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Ministry of Petroleum

Iranian Seismic Design Code for Petroleum Facilities & Structures

(3rd Edition)

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معاونت مهندسی، پژوهش و فناوری

جمهوری اسلامی ایران
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معاونین محترم وزیر و مدیران عامل شرکت‌های اصلی و فرعی

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موضوع: نسخه انگلیسی "آیین‌نامه طراحی لرزه‌ای تاسیسات و سازه‌های صنعت نفت"
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با سلام

پیرو ابلاغیه شماره ۱۸۴۴۲۱-۲۰/۲ مورخ ۹۵/۴/۲۷ وزیر محترم نفت و به منظور تدقیق طراحی لرزه‌ای تأسیسات و ساختمان‌ها و انطباق آن با شرایط پهنه‌بندی خطر لرزه‌ای کشور، بدینوسیله نسخه انگلیسی نشریه ۰۳۸ معاونت مهندسی، پژوهش و فناوری وزارت نفت با عنوان:

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به منظور استفاده در طرح‌ها و پروژه‌های صنعت نفت ارسال می‌دارد. خاطر نشان می‌گردد، نسخه الکترونیک مذکور از طریق پایگاه اطلاع رسانی اداره کل نظام فنی و اجرایی و ارزشیابی طرح‌ها به آدرس det-mop.ir قابل دریافت می‌باشد.

حبیب‌ا... بیطرف
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مدیران عامل محترم شرکت‌های اصلی و فرعی

معاونان محترم وزیر

موضوع: ابلاغ آیین‌نامه طراحی لرزه‌ای تاسیسات و سازه‌های صنعت نفت- ویرایش ۳

باسلام‌واحترام- به استناد بند «الف» از ماده (۳) قانون وظایف و اختیارات وزارت نفت، در راستای صیانت از تاسیسات و تجهیزات صنعت نفت و عمل به الزامات اقتصاد مقاومتی و حفظ محیط زیست و توسعه پایدار و به منظور یکسان‌سازی طراحی لرزه‌ای سازه‌ها و تاسیسات و انطباق آن با شرایط پهنه‌بندی خطر لرزه‌ای کشور، به پیوست «آیین‌نامه طراحی لرزه‌ای تاسیسات و سازه‌های صنعت نفت- ویرایش ۳»، برای استفاده در طراحی تاسیسات جدید و طرح‌های توسعه‌ای ابلاغ می‌گردد.

رعایت کامل مفاد این آیین‌نامه بدون جایگزینی بخشی از مفاد آن در سایر آیین‌نامه‌ها و بالعکس، از طرف مجریان طرح‌ها، مشاوران، پیمانکاران، سازندگان و عوامل دیگر الزامی است. چنانچه آیین‌نامه در مواردی فاقد ضوابط طراحی باشد، استفاده از سایر آیین‌نامه‌های معتبر درصورت عدم تناقض با مفاد این آیین‌نامه مجاز می‌باشد.

نسخه الکترونیکی آیین‌نامه از طریق تارنمای معاونت امور مهندسی به آدرس dea.mop.ir قابل دسترسی و دریافت می‌باشد.

بیژن زنگنه

Preface

With respect to Iran seismicity and probability of seismic damages especially for petroleum facilities and plants, in recent years, Iranian Ministry of Petroleum (MOP) - Department of Engineering has decided to establish a technical code for seismic design of petrochemical plants, refineries, pipelines, etc.
This is the third edition of this code. It has been written by a committee composed of Iranian structural and earthquake engineers and faculty members to be applied to the national petroleum facilities.
The first version of this governmental document was published in 2007 as a guideline of design practice for all of the major and minor companies in MOP. In 2010, three years after publishing the first version, the technical committee provided the second revision, which became mandatory. Finally, the third edition was prepared and published in 2015 and 2016, respectively. The Iranian Standard No. 2800 (National Standard for Seismic Design of Buildings) committee has also endorsed this code for industrial structures in its fourth edition.
In this revision, different chapter of the code have been revised according to the latest national and international regulations, guidelines and standards about seismic design of industrial structures and facilities. One chapter is now added to the code called "Chapter 10: Structures with Damping Systems". In addition, two appendices are allocated to the text and some technical and grammatical modifications and modifications have been applied.
The sincere efforts by all of the university professors, members of the technical committee, consulting engineers and staff of *Department of Engineering, Research and Technology* of MOP that resulted in codification of the developed manuscript, are appreciated. We are looking forward to hearing from consulting engineers, contractors, researchers and executive managers about the recommendations on the code and any error, infirmity in the text.
For any question, the telephone No. +98 (21) 888 10 456 is available. The electronic version of this document is available at det-mop.ir.

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Disclaimer

The material presented in this code, has been prepared in accordance with the recognized engineering principles. This code should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the material herein is not intended as a representation or warranty on the part of the Iranian Petroleum Standard (IPS) or any other person named herein, that this information is suitable for any general or particular use or promises freedom for infringement of any patent or patents. Anyone making use of this code, assumes all liability from such use.
This document is the English version of the Iranian Seismic Design Code for Petroleum Facilities and Structures, third Edition, 2016. If there seems to be any conflict or anomaly between this document and the original code, call this department as named above for probable explanations.

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Ministry of Petroleum (MOP) acknowledges the efforts of the committee for Iranian Seismic Design Code for Petroleum Facilities and Structures. The committee comprised individuals from many backgrounds, including consulting engineers, researchers, university faculty members, as listed below.

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Chapter 1
Introduction

1. Introduction

1.1 Objective

This code provides minimum requirements for seismic design of oil industry structures and equipment, (based on the scope mentioned in Section 1.2), such that consideration of which minimizes probability of interruption in efficiency and operation of the facilities in light and moderate earthquakes, and probability of extensive damages in strong earthquakes.
By following these requirements, the structure is expected to behave so that in light earthquakes, elements remain in the linear range and in moderate and strong earthquakes, depending on the importance of structure, damages are limited.

1.2 Scope

This code includes seismic design of essential buildings, non-building structures, and industrial equipment and components, as referred to in relevant chapters. In this code, three Seismic Hazard Levels (in accordance with Chapter 3) are considered; one or two of which, will be used relevantly.
Regulations for seismic evaluation of existing structures and equipment have not been developed in this code. For existing structures, Reference [1] could be used.

1.3 Design Basis

Seismic design criteria in this code are based on force method followed by displacement control, which is adequate to reach the objective (Section 1.1). The designer could use other valid methods such as performance-based design according to valid references. However, requirements of this code are mandatory.

1.4 Arrangement

This code is consists of 14 chapters and 3 appendices. The first five chapters contain general seismic design criteria including introduction, loading, seismic hazard analysis, analysis methods, and soil-structure interaction. Chapter 6 is dedicated to essential building. In Chapter 7, general and common regulations of non-building structures are presented. Seismic design criteria for Non-structural components including architectural components, and electrical and mechanical equipment are presented in Chapter 8. In cases of using base isolation, or damping systems, the provisions of Chapters 9 and 10 are necessary, respectively. In Chapters 11 to 14, specific seismic design criteria of some non-building structures such as chimney, storage tank, pipeline and offshore structure are presented.

1.5 Unit System

Although it has been tried to use non-dimensional equations valid in any unit system, when using the appropriate units is required due to existence of coefficients having dimensions, International System of units (SI) and subsidiary units are mentioned and used.

1.6 Abbreviations

Symbols and abbreviations used in this code are generally consistent with the other valid references. Symbols used in this code are presented here. In addition, it should be noted that in some cases, the same symbol has been used for several definitions.

A	Base design acceleration
A_B	Area of the base of structure
A_C	Tank convective mass seismic coefficient
A_f	Area of effective foundation
A_g	Maximum acceleration perpendicular to wave propagation direction for design earthquake
A_i	Tank impulsive mass seismic coefficient

A_k	The load or load effect resulting from extraordinary event A
A_p	Cross sectional area of pile or pipe
A_f	Cross sectional area of wall-base anchorage cable, strand or reinforcement in a tank
A_{si}	Web area of shear wall i in the direction of interest
A_v	Tank earthquake vertical acceleration coefficient
A_x	Torsional amplification factor in level x
a	Average dimension of the effective foundation in the direction of interest
a_i	Combination of modal accelerations at level i
a_j	Dynamic modification factor for the j^{th} DOF to determine soil radiation damping
a_p	Component amplification factor
a_0	Parameter for defining soil radiation damping
B	Response factor according to Reference [2]
B	Foundation width (smaller dimension)
B_D	Numerical coefficient related to the effective damping of the isolation system for design earthquake
B_g	Dimension of pile group
B_M	Numerical coefficient related to the effective damping of the isolation system for rare earthquake
B_{V+I}	Numerical coefficient for determining minimum the base shear in a structure with damping device
B_{mD}	Numerical coefficient for determining base shear in a structure with damping device
B_{mM}	Numerical coefficient for determining roof displacement in a structure with damping device in the rare earthquake
B_R	Numerical coefficient for determining base shear in a structure with damping device in the residual mode
B_{1D}	Numerical coefficient for determining base shear in a structure with damping device in the fundamental mode
B_{1M}	Numerical coefficient for determining maximum roof displacement in a structure with damping device in the fundamental mode
B_{1E}	Numerical coefficient for determining design roof displacement in a structure with damping device in the fundamental mode
b	Parameter related to seismicity of the site
b	A parameter to determine the modified base shear amplification factor of non-structural component
b	The shortest plan dimension of the structure measured perpendicular to d
b	Dimension of foundation perpendicular to the direction of interest
b_p	Width of rectangular glass
C	Velocity of seismic wave propagation
C_c	Coefficient for tank convective mass period determination
C_d	Deflection amplification factor
C_i	Non-dimensional coefficient for tank impulsive mass period determination
C_j	Coefficient of the concentrated damping at centroid of foundation in the j^{th} DOF
C_m	Modified component base shear amplification factor

C_{mFD}	Force coefficient dependent on displacement to determine design seismic force at the maximum acceleration in mode m
C_{mFV}	Force coefficient dependent on velocity to determine design seismic force at the maximum acceleration in mode m
C_p	Component base shear amplification factor
C_{sm}	Seismic response coefficient for mode m
C_{sR}	Seismic response coefficient for residual mode
C_T	Period modification factor proportional to slenderness of chimney
C_{s1}	Seismic response coefficient for fundamental mode
C_t	Structural period coefficient
C_{Tu}	Analytical period upper bound coefficient
C_u	Seismic response coefficient
C_{u1}	Minimum seismic response coefficient
C_{u2}	Minimum seismic response coefficient
C_{vx}	Lateral force vertical distribution factor
C_{vxm}	Vertical distribution factor for mode m at level x
C_w	A parameter to determine fundamental period of structures having concrete shear walls
C_{1FD}	Force coefficient dependent on displacement to determine the design seismic force at maximum acceleration in the fundamental mode
C_{1FV}	Force coefficient dependent on velocity to determine design seismic force at the maximum acceleration in the fundamental mode
c	Spectral acceleration corresponding to average failure capacity of structure
$\bar{c}$	Average site spectral acceleration derived from attenuation equation
c	Risk-targeted spectral acceleration corresponding to one percent probability of collapse
$c_{10\%}$	Spectral acceleration corresponding to two percent probability of exceedance in 50 years, equivalent to ten percent probability of collapse
c	Cohesion of soil
c_1	Average of the clearances (gaps) at both sides between the vertical glass edges and its frame
c_2	Average of the clearances (gaps) at top and bottom between the horizontal glass edges and its frame
D	Dead load effect
D	Nominal diameter of tank or outside diameter of the pipe
D	Distance from bottom of foundation to ground level
D_1	A parameter for determination of the liquid convective mass period
D_{1M}	Fundamental mode maximum displacement at the center of rigidity of the roof level of the structure in the direction of interest
D_D	Design displacement at the center of rigidity of the isolation system in the direction of interest
D_c	Relative horizontal displacement (drift), measured over the height of the glass panel of interest, which causes the initial glass-to-frame contact
D_f	A parameter to determine rigidity of a single or strip foundation
D_i	Length of shear wall i
D_M	Maximum displacement at the center of rigidity of the isolation system in the direction of interest

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 of interest
D_{mM}	Maximum displacement at the center of rigidity of the roof level of the structure due to the m^{th} mode of vibration in the direction of interest
D_{min}	Minimum inside diameter of pipe including its out of roundness defect
D_{RM}	Residual mode maximum displacement at the center of rigidity of the roof level of the structure in the direction of interest
D_p	Component relative seismic displacement
D_{pl}	Seismic design relative displacement
D_n	Nominal pipe diameter
D_{RD}	Residual mode design displacement at the center of rigidity of the roof level of the structure in the direction of interest
D_s	5% damped response spectrum conversion factor
D_{TD}	Total design displacement
D_{TM}	Total maximum displacement
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
D_{1D}	Fundamental mode design displacement at the center of rigidity of the roof level of the structure in the direction of interest
D_{1M}	Fundamental mode maximum displacement at the center of rigidity of the roof level of the structure in the direction of interest
D'_D	Design displacement at the center of rigidity of the isolation system in the direction of interest, in dynamic analysis
D'_M	Maximum displacement at the center of rigidity of the isolation system in the direction of interest, in dynamic analysis
d	The larger plan dimension of the structure
d	Height of effective sidewall contact, Chapter 5
d_i	Height of the i^{th} course of the tank shell
$\bar{d}_b$	Average diameter of chimney at bottom
E	Seismic load effect
E_t	Effective Young's modulus for tank shell material
E_{ch}	Young's modulus for chimney shell material
E_f	Young's modulus of the foundation material (concrete)
E_h	Horizontal seismic load effect
E_{loop}	Energy dissipated per cycle of loading
E_p	Modulus of elasticity of the pile material
E_p	Primary modulus of elasticity of the pipe material
E_s	Modulus of elasticity of the tank anchorage
E_V	Vertical seismic load effect
e	Napier number equal to 2.7183
e	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, taken as 5% of the longest plan dimension of the structure perpendicular to the direction of the lateral force
F	Load effect due to fluids with well-defined pressures at maximum heights

F^-	Negative forces at Δ^-
F^+	Positive forces at Δ^+
F_a	Site spectra modification factor in constant acceleration region
F_b	Net upward force per unit length of pipeline due to buoyancy
F_C	Allowable longitudinal shell compressive stress
F_D^-	Value of force at a negative displacement equal to D_D
F_D^+	Value of force at a positive displacement equal to D_D
F_i	Seismic lateral force at level i
F_i	Lateral force applied to the center of mass of element/segment i
F_{im}	Inertial force at Level i (or point mass i) in the m^{th} mode of vibration of the structure in the direction of interest
F_{iR}	Inertial force at Level i (or point mass i) in the residual mode of vibration of the structure in the direction of interest
F_{i1}	Inertial force at Level i (or point mass i) in the fundamental mode of vibration of the structure in the direction of interest
F_M^-	Value of force at a negative displacement equal to D_M
F_M^+	Value of force at a positive displacement equal to D_M
F_p	Design force for each anchor
F_p	Seismic force applied to non-structural component
F_p	Ratio of design to service pressure
F_{pi}	Force acting at the center of the i^{th} mass of the non-structural component
F_{stop}	Maximum design friction force
F_{px}	Diaphragm seismic design force
F_{ty}	Minimum specified yield stress of the shell course
F_v	Site-specific spectrum modification factor in the constant velocity region
F_w	Capacity of weld under simultaneous effects of shear and tension
F_x	Lateral seismic design force at level x
F_{xm}	Lateral force of level x in mode m
F_y	Minimum specified yield strength of the bottom annulus
f	Friction coefficient between soil and pipe
f_E	Seismic soil shear modulus reduction coefficient
f_g	Soil shear modulus amplification coefficient caused by weight of structure
$f_M(m)$	Probability density function for earthquake magnitude
$f_R(r)$	Probability density function for distance
$f_{capacity}(c)$	Function for fragility curve of structure
f'_c	Compressive strength of concrete
G	Effective shear modulus of subgrade in soil-structure interaction calculations (corresponding to large soil strains)
G_0	Shear modulus of subgrade in free-field conditions (corresponding to small soil strains without presence of structure or earthquake)
G_p	Shear modulus of elastomeric bearing pad of tank
g	Acceleration of gravity

g_e	Effective specified gravity including vertical seismic effects
H	Load effect due to lateral earth pressure, ground water pressure, or pressure of bulk materials
H	Structure height from base level
H_g	Specified length of pile group including pile cap in contact with stiff soil
H_l	Maximum height of the liquid
H_{rw}	Height of retaining wall
H_s	Depth of soil above the center of the pipeline
h	Average roof height of structure with respect to the base level
h	Vertical distance between centroid of lateral side of foundation having effective contact with soil from the ground level
$\bar{h}$	Height of mass center of the structure from base level
$\bar{h}$	Effective height of structure
h_{av}	Average of heights of attachment points of the non-structural component to the building, calculated from the base
h_c	Height of the center of action of the lateral seismic force related to the convective liquid from the bottom of the tank for determination of ring wall moment
h_{ch}	Chimney height with respect to the base
h_{cs}	Height of the center of action of the lateral seismic force related to the convective liquid from the bottom of the tank for determination of slab moment
h_i	Height of shear wall i or, center of mass of segment i or, roof or level i from base
h_i	Height of the center of action of the lateral seismic force related to the impulsive liquid from the bottom of the tank for determination of ringwall moment
h_i	Height from the bottom of the tank shell to the center of action of the lateral seismic force related to the impulsive liquid force for the ringwall moment
h_{is}	Height of the center of action of the lateral seismic force related to the impulsive liquid from the bottom of the tank for determination of slab moment
h_p	Height of the rectangular glass panel
h_r	Height of structure from base level to roof level
h_r	Height of the tank roof center of gravity from the bottom
h_s	Thickness of soft soil layer
h_s	Height of the tank shell center of gravity from the tank bottom
h_s	Vertical distance between floor to the floor to which non-structural component is attached
h_{sp}	Height of soil above pipeline
h_{sx}	Story height below level x
h_w	Height of water level above pipeline
h_x	Height of level x, calculated from base level or foundation centroid level
h_x	Height of higher level (x) where component is attached to structure
h_y	Height of lower level (y) where component is attached to structure
h_z	Height of segment z, calculated from base level
I	Importance factor of the structure
I_f	Moment of inertia of the foundation section (uncracked) about the axis perpendicular to direction of interest
I_g	Uncracked gross section moment of inertia

I_L	Importance factor for pipeline
I_p	Component importance factor
I_{pf}	Moment of inertia of effective foundation plan about area central axis perpendicular to direction of interest
IM	Intensity measure for earthquake
$\overline{IM}$	Average value of intensity measure at site, derived from GMPE
i	Stress intensification factor
i	Number of the closest segment with its mass center being above level z
J	Anchorage ratio
J_v^z	Correction factor for higher mode effects at level z
$\overline{K}$	Effective lateral stiffness of structure with rigid supports
K_{Dmax}	Maximum effective stiffness of the isolation system at the design displacement in the horizontal direction of interest
K_{Dmin}	Minimum effective stiffness of the isolation system at the design displacement in the horizontal direction of interest
K_{eff}	Effective stiffness of an isolator unit
K_h	Horizontal stiffness of foundation
$K_{j,emb}$	Spring stiffness coefficient at j^{th} DOF considering embedment to the depth D
$K_{j,sur}$	Spring stiffness coefficient at j^{th} DOF not considering embedment to the depth D
K_{Mmax}	Maximum effective stiffness of the isolation system at the maximum displacement in the horizontal direction of interest
K_{Mmin}	Minimum effective stiffness of the isolation system at the maximum displacement in the horizontal direction of interest
K_p	Stiffness of the industrial component and its connections to the structure
K_{sr}	Rotational spring constant of pile foundations about each horizontal axis
K_{st}	Vertical stiffness of deep foundation
K_{sv}	Rotational spring constant of pile foundations about vertical axis
K_θ	Rotational stiffness of foundation
k	Distribution exponent related to structural natural period
k_0	Coefficient of soil pressure at rest
k_a	Amplification factor for diaphragm flexibility
k_a	Horizontal stiffness per unit shell length at tank base
k_h	Horizontal seismic coefficient of soil
k_{sh}	Horizontal subgrade modulus
k_{sv}	Vertical subgrade modulus
L	Reduced live load effect
L	Foundation length (larger dimension)
L_f	Column tributary length of the foundation
L	Width of annular plate measured from the inside of the shell
L	Length of permanent ground deformation zone
L_a	Unanchored pipe length
L_b	Length of pipe in buoyancy zone

L_e	Effective length of pipeline over which friction force, t_u, acts.
L_f	Span length of flexible diaphragm
L_g	Outer dimension of pile group
L_{max}	Maximum permitted pipe span between lateral and vertical seismic restraints
L_p	Length of pile
L_p	Length of elastomeric bearing pad of tanks
L_r	Roof live load effect
L_{req}	Width of the bottom annular plate, measured from inside of the shell, over which portion of fluid resisting overturning acts
L_s	Effective length of anchorage taken as 35 times the anchor diameter plus the sleeve length between wall and base.
L_T	Recommended distance between gravity load supports
L_z	Length of permanent ground deformation zone
L_0	Length of pipe segment
l_i	Distance from attachment point to i^{th} mass of non-structural component
M	Overturning moment at foundation level without accounting for interaction effects
M_a	Resultant moment amplitude due to relative anchor movement
M_i	Resultant moment amplitude due to inertia forces
M_{rw}	Ringwall moment
M_s	Slab moment (of the tank)
M_t	Torsional moment of story
M_{ta}	Torsional moment of story considering accidental torsion effect in plan
M_w	Moment magnitude of earthquake
M_z	Moment at level h_z
m	Number of shear walls in the building effective in resisting lateral forces in the direction of interest
m_c	Convective portion of the liquid mass
m_f	Mass of the tank bottom plate
m_{fd}	Total mass of tank foundation
m_g	Mass of soil directly over tank foundation footing
m_i	Impulsive portion of the liquid mass
m_{max}	Earthquake maximum magnitude
m_{min}	Earthquake minimum magnitude
m_p	Total mass of the tank contents based on the design specific gravity of the product
m_r	Total mass of fixed tank roof including framing, knuckles, any permanent attachment and 10% of roof design snow mass
m_s	Total mass of tank shell and appurtenances
m_t	Mass of empty tank including shell, roof, bottom and attached parts
N_c	Bearing capacity factor for cohesive soil
N_c	Convective hoop membrane force in unit vertical length of tank shell
N_{ch}	Horizontal bearing capacity factor for soil cohesion
N_{cv}	Uplift bearing capacity factor for soil cohesion

N_q	Bearing capacity factor for soil
N_{qv}	Uplift bearing capacity factor for soil friction
N_g	Number of piles in a pile group
N_h	Product hydrostatic membrane force in unit vertical length of tank shell
N_i	Impulsive hoop membrane force in unit vertical length of tank shell
N_{qh}	Horizontal bearing capacity factor for soil friction
N_s	Total seismic hoop force per unit length of the shell
N_γ	Bearing capacity factor for soil
n	Number of stories/levels/floors/concentrated masses/component masses
n	Number of vibration modes
n	Identifier for upper component
n	Number of courses of tank shell
n	Parameter for the material behavior relation
n'	Number of non-structural component segments
n_A	Number of circumferential anchor bolts or anchor strips
n_C	Number of chained restraint joints
PE	Probability of exceedance
P	Uplift force caused by combined dead and earthquake loads
P_{Aa}	Design load for each anchor in ASD method
P_{Au}	Design load for each anchor in SD method
P_f	Total design bearing force in unit length of annular footing in SD method
P_{fs}	Design seismic bearing force in unit length of annular footing in SD method
P_i	Design internal pressure for tank
P_n	Nominal anchor pull-out capacity
P_{nr}	Anchor pull-out capacity
P_p	Maximum internal operating pressure of the pipe
P_u	Maximum lateral resistance of soil per unit length of pipe
P_v	Vertical earth pressure
P_w	Applied tensile forces of weld
P_x	Total vertical load over level x
PGV	Peak ground velocity at considered location
p_c	Lateral force intensity caused by convective mass at considered height based on a trapezoidal distribution
p_i	Lateral force intensity caused by impulsive mass at considered height based on a trapezoidal distribution
Q_{DSD}	Force in an element of the damping system required to resist design seismic forces of displacement- dependent damping devices
Q_d	Soil bearing strength in unit length of pipe
Q_E	Effects of horizontal seismic forces
Q_{mDSV}	Forces 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 of interest

Q_{mSFRS}	Force in an element of the damping system due to the design seismic force of the m^{th} mode of vibration of the structure in the direction of interest
Q_u	Soil uplift strength at unit length of pipe
q_c	Circumferential horizontal distribution of the hydrodynamic pressure on tank shell for each height caused by convective liquid
q_H	Hysteresis loop adjustment factor
q_i	Circumferential horizontal distribution of the hydrodynamic pressure on tank shell for each height caused by impulsive liquid
R	Rain load effect
R	Outer radius of pipe
R	Tank response factor
R_1	Anchor pull-out capacity modification factor based on concrete strength
R'_1	Anchor shear capacity modification factor based on concrete strength
R_2	Anchor pull-out capacity modification factor based on width of cracks
R_3	Anchor pull-out capacity modification factor based on sensitive content
R'_3	Anchor shear capacity modification factor based on sensitive content
R_4	Anchor pull-out capacity modification factor based on inspection quality
R'_4	Anchor shear capacity modification factor based on inspection quality
R_I	Response modification factor of isolated structure
R_{max}	Maximum site to fault distance, corresponding to distance definition used in GMPE
R_{min}	Minimum site to fault distance, corresponding to distance definition used in GMPE
R_p	Component response modification factor
R_u	Response modification factor in SD method
R_w	Response modification factor in ASD method
R_c	Response modification factor for liquid convective mass, equal to 1.5 for steel tanks and 1.0 for concrete tanks
r	Parameter in the relation of material behavior
r	Characteristic dimension of foundation (radius of equivalent circular foundation)
r_{ch}	Radius of gyration of chimney shell section
S	Snow load effect
S_1	Spectral acceleration parameter (g), corresponding to the rare earthquake for a period of 1 sec. at bedrock, from Site-Specific Hazard Study
S_a	Risk-targeted spectral acceleration
S_a	5% damped spectral acceleration parameter (g)
S_{aser}	Service earthquake spectral acceleration (g) form Site-Specific Hazard Study
$\bar{S}_a$	Spectral acceleration parameter (g), considering soil-structure interaction
S_{am}	Spectral acceleration parameter of mode m
S_{D1}	Design, 5% damped, spectral response acceleration parameter (g) at the period of 1 sec.
S_{DS}	Design, 5% damped, spectral response acceleration parameter (g) at short periods (0.2 sec.)
S_{M1}	The rare earthquake, 5% damped, spectral response acceleration parameter (g) at the period of 1 sec., adjusted for site class effects
S_{MS}	The rare earthquake, 5% damped, spectral response acceleration parameter (g) at short periods (0.2 sec.), adjusted for site class effects

S_n	Distance between n^{th} pile and axis of rotation (passing through foundation plan centroid)
S_p	Center-to-center spacing between elastomeric bearing pads of tank
S_p	Longitudinal stress in pipe due to internal pressure
S_r	Longitudinal stress in pipe due to temperature variation
S_s	Spectral acceleration parameter (g), corresponding to the rare earthquake for a period of 0.2 sec. on bedrock, from Site-Specific Hazard Study
S_s	Center-to-center spacing between consecutive anchors at shell perimeter
S_s	Allowable seismic stress at -30 ºC to +40 ºC temperatures
S_u	Undrained shear strength of soil
T	Self-straining load effect
T	Earthquake return period for desired Seismic Hazard Level
T	Fundamental period of structure on a rigid base
T_0	Soil dependent parameter, in the notion of period
$\bar{T}$	Fundamental period of structure supported on springs
T_1	Fundamental period
T_1	Temperature at the time of pipe installation
T_2	Temperature at the time of pipe operation
T_c	Natural period of the convective (sloshing) mode of behavior of the liquid
T_D	Effective period of the seismically isolated structure at the design displacement in the direction of interest
T_i	Natural period of the impulsive mode of tank and liquid
T_m	Natural period for mode m
T_M	Effective period of the seismically isolated structure at the maximum displacement in the direction of interest
T_p	Fundamental period of industrial/non-structural component
T_R	Period of the residual mode of vibration of the structure in the direction of interest
T_S	Soil dependent parameter, in the notion of period
T_{1D}	Effective period of the fundamental mode of vibration of the structure at the design displacement in the direction of interest
T_{1M}	Effective period of the fundamental mode of vibration of the structure at the maximum displacement in the direction of interest
t	Service life of structure/component, usually taken as 50 years
t	Thickness of foundation
t	Thickness of pipe wall
t	Thickness of shell at desired location or course for calculating hoop stress
t_a	Thickness, excluding corrosion allowance, of the bottom annulus under the shell
t_b	Thickness, excluding corrosion allowance, of the bottom plate under the shell
t_b	Chimney thickness at bottom
t_c	Thickness of desired course of tank shell
t_e	Effective thickness of tank shell
t_g	Effective soil depth, measured from bottom of foundation downward
t_h	Chimney thickness at top

t_i	Thickness of i^{th} layer of soil
t_i	Thickness of i^{th} course of tank shell
t_h	Chimney thickness at top
t_p	Thickness of elastomeric bearing pad of tank
t_p	Nominal wall thickness of the pipe
t_r	Thickness of roof plate in tank
t_s	Thickness of bottom shell course minus the corrosion allowance
t_u	The ultimate friction force acting in axial direction of the pipe
V	Shear force caused by combined dead and earthquake loads
V_b	Minimum seismic design load for isolation system and structural elements below the isolation system
V_c	Design base shear due to the convective component of the effective sloshing mass
V_g	Peak ground velocity for desired importance factor of pipe
V_{g0}	Peak ground velocity for specified location of pipe in Seismic Hazard Level II
V_i	Maximum modified base shear
V_i	Design base shear due to impulsive component from effective mass of tank and contents
V_m	Base shear for mode m
V_{min}	Minimum base shear
V_{max}	Peak local tangential shear per unit length at shell to bottom joint in tank
V_n	Nominal shear capacity of anchor
V_{nr}	Shear capacity of anchor
V_p	Base shear or sum of the base shears at the supports of a non-structural component
V_R	Seismic base shear of the residual mode of vibration of the structure in a given direction
V_s	Effective soil shear wave velocity
V_s	Minimum design shear force for the structure above the isolation system
V_s	Upper allowable limit of base shear in self-anchored flat bottom tanks
$\bar{V}_s$	Average shear wave velocity in soil layers
V_{s0}	Average shear wave velocity in soil layers for free-filed condition at effective soil depth
V_{s0i}	Shear wave velocity in i^{th} soil layer for free field condition
V_{sgi}	Increased shear wave velocity for structure weight in i^{th} soil layer
V_{ser}	Base shear due to service earthquake
V_u	Seismic base shear
$\bar{V}_u$	Structural base shear including soil-structure interaction effects
V_w	Shear forces of weld
V_x	Seismic design story shear between levels x and $x-1$
V_z	Design shear at level z
V_1	Fundamental mode design base shear in direction of interest
W	Wind load effect
W	Effective seismic weight of the structure
W	Effective seismic weight of the structure above the isolation interface

$\bar{W}_1$	Effective weight of structure in fundamental mode, including live load share
$\bar{W}_R$	Effective weight of structure in residual mode
W_c	Weight of pipe content per unit length
W_{ch}	Total weight of the chimney
W_m	Effective seismic weight for mode m
W_m	Maximum strain energy for m^{th} mode in direction under consideration for modal displacement, δ_{im}
W_{mj}	Work done by j^{th} damper in a complete loop of dynamic response corresponding to m^{th} mode in direction of interest
W_p	Tributary load assigned to anchor from wall
W_p	Industrial/non-structural component operating weight
W_p	Weight of pipe per unit length
W_s	Total weight of soil displaced by pipe per unit length
W_z	Width of permanent ground deformation zone
w_a	Liquid weight adjacent to shell resisting uplift in annular region
w_{Aa}	Calculated design uplift load on anchors per unit circumferential length in ASD method
w_{As}	Calculated uplift load on anchors per unit circumferential length
w_{Au}	Calculated design uplift load on anchors per unit circumferential length in SD method
w_c	Computed crack width at anchor location
w_i	The portion of the total effective seismic weight of the structure, W, located at or assigned to level i
w_p	Width of elastomeric bearing pad
w_{pi}	Weight of the i^{th} mass of the non-structural component
w_{px}	The seismic weight tributary to the diaphragm and it's apparatuses at level x
w_t	Tank and roof weight acting at base of shell
w_{tr}	Tank weight acting at base of shell, excluding roof weight
w_x	Effective seismic weight at level x
x	Height exponent parameter in experimental period calculations
x_i	Distance from middle of i^{th} course to liquid surface level in tank
Y	Vertical distance from liquid surface level to analysis point
Y_u	Elastic uplift of a self-anchored tank bottom
y	Distance between the centers of rigidity of the isolation system and the element of interest measured perpendicular to the direction of seismic loading under consideration
y_{max}	Maximum lateral elastic displacement at top of the chimney
Z_e	Elastic section modulus of pipe cross section
z	Height of anchor with respect to the base
z	Height of component attachment point to structure with respect to the base
α	Vertical load effect estimation factor
α_a	Angle of anchor with respect to horizontal direction
α_s	Adhesion factor between soil and pipe
α_t	Linear coefficient of thermal expansion
α_ε	Ground strain coefficient

β	Angle between pipeline and crossed fault line
β_j	Embedment correction factor for stiffness coefficients of deep foundations, for j^{th} DOF
β_D	Effective damping ratio of the isolation system, corresponding to design displacement
β_{HD}	Component of effective damping ratio of the structure in the direction of interest due to post-yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand of design earthquake
β_{HM}	Component of effective damping ratio of the structure in the direction of interest due to post-yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand of maximum earthquake
β_{eff}	Effective damping ratio of an isolator unit
β_I	Inherent damping ratio
β_M	Effective damping ratio of the isolation system at the maximum displacement
β_{mD}	Total effective damping of the m^{th} mode of vibration of the structure in the direction of interest at the design displacement
β_{mM}	Total effective damping ratio of the m^{th} mode of vibration of the structure in the direction of interest at the maximum displacement
β_R	Total effective damping ratio in the residual mode of vibration of the structure in the direction of interest
β_{Vm}	Component of effective damping ratio of the m^{th} mode of vibration of the structure in the direction of interest due to viscous dissipation of energy by the damping system
Δ^+	Maximum positive displacement of isolator in each full-scale test cycle
Δ^-	Maximum negative displacement of isolator in each full-scale test cycle
Δ_a	Allowable story drift
Δ_a	Safety margin for attachment displacement
$\Delta_{allowable}$	Allowable joint displacement for segmented pipe, as specified by joint manufacturer
Δ_D	Design story drift
Δ_{D+L}	Displacement caused by gravity loads
Δ_f	Allowable seismic drift for glass exterior walls or interior partitions having glass panels
Δ_M	Maximum story drift
Δ_{RD}	Design story drift in residual mode
Δ_x	Total design drift at level x
Δ_{xe}	Design elastic drift at level x, excluding P-Δ effects
Δ_{1D}	Design story drift in fundamental mode
Δ_m	Design story drift in m^{th} mode
Δ_{oper}	Joint displacement during operation
$\Delta_{seismic}$	Maximum joint displacement due to seismic action
$\Delta_{oper+seismic}$	Maximum joint displacement due to combined seismic action and operation
Δ_p	Joint deformation caused by internal pressure
Δ_p	Added seismic lateral pressure distributed uniformly on retaining wall
Δ_p	Maximum pipe lateral displacement
Δ_{Qu}	Pipe displacement corresponding to soil uplift strength
Δ_{Qd}	Pipe displacement corresponding to soil bearing strength
Δ_t	Joint deformation caused by temperature gradient

Δ_t	Maximum mobilized displacement of soil, parallel to pipe longitudinal axis
$\bar{\Delta}_{xe}$	Elastic drift of story x, including P-Δ effects
$\Delta\acute{\sigma}_i$	Effective normal stress at i^{th} mass center, due to weight of structure
∇_D	Total design story velocity in the direction of interest
∇_M	Total maximum story velocity in the direction of interest
∇_{RD}	Design story velocity due to the residual mode of vibration of the structure in the direction of interest
∇_{1D}	Design story velocity due to the fundamental mode of vibration of the structure in the direction of interest
∇_{mD}	Design story velocity due to the m^{th} mode of vibration of the structure in the direction of interest
δ	Angle of friction between pipe and soil
δ_{ave}	Average of the displacements at the extreme points of the structure at level x excluding A_x
δ_{fax}	Component of fault displacement in axial direction of pipe
$\delta_{fax-design}$	Design fault displacement in axial direction of pipe
δ_{fb}	Average displacement of fault with unknown behavior
δ_{fn}	Average normal fault displacement
δ_{fr}	Average reverse fault displacement
δ_{fs}	Average strike slip fault displacement
δ_{ftr}	Component of fault displacement in transverse direction of pipe
$\delta_{ftr-design}$	Design fault displacement in transverse direction of pipe
δ_{fvt}	Component of fault displacement in vertical direction of pipe
$\delta_{fvt-design}$	Design fault displacement in vertical direction of pipe
δ_i	Lateral elastic displacement of the center of mass of element i
δ_{im}	Displacement of center of rigidity of i^{th} floor at m^{th} mode in direction of interest
δ_{iD}	Design displacement of i^{th} level
δ_{iM}	Maximum displacement of i^{th} level
δ_{imD}	Design displacement of i^{th} level at m^{th} mode
$\acute{\delta}_{max}$	Maximum lateral displacement at level x, including A_x
δ_{max}	Maximum lateral displacement at level x, excluding A_x
δ_M	Maximum lateral displacement of structure
δ_{MT}	Minimum allowable distance between adjacent structures
δ_s	Height of the sloshing wave above the product design height
δ_{iRD}	Residual mode design deflection of level i at the center of rigidity of the structure in the direction under consideration
δ_{i1D}	Fundamental mode design deflection of level i at the center of rigidity of the structure in the direction under consideration
$\bar{\delta}_x$	Inelastic lateral displacement at level x, in Equivalent Structure Method
δ_x	Inelastic lateral displacement at level x without considering interaction effects
δ_{xe}	Elastic lateral displacement at level x
δ_{xem}	Elastic displacement at level x for mode m
δ_{xm}	Total lateral displacement at level x for mode m

δ^1	Maximum longitudinal ground displacement
$\delta^1{}_{design}$	Design ground displacement in longitudinal direction
δ^t	Maximum transverse ground displacement
$\delta^t{}_{design}$	Design ground displacement in transverse direction
ε	Strain in pipe
ε_a	Axial strain in pipe
$\varepsilon_{allowable}$	Allowable strain in pipe
ε_b	Maximum bending strain in pipe
ε_{c-PGD}	Allowable strain for permanent ground deformation effect in pipe
ε_{c-wave}	Allowable strain for wave effect in pipe
ε_{cr-c}	Strain at onset of wrinkling
ε_{D+L}	Strain in the pipe due to gravity loads
ε_{oper}	Operational strain in pipe
ε_p	Strain in the pipe due to internal pressure
$\varepsilon_{seismic}$	Maximum strain in pipe due to seismic action
ε_t	Strain in pipe due to temperature change
ε_u	Ultimate tensile strain of pipe material
ε_y	Yield strain of material
ϕ	Internal angle of friction of the soil
ϕ	A parameter to determine non-structural component amplification factor
ϕ_{im}	Modal amplitude at level i for mode m
ϕ_{i1}	Modal amplitude at level i for fundamental mode
ϕ_{iR}	Modal amplitude at level i for residual mode
ϕ_{xm}	Modal amplitude at level x for mode m
Γ_m	Mode m participation factor
Γ_1	Fundamental mode participation factor
Γ_R	Residual mode participation factor
$\bar{\gamma}$	Effective unit weight of soil
γ_d	Dry unit weight of backfill
γ_t	Unit weight of soil
γ_w	Unit weight of water
η	Damping ratio
λ	A parameter to determine non-structural component base shear
λ_e	Apparent wavelength of seismic waves at ground surface
λ_{annual}	Annual rate of occurrence in site for desired IM, equal to $1/T$
μ	Number of occurred events between m_{max} and m_{min}, during desired time duration, divided by this duration
μ	Ductility factor of structure
μ_D	Effective ductility demand for design earthquake
μ_{eq}	Equivalent ductility factor

μ_f	Friction coefficient between tank bottom and foundation
μ_M	Effective ductility demand for maximum earthquake
μ_{max}	Maximum effective ductility demand
μ_p	Ductility factor of non-structural component
θ	Angle between earthquake direction and radius passing from considered point on tank shell
$\theta_{seismic}$	Joint seismic rotation
θ_{max}	Maximum stability index
θ_x	Stability index for level x
ρ	Redundancy factor
ρ	Soil density in the free-field condition
ρ_{ch}	Equivalent chimney density (including shell and cover)
ρ_L	liquid density (mass per unit volume)
$\sum E_D$	Total energy dissipated in the isolation system during a full cycle of response at the design displacement, D_D
$\sum E_M$	Total energy dissipated in the isolation system during a full cycle of response at the maximum displacement, D_M
σ	Stress in pipe
σ_{bf}	Bending stress induced in a relatively short section of continuous pipeline subjected to buoyancy
σ_c	Total shell longitudinal compressive stress in ASD method
σ_{cs}	Maximum seismic longitudinal compressive stress in self-anchored tank at the bottom of the shell
σ_h	Hydrostatic hoop stress
σ_s	Seismic hoop stress
σ_{IM}	Standard deviation of intensity measure values
σ_y	Yield stress of pipe material
σ_{yo}	Material yielding stress at normal operating temperature
σ_i'	Effective normal stress at i^{th} mass center, due to weight of soil
ν	Poisson's ratio
ν_f	Poisson's ratio of foundation material (concrete)
Ω_0	Over-strength factor
ξ_j	Soil radiation damping ratio
ξ_S	Soil large strain damping ratio
ξ_j^t	Damping ratio, equal to soil radiation damping ratio plus soil large strain damping ratio
$\bar{\xi}$	Equivalent damping ratio
ξ_0	Damping ratio of foundation
ξ_0'	Replaced damping ratio of foundation for soft subgrade soil
ψ	A parameter to determine soil radiation damping ratio
ψ	Dip angle

Chapter 2
Loading

2. Loading

2.1 General Provisions

Building or non-building structures and oil industry equipment can be designed by either “Allowable Stress Design (ASD)” or “Strength Design (SD)” methods in this code. In both methods, the load combinations of Section 2.2 shall be used. Members of these structures should satisfy specified requirements for the most critical load combination as well as combinations including over-strength factor if necessary according to Section 2.2.4. When using load combinations presented here, it is necessary to consider their compatibility with requirements of structural design codes.
In this code, seismic load criteria are discussed. For defining other loads, see Reference [3] as well as documents and specifications for design and installation of equipment. Other loads not specified herein shall be applied regarding the valid standards and references compatible with the given combinations.

2.2 Load Combination

In this code, basic combinations for both ASD and SD methods are presented in Sections 2.2.1 and 2.2.2, respectively. If needed, in addition to the loads specified in these sections and other chapters, for other loads such as operation, fire, blast loads, other valid references are allowed to be used.

2.2.1 Load Combinations for ASD Method

2.2.1.1 Basic Combinations

In ASD method, maximum forces in members and foundation shall be determined using the most critical load combination. When using combinations of this section, increase in the allowable stress is not permitted.
Basic load combinations in ASD method are as follows:

$$D \tag{2.1}$$

$$D + L \tag{2.2}$$

$$D + (L_r \ or \ S \ or \ R) \tag{2.3}$$

$$D + 0.75\ L + 0.75\ (L_r or \ S \ or \ R) \tag{2.4}$$

$$D + (0.6 \times 1.4\ W \ or \ 0.7\ E) \tag{2.5}$$

$$D + 0.75 \times 0.6 \times 1.4\ W + 0.75\ L + 0.75(L_r or \ S \ or \ R) \tag{2.6 a}$$

$$D + 0.75(0.7E) + 0.75\ L + 0.75S \tag{2.6 b}$$

$$0.6D + 0.6 \times 1.4\ W \tag{2.7}$$

$$0.6D + 0.7E \tag{2.8}$$

where:
D = dead load effect
L = (reduced) live load effect
L_r = roof live load effect
E = earthquake load effect, according to Section 2.2.3
S = snow load effect
W = wind load effect
F = lateral load effect due to fluids with well-defined pressures at maximum height
H = load effect due to lateral earth pressure, ground water pressure, or pressure of bulk materials
R = rain load effect
Note 1. When fluid loads, F, are present; they shall be included at their maximum height with the same load factor as dead load, D, in combinations 2.1 to 2.6, and 2.8.

Note 2. Where load effects due to lateral earth pressure, ground water pressure, or pressure of bulk materials, H, are present with increasing effects compared to the primary loads, they shall be included with a load factor of 1.0 in all combinations. Where the effect of H resists the primary load effects, it shall be included with a load factor of 0.6 where the load is permanent or a load factor of zero for all other conditions.
Note 3. In the above load combinations, it is necessary to omit effects of those loads that ignoring them causes critical conditions.
Note 4. If necessary, effects of the seismic load shall be included by load combinations including over-strength factor according to Section 2.2.4.

2.2.1.2 Load Combinations Including Self-Straining Loads

Where applicable, the structural effects of load case T (strains independent of other external loads such as temperature gradients, assembly defects, settlement and creep) shall be considered in combination with other loads. The load factor on load T shall be established considering the probability that the maximum effect of T will occur simultaneously with other applied loadings, and the potential adverse consequences, compatible with valid references or other chapters of this code. The fraction of T considered in combination with other loads shall not be less than 0.75.

2.2.1.3 Load Combinations for Non-Specified Loads

Where approved by the Authority Having Jurisdiction, the Responsible Design Professional is permitted to determine the combined load effects for non-specified loads using a probability-based method, accompanied by documentation from valid references.

2.2.2 Load Combinations for SD Method

2.2.2.1 Basic Combinations

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

$$1.4\,D \tag{2.9}$$

$$1.2\,D + 1.6\,L + 0.5\,(L_r \ or\ S\ or\ R) \tag{2.10}$$

$$1.2\,D + 1.6\,(L_r \ or\ S\ or\ R) + \ (L\ or\ 0.5 \times 1.4W) \tag{2.11}$$

$$1.2\,D + 1.4\,W + L + 0.5\,(L_r \ or\ S\ or\ R) \tag{2.12}$$

$$1.2\,D + E + L + 0.2S \tag{2.13}$$

$$0.9\,D + 1.4\,W \tag{2.14}$$

$$0.9\,D + \ E \tag{2.15}$$

Note 1. The load factor on L in combinations 2.11 through 2.13 is permitted to equal 0.5 for all occupancies in which unreduced live load is less than or equal to 4 kN/m^2 with the exception of garages or areas occupied as places of public assembly.
Note 2. Where fluid loads F are present, they shall be included with the same load factor as dead load D in combinations 2.9 through 2.13 and 2.15.
Note 3. Where load effects due to lateral earth pressure, ground water pressure, or pressure of bulk materials, H, are present with increasing effects compared with primary loads, they shall be included with a load factor of 1.6 in all combinations. Where the effect of H resists the primary load effects, it shall be included with a load factor of 0.9 where the load is permanent or a load factor of zero for all other conditions.
Note 4. Effects of one or more loads not acting shall be investigated.

2.2.2.2 Load Combinations Including Self-Straining Loads

Where applicable, the structural effects of load T shall be considered in combination with other loads. The load factor on load T shall be established considering the probability that the maximum effect of T will occur simultaneously with other applied loadings, and the potential adverse consequences, compatible with valid references or other chapters of this code. The load factor for T shall not be taken less than 1.0.

2.2.2.3 Load Combinations for Non-Specified Loads

Where required by the Authority Having Jurisdiction, the Responsible Design Professional is permitted to determine the combined load effect for non-specified loads using a probability-based method, accompanied by documentation from valid references.

2.2.3 Seismic Load Effects in Load components

All members of the structure, including those not part of the seismic force-resisting system, shall be designed using the seismic load effects of this section unless otherwise exempted by this code. In defining earthquake load for both ASD and SD methods, seismic load effects are derived at limit states in this section.

2.2.3.1 Horizontal Seismic Load Effect

The horizontal seismic load effect, E_h, shall be determined in accordance with the following Equation:

$$E_h = \rho Q_E \qquad 2.16$$

where:
ρ = redundancy factor, as defined in Section 4.6
Q_E = effects of horizontal seismic forces as defined in Chapter 4. Where required by Section 4.7, such effects shall result from application of horizontal forces simultaneously in two orthogonal directions. Where linear time history analysis, defined in Section 4.10.2, is performed, Q_E, if necessary, represents the effect of applied orthogonal pairs of ground motion record.

2.2.3.2 Vertical Seismic Load Effect

The vertical seismic load effect, E_v, shall be determined in accordance with the following Equation:

$$E_v = \alpha S_{DS} D \qquad 2.17$$

where:
S_{DS} = Design, 5% damped, spectral response acceleration parameter (g) at short periods (0.2 sec). In cases where site-specific spectrum is not necessary to use in accordance with Chapter 3 of this code, value of S_{DS} can be obtained by multiplying base design acceleration A, by response factor B in the constant acceleration part of the smoothed spectrum from Reference [2].
α = vertical load effect estimation factor, which can be taken as 0.2.

2.2.3.3 Combination of Seismic Horizontal and Vertical Load Effects

For considering the combined horizontal and vertical earthquake load effect, E; in load combinations of Section 2.2.1 and 2.2.2, it shall be substituted by Equations 2.18 and 2.19.
For this purpose, seismic load effects in load combinations 2.5, 2.6 and 2.13 shall be considered as Equation 2.18 and seismic load effects in Equation 2.8 shall be considered as Equation 2.19. It should be noted that in seismic demand determination for the structure-foundation interface; value of E_v in Equation 2.19 could be taken zero.

$$E = E_h + E_v \qquad 2.18$$

$$E = E_h - E_v \tag{2.19}$$

Simultaneous effect of horizontal and vertical seismic loads in ASD and SD methods shall be considered as follows:

A. ASD method:

$$(1.0 + 0.14\, S_{DS})\, D + 0.7\rho\, Q_E \tag{2.20}$$

$$(1.0 + 0.105\, S_{DS})\, D + 0.75(0.7\, \rho\, Q_E) + 0.75\, L + 0.75\, (L_r \; or\; S\; or\; R) \tag{2.21}$$

$$(0.6 - 0.14\, S_{DS})\, D + 0.7\rho\, Q_E \tag{2.22}$$

When fluid load F is present, it shall be included with the same load factor as dead load D in combinations 2.20 and 2.21.
Where load H is present with increasing effects to primary loads, it shall be included with a load factor of 1.0 in all combinations. Where the effect of H resists the primary load effects, it shall be included with a load factor of 0.6 where the load is permanent or a load factor of zero for all other conditions.

B. SD method:

$$(1.2 + 0.2\, S_{DS})\, D + \rho Q_E + L + 0.2S \tag{2.23}$$

$$(0.9 - 0.2\, S_{DS})\, D + \rho Q_E \tag{2.24}$$

The load factor for L in combination 2.23 is permitted to be taken 0.5 where unreduced live load is less than or equal to 4 kN/m^2, with the exception of garages or areas occupied as places of public assembly.
Where fluid load F is present, it shall be included with the same load factor as dead load D in combination 2.23.
Where load H is present with increasing effects to primary loads, it shall be included with a load factor of 1.6 in all combinations. Where the effect of H resists the primary load effects, it shall be included with a load factor of 0.9 where the load is permanent or a load factor of zero for all other conditions.

2.2.4 Combinations Including Over-strength Factor

In columns, beams, trusses and slabs supporting a discontinuous wall or a frame of the structure having irregularity type 4 in Tables 4.1 or 4.2, enough capacity to resist the maximum axial forces resulted from combinations in this section shall be provided. The connections of these discontinuous elements to supporting members shall also have adequate capacity to transmit design forces from elements to supports.
In cantilevered column systems, row F of Table 4.4; foundation and other overturning resistant members shall have enough capacity to resist forces resulted from combinations in this section. Axial force in these columns, resulted from combinations in Section 2.2.1 or 2.2.2, shall not exceed 15% of axial capacity of column in ASD or SD method, respectively.
In addition, in cases where according to design codes, utilizing combinations with over-strength factor is necessary, combinations of this section shall be applied either.

A. ASD method:

$$(1.0 + 0.14\, S_{DS})\, D + 0.7\, \Omega_0 Q_E \tag{2.25}$$

$$(1.0 + 0.105\, S_{DS})\, D + 0.75\, (0.7\, \Omega_0 Q_E) + 0.75\, L + 0.75\, (L_r \; or\; S\; or\; R) \tag{2.26}$$

$$(0.6 - 0.14\, S_{DS})\, D + 0.7\, \Omega_0 Q_E \tag{2.27}$$

Where fluid load F is present, it shall be included with the same load factor as dead load D in combinations 2.25 and 2.26.
Where load H is present with increasing effects to primary loads, it shall be included with a load factor of 1.0 in all combinations. Where the effect of H resists the primary load effects, it shall be included with a load factor of 0.6 where the load is permanent or a load factor of zero for all other conditions.
In ASD method, in case of using load combinations including over-strength factor, allowable stress may be increased up to 20%.

B. SD method:

$$(1.2 + 0.2\, S_{DS})\, D + \Omega_0 Q_E + L + 0.2\, S \qquad 2.28$$

$$(0.9 - 0.2\, S_{DS})\, D + \Omega_0 Q_E \qquad 2.29$$

where:
Ω_0= over-strength factor from Chapter 4 or 7.
The load factor for L in combination 2.28 is permitted to be taken 0.5 where unreduced live load is less than or equal to 4 kN/m^2, with the exception of garages or areas occupied as places of public assembly.
Where fluid load F is present, it shall be included with the same load factor as dead load D in combinations 2.28.
Where load H is present with increasing effects to primary loads, it shall be included with a load factor of 1.6 in all combinations. Where the effect of H resists the primary load effects, it shall be included with a load factor of 0.9 where the load is permanent or a load factor of zero for all other conditions.
The value of internal forces developed by combinations containing $\Omega_0 Q_E$ need not exceed the maximum force that can develop in the element as determined by a rational, plastic mechanism analysis or nonlinear analysis utilizing realistic expected values of material strengths.

2.2.5 Upward Force for Horizontal Cantilevers

In addition to above combinations, horizontal cantilevers shall be designed for a minimum upward force of 0.2 times the dead load.

2.3 Load Combination for Extraordinary Events

2.3.1 Scope

Extraordinary events are those other events with low probability of occurrence that are not mentioned in Section 2.2, such as explosions, vehicular impact and fire. These events could include secondary effects of earthquake, either. Strength and stability of oil industry equipment in D_2 and D_3 Seismic Design Categories (according to Section 4.5) shall be evaluated to ensure adequacy for resisting these effects and minimizing probability of chain or progressive collapse.

2.3.2 Load Combinations

2.3.2.1 Total Capacity

In SD method, for checking the capacity of a structure or structural element to withstand the effect of an extraordinary event, the following load combinations shall be considered:

$$1.2\, D + A_k + 0.5\, L + 0.2\, S \qquad 2.30$$

$$0.9\, D + A_k + 0.5\, L + 0.2\, S \qquad 2.31$$

where:
A_k = the load or load effect resulting from extraordinary event A, according to valid codes such as Reference [4].

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 that major damage and elimination of them, identified by the Responsible Design Professional, is probable shall be notionally removed, and the capacity of the damaged structure shall be evaluated using the following load combinations:

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

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

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 the second-order effects is permitted.

Chapter 3

Seismic Hazard Analysis

3. Seismic Hazard Analysis

3.1 General Provisions

In this code, regarding the structure type, one or two Seismic Hazard Levels will be used to design oil industry structures and equipment. These levels are presented in Section 3.4. To obtain design spectrum for each Seismic Hazard Levels, uncertainties in earthquake identification parameters, such as geographical location of epicenter, focal depth, fault displacement rate, directivity and rupture propagation rate, and mechanical properties of seismic source medium shall be considered. Site-specific ground motion hazard study is accomplished by two approaches: probabilistic and deterministic, which complement each other in estimation of ground strong motion hazard.

3.2 Scope

Site-Specific Hazard Study is mandatory for all oil industry complexes.
Exception: for structures in Function and Risk Category II (as described in Table 4.3) on Site Class I or II (according to Reference [2]), and structures in Function and Risk Category I, this study is not mandatory and hazard maps from Reference [2] may be used. If results of Site-Specific Hazard Study for desired site exists, these results shall be used for mentioned structures.

3.3 Definitions

Seismic Hazard: Threat to structural safety due to different levels of earthquake
Seismic Hazard Analysis: Procedure in which effective parameters of different levels of earthquake in site are identified and required calculations for determination of these parameters to use in structural analysis and design are performed.
Seismic Hazard Level: An event with specific probability of exceedance in a specified time duration for considered site, determined due to earthquake conditions.
Response Spectrum: Spectrum used for structural design for desired Seismic Hazard Level
Risk Targeted Response Spectrum: A spectrum, structure designed based on which, is anticipated to have 1% probability of collapse in MCE as stated in Section 3.4.3, in a certain time duration.
Ground Motion Prediction Equation (GMPE-Attenuation relation): Mathematical model coherent with region, which presents the average changes in strong motion effects considering faulting mechanism, source to site distance, magnitude and site class.
Uniform Hazard Spectrum: Spectrum whose values have equal probability of exceedance in a certain time duration for each structural period.
Probabilistic Seismic Hazard Analysis (PSHA): Procedure to measure probability of exceedance for desired seismic parameter (i.e.PGA, PGV and S_a) from a specific value in a certain time duration for specific site, including all active seismic sources.
Deterministic Seismic Hazard Analysis (DSHA): Procedure to determine seismic parameters for the most severe scenario at site caused by shortest site to source distance regarding the site class.
Reliability Analysis: Investigation of effects of uncertainties on each parameter used in seismic hazard analysis, specially exactness and coherence of the involved GMPE with site conditions.
Sensitivity Analysis: Determination of role of each model, parameters and inputs in seismic hazard analysis results.
Disaggregation Analysis: Determination of contribution percentage of probability of exceedance of the seismic response parameter corresponding to magnitude, M, and distance, R, values, caused by site seismic sources at period under consideration.
Site-Specific Hazard Study: All measures accomplished in order to analyze the hazards and risks caused by earthquake in site, including field investigations and statistical calculations.
Active Fault: Fault, in which an event is recorded in recent Quaternary period (almost from 11,000 years ago until today).

