61.08	8.5
42	25.19	61.08	25.12	61.18	8.5
43	25.12	61.18	25.08	61.32	9
44	25.08	61.32	25.08	61.41	9
45	25.08	61.41	25.16	61.5	8
46	25.16	61.5	25.17	61.61	8

Explanation:

a) For points on the border of two sectors, the average values of both sides should be used.
b) The coordinates of the beginning and end of each sector are given.

Chapter 16
Piping Systems

16.1. Introduction

This chapter covers the seismic design of centralized piping networks included in Chapter 8 and some accessory components of pipeline systems included in Chapter 13.

16.2. Definitions

The technical terms used in this chapter are defined below.

Pipe: A conduit with a circular cross-section that is used in the industry to transport liquid and gas.

Flange: A device that connects piping sections by use of bolted connections and gaskets.
Control valve: Control devices that are placed on the pipes and are responsible for turning the flow off and on.

Expansion connection: Connections that absorb the relative displacements of the system.

Nozzle: Entrance-exit sections of the pipings.

Special pipes: Pipes that are categorized under the hazard group I and are subject to the criteria of Section 16.8 and characterized with their special type of contents, diameter, volume between two control valves and the distance from the boundary of the industrial complex.

Guide: supports that are placed at a short distance from the pipes and while providing limited movement capability during operational loads such as thermal ones, prevent unconventional movement along the constraint.

16.2.1. Symbols

D : Diameter of the circle corresponding to the inner face of the sealing plate

d : Inner diameter of the pipe

D_e : Diameter of the circle corresponding to the center of the contact surface of the sealing plate

D_i : Inner diameter at the valve critical section

D_o : External diameter at the valve critical section location

F : Axial force

F_{MH} : Seismic force imposed to the center of mass

H : Distance between the center of mass and the connection point of the upper part of the control valve to its lower base

L_b : Distance between the center of mass and the critical section at the neck of the control valve

M : Bending moment

N : Width of the sealing plate

P : Internal pressure of pipe

P_e : Pressure equivalent to stress caused by gravity loads, seismic inertia and seismic displacement

P_{eq} : Equivalent stress in flanges

S : Allowable design stress

Z : Critical section modulus

β : Seismic amplification coefficient of the control valve relative to the pipe

σ_n : The design stress created at the critical point

σ_L : The stress in the critical section due to the internal pressure and reaction of the control devices inside the control valve.

16.3. Scope

Piping systems resting on a support structure with a total weight less than 20% of the weight of the entire pipe system and the support structure are modeled according to the criteria of this chapter. In this case, individual modeling and analysis of the piping system apart from the support structure is warranted,. For larger weight ratios, it is required to model the piping system and the supporting structure altogether according to Chapter 7. The criteria presented in this chapter can be used for the materials mentioned in Table 8.3.

16.4. Design earthquake and performance level

Design earthquake of the piping system corresponds to the second hazard level introduced in Chapter 3. The expected performance levels for piping system vary depending on the importance and required ductility. In general, brittle pipes and special pipes are expected to have linear or near-linear performance in the design earthquake. Moreover, consistent with the adopted R_{po} factor according to Table 8.3, other types of pipes may experience different levels of nonlinear deformation.

16.5. Load combinations and allowable stresses

For the pipes covered by Chapter 8, the design load combinations are according to those introduced in Chapter 2 for the allowable stress method. As an exception, for design of the control valves, the criteria of Section 16.6.3 should be followed. Allowable stresses for pipes, regardless of their attachments, are mentioned in Section 16.10. For control valves, the allowable stresses are according to Section 16.6.3. For other components of the piping system, the allowable atresses may be obtained from valid references. Calculation of the inertial forces of pipes shall be done according to the equations of Section 8.3.1. In determining the seismic force on different components of the piping system, including flange connections, expansion joints, nozzles, and supports, it is necessary to include the overstrength factor according to Table 8.3. In addition, for control valves, it is necessary to include the acceleration correction coefficients according to Table 16.2. It should be noted that if in checking adequacy of the connections the manufacturer has introduced special requirements for seismic behavior, it should be taken into consideration in addition to the criteria mentioned in this section.