3.4 Seismic Hazard Levels

All of the oil industry structures and equipment shall be designed regarding the related chapter, for acceleration response spectra corresponding to one or two of Seismic Hazard Levels defined in Sections 3.4.1 through 3.4.3. Procedure to obtain appropriate spectrum is presented in Section 3.7. Earthquake return period in three Seismic Hazard Levels of this code is based on Poisson's distribution of events.

3.4.1 Seismic Hazard Level I

This Seismic Hazard Level corresponds to slight to moderate magnitude earthquake (service earthquake), that if occurs during the operational period of structure, it is expected that primary structural elements remain elastic. Spectrum corresponding to this Seismic Hazard Level is called service spectrum.

Service spectrum may vary regarding structure type, probability of exceedance and return period. For Essential Building (Chapter 6), Pipeline (Chapter 13) and Offshore Structure (Chapter 14), probability of exceedance for service earthquake in 50 years will be 70%, 50% and 20%, respectively. These values are equivalent to return periods of 40, 75 and 200 years, respectively. For other structures, considering service earthquake is not mandatory.

3.4.2 Seismic Hazard Level II

In this code, Seismic Hazard Level II (Design Earthquake) may be determined using two methods:

In method 1, by utilizing seismic parameters corresponding to Seismic Hazard Level III on bedrock from a Site-Specific Hazard Study, and applying site coefficients for soil effects (Table 3.1), and multiplying the results by 2/3, seismic parameters for Seismic Hazard Level II can be obtained.

In method 2, this Seismic Hazard Level is a relatively high magnitude event, with an exceeding probability of 10% in 50 years for an Intensity Measure of the event. Corresponding spectrum of this Hazard Level is obtained by utilizing appropriate GMPE matching the soil conditions of site. The return period of this hazard level is 475 years.

Note: The lower bound of design spectrum obtained from both methods is equal to 80% of design spectrum from Reference [5], and upper bound is the spectrum from method 2, if method 1 is used.

For pipeline, values of exceedance probability and return period of design earthquake shall be determined according to pipeline Function and Risk Category mentioned in Chapter 13. Seismic Hazard Level II shall be used to design any structure and equipment in the petroleum industries. In cases where according to Section 3.2 Site-Specific Hazard Study is not mandatory, design spectrum from Reference [2] may be used.

3.4.3 Seismic Hazard Level III

This Seismic Hazard Level represents an event with a very high magnitude (rare earthquake), that exceeding probability of an Intensity Measure of which in 50 years is 2%, equivalent to a return period of 2475 years. Resulting spectrum of this earthquake is called rare earthquake spectrum.

Seismic Hazard Level III is used to control or design seismic isolators, and offshore structures.

3.5 Selection of Ground Motion Prediction Equation (GMPE)

Selecting an appropriate GMPE consistent with site conditions, due to having greatest uncertainty, is very important. The equation should be based on data from site under investigation, moment magnitude, M_w, and fault mechanism (strike slip or dip slip). In addition, the selected equation should be consistent with magnitude upper and lower bound and definition of distance.

In case of lack of sufficient data from local earthquakes, GMPEs in which data records are expanded using simulation methods may be used. At least three equations with above specifications ranked by valid procedures shall be used in a logic tree.

3.6 Hazard Analysis Methodology

3.6.1 General

Site-Specific Hazard Study shall account for the regional tectonic setting, geology, seismicity, the expected recurrence rates and maximum magnitudes of earthquakes on known faults and source zones, and site class. In such a study, near source events (less than 10 km site to source distance) liquefaction in susceptible areas and potential of tsunami shall be considered. To determine design response spectrum of this code, see Section 3.7. In addition, where needed, uniform hazard spectrum for acceleration, velocity and displacement for desired hazard levels should be developed. If possible, all historic and instrumental data for site under investigation should be collected as a catalogue. This catalogue should be modified by deletion of foreshocks and aftershocks and completion of incomplete ranges. Gathering following data for all events are necessary:

- Event time
- Epicenter location with reasonable preciseness
- Estimated magnitude (for historical events) and recorded magnitude (for instrumental events). These values should be transformed to moment magnitude, M_w utilizing experimental equations. Samples of these equations are addressed in Reference [6].
- Focal depth
- Faulting mechanism
- Estimation of uncertainty for above parameters

Site-Specific Hazard Study in probabilistic and deterministic approaches is accomplished in accordance with Sections 3.6.2 and 3.6.3, respectively.

3.6.2 Probabilistic Seismic Hazard Analysis (PSHA)

Stages in this approach, in accordance with logical tree patterns, are as follows:

1. Determination of seismic sources in region, affecting the strong ground motion. These sources could be determined as line (where rupture orientation is known) or area (where rupture is scattered). In both cases, distance probability distribution function could be assumed uniform. Acquisition of a valid non-uniform probability distribution is also permitted.
2. Determination of magnitude probability density function, $f_M(m)$, from Equations 3.1:

$$f_M(m) = \frac{\beta e^{-\beta(m-m_{min})}}{1-e^{-\beta(m_{max}-m_{min})}} \tag{3.1a}$$

$$\beta = b\, log\, e \tag{3.1b}$$

where:

m_{max} = Earthquake maximum magnitude, from valid approaches

m_{min} = Earthquake minimum magnitude. This value shall be selected by iteration, in a manner that the IM calculated by Equation 3.2, by eliminating this value from catalogue and substituting next minimum value, differs more than 5%, compared to IM result obtained without eliminating it.

b = Regional parameter related to local seismicity, which could be derived from valid equations, such as Gutenberg-Richter law.

e = Napier's constant, equal to 2.7183

3. Calculation of value, x, of Intensity Measure (IM), such as S_a, PGA, PGV etc. from Equation 3.2.

$$\lambda_{annual} = \mu \int_{M_{min}}^{M_{max}} \int_{R_{min}}^{R_{max}} \left[1-\phi\left(\frac{x-\overline{IM}}{\sigma_{IM}}\right)\right] f_R(r)\, f_M(m) dr\, dm \tag{3.2}$$

where:

λ_{annual} = The annual rate of occurrence in site for desired IM, equal to $1/T$

T = Earthquake return period for desired Seismic Hazard Level

μ = Number of occurred events between m_{max} and m_{min}, during the desired time duration, divided by duration

$\left[1-\phi\left(\frac{x-\overline{IM}}{\sigma_{IM}}\right)\right]$ = IM probability of exceedance from the supposed value x. Standard normal function, $\phi(y)$, is in the form of Equation 3.3.

$$\phi(y) = \frac{1}{\sqrt{2\pi}} e^{-y^2/2} \quad 3.3$$

$f_R(r)$ = Probability density function for distance

$\overline{IM}$ = Average value of intensity measure at site, derived from GMPE.

σ_{IM} = Standard deviation of intensity measure values

R_{max} = Maximum distance from site to fault, corresponding to distance definition used in GMPE

R_{min} = Minimum distance from site to fault, corresponding to distance definition used in GMPE

Note: In Equation 3.2, each seismic source has a specific m_{max}. In the procedure of calculating integrals, after upper bound of integral reaches this value, the procedure should be ceased for that source.

4. Probability of exceedance, PE, by assuming a Poisson's time distribution, may be calculated from Equation 3.4:

$$PE = 1 - e^{-t\lambda_{annual}} \quad 3.4$$

where:

t = Service life of structure/component, usually taken as 50 years

5. Computation of uniform hazard acceleration response spectrum (and if necessary, velocity and displacement spectra) above ground at site under investigation for the desired return period, using a consistent GMPE. If return period is 2475 years, spectra could be established on bedrock and then transformed to above ground spectra by applying site parameters from Section 3.7.
6. Construction of the final spectrum as a code defined spectrum, in accordance with Section 3.7.

For all above stages, reliability analysis and sensitivity analysis are recommended. In this case, range of changes in models and parameters should be determined by valid methods according to the region.

3.6.3 Deterministic Seismic Hazard Analysis (DSHA)

Steps of this approach of Site-Specific Hazard Study are as follows:

1. Specification of seismic sources (similar to Section 3.6.2) and determination of maximum possible magnitude of each source (by reviewing historical data or seismic studies).
2. Specification of shortest source to site distance
3. Selection of at least 3 site consistent GMPEs (in accordance with Section 3.5)
4. Computation of average Intensity Measures, IM, for each period, using an appropriate GMPE for each source.
5. Construction of site deterministic spectrum by using 1.5 times the maximum values of all sources, from article 4.
6. Using site deterministic spectrum for different Seismic Hazard Level spectra.

3.7 Acceleration Design Spectrum

Where according to Section 3.2, probabilistic determination of spectra for different Seismic Hazard Levels is needed, deterministic spectrum shall be determined according to Section 3.6.3. Upper bound of probabilistic spectra shall be limited to values of deterministic spectra.

3.7.1 Design Spectrum

In this code, design spectrum is constructed in accordance with Figure 3.1 pattern. In this figure, spectral acceleration value, S_a, is determined from Equations 3.5 to 3.8.

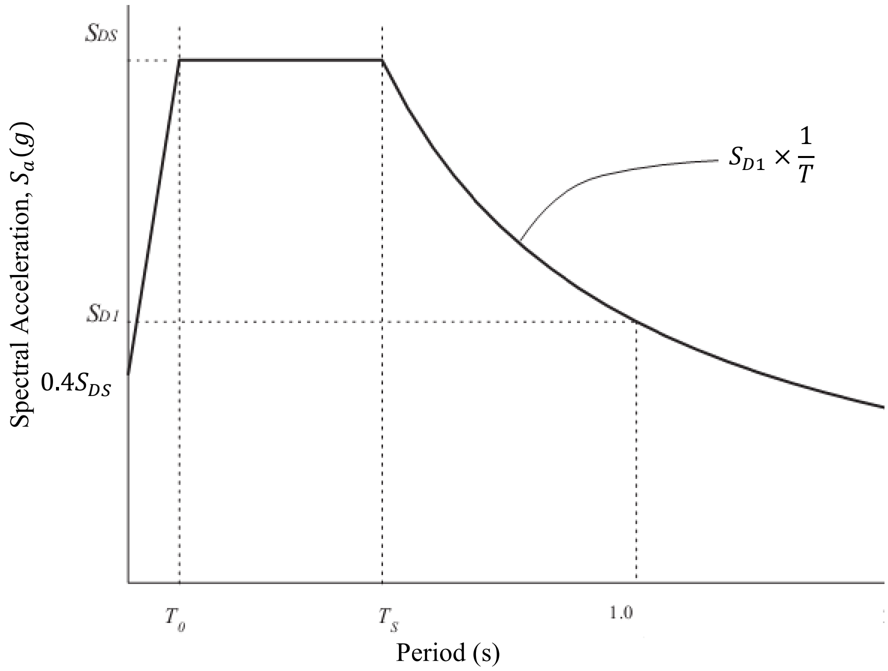


Figure 3.1 Design Code Spectrum

$S_a = 0.4\, S_{DS}$	$T = 0$	3.5
$S_a = S_{DS}\,(0.4 + 0.6\,\frac{T}{T_0})$	$0 \le T \le T_0$	3.6
$S_a = S_{DS}$	$T_0 \le T \le T_S$	3.7
$S_a = S_{D1} \times \frac{1}{T}$	$T \ge T_S$	3.8

In above equations, S_{DS} and S_{D1} are design spectrum acceleration parameters at short period (0.2 sec.) and 1 sec. period, respectively, at surface, determined from probabilistic seismic hazard analysis, in Seismic Hazard Level II assuming 5% damping ratio. If method 1 from Section 3.4.2 is used, spectral acceleration values on bedrock for desired periods are calculated, and then transformed to values at surface by Equations 3.9 and 3.10.

$$S_{DS} = \frac{2}{3} F_a S_S \qquad 3.9$$

$$S_{D1} = \frac{2}{3} F_v S_1 \qquad 3.10$$

where:

S_1 = spectrum acceleration parameter (g), corresponding to rare earthquake (Seismic Hazard Level III) for 1 sec. period at bedrock (shear wave velocity between 750 m/s and 1500 m/s), calculated from Site-Specific Study, assuming 5% damping ratio.

S_S = spectrum acceleration parameter (g), corresponding to rare earthquake (Seismic Hazard Level III) for short period (0.2 sec.) at bedrock, calculated from Site-Specific Hazard Study, assuming 5% damping ratio.

Site coefficients in the constant acceleration zone, F_a, and constant acceleration zone, F_v, are determined from Table 3.1, according to site conditions.

Table 3.1 Values of Site Coefficients for Seismic Hazard Level III

Values of F_a, according to Site Class and S_S					
Spectral Acceleration Value for Short Period on Rock, S_S					**Site Class**
$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$	
1.0	1.0	1.0	1.0	1.0	I
1.2	1.2	1.1	1.0	1.0	II
1.6	1.4	1.2	1.1	1.0	III
2.5	1.7	1.2	0.9	0.9	IV*
Values of F_v, according to Site Class and S_1					
Spectral Acceleration Value for 1 sec. Period on Rock, S_1					**Site Class**
$S_1 \leq 0.10$	$S_1 = 0.2$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.5$	
1.0	1.0	1.0	1.0	1.0	I
1.7	1.6	1.5	1.4	1.3	II
2.4	2.0	1.8	1.6	1.5	III
3.5	3.2	2.8	2.4	2.4	IV*

* For Site Class IV if any of following situations happens, instead of applying above site coefficients, precise procedures for transforming seismic parameters from bedrock to the surface shall be used:

1. Presence of liquefiable soil, quick or highly sensitive clays, and collapsible weakly cemented soils.
2. Peats and/or highly organic clays ($H > 3$m) of peat and/or highly organic clay where H represents for thickness of soil.
3. Very high plasticity clays ($H > 7.5$m with $PI > 75$).
4. Very thick soft/medium stiff clays ($H > 40$m) with $s_u < 50$ kPa.

T_0 and T_s are site class dependent periods, calculated as follows:

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}} \quad 3.11$$

$$T_s = \frac{S_{D1}}{S_{DS}} \quad 3.12$$

3.8 Risk-Targeted Acceleration Spectrum

In this code, it is allowed to use a risk-targeted spectrum instead of Seismic Hazard Level III spectrum. This spectrum is constructed based of 1% probability of structural collapse in 50 years. Steps are as follows:

1. Calculation of collapse spectral accelerations for fundamental mode period corresponding to 44 time histories from Reference [7], using Incremental Dynamic Analysis (IDA). These records should be made consistent with local seismicity by applying a spectral shape factor.
2. Calculation of structural fragility curve function, $f_{capacity}(c)$, from Equation 3.13, assuming log-normal distribution of spectral accelerations corresponding to structural collapse.

$$f_{capacity}(c) = \phi\left[\frac{\ln c - (\ln c_{10\%} + 1.024)}{0.8}\right]\frac{1}{0.8\,c} \qquad 3.13$$

3. Determination of collapse spectral acceleration probability of exceedance from Equation 3.14:

$$\mu\int_{min}^{max}\int_{R_{min}}^{R_{max}}\int_{c}\left[1-\phi\left(\frac{c-\bar{c}}{\sigma_{IM}}\right)\right]f_{capacity}(c)f_R(r)f_M(m)\,dc\,dr\,dn = 0.01 \qquad 3.14$$

where:
$\bar{c}$ = Average site spectral acceleration derived from attenuation equations
c = Spectral acceleration corresponding to 1% probability of collapse
$c_{10\%}$ = Spectral acceleration corresponding to 2% probability of exceedance in 50 years, equal to spectral acceleration corresponding to 10% probability of collapse

Chapter 4
Seismic Analysis Methods

4. Seismic Analysis Methods

4.1 General Provisions

Seismic loading and analysis can be performed by one of the elastic approaches including Equivalent Lateral Load Procedure (Section 4.8), Modal Response Spectrum Analysis (Section 4.9), and Seismic Response History Procedure (Section 4.10), or one of nonlinear approaches including pushover (NSP) or time history analysis considering required criteria.

Structure shall have appropriate horizontal and vertical seismic force-resisting systems including components with adequate stiffness, strength and ductility, so that the elements and attachments can resist elastic or inelastic deformations described in this code. The mathematical model of the structure should be consistent with material specifications and geometrical properties of structure but some simplifications such as assuming rigid subgrade are allowed, except otherwise suggested in this code such as the requirements of Chapters 5 and 13.

4.2 Classification of Structures by Configuration

Structures are classified to regular or irregular systems in horizontal and vertical directions. These classifications should be based on their structural configurations.

In Tables 4.1 and 4.2, horizontal and vertical irregularity conditions are described, respectively. In Figure 4.1, these irregularities are depicted.

Structure having an extreme weak story irregularity in height shall not be permitted. In addition, for structures assigned to Seismic Design Category (SDC) D_2 or D_3 having extreme torsional irregularity, extreme soft story and weak story are not allowed to use.

4.3 Classification of Structures by Function and Risk

Regarding their function and hazard exposure, structures are divided into four categories. Function and Risk Category and importance factor, I, are determined by probable human and economic losses caused by malfunction of the structure, as defined in Table 4.3.

Where operational access to a structure of Function & Risk Category IV is required through an adjacent structure, the adjacent structure shall conform to the requirements for Function & Risk Category IV structures.

If possible, location of structure with Function & Risk Category IV should be determined to have an appropriate distance from adjacent structures, especially structures of a lower Function and Risk Category.

Table 4.1 Vertical Structural Irregularities

Type of Irregularity and Description
a. Stiffness-Soft Story Irregularity: Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.
b. Stiffness-Extreme Soft Story Irregularity: Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.
c. Weight (Mass) Irregularity: Weight (mass) irregularity is defined to exist 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: Vertical geometric irregularity is defined to exist 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: In-plane discontinuity in vertical lateral force-resisting elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element more than the horizontal dimension of that element, or lateral force-resisting element stiffness in one story is less than the adjacent story.
f. Discontinuity in Lateral Strength–Weak Story Irregularity: Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.
g. Discontinuity in Lateral Strength–Extreme Weak Story Irregularity: Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above.

Table 4.2 Horizontal Structural Irregularities

Type of Irregularity and Description
a. Torsional Irregularity: Torsional irregularity requirements apply only to structures in which the diaphragms are rigid or semi rigid. Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion assuming $A_x = 1.0$ at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure.
b. Extreme Torsional Irregularity: Extreme torsional irregularity requirements apply only to structures in which the diaphragms are rigid or semi rigid. Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion assuming $A_x = 1.0$ at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure.
c. Reentrant Corner Irregularity: Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.
d. Diaphragm Discontinuity Irregularity: Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.
e. Out-of-Plane Offset Irregularity: Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resisance path, such as an out-of-plane offset of at least one of the vertical elements.
f. Non-Parallel System Irregularity: Non-parallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.

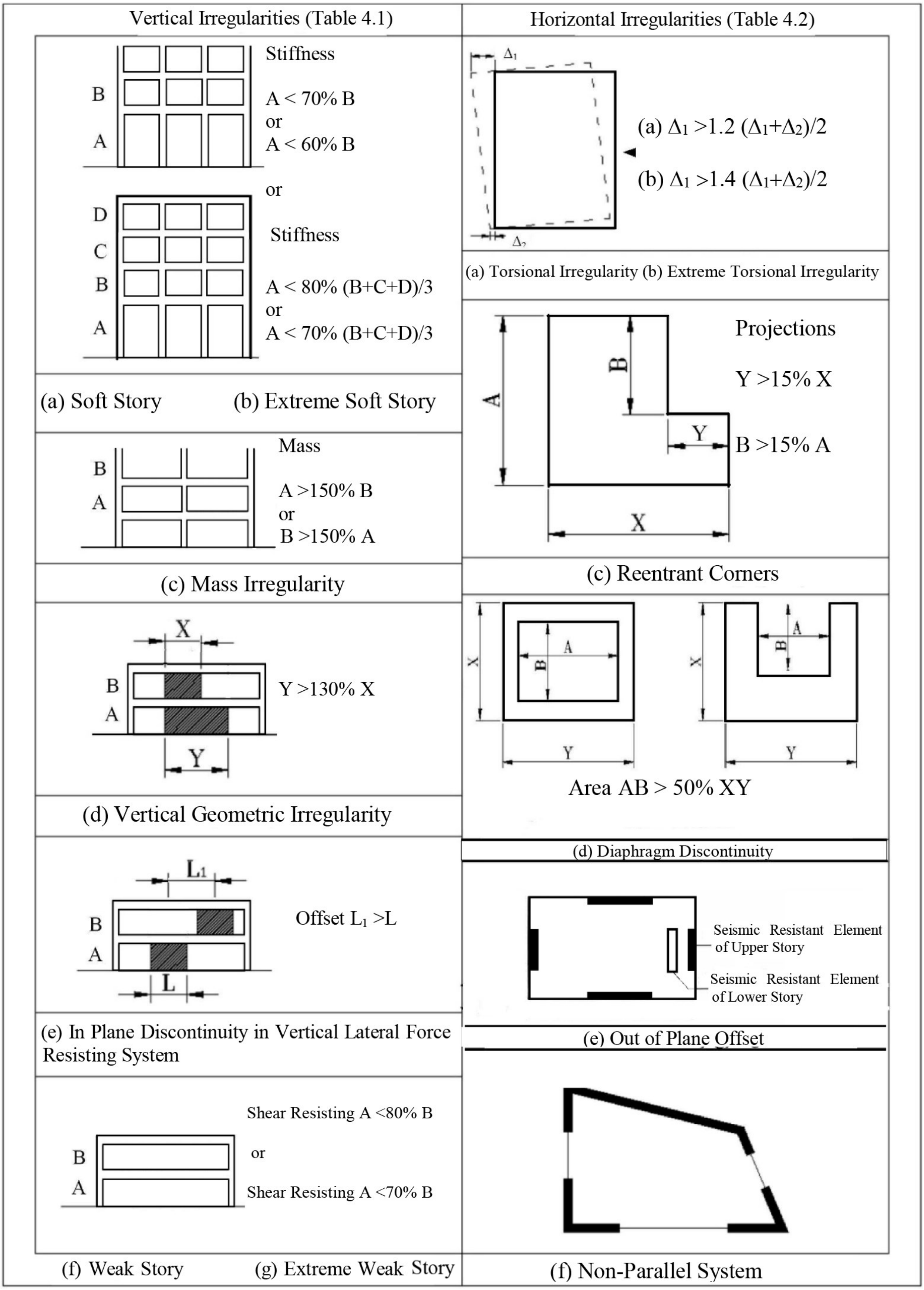


Figure 4.1 Irregularities Defined in This Code

Table 4.3 Importance Factors, Function and Risk Categorization

Structure Type	I
Function and Risk Category I : a. Structures that represent a low risk to human life in the event of failure and their damages lead to low level losses such as low-important storages b. Temporary facilities with service life less than 2 years	0.80
Function and Risk Category II : All other structures except those listed in Function & Risk Categories I, III, and IV	1.00
Function and Risk Category III : a. Structures, the failure of which could pose a substantial risk to human life, such as: - Building with a probability of gathering more than 300 people in a common location - Kindergarten or similar, with more than 150 people capacity - School or similar, with more than 250 people capacity - University or similar, with more than 500 people capacity - Medical centers with 50 or more beds, without emergency or surgery units b. Structures, not included in Function & Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure such as: - Documentation center - QC laboratory - Wastewater purification plants - Industrial wastewater purification plants - Mechanical & utility facilities servicing to Function & Risk Category III components c. Buildings and other structures not included in Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	1.25
Function and Risk Category IV : a. structures designated as essential facilities and should remain safe and serviceable after earthquake, such as: - Hospitals and other medical facilities with surgery and emergency units - Fire, first aid and police stations and related car parking places - Emergency shelters for flood, earthquake etc. - Control rooms - Mechanical facilities, utilities, and cooling systems providing services for Function and Risk Category IV equipment and structures. - Power stations and other general emergency facilities. - Telecommunication towers, fuel tanks and power stations - Firefighting water tanks and other facilities for storing water, substances and equipment for firefighting which are necessary for providing emergency services for other essential and hazardous facilities. - Air traffic control towers and emergency aircrafts hangars - Water storage structures and pumps needed for firefighting b. Facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, explosives or hazardous waste containing sufficient quantities of highly toxic substances where they exceed a threshold quantity to be dangerous to the public and is sufficient to pose a threat to the public if released.	1.5

4.4 Structural Systems and Seismic Factors

4.4.1 Systems with Uniform Stiffness in Height

Seismic force-resisting system may be chosen from one of the types indicated in Tables 4.4 for building type structures, and 7.1 or 7.2 for non-building type structures as described in this code. Also, for linear analysis of structure, designer should determine response modification factor, R_u, over-strength factor Ω_0, and deflection amplification factor, C_d, to obtain base shear, design forces of elements and design displacements of the floors. For building-type structures, these factors are defined in Table 4.4 and for other structures; they are defined in Tables 7.1 and 7.2. For other structural systems not defined in these tables, experimental results, calculations, or other valid references may be used, provided that approval of the committee issuing this code is taken.
Utilizing other structural systems and their seismic parameters mentioned in other valid references is permitted.

4.4.2 Two Stage Analysis Procedure

A two-stage Equivalent Lateral Load Procedure is permitted to be used for structures having a flexible upper portion above a rigid lower portion, provided the design of the structure complies with all of the following:

1. The stiffness of the lower portion shall be at least 10 times the stiffness of the upper portion.
2. The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure with fixed supports at the transition from the upper to the lower portion.

Two stage analysis procedure of this type of structure is performed by these steps:

1. The upper portion shall be designed as a separate structure using the appropriate values of R_u and ρ. The Equivalent Lateral Load Procedure or Modal Response Spectrum Analysis may be used for the analysis.
2. 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 amplified by the ratio of the R_u/ρ of the upper portion to R_u/ρ of the lower portion. This ratio shall not be less than 1.0. The Equivalent Lateral Load Procedure is used for the analysis.

4.5 Seismic Design Category

For every structure, Seismic Design Category is defined as follows:

- Seismic Design Category 1 (D_1): This category includes structures rather than Seismic Design Category D_2 and D_3. Also in cases which Site-Specific Hazard Study may be ignored, Seismic Design Category will be D_1.
- Seismic Design Category 2 (D_2): This category is related to structures with Function & Risk Category III in which, S_1 value is equal to or greater than 0.65.
- Seismic Design Category 3 (D_3): This category includes structures in Function & Risk Category IV where S_1 value is equal to or greater than 0.65.

Note:
For Site Class IV according to Reference [2], in sites where $S_{DS} \geq 0.75$, Seismic Design Categories D_2 and D_3 are considered for Function & Risk Categories III & IV respectively. The structures with Seismic Design Category D_2 and D_3 (excluding pipelines) shall not be constructed on sites where a known active fault (according to Chapter 3) will lead to ground failure at the structure location. When layout issues lead to construction on these sites for these Seismic Design Categories, confirmation of MOP-Department of Engineering for the calculations of seismic loading procedure shall be required.
Structural system height limits in different seismic design categories are described in Table 4.4. For seismic design category D_1 located where base design acceleration is less than or equal to 0.2 (g) (from Reference [2]), where stated NP, a height of 10 meters is permitted.

Table 4.4 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System [1]	R_u	Ω_0 [2]	C_d	Structural Height Limits (m)		
				D_1	D_2	D_3
A. BEARING WALL SYSTEMS						
1. Special reinforced concrete shear walls	5	2.5	5	50	50	30
2. Intermediate reinforced concrete shear walls	4.5	2.5	4.5	30	30	NP
3. Ordinary reinforced concrete shear walls	4	2.5	4	NP	NP	NP
4. Intermediate precast concrete shear walls	4	2.5	4	12	12	12
5. Special reinforced masonry shear walls	5	2.5	3.5	50	50	30
6. Intermediate reinforced masonry shear walls	3.5	2.5	2.25	NP	NP	NP
7. Light-frame walls with shear panels of all non-metal materials	2	2.5	2	NP	NP	NP
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 steel sheets	6.5	3	4.0	20	20	20
B. BUILDING FRAME SYSTEMS						
1. Steel eccentrically braced frames	8	2	4	50	50	30
2. Steel special concentrically braced frames	6	2	5	50	50	30
3. Steel ordinary concentrically braced frames [3]	3.25	2	3.25	10	10	NP
4. Steel buckling-restrained braced frames	8	2.5	5	50	50	30
5. Special reinforced concrete shear walls	6	2.5	5	50	50	30
6. Ordinary reinforced concrete shear walls	5	2.5	4.5	N.P.	N.P.	N.P.
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	50	50	30
9. Steel and concrete composite concentrically special braced frames	5	2	4.5	50	50	30
10. Steel and concrete composite concentrically ordinary braced frames	3	2	3	NP	NP	NP
11. Steel special shear walls	7	2	6	50	50	30
12. Steel and concrete composite plate shear walls	6.5	2.5	5.5	50	50	30
13. Special reinforced concrete shear walls	6	2.5	5	50	50	30
14. Special reinforced masonry shear walls	5.5	2.5	4	50	50	30
15. Intermediate reinforced masonry shear walls	4	2.5	4	NP	NP	NP
16. Light-frame walls with shear panels of all other non-metal materials	2.5	2.5	2.5	50	50	30
17. Light-frame (cold-formed steel) walls sheathed with steel sheets	7	2.5	4.5	50	50	30
C. MOMENT RESISTING FRAME SYSTEMS						
1. Steel special moment frames	8	3	5.5	NL	NL	NL
2. Steel intermediate moment frames [3]	4.5	3	4	10	NP[4]	NP
3. Steel ordinary moment frames [3]	3.5	3	3	NP	NP	NP
4. Concrete special moment frames	8	3	5.5	NL	NL	NL
5. Concrete intermediate moment frames	5	3	4.5	NP	NP	NP
6. Concrete ordinary moment frames	3	3	2.5	NP	NP	NP

Table 4.4 Design Coefficients and Factors for Seismic Force-Resisting Systems (continued)

Seismic Force-Resisting System [1]	R_u	Ω_0 [2]	C_d	Structural Height Limits (m)		
				D_1	D_2	D_3
7. Steel and concrete composite special moment frames	8	3	5.5	NL	NL	NL
8. Steel and concrete composite intermediate moment frames	5	3	4.5	NP	NP	NP
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	NL	NL
2. Steel special concentrically braced frames	7	2.5	5.5	NL	NL	NL
3. Steel buckling-restrained braced frames	8	2.5	5	NL	NL	NL
4. Special reinforced concrete shear walls	7	2.5	5.5	NL	NL	NL
5. Ordinary reinforced concrete shear walls	6	2.5	5	NP	NP	NP
6. Steel and concrete composite eccentrically braced frames	8	2.5	4	NL	NL	NL
7. Steel and concrete composite special concentrically braced frame	6	2.5	5	NL	NL	NL
8. Steel special plate shear walls	8	2.5	6.5	NL	NL	NL
9. Steel and concrete composite plate shear walls	7.5	2.5	6	NL	NL	NL
10. Steel and concrete composite special shear walls	7	2.5	6	NL	NL	NL
11. Steel and concrete composite ordinary shear walls	6	2.5	5	NP	NP	NP
12. Special reinforced masonry shear walls	5.5	3	5	NP	NP	NP
13. Intermediate reinforced masonry shear walls	4	3	3.5	NP	NP	NP
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF SEISMIC FORCES						
1. Steel special concentrically braced frames	6	2.5	5	10	NP	NP
2. Special reinforced concrete shear walls	6.5	2.5	5	50	30	30
3. Steel and concrete composite special concentrically braced frames	5.5	2.5	4.5	50	30	NP
4. Steel and concrete composite ordinary concentrically braced frames	3.5	2.5	3	NP	NP	NP
F. CANTILEVER COLUMN SYSTEMS [5]						
1. Special steel or reinforced concrete moment frames	2.5	1.25	2.5	10	10	10
2. Ordinary steel moment frames	1.25	1.25	1.25	NP	NP	NP
3. Intermediate concrete moment frames	1.5	1.25	1.5	NP	NP	NP
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	3	3	3	NP	NP	NP

1. For determining presented structural systems, refer to authoritative codes.
2. Where the tabulated value of the over-strength 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.
3. Steel ordinary concentrically braced frames are permitted in single-story buildings up to a structural height, 20 m where the dead load of the roof does not exceed 0.96 kN/m^2. In addition, the dead load of the exterior walls more than 10 m above the base tributary to the moment frames shall not exceed 0.96 kN/m^2.
4. In structures with floor dead loads less than 1.61 kN/m^2 and dead load of peripheral walls less than 0.96 kN/m^2, structural height is limited to 10 m.
5. The required axial strength of individual cantilever column elements, considering only the load combinations expressed in Sections 2.2.1 and 2.2.2 including seismic load effects, shall not exceed 15 percent of the available axial strength, including slenderness effects. Foundation and other elements used to provide overturning resistance at the base of cantilever column elements should be designed to resist the seismic load effects including over-strength factor of Section 2.2.4.

4.6 Redundancy Factor ρ

A redundancy factor, ρ, shall be assigned to the seismic force-resisting system in each of the two orthogonal directions for all structures.

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

1. Where Seismic Design Category is D_1 according to Section 4.5 and S_{DS} is less than 0.5 or S_{D1} is less than 0.2 based on the Site-Specific Hazard Study.
2. Drift, Δ_x calculations
3. P-Delta Effects
4. Design of non-structural components
5. Design of non-building structures with ground-supported industrial members (Section 7.3)
6. Design of collector elements, splices, and their connections for which the seismic load effects including over-strength factor of Section 2.2.4 are used.
7. Design of members or connections where the seismic load effects including over-strength factor of Section 2.2.4 are required for design.
8. Diaphragm loads determined according to Section 4.11
9. Structures with damping systems designed in accordance with Chapter 10.
10. Design of structural walls for out-of-plane forces, including their anchorage.
11. For regions having low relative seismic hazard, regarding Reference [2]

For all other cases, ρ value is considered to be 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0:

Condition 1: In each story resisting more than 35 percent of the base shear in the direction of interest, removing structural elements indicated in Table 4.5 shall not result in 35 percent or more reduction of story lateral strength.

Condition 2: Structures that are regular in plan at all levels provided that the seismic force-resisting systems consist of at least two bays of seismic force-resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height for light-frame construction (Table 4.4).

Table 4.5 Requirements for Each Story Resisting More than 35% of the Base Shear ($\rho = 1$)

Lateral Force-Resisting Element	Requirements
Braced frames	Removal of an individual brace, or connection
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 or wall pier with a height-to-length ratio greater than 1.0 within any story, or collector connections thereto, (Shear wall or pier is determined based on Figure 4.2.)
Cantilever columns	Loss of moment resistance at the base connections of any single cantilever column
Other	No requirements

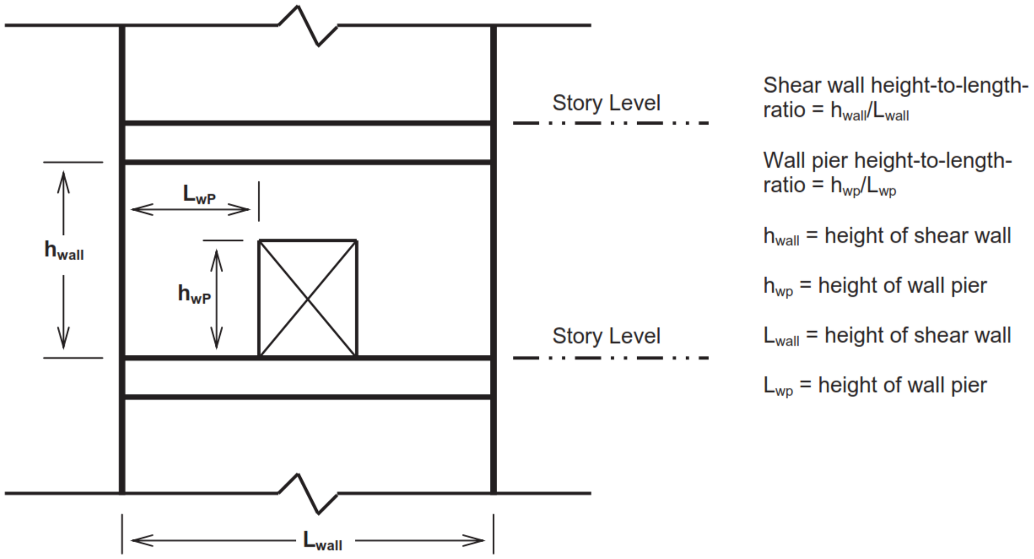


Figure 4.2 Shear Wall and Wall Pier Height-to-Length Ratio Determination

4.7 Seismic Load Applying

Horizontal seismic load direction should be considered so that maximum effects for structural elements appear. Structure can be analyzed for each horizontal seismic load effect independently in any orthogonal direction, unless in conditions mentioned below:

- Columns or walls which are parts of two or more seismic force-resisting systems, where seismic compressive axial load on which in each direction is equal to or greater than 20% of compressive axial load design capacity.
- Structures having horizontal irregularity type f in Table 4.2

In these cases, seismic load application is defined in each direction as follows:

A. In Equivalent Lateral Load Procedure (Section 4.8), Modal Response Spectrum Analysis (Section 4.9) and/or Linear Seismic Response History Procedure (Section 4.10.2), 100 percent of the forces for one horizontal direction plus 30 percent of the forces for the perpendicular direction shall be applied simultaneously and the most critical effect for structural and foundation design is selected

B. In Elastic Seismic Response History Procedure (Section 4.10.2) or Inelastic Seismic Response History Procedure (Section 4.10.3), a pair of orthogonal components of ground motion for both horizontal orthogonal directions shall be applied to the structure simultaneously.

4.8 Equivalent Lateral Load Procedure

4.8.1 General Provisions

In Equivalent Lateral Load Procedure, base shear is determined based on Section 4.8.2 and applied to structure according to Section 4.8.4. Using this procedure is permitted where:

- Buildings with Function & Risk Categories I & II, up to 2 stories from base level (For base level determination refer to Section 4.17)
- Light-frame systems in Function & Risk Categories I and II, up to 3 stories
- Regular structures with these conditions:
    - A. $T < 3.5T_S$
    - B. Structural height less than 50 m, and
    - C. Structures with light-frame systems
- Structures with height less than 50 m, having horizontal irregularities type “c” through “f” in Table 4.2 or all vertical irregular structures with type “e” through “g” in Table 4.1

In all other cases, Seismic Response History Procedures (Section 4.9 or 4.10) shall be performed. For non-building structures, criteria of relevant chapters shall be considered.

T_S can be derived from equation $T_S = 1_{sec} \times S_{D1}/S_{DS}$ and in this section only from Site-Specific Hazard Study (Chapter 3).

4.8.2 Seismic Base Shear

The seismic base shear, V_u, shall be determined from Equation 4.1.

$$V_u = \frac{S_a}{R_u/I} W \qquad 4.1$$

where:
S_a= mapped spectral response acceleration parameter (g), determined from site-specific ground motion hazard analysis (Chapter 3), using a 5% damping ratio. Instead, in cases permitted (Section 3.2), the design spectrum from Reference [2] can be used ($S_a = A.B$).
I= importance factor as prescribed in Table 4.3.
W= effective seismic weight of the structure, including dead loads and other loads, calculated from base level (Section 4.17). Where equivalent panel loads are included in dead load determination, it should not be less than 0.5 kN/m^2. In determination of W, following considerations shall be included:

1. In storage areas, a minimum value of 25% of the live load.
2. Partition live loading according to Reference [3]
3. Operational weight of permanent installed equipment
4. 20% of the greater value between snow load or roof live load (According to Reference [3])

For non-building structures, refer to Chapter 7.
R_u= response modification factor, from Chapter 4.
R_u is based on Strength Method. In order to use this factor in ASD Method, seismic load effects, E, in ASD load combinations (Section 2.2.1) are multiplied by factor of 0.7.

4.8.2.1 Minimum Base Shear

Base shear, V_u, for building type structures, shall not be considered less than the value resulted from Equation 4.2:

$$V_{min} = 0.044\, S_{DS}\, WI \ \geq 0.01W \qquad 4.2$$

Also, if $S_1 \geq 0.6$, minimum value of base shear shall not be considered less than value resulted from Equation 4.3:

$$V_{min} = 0.5\, S_1 W/(R_u/I) \qquad 4.3$$

4.8.3 Fundamental Period Determination

To determine the fundamental period of structures, analytical methods or experimental equations can be used. Some convenient experimental formulas for non-building structures are mentioned in Appendix 2. For building type structures, the experimental period may be determined from Equation 4.4:

$$T = C_t H^x \qquad 4.4$$

where:
H = structural height from base level (m) according to Section 4.17
x = height exponent parameter presented in Table 4.6
C_t = period coefficient listed in Table 4.6
For building structures, analytical to experimental fundamental period ratio shall not be greater than C_{Tu} in Table 4.7

Table 4.6 Parameters Used to Determine Structural Experimental Period

Lateral Force-Resisting System	C_t	x
Steel Moment Frame	0.072	0.8
Concrete Moment Frames	0.047	0.9
Eccentric Steel Braced Frames	0.073	0.75
Buckling Resisting Steel Braced Frames	0.073	0.75
Other Systems	0.050	0.75

Table 4.7 Upper Bound for Analytical Period Factor

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

Note 1. If infill walls limit the displacement of stories, the fundamental period coefficients shall be considered as "Other Systems" row in Table 4.6.
Note 2. The approximate fundamental period for masonry or concrete shear wall structures is permitted to be determined from Equation 4.5:

$$T = \frac{0.0062}{\sqrt{C_w}} H \qquad 4.5$$

Where C_w is calculated from Equation 4.6 as follows:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{m} \left(\frac{H}{h_i}\right)^2 \frac{A_{si}}{\left[1 + .083\left(\frac{h_i}{D_i}\right)^2\right]} \qquad 4.6$$

where:
A_B = area of the base of structure
m = number of shear walls in the building effective in resisting lateral forces in the direction under consideration
h_i = height of shear wall i
A_{si} = web area of shear wall i in the direction under consideration
D_i = length of shear wall i

4.8.4 Vertical Distribution of Seismic Forces

Lateral seismic design force at level x, (F_x); shall be determined from Equations 4.7 and 4.8.

$$F_x = C_{vx} V_u \qquad 4.7$$

$$C_{vx} = \frac{w_x {h_x}^k}{\sum_{i=1}^{n} w_i {h_i}^k} \qquad 4.8$$

where:
C_{vx} = vertical distribution factor
w_i and w_x = the portion of the effective seismic weight assigned to level i or x
h_i and h_x = height of level i or x, calculated from base level
n = number of stories (levels)
k = distribution exponent related to the structure period determined from Equation 4.9.

$$k = \begin{cases} 1 & T \leq 0.5 \\ 0.5T + 0.75 & 0.5 < T < 2.5 \\ 2 & T \geq 2.5 \end{cases} \tag{4.9}$$

Base level is defined according to Section 4.17.

4.8.5 Story Shear

The seismic design story shear between levels x and $x-1$ (V_x) shall be determined from Equation 4.10.

$$V_x = \sum_{i=x}^{n} F_i \tag{4.10}$$

where:
F_i = seismic lateral force at level i from Equation 4.7
The seismic design story shear shall be distributed to the various elements of the seismic force-resisting system in the story under consideration based on the relative lateral stiffness of the resisting elements and the diaphragm.

4.9 Seismic Response Spectrum Procedure

4.9.1 General Provisions

Seismic Response Spectrum Procedures shall be performed on 3D models. Two-dimensional analyses for regular structures with independent seismic force-resisting systems in each direction are permitted. If roofs, compared to seismic force-resisting elements; are not rigid (Section 3, Appendix 4 of Reference [2]), their flexibility shall be considered in the model.

4.9.2 Natural Periods and Modes

To perform a Seismic Response Spectrum Procedure, natural periods of structure, eigenvectors (ϕ), and modal masses in direction under consideration shall be determined. In each basic orthogonal direction of structure, sufficient number of modes shall be considered so that the sum of effective modal masses is not less than 90% of the total mass of the structure.

4.9.3 Modal Base Shear

Base shear for mode m. V_m , is calculated form Equation 4.1.

$$\begin{aligned} V_m &= C_{sm} W_m \\ W_m &= \left(\sum_{i=1}^{n} w_i \phi_{im}\right)^2 \Bigg/ \sum_{i=1}^{n} w_i \phi_{im}^2 \\ C_{sm} &= \frac{S_{am}}{R_u / I} \end{aligned} \tag{4.11}$$

where:

C_{sm} = seismic response coefficient for mode m
W_m = effective seismic weight for mode m
n = number of floors
ϕ_{im} = modal amplitude in level i for mode m
S_{am} = spectral acceleration parameter of mode m

4.9.4 Modal Lateral Forces

Lateral force for level x in mode m, F_{xm}, shall be calculated from Equations 4.12 and 4.13.

$$F_{xm} = C_{vxm} V_m \tag{4.12}$$

$$C_{vxm} = \frac{w_x \phi_{xm}}{\sum_{i=1}^{n} w_i \phi_{im}} \tag{4.13}$$

where:
C_{vxm} = vertical distribution factor for mode m at level x
ϕ_{xm} = modal amplitude at level x for mode m

4.9.5 Modal Displacements

Total lateral displacement at level x for mode m, δ_{xm}, shall be calculated from Equations 4.14 and 4.15.

$$\delta_{xm} = \frac{C_d \delta_{xem}}{I} \tag{4.14}$$

$$\delta_{xm} = \left(\frac{g}{4\pi^2}\right)(T_m^2 \phi_{xm} C_{sm}) \tag{4.15}$$

where:
δ_{xem} = elastic displacement at level x for mode m
T_m = natural period for mode m

4.9.6 Modal Combination

Considered response can be obtained from combination of the modes by one of following procedures. Story shear, overturning moment and drift for each level in various modes shall be calculated. Response values of above parameters in modes of interest shall be combined using one of the following methods to obtain respective design values:

- Square Root of the Sum of the Squares (SRSS) method, if modes are independent,
- Complete quadratic combination (CQC) method, if modes are dependent.

4.9.7 Modification of Responses

If design base shear obtained from this procedure is less than 85% of the value from Equivalent Lateral Load Procedure, design shear force shall be multiplied by ratio of base shear resulted from Equivalent Lateral Load Procedure to the base shear of Modal Response Spectrum Analysis.

4.10 Seismic Response History Procedures

4.10.1 General Provisions

In this method, the structure shall be analyzed using recorded ground motions. Ground motions shall be compatible to the faulting mechanism of seismic source, respective earthquake magnitude, distance to site and soil conditions. Seismic Response History Procedure is permitted to apply, assuming elastic or inelastic behavior of the structure as mentioned in following sections.

4.10.2 Elastic Response History Procedure

Forces and deflections in this method are derived with elastic structural behavior assumption. A regular structure (Section 4.2) with independent orthogonal lateral force resisting elements is permitted to be analyzed using a single component ground motion record (Section 4.10.2.1). Otherwise, regarding Section 4.7, a 3D modeling of the structure is necessary and it shall be analyzed using a set of ground motion records as mentioned in the following sections.

4.10.2.1 Elastic Analysis with Single Component Ground Motion Record

In this analytical method, a single component ground motion a used for structural analysis. Therefore, a number of appropriate individual ground motions are applied. These records shall be scaled such that the average value of the 5% damped response spectra for the suite of motions is not less than the design response spectrum for the site in periods ranging from $0.2T$ to $1.5T$, where T is the natural period of the structure in the fundamental mode for the direction of response being analyzed.

4.10.2.2 Elastic Analysis with Double Component Ground Motion Record

In this method, ground motions shall consist of pairs of two appropriately scaled horizontal ground motion acceleration components. Scaling pair of records and comparing them to the appropriate spectrum shall be done as follows:

A. For each component, a 5% damped response spectra is developed.
B. Two spectrum values are combined in the period range from $0.2T$ to $1.5T$ using the SRSS combination rule.
C. The above operations are repeated for every pair of records.
D. Each pair of motions shall be scaled such that in the period range from $0.2T$ to $1.5T$, average of the SRSS combined spectra from Article b does not fall more than 10% below the 1.3 times corresponding ordinate of the response spectrum used. A same scaling factor value for two horizontal components shall be used.

If rare earthquake (Section 3.4.3) is obtained considering 1% probability of exceedance in 50 years and directivity of the event, design spectrum values obtained from Site-Specific Hazard Study according to Chapter 3 are used for comparison.

At sites within 5 km of the active fault with high hazard potential, each pair of components shall be rotated to the fault-normal and fault-parallel directions of the causative fault and shall be scaled so that the average of the fault-normal components of resultant spectrum is not less than the rare earthquake corresponding spectrum for the period range from $0.2T$ to $1.5T$.

4.10.2.3 Modification of Responses

For each ground motion analyzed, the individual response parameters shall be multiplied by the following scalar quantities:

A. Base shear and element forces shall be multiplied by I/R_u .
B. Story drifts shall be multiplied by C_d/R_u.

Where the maximum scaled base shear from Article A, V_i, is less than 85% of the value of V determined from Section 4.8.2.1, the maximum scaled member forces, Q_{Ei} shall be modified again and multiplied by V_{min}/V_i.

Where the maximum scaled base shear from section A, V_i, is less than 85% of the value determined from Equation 4.3, maximum scaled drifts from Article B shall be multiplied by $0.85V_{min}/V_i$, where V_{min} is determined from Equation 4.3.

If at least seven ground motions are used in analysis, the maximum-scaled member forces and the design story drift used in the evaluation of drift in accordance with Table 4.8 are permitted to be taken respectively as the average of the scaled Q_E and drift values determined from the analyses and scaled as indicated in the preceding text. If fewer than seven ground motions are applied, the maximum member forces and the design story drift shall be taken as the maximum value of the scaled Q_E and drift values determined from the analyses. However, at least five ground motions should be considered.

Where this code requires consideration of the seismic load effects including over-strength factor Ω_0 in load combinations of Section 2.2, the value of $\Omega_0\Omega_E$ need not be taken larger than the maximum of the unscaled value, Q_{Ei}, obtained from the analyses.

4.10.3 Nonlinear Seismic Response History Procedure

Where a Nonlinear Seismic Response History Procedure is applied, a mathematical model of the structure shall be developed that represents the spatial distribution of mass throughout the structure and inelastic behavior of structural materials. The hysteretic behavior of elements shall be modeled consistent with suitable laboratory test data and shall account for all significant yielding, strength degradation, stiffness degradation, and hysteretic pinching indicated by such test data. Nominal strength of elements shall be based on expected values considering material over-strength, strain hardening, and hysteretic strength degradation.

4.10.3.1 Design Earthquake Response Parameters

Design of members shall include inelastic story drifts, individual member forces and member inelastic deformations.

If at least seven ground motions are used, the design values of member forces, member inelastic deformations, and story drift, are permitted to be taken as the average of the relevant values determined from the analyses. If fewer than seven ground motions are analyzed, these parameters shall be taken as the maximum value of the relevant values determined from the analyses.

Design member deformation shall not exceed two-thirds of a value that results in loss of ability to carry gravity loads, or results in deterioration of member strength to less than two-thirds of the peak value. The design story drift, obtained from the analyses shall not exceed 1.25 times the drift limits from Table 4.8.

4.10.3.2 Design Review

A design review of the seismic force-resisting system and the structural analysis shall be performed by an independent team of registered design professionals in the appropriate disciplines and others experienced in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under extreme cyclic loads.

The design review shall include, but need not be limited to, the following:

1. Review of any site-specific seismic criteria regarding the Chapter 3
2. Review of acceptance criteria used to demonstrate the adequacy of structural elements and systems to withstand the calculated force and inelastic deformation demands, together with laboratory and other data used to substantiate these criteria
3. Review of the preliminary design including selection of structural system and the configuration of structural elements
4. Review of the final design of the entire structural system and all supporting analyses

4.11 Diaphragm and its Components

Relative stiffness of the diaphragm and vertical elements of the seismic force-resisting system shall be included in structure analysis.

If the maximum in-plane deformation of the diaphragm under lateral load is less than half of the average relative lateral displacements of vertical members of the story, it shall be considered rigid, and if it is more than two times of this value, it shall be assumed flexible (Figure 4.3). If it can not be assumed to be rigid or flexible, the appropriate stiffness of the semi-rigid diaphragm shall be included in the analysis.

Diaphragms consisting of reinforced concrete slabs or composite cast in situ metal decks with a span-to-width ratio of three or less in regular structures are permitted to be assumed rigid.

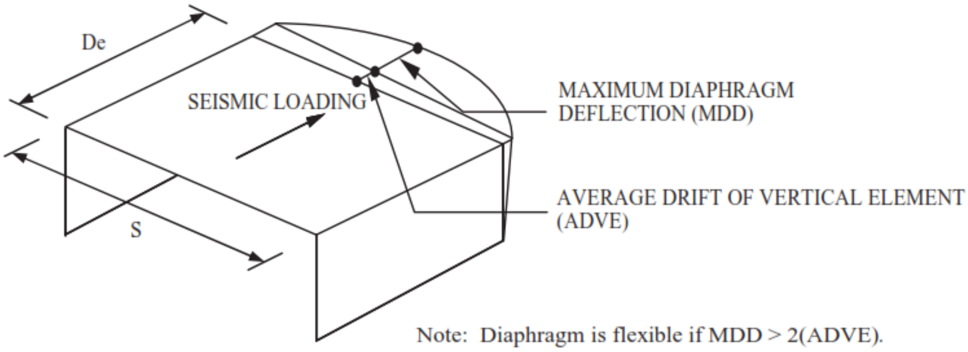


Figure 4.3 Evaluation of Diaphragm Rigidity

4.11.1 Diaphragm Design

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

For each level, diaphragms shall be designed to resist design seismic forces from two following values:

A. Lateral load obtained from Equivalent Lateral Load Procedure (Section 4.8) or Modal Response Spectrum Analysis (Section 4.9)

B. Lateral load, F_{px}, calculated from Equation 4.16.

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \tag{4.16}$$

The force determined from Equation 4.16 shall not be less than Equation 4.17 and need not exceed Equation 4.18.

$$F_{px} = 0.2\, S_{DS}\, I w_{px} \tag{4.17}$$

$$F_{px} = 0.4\, S_{DS}\, I w_{px} \tag{4.18}$$

where:

F_i = the design force applied to level i which is calculated from Equivalent Lateral Load Procedure or Modal Response Spectrum Analysis.

w_{px} = the seismic weight tributary to the diaphragm at level x

Where there are offsets in placement of lateral load resisting system elements above and below the diaphragm, or there are changes in relative lateral stiffness in the vertical elements, diaphragms shall be designed to resist forces resulted from these irregularities added to forces determined from Equation 4.16. The redundancy factor, ρ, to determine inertial forces F_i calculated in accordance with Equation 4.16, for all Function & Risk Categories shall equal 1.0. For transfer forces, the redundancy factor, ρ, shall be the same as that used for the structure.

4.11.2 Design of Collector Elements

Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces (Figure 4.4).

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

1. Forces calculated using the seismic load effects including over-strength factor (Section 2.2.4) with seismic forces determined by the Equivalent Lateral Load Procedure (Section 4.8) or the Modal Response Spectrum Analysis (Section 4.9)
2. Forces calculated using the seismic load effects including over-strength factor (Section 2.2.4) with seismic forces determined by Equation 4.16.
3. Forces calculated using combinations from Section 2.2.3, in which seismic force is determined from Equation 4.17.

4.11.3 Increase in Forces Due to Irregularities

For structures having a vertical structural irregularity of Type e in Table 4.1 or a horizontal structural irregularity of Type a to e in Table 4.2, the design forces determined from Section 4.11.1 shall be increased by 25 percent 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 over-strength factor of Section 2.2.4 need not be increased.

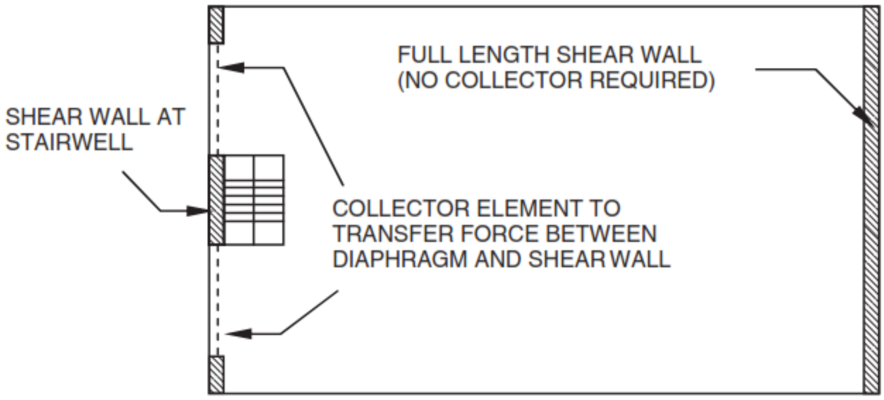


Figure 4.4 Collectors

4.12 Structural Wall

Structural walls are elements considered as parts of a vertical or lateral load resisting system. In addition to design code provisions, the requirements of this section shall be considered for structural wall design. If load on a gravity wall is greater than 3 kN/m, specifications of Section 4.12.1 shall be considered. Gravity wall is the structural wall, gravity load on which (excluding wall weight) is greater than 3 kN/m for concrete or masonry walls and 1.5 kN/m for steel walls.

4.12.1 Design for Out of Plane Forces

Structural wall and its connections shall be designed for a force perpendicular to the wall equal to $0.4S_{DS}I$ times the wall weight, but not less than 10% of the wall weight. Connections between wall elements and the structure shall have adequate ductility, rotational capacity and strength to withstand shrinkage, temperature variations, and foundation uneven settlings, simultaneously with seismic loads.