16.6. Seismic design of the piping system and its components

In this section, issues related to seismic design of pipe, flange, control valve, expansion joint, nozzle and support are described. In the seismic design of pipes and their components, the earthquake loads are applied in two horizontal orthogonal directions separately, each one along with the vertical component.

16.6.1. The piping system

Seismic design of pipes is carried out according to load combinations and allowable stresses presented in Section 16.5 and the analysis criteria described in Section 16.9. It is recommended that for all pipe systems that are designed according to this section, the highest design demand-capacity ratio of stresses occurs in the elbows. Compliance with this requirement is mandatory for special pipes.

16.6.2. Seismic design of flange

The equivalent stress in the flanges using the design loads shall be calculated based on Eq. 16.1:

$$P_{eq}=P+P_e \tag{16.1}$$

In this equation, P is the internal pressure of the pipe and P_e is the pressure equivalent to the stress caused by the gravity loads, seismic inertia and seismic displacement, which is calculated according to the following equation using the forces obtained from the analysis:

$$P_e=4F/(\pi D_e^2)+ 16M/(\pi D_e^3) \tag{16.2}$$

where M and F are the moment and axial force, respectively, and D_e is the diameter of the circle corresponding to the center of the contact surface of the sealing plate (twice the distance from the center of the contact surface of the sealing plate to the center of the pipe section) according to Fig. 16.1.

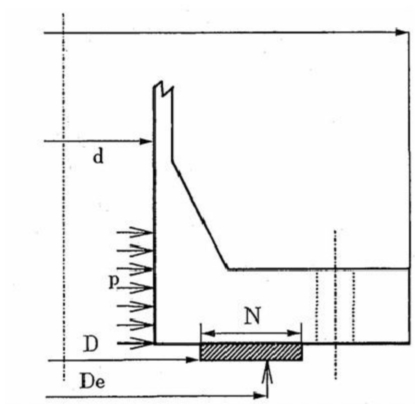


Figure 16.1. A typical flange section labeled with the related dimension parameters

In Fig. 16.1, N, D and d are the width of the sealing plate, the diameter of the circle corresponding to the inner face of the sealing plate (twice the distance from the inner face of the sealing plate to the center of the pipe) and the inner diameter of the pipe. In addition, P is the internal pressure of the pipe.

UDetails of using P_{eq} for calculating the radial, tangential and longitudinal stresses along with values of the allowable stresses can be obtained from valid references.

16.6.3. Seismic design of control valves

In the seismic design of control valves, the seismic inertial force on the upper head, in addition to the stresses caused by the internal pressure and the imposed force from their movable control part, according to Table 16.1, are used in the load combination.

Table 16.1. Load combination applicable in seismic design of control valves

Stress location	Load Combination		
	Inertial force	Pressure	Controller induced force
Valve neck	×	×	×

Figure 16.2 shows a side view of a typical control valve and the seismic inertial force acting on its center of valve-head mass. In this figure, L_b is the distance between the center of mass of the upper part of the control valve and the possible critical section below it, and H is the distance between the center of mass and the connection point of the upper part of the control valve to its lower base. D_o and D_i are the sizes of the outer and inner diameters at the critical section, respectively. Moreover, F_{MH} is the seismic force at the center of mass.

Tension in the critical section is calculated from the following equation:

$$\sigma_n = \frac{F_{MH}\, L_b}{Z} + \sigma_L \qquad 16.3$$

where σ_n is the total stress at the critical section, σ_L is the stress atthe critical section due to the internal pressure plus reaction of the control devices inside the control valve and Z is the section modulus of the critical section. In addition, F_{MH} is calculated from the following equation:

$$F_{MH}=\beta\, F_p \qquad 16.4$$

where F_p is calculated from Eq. 8.1 and β is obtained from Table 16.2.

In Table 16.2, D is considered equal to D_o. The allowable stresses used in the design of control valves are according to Table 16.3.