4.12.2 Anchorage of Structural Walls to Diaphragms and its Implied Loads

The anchorage of structural walls to supporting construction shall provide a direct connection capable of resisting F_p, from Equation 4.19. F_p shall not be less than $0.2K_aIw_p$ and need not be greater than 2.0.

$$F_p = 0.4\, S_{DS} k_a\, I w_p \qquad 4.19$$

$$K_a = 1 + L_f/30 \qquad 4.20$$

where:
F_p = design force in the individual anchors
S_{DS} = short period design spectral response acceleration parameter (Chapter 3)
I = the importance factor determined in accordance with Section 4.3
K_a = amplification factor for diaphragm flexibility
L_f = the span (m) of a flexible diaphragm that provides lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction of interest; use zero for rigid diaphragms.
w_p = the weight of the wall tributary to the anchor
Where the anchorage is not located at the roof and all diaphragms are not flexible, the value from Equation 4.19 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 average height of the roof above the base. Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.2 m.

4.13 Story Torsion

In structures with rigid or semi rigid diaphragms, seismic lateral load shall be distributed between story seismic resistant frames considering the torsional moment, M_t, resulting from eccentricity between the locations of center of mass and the center of rigidity of above stories.

4.13.1 Accidental Torsion

In addition to the torsional effects, the accidental torsion, M_{ta}, in plan shall be considered in lateral load distribution, caused by notional displacement 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 at both perpendicular directions regarding the mass center at all levels. In cases where orthogonal component effect is considered, this notional displacement need not to be included in two directions simultaneously and only the critical direction should be considered.

4.13.2 Torsional Moment Amplification Factor

Structures with rigid diaphragms, where torsional irregularities exist as defined in Table 4.2, shall have the effects accounted for by multiplying the accidental torsional moment, M_{ta}, at each level by a torsional amplification factor, A_x, at each level determined by Equation 4.21:

$$A_x = \left(\frac{\delta_{max}}{1.2\, \delta_{ave}}\right)^2 \qquad 4.21$$

where:
δ_{max} = maximum displacement at level x not considering A_x.
δ_{ave} = average of the displacements at the extreme points of structure at level x not considering A_x (Figure 4.5).
A_x shall not be considered less than 1.0 and need not be taken more than 3.0.
The more sever loading, with or without A_x, for each element shall be considered for design.

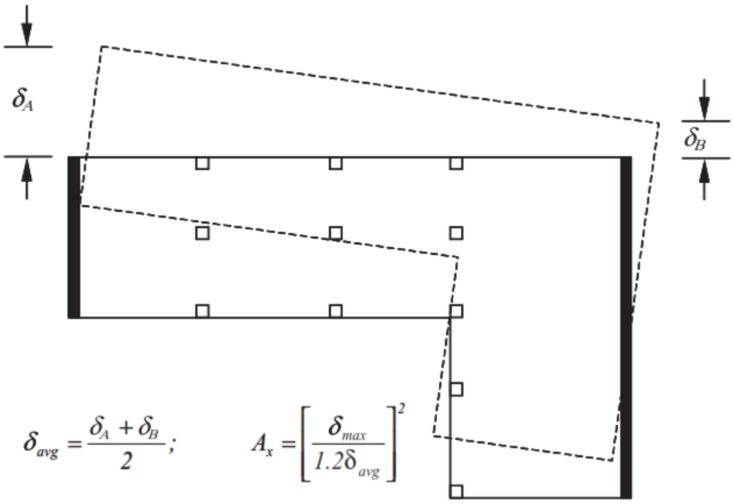


Figure 4.5 Average Lateral Displacement at Each Level

4.14 Story Displacement

To determine story displacements, the structure shall be modeled considering the stiffness and strength of elements with considerable roles in base shear distribution, and represents reasonable distribution of mass and stiffness. In addition, the modeling shall include:

1. Stiffness degrading in reinforced concrete and masonry elements, considering cracking effects and nonlinear behavior of materials.
2. Panel zone deformation effects on total story displacements in moment resisting frames.

4.14.1 Design Lateral Displacement and Design Drift

The design lateral displacement (nonlinear) at level x shall be calculated from Equation 4.22:

$$\delta_x = \frac{C_d\,\delta_{xe}}{I} \qquad 4.22$$

where:

δ_{xe}= elastic lateral displacement at level x, according to lateral load distribution from Equivalent Lateral Load Procedure or Modal Response Spectrum Analysis. For displacement determination, the redundancy factor, ρ, shall equal 1.0.

The total design drift at level x, Δ_x, is defined as the difference between displacements of centers of masses of two adjacent levels, as mentioned in Equation 4.23:

$$\Delta_x = \delta_x - \delta_{x-1} \qquad 4.23$$

To determine δ_{xe} in ASD & SD methods, earthquake load factor shall equal 1.0.

To determine displacements, considering limitation in period calculations according to Section 4.8.3 is not necessary. In addition, Equation 4.2 need not be considered.

Note 1. In structures having torsional irregularities, drift at each story is defined as the maximum differential displacements in each corresponding corner at top and bottom levels of the story.

Note 2. In linear analysis of concrete structures, to consider cracking effects, the effective flexural rigidities of elements shall be considered as follows:

- Beams: $0.35EI_g$
- Columns: $0.7EI_g$
- Uncracked walls in flexure: $0.7EI_g$
- Cracked walls in flexure: $0.35EI_g$

where, I_g is the moment of inertia for uncracked gross section excluding reinforcement.
In Figure 4.6, δ_{xe}, δ_x and Δ_x are depicted for $x = 1$ & $x = 2$ levels.

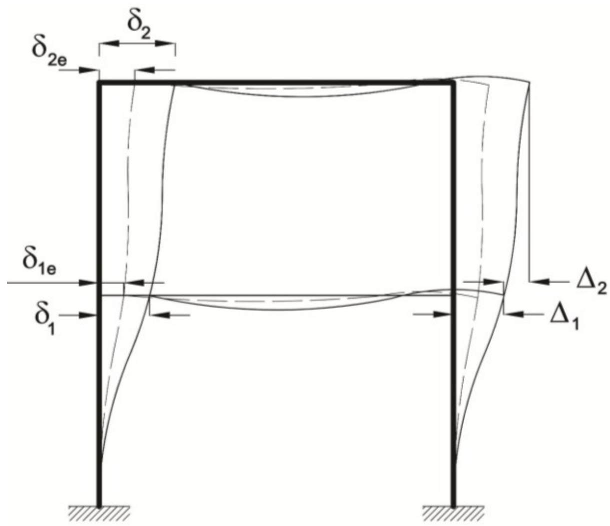


Figure 4.6 Elastic and Inelastic Displacements

4.14.2 Story Drift Control

Story drifts at level x, Δ_x/h_{sx}, shall not exceed values shown in Table 4.8.
h_{sx} is the height of story below level x.
Note 1. For building type moment frame structures, the allowable story drift, Δ_a from Table 4.8 shall be divided by the redundancy factor, ρ (Section 4.6).
Note 2. For one-story buildings where panels, exterior walls and roof are designed for story drifts, no drift limitation exists. In Table 4.8, masonry cantilever shear wall is a structure in which each wall is designed as a vertical element cantilevered from its base or foundation support and constructed so that moment transfer between shear walls (coupling) is negligible.

Table 4.8 Allowable Story Drift, Δ_a

Structure Type	**Function & Risk Category**		
	I-II	**III**	**IV**
Structures, other than masonry shear wall structures, 4 stories or less, with interior walls, partitions, ceilings, and exterior wall systems designed to accommodate the story drifts.	0.025	0.020	0.015
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	0.020	0.015	0.010

4.14.3 Separation Joint

The free space between adjacent structures at level x, shall not be less than the square root of sum of the squares of total displacements, δ_{MT}, of the structures according to Equation 4.24.

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \qquad 4.24$$

For each of adjacent structures, δ_M value is calculated from Equation 4.25.

$$\delta_M = \frac{C_d \delta'_{max}}{I} \qquad 4.25$$

where:
δ'_{max} = maximum elastic displacement in horizontal plane, at the critical location (probable point of collision) considering torsional moment amplification factor.
However, building shall be distanced from the boundary of site at least by δ_M.

4.15 Second Order Effects (P-Delta)

Second order effects including $P - \Delta$ & $P - \delta$, shall be considered in structural analysis and design.
$P - \delta$ effect is related to member curvature usually considered during design stage in most codes.
$P - \Delta$ effect is related to magnification of story drifts, element moments and shears caused by horizontal displacement of stories and shall be calculated in accordance with this section.
Structural analysis programs can be used for including $P - \Delta$ effects explicitly. Application method for $P - \Delta$ inclusion depends on structure type and analysis procedure as determined by structural analysis and design codes. However, specifications of this section shall also be considered.
Where, stability index, θ_x, is greater than 0.1, $P - \Delta$ effects shall be applied. However, stability index shall not be greater than θ_{max} in accordance with Equation 4.27. For calculation of stability index at each level, x, Equation 4.26 or an appropriate software may be applied.

$$\theta_x = \frac{P_x \Delta_{xe} I}{V_x h_{sx}} \qquad 4.26$$

$$\theta_{max} = \frac{0.5}{C_d} \leq 0.25 \qquad 4.27$$

where:
P_x = total vertical load over level x, including non-factored dead load and live load (this vertical load shall only be used to determine the stability index in both ASD and SD methods).
V_x = seismic shear force acting between levels x and $x - 1$, resulted from first order analysis.
Δ_{xe} = total design drift at level x from Equation 4.19, calculated by first order analysis
h_{sx} = the story height below level x
After applying $P - \Delta$ effects to determine end forces of each element, $P - \delta$ effects (secondary intensification effects along member length) shall be considered rationally. To achieve this, software having the ability to apply $P - \delta$ effects or design code regulations considering the geometrical nonlinear effects (including $P - \Delta$ or $P - \delta$) are permitted to be used.

4.16 Overturning

The structure shall be designed and controlled to resists seismic overturning moments.

4.17 Seismic Base Level

Seismic base level, is the level at which transition of seismic horizontal load effects to the structure is considered. In determination of this level, following notes shall be considered:

A. For buildings on level sites and without basement where the vertical elements of the seismic force-resisting system are supported at various elevations on top of footings, pile caps or perimeter foundation walls, the base is generally established as the lowest elevation of the elements supporting the vertical elements of the seismic force-resisting system.
B. For a building with a basement, if structure and retaining walls are independent and not connected, the base is generally established at top of foundation level.

In buildings on level sites with the retaining walls being part of the seismic force-resisting system, base level may be assumed to be at the first lower level story or same level of grade where:

A. No basement floors are flexible diaphragms.
B. Competent soil exists behind full height of retaining wall. Competent soil is defined stiff soil (not Site Class IV according to Reference [2] where N_{SPT} is greater than 20), not liquefiable for rare earthquake condition, not sensitive or very sensitive clays, and having enough cohesion.

In such a building, base level may be assumed to be at the first upper level of ground if:

A. Base level height from ground level should not be taken as greater than half of height of the below story.
B. Conditions of Section 4.4.2 (Two Stage Analysis Procedure) should be satisfied.
C. Perimeter structural walls are continuous from foundation to base level.

For more information and special cases such as sloping grade, refer to Appendix 1.

Chapter 5

Soil-Structure Interaction & Geotechnical Considerations

5. Soil-Structure Interaction & Geotechnical Considerations

Part 1. Soil-Structure Interaction

5.1 General Provisions

The requirements of this chapter increase the accuracy of structural modeling. These requirements are necessary for structures in Seismic Design Category D_3 (Refer to Section 4.5) and are recommended for the other structures when ($\bar{h}/V_{sT} > 1.0$) where $\bar{h}$ is the effective height of the structure at the center of gravity. For structures where the gravity load is effectively concentrated at a single level, this height is equal to height of that level, otherwise is taken as two-third of the total structural height from base level. V_s is the soil effective shear wave velocity (Section 5.2.3.3) and T is the fundamental period of structure assuming a rigid base in each orthogonal direction.

In soil-interaction analysis, soil flexibility is included in structural calculations. The methods of soil-structure interaction analysis are categorized into Direct Method (Section 5.2) and Substructure Method (Section 5.3). In Direct Method, a considerable part of soil supporting the structure is modeled in detailed gridlines (such as finite element). In Substructure Method, the support flexibility is included by using springs, masses and dampers at the support locations, the dynamic characteristics of which are variable with vibration frequency.

Using Substructure Method is permitted only in Equivalent Lateral Load Procedure (Section 4.8). Direct Method is permitted in all analysis procedures.

5.2 Direct Method

5.2.1 General

In this method, flexibility of foundation can be modeled by springs at the interface of structure and support. Furthermore, damping of total structure and soil system are modified. Damping modification in Equivalent Lateral Load Procedure and Modal Response Spectrum Analysis can be performed by presenting an equivalent damping ratio or by dampers corresponding to foregoing springs in Seismic Response History Procedure. For Equivalent Lateral Load Procedure and Modal Response Spectrum Analysis, design spectrum shall be calculated by equivalent damping ratio and fundamental period of the structure with flexible support shall be used for determination of spectral values.

In this method, effective shear modulus of soil layers is determined from Section 5.2.2. Springs stiffness of shallow and deep foundations in each DOF is presented in Section 5.2.3 & 5.2.4, respectively. Equivalent damping ratio is defined in Section 5.2.5 and damper coefficients are illustrated in Section 5.2.6.

5.2.2 Effective Shear Modulus of Soil Layers

Effective shear modulus of soil layers, G, is determined by consideration of structural weight and large strain of soil layers simultaneously according to this section.

5.2.2.1 Structural Weight Effect

Because of structural weight, shear modulus of soil layers adjacent to foundation subgrade are increased. Shear modulus amplification factor due to structural weight, f_g, is calculated from Equation 5.1.

$$f_g = \left(\frac{\sum_1^5 t_i/V_{s0i}}{\sum_1^5 t_i/V_{sgi}}\right)^2, \sum_1^5 t_i = t_g \qquad 5.1$$

In Equation 5.1, t_g is the effective soil depth, measured from shallow foundation subgrade downward levels and is considered as average of $\sqrt{BL}/2$, $\sqrt[4]{B^3L}/2$ and $\sqrt[4]{BL^3}/2$ in which:

L = footing length (larger dimension of footing)

B = footing width (smaller dimension of footing)

For considering variation of normal stress due to structural weight versus depth in Equation 5.1, soil effective depth shall be divided into five sublayers with thickness of t_i. V_{s0i} and V_{sgi} are shear wave velocity in the free-field condition (without consideration of structure) and increased shear wave velocity (with effect of structural weight) at the center of part i, respectively. V_{sgi} is calculated from Equation 5.2 as follows:

$$V_{sgi} = V_{s0i}\left(\frac{\sigma_i' + \Delta\sigma_i'}{\sigma_i'}\right)^{n/2} \qquad 5.2$$

In Equation 5.2, σ_i' and $\Delta\sigma_i'$ are effective vertical stresses in center of part i due to soil layers weight and structural weight, respectively. n value is equal to 0.5 for sand and 1.0 for clay from footing subgrade to the depth of t_g.

5.2.2.2 Soil Large Strain Effect

Because of seismic large strain of soil layers, shear modulus is reduced. Reduction factor for shear modulus, f_E, is obtained from Table 5.1. In this table, for intermediate values of $S_{DS}/2.5$, linear interpolation may be used. Where reduction factor is less than 0.1, it is recommended to use more precise methods to analyze soil-structure interaction, or utilize other appropriate foundation systems.

Table 5.1 Reduction Factor of Seismic Soil Shear Modulus, f_E

Site Class According to Reference [2]	**$S_{DS}/2.5$(g)**		
	≤0.1	**0.4**	**≥0.8**
I	1.00	0.95	0.90
II	0.95	0.75	0.60
III	0.90	0.50	0.10
IV	0.60	0.05	*

* Site-specific geotechnical investigation shall be performed

5.2.2.3 Effective Soil Shear Modulus and Effective Soil Shear Velocity

Effective soil shear modulus in soil-structure interaction calculations, G, is determined as follows:

$$G = f_g f_E G_0 \qquad 5.3$$

In Equation 5.3 G_0 is the soil shear modulus in free conditions (low strain without considering structure and earthquake), obtained from geotechnical tests or Equation 5.4:

$$G_0 = \rho V_{s0}^2 \qquad 5.4$$

In Equation 5.4, ρ is the soil density in free conditions and V_{s0} is the average of soil shear velocity for free condition in effective soil depth range, determined from Equation 5.5:

$$V_{s0} = \frac{t_g}{\sum(t_i/V_{s0i})} \qquad 5.5$$

In Equation 5.5, t_i is soil layer thickness and V_{s0i} is soil layer shear wave velocity in free conditions. These calculations are conducted for natural soil layers in the effective soil depth range.
Soil layer shear wave velocity, V_s, is calculated from Equation 5.6:

$$V_s = \sqrt{G/\rho} \tag{5.6}$$

5.2.3 Stiffness of Shallow Foundation

For evaluating stiffness of shallow foundation in each DOF, rigidity of foundation related to subgrade soil should be checked from Section 5.2.3.1 & 5.2.3.2. Then, the specification of equivalent springs in each DOF is obtained from Section 5.2.3.3.

5.2.3.1 Rigidity Control for Single Footings or Mats

If Equation 5.7a validates, the single footing or mat can be assumed to be rigid:

$$4k_{sv}\sum_{m=1}^{5}\sum_{n=1}^{5}\frac{sin^2\left[\frac{m\pi}{2}\right]sin^2\left[\frac{n\pi}{2}\right]}{\pi^4 D_f\left[\frac{m^2}{L^2}+\frac{n^2}{B^2}\right]^2+k_{sv}} < 0.03 \tag{5.7a}$$

$$D_f = \frac{E_f\,t^3}{12(1-v_f)^2}\,, k_{sv} = \frac{1.3G}{b(1-v)} \tag{5.7b}$$

where:
b = dimension of foundation perpendicular to direction under consideration (equal to L or B)
E_f & v_f =Young’s modulus of foundation material & Poisson’s ratio of foundation material (concrete)
t = thickness of foundation
v = Poisson’s ratio of subgrade soil
In mat foundation, B and L values should be calculated for the specified part of the foundation beneath each column, by considering column tributary area of the foundation. If all parts of the foundation are rigid, mat foundation shall be considered rigid, otherwise flexible.
For non-rectangular foundations, the equivalent rectangle with the same area, preserving the general proportionality shall be considered. For reinforced concrete columns, or in presence of pedestals or walls, dimension of the mentioned elements can be subtracted from corresponding foundation dimension and the resultant value is used for rigidity control.

5.2.3.2 Rigidity Control for Strip Foundations

Rigidity of strip foundation shall be controlled in each column tributary area. The strip foundation may be considered rigid when:

$$\frac{E_f I_f}{{L_f}^4} > \frac{2}{3}k_{sv}B \tag{5.8}$$

where:
I_f &L_f = moment of inertia (uncracked) and tributary length of the of the foundation section about the axis perpendicular to considered direction
For reinforced concrete columns, in presence of pedestals or walls perpendicular to the direction under consideration, thickness of column, pedestal or wall can be subtracted from foundation length and the resultant value is used for rigidity control.

5.2.3.3 Spring Stiffness Parameters

For rigid foundation, each of the following A & B methods may be used. For flexible foundation, Method C is applicable.

A. Rigid Foundation- Footing Equivalent Springs (excluding foundation):

Footings that are rigid with respect to the supporting soil can be excluded from structural model provided that uncoupled springs for each direction, as shown in Figure 5.1 a represent the foundation stiffness. These springs shall be located at column or wall bases and may be assumed to be independent. In Figure 5.1, the x axis is the longitudinal axis of the foundation. The spring stiffness in direction of j DOF including embedment correction factor at the depth D, $K_{j,emb}$, are determined from Equations 5.9:

$$K_{j,emb} = K_{j,sur} \times \beta_j \qquad 5.9$$

where:
j = x, y, z for translational D.O.Fs
j = xx, yy, zz for rotational D.O.Fs
$K_{j,sur}$ = spring stiffness for shallow foundation
β_j = embedment correction factor of foundation
$K_{j,sur}$ and β_j are determined from Equations 5.10 & 5.11, respectively. In these equations, three former equations are related to translational stiffness and three latter equations include rotational stiffness.

B. Rigid Foundation- Footing Equivalent Springs (including foundation):
For rigid foundations, the body of foundation can be included in structural modeling, so internal forces of foundation can be determined after structural analysis. In this method, for single footings and mat, the foundation may be modeled by shell elements and for strip foundation; beam element can be used to divide the foundation to smaller elements. In each joint of foundation elements, two perpendicular horizontal and one vertical spring can be used. Stiffness of each horizontal spring is calculated by multiplying the horizontal stiffness regarding Equations 5.9 to 5.11 by ratio of the joint tributary area to the foundation total area ($B \times L$). Stiffness of vertical spring in a specified joint is obtained by multiplying the stiffness per unit length in Figure 5.2 by tributary area of joint. If there is considerable uplift at the foundation bed, the corresponding effect shall be considered in the analysis appropriately (for example, by using compression-only springs or compression zone equivalent foundation).

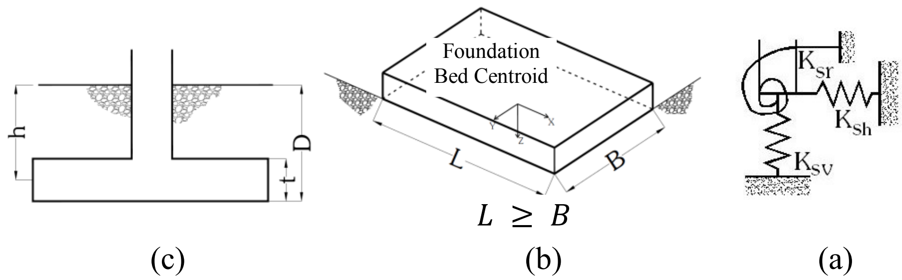


Figure 5.1 Parameters for Equations. 5.10 & 5.11

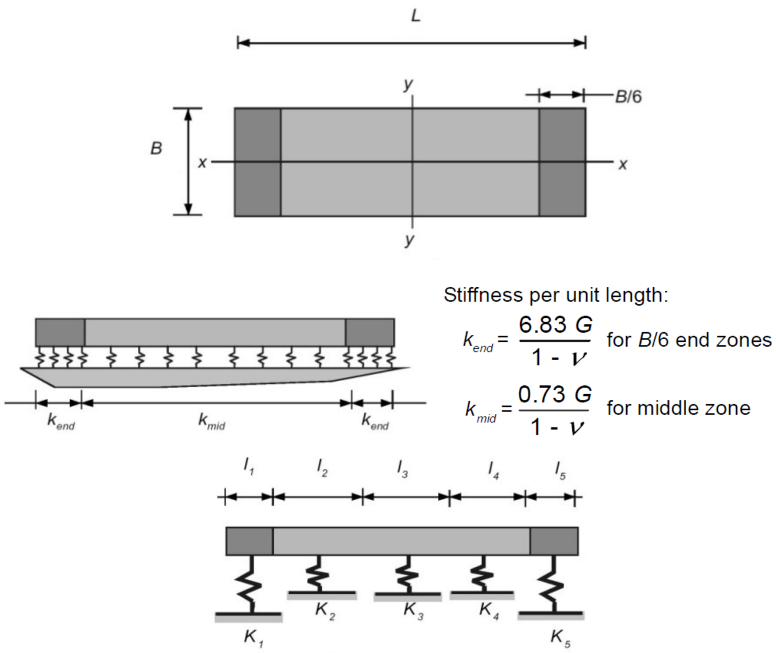
$$K_{x,sur} = \frac{GB}{2-v}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]$$
$$K_{y,sur} = \frac{GB}{2-v}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]$$
$$K_{z,sur} = \frac{GB}{1-v}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$$
$$K_{xx,sur} = \frac{GB^3}{1-v}\left[0.4\frac{L}{B} + 0.1\right] \qquad 5.10$$
$$K_{yy,sur} = \frac{GB^3}{1-v}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$$
$$K_{zz,sur} = GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]$$

$$\beta_x = \left[1 + 0.21\sqrt{\frac{D}{B}}\right]\left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$$
$$\beta_y = \beta_x$$
$$\beta_z = \left[1 + \frac{1}{21}\frac{D}{B}(2 + 2.6\,\frac{B}{L})\right]\left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]$$
$$\beta_{xx} = 1 + 2.5\frac{d}{B}\left[1 + \frac{2d_f}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right] \qquad 5.11$$
$$\beta_{yy} = 1 + 1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$$
$$\beta_{zz} = 1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$$

where:
D = distance from bottom of foundation to ground level
h = vertical distance between centroid of lateral side of foundation having effective contact with soil, and the ground level
d = height of effective sidewall contact that might be considered less than the total height of the foundation. Value of "d" is equal to "t" in Figure 5.1 and is limited to "D" if a retaining wall exists.
Figure 5.2 is shown as a typical view for rigid shallow footings. For rigid single footings and mat foundations and end zone is considered at all sides of foundation body, and in longitudinal direction of footing, the zone width is $L/6$. Furthermore, k_{end} and k_{mid} is determined from the results of represented equations in Figure 5.2 dividing by related zone length (B or L depending on case). Also, $K_{j,emb}$ is calculated by "k" value multiplying by tributary area of each spring and β_z factor. In this case, at the interface of two end zones (longitudinal and transverse); average of calculated stiffnesses should be considered.

C. Non-Rigid (Flexible) Foundation:

Flexible foundation shall be meshed same as previous section (Part A) and two horizontal and a vertical spring shall be assigned to each joint. Stiffness of horizontal springs is calculated similar to rigid foundations. Stiffness of vertical spring is determined by multiplying the tributary area of the joint by subgrade modulus K_{sv} (Equation 5.7b). In sites of Site Class I to III (According to Reference [2]), foundation model may be assumed to be constrained in horizontal direction and assignment of horizontal springs can be neglected.



Each spring stiffness: $K_{j,emb} = l_i k \beta_z$ where β_z is determined from Equation 5.11

Figure 5.2 Vertical Stiffness Modeling for Rigid Shallow Bearing Footings

5.2.4 Stiffness of Deep Foundation

For pile foundations with diameter up to 60 cm, structure can be assumed to be supported on a rigid diaphragm (Cap) and six springs (three translational and three rotational springs). Lateral stiffness of foundation is obtained from passive behavior of soil near to cap and piles regarding geotechnical investigations. For simplicity, lateral stiffness can be derived by multiplying horizontal subgrade modulus by lateral surface area of the smallest prism containing cap and pile, perpendicular to direction under consideration. Vertical stiffness of deep foundation, K_{sv}, is calculated from Equation 5.12. For pile foundations with diameter larger than 60 cm, the pile and the soil contact surface shall be modeled together with Winkler's Spring Model.

$$K_{sv} = \sum_{n=1}^{N_g} \left[\frac{A_p E_p}{L_p} \right] \qquad 5.12$$

where:
A_p = cross-sectional area of a pile
E_p = modulus of elasticity of pile material
L_p = length of pile
N_g = number of piles in a pile group

Rotational spring constant of pile foundations about each of horizontal axes, K_{sr}, shall be calculated using Equation 5.13.

$$K_{sr} = \sum_{n=1}^{N_g} \left[\frac{A_p E_p}{L_p} \right]_n S_n^2 \qquad 5.13$$

where:
S_n= distance between n^{th} pile and axis of rotation (passing through foundation plan centroid)
Rotational spring constant of pile foundations about vertical axis, K_{st}, shall be calculated using Equation 5.14.

$$K_{st} = \frac{1}{6} K_{sh} H_g \left({B_g}^3 + {L_g}^3 \right) \qquad 5.14$$

where:
K_{sh} = horizontal subgrade modulus
H_g = specified length of pile group including pile cap in contact with stiff soil
L_g & B_g = dimensions of pile group
If the skewed or non-prismatic piles are used, reliable equations shall be used for evaluation of spring constants.

5.2.5 Equivalent Damping Ratio

Equivalent damping ratio, $\bar{\xi}$, is calculated from Equation 5.15.

$$\bar{\xi} = \xi_0 + \frac{0.05}{(\bar{T}/T)^3} \quad , \; 0.05 \leq \bar{\xi} \leq 0.2 \qquad 5.15$$

where:
$\bar{T}$= Fundamental mode period for structure supported on springs defined in Section 5.2.3 determined from structural analysis.
ξ_0 = damping ratio of foundation calculated from Figure 5.3. In this figure, $\bar{h}$ is defined in Section 5.1 and r factor in Figure 5.3 is equivalent radius of foundation which is determined from Equation 5.16. For intermediate values of $\bar{h}/a$, r value may be calculated by linear interpolation.

$$r = \sqrt{\frac{A_f}{\pi}} \quad : \; \frac{\bar{h}}{a} \leq 0.5 \qquad 5.16a$$

$$r = \sqrt[4]{\frac{4I_{Pf}}{\pi}} \quad : \; \frac{\bar{h}}{a} \geq 1.0 \qquad 5.16b$$

where:
A_f = area of effective foundation assumed as an area circumambient of all footings. For mat foundation, effective area is equal to foundation area.
I_{Pf} = moment of inertia of effective foundation plan about foundation central axis perpendicular to direction under consideration
a = average dimension of effective foundation in the direction under consideration
Note 1. If the subgrade is soft soil with almost constant thickness, supported by a thick stiff soil, in Equation 5.15, ξ_0 shall be replaced by $\acute{\xi}_0$, calculated from Equation 5.17.

$$\acute{\xi}_0 = \left(\frac{4h_s}{\bar{V}_s \bar{T}} \right)^2 \xi_0 \qquad \frac{4h_s}{\bar{V}_s \bar{T}} < 1 \qquad 5.17a$$

$$\acute{\xi}_0 = \xi_0 \qquad \frac{4h_s}{\bar{V}_s \bar{T}} \geq 1 \qquad 5.17b$$

where:
h_s = thickness of soft soil layer

$\overline{V_s}$ = average shear wave velocity in soft soil layer at large strains, calculated from Equation 5.6 assuming $f_g = 1$ in Equation 5.3.

Note 2. Structural base shear, $\overline{V_u}$, shall be determined by spectral acceleration, $\bar{S_a}$. corresponding to $\bar{T}$ and $\bar{\xi}$. For this purpose, spectral acceleration, S_a, corresponding to $\bar{T}$, 5 percent damping ratio and site class, is determined and a reduction factor equal to 0.83 or 0.67 for $\bar{\xi}$ equal to 0.10 or 0.20, respectively; or intermediate factors with linear interpolation are applied to obtain $\bar{S_a}$. However, $\bar{S_a}$ shall not be taken smaller than 70 percent of spectral acceleration of the structure with fixed supports and 5 percent damping ratio. Lateral load distribution of stories, in Equivalent Lateral Load Procedure is performed according to Equation 4.7 with substituting V_u by $\overline{V_u}$.

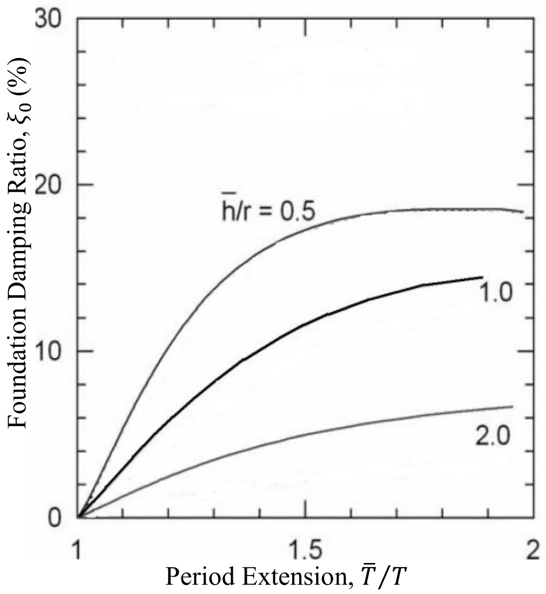


Figure 5.3 Foundation Damping Ratio

5.2.6 Damping Coefficients

Damping coefficient of lumped dampers at the centroid of footing in DOF j, C_j, is calculated from Equation 5.18:

$$C_j = \frac{K_{j,emb}\, \xi_j^t\, \bar{T}}{\pi} \quad 5.18$$

where:

j = x,y,z for translational D.O.Fs. For rotational D.O.Fs damper definition is not necessary.

$K_{j,emb}$ = corresponding spring stiffness according to Equation 5.9

$\bar{T}$= fundamental mode period for structure supported on springs defined in Section 5.2.3 determined from structural analysis excluding dampers

ξ_j^t= damping ratio calculated from Equation 5.19:

$$\xi_j^t = \xi_s + \xi_j \quad 5.19$$

where:

ξ_s = soil large strain damping ratio, which could be taken as 0.05

ξ_j = soil radiation damping ratio from Equation 5.20

$$\xi_x = \left[\frac{L}{B} + \left(\frac{2D}{B}\right)\left(\psi + \frac{L}{B}\right)\right]\left(\frac{GB}{K_{x,emb}}\right)\left(\frac{a_0}{a_x}\right)$$
$$\xi_y = \left[\frac{L}{B} + \left(\frac{2D}{B}\right)\left(1 + \psi\frac{L}{B}\right)\right]\left(\frac{GB}{K_{y,emb}}\right)\left(\frac{a_0}{a_y}\right) \quad 5.20$$
$$\xi_z = \left[\psi\left(\frac{L}{B}\right) + \left(\frac{2D}{B}\right)\left(1 + \frac{L}{B}\right)\right]\left(\frac{GB}{K_{z,emb}}\right)\left(\frac{a_0}{a_z}\right)$$

In Equation 5.20, values of Ψ and a_0 are determined from Equation 5.21 and 5.22, respectively. Dynamic correction factor, a_j, is equal to 1.0 for $j = x, y$ and shall be determined from Equation 5.23 for $j = z$.

$$\psi = \sqrt{2(1-v)/(1-2v)} \leq 2.5 \quad 5.21$$
$$a_0 = \frac{\pi B}{TV_s} \quad 5.22$$
$$a_z = 1.0 - \frac{\left(0.4 + \frac{0.2}{L/B}\right)a_0^2}{\frac{10}{1+3(L/B-1)} + a_0^2} \quad 5.23$$

5.3 Substructure Method

5.3.1 General

In this method, support of structure is assumed to be rigid, but equivalent fundamental period and equivalent damping ratio shall be used for structural lateral loads and base shear calculations. This method is allowed just for Equivalent Lateral Load Procedure (Section 5.3.2).

5.3.2 Equivalent Lateral Load Procedure

Provisions of this method is the same as Chapter 4, except that response factor or spectral acceleration in respect to period and damping ratio of equivalent structure shall be calculated by Section 5.3.2.1 and Section 5.3.2.2.

5.3.2.1 Equivalent Period

The equivalent period, $\bar{T}$, is determined from Equation 5.24:

$$\bar{T} = T\sqrt{1 + \frac{\bar{K}}{K_h}\left(1 + \frac{K_h\bar{h}^2}{K_\theta}\right)} \quad 5.24$$

where:
T = fundamental period of structure with a rigid base assumption
K_h, K_θ= horizontal and rotational stiffness of foundation according to Equations 5.9 to 5.14, respectively.
$\bar{h}$ = effective height of structure, equal to h if the major part of the mass of structure is lumped at height h, and equal to $0.7h$ in other cases.
$\bar{K}$ = lateral stiffness of structure with a rigid base, determined from Equation 5.25.

$$\bar{K} = \frac{\bar{W}}{g}\left(\frac{4\pi^2}{T^2}\right) \quad 5.25$$

$\bar{W}$= effective weight of structure, equal to W if the major mass of structure is lumped at a point, and equal to $0.7W$ in other cases. W is the seismic weight of structure according to Section 4.8.2.
In Non-mat foundations composed of continuous perpendicular strips, such as single footings connected with tie beams or strip foundations, the smallest area containing all single footings is assumed as an effective foundation. This effective foundation may be replaced with a rectangular foundation with the same aspect ratio, and then K_h and K_θ are calculated. If the foundation is not continuous, a rational method shall be used to determine equivalent stiffness, or generally, Direct Method (Section 5.2) can be applied.

5.3.2.2 Equivalent Damping Ratio

Equivalent damping ratio, $\bar{\xi}$, is calculated from Section 5.2.5 except that the period used in Equation 5.15 shall be determined from Equation 5.24.

5.3.2.3 Base Shear and Vertical Distribution of Shear Forces

Structure base shear including soil-structure interaction effects, $\bar{V_u}$ is calculated from Equation 4.1 based on spectral acceleration, $\bar{S}_a$ corresponding to $\bar{T}$ and $\bar{\xi}$. For practical applications, spectral acceleration, S_a can be determined using T, 5% damping ratio and Site Class according to Chapter 4, then reduced by factors of 0.83 and 0.67 for $\bar{\xi}$ equal to 0.10 or 0.20, respectively, or intermediate factors with linear interpolation. However, $\bar{S}_a$ value shall not be less than $0.7S_a$ for the same structure with rigid supports and damping ratio equal to 5%. Distribution of Lateral forces at the stories is according to Equation 4.7 with substitution of V_u by $\bar{V_u}$.

5.3.2.4 Lateral Displacement at Each Level

In Substructure Method, inelastic lateral displacement at level x, $\bar{\delta}_x$, is computed from Equation 5.26.

$$\bar{\delta}_x = \frac{\bar{V_u}}{V_u}\left(\delta_x + h_x \frac{M}{K_\theta}\right) \tag{5.26}$$

where:
V_u= structural base shear without considering interaction effects according to Equation 4.1
δ_x= inelastic lateral displacement at level x without considering interaction effects
h_x= height of level x with respect to level of foundation base center
M = overturning moment at level of foundation base without including interaction effects
δ and M are obtained from Equation 4.7 with applying lateral forces. $P - \Delta$ effects shall be considered according to Equation 4.15.

Part 2. Geotechnical Considerations

5.4 Scope

In this section, general requirements for site investigations and mitigation of geotechnical seismic hazards are presented. In Section 5.5, soil data gathering and site hazards are introduced and section 5-6 discusses about seismic soil pressures on retaining walls.

5.5 Site Investigation

Site investigation includes gathering of bearing soil data according to Section 5.5.1 and seismic hazards according to Section 5.5.2.

5.5.1 Required Information for the Bearing Soil

Following data should be provided by site investigation and geotechnical tests:
Soil type and structure, relative compactness and soil layer configuration should be determined to the depth where normal stress is less than 10 percent of structural weight divided by the area of total foundation plan. For structure with friction pile system, minimum depth of investigation according to

this approach shall be specified by assuming the structural weight at a depth equal to two-thirds of pile length. For a bearing pile system, minimum depth of investigation is assumed to be equal to pile length plus five times the pile diameter. Details of these data for each layer should include weight per unit volume, friction angle, soil shear strength and undrained shear strength for clays, compressibility, low strain shear modulus and Poisson's ratio. Also, effect of stress change on shear strength, friction angle and soil shear modulus should be evaluated. Furthermore, water table level and seasonal changes shall be specified.

Load-Deflection relation for shallow and deep foundations in each DOF are determined from Section 5.2.3 and 5.2.4, respectively.

5.5.2 Site Seismic Hazards

Site hazards due to instabilities during ground motions include shallow faulting, liquefaction, differential settlements, compactness, landslide and flooding where described in Sections 5.5.2.1 to 5.5.2.5. These hazards should be assessed in relation to considered hazard level.

If hazards are determined from authoritative maps, these studies should be based on local evaluations, soil strength and stiffness specifications.

5.5.2.1 Shallow Faulting

Construction at the vicinity of active faults is not permitted nowise. For pipelines crossing active faults, refer to Chapter 13.

5.5.2.2 Liquefaction

Liquefaction is a process in which, in saturated and loose soils, shear stiffness and strength drop due to increasing pore water pressure in earthquake or any other rapid loading pattern.

For assessment of liquefaction potential, liquefiable soil of foundation bed and site seismicity should be evaluated. This investigation includes soil data (soil type, plasticity specifications, grading, density, solidarity specifications, water table depth etc.) and seismic data (include PGA in ground surface and probable earthquake magnitude).

If site soil has any of the following conditions, it is not categorized into liquefiable soil:

1. Site is classified into Class I in Reference [2]
2. Soil composed of stiff and very stiff clays or silts
3. Fine grade soil without high sensitivity
4. Non-cohesive soil exists and value of minimum normalized strength in Standard Penetration Test named as $N_{1(60)}$ is greater than 30 at depths below water table.
5. Soil has a minimum 20% clay and liquid limit (LL) is greater than 35.
6. Maximum upper level of water table is at least 10 meters less than the deepest foundation or 15 meters lower than ground level, which one is closer to surface, provided that if the ground is slopped or wall mount exists, slope or mount is not continuous to the below of underground water table.

If liquefiable soil exists, following solutions shall be considered relating to the priority:

1. Site change to another site without liquefaction potential
2. Soil remediation for mitigation of liquefaction potential

5.5.2.3 Soil Settlement

Site subsurface soil investigation should include required data for assessing differential settlements of foundation bed in a seismic event. If differential settlement hazard is proved to exist, site soil should be rehabilitated with authoritative methods.

5.5.2.4 Landslide

Site study for evaluating landslide shall include slide susceptible soils that result in differential motions of foundation. In following cases, landslide potential shall be evaluated excluding cases where liquefaction potential exists:

1. Slopped surfaces with grading greater than 18 in degrees (3 horizontal to 1 vertical ratio)

2. Past background of instability (rotational or transitional landslide or rock fall)

If soil is not suspected to liquefaction according to Section 5.5.2.2 and shear strength loss while soil deformation is not expectable, pseudo static procedure may be used for slope stability analysis. Otherwise, dynamic procedure should be used for slope stability assessment. Seismic coefficient for pseudo static procedure should be determined based on considered hazard level. Landslide static safety factor for the site should be greater than one. Landslide and rock fall evaluation should be performed for upstream and downstream slopes.

5.5.2.5 Flood and Inundation

For structures with Function & Risk Category III & IV, following cases, which can produce flooding and inundation shall be assessed:

1. Upstream dam which is probable to collapse due to earthquake or fault rupture
2. Pipelines, water channels and water storage tanks at upstream subject to damages due to fault rupture, landslide or severe seismic events
3. Coastal areas subject to tsunami or areas in vicinity of bays and lakes subject to seiche
4. Lowland areas with high level of water table subject to seismic flooding and inundation

In addition to seismic flooding and inundation effects caused by earthquake, scouring of building foundation due to a rapid current around it should be investigated by authoritative approaches.

If any of mentioned hazards exists, appropriate measures for complementary studies or mitigations should be considered by authorities having jurisdiction.

5.6 Seismic Earth Pressures on Retaining Walls

For designing retaining walls, an additional seismic soil pressure shall be considered. When there is no site-specific study for geotechnical issues, additional seismic lateral pressure on retaining wall for non-saturated soils with horizontal top-level upper than water table, may be estimated by Equation 5.27:

$$\Delta_p = 0.4 k_h \gamma_{\mathrm{t}} H_{rw} \qquad 5.27$$

where:

Δ_p = additional seismic lateral pressure distributed uniformly on retaining wall

k_h = horizontal seismic coefficient of soil which may be assumed equal to be $S_{DS}/2.5$

γ_{t} = unit weight of soil

H_{rw} = height of retaining wall

S_{DS} = spectral acceleration parameter according to Chapter 3

The retaining wall shall be designed for total lateral pressure (H in Chapter 2), including additional seismic lateral pressure and non-factored static lateral pressure of soil.

Chapter 6

Essential Building

6. Essential Building

6.1 General Provisions

6.1.1 Scope

In this chapter, complementary requirements for design of essential building (Function and Risk Category IV from Table 4.3) for seismic loads, in addition to requirements from Chapters 2 to 5, are presented. This structure shall be designed for design earthquake (Seismic Hazard Level II) and service earthquake (Seismic Hazard Level I), as described in Chapter 3. This chapter does not include non-building structure requirements. Where base isolation or damper systems are used, design shall conform to requirements of Chapters 9 and 10.

6.1.2 Types of Essential Buildings

Definition of essential building in this code is the same as building with Function and Risk Category IV in Table 4.3. In this category, post-earthquake functionality is necessary and any failure in operation could pose an increase in casualties and detriments.
Hospitals and clinics, fire stations, water stations and installations, power plant buildings and installations, telecommunication centers, disciplinary buildings, first aid stations, control rooms, and any building containing essential facilities, inconstant toxic or flammable chemicals are classified as essential building. Any other building with client approval could be classified as essential building either.

6.1.3 Design Earthquake

Design Earthquake is an event with an exceeding probability of 10% in 50 years (475 years return period), which is the same as Seismic Hazard Level II, Section 3.4.2.

6.1.4 Service Earthquake

Service Earthquake is the same as Seismic Hazard Level I, Section 3.4.1.

6.1.5 Site-Specific Ground Motion Hazard Analysis

Site-specific ground motion analysis is necessary for essential building and shall be done regarding Chapter 3 of this code. It is not permitted to construct essential buildings with Seismic Design Category D_3 on sites with faulting potential by an active fault based on Site-Specific Ground Motion Hazard Analysis.

6.1.6 Importance Factor

For essential building, importance factor, I, is set to 1.5.

6.1.7 Geotechnical studies

For essential building, geotechnical studies are necessary according to provision of Chapter 5. For building in Seismic Design Category D_3, considering soil-structure interaction, as mentioned in Chapter 5, is recommended.

6.1.8 Classification of Building by Configuration

Determination of vertical and horizontal irregularities shall be done using from Tables 4.1 and 4.2, respectively. For an irregular building, a three dimensional analysis shall be performed. Construction of an essential building more than one story from base level with extreme torsional irregularity (section b of Table 4.2), with extreme soft story (section b of Table 4.1) or with extreme weak story irregularity (section g of Table 4.1) is not permitted.

6.1.9 Structural Systems

In essential buildings, structural systems presented in Table 4.4 shall be used provided that the term "N.P." is not mentioned on rows for all Seismic Design Categories.

6.2 Analysis and Design for Design Earthquake

6.2.1 Analysis Methods

To analyze an essential building, Equivalent Lateral Load Procedure (Section 4.8) considering limit of Section 4.8.1, Modal Response Spectrum Analysis (Section 4.9) or Seismic Response History Procedure (Section 4.10) are permitted to use. To control performance of building, one of nonlinear analysis methods including static (appendix II, Reference [2]) or Seismic Response History Procedure (Section 4.10.3) may be used.

6.3 Service Earthquake

6.3.1 Strength Control

Essential building shall be controlled to have linear behavior during service earthquake (Seismic Hazard Level I, Section 3.4.1), for load combination combined of service earthquake loads with other service loads, without load factors. To achieve so, in steel structures, stress in members shall not exceed yielding stress. In concrete structures, without any strength reduction factor, member internal forces shall not exceed nominal strengths. $P-\Delta$ effect in increasing internal forces shall be considered regarding Section 4.15.
Base shear in service earthquake, V_{ser}, in Equivalent Lateral Load Procedure is calculated from Equation 6.1.

$$V_{ser} = S_{aser} I W \tag{6.1}$$

where:
S_{aser}= service earthquake spectral acceleration (g), from Site-Specific Ground Motion Hazard Analysis (Chapter 3)
W= effective seismic weight of the structure according to Section 4.8.2

6.3.2 Drift Control

Drifts shall be checked by requirements of Reference [2] for service earthquake.

6.4 Non-Structural Components and Elements

Stability of non-structural components and elements shall be controlled, regarding requirements of Chapter 8.

Chapter 7

Non-Building Structures

7. Non-Building Structures

7.1 General Provisions

7.1.1 Definitions

Non-building structures include all stand-alone structures, which carry gravity loads and may be used for carrying seismic effects.

Structures similar to bridges, electricity transmission towers, hydraulic structures, buried pipelines and their attachments and nuclear reactors are not discussed here because of special cautions about their response properties and environmental effects.

Non-building structures are divided into two groups: in the first group, industrial component is supported by other structures, such as pipelines, finfans or furnaces supported on frames. In the second group, industrial component is self-supported on ground, such as pumps, vessels and heat exchangers supported through short pedestals, steel skirts or foundations. In this chapter, seismic analysis and design requirements for both types are presented in Section 7.2 and 7.3, respectively.

7.1.2 Seismic Design

Seismic design of non-building structures for resisting ground motion should include enough stiffness, strength and ductility provisions according to the following considerations:

A. Required strength and other design criteria shall satisfy the provisions of this chapter and special considerations of industrial component manufacturer.

B. In cases where required strength or other design criteria are not presented, or the non-building structure is not mentioned, these criteria shall be determined from manufacturer's special provisions. If the design criteria of special provisions represented by manufacturer are based on allowable stress or allowable strength design, seismic design forces shall be combined with other loads presented in Section 2.2.1 and Chapter 2 provisions should be observed. Allowable values in special provisions shall be considered. However, seismic detailing shall be observed according to special provisions with manufacturer responsibility.

7.1.3 Structural Analysis and Procedure Selection

Non-building structure should be analyzed and designed with one of the following procedures:

Equivalent Lateral Load Procedure according to Section 4.8, Modal Response Spectrum Analysis according to Section 4.9, Elastic Seismic Response History Procedure according to Section 4.10.2 and Inelastic Seismic Response History Procedure specified in Section 4.10.3.

7.2 Non-Building Structures Supported by Other Structures

These types of non-building structures include supporting structure, industrial components and their connections and attachments. In this type, industrial component(s) such as finfans, horizontal or vertical vessels, heat exchangers or pipes are supported by other structures such as MRFs, braced frames or other non-building structure, and industrial component is not a member of the main seismic force-resisting system.

In these structures, for designing industrial component and connections, regarding the component-to-structure weight ratio, one of the following conditions will happen:

A. Weight of industrial component is less than 25% of the total effective seismic weight of the non-building structure. In this condition, for designing the industrial component and its attachments, seismic design force, F_p may be calculated using requirements of Chapter 8 or provisions of Section 7.2.3.

   In a non-building structure with multiple industrial components including vessels, tanks and exchangers supported on the structure, if structural weight of each industrial component is less than 25% of total non-building structure, provided that total weight of industrial components is not less than the total weight of non-building structure, requirements of Section 7.2.3 are recommended for seismic design of those components and attachments.

B. Weight of industrial component is equal or greater than 25% of the total effective seismic weight of the non-building structure. In this condition, industrial component effects on dynamic response of non-building structure cannot be neglected and interaction effects of industrial components and supporting structure shall be considered in structural analysis. Therefore, seismic design forces for industrial component and its attachments are calculated from Section 7.2.3. To design the supporting structure for both "A" and "B" conditions, requirements of Sections 7.2.1 and 7.2.2 shall be used.

Flowcharts for seismic design of non-building structures (According to Chapter 7 provisions), and for seismic design of non-structural components are presented in Appendix 3 of this code according to their mass and stiffness properties.

7.2.1 Seismic Design of the Supporting Structure

Non-building structures having specific seismic design criteria presented in specifications of vendors and manufacturers, should be analyzed and designed according to specifications considering requirements of this chapter. Otherwise, provisions of this section shall be applied.

In this section, general requirements for selection of the analysis method for supporting structure are presented. For non-building structures where weight of industrial component or components is equal to or greater than 25% of the total weight of the non-building structure, interaction effects between component and structure shall be considered.

7.2.1.1 Analysis Method

Both elastic and inelastic procedures are permitted to be used for seismic analysis of the supporting structure. In this section, requirements for elastic analysis procedures are specified. For inelastic analysis procedures, provisions of Appendix of Reference [2] or other valid references may be used. Elastic analysis procedures can be performed in both static (Equivalent Lateral Load Procedure) and dynamic methods (Modal Response Spectrum Analysis or Seismic Response History Procedure). For irregular structures (According to Tables 4.1 and 4.2), or when interface components exist, dynamic methods shall be used.

In cases where each of adjacent and connected structures responds as a single DOF system, if product of sum of link elements stiffness times sum of the softness values for adjacent structures is less than 0.2, interaction effects between these structures may be neglected.

7.2.1.2 Equivalent Lateral Load Procedure

If supporting structure satisfies the conditions of Section 4.8.1, Equivalent Lateral Load Procedure could be used.

In this procedure, seismic effects on structures are substituted by equivalent lateral forces. These forces depend on ground motion acceleration, weight, dynamic properties, ductility and importance of the structure. Furthermore, application point of this force or vertical distribution along height of structure depends on weight of industrial component in comparison with total weight of non-building structure.

Effective base shear in each direction in this procedure shall be calculated using Equation 7.1 and distributed as mentioned in this section.

$$V_u = C_u W \tag{7.1}$$

where:

C_u = seismic response coefficient from Equation 7.2

W = effective seismic weight of the non-building structure

This weight includes dead weight of the supporting structure and supported components, plus operational weight of the contents of the components such as tanks, vessels, pipes, etc. In addition, where the snow or ice load is more than $0.25W$, it shall be included in W.

$$C_u = \frac{S_a}{R_u/I} \tag{7.2}$$

where:
S_a= mapped spectral response acceleration parameter (g), determined from hazard analysis corresponding to Seismic Hazard Level II, according to Chapter 3
R_u= response modification factor for supporting structure from Section 7.2.2.1
I= importance factor for supporting structure from Section 7.2.2.2
Where for determining seismic coefficient, C_u, response modification factor, R_u, is obtained from Table 7.1 or Table 4.4. This coefficient shall not be taken less than value obtained from Equation 7.3.

$$C_{u1} = 0.044\, S_{DS} I \geq 0.01 \tag{7.3}$$

In addition, if the structure is constructed on a site with $S_1 \geq 0.6$, seismic coefficient shall not be less than value obtained from Equation 7.4, either:

$$C_{u2} = \frac{0.5 S_1}{R_u/I} \tag{7.4}$$

For determining seismic coefficient, if response modification factor, R_u, is obtained from Table 7.2, the minimum seismic coefficient, C_u, shall be taken from Equation 7.5a, and in addition if $S_1 \geq 0.6$ it shall not be less than the value calculated from Equation 7.5b, excluding tanks, vessels and chimneys placed on a supporting structure, for which Equations 7.3 and 7.4 shall be used.

$$C_{u1} = 0.044 S_{DS} I \geq 0.03 \tag{7.5a}$$

$$C_{u2} = \frac{0.8 S_1}{R_u/I} \tag{7.5b}$$

where fundamental period of the non-structural building is less than 0.06 sec. design base shear for the non-structural building shall be calculated from Equation 7.6.

$$V_u = 0.3\, S_{DS} I W \tag{7.6}$$

where S_1 and S_{DS} are spectral acceleration parameters according to Chapter 3.

7.2.1.3 Elastic Dynamic Procedure

It is allowed to analyze every non-building structure using elastic dynamic procedure. This procedure can be performed in various methods, such as Modal Response Spectrum Analysis or Seismic Response History Procedure according to Chapter 4 provisions and seismic design parameters of Section 7.2.2.

7.2.2 Seismic Design Parameters

In seismic design of a non-building structure, following provisions shall be considered.

7.2.2.1 Structural Response Modification Factor, R_u

For non-building structures in which the weight of each industrial component is less than 25% of the total weight of the structure, response modification factor shall be derived regarding the supporting structure, from Table 4.4 or 7.1.
For non-building structures in which the weight of each industrial component is equal to or more than 25% of the total weight of the structure, response modification factor shall be calculated regarding period of the industrial component, T_p (Section 7.2.2.3) as follows:

- Where the fundamental period, T_p, of the industrial component and its connections, is equal to or greater than 0.06 sec., the industrial component and supporting structure shall be modeled together in a combined model and the R_u value of the combined system taken as the lesser value of the supporting structure, Table 4.4 or 7.1, or the industrial component, Table 7.2.

- Where the fundamental period, T_p, of the industrial component and its connections is less than 0.06 sec., the R_u value shall be taken regarding the supporting structure from Table 4.4 or 7.1.

To determine height limitations in Tables 7.1 or 7.2, the elevation of the highest point of the supporting structure shall be considered.

Table 7.1 Seismic Coefficients for Supporting Structure

Seismic Force-Resisting System	R_u	Ω_0	C_d	**Structural Height Limits (m)**		
				D_1	D_2	D_3
1. Steel storage racks	4	2	3.5	NL	NL	NL
2. Building frame systems						
Steel special concentrically braced frames	6	2	5	50	50	30
Steel ordinary concentrically braced frames	3.25	2	3.25	10[a]	10[a]	NP[a]
With permitted height increase	2.5	2	2.5	50	50	30
With unlimited height	1.5	1	1.5	NL	NL	NL
3. Moment resisting frame systems						
Steel special moment frames	8	3	5.5	NL	NL	NL
Special reinforced concrete moment frames	8	3	5.5	NL	NL	NL
Steel intermediate moment frames	4.5	3	4	10[b,c]	NP[b,c]	NP[b,c]
With permitted height increase	2.5	2	2.5	50	50	30
With unlimited height	1.5	1	1.5	NL	NL	NL
Intermediate reinforced concrete moment frames	5	3	4.5	NP	NP	NP
With permitted height increase	3	2	2.5	15	15	15
With unlimited height	0.8	1	1	NL	NL	NL
Steel ordinary moment frames	3.5	3	3	NP[b,c]	NP[b,c]	NP[b,c]
With permitted height increase	2.5	2	2.5	30	30	NP[b,c]
With unlimited height	1	1	1	NL	NL	NL
Concrete ordinary moment frames	3	3	2.5	NP	NP	NP
With permitted height increase	0.8	1	1	15	15	15

a. When used to support pipelines, a height up to 20 m is permitted.
b. Steel ordinary moment frames and intermediate moment frames are permitted in pipe racks up to a height of 20m where the moment joints of field connections are constructed of bolted end plates.
c. Steel ordinary and intermediate moment frames other than note b are permitted in pipe racks up to a height of 10 m.

Table 7.2 Seismic Coefficients for Industrial Component

Seismic Force-Resisting System	R_u	Ω_0	C_d	Structural Height Limits (m)		
				D_1	D_2	D_3
1. Elevated tanks, vessels, bins or hoppers on:						
Symmetrically braced legs (not similar to buildings)	3	2	2.5	50	30	30
Unbraced legs or asymmetrically braced legs (not similar to buildings)	2	2	2.5	30	18	18
2. Horizontal, saddle supported welded steel vessels	3	2	2.5	NL	NL	NL
3. Tanks or Vessels on supporting structure	Values are determined from Table 7.1 or Table 4.4 depending on supporting structure system					
4. Flat-bottom ground-supported tanks						
Steel or fiber-reinforced polymer						
Mechanically anchored	3	2	2.5	NL	NL	NL
Self-anchored	2.5	2	2	NL	NL	NL
Reinforced or precast concrete						
Reinforced non-sliding base	2	2	2	NL	NL	NL
Anchored flexible base	3.25	2	2	NL	NL	NL
Unanchored and unconstrained flexible base	1.5	1.5	1.5	NL	NL	NL
All other	1.5	1.5	1.5	NL	NL	NL
5. Cast-in-place concrete silos having walls continuous to the foundation	3	1.75	3	NL	NL	NL
6. Reinforced masonry structures not similar to buildings detailed as intermediate reinforced masonry shear walls	3	2	2.5	15	15	15
7. Reinforced masonry structures not similar to buildings detailed as ordinary reinforced masonry shear walls	2	2.5	1.75	NP	NP	NP
8. Reinforced masonry structures not similar to building structures	1.25	2	1.5	NP	NP	NP
9. Trussed towers (freestanding or guyed), guyed stacks, and chimneys	3	2	2.5	NL	NL	NL
10. Concrete chimneys and stacks	2	1.5	2	NL	NL	NL
11. All steel and reinforced concrete distributed mass cantilever structures including stacks, chimneys, silos, skirt-supported vertical vessels and single pedestal or skirt supported						
Welded steel	2	2	2	NL	NL	NL
Welded steel with special detailing	3	2	2	NL	NL	NL
Prestressed or reinforced concrete	2	2	2	NL	NL	NL
Prestressed or reinforced concrete with special detailing	3	2	2	NL	NL	NL

Table 7.2 Seismic Coefficients for Industrial Component (continued)

Seismic Force-Resisting System	R_u	Ω_0	C_d	Structural Height Limits (m)		
				D_1	D_2	D_3
12. Cooling towers						
Reinforced concrete or steel	3.5	1.75	3	NL	NL	NL
Wood frames	3.5	3	3	15	15	NL
13. Telecommunication towers						
Steel truss	3	1.5	3	NL	NL	NL
Steel pole	1.5	1.5	1.5	NL	NL	NL
Concrete pole	1.5	1.5	1.5	NL	NL	NL
Wooden pole	1.5	1.5	1.5	NL	NL	NL
Steel frame	3	1.5	1.5	NL	NL	NL
Concrete frame	3	1.5	1.5	NL	NL	NL
Wooden frame	1.5	1.5	1.5	NL	NL	NL
14. Inverted pendulum type structures (except elevated tanks, vessels, bins, and hoppers)	2	2	2	NL	NL	NL
15. Signs and billboards	3	1.75	3	NL	NL	NL
16. All other self-supporting structures, tanks, or vessels not covered in the above or by reference standards that are similar to buildings	1.25	2	2.5	15	15	15

7.2.2.2 Importance Factor

The importance factor for non-building structures, I, considering functionality, shall be selected from Table 4.3, unless manufacturer represents larger values.

7.2.2.3 Fundamental Period

A. Industrial component:
Fundamental period of industrial component, T_p, is permitted to be computed from Equation 7.7, if the component and its connections can be modeled as a single-degree of freedom mass-spring system with a reasonable accuracy.

$$T_p = 2\pi \sqrt{\frac{W_p}{K_p g}} \tag{7.7}$$

where:
W_p = industrial component operating weight
K_p = stiffness of the industrial component and its connections to the structure, equal to a load applied to the center of mass of the component causing a unit displacement at that point from component support.
g = ground gravity acceleration
Fundamental period of industrial components is also permitted to be determined from experimental or analytical procedures, as well as equations mentioned in Appendix 2.
B. Non-building structure:
Experimental equations from Chapter 4 shall not be used to determine the fundamental period of non-building structures. This value shall be computed considering dynamic characteristics and deformation specifications of seismic force-resisting systems in an appropriate analytical procedure. In addition, Equation 7.8 is permitted to be used for different types of non-building structures.

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i {\delta_i}^2}{g \sum_{i=1}^{n} F_i \, \delta_i}} \qquad 7.8$$

where:
F_i = lateral force applied to the center of mass of element i
δ_i = lateral elastic displacement of the center of mass of element i, caused by F_i.
w_i = seismic effective weight of element i
n = number of lumped masses in model
In addition, simplified equations of Appendix 2 are permitted to be used to compute fundamental period of common non-building structures.

7.2.2.4 Lateral Load Distribution

Where weight of the industrial component is less than 25% of the total weight of the non-building structure, it can be applied at the level of connection to the non-building structure. Where weight of the industrial component is equal to or more than 25% of the total weight of the non-building structure, vertical load distribution shall be considered along the height of the industrial component, as well as non-building structure.
Lateral seismic force at level x, F_x, shall be calculated from Equation 7.9:

$$F_x = C_{vx} V_u \qquad 7.9$$

where:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad 7.10$$

where:
C_{vx} = vertical distribution factor
V_u = base shear from Equations 7.1 or 7.6
w_x = seismic effective weight of level x, including light industrial components situated at that level
h_x = height of level x, calculated from base level
h_i = height of level i, calculated from base level
k = distribution exponent calculated as follows:

- For structures having a fundamental period of 0.5 sec. or less: $k = 1$
- For structures having a fundamental period of 2.5 sec. or more: $k = 2$
- For structures having a fundamental period between 0.5 and 2.5 sec., k shall be determined by linear interpolation.

7.2.2.5 Drift Limitations

Controlling story drift limitations as described in Chapter 4 for building structures, is not mandatory for non-building structures, provided that appropriate analysis shows that exceeding limitations mentioned in Section 4.14.2, does not jeopardize stability of the structure and connecting elements such as pipelines and manholes.
Note:
Drift ratio limitations shall be checked for piperacks, where drift values from Equation 4.23 shall be used to control impact and design seismic joints between structures, in accordance with Section 4.14.2 and Section 4.14.3.
To control drift limitations in non-building structures with lightweight industrial components, refer to Section 8.2.
$P - \Delta$ effects should be considered according to Section 4.15 in structural analysis.

7.2.3 Seismic Design Requirements for the Industrial Component and its Connections

For a non-building structure where the weight of industrial component is more than 25% of the total weight of the structure, interaction effect of industrial components and supporting structure shall be considered. If period of the component modeled along with its connections to the structure is 0.06 sec. or greater, they shall be modeled together in a combined model. In this model, seismic weight and stiffness shall be distributed appropriately. Base shear in equivalent static procedure shall be calculated from Section 7.2.1.2, selecting the minimum value of R_u for the structure from Tables 7.1 or 4.4, or the component response modification factor, from Table 7.2.
If period of the component along with its connections to the structure is less than 0.06 sec., the component and its connections shall be designed for F_p considering requirements from Chapter 8, using R_u from Table 7.2 instead of R_p, and considering $a_p = 1$. In this case, interaction effects between industrial component and supporting structure may be neglected.

7.3 Non-Building Structure with Ground Supported Component

In these structures, industrial component such as horizontal or vertical vessels, heat exchangers, pumps, etc. are situated on a mass concrete foundation or a rigid pedestal. Depending on the fundamental period of the structure, one of the following conditions might happen:
A. The structure is not rigid, with a period of 0.06 sec. or greater.
B. The structure is rigid, with a period of less than 0.06sec.
The structural period shall be calculated from Section 7.2.2.3 generally. In Section 7.3.1 and 7.3.2, seismic design requirements for rigid and non-rigid non-building structures are presented, respectively.

7.3.1 Seismic Design Requirements for Non-Rigid Non-Building Structure with Ground Supported Industrial Component

If period of the structure is 0.06 sec. or greater, requirements from Section 7.2.3 are permitted to be used, to design component and its connections.
Response modification factor for, R_u, shall be determined from Table 7.2.

7.3.2 Seismic Design Requirements for Rigid Non-Building Structure with Ground Supported Industrial Component

If period of the structure is less than 0.06 sec., component and its connections to the foundation shall be designed for a base shear, V_u, from Equation 7.11.

$$V_u = 0.3\, S_{DS} W_p I \qquad 7.11$$

The above base shear shall be distributed vertically proportional to the mass.

Chapter 8

Non-Structural Components

8. Non-Structural Components

8.1 General

8.1.1 Scope

Non-structural components (mechanical or electrical components and architectural elements), are the components connected to the floors or walls of structure. Although these components are not parts of the primary load carrying system of the structure, but may experience noticeable seismic forces during earthquakes. Examples of these components in petroleum industries are tanks, vessels, plumbing, ductworks, escalators, conveyors, small stacks and chimneys, antennas, cranes, computers, control panels, transformers, emergency and standby power systems, fire protection sprinkler piping systems, boilers, heat exchangers, and rotating machineries. Examples of architectural elements are stairways, partitions, precast facades, signs and billboards, lighting systems and suspended ceilings.
The regulations of this chapter can be applied to those non-structural components whose individual weights are less than one fourth of the total weight of the structure, otherwise these components fall under regulations of Chapter 7. For more discussions about selection of analysis method for non-structural components and recognizing from non-building structure, refer to Appendix 3.
To design elements or components and their connections, the required capacity for element and its anchorage shall be determined from Sections 8.3 and 8.4. In addition, specific provisions mentioned in Section 8.5 for architectural components and Section 8.6 for mechanical and electrical components shall be considered. In addition, the ductility issues for components exposed to differential displacements should be considered according to Section 8.2.
The following non-structural components are exempt from the requirements of this section:

1. Temporary or movable equipment.
2. Mechanical and electrical components in Seismic Design Category D_1 with importance factor, I_p, equal to 1.0.
3. Mechanical and electrical components in Seismic Design Categories D_2 and D_3 where all of the followings apply:
   A. The component importance factor, I_p, is equal to 1.0;
   B. Connection between component and structure is capable of transmitting tension;
   C. Flexible connections are provided between the component and associated ductwork, piping, and conduit; and either
      1. The component weighs 1.8 kN or less and has a center of mass located 1.20 m or less above the adjacent floor level;
      2. The component weighs 0.09 kN or less or, in the case of a distributed system, 0.07 kN/m or less.

8.1.2 Importance Factor for Non-Structural Components

The component importance factor, I_p, shall be taken as 1.5 if any of the following conditions apply:

- The component is required to function for life-safety purposes after an earthquake, including fire protection systems, automatic shut off valves and emergency stairways.
- The component conveys, supports, or otherwise contains toxic or explosive substances.
- The component is in or attached to a structure with Function & Risk Category IV and it is needed for continued operation of the facility or its failure could impair the continued operation of the facility.

In other cases, $I_p = 1.0$.

8.1.3 Vendor Specific Provisions

Where vendor specific provisions are in terms of allowable stress or allowable strength, seismic design forces shall be combined with other loads according to Section 2.2.1, and in addition to requirements of Chapter 2 shall be checked with allowable values described in specific provisions. However, details shall be observed according to specific provisions under vendor responsibilities. In any case, the

provisions of this chapter should be evaluated to determine the minimum design seismic force, interaction effects of non-structural components, displacements and design of support anchors.
If the design reference is based on allowable stress design method, the seismic loads determined in accordance with Sections 8.3.1.2, 8.3.2.2 or 8.3.3 depending on case, shall be multiplied by a factor of 0.7 and used with effects of dead, live and operating loads in component design.

8.1.4 Special Certification Requirements for Certain Non-Structural Components

Certifications shall be provided for special non-structural components as follows:

1. Active components that must remain operable following the design earthquake shall be certified by the vendor as operable exclusively based on approved shake table testing or experience data in accordance with Section 8.1.6 unless it can be shown that the component is inherently stronger by comparison with similar seismically qualified components.
2. Components with hazardous substances assigned a component importance factor, I_p, of 1.5; shall be certified by the manufacturer as maintaining containment following the design earthquake ground motion by analysis, experience data, or approved shake table testing in accordance with Section 8.1.6.

8.1.5 Consequential Damage

The functional and physical interrelationship of components, their supports, and their effect on each other shall be considered so that the failure of a component shall not cause the failure of an essential architectural, mechanical, or electrical component.

8.1.6 Determined Capacity Based on Experimental or Experience Data

As an alternative to the analytical requirements of Sections 8.3.1.2, 8.3.2.2 or 8.3.3, it is possible to determine seismic capacity of non-structural components and their connections with experimental or experience data. Authorities shall approve the procedure. In any case, the required capacity shall not be less than values corresponding to above sections, depending on case.

8.2 Relative Displacement

Non-structural components included in this chapter shall be controlled for the seismic relative displacement according to this section and lateral displacement of other loads for stability and strength criteria.
With multiplying D_p by the structure important factor, I, according to Chapter 4, the relative seismic displacement, D_{pI}, is obtained. D_p is the calculated relative displacement according to Equations 8.1 to 8.4.
For existing non-structural components in same structure, the relative displacement between connecting points of non-structural components to the structure in x and y elevations shall be calculated from Equation 8.1.

$$D_p = \delta_x - \delta_y \tag{8.1}$$

D_p, calculated from Equation 8.1, is not required to be taken as greater than Equation 8.2.

$$D_p = \left(h_x - h_y\right)\frac{\Delta_a}{h_s} \tag{8.2}$$

For two connection points of non-structural components on separate structures “A” and “B”, relative displacement between these two points shall be calculated from Equation 8.3.

$$D_p = |\delta_x|_A + \left|\delta_y\right|_B \tag{8.3}$$

D_p, calculated from Equation 8.3, is not required to be taken as greater than value determined from Equation 8.4.

$$D_p = \left(\frac{h_x \Delta_a}{h_s}\right)_A + \left(\frac{h_x \Delta_a}{h_s}\right)_B \quad 8.4$$

In Equations 8.1 to 8.4:
δ_x, δ_y = structural design lateral deflection at level x or y according to Chapter 4
h_x, h_y = height of consecutive levels x and y where component is attached to structure $(h_x \geq h_y)$
Δ_a = allowable story drift for structures A and B
h_s = floor to floor height of the story to which the non-structural component is connected

8.3 Analysis Methods

Non-structural components are categorized into three groups: rigid, flexible and suspended.
If a component is rigid, its dynamic characteristics basically is dependent on its stiffness and ductility of anchors. In this case, the non-structural components can be modeled as SDOF system with stiffness and ductility of anchors and a mass equal to seismic mass of component.
If a non-structural component is flexible, this component shall be modeled as a MDOF system with distributed mass and stiffness like a building-type structure. A non-structural component may be connected to the supporting structure in multiple locations. In this case, these connections shall be considered in the modeling of non-structural component. For suspended non-structural components, the possibility of collision to adjacent components and its effects shall be investigated in the analysis.
It is recommended to model structure with the supported non-structural components as a whole with full geometric conditions and interaction effects between non-structural components and supporting structure using dynamic analysis. However, analysis of non-structural component without consideration of its supporting structure may be conducted using one of following analysis methods; Equivalent Lateral Load Procedure (Section 8.3.1), Simplified Interaction Method (Section 8.3.2) and Story Spectrum Method (Section 8.3.3).
Equivalent Lateral Load and Story Spectrum Method can only be used when the non-structural component is similar to an SDOF system. Otherwise, Simplified Interaction Method shall be used.

8.3.1 Equivalent Lateral Load Procedure

8.3.1.1 General

In this method, non-structural component is assumed to be an SDOF system, and applied lateral force can be calculated according to Section 8.3.1.2. This force shall be used to design of component and its connections. The effects of relative displacements of structure on the component shall be considered according to Section 8.2.

8.3.1.2 Seismic Design Force

Seismic force shall be applied independently in two orthogonal horizontal directions on non-structural component or in one critical horizontal direction, and be distributed in proportion of mass over the component. For vertically cantilevered systems, however, the seismic force shall be assumed to act in any critical horizontal direction. This force, F_p, is determined in accordance with Equation 8.5.

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \quad 8.5$$

F_p needs not be taken greater than Equation 8.6 and shall not be taken less than Equation 8.7.

$$F_p = 1.6 S_{DS} I_p W_p \quad 8.6$$

$$F_p = 0.3 S_{DS} I_p W_p \quad (8.7)$$

When the fundamental period of non-structural component, T_p, determined from Equation 8.16, is greater than $T_f = (1 + 0.25z/h)T_s$, the value of F_p calculated from Equations 8.5 and 8.6 may be scaled down by a factor T_f/T_p.

where:

S_{DS}, S_{D1} = Design spectrum acceleration parameters according to Chapter 3.

a_p = component amplification factor (Tables 8.1 and 8.2).

I_p = component importance factor according to Section 8.1.2.

W_p = non-structural component operating weight

R_p = component response modification factor according to Tables 8.1 and 8.2. In special cases, if R_p values are not addressed in those tables, it is permitted to use R_u from Table 7.2 instead of R_p.

z = height of component attachment point to structure with respect to the base. For items at or below the base, z shall be taken as zero. The value of z need not exceed h .

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

T_s = soil-dependent coefficient in terms of period according to Chapter 3.

For design purposes, the effects of this force shall be combined with service loads applied on that component, assuming over-strength factor to be $\Omega_0 = 1.0$. In this calculation, the redundancy factor may be assumed as $\rho = 1.0$. In addition, vertical component of seismic force shall be considered equal to $\pm 0.2 S_{DS} W_p$ simultaneously with horizontal forces. The concurrent vertical seismic force need not be considered for lay-in access floor panels and lay-in ceiling panels.

If non-seismic lateral forces are greater than F_p, these forces will govern for component design. In any case, requirements of Section 8.5 and 8.6 shall be considered.

In Equation 8.5, using smaller response modification factor, a_p, from independent dynamic analysis is permitted, however, a_p shall not be taken less than 1.0.

Where the mechanical or electrical component is placed over base isolation systems, in all two directions nearby the component, bumper restraints or snubbers shall be provided. If the clear distance between bumper and component is more than 6 mm, the design force for non-structural component shall be taken as $2F_p$, otherwise the it can be taken as F_p.

Table 8.1 Response Coefficients for Architectural Components

Architectural Components	[1]a_p	R_p
Interior non-structural walls and partitions		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever elements		
Unbraced or braced to structural frame below its center of mass, like parapets and cantilever interior non-structural walls, chimneys where laterally braced or supported by the structural frame	2.5	2.5
Braced to structural frame above its center of mass like parapets, chimneys, exterior non-structural walls	1.0	2.5
Exterior non-structural wall elements and connections		
Wall element, body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1
Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Penthouses (except where framed by an extension of the building frame)	2.5	3.5
Ceilings	1.0	2.5
Cabinets for storage of contents and laboratory equipment	1.0	2.5
Access floors		
Access floors	1.0	1.5
Special access floors	1.0	2.5
Appendages and ornamentations	2.5	2.5
Signs and billboards	2.5	3.0
Other rigid components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
Other flexible components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5
Stairways and access	1.0	2.5

1. A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1.0$ is for rigid components and rigidly attached components. The value of $a_p = 2.5$ is for flexible components and flexibly attached components.

Table 8.2 Coefficients for Mechanical and Electrical Components

Mechanical and Electrical Components	[1] a_p	R_p
Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing	2.5	6.0
Wet-side HVAC, 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	2.5
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 7.	1.0	2.5
Skirt supported pressure vessels not within the scope of Chapter 7.	2.5	2.5
Elevator and escalator components	1.0	2.5
Generators, batteries, inverters, motors, transformers, and other electrical components constructed of high deformability materials	1.0	2.5
Motor control centers, panel boards, switch gear, instrumentation cabinets, and other components constructed of sheet metal framing	2.5	6.0
Communication equipment, computers, instrumentation, and controls	1.0	2.5
Roof-mounted stacks, cooling and electrical towers laterally braced below their center of mass	2.5	3.0
Roof-mounted stacks, cooling and electrical towers laterally braced above their center of mass	1.0	2.5
Lighting fixtures	1.0	1.5
Other mechanical or electrical components	1.0	1.5
Vibration Isolated Components and Systems		
Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter stops	2.5	2.5
Spring isolated components and systems and vibration isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops	2.5	2.0
Internally isolated components and systems	2.5	2.0
Suspended vibration isolated equipment including in-line duct devices and suspended internally isolated components	2.5	2.5
Distribution Systems		
Piping in accordance with ASME B31, including in-line components with joints made by welding or brazing	2.5	12.0
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	2.5	6.0
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	2.5	9.0
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	2.5	4.5
Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and non-ductile plastics	2.5	3.0
Ductwork, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing	2.5	9.0
Ductwork, including in-line components, constructed of high- or limited-deformability materials with joints made by means other than welding or brazing	2.5	6.0
Ductwork, including in-line components, constructed of low-deformability materials, such as cast iron, glass, and non-ductile plastics	2.5	3.0
Bus ducts, Plumbing	1.0	2.5
Manufacturing or process conveyors (no personnel)	2.5	3.0
Electrical conduit and cable trays	2.5	6.0

1. A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1.0$ is for rigid components and rigidly attached components. The value of $a_p = 2.5$ is for flexible components and flexibly attached components.

8.3.2 Simplified Interaction Procedure

8.3.2.1 General

This method is developed for analysis of non-structural components based on simplification of analytical analysis of secondary system in interaction with supporting structure. The internal forces shall be calculated and applied for component design in accordance with Section 8.3.2.2. It is also necessary to control non-structural component for seismic relative displacement according to Section 8.2, in combination with lateral displacement due to other loads.
In this procedure, flexibility and mass distribution of non-structural components are considered.

8.3.2.2 Equivalent Lateral Load

Distribution of lateral load shall be calculated according to Equation 8.8.

$$F_{pi} = \frac{w_{pi} l_i}{\sum_{i=1}^{n} w_{pi} l_i} V_p \qquad 8.8$$

where:
F_{pi} = force acting at the center of the i^{th} mass of the component
w_{pi} = weight of the i^{th} part of the component
l_i = distance from the attachment point to center of the i^{th} mass of component in case where there is only one single connecting point (See Figure 8.1a), or distance from the lower or upper attachment point to center of the i^{th} mass of the non-structural components in case of a non-structural component with two attachment points (See Figure 8.1b). For the latter case, the lower attachment point is selected when the i^{th} mass is located below the point at which the element attains its maximum deflection when each mass is subjected to a lateral force equal to its own weight; otherwise, the upper attachment point is selected (See Figure 8.1b).
n = number of the concentrated masses of the component
V_p = base shear or sum of the shears at the supports of the non-structural element, calculated according to Equation 8.9

$$V_p = \frac{S_a}{\lambda / I_p} C_p W_p \qquad 8.9$$

where:
S_a = spectral response acceleration parameter (g) according to Chapter 3.
C_p = component base shear amplification factor from Equation 8.13 considering provisions of Section 8.3.2.3
λ is determined from Equation 8.10

$$\lambda = \begin{cases} \mu_{eq} & T \geq 0.5 \\ \sqrt{2\mu_{eq} - 1} & 0.5 > T \geq 0.125 \\ 1 + \dfrac{33T - 1}{25T}\left(\sqrt{2\mu_{eq} - 1} - 1\right) & 0.125 > T \geq 0.03 \\ 1 & T < 0.03 \end{cases} \qquad 8.10$$

where:
T= fundamental period of structure
μ_{eq}= equivalent ductility factor calculated from Equation 8.11

$$\mu_{eq} = \left[\frac{1}{n + n'}\left(\frac{n}{\mu} + \frac{n'}{\mu_p}\right)\right]^{-1} \qquad 8.11$$

where:
μ = ductility factor of structure in accordance with Equation 8.12
μ_p = ductility factor of non-structural component, derived from an equation similar to Equation 8.12, by substituting R_p from Tables 8.1 or 8.2, $\Omega_0 = 1.0$ and fundamental period, T_p, obtained from Equation 8.16
n = number of stories of structure
n' = number of non-structural component parts according to Figure 8.1

$$\mu = \begin{cases} \frac{R_u}{\Omega_0} & T \geq 0.5 \\ 0.5\left[\left(\frac{R_u}{\Omega_0}\right)^2 + 1\right] & T < 0.5 \end{cases} \tag{8.12}$$

Where R_u and Ω_0 are response coefficient and over-strength factor of the structure, respectively.
Amplification factor in Equation 8.9 is:

$$C_p = \frac{1}{2\sqrt{\left|\frac{W_p}{W} - \frac{0.0025}{\phi_0^2}\right|}} \leq 12.5\phi_0 \tag{8.13}$$

where:
W= effective seismic weight of the structure
ϕ_0= a parameter given by Equation 8.14 as following:

$$\phi_0 = \frac{Wh_{av}}{\sum_{i=1}^{N} w_i h_i} \tag{8.14}$$

where:
h_i = height of level i, calculated from the base level
h_{av} = average heights of attachment points of the building to the non-structural component, calculated from the base level
w_i= seismic effective weight of level i
For non-structural components, that have more than two elements attached to structure, the same procedure as above can be used for each region between the two adjacent connections.
Limitations of Equation 8.2 and 8.3 are valid for the sum of F_{pi} values.

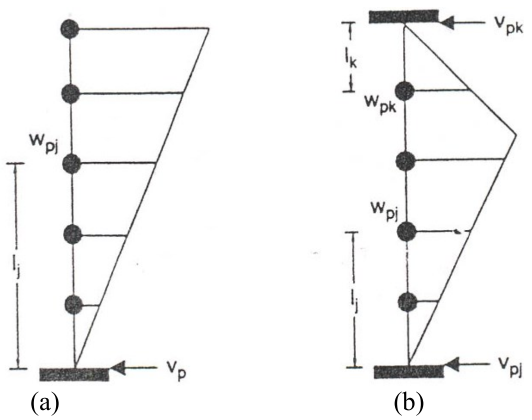


Figure 8.1 Hypothetical 1st Mode Shape for Non-Structural Components by Connections in One or Two Points

8.3.2.3 Modified Amplification Factor

If T_p is calculated or determined, the amplification factor, C_p from Equation 8.13 which is derived conservatively in an amplified condition, may be replaced by a modified amplification factor, C_p, calculated from Figure 8.2. In Figure 8.2, b is determined from Equation 8.15.

$$b = \frac{1}{2}\phi_0\sqrt{W_p/W} \qquad 8.15$$

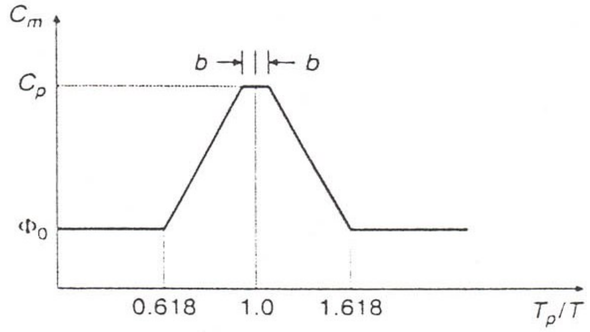


Figure 8.2 Variation of Modified Amplification Factor with Respect to Different Fundamental Periods

8.3.2.4 Fundamental Period of Non-Structural Component

The fundamental period of the non-structural component, T_p, shall be calculated from Equation 8.16 provided that the non-structural component and its connections can be modeled approximately with mass-spring SDOF system:

$$T_p = 2\pi\sqrt{W_p/K_p g} \qquad 8.16$$

where:
K_p = total lateral stiffness of component and its connection to structure.
The natural period of non-structural components can also be derived from experimental and analytical procedures.

8.3.3 Floor Spectrum Procedure

Where more accuracy is required for determination of acceleration and displacement of installation location of a non-structural component, using the floor spectrum is recommended. The lateral force applied to non-structural component, F_p, shall be calculated from Equation 8.17.

$$F_p = \frac{a_i a_p W_p}{(R_p/I_p)} A_x \qquad 8.17$$

where:
a_i = total spectral accelerations of the i^{th} floor, calculated by spectral combination with $R_p = 1$.
A_x = torsional amplification factor of the structure (Equation 4.21)
In Equation 8.17, using smaller values of amplification factor, a_p, calculated from an independent dynamic analysis of non-structural component is permitted. However, a_p shall not be taken smaller than one.
The limitations of Equation 8.2 and 8.3 are also valid for this procedure. Evaluation of the seismic relative displacement according to Section 8.2 with combination of lateral displacements due to other loads is also required.

8.4 Anchorage of Non-Structural Components

Where the non-structural components are anchored to the base via short expandable anchors, short chemical bolts, or in-situ bolts with low flexibility, the maximum response factor R_p in Equations 8.5 and 8.17 is taken as 1.5. The embedded anchorage in concrete or masonry materials shall resist the minimum of connection strength, 1.3 times the calculated connection force, and the maximum force that can be transferred through the component to the connection. In calculation of anchorage forces, the connection conditions such as eccentricity and leveraging shall be considered. Forces within anchors shall be distributed with respect to the stiffness and redistribution of anchors forces after yielding.

According to this section, non-structural components shall be attached or anchored to their supporting structure. Frictional resistance due to gravity load shall not be considered in design of component connection to the structure. For load transmission between the component and structure, a continuous path shall be provided with sufficient strength and stiffness. When these forces govern for members or connections of structures located in the path of transmission, these members and connections shall be designed for those loads.

In case of using Power Actuated Fasteners like bullet anchor (cotter anchor) in concrete or steel to transfer tensile forces and for masonry structures, a seismic resistance certificate is required according to Section 8.1.4. Use of Power Actuated Fasteners in concrete to support acoustical tile or lay-in panel suspended ceiling applications and distributed systems where the service load on any individual fastener does not exceed 0.4 kN and in steel where the service load on any individual fastener does not exceed 1.1 kN, is permitted.

Friction clips shall not be used for supporting sustained loads in addition to resisting seismic forces. C-type beam and large flange clamps are permitted for hangers provided that they are equipped with restraining straps. Lock nuts or equivalent shall be provided to prevent loosening of threaded connections

Anchors in masonry shall be placed so that the connection strength does not exceed the yield strength of the anchor. Otherwise, the design strength of the anchors shall be at least 2.5 times the factored forces transmitted by the component.

8.4.1 Design Force in the Attachment

Value of F_p as determined in Section 8.3 is used to design connections, except that R_p shall not be taken as larger than 6. In determining the design force in each attachment, installation conditions such as eccentricity and leverage effects shall be considered. Force distribution between multiple attachments at one location shall take into account the stiffness and ductility of the attachments and the ability to redistribute loads to other attachments after yielding.

8.4.2 Anchors in Concrete

8.4.2.1 General

Anchors in concrete shall be designed in accordance with this section, Reference [8] or other valid codes.

8.4.2.2 Anchors Capacity

The pull-out capacity of an anchor, P_{nr}, shall be calculated from multiplying nominal pull-out capacity, P_n, to the capacity reduction factors according to Equation 8.18.

$$P_{nr} = R_1 R_2 R_3 R_4 P_n \tag{8.18}$$

where:

R_1 to R_4= pull-out capacity reduction factors for concrete strength, crack amount in concrete, sensitive content, and inspection quality, respectively.

The shear capacity of an anchor, V_{nr}, shall be calculated by multiplying the nominal shear capacity, V_n, to the reduction factors according to Equation 8.19.

$$V_{nr} = R'_1 R'_3 R'_4 V_n \tag{8.19}$$

where:
R'_1, R'_3 and R'_4= reduction factors for concrete strength, sensitive content, and inspection quality, respectively.
Nominal pull-out capacity, P_n, and shear capacity, V_n, can be calculated in accordance with Reference [8] or other valid codes.

8.4.2.2.1 Reduction Factors for Concrete Strength, R_1 and R'_1

For expansion anchors:

$$R_1 = \begin{cases} 1.0 & f'_c \geq 28 \\ \dfrac{f'_c}{28} & 28 > f'_c \geq 14 \\ 0.0 & f'_c < 14 \end{cases} \tag{8.20}$$

$$R'_1 = \begin{cases} 1.0 & f'_c \geq 25 \\ \dfrac{f'_c}{70} + 0.65 & 25 > f'_c \geq 14 \\ 0.0 & f'_c < 14 \end{cases} \tag{8.21}$$

For cast-in-place bolts and headed studs, cast-in-place J-bolts and grouted-in-place bolts:

$$R_1 = R'_1 = \begin{cases} 1.0 & f'_c \geq 25 \\ \sqrt{\dfrac{f'_c}{25}} & 28 > f'_c \geq 17.5 \\ 0.0 & f'_c < 17.5 \end{cases} \tag{8.22}$$

8.4.2.2.2 Reduction Factor for Cracked Concrete, R_2

For expansion anchors:

$$R_2 = \begin{cases} 1.0 & 0.25 > w_c \\ 0.75 & 0.25 \leq w_c < 0.5 \\ 0.0 & 0.50 \leq w_c \end{cases} \tag{8.23}$$

For cast-in-place bolts and headed studs, grouted-in-place bolts:

$$R_2 = \begin{cases} 1.0 & 0.25 > w_c \\ 1.08 - 0.32 w_c & 0.25 \leq w_c < 1.5 \\ 0.0 & 1.50 \leq w_c \end{cases} \tag{8.24}$$

Equations for J-bolts are same as bolts, unless when $w_c > 0.5$, R_2 shall be taken zero ($R_2 = 0$).
where:
f'_c = compressive strength of concrete (MPa)
w_c = computed crack width at the place of anchor (mm)

8.4.2.2.3 Reduction Factors for Sensitive Contents, R_3 and R'_3

For expansion anchors which support components with Function & Risk Category IV (Table 4.3), or component with special condition that is presented in Section 8.1.2, R_3 and R'_3 shall be considered 0.75 and in other cases shall be considered as one.

8.4.2.2.4 Inspection Quality Reduction Factors, R_4 and R'_4

In the case of high accuracy inspection, the reduction factors R_4 and R'_4 shall be considered one and in other cases shall be considered as 0.75.

8.4.2.3 Anchors Design

8.4.2.3.1 Expansion Anchors, Cast-in-Place Bolts and Headed Studs, Grouted-in-Place Bolts

The simultaneous effects of shear and tension for expansion anchors, cast-in-place bolts and headed studs, grouted-in-place bolts can be controlled according to Equation 8.25.

$$\begin{cases} \dfrac{V}{V_{nr}} \leq 0.3 & \rightarrow \quad \dfrac{P}{P_{nr}} \leq 1.0 \\ 0.3 < \dfrac{V}{V_{nr}} \leq 1.0 & \rightarrow \quad 0.7\dfrac{P}{P_{nr}} + \dfrac{V}{V_{nr}} \leq 1.0 \end{cases} \tag{8.25}$$

where:
P = tension force caused by the combined dead and seismic loads
V = shear force caused by the combined dead and seismic loads

8.4.2.3.2 Anchors Welded to Concrete Embedded or Exposed Steel

The simultaneous effects of shear and tension shall be controlled according to Equation 8.26.

$$\left(\frac{P_w}{F_w}\right)^2 + \left(\frac{V_w}{F_w}\right)^2 \leq 1 \tag{8.26}$$

where:
P_w = applied tensile forces to the weld
V_w = applied shear forces to the weld
F_w = capacity of weld under simultaneous effects of shear and tension

8.5 Architectural Components

8.5.1 General

Architectural components, and their supports and attachments, shall satisfy the requirements of this section as well as previous sections. Components supported by chains or otherwise suspended from the structure are not required to satisfy the requirements of Section 8.2 and Section 8.3 for seismic force and relative displacement, if they meet all of the following criteria:

1. The design load for such items shall be equal to 1.4 times the operating weight acting down with a simultaneous horizontal load equal to 1.4 times the operating weight, corresponding to the most unfavorable effects.
2. Seismic interaction effects with structure and adjacent components shall be considered in accordance with Section 8.1.5.
3. The connection to the structure shall allow a 360° range of motion in the horizontal plane.

8.5.2 Vertical Deflection

Architectural components shall be designed considering vertical deflection due to joint rotation of cantilever structural members.

8.5.3 Exterior Non-Structural Wall Elements and Connections

Exterior non-structural wall panels or elements that are attached to or enclose the structure shall be designed to accommodate the seismic relative displacements defined in Section 8.2 and movements due to temperature variations. Such elements shall be supported by means of positive and direct structural supports or by mechanical connections with the ability of tension transfer and fasteners in accordance with the following requirements:

1. Connections and panel joints shall allow for the story drift caused by relative seismic displacements determined in Section 8.2, or 15 mm, whichever is greater.
2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections that provide equivalent sliding or ductile capacity.
3. The connecting member itself shall have sufficient ductility and rotation capacity to preclude fracture of concrete or brittle failure at or near welds.
4. All fasteners in the connecting system such as bolts, inserts, welds, and dowels and the body of the connectors shall be designed for the force F_p determined by Section 8.3.
5. Where anchorage is achieved using flat straps embedded in concrete or masonry, such straps shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel or to assure that pull-out of anchorage is not the initial failure mechanism.

8.5.4 Out of Plane Bending

For flat components, it is necessary to evaluate transverse or out-of-plane bending or deformation of a component or system that is subjected to forces as determined in Section 8.3. These amounts shall not exceed the deflection capability of the component or system.

8.5.5 Suspended Ceiling

Suspended ceilings with areas less than or equal to 15 m^2 that are surrounded by walls or soffits that are laterally braced to the structure above are exempt from the requirements of this section. In addition, suspended ceilings constructed of screw- or nail-attached gypsum board on one level that are surrounded by and connected to walls or soffits that are laterally braced to the structure above are exempt from the requirements of this section.

The weight of the suspended ceiling, W_p, shall include the ceiling grid, ceiling tiles or panels, light fixtures if attached to, clipped to, or laterally supported by the ceiling grid, and other components that are laterally supported by the ceiling. This value shall not be taken as less than 0.2 kN/m^2. The seismic force, F_p, shall be transmitted in a safe manner through the ceiling attachments to the building structural elements or the ceiling–structure boundary.

As an alternate to providing large clearances around sprinkler system penetrations through ceilings, the sprinkler system and ceiling grid are permitted to be designed and tied together as an integral unit.

Acoustical tile or lay-in panel ceilings shall also comply with the following:

1. The width of the perimeter supporting closure angle or channel shall not be less than 50 mm. Where perimeter supporting clips are used, they shall be qualified in accordance with an approved test criteria. In each orthogonal horizontal direction, one end of the ceiling grid shall be attached to the closure angle or channel. The other end in each horizontal direction shall have a 20 mm clearance from the wall and shall rest upon and be free to slide on a closure angle or channel.
2. For ceiling areas exceeding 230 m^2, a seismic separation joint or full height partition that breaks the ceiling up into areas not exceeding 230 m^2, each with a ratio of the long to the short dimension less than or equal to 4, shall be provided unless structural analyses are performed for the ceiling bracing system for the prescribed seismic forces that demonstrate ceiling penetrations and closure angles or channels provide sufficient clearance to accommodate the anticipated lateral displacement. Each area shall be provided with closure angles or channels in accordance with Article 1 and horizontal restraints or bracing.

8.5.6 Access Floor

8.5.6.1 General

The effective weight of the access floor, W_p, shall include the weight of the floor system, 100% of the weight of all equipment fastened to the floor, and 25% of equipment supported by but not fastened to the floor. The seismic force, F_p, shall be transmitted from the top surface of the access floor to the supporting structure. Overturning effects of equipment fastened to the access floor panels also shall be considered. The ability of "slip on" heads for pedestals shall be evaluated for suitability to transfer overturning effects of equipment. Where checking individual pedestals for overturning effects, the maximum concurrent axial load shall not exceed the portion of W_p assigned to the pedestal under consideration.

8.5.6.2 Special Access Floors

Access floors shall be considered to be "special access floors" if they are designed to comply with the following considerations:

1. Connections transmitting seismic loads consist of mechanical fasteners, anchors satisfying the requirements of Section 8.4, welding, or bearing surfaces.
2. Seismic loads are not transmitted by friction; power actuated fasteners, adhesives, or by friction produced solely by the effects of gravity.
3. The design analysis of the bracing system includes the destabilizing effects of individual members buckling in compression.
4. Bracing and pedestals are of structural or mechanical shapes produced to ASTM specifications that specify the minimum required mechanical properties. Electrical tubing shall not be used.
5. Floor stringers that are designed to carry axial seismic loads and are mechanically fastened to the supporting pedestals are used.

8.5.7 Partitions

Partitions that are tied to the ceiling and all partitions greater than 1.8 m in height shall be laterally braced to the building structure. Such bracing shall be independent of any ceiling lateral force bracing. Bracing shall be spaced to limit horizontal deflection at the partition head to be compatible with ceiling deflection requirements as determined in Section 8.5.5 for suspended ceilings and elsewhere in this section for other systems.

Provisions of this article need not be applied to the partitions that meet all of the following conditions:

1. The partition height does not exceed 2.7 m
2. The linear weight of the partition does not exceed the product of 0.5 kN times the height (m) of the partition.
3. The partition horizontal seismic load does not exceed 0.25 kN/m^2

8.5.8 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions

Glass in glazed curtain walls, glazed storefronts and glazed partitions shall meet the relative displacement requirement of Δ_f equal to $1.25 \times I \times D_p$, but should not be taken less than 15 mm. D_p can be calculated from Equation 8.1 and importance factor, I, is extracted from Table 4.3.

Glass in glazed curtain walls, glazed storefronts, and glazed partitions shall meet the relative displacement requirement of Equation 8.27:

$$\Delta_f \geq max\left(1.25 \times I \times D_p, 15mm\right) \qquad 8.27$$

where;

Δ_f = The drift causing glass fallout from the curtain wall, storefront, or partition, and shall be determined in accordance with Reference [9] or by engineering analysis.

D_p = The relative seismic displacement that the component must be designed to accommodate (Section 8.2) and shall be applied over the height of the glass component under consideration.
I = The importance factor determined in accordance with Table 4.3
In the following cases, this section need not to be applied:

1. Glass with sufficient clearances from its frame such that physical contact between the glass and frame will not occur at the relative seismic displacement equal to $1.25D_p$. Relative displacement, measured over the height of the glass panel under consideration, which causes initial glass-to-frame contact, D_c, for rectangular glass panels within a rectangular wall frame is determined by Equation 8.28:

$$D_c = 11 + h_p c_2 b_p c_1 \geq 1.25D_p \qquad 8.28$$

   where;
   h_p= height of the rectangular glass panel
   b_p= width of the rectangular glass panel
   c_1= average of the clearances (gaps) on both sides between the vertical glass edges and the frame
   c_2= average of the clearances (gaps) top and bottom between the horizontal glass edges and the frame
2. Fully tempered monolithic glass in Function & Risk Categories I, II, and III located no more than 3 m above a walking surface need not comply with this requirement.
3. Annealed or heat-strengthened laminated glass in a single thickness with an interlayer no less than 0.75 mm that is captured mechanically in a wall system glazing pocket, and whose perimeter is secured to the frame by a wet glazed gunable curing elastomeric sealant perimeter bead of 15 mm minimum glass contact width, or other approved anchorage system need not comply with this requirement.

8.6 Mechanical and Electrical Components

8.6.1 General

Mechanical and electrical components and their supports shall satisfy the requirements of this section. The attachment of mechanical and electrical components and their supports to the structure shall meet the requirements of Section 8.4.
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 6 mm. If the nominal clearance specified on the construction documents is not greater than 6 mm, the design force is permitted to be taken as F_p.
Light fixtures, lighted signs, and ceiling fans not connected to ducts or piping, which are supported by chains or otherwise suspended from the structure, are not required to satisfy the seismic force and relative displacement requirements of Section 8.2 provided they meet all of the following criteria:

1. The design load for such items shall be equal to 1.4 times the operating weight acting down with a simultaneous horizontal load equal to 1.4 times the operating weight. The horizontal load shall be applied in the direction that results in the most critical loading for the design.
2. Seismic interaction effects with structure and adjacent components shall be considered in accordance with Section 8.1.5.
3. The connection to the structure shall allow a 360 ^{o} range of motion in the horizontal plane.

Where design of mechanical and electrical components for seismic effects are required, consideration shall be given to the dynamic effects of the vibration of components, their contents, and where appropriate, their supports and attachments. In such cases, the interaction between the components and the supporting structures, including other mechanical and electrical components, shall also be considered.
When designing the anchorage of such components to their supports, frictional resistance due to operating weight shall not be included.

8.6.2 Mechanical Component

Mechanical components with I_p greater than 1.0 shall be designed for the seismic forces and relative displacements defined in Section 8.2 and 8.3 and shall satisfy the following additional requirements:

1. Provision shall be made to eliminate seismic impact for components vulnerable to impact, for components constructed of non-ductile materials, and in cases where material ductility will be reduced due to service conditions (e.g., below zero temperature applications).
2. The possibility of loads imposed on components by attached utility or service lines, due to differential movement of support points on separate structures, shall be evaluated.
3. Where piping or HVAC ductwork components are attached to structures that could displace relative to one another and for isolated structures where such components cross the isolation interface, the components shall be designed to accommodate the seismic relative displacements defined in Chapter 9.

8.6.3 Electrical Component

Electrical components with I_p greater than 1.0 shall be designed for the seismic forces and relative displacements defined in Sections 8.2 and 8.3 and shall satisfy the following additional requirements:

1. Provisions shall be made to eliminate seismic impact between components.
2. Loads imposed on the components by attached utility or service lines that are attached to separate structures shall be evaluated.
3. Batteries on racks shall have wrap-around restraints to ensure that the batteries will not fall from the racks. Spacers shall be used between restraints and cells to prevent damage to cases. Racks shall have adequate capacity for lateral loads.
4. Internal coils of dry type transformers shall be positively attached to their supporting substructure within the transformer enclosure.
5. Electrical control panels, computer equipment, and other items with slide-out components shall have a latching mechanism to hold the components in place.
6. Electrical cabinet design shall comply with the applicable standards. Cutouts in the lower shear panel that have not been made by the manufacturer and reduce significantly the strength of the cabinet shall be specifically evaluated.
7. Securing control box, computers and other components which have elements that may fall down by appropriate fasteners.
8. The attachments for additional external items weighing more than 0.45 kN shall be specifically evaluated if connection specifications are not provided by the manufacturer.
9. Where conduit, cable trays, or similar electrical distribution components are attached to structures that may displace relative to one another and for isolated structures (Chapter 9) where such components cross the isolation interface, the components shall be designed to accommodate the seismic relative displacements defined in Section 8.2

8.6.4 Component Supports

Mechanical and electrical component and the means by which they are attached to the component shall be designed for the forces and displacements determined in Section 8.2 and 8.3. Such supports include structural members, braces, frames, skirts, steel legs, saddles, concrete and steel pedestals, cables, guys, stays, snubbers, and tethers, as well as elements forged or cast as a part of the mechanical or electrical component. In addition, displacement shall be checked according to Section 8.2.

8.6.4.1 Standard Supports

If standard supports are used according to appropriate provisions, their seismic efficiency shall be determined by test or load rating analysis. In addition, the stiffness of the support, where appropriate, shall be designed such that the seismic load path for the component performs its intended function.

8.6.4.2 Design for Relative Displacement

Component supports shall be designed to accommodate the seismic relative displacements between points of support determined in accordance with Section 8.2.

8.6.4.3 Support Attachment to Component

The means by which supports are attached to the component, except where integral, shall be designed to accommodate both the forces and displacements determined in accordance with Section 8.2 and 8.3. If the value of $I_p = 1.5$ for the component, the local region of the support attachment point to the component shall be evaluated for the effect of load transfer on the component wall.

8.6.4.4 Conduit, Cable Tray, and Other Electrical Distribution Systems (Raceways)

Raceways shall be designed for seismic forces and seismic relative displacements as required in Section 8.2. Conduits greater than 65 mm trade size and attached to panels, cabinets, or other equipment subject to seismic relative displacement shall be provided with flexible connections or designed for seismic forces and seismic relative displacements.
In the following conditions, design of raceways for seismic forces and displacements according to Section 8.2 & 8.3 is not required:

1. Trapeze assemblies are used to support raceways and the total weight of the raceway supported by trapeze assemblies is less than 0.015 kN/m.
2. The raceway is supported by hangers and each hanger in the raceway run is 300 mm or less in length from the raceway support point to the supporting structure. Where rod hangers are used, they shall be equipped with swivels to prevent inelastic bending in the rod.
3. Where the conduit is less than 65 mm trade size.

8.6.4.5 Other Requirements

The following additional requirements shall be applied to mechanical and electrical component supports:

1. Reinforcement (e.g., stiffeners or Belleville washers) shall be provided at bolted connections through sheet metal equipment housings as required to transfer the equipment seismic loads specified in this section from the equipment to the structure.
2. Where weak-axis bending of cold-formed steel supports is relied on for the seismic load path, such supports shall be specifically evaluated.
3. Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction, and vertical restraints shall be provided where required to resist overturning. Isolator housings and restraints shall be constructed of ductile materials. A viscoelastic pad or similar material of appropriate thickness shall be used between the bumper and components to limit the impact load. The provisions of Section 8.6.1 shall be considered for components mounted on vibration isolators.
4. In the case where one of the following conditions happens, the supports of electrical distribution components should be designed for following forces and relative displacements in accordance with Section 8.2 and 8.3:
    A. $I_p = 1.5$ and the cable conduit diameter is larger than 65 mm
    B. Hangers and trays and ductworks with $I_p = 1.5$ and total weight more than 0.15 kN/m
    C. The support is cantilevered, from the floor
    D. The support has bracings to restrict displacement
    E. The support is a rigid frame with welded connections
5. For piping, boilers, and pressure vessels, attachments to concrete shall be suitable for cyclic loads.
6. For mechanical equipment, drilled and grouted in-place anchors for tensile load applications shall use either expansive cement or expansive epoxy grout.

8.6.5 Utility and Service Lines

At the interface of adjacent structures or portions of the same structure that may move independently, utility lines shall be provided with adequate flexibility to accommodate the anticipated differential movement between the portions that move independently. Differential displacement calculations shall be determined in accordance with Section 8.2. The possible interruption of utility service in structures of Function & Risk Category IV (Table 4.3) shall be considered in relation to designated seismic systems, especially in Site Class IV (according to Reference [2]) and site with $S_{DS} \geq 0.33$.

8.6.6 Ductwork

HVAC and other ductwork shall be designed for seismic forces and seismic relative displacements as required in Section 8.2 and 8.3. Design for the displacements across seismic joints shall be required for ductwork according to Section 8.2 with $I_p = 1.5$ without consideration of the exceptions below.

The following exceptions pertain to ductwork not designed to carry toxic or flammable gases or used for smoke control according to Section 8.2:

1. Trapeze assemblies are used to support ductwork and the total weight of the ductwork supported by trapeze assemblies is less than 0.15 kN/m.
2. The ductwork is supported by hangers and each hanger in the duct run is 300 mm or less in length from the duct support point to the supporting structure. Where rod hangers are used, they shall be equipped with swivels to prevent inelastic bending in the rod.
3. Where provisions are made to avoid impact with larger ducts or mechanical components or to protect the ducts in the event of such impact; and HVAC ducts have a cross-sectional area of less than 0.55 m^2, or weigh 0.25 kN/m or less.

Components that are installed in-line with the duct system and have an operating weight greater than 0.33 kN, such as fans, heat exchangers, and humidifiers, shall be supported and laterally braced independent of the duct system and such braces shall meet the force requirements of Section 8.3. Appurtenances such as dampers, louvers, and diffusers shall be positively attached with mechanical fasteners. Unbraced piping attached to in-line equipment shall be provided with adequate flexibility to accommodate the seismic relative displacements of Section 8.2.

8.6.7 Piping Systems

Unless otherwise noted in this section, piping systems shall be designed for the seismic forces and seismic relative displacements of Section 8.2 and 8.3.

Where other applicable material standards or recognized design bases are not used, piping design including consideration of service loads shall be based on the following allowable stresses:

1. For piping constructed with ductile materials (e.g., steel, aluminum, or copper), 90 percent of the minimum specified yield strength.
2. For threaded connections in piping constructed with ductile materials, 70 percent of the minimum specified yield strength.
3. For piping constructed with non-ductile materials (e.g., cast iron or ceramics), 10 percent of the material minimum specified tensile strength.
4. For threaded connections in piping constructed with non-ductile materials, 8 percent of the material minimum specified tensile strength

Piping not detailed to accommodate the seismic relative displacements at connections to other components shall be provided with connections having sufficient flexibility to avoid failure of the connection between the components.

Exception: Design of piping systems and attachments for the seismic forces and relative displacements of Section 8.2 is not required where one of the following conditions apply:

1. Trapeze assemblies are used to support piping whereby no single pipe exceeds the limits set forth in 3A and 3B below, and the total weight of the piping supported by the trapeze assemblies is less than 0.15 kN/m.
2. Piping is supported by hangers and each hanger in the piping run is 300 mm or less in length from the top of the pipe to the supporting structure. Where pipes are supported on a trapeze, the trapeze shall be supported by hangers having a length of 300 mm or less. Where rod hangers are used, they shall be equipped with swivels, eye nuts, or other devices to prevent bending in the rod.

3. Piping having an R_p in Table 8.2 of 4.5 or greater is used and provisions are made to avoid impact with other structural or non-structural components or to protect the piping in the event of such impact and where the following size requirements are satisfied:
   A. For I_p greater than 1.0, the nominal pipe size shall be 25 mm or less.
   B. For $I_p = 1.0$, the nominal pipe size shall be 80 mm or less.

8.6.8 Elevators and Escalators

Elevators operating with a speed of 45 m/min or greater shall be provided with seismic switches. Seismic switches shall provide an electric signal indicating that structural motions are of such a magnitude that the operation of the elevators may be impaired.
In cases where seismic switches cannot be located near a column in accordance, they shall have two horizontal axes of sensitivity and have a trigger level set to 20 percent of the acceleration of gravity where located at or near the base of the structure and 50 percent of the acceleration of gravity in all other locations.
In facilities where the loss of the use of an elevator is a life-safety issue, the elevator shall only be used after the seismic switch has been triggered provided that:
1. The elevator shall operate no faster than the service speed.
2. Before the elevator is occupied, it is operated from top to bottom and back to top to verify that it is operable.

In addition, retainer plates are required at the top and bottom of the car and counterweight.

8.6.9 Other Mechanical and Electrical Components

For mechanical components with hazardous substances assigned a component importance factor, I_p, of 1.5, and for boilers and pressure vessels, the design strength for seismic loads shall be based on the following material properties:
1. For mechanical components constructed with ductile materials (e.g., steel, aluminum, or copper), 90 percent of the minimum specified yield strength.
2. For threaded connections in components constructed with ductile materials, 70 percent of the minimum specified yield strength.
3. For mechanical components constructed with non-ductile materials (e.g., plastic, cast iron, or ceramics), 10 percent of the material minimum specified tensile strength.
4. For threaded connections in components constructed with non-ductile materials, 8 percent of the material minimum specified tensile strength.

Chapter 9

Base Isolation Systems

9. Base Isolation Systems

9.1 General Provisions

Analysis and design of base isolation systems, structural elements and members of isolated superstructures and foundation are discussed in this chapter. Although the base isolation system is more effective for stiffer and shorter structures, using this system in other structures leads to improvement of structural behavior and decreasing non-structural element damages.

9.1.1 Variations in Material Properties

The analysis of seismically isolated structures, including the substructure, isolators, and superstructure, shall consider variations in seismic isolator material properties over the projected life of the structure including changes due to aging, contamination, environmental exposure, loading rate, scragging, and temperature.

9.1.2 Definitions

Superstructure: Top side of the structure, which is seismically isolated
Substructure: Bottom side of the structure that moves with ground simultaneously.
Isolation Interface: Boundary between the superstructure and substructure
Isolation System: The system that includes all isolator units and all connections to superstructure and bottom support, rigid slab of force transmitter, elements of wind-resistant system, dampers (if exist) and displacement limiter system.
Isolator Unit: A horizontally flexible and vertically stiff structural element of the isolation system that permits large lateral deformations. Isolator units are permitted to be used either as part of, or in addition to, the weight-supporting system of the structure.
Displacement Restraint System: A collection of structural elements that limits lateral displacement of seismically isolated structures due to the rare earthquake (Seismic Hazard Level III according to Chapter 3 provisions).
Design Displacement: The design earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.
Total Design Displacement: The design earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system or an element thereof.
Maximum Displacement: The rare earthquake lateral displacement (Seismic Hazard Level III according to Chapter 3 provisions), excluding additional displacement due to actual and accidental torsion.
Total Maximum Displacement: The maximum considered earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system or elements thereof, design of structure separations, and vertical load testing of isolator unit prototypes.
Effective Damping: Value of the equivalent viscous damping corresponding to energy dissipated during cyclic response of the isolation system.
Effective Stiffness: Value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.

9.2 General Design Requirements

Performance of base isolation elements shall be controlled by analysis and test for total maximum displacement according to provisions of this chapter. In addition to the specifications of this chapter, importance factor, over-strength factor, displacement magnification factor, structural height limitations and seismic force-resisting system for superstructure are determined similar to the non-isolated structure.

9.2.1 Importance Factor

The importance factor, I, shall be taken as 1.0 for a seismically isolated structure, regardless of its Function & Risk Category.

9.2.2 Classification of Structures by Configuration

Regularity of each structure shall be determined based on the superstructure from Tables 4.1 and 4.2.

9.3 Ground Motion

9.3.1 Site-Specific Spectrum

Development of site-specific spectrum corresponding to design earthquake according to Section 3.4.2 and rare earthquake according to Section 3.4.3 is needed for analysis and design of all isolated structures. Spectrum corresponding to the rare earthquake is used for control of total maximum displacement, overturning and the tests needed for isolation device.

9.3.2 Ground Motion Histories

Ground motion histories for response history procedure is selected according to Section 4.10 provisions except that $0.2T$ and $1.5T$ shall be replaced by $0.5T_D$ and $1.25T_M$, respectively, where T_D and T_M are defined in Sections 9.5.2.2 and 9.5.2.4.

9.4 Analysis Procedure Selection

Analysis method is selected regarding Section 9.4.1 to 9.4.3.