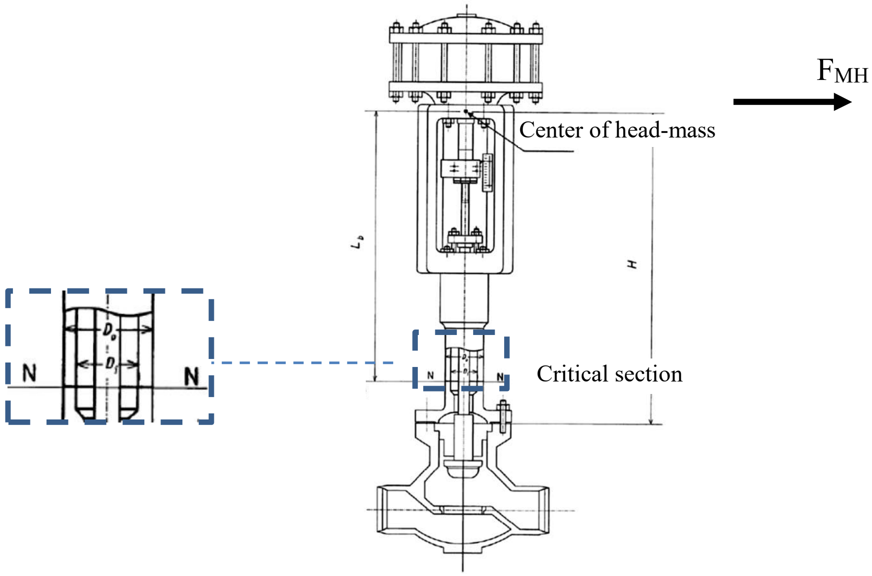


Fig. 16.2. A typical control valve labeled with dimension parameters

Table 16.2. Magnification factor of the acceleration induced at the control valve relative to the attached pipe

Support condition	**Criteria**	**Natural frequency**	Magnification factor β
Additionally supported valves		20 Hz and above	1
No support other than pipe	$\frac{H}{\sqrt{D}} \leq 40$		1
	$40 < \frac{H}{\sqrt{D}} \leq 60$	Less than 20 Hz	$0.1\frac{H}{\sqrt{D}} - 3$
	$60 < \frac{H}{\sqrt{D}}$		3

Table 16.3. Allowable stresses

Valve Type	**Allowable stress**
Valves supported by special pipes	0.5 S
Other Valves	S

In Table 16.3, S is the allowable stress according to Section 16.10.

16.6.4. Seismic design of expansion joints

The load combinations and allowable stresses used in seismic design of expansion joints are according to the general criteria of Section 16.5. Moreover, calculation of the stress imposed on the connection can be done based on valid references.

16.6.5. Seismic design of nozzles

The load combinations and the overstrength factor are according to the general rules of Section 16.5.

16.6.6. Seismic design of supports

The load combinations and the overstrength coefficients are according to the general rules of Section 16.6.

16.7. Strength reduction and overstrength factors

To determine the strength reduction factors for all pipe types, the values presented in Table 8.3 are applicable. As an exception, for the special pipes included in Section 16.8 and chapter 8, in addition to the criteria presented in that chapter, the calculations shall be repeated under the following assumptions and the more critical values shall be used for design.

1) R_{po} shall be taken 1 and 1.25 accoring to the toxcicity of the contents for groups A and B, respectively, and,
2) The importance factor I_p shall be taken as unity.

16.8. Special pipes

Pipes that satisfy all of the following criteria, are categorized as special pipes:
- Diameter: Pipes with an outer diameter greater than or equal to 45 mm,
- Volume: Volume of the contents is larger than or equal to 3 m^3 between the two successive control valves,
- Content type: Gases that fall under one of the two groups A and B as follows:

- Group A toxic gases: chlorine, hydrocyanide acid, nitrogen dioxide, fluorine and phosgene.
- Group B toxic gases: hydrogen chloride, boron trifluoride, sulfur dioxide, hydrogen fluoride, bromomethane, hydrosulphuric.
- Distance from the boundaries: boundary in industrial complexes refers to one or a combination of the following:
- Sea, lake, river and service water tanks used in the industrial complex
- Railway line for transporting products of the industrial complex
- The area reserved for future expansion of industrial facilities
- Public infrastructure
- Spaces used for commercial activities such as production areas, warehouses, etc.
- The routes connecting each pair of the boundaries mentioned above

According to the above definition, special pipes are classified into two "very critical" and "critical" groups within Table 16.4 in terms of impact on the environment in the case of damage.