9.4.1 Equivalent Lateral Load Procedure Scope of Use

The Equivalent Lateral Load Procedure is permitted to be used for design of a seismically isolated structure provided that:

1. The structure is located at a site with S_1 less than 0.60, where S_1 is 5% damped mapped spectral acceleration (g) on bedrock, corresponding to the rare earthquake for a period of 1 sec.
2. The structure is located on a Site Class I, II and III according to Reference [2].
3. The structure above the isolation interface is less than or equal to four stories or 20 m in structural height.
4. The effective period of the isolated structure at the maximum displacement, T_M (Section 9.5.2.4), is less than or equal to 3.0 seconds.
5. The effective period of the isolated structure at the design displacement, T_D (Section 9.5.2.2), is greater than three times the elastic, fixed-base period of the structure above the isolation system.
6. The structure above the isolation system has a regular configuration.
7. The isolation system meets all of the following criteria:
   A. The effective stiffness of the isolation system at the design displacement (Section 9.11.4.1) is not less than one-third of the effective stiffness at 20% of the design displacement.
   B. The isolation system is capable of producing a restoring force as specified in Section 9.7.1.4.
   C. Each block of isolation system does not limit the rare earthquake displacement to less than the total maximum displacement (Section 9.5.3).

9.4.2 Modal Response Spectrum Analysis Scope of Use

Modal Response Spectrum Analysis is permitted to be used for design of a seismically isolated structure only where articles 2 and 7 of Section 9.4.1 are satisfied.

9.4.3 Seismic Response History Procedure Scope of Use

Seismic Response History Procedure is permitted for design of any seismically isolated structure.

9.5 Equivalent Lateral Load Procedure

9.5.1 Characteristics of Base Isolation System

The base isolation characteristics are determined from tests according to Section 9.11. If base isolation system includes wind-resistant elements, their effects shall be considered.

9.5.2 Lateral Displacements

9.5.2.1 Design Lateral Displacement

Design lateral displacement of base isolation system in the direction of consideration, D_D, is calculated from Equation 9.1:

$$D_D = gS_{D1}T_D\frac{1}{4\pi^2 B_D} \tag{9.1}$$

where:
g = ground gravity acceleration
S_{D1} = Spectral response acceleration parameter according to the definitions of Chapter 3
T_D = effective period of the seismically isolated structure in seconds, at the design displacement in the direction under consideration, as prescribed in Section 9.5.2.2.
B_D= numerical coefficient related to the effective damping of the isolation system for design earthquake level (Seismic Hazard Level II according to Chapter 3), as set forth in Table 9.1.

Table 9.1 Damping Coefficient, B_D or B_M

B_D or B_M Factor	Effective Damping Ratio of Isolator in Design (S.H.L. II) or Rare Earthquake (S.H.L. III)
0.8	≤ 2
1.0	5
1.2	10
1.5	20
1.7	30
1.9	40
2.0	≥ 50

9.5.2.2 Effective Period at Design Displacement

The effective period of the isolated structure at design displacement, T_D, shall be determined using characteristics of the isolation system and Equation 9.2.

$$T_D = 2\pi\sqrt{\frac{W}{K_{D\,min}\,g}} \tag{9.2}$$

where:
$K_{D\,min}$ = minimum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration, as prescribed by Section 9.11.4.1.
W = effective seismic weight of the superstructure

9.5.2.3 Maximum Lateral Displacement

The maximum lateral displacement of base isolation system in the direction under consideration, D_M, is calculated from Equation 9.3:

The maximum displacement of the isolation system, D_M, in the most critical direction of horizontal response shall be calculated using Equation 9.3.

$$D_M = gS_{M1}T_M \frac{1}{4\pi^2 B_M} \quad 9.3$$

where:
S_{M1} = rare earthquake 5 percent damped spectral acceleration parameter (g)
T_M = effective period, in seconds, of the seismically isolated structure at the maximum displacement in the direction under consideration, according to Section 9.5.2.4.
B_M = numerical coefficient related to the effective damping of the isolation system for rare earthquake level (Seismic Hazard Level III according to Chapter 3) as set forth in Table 9.1.

9.5.2.4 Effective Period at Maximum Displacement

The effective period of the isolated structure at maximum displacement, T_M, shall be determined using characteristics of the isolation system and Equation 9.4.

$$T_M = 2\pi \sqrt{\frac{W}{K_{M\,min}\, g}} \quad 9.4$$

where:
$K_{M\,min}$= minimum effective stiffness of the isolation system at the maximum displacement in the horizontal direction under consideration, as prescribed by Section 9.11.4.1.
W = effective seismic weight of the superstructure

9.5.3 Total Lateral Displacement

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, for each device of the isolation system shall include additional displacement due to actual and accidental torsion calculated from the spatial distribution of the lateral stiffness of the isolation system and the most critical location of eccentric mass (See Figure 9.1). D_{TD} and D_{TM} for devices of an isolation system with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by Equations 9.5 and 9.6.

$$D_{TD} = D_D \left[1 + y\left(\frac{12e}{b^2 + d^2}\right)\right] \quad 9.5$$

$$D_{TM} = D_M \left[1 + y\left(\frac{12e}{b^2 + d^2}\right)\right] \quad 9.6$$

where:
b = shortest plan dimension of structure measured perpendicular to d
d = longest plan dimension of structure
e = actual eccentricity measured in plan between the center of mass of the superstructure and the center of rigidity of the isolation system, plus accidental eccentricity, taken as 5% of the longest plan dimension of the structure perpendicular to the direction of force under consideration
y = distance between the centers of rigidity of the isolation system and the element of interest measured perpendicular to the direction of seismic loading under consideration.
D_{TD} and D_{TM} values determined from Equations 9.5 and 9.6, shall not be less than 1.1 times D_D and D_M, respectively.

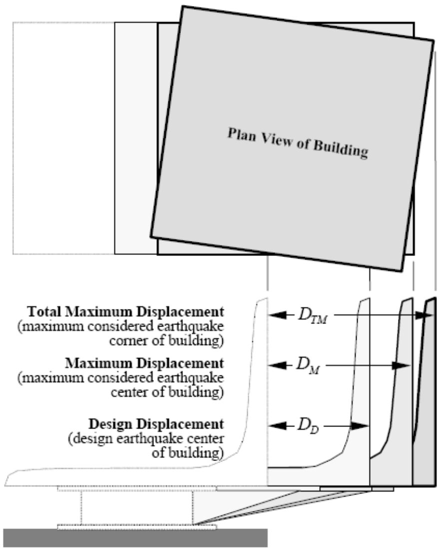


Figure 9.1 Displacement Definition

9.5.4 Lateral Force

Lateral force for structure and isolator are specified in this section.

9.5.4.1 Isolation System and Structural Elements below the Isolation System

The isolation system and substructure system (Figure 9.2) shall be designed and constructed to withstand a lateral seismic force, V_b, calculated by Equation 9.7.

$$V_b = K_{Dmax} D_D \qquad 9.7$$

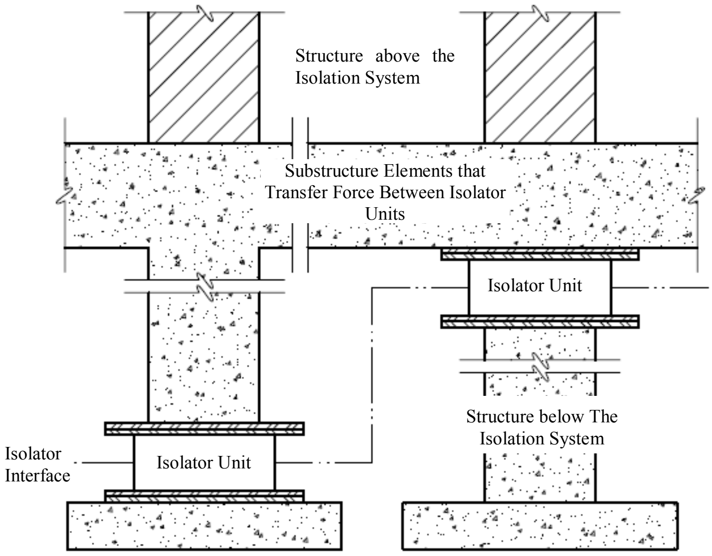


Figure 9.2 Isolator and Connected Structural Elements

where:

K_{Dmax} = maximum effective stiffness of the isolation system at the design displacement under consideration as prescribed by Section 9.11.4.1

D_D = design displacement at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Section 9.5.2.1.

9.5.4.2 Structural Elements above the Isolation System

The superstructure (See Figure 9.2) shall be designed to withstand a minimum shear force, V_s, as prescribed by Equation 9.8.

$$V_s = \frac{K_{D\,max} D_D}{R_I} \tag{9.8}$$

where:
R_I = numerical coefficient equal to $0.375R_u$ where R_u is the response modification factor of non-isolated structure. R_I value shall not be taken greater than 2 and need not be taken less than 1.

9.5.4.3 Limits on Lateral Seismic Force

The value of V_s shall not be taken less than the following:
1. The base shear calculated for a fixed-base structure with the effective seismic weight, W, and a period equal to T_D.
2. The base shear corresponding to the design wind load.
3. The lateral seismic force required to fully activate the isolation system (According to Section 9.11), e.g., the yield level of damping system, the ultimate capacity of a wind-resistant security system, or break-away friction level of a sliding system multiplied by 1.5.

If V_s value is increased according to this article, isolation system should be designed again.

9.5.5 Vertical Distribution of Force

The base shear V_s shall be distributed over the height of the superstructure using Equation 9.9.

$$F_x = V_s \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{9.9}$$

where:
V_s = base shear of superstructure calculated from Section 9.5.4.2
w_x , w_i = seismic effective weight of level x or i
h_x , h_i = height of level x or i calculated from base level
F_x = lateral force at story or level x
n = number of stories or levels

9.5.6 Drift Limit

Lateral displacement at level x, δ_x, is determined from Equation 9.10.

$$\delta_x = R_I \delta_{xe} \tag{9.10}$$

where:
δ_{xe}= elastic lateral displacement at level x
After calculation of δ_x, story drift is calculated from Equation 4.23. This value shall not exceed $0.015h_{sx}$, where h_{sx} is the height of the story x.

9.6 Dynamic Analysis Procedures

Dynamic analysis of seismically isolated structures is performed according to the requirements of this section.

9.6.1 Modeling

The isolation system and the superstructures are modeled according to this section and Chapter 4 modeling provisions such as stiffness reduction for concrete members, panel zone effect, diaphragm rigidity, $P - \Delta$ effect, etc.

9.6.1.1 Isolation System

The isolation system shall be modeled using force-deformation characteristics developed and verified by test in accordance with the requirements of Section 9.11. The isolation system shall be modeled with sufficient detail to:

1. Account for the spatial distribution of isolator units.
2. Calculate translation, in both horizontal directions, and torsion of the structure above the isolation interface considering the most disadvantageous location of eccentric mass of superstructure.
3. Assess overturning/uplift forces on individual isolator units.
4. Account for the effects of vertical load, bilateral load, and/or the rate of loading if the force-deflection properties of the isolation system are dependent on one or more of these attributes.

The total design displacement and total maximum displacement across the isolation system shall be calculated using a model of the isolated structure that incorporates the force-deflection characteristics of nonlinear elements of the isolation system and the seismic force-resisting system of superstructure.

9.6.1.2 Superstructure

The maximum displacement of each floor and design forces and displacements in elements of the seismic force-resisting system of superstructure are permitted to be calculated using a linear elastic model of the isolated structure if both of the following conditions are met:

1. Stiffness properties assumed for the inelastic components of the isolation system are based on the maximum effective stiffness of the isolation system.
2. All elements of the seismic force-resisting system of remain elastic for the design earthquake.

9.6.2 Dynamic Procedures

Dynamic analysis is performed by one of the Modal Response Spectrum Analysis or Seismic Response History Procedures according to Sections 4.9, 4.10 and this section.

9.6.2.1 Input Earthquake

The design earthquake ground motions (Seismic Hazard Level II according to Chapter 3) shall be used to calculate the total design displacement of the isolation system and the lateral forces and displacements in the superstructure. The maximum considered earthquake (Seismic Hazard Level III) shall be used to calculate the total maximum displacement of the isolation system.

9.6.2.2 Modal Response Spectrum Analysis

This procedure is performed according to Section 9.4. Effective damping ratio for the fundamental mode in the direction of interest, shall not be greater than the effective damping of the isolation system or 30 percent of critical, whichever is less. Modal damping ratios for higher modes shall be selected consistent with those that would be appropriate for response-spectrum analysis of the superstructure system assuming a fixed base. Modal Response Spectrum Analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the ground motion in the critical direction and 30 percent of the ground motion in the perpendicular, horizontal direction. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements. The design shear at any story shall not be less than the story shear resulting from application of the story forces calculated by Equation 9.9 and a value of V_S equal to the base shear obtained from the response-spectrum analysis in the direction of interest.

9.6.2.3 Seismic Response History Procedure

This procedure is performed according to Section 4.10. Total maximum displacement of isolator system (Section 9.5.3.2) is calculated as the vectorial sum of the two orthogonal displacements.

9.6.3 Minimum Lateral Displacements and Design Forces

9.6.3.1 Isolation System and Substructure

The isolation system, foundation, and all structural elements below the isolation system shall be designed using seismic lateral force without considering R_I coefficient. The design lateral force shall not be taken as less than $0.9V_b$.
The total design displacement and total maximum displacement of the isolation system shall not be taken as less than $0.9D_{TD}$ and $0.8D_{TM}$ as prescribed by Section 9.5.3. For calculating the values of D_{TD} and D_{TM} , it is noted that D_D' shall be used in lieu of D_D and D_M' shall be used in lieu of D_M as prescribed in Equations 9.11 and 9.12.

$$D_D' = \frac{D_D}{\sqrt{1+\left(\frac{T}{T_D}\right)^2}} \quad 9.11$$

$$D_M' = \frac{D_M}{\sqrt{1+\left(\frac{T}{T_M}\right)^2}} \quad 9.12$$

where:
T = Fixed-base period of the superstructure

9.6.3.2 Superstructure

The superstructure system shall be designed with V_s distributed over the height of the superstructure according to Section 9.5.5. Where the Seismic Response History Procedure is used, the design base shear on the superstructure, if irregular in configuration, shall not be taken less than $0.8V_s$ and if regular in configuration, shall not be taken less than $0.6V_s$. Where Modal Response Spectrum Analysis is used, minimum base shear, if regular in configuration, shall not be taken less than $0.8V_s$ and if regular in configuration, shall not be taken less than V_s.

9.6.3.3 Scaling of Results

Where base shear determined using either Modal Response Spectrum Analysis or Seismic Response History Procedure, is less than the minimum values prescribed by Sections 9.6.3.1 and 9.6.3.2, all response parameters, including member forces and moments, shall be adjusted upward proportionally.

9.6.3.4 Lateral Drifts and Design Forces

Maximum story drift corresponding to the design lateral force including displacement due to vertical deformation of the isolation system shall not exceed the following limits:

1. The maximum story drift of the superstructure calculated by Modal Response Spectrum Analysis shall not exceed $0.015h_{sx}$
2. The maximum story drift of the structure above the isolation system calculated by Nonlinear Seismic Response History Procedure (based on the force-deflection characteristics of nonlinear elements of the seismic force-resisting system) shall not exceed $0.020h_{sx}$. Displacement shall be calculated with the C_d of the isolated structure being equal to R_I as defined in Equation 9.10. The $P-\Delta$ effects resulted from lateral displacement of the superstructure in the maximum considered earthquake shall be investigated if the story drift ratio exceeds $0.010/R_I$.

The specifications and limitations for displacement of isolated structures with respect to analysis procedure are shown in Table 9.2.

Table 9.2 Seismic Design Parameters

Design Parameter	Equivalent Lateral Load Procedure	Dynamic Procedure	
		Modal Response Spectrum	Seismic Response History
Design Lateral Displacement, D_D	$D_D = (g/4\pi^2)(S_{D1}T_1T_D/B_D)$	-	-
Total Lateral Displacement, D_{TD}	$D_{TD} \geq 1.1D_D$	$\geq 0.9D_{TD}$	$\geq 0.9D_{TD}$
Maximum Lateral Displacement, D_M	$D_M = (g/4\pi^2)(S_{M1}T_1T_M/B_M)$	-	-
Total Maximum Lateral Displacement, D_{TM}	$D_{TM} \geq 1.1D_M$	$\geq 0.8D_{TM}$	$\geq 0.8D_{TM}$
Design Shear of Substructure, V_b	$V_b = K_{Dmax}D_D$	$\geq 0.9V_b$	$\geq 0.9V_b$
Design Shear for Regular Structure, V_s	$V_s = K_{Dmax}D_D/R_I$	$\geq 0.8V_s$	$\geq 0.6V_s$
Design Shear for Irregular Structure, V_s	$V_s = K_{Dmax}D_D/R_I$	$\geq 1.0V_s$	$\geq 0.8V_s$
Lateral Drift (Use R_I instead of C_d)	$0.015h_{sx}$	$0.015h_{sx}$	$0.020h_{sx}$

9.7 Miscellaneous Specifications

The isolation system and superstructure shall satisfy the requirements of this section.

9.7.1 Isolation System

9.7.1.1 Environmental Conditions

In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall provide for other environmental conditions including aging effects, creep, fatigue, operating temperature, and exposure to moisture or damaging substances.

9.7.1.2 Wind Force

Isolated structures shall resist design wind loads at all levels of superstructure. At the isolation interface, a suitable system shall be provided to limit lateral displacement of the isolation system to a value equal to permitted drift values presented in Section 9.5.6.

9.7.1.3 Fire Resistance

Fire resistance for the isolation system shall meet the same requirements for the main structural members such as columns, walls, or others such as gravity-bearing elements in the same region of the structure.

9.7.1.4 Lateral Restoring Force

The isolation system shall be configured to produce a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50% of the total design displacement.

9.7.1.5 Displacement Restraint

Distance between isolated structure and displacement restraint shall not be less than total maximum design displacement, D_{TM}, unless the seismically isolated structure is designed in accordance with the following criteria:

1. Maximum considered earthquake response is calculated in accordance with the dynamic analysis requirements, according to Section 9.6 explicitly considering the nonlinear characteristics of the isolation system and the superstructure.
2. The ultimate capacity of the isolation system and substructure element shall exceed the strength and displacement demands of the maximum considered earthquake.
3. The superstructure is checked for stability and ductility demand of the maximum considered earthquake.

4. The displacement restraint does not become effective at a displacement less than 0.75 times the total design displacement unless it is demonstrated by analysis that earlier engagement does not result in unsatisfactory performance.

9.7.1.6 Vertical-Load Stability

Each element of the isolation system shall be designed to be stable under the design vertical load when subjected to a horizontal displacement equal to the total maximum displacement. The design vertical load shall be computed using load combination of Equation 2.13 for the maximum vertical load and load combination of Equation 2.29 for the minimum vertical load where S_{DS} in these equations is replaced by S_{MS}.

9.7.1.7 Overturning

The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations of maximum earthquake. Local uplift of individual elements shall not be allowed unless the resulting deflections do not cause overstress or instability of the isolator units or other structure elements.

9.7.1.8 Inspection and Replacement

Access for inspection and replacement of all components of the isolation system shall be provided by a registered design professional team.

9.7.1.9 Quality Control

A quality control testing program for isolator units shall be established by the registered design professional, responsible for the structural design.

9.7.2 Structural System

9.7.2.1 Horizontal Distribution of Force

A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces (due to non-uniform ground motion) from one part of the structure to another.

9.7.2.2 Building Separations

Minimum separations between the isolated structure and surrounding retaining walls or other fixed obstructions shall not be less than the total maximum displacement corresponding to Section 9.5.3.

9.7.2.3 Non-Building Structures

Non-building structures shall be designed with design displacements and forces calculated in accordance with Section 9.5 or 9.6 and Chapter 7.

9.8 Structural and Non-Structural Elements

Structural and non-structural elements shall be designed based on this section and the other specified chapters.

9.8.1 Superstructure Elements

Elements of the superstructure system shall be designed using forces and displacements presented in previous sections.

9.8.2 Elements of Isolation Interface

Elements of the isolation interface shall be resistant to the total maximum displacement, D_{TM}.

9.8.3 Elements below the Isolation System

These elements shall be designed according to Chapters 4, 7 and 8.

9.9 Foundation

The foundation shall be designed using design forces calculated in accordance with Section 9.5 or 9.6.

9.10 Design Review

A design review of the isolation system and the related test program shall be performed by an independent engineering team including persons licensed in the appropriate disciplines and experienced in seismic analysis methods and the theory and application of seismic isolation. Reviewing the isolation system design shall include, but not be limited to, the following:

1. Review of the site-specific seismic criteria including development of the site-specific spectra and ground motion histories and all other design criteria developed specifically for the project.
2. Review of the preliminary design including the determination of the total design displacement, the total maximum displacement, and the lateral force
3. Overview and observation of prototype testing (Section 9.11).
4. Review of the final design of the entire structural system and all supporting analyses.
5. Review of the isolation system quality control testing program (Section 9.7.1.9).

9.11 Testing

The deformation characteristics and damping ratio values of the isolation system used in the design and analysis of seismically isolated structures shall be based on this section. The isolation system components to be tested shall include the wind-restraint system if such a system is used in the design.

9.11.1 Prototype Tests

Prototype tests shall be performed separately on two full-size specimens (or sets of specimens, as appropriate) of each predominant type and size of isolator unit of the isolation system.

9.11.1.1 Recording

For each cycle of each test, the force-deflection and hysteretic behavior of the test specimen shall be recorded.

9.11.1.2 Loading Sequences and Cycles

The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the average dead load plus one-half of the effects due to live load on all isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force, if needed.
2. Three fully reversed cycles of loading at each of the following increments of the total design displacement: $0.25D_D$, $0.5D_D$, D_D, D_M
3. Three fully reversed cycles of loading at the total maximum displacement, D_{TM}.
4. $n = 30S_{D1}/S_{DS}B_D$, but not less than 10, fully reversed cycles of loading at 1.0 times the total design displacement, D_{TD}

If an isolator unit is also a vertical-load-carrying element, then item 2 of the sequence of cyclic tests specified in the preceding text shall be performed for two additional vertical load cases specified in load combinations of Equations 2.13 and 2.29. The load increment due to earthquake overturning, E or 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 taken as the typical or average downward force on all isolator units of a common type and size.

9.11.1.3 Units Dependent on Loading Rates

The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if the measured property (effective stiffness or effective damping) at the design displacement when tested at any frequency in the range of 0.1 to 2.0 times the inverse of T_D is different from the property when tested at a frequency equal to the inverse of T_D by more than 15 percent.

If the force-deflection properties of the isolator units are dependent on the rate of loading, each set of tests specified in Section 9.11.1.2 shall be performed dynamically at a frequency equal to the inverse of the effective period, T_D. If reduced-scale prototype specimens are used to quantify rate-dependent properties of isolators, the reduced-scale prototype specimens shall be of the same type and material and be manufactured with the same processes and quality as full-scale prototypes and shall be tested at a frequency that represents full-scale prototype loading rates.

9.11.1.4 Units Dependent on Bilateral Load

The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load if the effective stiffness where subjected to bilateral loading is different from the effective stiffness where subjected to unilateral loading, by more than 15%. If the force-deflection properties of the isolator units are dependent on bilateral load, the tests shall be augmented to include bilateral load at the following increments of the total design displacement, D_{TD}: 1:0.25, 1.0:0.5, 1.0:0.75 and 1:1.

9.11.1.5 Maximum and Minimum Vertical Load

Isolator units that carry vertical load shall be statically tested for maximum and minimum downward vertical load at the total maximum displacement. In these tests, the combined maximum and minimum vertical loads shall be taken as $(1.2 + 0.2S_{MS})D + \rho Q_E + L + 0.2S$ and $(0.9 - 0.2S_{MS})D + \rho QE + 1.6H$, respect to any one isolator of a common type and size. Vertical component of earthquake, Q_E, shall be based on the peak response due to the maximum considered earthquake.

9.11.1.6 Sacrificial Wind-Restraint System

If a sacrificial wind-restraint system is to be utilized, its ultimate capacity shall be established by test.

9.11.1.7 Testing Similar Units

Prototype tests are not required if an isolator unit is of similar size and of the same type and material as a prototype isolator unit that has been previously tested using the specified sequence of tests.

9.11.2 Determination of Force-Deflection Characteristics

The force-deflection characteristics of the isolation system shall be based on the cyclic load tests of prototype isolator specified in Section 9.11.1. As required, the effective stiffness of an isolator unit, K_{eff}, shall be calculated for each cycle of loading as prescribed by Equation 9.13.

$$K_{eff} = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|} \tag{9.13}$$

where:
F^+ and F^- = positive and negative forces at Δ^+ and Δ^-, respectively
Δ^+ and Δ^- = maximum positive and negative displacements, respectively, of isolator in each full-scale test cycle
As required, the effective damping, β_{eff}, of an isolator unit shall be calculated for each cycle of loading by Equation 9.14.

$$\beta_{eff} = \frac{2}{\pi} \frac{E_{loop}}{K_{eff}(|\Delta^+| + |\Delta^-|)^2} \tag{9.14}$$

where:
E_{loop} = the energy dissipated per cycle of loading

9.11.3 Test Specimen Adequacy

The performance of the test specimens shall be deemed adequate if the following conditions are satisfied:

1. The force-deflection plots for all tests specified in Section 9.11.1 have a positive incremental force-resisting capacity.
2. For each increment of test displacement specified in item 2 of Section 9.11.1.2 and for each vertical load case specified in the same section:
   A. For each test specimen, the difference between effective stiffness at each of the three cycles of test and average value of effective stiffness is no greater than 15%.
   B. For each cycle of test, the difference between effective stiffness of the two test specimens of a common type and size of the isolator unit (Section 9.11.1) and the average effective stiffness is no greater than 15%.
3. For each specimen there is no greater than a 20% change in the initial effective stiffness over the cycles of test specified in item 4 of Section 9.11.1.2.
4. For each specimen there is no greater than a 20% decrease in the initial effective damping over the cycles of test specified in item 4 of Section 9.11.1.2.
5. All specimens of vertical-load-carrying elements of the isolation system remain stable where tested in accordance with Section 9.11.1.5.

9.11.4 Design Properties of the Isolation System

9.11.4.1 Maximum and Minimum Effective Stiffness

At the design displacement, the maximum and minimum effective stiffness of the isolated system, K_{Dmax} and K_{Dmin}, shall be based on the cyclic tests of item 2 of Section 9.11.1.2 (See Figure 9.3) and calculated using Equation 9.15 and 9.16.

$$K_{Dmax} = \frac{\sum|F_D^+|_{max} + \sum|F_D^-|_{max}}{2D_D} \qquad 9.15$$

$$K_{Dmin} = \frac{\sum|F_D^+|_{min} + \sum|F_D^-|_{min}}{2D_D} \qquad 9.16$$

where:
$\sum|F_D^+|_{max}$= summation of absolute maximum isolator forces corresponding to positive D_D
$\sum|F_D^-|_{max}$= summation of absolute maximum isolator forces corresponding to negative D_D
$\sum|F_D^+|_{min}$= summation of absolute minimum isolator forces corresponding to positive D_D
$\sum|F_D^-|_{min}$= summation of absolute minimum isolator forces corresponding to negative D_D

At the maximum displacement, the maximum and minimum effective stiffness of the isolation system, k_{Mmax} and k_{Mmin}, shall be based on the cyclic tests of item 3 of Section 9.8.1.2 and calculated using Equations 9.17 and 9.18.

$$K_{Mmax} = \frac{\sum|F_M^+|_{max} + \sum|F_M^-|_{max}}{2D_M} \qquad 9.17$$

$$K_{Mmin} = \frac{\sum|F_M^+|_{min} + \sum|F_M^-|_{min}}{2D_M} \qquad 9.18$$

where:
$\sum|F_M^+|_{max}$= summation of absolute maximum isolator forces corresponding to positive D_M
$\sum|F_M^-|_{max}$= summation of absolute maximum isolator forces corresponding to negative D_M
$\sum|F_M^+|_{min}$= summation of absolute minimum isolator forces corresponding to positive D_M
$\sum|F_M^-|_{min}$= summation of absolute minimum isolator forces corresponding to negative D_M

9.11.4.2 Effective Damping Ratio

At the design displacement, the effective damping of the isolation system, β_D, shall be based on the cyclic tests of item 2 of Section 9.11.1.2 and calculated using Equation 9.19.

$$\beta_D = \frac{1}{2\pi}\left[\frac{\sum E_D}{K_{Dmax} D_D^2}\right] \tag{9.19}$$

where:
$\sum E_D$ = total energy dissipated per cycle of design displacement response. This value shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at a test displacement equal to D_D and shall be based on forces and deflections from the cycle of prototype testing at test displacement D_D that produces the smallest values of effective damping ratio.
At the maximum displacement, the effective damping ratio of the isolation system, β_M, shall be based on the cyclic tests of item 2 of Section 9.11.1.2 and calculated using Equation 9.20.

$$\beta_M = \frac{1}{2\pi}\left[\frac{\sum E_M}{K_{Mmax} D_M^2}\right] \tag{9.20}$$

where:
$\sum E_M$ = total energy dissipated per cycle in all isolator units measured at a test displacement equal to D_M. This value shall be calculated from sum of the energy values dissipated per cycle of design displacement response and shall be based on forces and deflections from the cycle of prototype testing that produces the smallest value of effective damping ratio.

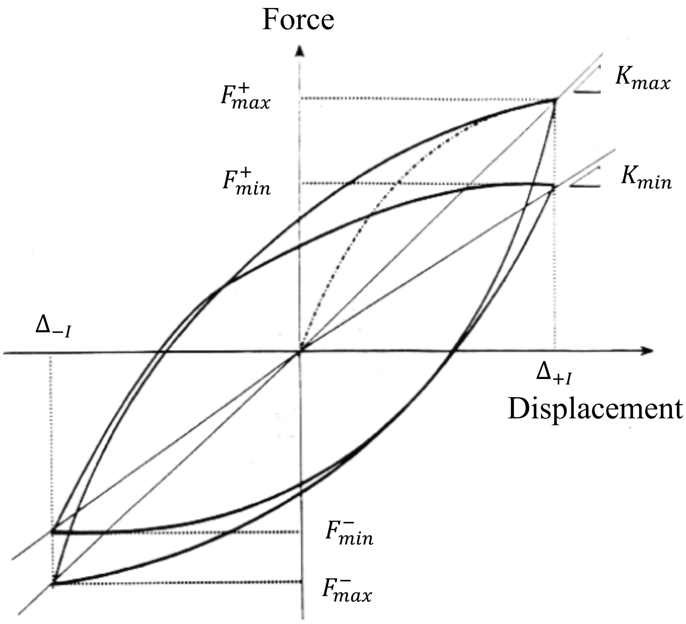


Figure 9.3 Stiffness Effects on Cyclic Behavior of Isolators

Chapter 10

Structures with Damping Systems

10. Structures with Damping Systems

10.1 General

10.1.1 Scope

Every structure with a damping system and every portion thereof shall be designed and constructed in accordance with the requirements of this code as modified by this chapter. Where damping devices are used across the isolation interface of a seismically isolated structure, displacements, velocities, and accelerations shall be determined in accordance with Chapter 9.

10.1.2 Definitions

Damping Device:

A flexible structural element of the damping system that dissipates energy due to relative motion of each end of the device. Damping devices include all pins, bolts, gusset plates, brace extensions, and other components required to connect damping devices to the other elements of the structure. Damping devices may be classified as either displacement-dependent or velocity-dependent, or a combination thereof, and may be configured to act in either a linear or a nonlinear manner.

Damping System:

The collection of structural elements that includes all the individual damping devices, all structural elements or bracing required to transfer forces 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 substantially independent of the relative velocity between each of the devices and/or the excitation frequency.

Velocity Dependent Damping Device:

The force-displacement relation for a velocity dependent damping device is primarily a function of the relative velocity between each end of the device and can be a function of the relative displacement between them.

10.2 General Design Requirements

10.2.1 System Requirements

Design of the structure shall consider the basic 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 meet the forces defined in Section 10.2.1.1. Combination of the seismic force-resisting system and the damping system is permitted to be used to meet the drift requirement.

10.2.1.1 Seismic Resisting System

Structures that contain a damping system are required to have a seismic force-resisting system that, in each lateral direction, conforms to one of the types indicated in Table 4.4.
Design of the seismic force-resisting system in each direction shall satisfy the requirements of Section 10.7 and the following:

A. The seismic base shear used for design of the seismic force-resisting system shall not be less than V_{min}, where V_{min} is determined as the greater of the values computed using Equations 10.1 and 10.2:

$$V_{min} = \frac{V_u}{B_{V+I}} \qquad 10.1$$

$$V_{min} = 0.75\,V_u \qquad 10.2$$

where:
V_u = seismic base shear in the direction of interest, determined in accordance with Section 4.8.
B_{V+I}= numerical coefficient as set forth in Table 10.1 for effective damping equal to the sum of viscous damping in the fundamental mode of vibration of the structure $(m = 1)$ in the direction of interest, B_{Vm}, plus inherent damping, β_1.
The fundamental mode of vibration (shown as $m = 1$ in this Chapter) is the mode that the maximum mass participation is occurred in the direction of interest and that is not necessarily equals to maximum period. Although for normal structures usually the fundamental mode is corresponding to the maximum period.
Exception: The seismic base shear used for design of the seismic force-resisting system shall be taken as $1.0V_u$, if either of the following conditions apply:
1. In the direction of interest, the damping system has less than two damping devices on each floor level, configured to resist torsion
2. The seismic force-resisting system has horizontal irregularity Type b (Table 4.1) or vertical irregularity Type b (Table 4.2).

B. Minimum strength requirements for elements of the seismic force-resisting system that are also elements of the damping system or are otherwise required to resist forces from damping devices shall meet the additional requirements of Section 10.7.2.

10.2.1.2 Damping System

Elements of the damping system shall be designed to remain elastic for design loads including unreduced seismic forces of damping devices as required in Section 10.7.2.1, unless it is shown by analysis or test that inelastic response of elements would not adversely affect damping system function and inelastic response is limited in accordance with the requirements of Section 10.2.7.6.

Table 10.1 Damping Coefficient B_{1M} , B_{1E}, B_{1D}, B_{mM} , B_{mD} , B_R, B_{V+I}
(Where Period of the Structure ≥ T_0)

Effective Damping Ratio, β (Percent of Critical Damping)	Damping Coefficient
≤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

10.2.2 Ground Motion

10.2.2.1 Design Spectra

Spectra for the design earthquake ground motions (Seismic Hazard Level II) and maximum considered earthquake ground motions (Seismic Hazard Level III) developed in accordance with Section 3.4 shall be used for the design and analysis of a structure with a damping system. Site-specific design spectra shall be developed and used for design of a structure with a damping system if either of the following conditions apply:

A. The structure is located on a Site Class IV (based on Reference [2]).
B. The structure is located at a site with S_1(g) greater than or equal to 0.6 where S_1 is the spectral acceleration parameter at a period of 1.0 second located on the rock bed in rare earthquake (Two percent probability of exceedance in 50 years corresponding to Seismic Hazard Level III).

10.2.2.2 Ground Motion Histories

Ground motion histories for the design and the rare earthquake shall be used for design and analysis of all structures with a damping system if either of the following conditions apply:

A. The structure is located at a site with S_1 greater than or equal to 0.6.
B. The damping system is explicitly modeled.

The ground motion for Seismic Response History Procedure is selected in accordance with Section 4.10 considering fundamental period range from $0.5T_{1D}$ to $1.25T_{1M}$ in lieu of $0.2T$ to $1.5T$. T_{1D} and T_{1M} are the fundamental period of the structure in design and maximum displacements respectively. These values are determined for Modal Response Spectrum and Equivalent Lateral Load Procedures according to Section 10.4.2.5 or 10.5.2.5 respectively.

10.2.3 Procedure Selection

A structure with a damping system shall be designed using linear procedures, nonlinear procedures, or a combination of linear and nonlinear procedures, as permitted in this section.

Regardless of the analysis method used, the peak dynamic response of the structure and elements of the damping system shall be confirmed by using the Nonlinear Response History Procedure if the structure is located at a site with S_1 greater than or equal to 0.6.

10.2.3.1 Nonlinear Procedures

The nonlinear procedures of Section 10.3 are permitted to be used for design of all structures with damping systems.

10.2.3.2 Modal Response Spectrum Procedure

The response spectrum procedure of Section 10.4 is permitted to be used for design of a structure with a damping system provided that:

A. In the direction of interest, the damping system has at least two damping devices in each story, configured to resist torsion.
B. The total effective damping of the fundamental mode, $\beta_{mD}(m = 1)$, of the structure in the direction of interest is not greater than 35 percent of critical.

10.2.3.3 Equivalent Lateral Load Procedure

The Equivalent Lateral Load Procedure of Section 10.5 is permitted to be used for design of a structure with a damping system provided that:

A. In the direction of interest, the damping system has at least two damping devices in each story, configured to resist torsion.
B. The total effective damping of the fundamental mode, $\beta_{mD}(m = 1)$, of the structure in the direction of interest is not greater than 35 percent of critical.
C. The seismic force-resisting system does not have horizontal irregularity Type a or b, (Table 4.2) or vertical irregularity Type a, b, c, or d (Table 4.1).
D. Floor diaphragms are rigid as defined in Section 4.11.
E. The height of the structure above the base 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 rare earthquake ground motions and consideration of the following:

A. Low cycle, large displacement degradation due to seismic loads.
B. High cycle, small displacement degradation due to wind, thermal, or other cyclic loads.
C. Forces or displacements due to gravity loads.
D. Adhesion of device parts due to corrosion or abrasion, biodegradation, moisture, or chemical exposure.
E. Exposure to environmental conditions, including, but not limited to, temperature, humidity, moisture, radiation (e.g., ultraviolet light), and reactive or corrosive substances (e.g., salt water).

Damping devices subject to failure by low cycle fatigue shall resist wind forces without slip, movement, or inelastic cycling.

The design of damping devices shall incorporate the range of thermal conditions, device wear, manufacturing tolerances, and other effects that cause device properties to vary during the design life of the device.

10.2.4.2 Multi-Axis Movement

Connection points of damping devices shall provide sufficient articulation to accommodate simultaneous longitudinal, lateral, and vertical displacements of the damping system.

10.2.4.3 Inspection and Periodic Testing

Means of access for inspection and removal of all damping devices shall be provided. The registered design professional responsible for design of the structure shall establish an appropriate inspection and testing schedule for each type of damping device to ensure that the devices respond in a dependable manner throughout their design life. The degree of inspection and testing shall reflect the established in-service history of the damping devices and the likelihood of change in properties over the design life of the devices.

10.2.4.4 Quality Control

As part of the quality assurance plan developed in accordance with reliable references, the registered design professional responsible for the structural design shall establish a quality control plan for the manufacture of damping devices. As a minimum, this plan shall include the testing requirements of Section 10.9.

10.3 Nonlinear Procedures

The stiffness and damping properties of the damping devices used in the models shall be based on or verified by testing of the damping devices as specified in Section 10.9. The nonlinear force deflection characteristics of damping devices shall be modeled, as required, to explicitly account for device dependence on frequency, amplitude, and duration of seismic loading.

10.3.1 Nonlinear Response History Procedure

A Nonlinear Response History Procedure shall utilize a mathematical model of the structure and the damping system as provided in Section 4.10.3 and this section. The model shall directly account for the nonlinear hysteretic behavior of elements of the structure and the damping devices to determine its response.

Inherent damping of the structure shall not be taken as greater than 5 percent of critical unless test data consistent with levels of deformation at or just below the effective yield displacement of the seismic force-resisting system support higher values.

If the calculated force in an element of the seismic force-resisting system does not exceed 1.5 times its nominal strength, that element is permitted to be modeled as linear.

10.3.1.1 Damping Device Modeling

Mathematical models of displacement dependent damping devices shall include the hysteretic behavior of the devices consistent with test data and accounting for all significant changes in strength, stiffness, and hysteretic loop shape. Mathematical models of velocity dependent damping devices shall include the velocity coefficient consistent with test data. If this coefficient changes with time and/or temperature, such behavior shall be modeled explicitly. The elements of damping devices connecting damper units to the structure shall be included in the model.

Exception: If the properties of the damping devices are expected to change during the duration of the time history analysis, the dynamic response is permitted to be enveloped by the upper and lower limits of device properties. All these limit cases for variable device properties must satisfy the same conditions as if the time dependent behavior of the devices were explicitly modeled.

10.3.1.2 Response Parameters

In addition to the response parameters given in Section 4.10.3.1, for each ground motion used for response history analysis, individual response parameters consisting of the maximum value of the discrete damping device forces, displacements, and velocities, in the case of velocity dependent devices, shall be determined. If at least seven pairs of ground motions are used for response history analysis, the design values of the damping device forces, displacements, and velocities are permitted to be taken as the average of the values determined by the analyses. If less than seven pairs of ground motions are used for response history analysis, the design damping device forces, displacements, and velocities shall be taken as the maximum value determined by the analyses. A minimum of three pairs of ground motions shall be used.

10.3.2 Nonlinear Static Procedure

The nonlinear modeling described in Section 4.10.3 and the lateral loads described in Section 4.8 shall be applied to the seismic force-resisting system. The resulting force displacement curve shall be used in lieu of the assumed effective yield displacement, D_Y, of Equation 10.56 to calculate the effective ductility demand due to the design earthquake ground motions, μ_D, and due to the maximum considered earthquake ground motions, μ_M, in Equations 10.54 and 10.55, respectively. The value of (R_u/C_d) shall be taken as 1.0 in Equations 10.6, 10.7, 10.10, and 10.11 for the Modal Response Spectrum Procedure, and in Equations 10.25, 10.26, and 10.34 for the Equivalent Lateral Load Procedure.

10.4 Modal Response Spectrum Procedure

Where the Modal Response Spectrum Procedure is used to analyze a structure with a damping system, the requirements of this section shall apply.

10.4.1 Modeling

A mathematical model of the seismic force-resisting system and damping system shall be constructed that represents the spatial distribution of mass, stiffness, and damping throughout the structure. The model and analysis shall comply with the requirements of Section 4.9 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 models shall be based on or verified by testing of the damping devices as specified in Section 10.9. The elastic stiffness of elements of the damping system other than damping devices shall be explicitly modeled. Stiffness of damping devices shall be modeled depending on damping device type as follows:

A. <u>Displacement dependent damping devices:</u>

Displacement dependent damping devices shall be modeled with an effective stiffness that represents damping device force at the response displacement of interest (e.g., design story drift). Alternatively, the stiffness of hysteretic and friction damping devices is permitted to be excluded from response spectrum analysis provided design forces in displacement dependent damping devices, Q_{DSD}, are applied to the model as external loads (Section 10.7.2.5).

B. <u>Velocity dependent damping devices:</u>

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.

10.4.2 Seismic Force-Resisting System

10.4.2.1 Seismic Base Shear

The seismic base shear, V_u, of the structure in a given direction shall be determined as the combination of modal components, V_m, subject to the limits of Equation 10.3 and the magnitude is determined based on Section 10.2.1.1:

$$V_u \geq V_{min} \tag{10.3}$$

The seismic base shear, V_u, of the structure shall be determined by the Square Root of the Sum of the Squares (SRSS) or Complete Quadratic Combination (CQC) of modal base shear components, V_m.

10.4.2.2 Modal Base Shear

Modal base shear of the m^{th} mode of vibration, V_m, of the structure in the direction of interest shall be determined in accordance with Equations 10.4:

$$V_m = C_{sm} W_m \tag{10.4a}$$

$$W_m = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im}\right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \tag{10.4b}$$

where

C_{sm} = seismic response coefficient of the m^{th} mode of vibration of the structure in the direction of interest as determined from Section 10.4.2.4 $(m = 1)$ or Section 10.4.2.6 $(m > 1)$

W_m= effective seismic weight of the m^{th} mode of vibration of the structure

w_i = the portion of the total effective seismic weight of the structure W, located or assigned to level i

ϕ_{im} = modal shape amplitude at the i^{th} level of the structure in the m^{th} mode of vibration in the direction of interest

n = Number of vibration modes considered to cover 90% of effective seismic weight

10.4.2.3 Modal Participation Factor

The modal participation factor of the m^{th} mode of vibration, Γ_m, of the structure in the direction of interest shall be determined in accordance with Equation 10.5:

$$\Gamma_m = \frac{\overline{W}_m}{\sum_{i=1}^{n} w_i \phi_{im}} \tag{10.5}$$

10.4.2.4 Fundamental Mode Seismic Response Coefficient

The fundamental mode $(m = 1)$ seismic response coefficient, C_{s1}, in the direction of interest shall be determined in accordance with Equations 10.6 and 10.7:

$$C_{s1} = \left(\frac{R_u}{C_d}\right) \frac{S_{DS}}{\Omega_0 B_{1D}} \qquad for \quad T_{1D} < T_s \tag{10.6}$$

$$C_{s1} = \left(\frac{R_u}{C_d}\right) \frac{S_{D1}}{T_{1D}(\Omega_0 B_{1D})} \quad for \quad T_{1D} \geq T_s \tag{10.7}$$

where

T_s = period defined in Equation 3.12

R_u, Ω_0, C_d = structure coefficients as set forth in Table 4.4

B_{1D} = numerical coefficient as set forth in Table 10.1 for effective damping equal to β_{ml} $(m = 1)$ and period of structure equal to T_{1D}
S_{DS}, S_{D1} = spectral acceleration parameter as defined in Chapter 3

10.4.2.5 Effective Fundamental Mode Period Determination

The effective fundamental mode $(m = 1)$ period at the design earthquake (Seismic Hazard Level II), T_{1D}, and at rare earthquake (Seismic Hazard Level III), T_{1M}, shall be based on either explicit consideration of the post yield force deflection characteristics of the structure or determined in accordance with Equations 10.8 and 10.9 considering fundamental period for structure, T_1:

$$T_{1D} = T_1\sqrt{\mu_D} \qquad 10.8$$

$$T_{1M} = T_1\sqrt{\mu_M} \qquad 10.9$$

10.4.2.6 Higher Mode Seismic Response Coefficient

Higher mode seismic response coefficient, C_{sm}, of the m^{th} mode of vibration $(m > 1)$ of the structure in the direction of interest shall be determined in accordance with Equations 10.10 and 10.11:

$$C_{sm} = \left(\frac{R_u}{C_d}\right)\frac{S_{DS}}{\Omega_0 B_{mD}} \qquad for \quad T_m < T_s \qquad 10.10$$

$$C_{sm} = \left(\frac{R_u}{C_d}\right)\frac{S_{D1}}{T_m(\Omega_0 B_{mD})} \qquad for \quad T_m \geq T_s \qquad 10.11$$

where
T_m = period of the m^{th} mode of vibration of the structure in the direction under consideration
B_{mD} = numerical coefficient as set forth in Table 10.1 for effective damping equal to β_{mD} and fundamental period of the structure equal to T_m

10.4.2.7 Design Lateral Force

Design lateral force at Level i due to the m^{th} mode of vibration, F_{im}, of the structure in the direction of interest shall be determined in accordance with Equation 10.12:

$$F_{im} = w_i \phi_{im} \frac{\Gamma_m}{\overline{W}_m} V_m \qquad 10.12$$

Design forces in elements of the seismic force-resisting system shall be determined by the SRSS or CQC of modal design forces.

10.4.3 Damping System

Design forces in damping devices and other elements of the damping system shall be determined on the basis of the floor deflection, story drift, and story velocity response parameters described in the following sections.
Displacements and velocities used to determine 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 calculated for both the design earthquake ground motions and rare earthquake ground motions, respectively, in accordance with the following sections.

10.4.3.1 Design Earthquake Floor Deflection

The deflection of structure due to the design earthquake ground motions at Level i in the m^{th} mode of vibration, δ_{imD}, of the structure in the direction of interest shall be determined in accordance with Equation 10.13:

$$\delta_{imD} = D_{mD}\phi_{im} \qquad 10.13$$

where

D_{mD} = displacement of the roof level of the m^{th} mode of vibration under consideration Section 10.4.3.2

The total design deflection at each floor of the structure shall be calculated by the SRSS or complete quadratic combination (CQC) of modal design earthquake deflections.

10.4.3.2 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 of interest shall be determined in accordance with Equations 10.14 and to 10.15:

For $m = 1$:

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1\frac{S_{DS}T_{1D}^2}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1\frac{S_{DS}T_1^2}{B_{1E}} \quad , \quad T_{1D} < T_s \qquad 10.14a$$

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1\frac{S_{D1}T_{1D}}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1\frac{S_{D1}T_1}{B_{1E}} \quad , \quad T_{1D} \geq T_s \qquad 10.14b$$

For $m > 1$:

$$D_{mD} = \left(\frac{g}{4\pi^2}\right)\Gamma_m\frac{S_{D1}T_m}{B_{mD}} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_m\frac{S_{DS}T_m^2}{B_{mD}} \qquad 10.15$$

where

B_{1E} = numerical coefficient as set forth in Table 10.1 for the effective damping equal to $\beta_I + \beta_{V1}$ and period equal to T_1

Γ_m = participation factor in the m^{th} mode of vibration of the structure in the direction of interest using Equation 10.5

10.4.3.3 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 of interest shall be calculated in accordance with Section 4.14.1 using modal roof displacements of Section 10.4.3.2.

Total design story drift, Δ_D, shall be determined by the SRSS or complete quadratic combination (CQC) of modal design earthquake drifts.

10.4.3.4 Design Earthquake Story Velocity

Design story velocity in the fundamental mode, ∇_{1D}, and higher modes, ∇_{mD} $(m > 1)$, of the structure in the direction of interest shall be calculated in accordance with Equations 10.16 and 10.17:

$$\nabla_{1D} = 2\pi\frac{\Delta_{1D}}{T_{1D}} \qquad 10.16$$

$$\nabla_{mD} = 2\pi\frac{\Delta_{mD}}{T_m} \qquad 10.17$$

Total design story velocity, ∇_D, shall be determined by the SRSS or complete quadratic combination (CQC) of modal design velocities.

10.4.3.5 Maximum Considered Earthquake Response

Total modal maximum floor deflection at Level i design story drift values, and design story velocity values shall be based on Sections 10.4.3.1, 10.4.3.3, and 10.4.3.4, respectively, except design roof displacement shall be replaced by maximum roof displacement. Maximum roof displacement of the structure in the direction of interest in the fundamental mode, D_{1M}, and higher modes, D_{mM} ($m > 1$), shall be calculated in accordance with Equations 10.18 and 10.19:

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_{1M}^2}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_1^2}{B_{1E}} \quad , \quad T_{1M} < T_s \tag{10.18a}$$

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_{1M}}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_1}{B_{1E}} \quad , \quad T_{1M} \geq T_s \tag{10.18b}$$

$$D_{mM} = \left(\frac{g}{4\pi^2}\right)\Gamma_m \frac{S_{M1}T_m}{B_{mM}} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_m \frac{S_{MS}T_m^2}{B_{mM}} \tag{10.19}$$

where
B_{mM} = numerical coefficient as set forth in Table 10.1 for effective damping equal to β_{mM} and period of the structure equal to T_m
S_{MS} = maximum considered earthquake spectral acceleration parameter, 5% damped, at short periods (0.2 sec), calculated in accordance with Equation $S_{MS} = F_a S_S$ (According to Chapter 3)
S_{M1} = maximum considered earthquake spectral acceleration, 5% damped, at periods 1.0 sec, calculated in accordance with Equation $S_{M1} = F_v S_1$ (According to Chapter 3)

10.5 Equivalent Lateral Load Procedure

Where the Equivalent Lateral Load Procedure is used to design structures with a damping system, the requirements of this section shall apply.

10.5.1 Modeling

Elements of the seismic force-resisting system shall be modeled in a manner consistent with the requirements of Section 4.8. For purposes of analysis, the structure shall be considered to be fixed at the 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 effective damping is calculated in accordance with the procedures of Section 10.6 and used to modify response as required in Sections 10.5.2 and 10.5.3.
The stiffness and damping properties of the damping devices used in the models shall be based on or verified by testing of the damping devices as specified in Section 10.9.

10.5.2 Seismic Force-Resisting System

10.5.2.1 Seismic Base Shear

The seismic base shear, V_u, of the seismic force-resisting system in a given direction shall be determined as the combination of the two modal components, V_1 and V_R, in accordance with Equation 10.20:

$$V_u = \sqrt{{V_1}^2 + {V_R}^2} \geq V_{min} \tag{10.20}$$

where

V_1= design value of the seismic base shear of the fundamental mode in a given direction of response, as determined in Section 10.5.2.2
V_R= design value of the seismic base shear of the residual mode in a given direction, as determined in Section 10.5.2.6
V_{min} = minimum allowable value of base shear permitted for design of the seismic force-resisting system of the structure in direction of the interest, as determined in Section 10.2.2.1

10.5.2.2 Fundamental Mode Base Shear

The fundamental mode base shear, V_1, shall be determined in accordance with Equation 10.21:

$$V_1 = C_{s1}\overline{W}_1 \tag{10.21}$$

where
C_{s1} = the fundamental mode seismic response coefficient, as determined in Section 10.5.2.4
$\overline{W}_1$= the effective fundamental mode seismic weight including portions of the live load as defined by Equation 10.4b for $m = 1$

10.5.2.3 Fundamental Mode Properties

The fundamental mode shape vector, 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 Equations 10.22 and 10.23:

$$\phi_{i1} = \frac{h_i}{h_r} \tag{10.22}$$

$$\Gamma_1 = \frac{\overline{W}_1}{\sum_{i=1}^{n} w_i\phi_{i1}} \tag{10.23}$$

where
h_i = the height above the base to Level i
h_r = the height of the structure above the base to the roof level
w_i = 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 by dynamic analysis using the elastic structural properties and deformational characteristics of the resisting elements, or using Equation 10.24 as follows:

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{n} w_i\delta_i^2}{g\sum_{i=1}^{n} F_i\delta_i}} \tag{10.24}$$

where
F_i = lateral force at Level i of the structure distributed in accordance with Section 4.8.4
δ_i = elastic deflection at Level i of the structure due to applied lateral forces F_i

10.5.2.4 Fundamental Mode Seismic Response Coefficient

The fundamental mode seismic response coefficient, C_{s1}, shall be determined using Equation 10.25 or 10.26:

$$C_{s1} = \left(\frac{R_u}{C_d}\right)\frac{S_{DS}}{\Omega_0 B_{1D}} \qquad for \quad T_{1D} < T_s \tag{10.25}$$

$$C_{s1} = \left(\frac{R_u}{C_d}\right) \frac{S_{D1}}{T_{1D}(\Omega_0 B_{1D})} \quad for \quad T_{1D} \geq T_s \tag{10.26}$$

10.5.2.5 Fundamental Mode Period Determination

The effective fundamental mode $(m = 1)$ period at the design earthquake, T_{1D}, and at the maximum considered earthquake, T_{1M}, shall be based on explicit consideration of the post yield force-deflection characteristics of the structure or shall be calculated using Equations 10.27 and 10.28:

$$T_{1D} = T_1\sqrt{\mu_D} \tag{10.27}$$

$$T_{1M} = T_1\sqrt{\mu_M} \tag{10.28}$$

10.5.2.6 Residual Mode Base Shear

Residual mode base shear, V_R, shall be determined in accordance with Equation 10.29:

$$V_R = C_{sR}\overline{W}_R \tag{10.29}$$

where
C_{sR} = the residual mode seismic response coefficient as determined in Section 10.5.2.8
$\overline{W}_R$ = the effective residual mode effective weight of the structure determined using Equation 10.32

10.5.2.7 Residual Mode Properties

Residual mode shape, ϕ_{iR}, participation factor, Γ_R, effective residual mode seismic weight of the structure, $\overline{W}_R$, and effective period, T_R, shall be determined using Equations 10.30 through 10.33:

$$\phi_{iR} = \frac{1 - \Gamma_1\phi_{i1}}{1 - \Gamma_1} \tag{10.30}$$

$$\Gamma_R = 1 - \Gamma_1 \tag{10.31}$$

$$\overline{W}_R = 1 - \overline{W}_1 \tag{10.32}$$

$$T_R = 0.4T_1 \tag{10.33}$$

10.5.2.8 Residual Mode Seismic Response Coefficient

The residual mode seismic response coefficient, C_{sR}, shall be determined in accordance with Equation 10.34:

$$C_{sR} = \left(\frac{R_u}{C_d}\right) \frac{S_{DS}}{\Omega_0 B_R} \tag{10.34}$$

where
B_R = numerical coefficient as set forth in Table 10.1 for effective damping equal to β_R, and period of the structure equal to T_R

10.5.2.9 Design Lateral Force

The design lateral force in elements of the seismic force-resisting system at Level i due to fundamental mode response, F_{i1}, and residual mode response, F_{iR} of the structure in the direction of interest shall be determined in accordance with Equations 10.35 and 10.36:

$$F_{i1} = w_i \phi_{i1} \frac{\Gamma_1}{\overline{W}_1} V_1 \qquad 10.35$$

$$F_{iR} = w_i \phi_{iR} \frac{\Gamma_R}{\overline{W}_R} V_R \qquad 10.36$$

Design forces in elements of the seismic force-resisting system shall be determined by taking the SRSS of the forces due to fundamental and residual modes.

10.5.3 Damping System

Design forces in damping devices and other elements of the damping system shall be determined on the basis of the floor deflection, story drift, and story velocity response parameters described in the following sections.
Displacements and velocities used to determine 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 calculated for both the design earthquake ground motions and the maximum considered earthquake ground motions, respectively, in accordance with the following sections.

10.5.3.1 Design Earthquake Floor Deflection

The total design deflection at each floor of the structure in the direction of interest shall be calculated as the SRSS of the fundamental and residual mode floor deflections. The fundamental and residual mode deflections due to the design earthquake ground motions, δ_{iRD} and δ_{i1D}, at the center of rigidity of Level i of the structure in the direction of interest shall be determined using Equations 10.37 and 10.38:

$$\delta_{i1D} = D_{1D} \phi_{i1} \qquad 10.37$$

$$\delta_{iRD} = D_{RD} \phi_{iR} \qquad 10.38$$

where
D_{1D} = fundamental mode design displacement at the center of rigidity of the roof level of the structure in the direction under consideration, Section 10.5.3.2

10.5.3.2 Design Earthquake Roof Displacement

Fundamental and residual mode displacements due to the design earthquake ground motions, D_{1D} and D_{RD}, at the center of rigidity of the roof level of the structure in the direction of interest shall be determined using Equations 10.39 and 10.40:

$$D_{1D} = \left(\frac{g}{4\pi^2}\right) \Gamma_1 \frac{S_{DS} T_{1D}^2}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right) \Gamma_1 \frac{S_{DS} T_1^2}{B_{1E}} \quad , \quad T_{1D} < T_s \qquad 10.39a$$

$$D_{1D} = \left(\frac{g}{4\pi^2}\right) \Gamma_1 \frac{S_{D1} T_{1D}}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right) \Gamma_1 \frac{S_{D1} T_1}{B_{1E}} \quad , \quad T_{1D} \geq T_s \qquad 10.39b$$

$$D_{RD} = \left(\frac{g}{4\pi^2}\right) \Gamma_R \frac{S_{D1} T_R}{B_R} \leq \left(\frac{g}{4\pi^2}\right) \Gamma_m \frac{S_{DS} T_R^2}{B_R} \qquad 10.40$$

10.5.3.3 Design Earthquake Story Drift

Design story drifts, Δ_D, in the direction of interest shall be calculated using Equation 10.41:

$$\Delta_D = \sqrt{\Delta_{1D}^2 + \Delta_{RD}^2} \qquad 10.41$$

where

Δ_{1D} = design story drift due to the fundamental mode of vibration of the structure in the direction of interest

Δ_{RD} = design story drift due to the residual mode of vibration of the structure in the direction of interest

Modal design story drifts, Δ_{1D} and Δ_{RD}, shall be determined as the difference of the deflections at the top and bottom of the story under consideration using the floor deflections of Section 10.5.3.2.

10.5.3.4 Design Earthquake Story Velocity

Design story velocities,∇_D, in the direction of interest shall be calculated in accordance with following equations:

$$\nabla_D = \sqrt{\nabla_{1D}^2 + \nabla_{RD}^2} \qquad 10.42$$

$$\nabla_{1D} = 2\pi \frac{\Delta_{1D}}{T_{1D}} \qquad 10.43$$

$$\nabla_{RD} = 2\pi \frac{\Delta_{RD}}{T_R} \qquad 10.44$$

where

∇_{1D} = design story velocity due to the fundamental mode of vibration of the structure in the direction of interest

∇_{RD}= design story velocity due to the residual mode of vibration of the structure in the direction of interest

10.5.3.5 Maximum Considered Earthquake Response

Total and modal maximum floor deflections at Level i, design story drifts, and design story velocities shall be based on the equations in Sections 10.5.3.1, 10.5.3.3, and 10.5.3.4, respectively, except that design roof displacements shall be replaced by maximum roof displacements. Maximum roof displacements shall be calculated in accordance with Equations 10.45 and 10.46:

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_{1M}^2}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{MS}T_1^2}{B_{1E}} \quad , \quad T_{1M} < T_s \qquad 10.45a$$

$$D_{1M} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_{1M}}{B_{1M}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{S_{M1}T_1}{B_{1E}} \quad , \quad T_{1M} \geq T_s \qquad 10.45b$$

$$D_{RM} = \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{M1}T_R}{B_R} \leq \left(\frac{g}{4\pi^2}\right)\Gamma_R \frac{S_{MS}T_R^2}{B_R} \qquad 10.46$$

10.6 Damped Response Modification

As required in Sections 10.4 and 10.5, response of the structure shall be modified for the effects of the damping system.

10.6.1 Damping Coefficient

Where the period of the structure is greater than or equal to T_0, the damping coefficient shall be as prescribed in Table 10.1. Where the period of the structure is less than T_0, the damping coefficient shall be linearly interpolated between a value of 1.0 at a 0-second period for all values of effective damping and the value at period T_0as indicated in Table 10.1.

10.6.2 Effective Damping

The effective damping at the design displacement, β_{mD}, and at the maximum displacement, β_{mM}, of the m^{th} mode of vibration of the structure in the direction under consideration shall be calculated using Equations 10.47 and 10.48:

$$\beta_{mD} = \beta_I + \beta_{Vm}\sqrt{\mu_D} + \beta_{HD} \quad 10.47$$

$$\beta_{mM} = \beta_I + \beta_{Vm}\sqrt{\mu_M} + \beta_{HM} \quad 10.48$$

where
β_{HD} = component of effective damping of the structure in the direction of interest due to post yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand, μ_D
β_{HM} = component of effective damping of the structure in the direction of interest due to post yield hysteretic behavior of the seismic force-resisting system and elements of the damping system at effective ductility demand, μ_M
β_I = component of effective damping of the structure due to the inherent dissipation of energy by elements of the structure, at or just below the effective yield displacement of the seismic force-resisting system
β_{Vm} = component of effective damping of the m^{th} mode of vibration of the structure in the direction of interest due to viscous dissipation of energy by the damping system, at or just below the effective yield displacement of the seismic force-resisting system
Unless analysis or test data supports other values, the effective ductility demand of higher modes of vibration in the direction of interest shall be taken as 1.0.

10.6.2.1 Inherent Damping

Inherent damping, β_I, shall be based on the material type, configuration, and behavior of the structure and non-structural components responding dynamically at or just below yield of the seismic force-resisting system. Unless analysis or test data supports other values, inherent damping shall be taken as not greater than 5 percent of critical for all modes of vibration.

10.6.2.2 Hysteretic Damping

Hysteretic damping of the seismic force-resisting system and elements of the damping system shall be based either on test or analysis or shall be calculated using Equations 10.49 and 10.50:

$$\beta_{HD} = q_H(0.64 - \beta_I)(1 - \frac{1}{\mu_D}) \quad 10.49$$

$$\beta_{HM} = q_H(0.64 - \beta_I)(1 - \frac{1}{\mu_M}) \quad 10.50$$

where
q_H = hysteresis loop adjustment factor, as defined in Section 10.6.2.2.1
Unless analysis or test data supports other values, the hysteretic damping of higher modes of vibration in the direction of interest shall be taken as zero.

10.6.2.2.1 Hysteresis Loop Adjustment Factor

The calculation of hysteretic damping of the seismic force-resisting system and elements of the damping system shall consider 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 full hysteretic loop area of the seismic force-resisting system used for design shall be taken as equal to the factor, q_H, calculated using Equation 10.51:

$$q_H = 0.67\frac{T_s}{T_1} \tag{10.51}$$

where
T_s = period defined by the Equation 11.3

The value of q_H shall not be taken as greater than 1.0 and need not be taken as less than 0.5.

10.6.2.3 Viscous Damping

Viscous damping of the m^{th} mode of vibration of the structure, β_{Vm}, shall be calculated using Equations 10.52 and 10.53:

$$\beta_{Vm} = \frac{\sum_j W_{mj}}{4\pi W_m} \tag{10.52}$$

$$W_m = \frac{1}{2}\sum_j F_{im}\delta_{im} \tag{10.53}$$

where
W_{mj} = work done by 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 of interest at modal displacements, δ_{im}
W_m = maximum strain energy in the m^{th} mode of vibration of the structure in the direction of interest at modal displacements, δ_{im}
F_{im} = m^{th} mode inertial force at Level i
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
Viscous modal damping 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.
The work done by individual damping devices shall be reduced 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.

10.6.3 Effective Ductility Demand

The effective ductility demand on the seismic force-resisting system due to the design earthquake ground motions, μ_D, and due to the maximum considered earthquake ground motions, μ_M, shall be calculated using Equations 10.54 to 10.56:

$$\mu_D = \frac{D_{1D}}{D_Y} \geq 1.0 \tag{10.54}$$

$$\mu_M = \frac{D_{1M}}{D_Y} \geq 1.0 \tag{10.55}$$

$$D_Y = \left(\frac{g}{4\pi^2}\right)\left(\frac{\Omega_0 C_d}{R_u}\right)\Gamma_1 C_{s1} T_1^2 \tag{10.56}$$

The design ductility demand, μ_D, shall not exceed the maximum value of effective ductility demand, μ_{Max}, given in Section 10.6.4.

10.6.4 Maximum Effective Ductility Demand

For determination of the hysteresis loop adjustment factor, hysteretic damping, and other parameters, the maximum value of effective ductility demand, μ_{Max}, shall be calculated using Equations 10.57 and 10.58:

$$\mu_{Max} = 0.5\left[\left(\frac{R_u}{\Omega_0 I}\right)^2 + 1\right] \qquad for\ T_{1D} \leq T_s \qquad 10.57$$

$$\mu_{Max} = \frac{R_u}{\Omega_0 I} \qquad for\ \ T_1 > T_s \qquad 10.58$$

where

I = the importance factor determined in accordance with Section 4.3

For $T_1 < T_S < T_{1D}$, μ_{Max} shall be determined by linear interpolation between the values of Equations 10.57 and 10.58.

10.7 Seismic Load Conditions and Acceptance Criteria

For the nonlinear procedures of Section 10.3, the seismic force-resisting system, damping system, loading conditions and acceptance criteria for response parameters of interest shall conform to Section 10.7.1. Design forces and displacements determined in accordance with the Modal Response Spectrum Analysis (Section 10.4) or the Equivalent Lateral Load Procedure (Section 10.5) shall be checked using the strength design criteria of this standard and the seismic loading conditions of Section 10.7.1 and 10.7.2.

10.7.1 Nonlinear Procedures

Where nonlinear procedures are used in analysis, the seismic force-resisting system, damping system, seismic loading conditions, and acceptance criteria shall conform to 10.7.1.1 to 10.1.1.2.

10.7.1.1 Seismic Force-Resisting System

The seismic force-resisting system shall satisfy the strength requirements of Chapter 4 using the seismic base shear, V_{min} given by Section 10.2.2.1. The story drift shall be determined using the design earthquake ground motions.

10.7.1.2 Damping Systems

The damping devices and their connections shall be sized to resist the forces, displacements, and velocities from the maximum considered earthquake ground motions.

10.7.1.3 Combination of Load Effects

The effects on the damping system due to gravity loads and seismic forces shall be combined in accordance with Section 2.2.2 using the effect of horizontal seismic forces, Q_E, determined in accordance with the analysis. The redundancy factor, ρ, shall be taken equal to 1.0 in all cases, and the seismic load effect with over-strength factor of Section 2.2.4 need not apply to the design of the damping system.

10.7.1.4 Acceptance Criteria for the Response Parameters of Interest

The damping system components shall be evaluated using the strength design criteria of this code using the seismic forces and seismic loading conditions determined from the nonlinear procedures and without reducing strength. The members of the seismic force-resisting system need not be evaluated where using the nonlinear procedure forces.

10.7.2 Modal Response Spectrum and Equivalent Lateral Load Procedures

Where Modal Response Spectrum or Equivalent Lateral Load Procedures are used in analysis, the seismic force-resisting system, damping system, seismic loading conditions, and acceptance criteria shall conform to Sections 10.7.2.1 to 10.7.2.6.

10.7.2.1 Seismic Force-Resisting System

The seismic force-resisting system shall satisfy the requirements of Section 4 using seismic base shear and design forces determined in accordance with Section 10.4.2 or 10.5.2.
The design story drift, Δ_D, as determined in either Section 10.4.3.3 or 10.5.3.3 shall not exceed R_u/C_d times the allowable story drift, as obtained from Table 4.8, considering the effects of torsion as required in Section 4.13.

10.7.2.2 Damping System

The damping system shall satisfy the requirements of Chapter 4 for seismic design forces and seismic loading conditions determined in accordance with this section.

10.7.2.3 Combination of Load Effects

The effects on the damping system and its components due to gravity loads and seismic forces shall be combined in accordance with Section 2.2 using the effect of horizontal seismic forces, Q_E, determined in accordance with Section 10.7.2.5. The redundancy factor, ρ, shall be taken equal to 1.0 in all cases, and the seismic load effect with over-strength factor of Section 2.2.4 need not apply to the design of the damping system.

10.7.2.4 Modal Damping System Design Forces

Modal damping system design forces shall be calculated based on the type of damping devices and the modal design story displacements and velocities determined in accordance with either Section 10.4.3 or 10.5.3.
Modal design story displacements and velocities shall be increased as required to envelop the total design story displacements and velocities determined in accordance with Section 10.3 where peak response is required to be confirmed by Modal Response History Procedure.

A. Displacement dependent damping devices: Design seismic force in displacement dependent damping devices shall be based on the maximum force in the device at displacements up to and including the design story drift, Δ_D.
B. Velocity dependent damping devices: Design seismic force in each mode of vibration in velocity dependent damping devices shall be based on the maximum force in the device at velocities up to and including the design story velocity for the mode of interest.

Displacements and velocities used to determine design forces in damping devices at each story shall account for the angle of orientation of the damping device from the horizontal and consider the effects of increased floor response due to torsional motions.

10.7.2.5 Seismic Load Conditions and Combination of Modal Responses

Seismic design force, Q_E in each element of the damping system shall be taken as the maximum force of the following three loading conditions:

1. Stage of maximum displacement: Seismic design force at the stage of maximum displacement shall be calculated in accordance with Equation 10.59:

$$Q_E = \Omega_0 \sqrt{\sum_m (Q_{mSFRS})^2} \pm Q_{DSD} \qquad 10.59$$

where

Q_{mSFRS} = force in an element of the damping system equal to the design seismic force of the m^{th} mode of vibration of the structure in the direction of interest
Q_{DSD} = force in an element of the damping system required to resist design seismic forces of displacement dependent damping devices
Seismic forces in elements of the damping system, Q_{DSD}, shall be calculated by imposing design forces of displacement dependent damping devices on the damping system as pseudo static forces.
Design seismic forces of displacement dependent damping devices shall be applied in both positive and negative directions at peak displacement of the structure.

2. Stage of maximum velocity: Seismic design force at the stage of maximum velocity shall be calculated in accordance with Equation 10.60:

$$Q_E = \sqrt{\sum_{\mathrm{m}}(Q_{mDSV})^2} \tag{10.60}$$

where
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 of interest
Modal seismic design forces in elements of the damping system, Q_{mDSV}, shall be calculated by imposing modal design forces of velocity dependent devices on the non-deformed damping system as pseudostatic forces. Modal seismic design forces shall be applied in directions consistent with the deformed shape of the mode of interest. Horizontal restraint forces shall be applied at each floor Level i of the non-deformed 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. At each floor Level i, restraint forces shall be proportional to and applied at the location of each mass point.

3. Stage of maximum acceleration: Seismic design force at the stage of maximum acceleration shall be calculated in accordance with Equation 10.61:

$$Q_E = \sqrt{\sum_{\mathrm{m}}(C_{mFD}\, \Omega_0\, Q_{m\mathrm{SFRS}} + C_{mFV} Q_{mDSV})^2} \pm\ Q_{DSD} \tag{10.61}$$

The force coefficients, C_{mFD} and C_{mFV}, shall be determined from Tables 10.2 and 10.3, respectively, using values of effective damping determined in accordance with the following requirements:
For fundamental mode response ($m = 1$) in the direction of interest, the coefficients, C_{1FD} and C_{1FV}, shall be based on the velocity exponent, α, that relates device force to damping device velocity.
The effective fundamental mode damping shall be taken as equal to the total effective damping of the fundamental mode less the hysteretic component of damping (β_{1D}– β_{HD} or β_{1M}– β_{HM}) at the response level of interest ($\mu = \mu_D$ or $\mu = \mu_M$).
For higher mode ($m > 1$) or residual mode response in the direction of interest, the coefficients, C_{mFD} and C_{mFV}, shall be based on a value of α equal to 1.0. The effective modal damping shall be taken as equal to the total effective damping of the mode of interest (β_{mM}–β_{mD}). For determination of the coefficient C_{mFD}, the ductility demand shall be taken as equal to that of the fundamental mode ($\mu = \mu_D$ or $\mu = \mu_M$).

Table 10.2 Force Coefficient, C_{mFD}[1,2]

Effective Damping	$\mu \leq 1.0$				$C_{mFD} = 1$[3]
	$\alpha \leq 0.25$	$\alpha = 0.5$	$\alpha = 0.75$	$\alpha \geq 1.0$	
≤ 0.05	1.00	1.00	1.00	1.00	$\mu \geq 1.0$
0.1	1.00	1.00	1.00	1.00	$\mu \geq 1.0$
0.2	1.00	0.95	0.94	0.93	$\mu \geq 1.1$
0.3	1.00	0.92	0.88	0.86	$\mu \geq 1.2$
0.4	1.00	0.88	0.81	0.78	$\mu \geq 1.3$
0.5	1.00	0.84	0.73	0.71	$\mu \geq 1.4$
0.6	1.00	0.79	0.64	0.64	$\mu \geq 1.6$
0.7	1.00	0.75	0.55	0.58	$\mu \geq 1.7$
0.8	1.00	0.7	0.5	0.53	$\mu \geq 1.9$
0.9	1.00	0.66	0.5	0.5	$\mu \geq 2.1$
≥ 1.0	1.00	0.62	0.5	0.5	$\mu \geq 2.2$

1. Unless analysis or test data support other values, the force coefficient C_{mFD} for viscoelastic systems shall be taken as 1.0.
2. Interpolation shall be used for intermediate values of velocity exponent, α, and ductility demand, μ.
3. 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, C_{mFV}[1,2]

Effective Damping Ratio	$\alpha \leq 0.25$	$\alpha = 0.50$	$\alpha = 0.75$	$\alpha \geq 1.0$
≤ 0.05	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

1. Unless analysis or test data support other values, the force coefficient C_{mFD} for viscoelastic systems shall be taken as 1.0.
2. Interpolation shall be used for intermediate values of velocity exponent, α.

10.7.2.6 Inelastic Response Limits

Elements of the damping system are permitted to exceed strength limits for design loads provided it is shown by analysis or test that:

A. Inelastic response does not adversely affect damping system function.

B. Element forces calculated in accordance with Section 10.7.2.5, using a value of Ω_0 taken as equal to 1.0, do not exceed the strength required to satisfy the load combinations of Section 2.2.2.

10.8 Design Review

A design review of the damping system and related test programs shall be performed by an independent team of registered design professionals in the appropriate disciplines and others experienced in seismic analysis methods and the theory and application of energy dissipation systems.

The design review shall include, but need not be limited to, the following:

A. Review of site-specific seismic criteria including the development of the site-specific spectra and ground motion histories and all other project specific design criteria.
B. Review of the preliminary design of the seismic force-resisting system and the damping system, including design parameters of damping devices.
C. Review of the final design of the seismic force-resisting system and the damping system and all supporting analyses.
D. Review of damping device test requirements, device manufacturing quality control and assurance, and scheduled maintenance and inspection requirements.

10.9 Testing

The force velocity displacement and damping properties used for the design of the damping system shall be based on the prototype tests specified in this section.

The fabrication and quality control procedures used for all prototype and production damping devices shall be identical.

10.9.1 Prototype Tests

The following tests shall be performed separately on two full size damping devices of each type and size used in the design, in the order listed as follows.

Representative sizes of each type of device are permitted to be used for prototype testing, provided both of the following conditions are met:

A. Fabrication and quality control procedures are identical for each type and size of device used in the structure.
B. Prototype testing of representative sizes is accepted by the registered design professional responsible for design of the structure.

Test specimens shall not be used for construction, unless they are accepted by the registered design professional responsible for design of the structure and meet the requirements for prototype and production tests.

10.9.1.1 Data Recording

The force-deflection relationship for each cycle of each test shall be recorded.

10.9.1.2 Sequence and Cycles of Testing

For the following test sequences, each damping device shall be subjected to gravity load effects and thermal environments representative of the installed condition. For seismic testing, the displacement in the devices calculated for the maximum considered earthquake ground motions, termed herein as the maximum device displacement, shall be used.

1. Each damping device shall be subjected to the number of cycles expected in the design windstorm, but not less than 2,000 continuous fully 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 ($f_1 = 1/T_1$).

Exception: Damping devices need not be subjected to these tests if they are not subject to wind induced forces or displacements or if the design wind force is less than the device yield or slip force.

2. Each damping device shall be loaded with five fully reversed, sinusoidal cycles at the maximum earthquake device displacement at a frequency equal to $1/T_{1M}$ as calculated in Section 10.4.2.5.

Where the damping device characteristics vary with operating temperature, these tests shall be conducted at a minimum of three temperatures (minimum, ambient, and maximum) that bracket the range of operating temperatures.

Exception: Damping devices are permitted to be tested by alternative methods provided all of the following conditions are met:

1. Alternative methods of testing are equivalent to the cyclic testing requirements of this section.
2. Alternative methods capture the dependence of the damping device response on ambient temperature, frequency of loading, and temperature rise during testing.

3. Alternative methods are accepted by the registered design professional responsible for the design of the structure.

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 percent for changes in testing frequency from $1/T_{1M}$ to $2.5/T_1$, then the preceding tests shall also be performed at frequencies equal to $1/T_1$ and $2.5/T_1$.

If reduced-scale prototypes are used to qualify the rate-dependent properties of damping devices, the reduced-scale prototypes should be of the same type and materials, and manufactured with the same processes and quality control procedures, as full scale prototypes, and tested at a similitude scaled frequency that represents the full scale loading rates.

10.9.1.3 Testing Similar Devices

Damping devices need not be prototype tested provided that both of the following conditions are met:

1. All pertinent testing and other damping device data are made available to and are accepted by the registered design professional responsible for the design of the structure.
2. The registered design professional substantiates the similarity of the damping device to previously tested devices.

10.9.1.4 Determination of Force-Velocity-Displacement Characteristics

The force-velocity-displacement characteristics of a damping device shall be based on the cyclic load and displacement tests of prototype devices specified in the preceding text. Effective stiffness of a damping device, k_{eff}, shall be calculated for each cycle of deformation using Equation 13.9.

10.9.1.5 Device Adequacy

The performance of a prototype-damping device shall be deemed adequate if all of the conditions listed below are satisfied. The 15 percent limits specified in Sections 10.9.1.5.1 and 10.9.1.5.2 are permitted to be increased by the registered design professional responsible for the design of the structure provided that the increased limit has been demonstrated by analysis not to have a deleterious effect on the response of the structure.

10.9.1.5.1 Displacement Dependent Damping Devices

The performance of the prototype displacement dependent damping devices shall be deemed adequate if the following conditions, based on tests specified in Section 10.9.1.2, are satisfied:

1. For Test A, no signs of damage including leakage, yielding, or breakage.
2. For Tests B and C, the maximum force and minimum force at zero displacement for a damping device for any one cycle does not differ by more than 15 percent from the average maximum and minimum forces at zero displacement as calculated from all cycles in that test at a specific frequency and temperature.
3. For Tests B and C, the maximum force and minimum force at maximum device displacement for a damping device for any one cycle does not differ by more than 15 percent from the average maximum and minimum forces at the maximum device displacement as calculated from all cycles in that test at a specific frequency and temperature.
4. For Tests B and C, the area of hysteresis loop, E_{loop}, of a damping device for any one cycle does not differ by more than 15 percent from the average area of the hysteresis loop as calculated from all cycles in that test at a specific frequency and temperature.
5. The average maximum and minimum forces at zero displacement and maximum displacement, and the average area of the hysteresis loop, E_{loop}, calculated for each test in the sequence of Tests B and C, shall not differ by more than 15 percent from the target values specified by the registered design professional responsible for the design of the structure.

10.9.1.5.2 Velocity Dependent Damping Devices

The performance of the prototype velocity dependent damping devices shall be deemed adequate if the following conditions, based on tests specified in Section 18.9.1.2, are satisfied:

1. For Test A, no signs of damage including leakage, yielding, or breakage.
2. For velocity dependent damping devices with stiffness, the effective stiffness of a damping device in any one cycle of Tests B and C does not differ by more than 15 percent from the average effective stiffness as calculated from all cycles in that test at a specific frequency and temperature.
3. For Tests B and C, the maximum force and minimum force at zero displacement for a damping device for any one cycle does not differ by more than 15 percent from the average maximum and minimum forces at zero displacement as calculated from all cycles in that test at a specific frequency and temperature.
4. For Tests B and C, the area of hysteresis loop, E_{loop}, of a damping device for any one cycle does not differ by more than 15 percent from the average area of the hysteresis loop as calculated from all cycles in that test at a specific frequency and temperature.
5. The average maximum and minimum forces at zero displacement, effective stiffness (for damping devices with stiffness only), and average area of the hysteresis loop, E_{loop}, calculated for each test in the sequence of Tests 2 and 3, does not differ by more than 15 percent from the target values specified by the registered design professional responsible for the design of the structure.

10.9.2 Production Testing

Prior to installation in a building, damping devices shall be tested to ensure that their force-velocity-displacement characteristics fall within the limits set by the registered design professional responsible for the design of the structure. The scope and frequency of the production-testing program shall be determined by the registered design professional responsible for the design of the structure.

Chapter 11

Chimney

11. Chimney

11.1 General Provisions

This chapter covers requirements for seismic analysis and design of shell of chimney and the other similar structures. Chimneys are constructed by reinforced concrete, steel, or other appropriate materials and have two main parts:

1. Chimney shell, which has a structural role and shall be designed for the effects of gravity, temperature, wind and earthquake.
2. Chimney lining, which does not have a structural role and acts as a shield for shell against high temperatures, degrading and erosion.

Chimneys are classified statically into self-support and anchored. Chimneys that are an apparatus of another structure shall be designed for the requirements of Chapters 7 or 8.

Height and upper diameter of chimney shall be determined based on environmental regulations in accordance with effluent gasses. Bottom diameter, shell thickness and foundation design shall be based on requirements of this chapter.

Chimney shall be designed for Seismic Hazard Level II, Section 3.4.2.

11.2 Modeling

Analytical model of chimney shall be adequately precise, to ensure consideration of mass and stiffness changes in shell and lining, and foundation conditions in model. Chimney model shall be divided into at least 10 segments in height.

Modeling could be done with two methods:

1. Modeling with shell elements (usually with FEM)
2. Modeling as a cantilever beam

First method is more precise, with possibility of considering shell-lining interaction.

In addition to requirements from Chapter 5, soil-structure interaction is suggested to be considered where a high-rise chimney is going to be constructed on soft soil.

Where lining is supported by shell at any level, modeling shall include shell and lining and their interaction.

Where chimney is guyed by cables or frames, interaction effects of supporting structure shall be considered in modeling, analysis and design of chimney.

11.3 Analysis Procedures

In this chapter, Equivalent Lateral Load Procedure, Modal Response Spectrum Analysis and Seismic Response History Procedure for seismic analysis of chimney are presented. Where nonlinear static or dynamic analyses are used permitted by other valid references, their requirements shall be considered, provided that design is checked by an independent authorized group of experts. Equivalent Lateral Load Procedure could only be applied to estimate primary structural specifications of chimney but for final design, one of dynamic procedures shall be used.

Vertical component of earthquake need not be considered in analysis and design of chimney.

11.3.1 Equivalent Lateral Load Procedure

11.3.1.1 Base Shear

Value of base shear, V_u, shall be determined from Equation 11.1:

$$V_u = C_u W_{ch} \qquad 11.1$$

where:

C_u = seismic response coefficient from Equation 11.2

W_{ch} = total weight of the chimney

$$C_u = \frac{S_a I}{R_u} \tag{11.2}$$

where:
S_a = spectral acceleration (in g), according to Chapter 3.
I = importance factor of chimney, from Table 4.3 unless greater values are provided by vendor.
R_u = response modification factor from Table 7.2.
For chimneys located in a area with $S_1 \geq 0.6$, seismic response coefficient, C_u, shall not be less than values of Equation 7.5b and for the others, it shall not be less than values of Equation 7.5a. Fundamental period of chimney, T, could be estimated from Equation 11.3a for chimneys with variable cross section and Equation 11.3.b for cylindrical chimneys with uniform cross section.

$$T = \frac{5h_{ch}^2}{\bar{d}_b}\sqrt{\frac{\rho_{ch}}{E_{ch}}}\left[\frac{t_h}{t_b}\right]^{0.3} \tag{11.3a}$$

$$T = C_T h_{ch}\sqrt{\frac{\rho_{ch}}{E_{ch}}} \tag{11.3b}$$

where:
h_{ch} = chimney height from the base level
$\bar{d}_b$ = crust average diameter of chimney at bottom
ρ_{ch} = density of chimney material (including shell and cover)
E_{ch} = effective elasticity modulus of chimney shell material
t_h = chimney shell thickness at top
t_b = chimney shell thickness at bottom
C_T = correction factor for period proportioned to the slenderness of chimney according to Table 11.1.
r_{ch} = radius of gyration for chimney cross section

Table 11.1 Correction factor of fundamental period for a cylindrical chimney

h_{ch}/r_{ch}	5	10	15	20	25	30	35	40	45	50 and greater
C_T	14.4	21.2	29.6	38.4	47.2	56.0	65.0	73.8	82.8	$1.8(h_{ch}/r_{ch})$

11.3.1.2 Distribution of Lateral Force

Vertical distribution of lateral force shall be determined from Equation 11.4:

$$F_i = V_u \frac{w_i h_i^2}{\sum_{i=1}^{n} w_i h_i^2} \tag{11.4}$$

where:
F_i = lateral force applied on mass center of i^{th} segment.
w_i = weight of segment i
h_i = height of segment i, calculated from base level
n = number of segments
Design shear at level z (h_z), V_z, shall be determined from Equation 11.5

$$V_z = J_v^z \sum_{j=i}^{n} F_j \tag{11.5}$$

where:
i = the number of the segment adjacent to level z, with mass center above it.
J_v^z = correction factor for higher mode effects at level z, from Equation 11.6:

$$J_v^z = 8\left(J_v^{top} - J_v^{0.5h}\right)\left(\frac{h_z}{h} - \frac{1}{2}\right)^3 + J_v^{0.5h} \qquad 0.5 \le \frac{h_z}{h} \le 1 \tag{11.6a}$$

$$J_v^z = 1 - 2\left(1 - J_v^{0.5h}\right)\left(\frac{h_z}{h}\right) \qquad 0 \le \frac{h_z}{h} \le 0.5 \tag{11.6b}$$

where:

$$J_v^{top} = 0.96 + 0.23T \tag{11.7}$$

$$J_v^{0.5h} = 0.43 + \frac{1}{2T^{1.5}} \tag{11.8}$$

11.3.1.3 Distribution of Moment

Moment at level h_z, M_z, shall be determined from Equation 11.9:

$$M_z = J_m^z \left[\sum_{j=i}^{n} F_j\left(h_j - h_z\right)\right] \tag{11.9}$$

where:

$$J_m^z = 11.1\left(J_m^{base} - J_m^{0.3h}\right)\left(0.3 - \frac{h_z}{h}\right)^2 + J_m^{0.3h} \qquad 0 \le \frac{h_z}{h} \le 0.3 \tag{11.10a}$$

$$J_m^z = 2.78\left(J_m^{0.9h} - J_m^{0.3h}\right)\left(\frac{h_z}{h} - 0.3\right)^2 + J_m^{0.3h} \qquad 0.3 \le \frac{h_z}{h} \le 1.0 \tag{11.10b}$$

where:

$$J_m^{0.9h} = 1.15 + 0.025T^2 \tag{11.11}$$

$$J_m^{0.3h} = 0.3 + 0.004(6 - T)^3 \tag{11.12}$$

$$J_m^{0.3h} = 0.4 + \frac{(6 - T)^3}{300} \tag{11.13}$$

11.3.2 Dynamic Procedures

Dynamic procedures can be performed with both Modal Response Spectrum Analysis and Seismic Response History Procedure, considering requirements from Chapter 4 and assuming an appropriate damping ratio. Appropriate damping ratio for concrete and steel shells are 5% and 1.5%, respectively. To determine a spectrum with damping ratio equal to η%, 5% damped spectrum may be multiplied by factor D_s from Equation 11.14.

$$D_s = \frac{-Ln\left(\frac{\eta}{100}\right)}{Ln(20)} \qquad 11.14$$

For non-circular chimneys, when Seismic Response History Procedure is used, pairs of horizontal records shall be used. In Modal Response Spectrum Analysis, SRSS combination of results for two orthogonal directions shall be performed.

11.3.2.1 Modal Response Spectrum Analysis

Modal Response Spectrum Analysis shall be performed in accordance with requirements of Chapters 3 and 4.
Dynamic response of chimney shall be calculated based on at least first five effective modes, provided that the cumulative effective modal mass ratio is not less than 90%, using SRSS or CQC rules.

11.3.2.2 Seismic Response History Procedure

When Seismic Response History Procedure is used, requirements of Section 4.10 shall be considered.

11.4 Design Requirements

11.4.1 Overturning

The minimum factor of safety against overturning shall be 1.5 using unfactored loads.

11.4.2 Displacement Check

The maximum linear lateral deflection of the top of a chimney, y_{max} (m) before applying load factors shall not exceed the limit set forth by Equation 11.15:

$$y_{max} = 0.0033h \qquad 11.15$$

The displacement limit is compared against the deflection calculated using uncracked concrete sections and a fixed base.
The maximum lateral relative displacement of steel chimneys due to unfactored wind loads shall be limited to 1/200 of the height of non-anchored part of the chimney.

11.4.3 Load Combinations

In SD method, load combinations including seismic loads are defined in Equations 11.16.

$$1.2D + 1.2T + E \qquad 11.16a$$

$$0.9D + 1.2T + E \qquad 11.16b$$

In ASD method,

$$D + 0.9T + 0.75(0.7E) \qquad 11.17a$$

$$D + 0.7E \qquad 11.17b$$

$$0.6D + 0.9T + 0.7E \qquad 11.17b$$

where:
T = thermal effects at service temperature of chimney
To determine wind loads, distribution and combinations containing wind loads, valid references should be consulted.

11.4.4 Design Criteria and Detailing

In general, the dimensions of the chimneys are determined based on operating conditions, environmental issues, location of adjacent structures, and so on, and are designed using the regulations and relevant standards. However, compliance with the requirements of this section is required as the minimum design requirements.

11.4.4.1 Design Criteria for Concrete Chimneys

Concrete chimneys shall be designed according to requirements of Reference [10], except that the base shear and relevant coefficients shall be selected from Section 11.3.1 of this code. In addition, following requirements around openings shall be considered.

Not more than 50% of bars shall be spliced along any horizontal or vertical plane. Also, if the opening area is larger than 10% of the total area, sections at opening shall be designed for vertical and horizontal loads and bending moments perpendicular to effective area, considering an over-strength factor equal to 1.5. Location of applying this factor is below and over the opening, with a distance equal to half-width of the largest opening at location under consideration. In this region, adequate reinforcement considering appropriate development length shall be provided.

Details of opening adjacent parts shall be derived from Reference [8] for columns. Details shall be followed at a minimum length equal to two times shell thickness in transverse direction and two times shell thickness at top and bottom of opening in longitudinal direction, not less than required by the development length.

Where opening is adjacent to foundation, and applying the above requirements is not possible, reinforcement of opening sides shall develop in foundation. Reinforcement ratio in this region is calculated from Reference [8] for compression members.

11.4.4.2 Design Criteria for Steel Chimneys

For providing stability of stand-alone steel chimneys with a height of more than 40 m, it is necessary to increase the diameter at the base level. This part is called skirt and its minimum height is one-third of the total height of the chimney. The outer diameter of the chimney, without insulation layer, at the highest level must be at least one-twentieth of its cylindrical height. In the case of chimneys with an insulation layer, the outer diameter of the chimney must be at least 0.04 times its cylindrical height. The minimum outside diameter of the chimney at the base level should be 1.6 times the outer diameter of the chimney at its highest level.

Shell thickness shall be determined based on lateral displacement and response of chimney. It is necessary to add the permitted corrosion thickness allowance to the computational shell thickness, according to the related standard. Shell thickness including corrosion allowance, shall not be less than the greater of 6 mm and 1/500 outer diameter of the shell at any point.

Chapter 12

Storage Tank

12. Storage Tank

12.1 General Provisions

In this chapter, minimum requirements and regulations for design and construction of storage tanks for seismic load effects of design earthquake are presented. Design earthquake, is the Seismic Hazard Level II as defined in Section 3.4.2. Considering service earthquake, Seismic Hazard Level I from Section 3.4.1, is not necessary for storage tanks.

12.1.1 Scope

Provisions of this chapter include ground and elevated storage tanks for water and petroleum products. Storage tanks shall be classified by Function & Risk Category in accordance with Section 12.2.4. In any case, requirements from other chapters of this code shall be satisfied. For storage tanks constructed on isolation systems, requirements from Chapter 9 shall be considered either.

12.1.2 Types of Storage Tanks

Tanks can be classified as ground or elevated, atmospheric or pressured, and supported by foundation or other structure. Ground tanks might be completely on earth, semi buried or buried. Elevated tanks are installed on a supporting structure higher than ground level. Pressure tanks, usually with smaller sizes, are supported by foundation, specific pedestals, or other structure. Storage tanks might be made of metal, reinforced concrete, prestressed concrete, masonry materials, or other appropriate materials. Ground metal storage tanks are classified as self-anchored and mechanically anchored. If the ground metal tank resists overturning moment by weight of the tank and containing liquid, it will be classified as self-anchored. Otherwise, if anchorage is provided by means of anchor bolts.

12.1.3 Site-Specific Studies

For storage tanks in Function & Risk Category III and IV, and all tanks in Function & Risk Category II located near an active fault (less than 10 kilometers distant), a site-specific study, as mentioned in Chapter 3, is mandatory. In addition, it shall be done for tanks on Site Class IV (regarding Reference [2]). In other cases, site-specific study shall be done based on the client request.

12.1.4 Geotechnical Studies

Geotechnical and geo-seismic studies including definition of soil layers under storage tanks, to satisfy design requirements of this chapter are necessary. In these studies, critical factors such as liquefaction potential, landslide and slope instability shall be considered. In addition, in designing tank foundation, following critical factors shall be considered:

A. Site is situated on mountain or hill slope, such that the tank bottom is partly on natural stiff ground and partly on embankment.
B. Existence of compactable organic soil or embankment at site.
C. Existence of plastic clay soils with high rates of long term settlement.
D. Deep drilling and excavation near tank foundation
E. Noticeable decrease of underground water table caused by construction of wells near tank, and consequent soil settlements.
F. Probability of buoyancy for foundations exposed to flood, increasing of the underground water level, or other reasons.
G. Probability of chemical degrading of soil

Loose and inappropriate soli shall be modified using efficient methods, such as:

A. Removing inappropriate soil layers and substituting with suitable compacted soil
B. Using micropiles to increase density and strength of soft soils
C. Consolidating soft soil by pre-loadi
D. Soil stabilization by chemical methods or grouting.
E. Using deep piles to transfer loads to the bedrock.
F. Controlled and slow water refilling of tank, followed by hydraulic tests, or separately, to consolidate the underlying soil.

G. Arranging appropriate foresights in order to prevent and decrease chemical degradation of beneath soil.
H. Prediction of an appropriate drainage system to discharge water and chemical materials.

12.2 General Seismic Analysis and Design Provisions

It is permitted to analyze storage tanks with Equivalent Lateral Load or dynamic procedures. Section 12.3 illustrates basis of the Equivalent Lateral Load Procedure for ground-supported storage tanks. In case of performing dynamic analyses for ground-supported storage tanks, hydrodynamic effects, fluid-structure interaction and soil-structure interaction effects shall be considered in modeling, using valid methods. However, ground-supported storage tanks shall resist a minimum of 85% of static loads mentioned in Section 12.3. Analysis of elevated tanks shall be in accordance with Section 12.7. Supporting structures for pressure tanks shall be analyzed in accordance with Chapter 7. Where base isolation systems are used, requirements from Chapter 9 shall be considered and in case of using damping systems, in tank analysis and design, their interaction with tank shall be considered using provisions of Chapter 10 of this code, or other valid references.

12.2.1 Modeling

In modeling of ground-supported storage tanks, flexibility of wall and bottom, rocking and sliding actions, which may increase the impulsive liquid period (Section 12.3.2) and change seismic responses, shall be considered. For a self-anchored tank with a rigid base, separation between tank and base may result in decrease of hydrodynamic forces but increase of wall compressive longitudinal stresses and probability of wall buckling. For a self-anchored tank with a flexible base, increase of wall compressive longitudinal stresses is smaller, but sinking is probable. Pipe and other attachment connections shall have an adequate ductility to tolerate deflections. It is recommended not to attach stairs to the ground, otherwise interaction effects between staircase and tank shall be considered in modeling.
Supporting structure of elevated or pressure tanks can be modeled as other industrial structures, considering requirements from Chapter 7.

12.2.2 Impulsive and Convective Liquid

For simplicity of analysis, the contained liquid mass in a tank may be divided into equivalent impulsive and convective liquid masses. Impulsive liquid is that part of the fluid, which is supposed to be bounded to and move along with the structure. The convective liquid is the sloshing part of the fluid mass at upper part of the tank. In most cases, main part of the base shear and overturning moment is caused by the impulsive liquid. Free surface movement in tank is determined by the convective liquid mass properties. Calculation of these properties in Equivalent Lateral Load Procedure for ground tanks is mentioned in Section 12.3.

12.2.3 Seismic Damages

To design a storage tank, different kinds of potential seismic damage, especially those mentioned below shall be considered:

- Elephant foot buckling caused by wall compressive stresses.
- Damage in roof and wall upper parts caused by sloshing.
- Shell disruption and stress concentration near to anchors to the base or foundation.
- Sliding of ground tank caused by seismic horizontal forces prevailing to the frictional strength.
- Damage to rigid connections of pipes and other attached equipment, disruption of bottom-shell welds, differential settlement of foundation caused by bottom uplift in self or semi-anchored tanks.
- Fire followed by earthquake, mostly caused by connection failure or flammable liquid leakage from roof especially in floating roof tanks.

12.2.4 Classification of Tanks by Function and Risk

Storage Tanks, regarding the function and probable risk imposed to the life and environment, and economic aspects, are classified into four categories with different importance factors, mentioned in Table 12.1.

Function & Risk Category I. Low-importance tanks: including small temporary tanks with low human risk if damaged. For this group, requirements of this chapter are not mandatory.

Function & Risk Category II. Ordinary-importance tanks: including tanks that damage to or stop of their functions will not impose human or great economic losses. Ground non-potable or non-firefighting water storage tanks and not flammable and toxic chemical materials storage tanks, provided that liquid leaking does not impose functional problems in other structures with higher importance categories, are classified in this group.

Function & Risk Category III. High-importance tanks: including tanks that their damage caused by design earthquake shall be limited and they shall return to operation in a reasonably limited time. These tanks are so important that only for a limited rehabilitation period they can be exempted of operation. Tanks containing stable toxic chemicals, low flammable liquids, ground and elevated potable water tanks with a capacity more than 100 m^3 are classified in this group. These tanks are permitted to have limited inelastic deformations in design earthquake.

Function & Risk Category IV. Very high-importance tanks (essential tanks): including tanks, that preserving their contained liquid or being conserving their operation after earthquake is necessary. Damage of such tanks can impose noticeable human casualty or environmental losses. Tanks containing unstable or detonable toxic chemicals, inflammable liquids and firefighting water tanks are classified into this group. In addition, storage tanks that their damage can impose a long time delay in production process are in this group. An essential tank shall generally behave elastically in design earthquake.

Table 12.1 Importance Factor of Storage Tanks

Function & Risk Category	Importance Factor
II	1
III	1.25
IV	1.5

12.2.5 Load Combinations

12.2.5.1 Allowable Stress Design (ASD) Method

Design of steel ground storage tanks shall be performed using ASD method. Load combinations in ASD method shall be taken according to considered from Section 2.2.1, excluding 0.7 coefficient (due to using a response factor for allowable stress level for calculating earthquake effects). In those combinations, fluid load, F, and lateral pressure load, H, shall be included.

12.2.5.2 Strength Design (SD) Method

In design of concrete storage tanks, concrete supporting structures, steel piers of elevated tanks, and similar items, load combinations in strength method from Section 2.2 may be used. In those combinations, fluid load, F, and lateral pressure load, H, shall be included.

12.2.5.3 Allowable Strength (AS) Method

For designing steel piers of elevated tanks and similar components by ASD method, ASD load combinations from Section 2.2.1 may be used.

12.3 Equivalent Lateral Load Procedure for Ground Storage tanks

12.3.1 Basis

In this procedure, only first vibration mode for impulsive and convective liquid masses are considered. For impulsive liquid mass plus tank structure, and for convective liquid mass, damping ratio is considered to be 5% and 0.5%, respectively. Spectral ordinate with 0.5% damping ratio is permitted to be assumed equal to 1.5 times the spectral ordinate with 5% damping ratio.
For each force or deformation component, effects of impulsive mass displacement (including impulsive liquid mass and tank solid parts), convective mass displacement and vertical earthquake component effect (if necessary regarding Section 12.3.10) shall be combined using SRSS method.

12.3.2 Fundamental Period

Fundamental period (sec.) of the tank structure plus impulsive mass is calculated from Equation 12.1.

$$T_i = C_i H_L \sqrt{\frac{\rho_L D}{2 t_e E_t}} \qquad 12.1$$

where:
H_L = maximum height of the liquid (m)
D = nominal diameter of tank (internal diameter in cylindrical tanks and internal length along earthquake direction under consideration in rectangular tanks (m)
t_e = effective thickness of tank shell (m)
ρ_L =liquid density (mass per unit volume-kg/m^3)
E_t = effective Young modulus of tank shell material (Pa)
C_i = dimensionless coefficient for tank impulsive mass period determination mentioned in Table 12.2

Effective thickness of tank shell shall be derived considering the changes in material, thickness and shell longitudinal stresses. For shells with a uniform material, average thickness value is permitted to be used as effective thickness. In addition, Equation 12.2 may be used to determine effective thickness.

$$t_e = \frac{\sum_{i=1}^{n} t_i d_i x_i}{\sum_{i=1}^{n} d_i x_i} \qquad 12.2$$

where:
t_i = thickness of course i of shell plate
d_i = width (height) of course i of shell plate
x_i = distance between middle of course i and liquid level
n = number of courses (shell courses with their middle level being above liquid level shall be excluded from calculation).
Fundamental period of liquid convective mass (sec) can be calculated from both of the following equations:

$$T_c = 2\pi \sqrt{\frac{D_1}{g}} \quad , \quad D_1 = \frac{D}{3.67 tanh\left(\frac{3.67 H_L}{D}\right)} \qquad 12.3a$$

$$T_c = C_c \sqrt{\frac{D}{2}} \qquad 12.3b$$

where:

g = acceleration of gravity (m/s^2)
C_c = coefficient for tank impulsive mass period determination $(s/\sqrt{m})$ from Table 12.2
For rectangular tanks, the factor “3.67” in Equation 12.3a shall be replaced by “3.16”.

Table 12.2 Period Calculation Coefficients

H_L/D	0.15	0.25	0.35	0.50	0.75	1.00	1.25	1.50
C_i	9.28	7.74	6.97	6.36	6.06	6.21	6.56	7.03
C_c	2.09	1.74	1.60	1.52	1.48	1.48	1.48	1.48

12.3.3 Impulsive and Convective Liquid Mass

Impulsive liquid mass, m_i, and convective liquid mass, m_c, can be calculated regarding the total liquid mass, m_p, from Equations 12.4 and 12.5.

$$m_i = \frac{tanh\left(0.866\frac{D}{H_L}\right)}{0.866\frac{D}{H_L}} m_p \qquad \frac{D}{H_L} \geq \frac{4}{3} \qquad 12.4a$$

$$m_i = \left[1.0 - 0.218\frac{D}{H_L}\right] m_p \qquad \frac{D}{H_L} < \frac{4}{3} \qquad 12.4b$$

$$m_c = 0.230\frac{D}{H_L}\ tanh\left(\frac{3.67H_L}{D}\right) m_p \qquad 12.5$$

For rectangular tanks, Equation 12.4 shall be used for all D/H_L ratios, and “0.23” and “3.67” in Equation 12.5 shall be replaced by “0.264” and “3.16”, respectively.

12.3.4 Design Forces and Base Shear

Lateral equivalent force for impulsive liquid, roof, bottom and shell of the tank is calculated by multiplying their weight by seismic coefficient A_i, from Section 12.3.7. Lateral equivalent force for convective liquid is calculated by multiplying its weight by seismic coefficient A_c, from Section 12.3.8. Total design base shear, V_u, is calculated from Equation 12.6.

$$V_u = \sqrt{V_i^2 + V_c^2} \qquad 12.6$$

where:
V_i = base shear corresponding to tank impulsive mass including impulsive liquid, tank roof, bottom and shell, from Equation 12.7.
V_c= base shear corresponding to convective liquid mass, from Equation 12.8.

$$V_i = A_i g\left(m_i + m_r + m_f + m_s\right) \qquad 12.7$$

$$V_c = A_c g m_c \qquad 12.8$$

where:
m_r = total mass of fixed tank roof including framing, knuckles, any permanent attachment and 10% of roof design snow mass.

m_f = mass of the tank bottom
m_s = total mass of tank shell and appurtenances

12.3.5 Ringwall Moment

Ringwall moment, M_{rw}, is a portion of the total seismic overturning moment that acts at the base of the tank shell perimeter. This moment is used to calculate vertical axial forces at the base of the tank shell perimeter and shell thickness design, vertical load applied on shell strip foundation, and anchorage forces of the tank to foundation. If tank is built on a mat foundation (concrete slab), M_{rw} value is just used to control shell thickness and tank anchorage. In cylindrical tanks, this moment is called the annular ring moment. Value of this moment shall be determined by Equation 12.9.

$$M_{rw} = \sqrt{[A_i g(m_i h_i + m_s h_s + m_r h_r)]^2 + [A_c g m_c h_c]^2} \qquad 12.9$$

where:
h_i = height of the center of action of the lateral seismic force related to the impulsive liquid from the bottom of the tank for determination of ringwall moment (Equation 12.10).
h_s = height of the tank shell center of gravity from the tank bottom
h_r = height of the tank roof center of gravity from the bottom
h_c = height of the center of action of the lateral seismic force related to the convective liquid from the bottom of the tank for determination of ringwall moment (Equation 12.11).

$$when\ \frac{D}{H_L} \geq \frac{4}{3} \qquad h_i = 0.375 H_L \qquad 12.10a$$

$$when\ \frac{D}{H_L} < \frac{4}{3} \qquad h_i = \left(0.500 - 0.094\frac{D}{H_L}\right) H_L \qquad 12.10b$$

$$h_c = \left[1 - \frac{cosh\left(\frac{3.67H_L}{D}\right) - 1}{\frac{3.67H_L}{D} sinh\frac{3.67H_L}{D}}\right] H_L \qquad 12.11$$

For rectangular tanks, “3.67” in Equation 12.11 shall be replaced by “3.16”.
To compare soil stress under shell strip foundation (annular ring in cylindrical tanks) with the allowable stress of soil, load combinations from Section 2.2.1 without increase in allowable stress shall be used.

12.3.6 Slab Moment

Slab moment, M_s, depends on seismic force distribution on tank shell and bottom, and shall be determined from Equation 12.12. This value is used for foundation slab, pile cap design and soil stress evaluation for mat foundations.

$$M_s = \sqrt{[A_i g(m_i h_{is} + m_s h_s + m_r h_r)]^2 + [A_c g m_c h_{cs}]^2} \qquad 12.12$$

where:
h_{is} = height of the center of action of the lateral seismic force related to the impulsive liquid from the bottom of the tank for determination of slab moment (Equation 12.13).
h_{cs} = height of the center of action of the lateral seismic force related to the convective liquid from the bottom of the tank for determination of slab moment (Equation 12.14).

$$when\ \ \frac{D}{H_L} \geq \frac{4}{3} \qquad h_{is} = \left[\frac{0.866\frac{D}{H_L}}{2tanh\left(0.866\frac{D}{H_L}\right)} - \frac{1}{8}\right]H_L \tag{12.13a}$$

$$when\ \ \frac{D}{H_L} < \frac{4}{3} \qquad m_i = \left(0.500 + 0.06\frac{D}{H_L}\right)H_L \tag{12.13b}$$

$$h_{cs} = \left[1 - \frac{cosh\left(\frac{3.67H_L}{D}\right) - 1.937}{\frac{3.67H_L}{D}sinh\left(\frac{3.67H_L}{D}\right)}\right]H_L \tag{12.14}$$

For rectangular tanks, “3.67” and “1.937” in Equation 12.14 shall be replaced by “3.16” and “2.01”, respectively.
To compare soil stress under shell strip foundation (annular ring in cylindrical tanks) with soil allowable stress of soil, load combinations from Section 2.2.1 without increase in allowable stress shall be used.

12.3.7 Impulsive Mass Seismic Coefficient

Impulsive mass seismic coefficient, A_i, shall be determined from Equation 12.15.

$$A_i = \frac{S_a I}{R} \geq (A_i)_{min} \tag{12.15a}$$

$$(A_i)_{min} = \begin{cases} 0.01 & S_1 < 0.6 \\ \frac{0.5S_1 I}{R} & S_1 \geq 0.6 \end{cases} \tag{12.15b}$$

where:
S_a = mapped spectral response acceleration parameter (g) with 5% damping ratio, corresponding to T_i (Equation 12.1). It is permitted to use S_{DS} instead, without referring to Equation 12.1.
S_1 = mapped spectral response acceleration parameter (g) corresponding to rare earthquake (Seismic Hazard Level III) at 1 sec. on bedrock, as defined in Chapter 3.
R = response modification factor, equal to R_w for steel tanks and R_u for concrete tanks from Table 12.3
I = importance factor from Table 12.1

12.3.8 Convective Mass Seismic Coefficient

Convective mass seismic coefficient, A_c, shall be determined from Equation 12.16.

$$A_c = \frac{1.5S_a I}{R_c} \leq A_i \tag{12.16}$$

where:
S_a = mapped spectral response acceleration parameter (g) with 5% damping ratio, corresponding to T_c (Equation 12.3).
R_c = response modification factor for convective liquid mass, permitted to be supposed equal to 2 and 1 for steel and concrete tanks, respectively.
Factor “1.5” in above equation is used to consider 5% damping for convective mass spectrum. If in Equation 12.16, 0.5% damped site-specific response spectrum is used, this factor shall be removed.

12.3.9 Response Modification, Over-strength and Displacement Amplification Factors

Response modification factor, R, over-strength factor, Ω_0, and lateral displacement amplification factor, C_d, for storage tanks shall be determined from Table 12.3. Response modification factor value for convective mass, R_c, in concrete tanks is equal to 1 and in steel tanks and other materials is equal to 2. Design lateral displacement at each level of tank shall be determined by multiplying elastic displacement by C_d. For reinforced and prestressed concrete tank definition, refer to Section 12.5.1.

Table 12.3 Seismic Parameters for Ground Storage Tanks

Type of Tank	R_w	R_u	Ω_0	C_d
Mechanically anchored, steel or fiber-reinforced polymer tank	4	3	2	2.5
Self-anchored, steel or fiber-reinforced polymer tank	3.5	2.5	2	2
Non-sliding base reinforced or prestressed concrete tank	-	2	2	2
Anchored flexible base reinforced or prestressed concrete tank	-	3.25	2	2
Unanchored and unconstrained flexible base reinforced or prestressed concrete tank	-	1.5	1.5	1.5
All other	-	1.5	1.5	1.5

For non-sliding buried reinforced or prestressed concrete tanks, it is permitted to use $R_u = 2.8$. Buried tank is defined a tank in which liquid level is lower than the surrounding ground level. For semi buried tanks, linear interpolation is permitted to determine R_u.

12.3.10 Vertical Seismic Effects

Earthquake vertical acceleration coefficient, A_v, is calculated from Equation 12.17.

$$A_v = 0.2S_{DS}I \tag{12.17}$$

where:
S_{DS}= 5% damped spectral response acceleration parameter (g) at short periods, 0.2 sec. (Section 2.2.3.2).
Vertical component shall be applied both upward and downward. Vertical acceleration effects need not be combined simultaneously for determining loads, forces, and resistance to overturning in the tank shell. Vertical seismic effects shall be considered for:

A. Shell hoop tensile stresses (Section 12.3.13)
B. Shell vertical compression membrane force (Section 12.4.2.1)
C. Anchorage design (Section 12.4.1.2)
D. Design of fixed roof components (Section 12.4.10)
E. Sliding (Section 12.6.3)
F. Foundation design (Section 12.6.2)

12.3.11 Lateral Force Distribution

Horizontal distribution of the hydrodynamic pressure on the tank shell at each level caused by impulsive liquid, q_i, and convective liquid, q_c, shall be calculated from Equations 12.18 and 12.19, respectively.

$$q_i = \frac{2p_i cos\theta}{\pi D} \tag{12.18}$$

$$q_c = \frac{2p_c cos\theta}{\pi D} \tag{12.19}$$

where:

p_i = lateral force intensity caused by impulsive mass at considered height based on a trapezoidal distribution
p_c = lateral force intensity caused by convective mass at considered height based on a trapezoidal distribution
θ = angle between earthquake direction and radius passing from considered point on tank shell
Distribution of pressure from above equations for $-\pi/2 \le \theta \le \pi/2$ on tank shell is pressure from inside to outside and at the other half is suction toward the inside of the tank shell.
For rectangular tanks, p_i and p_c intensities shall be divided equally between two walls perpendicular to the earthquake direction, and distributed uniformly along the wall length. This distribution, for one wall is pressure from inside to outside, and for the other wall is suction toward inside. Figure 12.1 illustrates impulsive and convective liquid lateral load distribution for cylindrical and rectangular tanks.
Lateral load distribution for solid parts of the tank, such as fixed roof, bottom and shell is proportional to mass distribution in these parts.