16.9. Analysis Method

The criteria presented in this chapter are complementary to the approach presented in Section 8.3.1 and may not be used with the calculations included in Sections 8.3.2 and 8.3.3. It is necessary to satisfy the criteria of Section 8.3.4 regarding the reverse displacements of the consecutive supporting structures in the calculations. For this purpose, it is necessary to consider the displacements of successive supporting structures with different signs in the calculations. If it is intended to model a set of pipes and support structures in an integrated way, it is required to consider the same loading strategy in the modeling too. In addition, in the latter case, it is necessary to consider the effect of amplification of the ground surface acceleration as a result of seismic response of the supporting structure and the pipe according to Eq. 16.5.

$$a_p = 0.4 S_{DS} I_P \left[\frac{a_h}{R_b}\right]\left[\frac{a}{R_{po}}\right] \qquad 16.5$$

In this equation, a_p is the acceleration per unit length of the pipe. For both adjacent supports, regardless of whether they are located on one or two supporting structures, the acceleration of the two successive supports is considered to have the same sign, and a_p takes the average of the acceleration of the adjacent supports. Also, the criteria of Section 16.6 may be applied for the design of the piping components in this analysis approach.

If separation occurs between the pipe and the supporting seats under thermal loads, it is necessary to consider the effect of this separation in the seismic analysis and apply the boundary conditions accordingly. In the case where the guide supports are located with several millimeters of a gap distance at the pipe, it is necessary to not include the gap in the modeling and assume the boundary condition such that the pipe is clamped to the supporting structure along the guide constraints. If the distance between the guide and pipe is more than 6 mm, it is necessary to increase the seismic inertia force according to the criteria of Table 8.3.

In the seismic design of special pipes, in addition to the adequacy of design against earthquake, it is necessary to alleviate the maximum stresses in the elbows.

16.10. General notes

Except for the cases mentioned as exceptions, the piping system must be designed for the relative forces and displacements stipulated in Section 8.3. If flexible connections with the ability to tolerate the relative displacements are used in the pipe route, there is no need to design the pipe to withstand seismic relative displacements.

In cases where other material reference standards are not used, design of the piping system shall be done using the following allowable stresses:

a) For pipes made of ductile materials (steel, aluminum or copper), 90% of the minimum characteristic yield limit,

b) If threaded connections are used in pipes made of ductile materials, 70% of the minimum characteristic yield limit,

c) For pipes made of non-ductile materials (such as cast iron or ceramics), 10% of the minimum characteristic tensile strength,

d) In the case of using threaded connections in the pipes made of non-ductile materials, 8% of the minimum characteristic tensile strength.

Weight of the hanging equipments located in the path of the pipes and connected to them, such as valves, pumps, air separators and tanks, where they do not have separate lateral restraints, shall be considered in the design of the pipe restraints. If these equipments have independent restrainers, their connections to the the pipes shall have enough flexibility to withstand relative displacements.

Table 16.4. Criteria to distinguish special pipes

Content	**Distance to the boundary (m)**	**Weight of content (ton)**					
		Less than 5	Between 5 and 20	Between 20 and 30	Between 30 and 100	Between 100 and 500	500 and more
Toxic gas type A	Less than 100	Critical	Critical	Critical	Very critical	Very critical	Very critical
	Between 100 and 200	-	Critical	Critical	Very critical	Very critical	Very critical
	Between 200 and 500	-	-	Critical	Very critical	Very critical	Very critical
	Between 500 and 1000	-	-	-	Critical	Very critical	Very critical
	1000 and more	-	-	-	-	Critical	Very critical
Toxic gas type B	Less than 50	Critical	Critical	Critical	Very critical	Very critical	Very critical
	Between 50 and 200	-	Critical	Critical	Very critical	Very critical	Very critical
	Between 200 and 500	-	-	Critical	Very critical	Very critical	Very critical
	Between 500 and 1000	-	-	-	Critical	Very critical	Very critical
	1000 and more	-	-	-	-	Critical	Very critical

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Important Note: Finally, the ASCE7-22 standard is the reference.

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