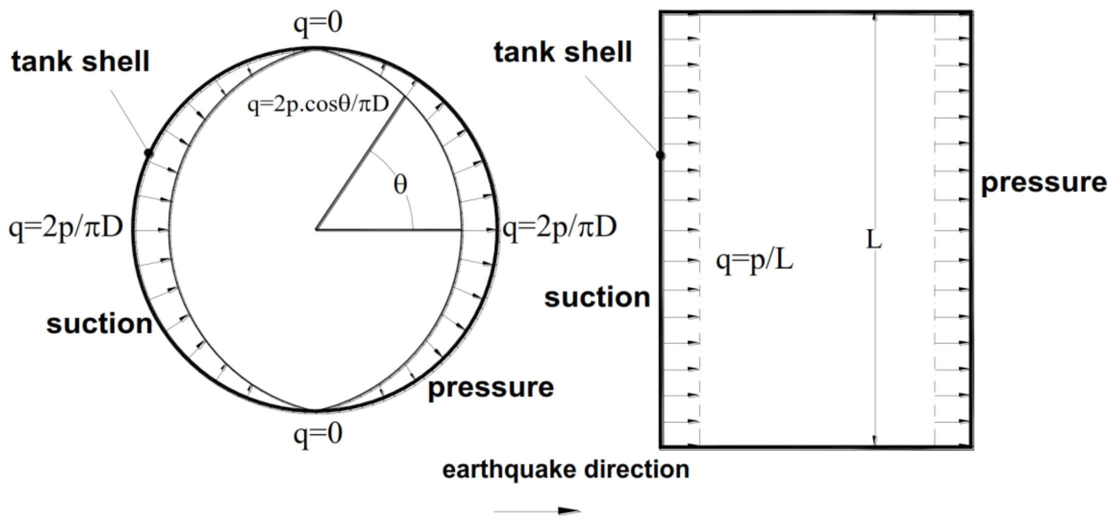


Figure 12.1 Horizontal Distribution of Earthquake Induced Hydrodynamic Impulsive and Convective Lateral Loads in Cylindrical and Rectangular Tanks

12.3.12 Freeboard

Freeboard from liquid level shall be determined considering the height of the sloshing wave and structural characteristics. The maximum sloshing wave height, δ_s, from the liquid design level, H_L, shall be calculated from Equation 12.20.

$$\delta_s = \frac{0.63DS_{D1}I}{T_c} \qquad T_c \le 4sec \qquad 12.20a$$

$$\delta_s = \frac{2.5DS_{D1}I}{T_c^2} \qquad T_c > 4sec \qquad 12.20b$$

A freeboard from the liquid level to the bottom level of the roof shall be determined and considered as follows:

A. For Function & Risk Category II, there is no need to consider freeboard, but if due to any other issue, a freeboard is meant to be specified, for economic considerations, it needs not to be more than $0.7\delta_s$.
B. For Function & Risk Category III, if $S_{DS} < 0.33$, there is no need to consider freeboard, but if due to any other issue, a freeboard is meant to be specified, for economic considerations, it needs not to be more than $0.7\delta_s$.

C. For Function & Risk Category III, if $S_{DS} \geq 0.33$, a minimum freeboard equal to $0.7\delta_s$ shall be specified.
D. For Function & Risk Category IV a minimum freeboard equal to δ_s shall be specified.
Note: for articles C and D, if one of the following alternatives is provided, there will be no need to consider freeboard:
- Secondary containment is provided to control the liquid spill
- The roof and tank shell are designed to contain the sloshing liquid.

12.3.13 Dynamic Liquid Hoop Forces in Tank Shell

Dynamic hoop forces per the vertical unit length of the cylindrical shell, due to the seismic motion of the convective mass, N_i, and impulsive mass, N_c, shall be determined by Equation 12.21 and 12.22.

$$N_i = 0.864 A_i \rho_L g D H_L \left[\frac{Y}{H_L} - 0.5\left(\frac{Y}{H_L}\right)^2\right] tanh\left(0.866\frac{D}{H_L}\right) \qquad \frac{D}{H_L} \geq \frac{4}{3} \qquad 12.21a$$

$$N_i = 0.532 A_i \rho_L g D^2 \left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right] \qquad \frac{D}{H_L} < \frac{4}{3}, Y < 0.75D \qquad 12.21b$$

$$N_i = 0.264 A_i \rho_L g D^2 \qquad \frac{D}{H_L} < \frac{4}{3}, Y \geq 0.75D \qquad 12.21c$$

$$N_c = \frac{0.189 A_c \rho_L g D^2 cosh\left[\frac{3.67(H_L - Y)}{D}\right]}{cosh\left[\frac{3.67 H_L}{D}\right]} \qquad for\ all\ values\ of\ \frac{D}{H_L} \qquad 12.22$$

where:
ρ_L: liquid density (kg/m^3)
Equations 12.21 and 12.22 are for thin-shell cylindrical tanks. In addition, by considering distribution of forces from Section 12.3.11, internal forces of shell can be calculated.
Total seismic hoop forces per unit length of the shell, shall be determined from Equation 12.23.

$$N_s = \sqrt{N_i^2 + N_c^2 + (A_v N_h)^2} \qquad 12.23$$

Seismic hoop stress, $\sigma_s = N_s/t$, considering load combinations of Section 2.2, shall be combined (both positively and negatively) with hydrostatic hoop stresses, $\sigma_s = N_h/t$, considering Section 12.2.5.1.
In above equations:
Y = distance from liquid surface to analysis point in meters (positive downward).
N_h = hydrostatic hoop force per vertical unit length (N/m). It could be derived by prevalent methods or from equation $N_h = 0.5gD(Y - 0.3)\rho_L$
t = shell thickness at analysis point, excluding corrosion allowance (m)

12.3.14 Soil-Structure Interaction

If necessary regarding Chapter 5, soil-structure interaction shall be considered. When interaction is considered using Chapter 5 requirements, tank shall be mechanically anchored to a strip or mat reinforced concrete foundation. Base shear and overturning moment values for convective mass shall not be taken less than 80% of the corresponding values without interaction. Effective damping ratio for structure-foundation system shall not be taken more than 20%.

12.4 Seismic Design of Ground Cylindrical Steel Tanks [11]

Steel storage tanks shall be designed by ASD method considering Section 12.2.5.1

12.4.1 Overturning

Overturning resistance at the base of the tank can be provided by one of the following measures:
- Shell weight, roof weight and shell adjacent liquid weight for self-anchored tanks
- Anchorage in mechanically anchored tanks

A tank can be classified as self-anchored if following conditions are met:
1. The anchorage ratio, J, is less than one.
2. Width of annular ring, considered in determination of resisting force, does not exceed 3.5 times tank diameter.
3. Pressure stress on shell, is calculated from Section 12.4.2.1
4. Required thickness for annular ring does not exceed lowest shell course thickness.
5. Flexibility of pipes at connection to tank is provided.

Otherwise, a mechanical anchorage shall be used. Anchorage ratio, J, shall be determined from Equation 12.24.

$$J = \frac{M_{rw}}{\frac{\pi}{2}D^2[w_t(1-A_v)+w_a]} \qquad 12.24$$

where M_{rw} is in N.m and:
w_t = weight of the unit length of the shell applied on the bottom (including shell weight and the weight of that part of the fixed roof and roof apparatuses, plus 10% of snow load of roof applied on the shell) (N/m).
w_a= shell adjacent liquid weight (N/m) from Section 12.4.1.1 for self-anchored tank not included for mechanically anchored tanks (Section 12.4.1.2).
For self-anchored tanks with $J > 0.5$, which meaning cases when tank uplift happen, requirements for flexibility of piping and attachments of Sections 12.4.7 to 12.4.9 shall be met.
Requirements for each type of tank are as follows:

12.4.1.1 Self-Anchored Tank

Resistance to overturning moment at the base of the tank shell in self-anchored tanks is provided by the moment initiated from shell weight, roof weight applied on the shell, and shell's adjacent shell liquid weight. Adjacent liquid weight per shell unit length can be calculated from Equation 12.25.

$$w_a = t_a\sqrt{F_y \rho_L g_e H_L} \le 0.02\rho_L g_e H_L D \qquad 12.25$$

$$g_e = g(1-A_v) \qquad 12.26$$

Equation 12.25 is applicable for both constant thickness bottom plate and extended thickness bottom plate (annular ring). Cases where $w_a > 0.02\rho_L g_e H_L D$, value of L shall be taken $0.035D$ and value of w_a shall be taken $0.02\rho_L g_e H_L D$. Smaller values of L compared to Equation 12.27 may be used. Consequently, w_a shall be taken $0.59\rho_L g_e H_L L$.

In the above equations:
t_a = thickness, excluding corrosion allowance, of the bottom annulus under the shell (mm). Where the width of this plate from the inner part of the shell to the inside part of the tank is less than L_{req} from Equation 12.27, t_a shall be replaced by t_b in Equation 12.25 (Figure 12.2).
t_b = thickness, excluding corrosion allowance, of the bottom plate (mm).
F_y = minimum specified yield stress of the bottom annulus (MPa)
g = acceleration of gravity (m/s^2)
g_e = effective specified gravity including vertical seismic effects (m/s^2) according to Equation 12.26.
L_{req} = required width of thickened bottom annular ring measured from the inside of the shell, calculated from Equation 12.27 in m. It is supposed that liquid above this length contributes resisting the overturning moment.

L = width of bottom plate measured from the inside of the shell (m).
t_a shall not be less than t_b in any case.

$$L_{req} = \frac{2+\sqrt{2}}{2} t_a \sqrt{\frac{F_y}{\rho_L g_e H_L}} \leq 0.035D \qquad 12.27$$

Thickness of annulus under shell is permitted to be more than the thickness of the shell at the base, but t_a used in Equation 12.25 shall not be taken more than shell thickness at base (excluding corrosion allowance).

If $L < L_{req}$, in Equation 12.25, t_a shall be replaced by t_b

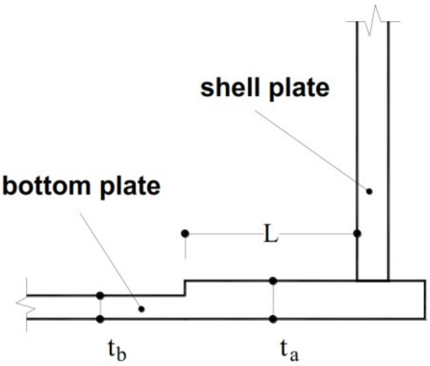


Figure 12.2 Dimensions of Shell Annulus and Bottom Plates

12.4.1.2 Mechanically Anchored Tank

In mechanically anchored tanks, overturning resistant moment is provided by anchorage only, considering requirements of this section.
Seismic anchor force per unit length of tank perimeter, w_{AS}, shall be calculated from Equation 12.28.

$$w_{AS} = \frac{4M_{rw}}{\pi D^2} + A_v w_t \qquad 12.28$$

In ASD method, anchor design force, w_{Aa}, per unit length of tank perimeter (N/m) is the maximum value calculated from Equations 12.29a and 12.29b, which corresponds to load combination including earthquake effects, and earthquake and tank design pressure, respectively.

$$w_{Aa} = 0.75(w_{AS} - w_t) \qquad 12.29a$$

$$w_{Aa} = 0.75\left[(F_P P_i - 80t_r)\frac{D}{4} + \frac{4M_{rw}}{\pi D^2}\right] - 0.75 w_{tr}(1 - A_v) \qquad 12.29b$$

where:
F_P = pressure combination factor (ratio of operation pressure to design pressure)
P_i = inside design pressure (Pa)
t_r = roof plate thickness (mm)
w_{tr} = weight of unit perimeter length, applied from shell to bottom (including shell weight, without fixed roof weight).
In case of using n_A circumferential anchor bolts or anchor strips with equal distances, design force for each anchor shall be calculated from Equation 12.30.

$$P_{Aa} = w_{Aa}\left(\frac{\pi D}{n_A}\right) \qquad 12.30$$

When ASD method is used, allowable stress of the anchor bolts or anchor strips, and shell plate at attachment to bottom plate, shall not be more than 80% of the specified yield stress.
Anchor attachments to the shell and foundation shall be designed for the minimum anchor bolt or strip yield force (equal to as-built cross-sectional area of the anchor multiplied by the minimum specified yield stress) and also $\Omega_0 P_{Au}$ (AS method) or $1.5\Omega_0 P_{Aa}$ (ASD method). Increase in anchor cross-sectional area for corrosion or other factors shall be considered in anchor attachment design.

12.4.2 Maximum Longitudinal Shell Compressive Stress

12.4.2.1 Self-Anchored Tank

The maximum seismic longitudinal compressive stress in self-anchored tank at the shell bottom shall be determined by Equations 12.31:

$$\sigma_{cs} = \left[A_v w_t + \frac{4M_{rw}}{\pi D^2}\right]\frac{1}{1000t_s} \qquad when\, J \leq 0.5 \qquad 12.31a$$

$$\sigma_{cs} = \left[\frac{w_t(1+A_v)+w_a}{0.607-0.527J^{2.3}} - w_t - w_a\right]\frac{1}{1000t_s} \qquad when\, J > 0.5 \qquad 12.31b$$

If $J \leq 0.5$, tank will not have a computable uplift. If $0.5 < J \leq 1$, tank will uplift, but stays stable, provided that shell compressive stress does not exceed allowable stress. Considering load combination 2.6b, total shell longitudinal stress, σ_c (MPa), in ASD method shall be determined from Equation 12.32.

$$\sigma_c = 0.75\sigma_{cs} + \frac{w_t}{1000t_s} \qquad 12.32$$

where:
t_s = thickness, excluding corrosion allowance, of the lowest shell course (mm).

12.4.2.2 Mechanically Anchored Tank

The maximum seismic longitudinal compressive stress in a self-anchored tank at the bottom of the shell shall be determined by Equation 12.31a and the total shell longitudinal stress in ASD method shall be determined from Equation 12.32.

12.4.3 Allowable Shell Vertical Compressive Stress

The maximum longitudinal shell compressive stress, Equation 12.32, shall be less than the allowable stress F_C (MPa):

$$F_C = \frac{62t_s}{D} \qquad \frac{\rho_L g H_L D^2}{t_c^2} \geq 440000 \qquad 12.33a$$

$$F_C = \frac{62t_s}{2.5D} + 0.056\sqrt{\rho_L g H_L} \leq 0.375F_{ty} \qquad \frac{\rho_L g H_L D^2}{t_c^2} < 440000 \qquad 12.33b$$

In above equations:
F_C = Allowable longitudinal shell compressive stress (MPa).
t_C = thickness of shell course under consideration (mm).
F_{ty} = minimum specified yield stress of the shell course (MPa).
In Equations 12.33, H_L and D are in meters, ρ_L is in kg/m^3 and g is in m/s^2. In determination of F_C, internal pressure effects of contained liquid are considered.

If the thickness of the bottom shell course calculated to resist the seismic overturning moment is greater than the thickness required for hydrostatic pressure, then the calculated thickness of each upper shell course for hydrostatic pressure may be increased in the same proportion (ratio of the bottom shell course thickness, calculated considering the overturning moment, to the calculated thickness only for hydrostatic pressure). In case a separate analysis is performed to determine the compressive stress (considering overturning moment) at lowest point of each shell course, it can be used to control the thickness of corresponding course. Corrosion allowance thickness shall be added to the calculated thicknesses afterward.

12.4.4 Allowable Hoop Stresses

The maximum seismic hoop stress, according to Section 12.3.13, after combining with hydrostatic hoop stress considering Section 12.2.5.1, shall not be greater than the lesser of the allowable shell stress (in the referred standard without 33% increase) and $0.67F_y$ times the shell joint efficiency factor, where F_y is the lesser of the yield stress of the shell material or weld material. Joint efficiency factor, where radiography tests are performed is set to 0.85, otherwise 0.7.

12.4.5 Mechanical Anchorage Detailing Requirements

When mechanical anchorage is required, at least six anchors shall be provided. Spacing between anchors shall not exceed 3 m. In tanks with a diameter less than 15 m, maximum distance between anchors is 1.8 m. When anchor bolts are used, they shall have a minimum diameter of 25 mm, excluding any corrosion allowance. Carbon steel anchor straps shall be 6 mm minimum thickness and have a minimum corrosion allowance of 1.5 mm on each surface. An un-embedded length between 75 mm and 300 mm shall be provided

Provided of anchor strength in concrete should exceed anchor yield strength. Hooked anchor bolts (L- or J-shaped embedded bolts) or other anchorage systems based solely on bond or mechanical friction shall not be used. End plates are allowed to be used in order to provide required strength for anchor bolts or straps. 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 and meet the requirements of valid references.

12.4.6 Local Shear Transfer

Capacity for local transfer of the shear from the roof to the shell and from which to the base shall be provided. For cylindrical tanks, the peak local tangential shear per unit length, V_{max}, shall be calculated by Equation 12.34.

$$V_{max} = \frac{\Omega_0 V_u}{\pi D} \qquad 12.34$$

Tangential shear shall be transferred through the welded connections to the steel bottom. The shear stress in the weld shall not exceed 80% of the weld metal yield stress and 90% of the base metal yield stress. V_u shall be calculated from Equation 12.6, excluding mass of the bottom plate, m_f.

12.4.7 Piping System Flexibility

When designing piping systems connected to tanks, the potential movement of the connection points during earthquakes shall be considered and sufficient flexibility to avoid release of the product due to failure of the piping system shall be provided. The piping system and supports shall be designed so as to not 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.

Piping systems shall be designed so that for the minimum displacements in Table 12.4, resultant stresses in the piping, supports and tank connection, in combination with other load effects according to Section 12.2.5.1, do not exceed allowable stresses (without the 33% increase for allowable stresses). Where precise analyses are done, displacement results are permitted to be used instead of design piping system. For attachment points located above the support or foundation elevation, the displacements in Table 12.4 shall be increased to account for drift of the tank. The values given in Table 12.4 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. The maximum elastic uplift of a self-anchored tank bottom, Y_u, in mm may be approximated by Equation 12.35, considering Section 12.4.1.

$$Y_u = \frac{12.1 F_y L^2}{t_b} \qquad 12.35$$

The piping system and tank connection shall also be designed to tolerate the design displacement ($1.4C_d$ times the displacements given in Table 12.4 or Equation 12.35), without rupture, although permanent deformations and inelastic behavior in the piping supports and tank shell is permitted.

Table 12.4 Elastic Design Displacements of the Shell Bottom Course from Support or Foundation

Type of Tank	**Elastic Design Displacement (mm)**
Mechanically anchored tank	
Upward displacement	25
Downward displacement	13
Horizontal displacement (radial or tangential)	13
Self-anchored tank	
Upward displacement	
Anchorage ratio less than 0.5	25
Anchorage ratio more than 0.5	100
Downward displacement	
Tank with a ringwall or mat foundation	13
Tank with a berm foundation	25
Horizontal displacement (radial or tangential)	50

12.4.8 Connections to Adjacent Structures

Equipment, piping, and walkways or other appurtenances attached to the tank or adjacent structures shall be designed to accommodate the design seismic displacements of the tank plus the displacement of the other structure, considering Section 8.2.

12.4.9 Connections

Connections and attachments for anchorage against lateral forces shall be designed to develop the yield strength of the anchor (based on minimum specified yield stress in direct tension or plastic bending moment), or 4 times the calculated element loads.

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 pipe bottom connection to a self-anchored flat-bottom tank shall be located inside the shell with a minimum distance of L_{req} (Section 12.4.1.1) plus 300 mm.

12.4.10 Internal Components

The attachments of internal equipment and accessories, which are attached to the primary liquid or pressure-retaining shell or bottom, shall be designed for the lateral loads due to the sloshing liquid in addition to the inertial forces. To include convective liquid effects, pressure distribution from Equation 12.19 can be used.

For seismic design of roof framing, vertical component of ground motion, Section 12.3.10, shall be considered. For seismic design of internal columns and their attachments, seismic lateral force from convective liquid shall be considered. To do so, pressure distribution from Equation 12.19 with $\theta = 0$ is permitted to be used.

12.5 Seismic Design of Ground Concrete Tank

12.5.1 Design Method

Reinforced or prestressed concrete tank can be designed and constructed buried, semi-buried or ground-supported. Response modification factor, over-strength factor, and lateral deflection amplification factor for ground concrete tanks shall be determined from Table 12.3. In non-flexible tanks, wall-base joint can be fixed or hinged. Figure 12.3 shows a non-flexible wall tank with both fixed and hinged joint connections. For flexible wall tanks, to limit lateral sliding, lateral anchorage or restraints can be used. In addition, these tanks are permitted to be designed without any anchorage or restraint (Figure 12.4).
The convective period for all types of tanks and impulsive period for non-flexible tanks shall be calculated from Section 12.3.2 and impulsive liquid period for flexible tanks shall be derived from Section 12.5.2.
For buried or semi-buried tanks, dynamic soil pressure forces shall be included when having an exceeding effect, without including reducing effects of liquid pressure. Dynamic soil pressure can be assumed A_i times the static soil pressure, where A_i is calculated from Equation 12.15.

12.5.2 Fundamental Period of Flexible Base Tank

Fundamental period of the impulsive mass of a flexible base tank shall be determined from Equation 12.36, but shall not be taken more than 1.25 sec.

$$T_i = 2\pi\sqrt{\frac{2(m_s + m_r + m_i)}{\pi D k_a}} \qquad 12.36$$

where:
k_a = horizontal stiffness per unit wall length, from Equation 12.37

$$k_a = \frac{A_s E_s cos^2\alpha_a}{L_s S_s} + \frac{2G_p w_p L_p}{t_p S_p} \qquad 12.37$$

where:
A_s = cross sectional area of wall-base anchorage cable, strand or reinforcement
E_s = modulus of elasticity of anchorage
L_s = effective length of anchorage taken as 35 times the anchor diameter plus the sleeve length between wall and base.
S_s = center-to-center spacing between consecutive anchor loops at wall perimeter
G_p = shear modulus of elastomeric bearing pad
w_p = width of elastomeric bearing pad
L_p = length of elastomeric bearing pad
S_p = center-to-center spacing between elastomeric bearing pads
t_p = thickness of elastomeric bearing pad
α_a = angle of anchor with horizontal direction

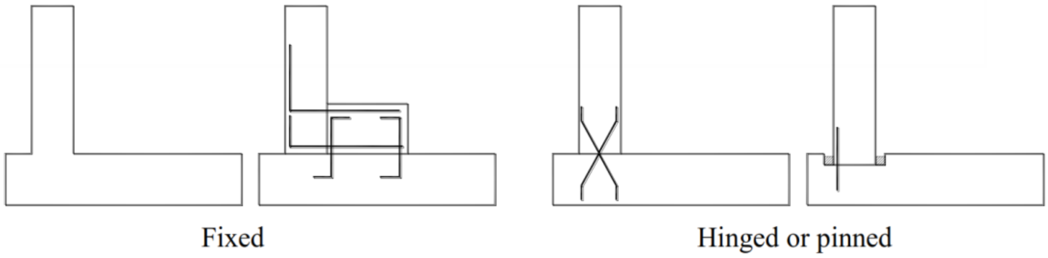


Figure 12.3 Non-Flexible Base Connections

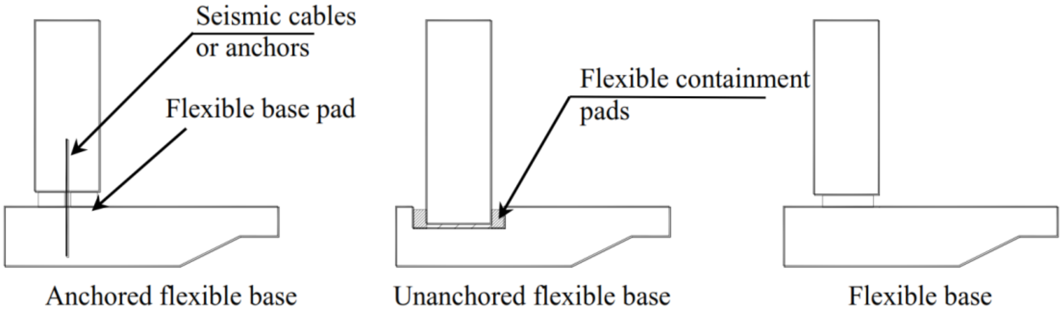


Figure 12.4 Flexible Base Connections

12.6 Seismic Design of Ground Tank Foundation

12.6.1 Design Principles

Tank foundation shall be designed for earthquake loads according to Sections 12.1.4 and 12.3.4. For steel tanks, ultimate seismic moment on an annular foundation and on a mat foundation shall be calculated from Equations 12.9 and 12.12, respectively.

12.6.2 Design Forces for Cylindrical Steel Tank Foundation

Foundations for mechanically anchored tanks shall be proportioned to resist peak anchor uplift and overturning bearing pressure. The liquid and soil load directly over the ringwall and footing may be used to resist the maximum anchor uplift on the foundation, provided the ringwall and footing are designed to carry this eccentric loading. The liquid weight shall not be used to reduce the anchor load.
When the liquid weight directly over the ringwall and footing is used to resist the anchor uplift on the foundation, vertical component of the earthquake shall be considered by multiplying a factor of $(1 - A_v)$ by liquid weight. In foundation design, the foundation ringwall and footing shall be designed to resist the eccentric loads with and without the vertical seismic accelerations. When used to evaluate the bearing (downward) load, liquid pressure over the ringwall shall be multiplied by a factor of $(1 + A_v)$ and the foundation ringwall and footing shall be designed to resist the eccentric loads with and without the vertical seismic accelerations.
The overturning stability for a mechanically anchored tank system excluding vertical seismic effects, defined in Equation 12.38, shall be established.

$$\frac{0.5Dg\left(m_p + m_t + m_{fd} + m_g\right)}{M_s} \geq 2.0 \qquad 12.38$$

where:
m_t = total mass of empty tank including shell, roof, bottom and attached parts.
m_{fd} = total mass of foundation
m_g = mass of soil directly over foundation
m_p = total mass of the liquid

M_s shall be determined from Equation 12.12.
Ringwalls for self-anchored tanks shall be designed to resist seismic compression force, P_{fs} , from Equation 12.39, per unit length of the ringwall, and longitudinal force caused by weight of the shell, w_t.

$$P_{fs} = \frac{4M_{rw}}{\pi D^2} + A_v w_t \qquad 12.39$$

Soil pressure check in ASD method shall be based on Section 12.2.5.1.
Where foundation is designed using SD method, applied pressure per unit length on the ringwall, P_f shall be calculated from Equation 12.40.

$$P_f = P_{fs} + 1.2w_t \qquad 12.40$$

12.6.3 Sliding Resistance in Flat Bottom Tanks

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 , shall be determined from tests or valid references, based on tank bottom and foundation or subgrade materials, but shall not exceed 0.56. Self-anchored storage tanks shall be proportioned such that the calculated seismic base shear, V_u from Equation 12.6, does not exceed V_s from Equation 12.41:

$$V_s = \mu_f g\left(m_s + m_r + m_f + m_p\right)(1 - A_v) \qquad 12.41$$

Lower values of μ_f should be used if the interface of the bottom to supporting foundation does not justify the above friction value above (e.g., when a leak detection membrane exists beneath the bottom with a lower friction factor, smooth bottoms, etc.). For mechanically anchored steel tanks, sliding control is not necessary, for sliding values up to 25 mm are probable.

12.6.4 Shell Support of Steel Tanks

Self-anchored tanks resting on concrete ringwalls or slabs shall have a uniformly symmetric supported annulus under the shell. Uniform support shall be provided by one of the following methods:
A. Shimming and grouting the annulus
B. Using fiberboard or other suitable padding
C. Using double butt-welded bottom or annular symmetric plates resting directly and uniformly on the foundation,
D. Using closely spaced shims (without structural grout) provided that the localized bearing loads be considered in the tank wall and foundation to prevent local crippling and spalling.
In methods B, C and D, the foundation level allowance shall be less than permitted allowance range.
Mechanically anchored tanks shall be shimmed and grouted.

12.6.5 Earth Foundation without Ringwall

For low-height tanks where soil has an adequate bearing capacity and settlements are acceptable, satisfactory foundations may be constructed from earth materials. Crushed stone combined with compacted sand and gravel can be used. Earth foundation shall provide sufficient resistance for seismic forces mentioned in this chapter.

12.6.6 Earth Foundation with a Concrete Ringwall

For large tanks and tanks with heavy or tall shells and/or self-supported roofs, which impose a substantial load on the foundation under the shell, an earth foundation with a concrete ringwall can be used. As an alternative to the concrete ringwall noted in this section, a crushed stone ringwall may be used. Ringwall thickness shall be designed and constructed so that the differential settlements between middle and side parts of the tank bottom stay limited. Greater settlements may lead to high bending stresses in the bottom plate adjacent to the ringwall. In a concrete ringwall, reinforcement shall be determined through concrete foundation design procedure, while minimum temperature and shrinkage reinforcement shall be provided.

The ringwall shall be reinforced to resist the direct hoop tension resulting from the lateral earth pressure on the ringwall's inside face. This lateral earth pressure shall be assumed to be at least 50% of the vertical pressure.

The ringwall shall be reinforced to resist the bending and torsion moments and shear forces resulting from lateral loads. Usually, the overturning moment from Equation 12.9 will cause an increase in ringwall thickness, compared to the thickness designed for vertical loads. This increase may have a disadvantage of increasing differential settlement at tank bottom in vertical loads. To avoid it, effects of thickness increase on foundation behavior under vertical loads shall be studied and practical solutions considering site conditions shall be provided.

12.6.7 Concrete Slab Foundation for Ground Tanks

For low bearing capacity soils or soils having high potential for settlement, a reinforced concrete slab can be used. Piles beneath the slab may be required for proper load transmission to lower soil layers. To analyze concrete slabs, plate on elastic foundation models with common software can be used. Where using equivalent static procedure, Equation 12.12 shall be used to determine transmitted bending moment to the slab.

12.7 Seismic Design of Elevated Tanks

12.7.1 Design Requirements

Supporting structure and anchors shall be designed considering Chapter 7 requirements. Pipes and attached apparatuses shall be designed considering the requirements of Chapter 8. Importance factor for elevated tanks is defined in Section 12.2.4.

Seismic coefficients for elevated tanks are derived from Table 7.1 or 7.2. Structures supporting equipment other than elevated tanks shall be designed regarding requirements of Chapters 7 and 8.

Anchor attachments for steel elevated tanks on concrete pedestals shall be designed for Ω_0 times the calculated anchor force.

12.7.2 Analysis Procedure

Where the total weight of the main supporting pier is more than 10% of the total full tank, dynamic analysis shall be done; otherwise, equivalent static procedure is permitted.

12.7.3 Lateral Deformation and P-Delta Effects

Vertical load, P, used to determine stability index, Section 4.15, includes 1.2 times of the total dead weight, total live load, 1.2 times the maximum liquid weight in tank and 0.2 times the snow load. In Equation 4.26, Δ is the linear displacement of the center of mass of full tank corresponding to the base shear V_u, and height of the center of mass over the foundation level, h.

If supporting pier is a multi-story structure (with or without lateral bracing), stability index for each story shall be checked regarding requirements of Chapter 4. To determine deformation and period for tanks with concrete legs, uncracked gross section moment of inertia, I_g, without any reduction is permitted to be used.

Chapter 13

Pipeline

13. Pipeline

13.1 General

Preliminary design of pipeline is usually performed to fulfill some processing and mechanical needs such as pressure, temperature, fluid type, etc., which are out of scope of this code. In this chapter, seismic check and design of pipeline for seismic hazards are presented.

Pipelines are categorized into continuous and segmented types. Steel welded pipelines are considered as continuous whereas segmental pipelines include cast iron pipeline with washer joints, ductile iron pipeline with rubber washer, asbestos pipeline, etc.

Pipeline shall also be controlled for any probable seismic hazard, which this chapter deals with. Analysis procedures and general design criteria of pipeline systems for some of general seismic hazards are presented in this chapter. For specific local hazards, seismic evaluation shall be based on Site-Specific Hazard Study research reports.

In seismic hazardous sites, preparations for fluid cut-off and rapid replacement of damaged parts of pipeline shall be considered.

In Section 13.2, Function & Risk Category of pipeline system are specified. Details of seismic loading and analysis procedures for buried pipelines are presented in Section 13.3, above ground pipelines are presented in Section 13.4, and pipelines supported on a structure are presented in Section 13.5.

Pipeline seismic analysis may be performed either with Equivalent Lateral Load Procedure, as mentioned in Section 13.3.1 and 13.4.1, or for more precise dynamic procedure, with the requirements of Section 13.3.2 and 13.4.2.

13.2 Function & Risk Category

Regarding function & risk, pipelines are categorized into four categories:

Function & Risk Category I: Pipeline with very low importance, damage in which will pose negligible effects on life safety, environment and operation of facilities, without emergency needs to repair.

Function & Risk Category II: Pipeline with ordinary usage, excluding pipelines in Function & Risk Category I, III and IV, such as low-pressure oil or gas pipeline.

Function & Risk Category III: Pipeline with important usage, including pipeline that its damage can pose a risk to people. Main distribution pipeline and pipeline with a high economical loss if damaged, such as medium pressure oil and gas pipeline, vitally providing energy but able be stopped for short period maintenance objectives, lays in this category.

Function & Risk Category IV: Pipeline with essential usage, including high pressure or temperature pipelines with flammable fluids or toxic materials, pipeline that shall stay operational during and after design earthquake such as fire pipelines, and pipeline that its damage can pose severe casualties or huge environmental losses, lays in this use category.

13.3 Buried Pipeline

Seismic hazards directly related to buried pipeline damage can be classified as follows:

1. Wave propagation
2. Permanent ground deformation due to:
    - Faulting
    - Landsliding
    - Liquefaction (Including subsidence, lateral spreading and uplift)

Analysis of buried pipeline for earthquake waves and large ground deformations shall be done according to requirements of this section. Design earthquake for Function & Risk Categories II, III and IV has a return period of 475, 975 and 2475 years, respectively. In addition, it is permitted to use design earthquake (Seismic Hazard Level II from Section. 3.4.2) including velocity, acceleration and displacement for all Function & Risk Categories, by applying importance factors, I_L, from Table 13.1. For Function & Risk Category I, there is no need to consider seismic provisions.

Table 13.1 Pipeline Importance Factor, I_L, for Different Function & Risk Categories

Function & Risk Category	Wave Propagation	Faulting	Ground Permanent Longitudinal and Transverse Deformations	Landsliding
IV	1.5	2.3	1.5	2.6
III	1.25	1.5	1.35	1.6
II	1.0	1.0	1.0	1.0

Generally, it is suggested to benefit from the inelastic capacity of pipeline, although critical parts that can cause widespread casualties or environmental losses shall stay in zone of linear behavior.
If a specific stress-strain relation of pipeline materials is not selected, Equation 13.1 is permitted to be used as a reliable estimation:

$$\varepsilon = \frac{\sigma}{E_p}\left[1 + \frac{n}{1+r}\left(\frac{\sigma}{\sigma_y}\right)^r\right] \qquad 13.1$$

where:
ε = strain
σ = stress
E_p = primary modulus of elasticity of pipe material
σ_y = yield stress of pipe material
n, r = behavioral parameters of pipeline material which for some of standard pipelines in Reference [12] can be found in Table 13.2.
For other types of pipelines, model parameters shall be derived from test or valid references.

Table 13.2 Parameters of Steel Pipes

Grade of Pipe	**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 service loads application, seismic load is applied to pipeline and then resulted strains are calculated. These strains shall be less than allowable values.
Seismic stresses (or strains) shall be combined with stresses (or strains) resulted from internal pipe pressure or temperature, from Equations 13.2 and 13.3.
The longitudinal stress in pipe due to internal pressure, S_P, may be calculated as:

$$S_P = \frac{P_p D \nu}{2 t_p} \qquad 13.2$$

where:
P_p = maximum internal operating pressure of the pipe
D = outside diameter of the pipe
ν = Poisson's ratio (generally taken as 0.3 for steel)
t_p = nominal wall thickness of the pipe
The longitudinal stress in pipe due to temperature variation, S_r, may be calculated as:

$$S_r = E_p \alpha_t (T_2 - T_1) \qquad 13.3$$

where:
α_t = linear coefficient of thermal expansion of pipe material

T_1 = temperature in pipe at the time of installation
T_2 = temperature in pipe at the time of operation
The maximum allowable strains for buried continuous pipelines conforming to Reference [12] are specified in Table 13.3. For other types of pipes, the allowable strain limit provided by the manufacturer and approved by authorities may be used.

Table 13.3 Allowable Strain Criteria for Buried Pipelines

Pipeline Application	Pipe Type	Allowable Strain	
		Tension	**Compression**
Continuous Oil and Gas Pipelines	Ductile Cast Iron Pipe	2%	For PGD: Onset of wrinkling (ε_{cr-c}) For wave propagation: 50% to 100% of the onset of wrinkling
	Steel Pipe	3%	
	Polyethylene Pipe	20%	
	Bends and Tees of Pipe	1%	
Continuous Water Pipelines	Steel and Iron Pipe	$0.25\ \varepsilon_u{}^1\ or\ 5\%$	For PGD: ε_{c-PGD} For wave propagation: ε_{c-wave}

[1] ε_u is the ultimate tensional strain of pipe material

The limiting compressive strain is considered as the strain at onset of wrinkling, ε_{cr-c}:

$$\varepsilon_{cr-c} = 0.175\frac{t_p}{R} \tag{13.4}$$

where:
R = outside radius of pipe
Other values presented in Table 3.13 are determined from Equations 13.5 to 13.7

$$\varepsilon_{c-PGD} = 0.88\frac{t}{R} \tag{13.5}$$

$$\varepsilon_{c-wave} = 0.75\left[0.5\frac{t}{D'} - 0.0025 + 3000\left(\frac{P_p D}{2E_p t}\right)^2\right] \tag{13.6}$$

$$D' = \frac{D}{1 - \frac{3}{D}(D - D_{min})} \tag{13.7}$$

where:
t = pipe wall thickness
D_{min}= minimum inside diameter of pipe with consideration of roughness and distortion (See Figure 13.1)

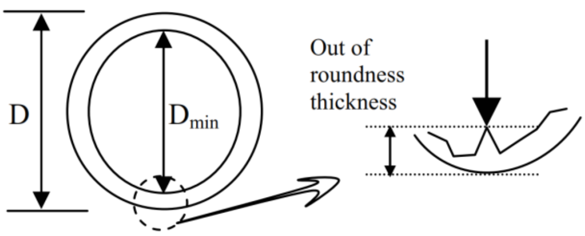


Figure 13.1 Determination of D_{min} [13]

Design strain for a continuous pipeline shall be less than allowable strain:

$$\varepsilon_{seismic} + \varepsilon_{oper} \leq \varepsilon_{allowable} \tag{13.8}$$

$\varepsilon_{allowable}$ = allowable strain of pipe according to Table 13.3
$\varepsilon_{seismic}$ = design strain of pipe due to seismic hazards
ε_{oper} = operational strain in the pipe equal to $\varepsilon_p + \varepsilon_t + \varepsilon_{D+L}$
ε_p = operational strain in the pipe due to internal pressure
ε_t = operational strain in the pipe due to temperature variation
ε_{D+L} = operational strain in the pipe due to gravity loads

For segmented pipelines, maximum displacement of joints between various parts of the pipe shall be less than allowable displacement according to Equation 13.9:

$$\Delta_{oper+seismic} \leq \Delta_{allowable} - \Delta_a \tag{13.9}$$

where:
$\Delta_{allowable}$ = allowable displacement of joint determined and presented by joint supplier
Δ_a = an allowance about 6 mm to cover the additional safety for pipe joint
$\Delta_{oper+seismic}$ = maximum displacement of joint due to service and seismic loads
Δ_{oper} = maximum service displacement of joint equal to $\Delta_p + \Delta_t + \Delta_{D+L}$
Δ_p = joint deformation due to internal pressure
Δ_t = joint deformation due to temperature variation
Δ_{D+L} = joint deformation due to gravity loads

For calculating the deformation due to service loads, gravity load, temperature and pressure effects shall be considered. In Equation 13.9, values of joint deformation are calculated as product of pipeline strain to pipe segment length.

13.3.1 Analysis for Wave Propagation in Equivalent Seismic Load Procedure

Generally, longitudinal axial strain of pipeline is defined as the seismic response to wave propagation. Bending strain due to ground curvature, regarding its low value, is negligible.
In this code, maximum ground velocity for specified Function & Risk Category, is considered as seismic design parameter. Peak ground design velocity for each category, V_g, can be derived according to Function & Risk Category and related return period corresponding to Section 13.2 or it could be determined from Equation 13.10:

$$V_g = V_{g0} I_L \tag{13.10}$$

where:
V_{g0} = peak ground velocity for a specific location in Seismic Hazard Level II according to Section 3.4.2
I_L = importance factor from Table 13.1

13.3.1.1 Pipeline Strain Determination for Wave Propagation

Axial strain of continuous pipeline caused by seismic waves is approximated by wave propagation velocity. As a general rule, to determine this strain, shear wave (S wave) velocity for areas within the epicentral distance of 5 times focal depth is used. Otherwise, the velocity of Rayleigh wave (R wave) is considered.

The maximum longitudinal axial strain, ε_a, that can be induced in the pipeline due to wave propagation, can be approximated from Equation 13.11.

$$\varepsilon_{seismic} = \frac{V_g}{\alpha_\varepsilon C} \leq \frac{t_u \lambda_e}{4 A_p E_p} \qquad 13.11$$

where:
α_ε = ground strain coefficient (2 for S wave and 1 for other types of waves)
C = velocity of seismic wave propagation (in lack of precise information, 2 km/sec is permitted to be used for S wave)
t_u = peak frictional force per unit length at soil-pipe interface, from Equation 13.12
λ_e = apparent wavelength of seismic waves at ground surface (often taken as 1 km in the absence of detailed information)
A_p = cross sectional area of 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 (MPa)
α_s = non-dimensional adhesion factor between soil and pipe, calculated from Equation 13.13:

$$\alpha_s = 0.608 - 1.23c - \frac{0.274}{1 + 100c^2} + \frac{0.695}{1 + 1000c^3} \qquad 13.13$$

$\bar{\gamma}$ = effective unit weight of soil
H_s = height of soil above the center of the pipe
k_0 = coefficient of soil pressure at rest
δ = interface angle of friction between pipe and soil, permitted to be taken as $f \times \phi$, where ϕ is the internal friction angle of the soil and f is friction factor between soil and pipe. Some values for f are suggested in Table 13.4.

Table 13.4 Friction Factor between Soil and Pipe

Pipe Coating	Friction Factor
Concrete	1.0
Coal Tar	0.9
Rough Steel	0.8
Smooth Steel	0.7
Fusion Bonded Epoxy	0.6
Polyethylene	0.6

For segmented pipelines, deflection at joint is controlled by Equation 13.9. $\Delta_{oper+seismic}$ value in this equation may be determined from summation of the service and seismic load displacements. In addition, joint rotation is determined from Equation 13.14.

$$\theta_{seismic} = 1.5 \frac{A_g}{C^2} L_0 \qquad 13.14$$

where:

A_g = peak ground acceleration in direction normal to wave propagation due to design level earthquake
L_0 = pipe segment length between two joints
Allowable joint rotation is specified and represented by supplier.

13.3.2 Analysis for Wave Propagation in the Dynamic Procedure

In dynamic modeling of pipeline, values such as nonlinear soil behavior parameters, wave propagation velocity in soil, and site governing frequency along pipeline route should be derived from valid procedures.

13.3.3 Analysis for Permanent Fault Displacement

It is strongly recommended that pipeline routes do not cross active faults. Where fault crossing for pipeline is inevitable, following issues should be cleared:

1. Fault mechanism (Slip, normal, reverse, oblique strike)
2. Fault activity and seismicity rate
3. Width and extension of faulting area
4. Strike direction with respect to pipeline route
5. Vertical or horizontal displacement magnitude proportionate to seismic hazard level

After specifying above issues, it is strongly recommended that pipeline is placed on fault in a manner that fault movement does not impose compression on pipeline. Empirical Equations 13.15a to 13.15d or other appropriate relationships from valid references can be used to determine probable fault displacements.

$$log\delta_{fs} = -6.32 + 0.90M_w \tag{13.15a}$$

$$log\delta_{fn} = -4.45 + 0.63M_w \tag{13.15b}$$

$$log\delta_{fr} = -0.74 + 0.08M_w \tag{13.15c}$$

$$log\delta_{fb} = -4.80 + 0.69M_w \tag{13.15d}$$

where:
δ_{fs}= strike slip fault displacement (m)
δ_{fn}= normal slip fault displacement (m)
δ_{fr}= reverse slip fault displacement (m)
δ_{fb}= displacement of a blind fault (m)
M_w= moment magnitude of earthquake

13.3.3.1 Displacement of Pipeline Crossing a Strike Slip Fault

For a strike slip fault, the fault movement along and transverse to the pipeline, δ_{fax} and δ_{ftr}, can be calculated from Equation 13.16.

$$\delta_{fax} = \delta_{fs}cos\beta \tag{13.16a}$$

$$\delta_{ftr} = \delta_{fs}sin\beta \tag{13.16b}$$

where:
β= angle of pipeline crossing the fault line (Figure 13.2)

13.3.3.2 Displacement of Pipeline Crossing a Normal Slip Fault

For a normal slip fault, the fault movement along, transverse and vertical to the pipeline, δ_{fax}, δ_{ftr}, and δ_{fyt} can be calculated from Equation 13.17.

$$\delta_{fax} = \delta_{fn} cos\psi . sin\beta \tag{13.17a}$$

$$\delta_{ftr} = \delta_{fn} cos\psi . cos\beta \tag{13.17b}$$

$$\delta_{fvt} = \delta_{fn} sin\psi \tag{13.17c}$$

where:
β= angle of pipeline crossing the fault line (Figure 13.3)
ψ= dip angle of the fault (Figure 13.3)

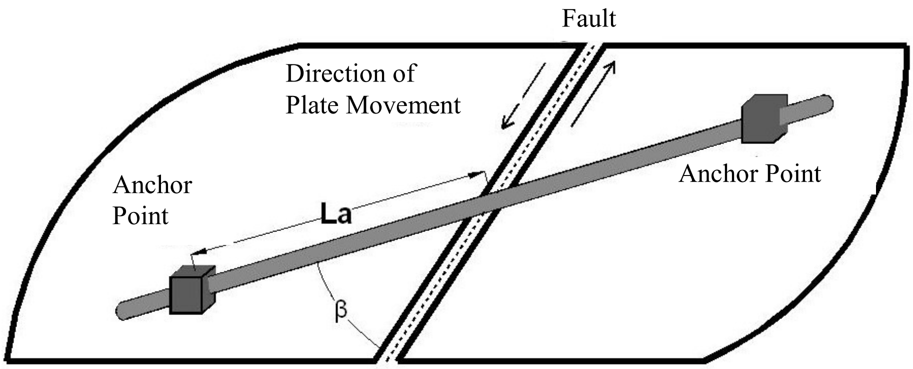


Figure 13.2 Pipeline Crossing a Strike Slip Fault

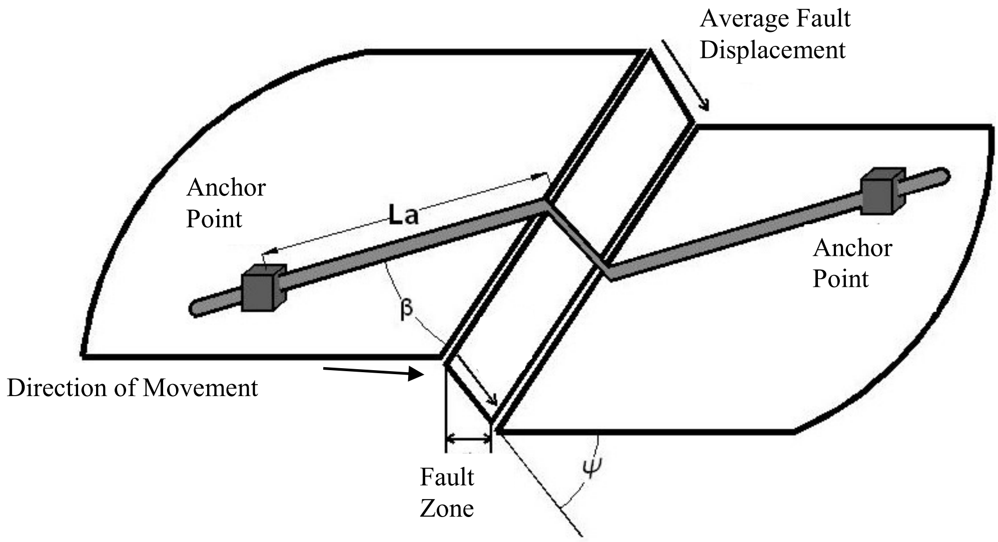


Figure 13.3 Pipeline Crossing a Normal Slip Fault

13.3.3.3 Displacement of Pipeline Crossing a Reverse Fault

In reverse faults, the displacement components are evaluated in the similar way as for the normal-slip fault, but with a negative slip ψ.

13.3.3.4 Displacement of Pipeline Crossing an Oblique Fault

For oblique faults, the strike slip displacement and normal slip (or reverse slip) displacement may be added algebraically in axial, transverse and vertical directions to the pipeline axis.

13.3.3.5 Design Fault Displacement

Design fault displacement can be evaluated by multiplying the importance factor (Table 13.1) with the expected fault displacement, as mentioned in Equation 13.18.

$\delta_{fax-design} = \delta_{fax} I_L$ design fault displacement in the axial direction of pipeline 13.18a

$\delta_{ftr-design} = \delta_{ftr} I_L$ design fault displacement in transverse direction of pipeline 13.18b

$\delta_{fvt-design} = \delta_{fvt} I_L$ design fault displacement in vertical direction of pipeline 13.18c

13.3.3.6 Strain in Pipe due to Fault Crossing

The average seismic strain of pipe due to fault crossing can be calculated from Equation 13.19.

$$\varepsilon_{seismic} = 2\left[\frac{\delta_{fax-design}}{2L_a} + \frac{1}{2}\left(\frac{\delta_{ftr-design}}{2L_a}\right)^2\right] \qquad 13.19$$

where:
L_a= unanchored pipe length that in fault crossing zone can be assumed to be the least of two following values:

a. Where there are no constraints such as bends, attachments etc. at fault crossing zone, the effective unanchored length of pipeline can be calculated from Equation 13.20.

$$L_a = \frac{\pi D t_p E_p \varepsilon_y}{t_u} \qquad 13.20$$

where:
ε_y= yield strain of pipe material

b. Any constraint (such as bends, knees, change in the soil above pipe, etc.) shall be considered as an anchor point. Length of pipeline from such anchor point to the fault line is considered as effective unanchored length.

Average seismic strain of pipe at fault crossing location is supposed to be the pipe design strain, $\varepsilon_{seismic}$, and shall satisfy the values of allowed strain from Table 13.3.
The factor 2 in Equation 13.19 is safety factor and used to counterbalance the non-conservatism involved in this model. The foregoing equation is just for initial approximation and the detailed design should be based on suitable nonlinear analysis.
In segmented pipelines, the fault offset is assumed to be accommodated equally by pipe joints located on each side of the fault line. The design displacement of the joints can be calculated by Equation 13.21 and compared with values represented by the supplier.

$$\Delta_{seismic} = \delta_{fax} I_L \qquad 13.21$$

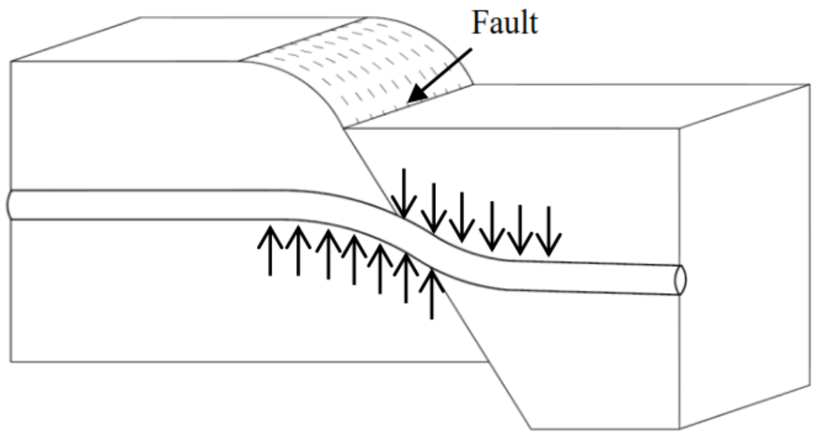
13.3.3.7 Finite Element Method

In this method, by means of an appropriate software and considering models including nonlinear behavior of soil and pipe materials and large displacements (geometrical nonlinearity), pipeline can be analyzed. By applying displacement to any desired point of the pipe-soil system as input, the effect of fault displacement can be considered. Utilizing such a software needs a good knowledge of soil and structure nonlinear behavior and applied aspects of finite element method.
To gain accurate results, a sufficient length of the pipeline at both sides of the fault shall be involved. To model soil behavior, from valid references, equivalent bulk elements or nonlinear springs can be

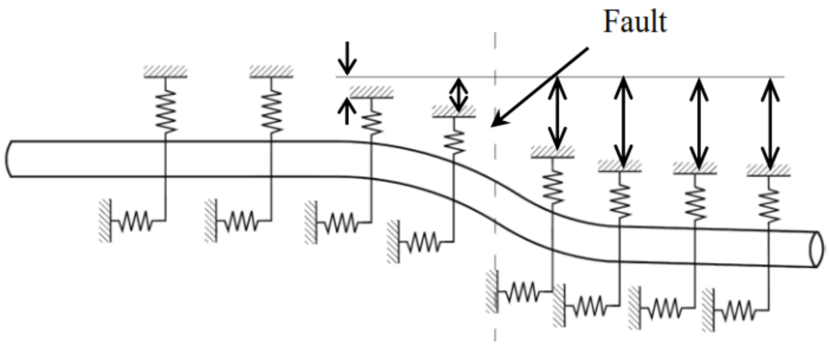
utilized (Figures 13.4 and 13.5). In Section 13.3.7, soil behavior modeling with equivalent springs is represented.

13.3.4 Analysis for Landslide

Landslide zones should be avoided through careful route selection. Figure 13.6 illustrates pipeline modeling for landslide. Loads caused by landslide can be assumed uniform at direction of the slide, as shown in Figure 13.6b, with a value calculated based on soil-pipe interaction behavior.
Where the slide direction is not perpendicular to the pipeline longitudinal axis, in addition to lateral component, axial seismic force component shall be considered. Pipeline anchored length at sides of sliding zone shall be determined from a try and error process. If at one side or both sides of the sliding zone, pipe is constrained by anchors and the distance between constraints to the sliding location is not more than the determined length from try and error process, effects of constraints shall be considered in analysis. Normally, unstiffened pipelines are subjected to nonlinear behavior at sliding. In this case, it is permitted to simulate the ductile pipeline with a beam with plastic hinges.
Permanent design displacement values for landslide can be calculated based on return period of the relevant Function & Risk Category (Section 13.2). In addition, these values are permitted to be calculated based on a 475-year return period and then multiplied by importance factor, I_L, from Table 13.1. Pipe strain due to landslide is determined according to Section 13.3.6.



a. Real Geometry



b. Idealized Modeling

Figure 13.4 Modeling of Pipeline for Fault Displacement

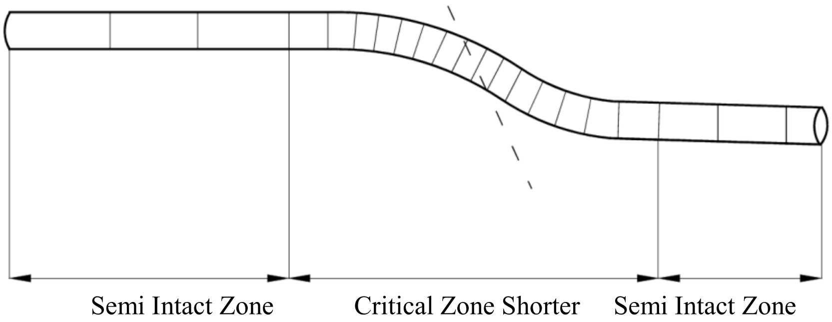


Figure 13.5 Modeling of Pipeline in FEM at Fault Crossing Location

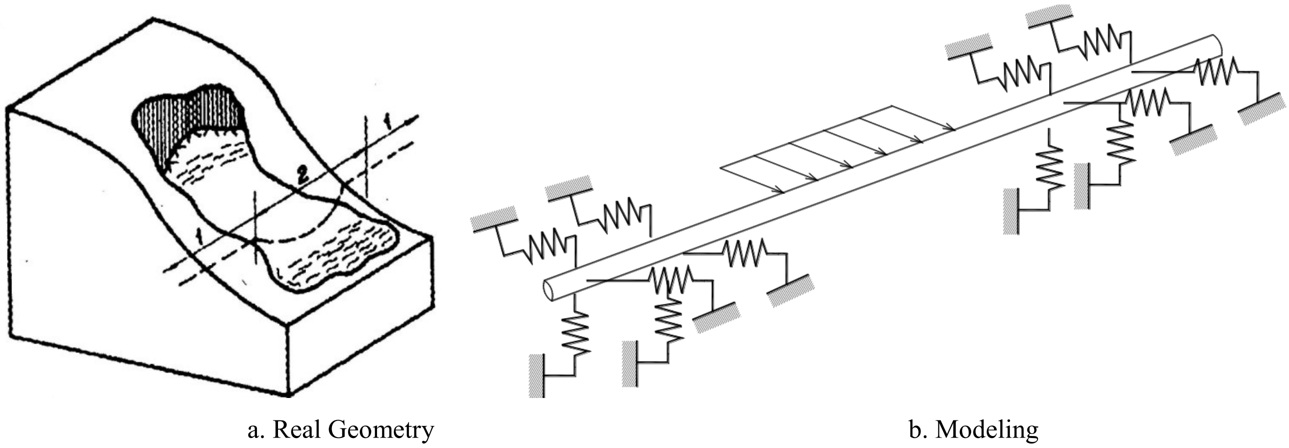


Figure 13.6 Modeling of Pipeline for Landslide

13.3.5 Analysis for Liquefaction

To determine pipeline route, it is suggested to avoid probable liquefaction zones. In liquefiable soil, analysis can be based on beam on elastic foundation assumption. In this method, pipeline foundation stiffness can be calculated from soil-pipe interaction formulas. Conservatively, soil stiffness for liquefiable soil can be taken as zero. Length of constrained parts at two sides of liquefiable zone shall be determined by try and error. As an initial value, a length equal to 25% of the length of the pipeline crossing the liquefiable zone is suggested. Permanent displacements caused by buoyancy due to liquefaction can be calculated based on return period corresponding to Function & Risk Category (Section 13.2) or ground permanent deformation factors from Table 13.1.

When liquefaction of soil occurs around the pipeline, buoyant forces are exerted on pipeline and must be resisted by suitable anchoring device. The net upward force per unit length of pipeline due to buoyancy may be calculated from Equation 13.22.

$$F_b = W_s - \left[W_p + W_c + (P_v - \gamma_w h_w)D\right] \qquad 13.22$$

where:

W_s = total weight of soil displaced by pipe per unit length

W_p = weight of pipe per unit length

W_c = weight of pipe content per unit length

P_v = vertical earth pressure from Equation 13.23

γ_w = unit weight of water

h_w = height of water above pipeline

Adherence of soil to pipe is neglected in the above equation for simplicity.

$$P_v = \gamma_w h_w + \gamma_d h_{sp} - 0.33\gamma_d h_w \qquad 13.23$$

where:
γ_d = unit weight of dry soil
h_{sp} = height of soil above pipeline

Bending stress induced for a relatively short section of continuous pipeline subjected to buoyancy, σ_{bf}, can be calculated from Equation 13.24. In this equation, F_b is in N/m.

$$\sigma_{bf} = \frac{F_b L_b^2}{10 Z_e} \qquad 13.24$$

where:
L_b= length of pipe in buoyancy zone (m)
Z_e= elastic section modulus of pipe cross section (m^3)
The maximum strain corresponding to above bending stress can be obtained by Equation 13.1. The maximum strain obtained can be considered as the design strain in pipe, $\varepsilon_{seismic}$, and should conform to the allowable strain as specified in Table 13.3, regarding the Equation 13.8.
For longer pipelines affected by buoyancy force, resistance against upward force can be approximated based on simultaneous cable and beam behavior of the pipe.
Pipeline investigation for permanent ground deformations due to lateral spreading caused by liquefaction is mentioned in Section 13.3.6.
The response of segmented pipeline subjected to liquefaction can be analyzed according to the location of the joint by using the equilibrium of forces and moment. In the analysis, the joint of the segmented pipe may be considered as a hinge joint and the extension and rotation of the joint is obtained. The extension of the joint can be considered as the design joint displacement of the pipeline and should conform to the allowable joint displacement.

13.3.6 Analysis for Permanent Ground Deformation Caused by Liquefaction and Landslide

In this section, the attention is restricted to the permanent ground deformation due to liquefaction-induced lateral spreading and landslide. From the geotechnical investigations, the spatial extent, i.e., length L_z, width W_z and maximum longitudinal ground displacement δ^l of permanent ground deformation zone, should be established. It is generally difficult to come out with a single value for the amount δ^l and spatial extent L_z and W_z. Hence, a range of above quantities are established, and the seismic check is carried out. The permanent design ground displacement in longitudinal direction may be taken as Equation 13.25:

$$\delta^l_{design} = \delta^l I_L \qquad 13.25$$

Generally, two types of models are used for buried pipelines subjected to longitudinal permanent ground deformation, with a uniform distribution assumption for the deformations, i.e. longitudinal ground deformation is uniform in the whole permanent deformation zone:

Case 1. The amount of ground movement, δ^l_{design}, is large and the pipe strain is controlled by length L_z of the permanent ground deformation zone. In this case, the maximum axial strain in pipe for both tension and compression is calculated from Equation 13.26.

$$\varepsilon_a = \frac{t_u L_z}{2\pi D t_p E_p}\left[1 + \frac{n}{1+r}\left(\frac{t_u L_z}{2\pi D t_p \sigma_y}\right)^r\right] \qquad 13.26$$

where:
n, r = parameters from Equation 13.1
t_u = peak frictional force per unit length at soil-pipe interface, from Equation 13.12

Case 2. Length of the permanent ground deformation zone, L_z, is large and the pipe strain is controlled by the amount of ground movement, δ^l_{design}. In this case, the maximum axial strain in pipe for both tension and compression is calculated from Equation 13.27.

$$\varepsilon_a = \frac{t_u L_e}{\pi D t_p E_p}\left[1 + \frac{n}{1+r}\left(\frac{t_u L_e}{\pi D t_p \sigma_y}\right)^r\right] \qquad 13.27$$

where:
L_e = effective length of pipeline over which friction force, t_u, acts and can be calculated from Equation 13.28.

$$\delta^l_{design} = \frac{t_u L_e^2}{\pi D t_p E_p}\left[1 + \left(\frac{2}{2+r}\right)\left(\frac{n}{1+r}\right)\left(\frac{t_u L_e}{\pi D t_p \sigma_y}\right)^r\right] \qquad 13.28$$

Pipe seismic strain, $\varepsilon_{seismic}$, for longitudinal permanent ground deformations, shall be taken as the lower of the strains obtained from Equations 13.26 and 13.27.
The expansion joints are provided to mitigate the effect of longitudinal permanent ground deformation in a continuous pipeline.
Like longitudinal permanent ground deformation, a range of values for δ^l and spatial extent (L_z and W_z) of transverse permanent ground deformation are quantified and the seismic check is carried out.
The design ground displacement in transverse direction, δ^t_{design}, can be calculated from Equation 13.29.

$$\delta^t_{design} = \delta^t I_L \qquad 13.29$$

where:
δ^t= maximum transverse ground displacement
The maximum bending strain in pipe, ε_b, may be conservatively calculated as the least of the values calculated by Equation 13.30.

$$\varepsilon_b = \pm\frac{\pi D \delta^t_{design}}{W_z^2} \qquad 13.30a$$

$$\varepsilon_b = \pm\frac{P_u W_z^2}{3\pi E_p t_p D^2} \qquad 13.30b$$

where:
P_u= maximum lateral resistance of soil per unit length of pipe, from Equation 13.31
The maximum strain obtained above shall be considered as pipe design strain, $\varepsilon_{seismic}$.

$$P_u = S_u N_{ch} D \qquad \text{for clay} \qquad 13.31a$$

$$P_u = \bar{\gamma} H_s N_{qh} D \qquad \text{for sand} \qquad 13.31b$$

where:
S_u= undrained shear strength of soil
N_{ch}= horizontal bearing capacity factor for clay according to Equation 13.32
N_{qh}= horizontal bearing capacity factor for sand according to Equation 13.33

$$N_{ch} = A_1 + A_2x + \frac{A_3}{(x+1)^2} + \frac{A_4}{(x+1)^3} \leq 9 \qquad 13.32$$

$$N_{qh} = A_1 + A_2x + A_3x^2 + A_4x^3 + A_5x^4 \qquad 13.33$$

where:
A_1 to A_5= the coefficients determined from Table 13.5
$x = \frac{H_s}{D}$

Simplified analytical expressions given above may be used for determining strain in the pipeline required for preliminary design. However, finite element analysis considering nonlinearity in pipe and soil is advised to be performed while designing important pipelines.

Table 13.5 Coefficients of Equations 13.32 and 13.33 with Respect to Soil Friction Angle, ϕ

Coefficient	ϕ (degrees)	A_1	A_2	A_3	A_4	A_5
N_{ch}	0	6.572	0.065	-11.063	7.119	-
N_{qh}	20	2.399	0.439	-0.030	1.059×10^{-3}	-1.754×10^{-5}
	25	3.332	0.839	-0.090	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}

For segmented pipelines, seismic deformation is considered as the maximum opening at the joint of the pipe, $\Delta_{seismic}$, due to permanent ground deformation as determined from Equation 13.34:

$$\Delta_{seismic} = \delta^l_{design} \qquad 13.34$$

where:
δ^l_{design}= design ground displacement in longitudinal direction
Joint design deformation shall be less than allowable deformation represented by supplier.
Number of joints in segmented pipelines depends on permanent ground displacement (PGD). In this type of pipelines, one joint may be provided at the head and one at the toe of the PGD zone. For small values of ground displacement, push-on type joints (joints without mechanical stops) may be used.
In the areas of large ground displacement, a chained joint can be designed to accommodate the deformations. Chained joints should be provided at both head and toe of the PGD zone, and at least three joints are to be installed outside the PGD zone boundary. The design joint displacement of each pipe segment may be calculated as:

$$\Delta_{seismic} = \left[\frac{\delta^l_{design}}{L/2}\right] L_0 \qquad 13.35$$

where:
L= length of permanent ground deformation zone.

The mechanical stops, which are used in chained joints, must be designed to accommodate maximum friction force, F_{stop} given by Equation 13.36:

$$F_{stop} = 2\left[\frac{n_c + 1}{2}\right] L_0 t_u \tag{13.36}$$

where:
n_c = number of chained joints at head or toe of the moving soil mass, that will expand to absorb total amount of PGD.
In any case, F_{stop} need not be greater than the yield strength of pipe.
The design joint displacement of a segmented pipeline for transverse PGD can be calculated as sum of axial extension and extension due to rotational effect. Thus, the resulting joint displacement can be written as:

$$\Delta_{seismic} = \frac{\pi^2 L_0 {\delta^t_{design}}^2}{W^2}\left[\frac{2D}{\delta^t_{design}}\right] \qquad 0.268 \le D/\delta^t_{design} \le 3.73 \tag{13.37a}$$

$$\Delta_{seismic} = \frac{\pi^2 L_0 \left(\delta^t_{design}\right)^2}{W^2}\left[1 + \left(\frac{D}{\delta^t_{design}}\right)^2\right] \qquad For\ other\ values\ of\ \ D/\delta^t_{design} \tag{13.37b}$$

The design joint displacement calculated above should conform to the allowable joint displacement criteria as given by supplier.

13.3.7 Modeling of Buried Pipeline with Equivalent Springs

For modeling soil behavior and soil pipe interaction, numerical methods may be used. Alternatively, pipe embedded in a semi-infinite soil medium may be modeled. Another procedure is the theory of Beam on Nonlinear Winkler Foundation (BNWF) model; where the soil is represented by independent springs (longitudinal, transverse and vertical components) lumped at discrete locations of the pipe. In such a procedure, the model is able to simulate stiffness of the soil surrounding the pipe. For a soil with given geotechnical parameters, the specifications of the springs can be calculated. Nonlinear behavior of these springs is modeled according to Figure 13.8.

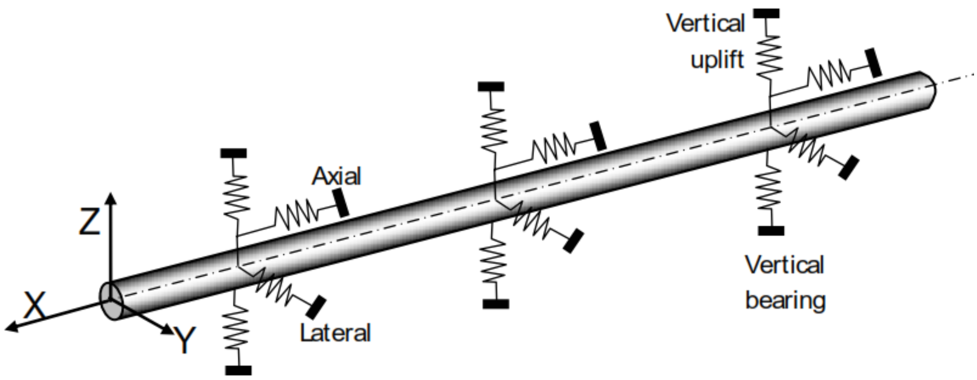


Figure 13.7 Modeling Soil-Pipe Interaction with Inelastic Springs

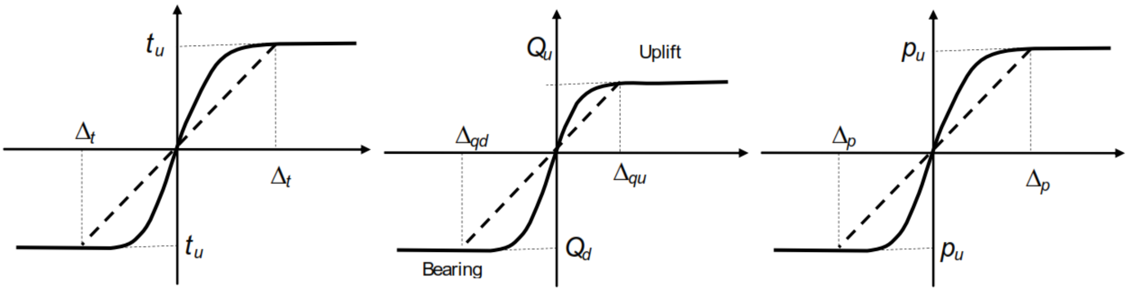


Figure 13.8 Soil Springs in a. vertical, b. axial and c. lateral directions

While analyzing the pipeline for permanent ground deformation (PGD), Nonlinear Static Procedure (NSP) may be used. In this procedure, considered deformation is applied to the fixed ends of soil springs. While analyzing the pipeline for seismic wave propagation effect, a valid Seismic Response History Procedure with application of considered time history to the fixed ends of soil springs and performing dynamic analysis may be used.
The values shown in Figure 13.8 with respect to soil and pipe specifications are described as follows. The spring's specifications, with consideration of construction method and corresponding to surrounding backfill soil or native soil are determined. The represented equations are specially for buried pipeline systems with a depth of 0.5 to 2.0 meters. For deeper pipelines, model shall consider surrounding soil specifications.

13.3.7.1 Axial Soil Spring

The properties of axial soil springs are estimated considering the soil properties of the backfill material used in the pipeline trench. The maximum axial soil strength per unit length of pipe, t_u, is calculated from Equation 13.12. The maximum mobilizing displacement of soil in axial direction of pipe, Δ_t, depends on soil type and can be derived from Table 13.6.

Table 13.6 Δ_t Values Corresponding to Soil Types

Soil Type	Δ_t (mm)
Dense Sand	3
Loose Sand	5
Stiff Clay	8
Soft Clay	10

13.3.7.2 Lateral Soil Spring

The properties of lateral soil spring are estimated considering the native soil at the site. The maximum lateral resistance of soil per unit length of pipe, P_u , can be calculated as:

$$P_u = N_{ch} cD + N_{qh} \bar{\gamma} H_s D \qquad 13.38$$

where N_{ch} and N_{qh} are determined from Equations 13.32 & 13.33.
The maximum lateral displacement, Δ_p, is calculated from Equation 13.39:

$$\Delta_p = 0.04(H_s + D/2) \leq (0.1 \sim 0.15)D \qquad 13.39$$

13.3.7.3 Vertical Soil Spring

The soil spring properties are different for uplift and bearing cases. For bearing soil spring, the properties of native soil at the site may be used. However, for uplift soil spring, the properties of backfill soil are to be considered. Soil behavior specifications in uplift case are determined from Equations 13.40 to 13.43:

$$Q_u = N_{cv}cD + N_{qv}\bar{\gamma}H_sD \qquad 13.40$$

where:
N_{cv}= Vertical uplift factor for soil cohesion according to Equation 13.41
N_{qv}= Vertical uplift factor for soil internal friction according to Equation 13.42

$$N_{cv} = 2\frac{H_s}{D} \leq 10 \qquad \textit{For values of } \frac{H_s}{D} \leq 10 \qquad 13.41$$

$$N_{qv} = \frac{\phi H_s}{44D} \leq N_q \qquad 13.42$$

N_q= bearing capacity factor, determined from Equation 13.43 or Figure 13.9:

$$N_q = exp(\pi tan\phi) \times tan^2(45 + \phi/2) \qquad 13.43$$

The mobilizing soil displacement at Q_u, Δ_{Qu}, is calculated from Equation 13.44:

$$\Delta_{Qu} = (0.01\sim 0.02)H_s \leq 0.1D \qquad \textit{For granular soils} \qquad 13.44a$$

$$\Delta_{Qu} = (0.1\sim 0.2)H_s \leq 0.2D \qquad \textit{For cohesive soils} \qquad 13.44b$$

The maximum soil resistance per unit length of pipeline in vertical bearing can be calculated with Equations 13.45 to 13.47:

$$Q_d = N_c cD + N_q\bar{\gamma}H_sD + N_\gamma\gamma\frac{D^2}{2} \qquad 13.45$$

where:
N_c and N_q are bearing capacity factors determined from Equations 13.46 & 13.47 or Figure 13.9:

$$N_c = [cot(\phi + 0.001)]\left\{exp[\pi tan(\phi + 0.001)]tan^2\left(45 + \frac{\phi + 0.001}{2}\right) - 1\right\} \qquad 13.46$$

$$N_\gamma = exp(0.18\phi - 2.5) \qquad 13.47$$

In cases where internal friction angle of soil, ϕ, is equal to zero, minimum value of the above equations is used. Corresponding displacement of maximum compression force, Δ_{Qd}, for granular soils is $0.1D$ and for cohesive soils is $0.2D$.

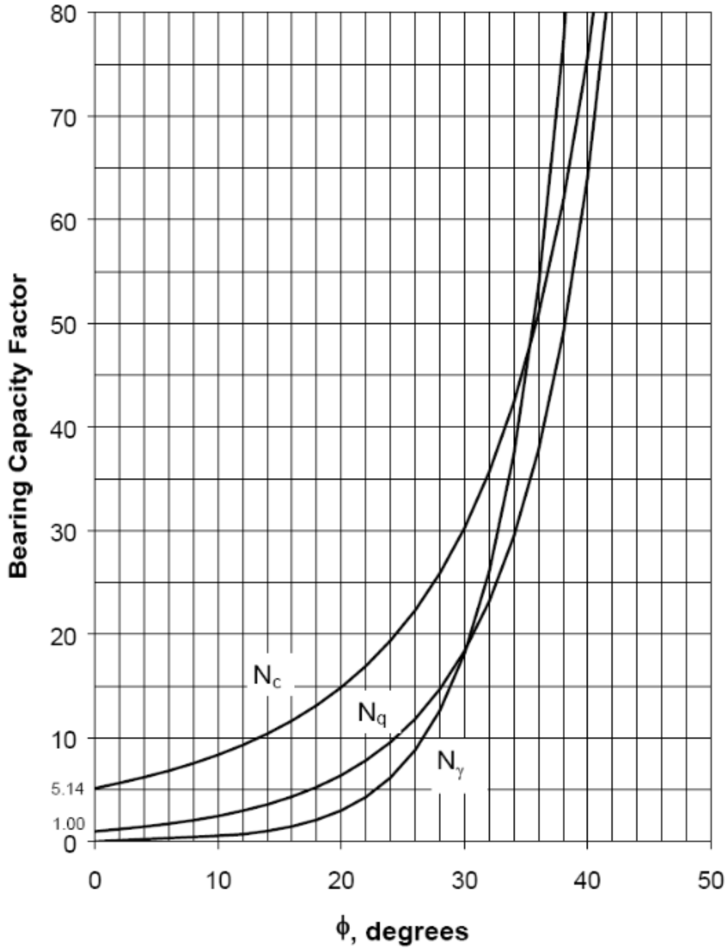


Figure 13.9 Bearing Capacity Factors for Different Soil Friction Values [13]

13.4 Above Ground Pipeline

Generally, above ground pipeline is supported by numerous sliding restraints in addition to anchor points. For seismic analysis of pipeline, friction between pipe and support shall be precisely modeled, as well as other effective factors. To resist against deformations due to temperature variations, pipeline includes expansion joints or special bends, based on diameter, which complicate structural geometry of pipeline. So, for seismic design of above ground pipeline, it is recommended to consider sliding supports with a negligible friction. These supports shall have adequate width or appropriate transverse constraint perpendicular to pipeline axis to prevent falling of pipeline due to lateral seismic forces. Allowable response values are presented in Section 13.4.6.

13.4.1 Analysis for Wave Propagation by Equivalent Lateral Procedure

To analyze above ground pipeline with Equivalent Lateral Load Procedure, the maximum relative displacement between two adjacent anchor points can be approximated with Site-Specific Hazard Study, and the pipeline can be analyzed as a multi-span beam for the maximum relative displacement. If direction of seismic waves is known or predictable, longitudinal and transversal movements of anchor points can be considered for analyses. Otherwise, the most unfavorable combination of support movements, exerting maximum values of stress in pipe, which can be calculated in an iterative method for different directions, shall be considered. In this method, pipeline analysis at bends and tees shall be done precisely.

13.4.2 Analysis for Wave Propagation in Dynamic Procedure

Analysis of above ground pipeline for wave propagation in dynamic procedure is same as buried pipeline, except that supports or supporting structures can be modeled as springs or structural elements. Seismic design can be based on one of the two following seismic component combinations:

1. The resultant (square root of the sum of the squares) of the larger horizontal component and the vertical component.
2. The resultant (square root sum of the squares) of both horizontal and vertical components.

Other provisions are the same as Section 13.3.2.

13.4.3 Analysis for Fault Crossing

To analyze an above ground pipeline for fault crossing, pipeline can be modeled as a multi-span beam with nonlinear spring supports along all three perpendicular directions at anchor points.
Number of anchor points in this procedure can be estimated by iteration. To do so, usually spans inside fault zone (at least one span) and two spans of pipeline at fault zone boundaries are initially considered. If end support forces due to fault displacements do not cross the boundary of linear behavior, analysis is acceptable, with a reasonable preciseness. Otherwise, one span shall be added to each side and calculations shall be repeated.
Section 13.3.3 can be considered to estimate fault displacements.

13.4.4 Analysis for Landslide

To analyze an above ground pipeline for landslide, procedure of Section 13.3.4 can be used, except that in this case, pipeline is modeled as a multi-span beam with nonlinear spring supports along all three perpendicular directions at anchor points.

13.4.5 Analysis for Liquefaction

To analyze an above ground pipeline for liquefaction, procedure of Section 13.3.5 can be used. To determine number of spans in modeling, Section 13.4.3 can be considered.

13.4.6 Acceptance Criteria of Above Ground Pipelines

Seismic design procedure and its stages are mentioned in Table 13.7. Procedure depends on pipeline classification to critical and non-critical pipes, magnitude of input seismic event, and pipe diameter. In Table 13.5, D_n is the nominal diameter of the pipe. In all cases, it is permitted to design the pipeline regarding analysis procedures mentioned in Section 13.4.6.2 and 13.4.6.3.

13.4.6.1 Prescribed Design Method

When design by rule is permitted in Table 13.7, the seismic qualification of piping systems may be established by providing lateral and vertical seismic restraints at a maximum spacing, L_{max}, from Equation 13.48 [14]:

$$L_{max} = min\begin{cases} 1.94L_T/S_{DS}^{0.25} \\ 0.211L_T\left(\sigma_{yo}/S_{DS}\right)^{0.5} \end{cases} \qquad 13.48$$

where:
L_{max} = maximum permitted pipe span between lateral and vertical seismic restraints
L_T = recommended span between weight supports (Table 13.8)
S_{DS} = Spectral acceleration parameter (g) for Seismic Hazard Level II at short period (0.2 sec)
σ_{yo} = material yielding stress at normal operating temperature (MPa)
In addition, straight pipes running longer than three times the span of Table 13.8, should be restrained longitudinally. The distance between lateral and vertical restraints should be reduced for pipe spans that contain heavy in-line components (with a total component weight in excess of 10% of the weight of the tabulated pipe span). Equation 13.48 is based on limiting deflection at the pipe mid-span to 50 mm and the maximum stress to $0.5\sigma_{yo}$.

Table 13.7 Seismic Design Requirements for Above Ground Pipelines, Applicable Sections (D_n in mm)

S_{DS} (g)	Non-Critical Pipelines (Function & Risk Category) II)			Critical Pipelines (Function & Risk Category III & IV)	
	$D_n \leq 50$	$50 < D_n < 150$	$D_n \geq 150$	$D_n \leq 50$	$D_n \geq 150$
≤ 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

Table 13.8 Suggested Pipe Support Spacing, 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)	Liquids Content	200	300	350	430	520	580	700	820	900	980
	Steam, Gas or Air Content	270	400	460	520	640	730	900	1070	1200	1280

13.4.6.2 Analytical Design Method

In analytical design of pipeline, the elastically calculated longitudinal stresses due to the design earthquake (calculated by static or dynamic analysis) shall comply with the Equation 13.49.

$$\frac{i \times \sqrt{M_i^2 + M_a^2}}{Z_e} < S_s \qquad 13.49$$

where:
i = stress intensification factor (from the applicable code, such as Reference [15])
M_i = resultant moment amplitude due to inertia
M_a = resultant moment amplitude due to relative anchor motion
S_s = allowable seismic stress for temperature spanning -30° C to +40° C, equal to 110 MPa for mild and low-alloy steel and 130 MPa for stainless steel
The resultant moment at a point may be the square root of the sum of the squares of the three moment components (the in plane, out-of-plane and torsional moments) at that point.

13.4.6.3 Alternative Design Methods

Where equation 13.49 can not be met, the piping system may be qualified by more detailed analysis techniques, including fatigue, inelastic or extreme limit state analysis.

13.4.6.4 Mechanical Joints

For critical pipelines (Function & Risk Categories III & IV), rotations, displacements and internal actions at mechanical joints must remain within the limits specified by the joint manufacturer.

13.4.6.5 Seismic Restraints

Seismic restraints are provided to prevent falling of pipeline from its supports. The seismic forces on seismic restraints and their attachment to building structures and anchorage to foundation shall be calculated by static or dynamic analysis. A total gap equal to the pipe radius for 50 mm (2 inch) nominal pipe size and smaller pipes, and 50 mm (2 inch) for larger pipes is permitted in the restrained direction, provided the seismic force, calculated on the basis of a zero gap, is multiplied by an impact factor of 2.

13.4.6.6 Components of Pipeline

The seismic and concurrent loads applied by pipe at component nozzles shall be determined as part of the seismic design of piping system.

13.4.6.7 Interactions

Piping systems shall be evaluated for seismic interactions. Credible and significant interactions shall be identified and resolved by analysis, testing or hardware modification.

13.5 Pipeline Supported by Structural Frame (Pipe Rack)

Piperacks shall be designed for requirements of Chapter 7. Design of other parts of pipeline is similar to Section 13.4 requirements.

Chapter 14

Offshore Structure

14. Offshore Structure

14.1 Scope

Analysis and design of fixed offshore platforms are included in this chapter. Seismic design of submersible, semi-submersible or jack-up platforms is not discussed in this code. Other load cases such as wind, wave, buoyancy, blasting, ship impact etc. are not considered. For these load cases refer to Reference [16].

14.2 General

For seismic design of jacket-type platforms in offshore areas, providing strength in service earthquake and providing ductility in rare earthquake are considered. Strength requirements (Section 14.4) are intended to provide a platform and it's primary members with adequate size to show elastic response such that major structural damage does not occur during service level earthquake (Section 14.3). The ductility requirements (Section 14.5) are intended to ensure that the platform has sufficient reserved capacity to prevent its collapse during rare earthquake motions, although structural damage may occur. Only vibratory ground motion is addressed in this code and other major concerns such as large soil deformations or instability because of liquefaction or landslide should be resolved by special studies.

14.3 Ground Motion

There are two hazard levels for seismic design of offshore structure that shall be based on special site investigation:
Service Ground Motion: The 1st level of earthquake introduced in Section 3.4.1 (Seismic Hazard Level I)
Rare Ground Motion: The 3rd level of earthquake represented by Section 3.4.3 (Seismic Hazard Level III), unless by means of economical and technical investigations, another return period is recommended by the consultant and approved by the client.

14.4 Strength Requirements

14.4.1 Design Basis

The platform shall be analyzed and designed for the service earthquake determined in accordance with Section 14.3 using elastic dynamic analysis procedures such as Modal Response Spectrum Analysis or Seismic Response History Procedure. For performing Modal Response Spectrum Analysis, the two perpendicular horizontal components of motion and vertical component obtained from special hazard analysis with a 5% damping ratio are used separately and the modal responses are combined with the methods that introduced in Section 14.4.3 and Chapter 4 provisions.
If there are different geotechnical specifications along the pile length, for evaluating the seismic horizontal spectrum, the soil specification near the pile cap should be considered. For determination of the vertical component of earthquake, the soil condition near the tip of pile should be considered.
If Seismic Response History Procedure is used, time histories shall be selected and scaled by provisions of Section 4.10.

14.4.2 Structural Modeling

The mass used in the dynamic analysis should consist of the mass of the platform including permanent mass of the jacket (structural framing, components and appurtenances), live service mass associated with permanent fixed components, 75% of the maximum supply and storage loads, 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 structural stiffness, mass and asymmetry. In computing the dynamic characteristics of braced, pile supported steel structures, a uniform damping ratio of 5% shall be used. Where substantiating data exist, other damping ratios may

be used. Damping ratio of 5% is specified for use in all modes unless another damping ratio, η (percent), is justified, where the following factor, D_s, is multiplied to the response:

$$D_s = \frac{-Ln\left(\frac{\eta}{100}\right)}{Ln(20)} \tag{14.1}$$

The factor, D_s, is appropriate for values of damping ratios between 2 and 10 percent.

14.4.3 Response Combination

For Modal Response Spectrum Analysis, two horizontal and one vertical spectrum obtained from site hazard analysis are used. If there exists only one horizontal spectrum, a same spectrum will apply for two horizontal directions and half of which will apply for vertical direction. 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 directional responses. For the Modal Response Spectrum Procedure, for an adequate representation of the response, as many modes should be considered as required so that in both main direction of the structure, cumulative modal effective mass ratio would not be less than 90% in direction under consideration.

Where the Seismic Response History Procedure is used, at least three time histories shall be used. For scaling the time history components individually (two horizontal and one vertical components), the method in Section 4.10.2 may be applied.

Earthquake loading shall be combined with other simultaneous loadings such as gravity, buoyancy, and hydrostatic pressure according to Section 2.2

14.4.4 Response Assessment

For assessment of the strength requirements, valid codes may be used. The basic allowable values may be increased by 30%[1] when ASD method is used.

Pile-soil performance and pile design requirements should be determined based on special studies. These studies should consider the design loadings of Section 14.4.3, as well as installation procedures, earthquake effects on soil properties and characteristics of the soils as appropriate to the axial or lateral capacity procedure being used.

14.5 Ductility Requirements

14.5.1 Objective

The intent of these requirements is to ensure that platforms in seismically active areas have adequate reserve capacity to prevent collapse under rare earthquake. Ductility control shall be performed analytically in accordance with provisions of Section 14.5.3. Where requirements of Section 14.5.2 for providing ductility are achieved, these controls are not mandatory.

14.5.2 Providing Ductility

Respect to these conditions, there is no need for an explicit analytical demonstration of adequate ductility (Section 14.5.3).

- If the structure is to be located in an area where the intensity ratio of rare earthquake response spectrum to service level earthquake response spectrum at the fundamental period is two or less.
- The piles are to be founded in soils that are stable in ground motions imposed by the rare earthquake.
- 8 or more legs is used for the structure
- Jacket legs, including any enclosed piles are designed for twice service seismic loads.
- Diagonal bracing in the vertical frames are configured such that shear forces between horizontal frames or in vertical runs between legs are distributed approximately equally between both tension and compression diagonal braces.
- "K" bracing is not used.

[1] In previous versions of codes for design the offshore structures, this multiplier was 70% for ASD method.

- Chevron bracing is not used unless the ability of a panel to transmit shear if the compression brace buckles is approved.
- Where these conditions are not met or where bracing is not used, including areas such as a portal frame existing between the jacket and the deck, the structural components should be designed for twice the service seismic loads.
- Horizontal members are provided between all adjacent legs at horizontal framing levels in vertical frames and these members have sufficient compression capacity to support the redistribution of loads resulting from the buckling of adjacent diagonal braces.
- The slenderness ratio (Kl/r) of primary diagonal bracings in vertical frames is limited to 80 and their ratio of diameter to thickness is limited to $0.07E/F_y$.
- All non-tubular members at connections in vertical frames are designed as compact sections in accordance with the Reference [17] or designed for twice the service seismic loads.

14.5.3 Ductility Control

Structure-foundation system shall be analyzed to demonstrate its ability to withstand the rare earthquake without collapse. 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 geometrical nonlinearity (P-delta) effect of axial loads acting through elastic and inelastic deflections of the structure and foundation shall be considered.

14.6 Additional Guidelines

14.6.1 Tubular Joints

Joints for primary structural members shall 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 according to valid references.
Connection capacity can be calculated by punch shear or nominal forces of bracings. The allowable punch shear and allowable connection capacities can be increased by 30% if ASD method is used[1].

For determining allowable punching shear capacity, nominal stresses in members can be calculated as the minimum of nominal capacity of members or combination including twice service level earthquake, gravity loads, hydrostatic pressure and buoyancy.

14.6.2 Deck Appurtenances and Equipment

Equipment, piping, and other deck appurtenances shall be connected to main structure 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 shall 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 shall be given to the design of restraints for critical piping and equipment whose failure would result in injury to personnel, hazardous material spillage, pollution, or hindrance to emergency response. Design acceleration value should include the effects of global structural dynamic response; and, if appropriate, local dynamic response of the deck and appurtenance itself (Refer to Chapter 8), but in lieu of Seismic Hazard Level II, service earthquake (Seismic Hazard Level I) shall be used and R_p shall be taken as one.
If ASD method is used, increasing allowable stresses for designing the equipment to deck connections is not valid.

[1] In previous versions of codes for designing the offshore structures, this multiplier was 70% for ASD method.

Appendix 1

Seismic Base Considerations

Appendix 1. Seismic Base Considerations

A1.1 General

Seismic base is the level where transmission of horizontal ground motion to structure occurs. In this appendix, factors affecting determination of seismic base are presented.

A1.2 Factors Affecting the Location of Seismic Base

Many factors affect the location of seismic base. Some of the factors are:

a. Location of the grade relative to floor levels
b. Soil conditions adjacent to the building
c. Stiffness and openings in the basement walls
d. Location and stiffness of vertical elements of the seismic force-resisting system
e. Location and extent of seismic separation joints
f. Depth of basement
g. Manner in which basement walls are supported
h. Proximity to adjacent buildings
i. Slope of grade

In the following sections, effects of above articles in determination of seismic base are discussed.

A1.3 Seismic Base Regarding Adjacent Soil

For a building without a basement, the base is generally established at foundation level. Where the vertical elements of the seismic force-resisting system are supported at different elevations of footings, pile caps or perimeter foundation walls, seismic base is considered at the lowest elevation of the tops of elements supporting the vertical elements of the seismic force-resisting system.

For a building with a basement located on a level site which includes retaining walls integral with structural system, seismic base is considered at the first non-soft diaphragm below the ground (See Figure A1.1) provided that, competent soil exists over the depth of retaining wall during lifetime of the building. Competent soil is defined as stiff soil, which has the following conditions:

- is not categorized into Site Class IV in Reference [2]
- N_{SPT} for the layers is 20 as a minimum
- should not be liquefiable in the Seismic Hazard Level III
- does not include quick and highly sensitive clays
- has adequate cohesion

In some cases, the base may be at the non-soft diaphragm level adjacent to but above grade. In order 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 base and the Section 4.4.2 conditions (Two-Stage Analysis) should be met. In addition, for a floor level above grade to be considered as the base, it should not generally be located above grade more than one-half of the basement story height (Figure A1.2).

A1.4 Seismic Base Regarding the Stiffness of Basement Walls and Vertical Elements of Seismic-Force Resisting System

If the base is located at the level closest to grade, the lateral stiffness of the basement walls should be substantially more than the stiffness of the vertical elements of the seismic force-resisting system.

A condition where the basement walls that extend above grade on a level site may not provide adequate stiffness is when they have many openings (Figure A1.3). Where the basement wall stiffness is inadequate, the base should be taken as the level of non-soft diaphragm close to but below grade. If all of the vertical elements of the seismic force-resisting system are located on top of basement walls and there are many openings in the basement walls, it may be appropriate to establish the base at the bottom of the openings. Another condition where the basement walls may not be stiff enough is when the vertical elements of the seismic force-resisting system are long concrete shear walls extending over the full height and length of the building (Figure A1.4). For this case, the appropriate location for the base is the foundation level of the basement walls.

Where the structure and retaining wall systems are independent and without any connection, seismic level is considered at the foundation level.

A1.5 Seismic Base Regarding the Location and Distribution of Seismic Separation

For a building with seismic separations extending through the height of the building including levels close to and below grade, the separate structures will not be supported by the soil against a basement wall on all sides in all directions. If there is only one joint through the building, assigning the base to the level close to grade may still be appropriate if the soil over the depth of the basement walls is stiff and the diaphragm is rigid. Stiff soils are required so that the seismic forces can be transferred between the soil and basement walls through both bearing and side friction. If the soil is not stiff, adequate side friction may not develop for movement in the direction perpendicular to the joint.

For large footprint buildings, seismic separation joints may extend through the building in two directions and there may be multiple parallel joints in a given direction. For individual structures within these buildings, substantial differences in the location of the center of rigidity for the levels below grade relative to levels above grade can lead to torsional response. For such buildings, the base should usually be at the foundation elements below the basement or the highest basement slab level where the separations are no longer provided.

A1.6 Seismic Base Regarding the Adjacent Building

If other buildings with basements are located adjacent to one or more sides of a building, it may be appropriate to locate the base at the bottom of the basement of adjacent building.

A1.7 Seismic Base Regarding the Sloping Grade

For sites with sloping grade, many of the same considerations for a level site are applicable. For example, on steeply sloped sites, the earth may be supported by individual retaining wall. In such cases, the base shall be located at top of foundation level (Figure A1.5).

In building where the retaining wall is a part of seismic force-resisting system (Figures A1.6 & A1.7), it is recommended that the base to be located at a level close to the elevation of grade on the side of the building where it is lowest.

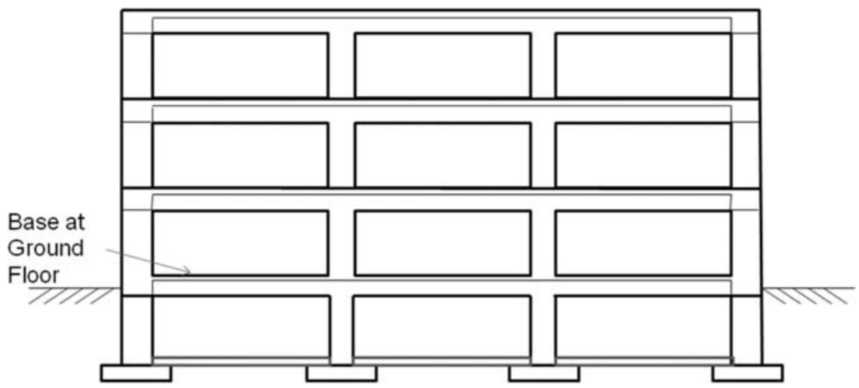


Figure A1.1 Building with Basement

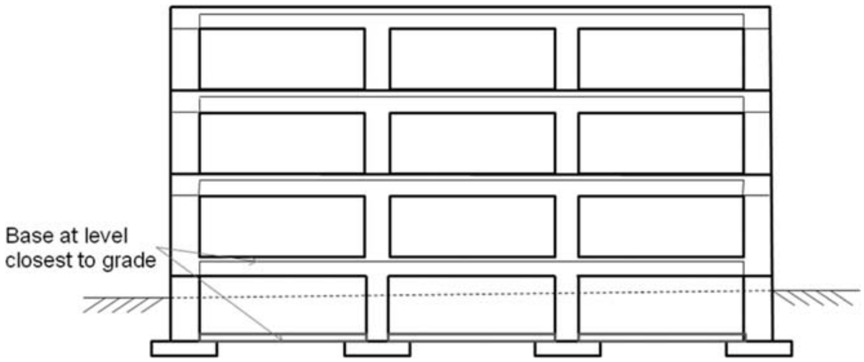


Figure A1.2 Building with Basement where the Base is at the Level Closest to Grade Elevation

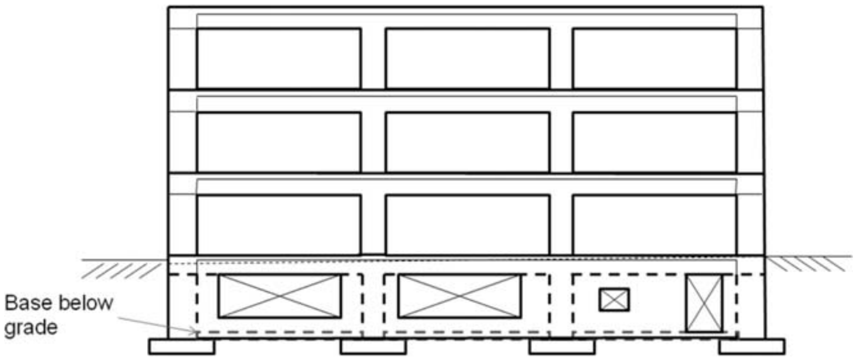


Figure A1.3 Building with the Basement Walls with Substantial Openings

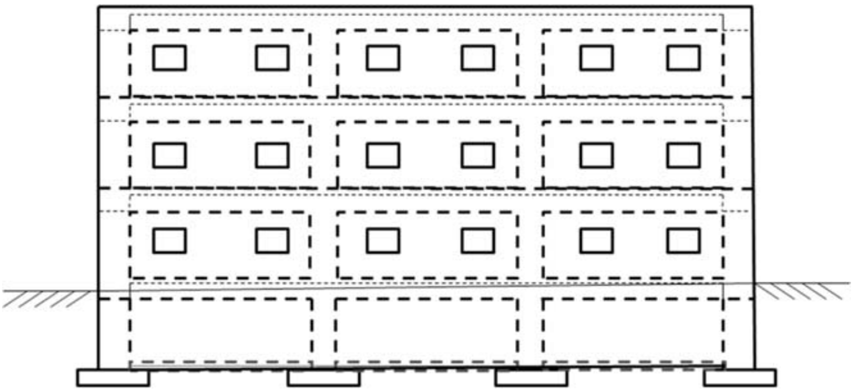


Figure A1.4 Building with Full Length Exterior Retaining Wall Disconnecting to the Structure

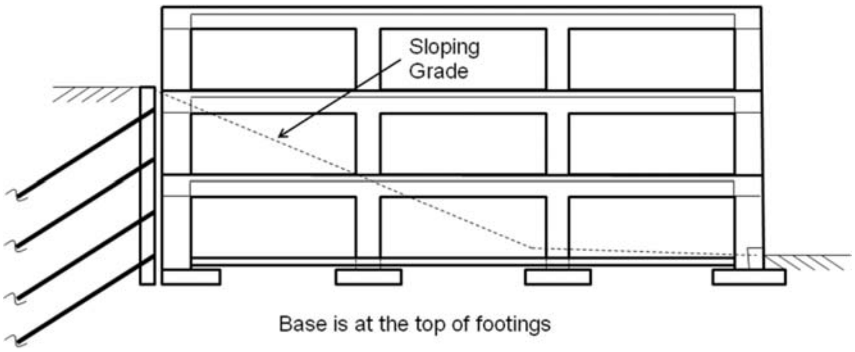


Figure A1.5 Building with Cantilever or Tied Retaining Wall, Disconnected to the Structural System

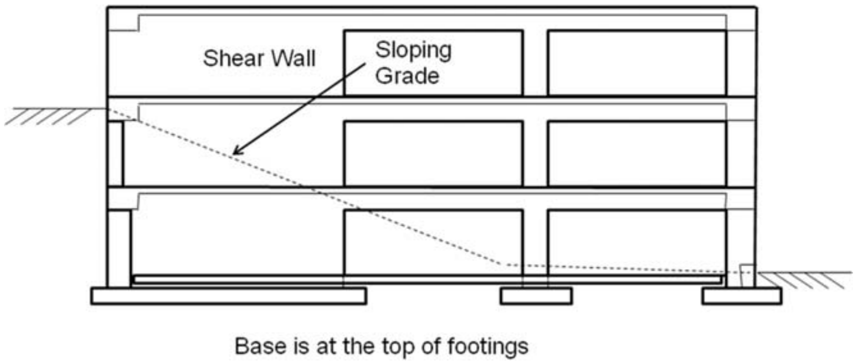


Figure A1.6 Building with Shear Walls which also Resis Soil Lateral Pressure

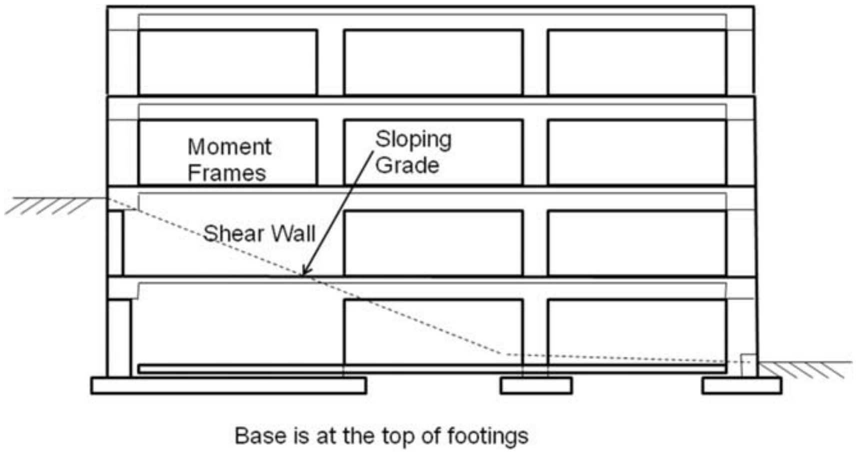


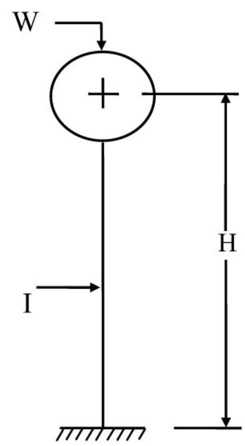
Figure A1.7 Building with Retaining Wall Connected to the Structural System

Appendix 2
Fundamental Period of Non-Building Structures

Appendix 2. Fundamental Period of Non-Building Structures

A2.1 Fundamental Period of Cantilever Bending System with Lumped Mass

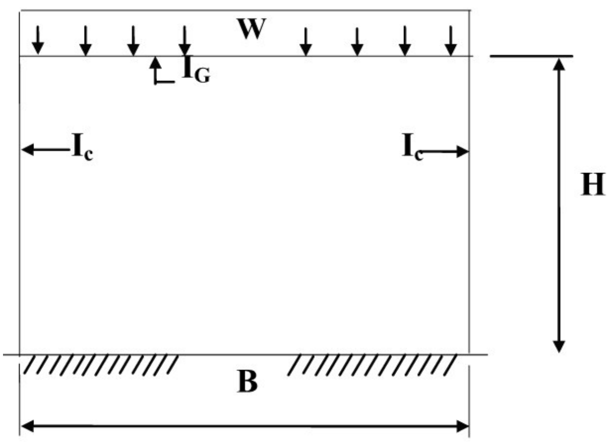
$$T = 2\pi\sqrt{\frac{WH^3}{3EIg}}$$



T= fundamental period (sec)
W= lumped weight (N)
H= catilever height (m)
E= modulus of elasticity (N/m^2)
I= moment of inertia (m^4)
g= acceleration of gravity (m/sec^2)

A2.2 Fundamental Period of Sway Single Degree of Freedom Moment Frame

$$T = 1.814\sqrt{\frac{\alpha WH^3}{EI_c g}}$$



W= total appied load (N)

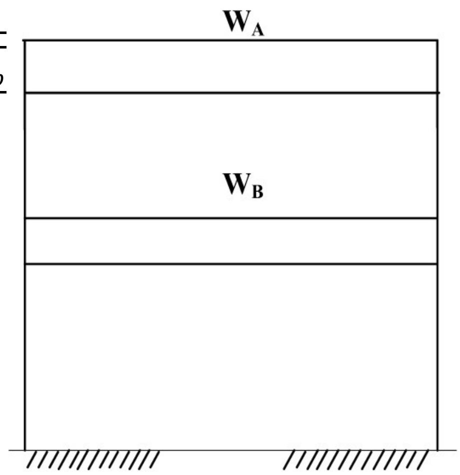
$$\alpha = \frac{2K+1}{K} \quad \text{for Pinned Base Columns}$$
$$\alpha = \frac{3K+2}{6K+1} \quad \text{for Fixed Base Columns}$$

$$K = \left(\frac{I_G}{I_c}\right)\left(\frac{H}{B}\right)$$

I_c & I_G are beam and column sectional moment of inertia, respectively. Other parameter units are the same as Section A2.1.

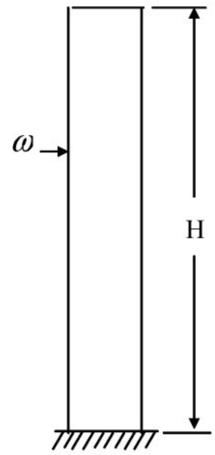
A2.3 Fundamental Period of Sway Two-Degree of Freedom Structure

$$T = 2\pi\sqrt{\frac{W_A C_{aa} + W_B C_{bb} + \sqrt{(W_A C_{aa} - W_B C_{bb})^2 + 4W_A W_B C_{ab}^2}}{2g}}$$



C_{aa} = displacemnet at level A, caused by unit load At level A
C_{bb} = displacemnet at level B, caused by unit load At level B
C_{ab} = displacemnet at level B, caused by unit load At level A
W_A and W_B = total vertical load on A or B level
g = acceleration of gravity
All units are in m and N.

A2.4 Fundamental Period of Cantilever Uniform Column

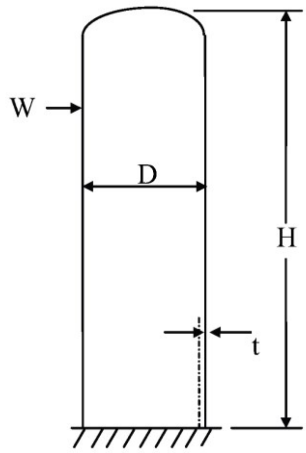


$$T = 1.79\sqrt{\frac{\omega H^4}{EIg}}$$

ω = weight per length (N/m)

Units of other parameters are the same as Section A2.1

A2.5 Fundamental Period of Cantilever Cylindrical Vessel



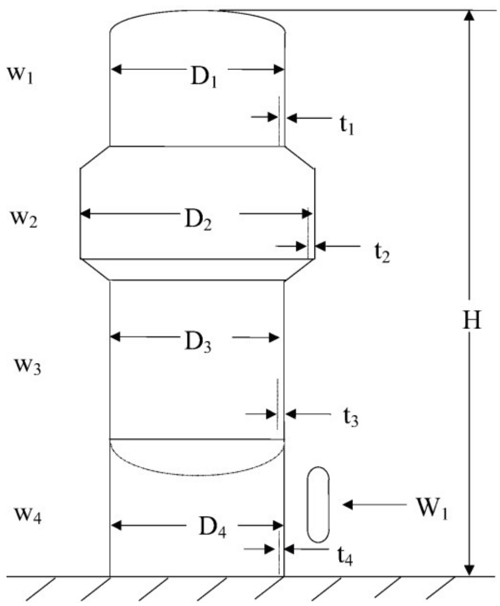
$$T = 0.0091\left(\frac{H}{D}\right)^2\sqrt{\frac{\omega D}{tE}}$$

ω = weight per length (N/m)
t = shell thickness (cm)
E = modulus of elasticity (N/m²)

Units of other parameters are the same as Section A2.1

A2.6 Fundamental Period Of Non-Uniform Cylindrical Vessel

$$T = \left(\frac{H}{100}\right)^2\sqrt{0.4\frac{\sum \omega\Delta\alpha + \frac{1}{H}\sum W\Delta\beta}{\sum ED^3t\Delta\gamma}}$$



H = total height (m)
ω = weight per length for each section (N/m)
W = weight of each lumped mass (N)
D = diameter (m)
t = shell thickness for each section (m)
E = modulus of elasticity (N/m²)
α, β & γ = coefficients dependent to the h_x/H, where h_x is the section height. $\Delta\alpha$ and $\Delta\gamma$ are the differences of these values from top to bottom of each section with a uniform weight distribution and constant diameter and thickness. β shall be calculated for each concentrated mass. Values are mentioned in the following table.

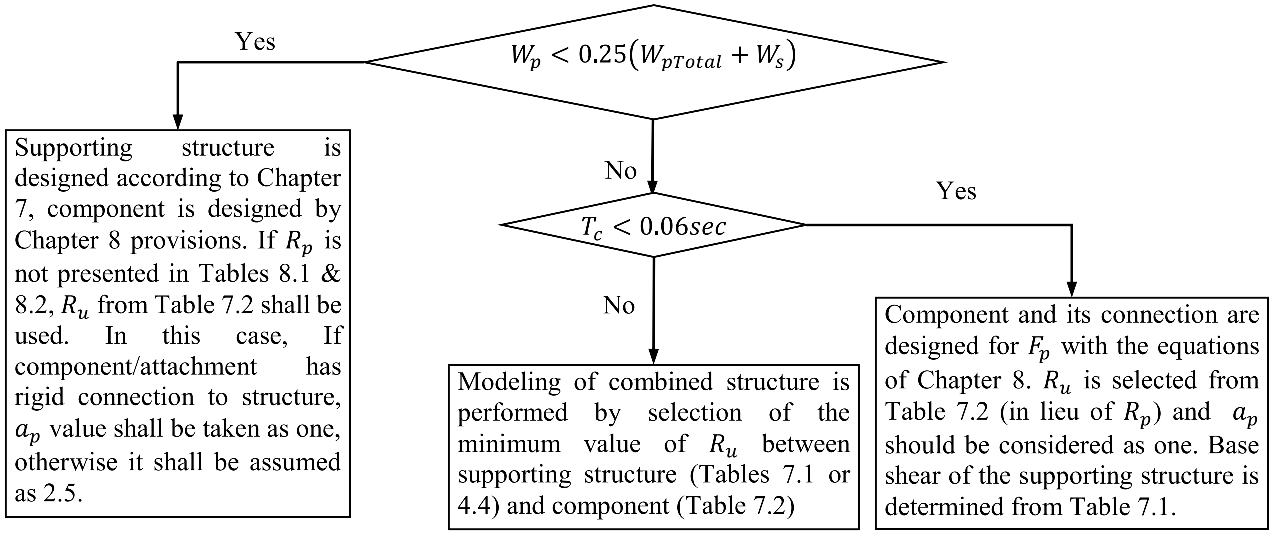
Table A1.1 α, β & γ Coefficients Corresponding to Section A2.6

h_x/H	α	β	γ	h_x/H	α	β	γ
1.00	2.103	8.347	1.000000	0.50	0.1094	0.9863	0.95573
0.99	2.021	8.121	1.000000	0.49	0.0998	0.9210	0.95143
0.98	1.941	7.898	1.000000	0.48	0.0909	0.8584	0.94683
0.97	1.863	7.678	1.000000	0.47	0.0826	0.7987	0.94189
0.96	1.787	7.461	1.000000	0.46	0.0749	0.7418	0.93661
0.95	1.714	7.248	0.999999	0.45	0.0578	0.6876	0.93097
0.94	1.642	7.037	0.999998	0.44	0.0612	0.6361	0.92495
0.93	1.573	6.830	0.999997	0.43	0.0551	0.5372	0.91854
0.92	1.506	6.626	0.999994	0.42	0.0494	0.5409	0.91173
0.91	1.440	6.425	0.999989	0.41	0.0442	0.4971	0.90443
0.90	1.377	6.227	0.999982	0.40	0.0395	0.4557	0.89679
0.89	1.316	6.032	0.999971	0.39	0.0351	0.4167	0.88864
0.88	1.256	5.840	0.999956	0.38	0.0311	0.3801	0.88001
0.87	1.199	5.652	0.999934	0.37	0.0275	0.3456	0.87033
0.86	1.143	5.467	0.999905	0.36	0.0242	0.3134	0.86123
0.85	1.090	5.285	0.999867	0.35	0.0212	0.2833	0.85105
0.84	1.038	5.106	0.999317	0.34	0.0185	0.2552	0.84032
0.83	0.938	4.930	0.999754	0.33	0.0161	0.2291	0.82901
0.82	0.939	4.758	0.999674	0.32	0.0140	0.2050	0.81710
0.81	0.892	4.589	0.999576	0.31	0.0120	0.1826	0.80459
0.80	0.847	4.424	0.999455	0.30	0.010293	0.16200	0.7914
0.79	0.804	4.261	0.999309	0.29	0.008769	0.14308	0.7776
0.78	0.762	4.102	0.999133	0.28	0.007426	0.12576	0.7632
0.77	0.722	3.946	0.998923	0.27	0.006249	0.10997	0.7480
0.76	0.683	3.794	0.998676	0.26	0.005222	0.09564	0.7321
0.75	0.646	3.645	0.998385	0.25	0.004332	0.08267	0.7155
0.74	0.610	3.499	0.998047	0.24	0.003564	0.07101	0.6981
0.73	0.576	3.356	0.997656	0.23	0.002907	0.06056	0.6800
0.72	0.543	3.217	0.997205	0.22	0.002349	0.05126	0.6610
0.71	0.512	3.081	0.996689	0.21	0.001878	0.04303	0.6413
0.70	0.481	2.949	0.996101	0.20	0.001485	0.03579	0.6207
0.69	0.453	2.820	0.995434	0.19	0.001159	0.02948	0.5602
0.68	0.425	2.694	0.904681	0.18	0.000893	0.02400	0.5769
0.67	0.399	2.571	0.993834	0.17	0.000677	0.01931	0.5536
0.66	0.374	2.452	0.992885	0.16	0.000504	0.01531	0.5295
0.65	0.3497	2.3365	0.99183	0.15	0.000368	0.01196	0.5044
0.64	0.3269	2.2240	0.99065	0.14	0.000263	0.00917	0.4783
0.63	0.3052	2.1148	0.98934	0.13	0.000183	0.00689	0.4512
0.62	0.2846	2.0089	0.98739	0.12	0.000124	0.00506	0.4231
0.61	0.2650	1.9062	0.98630	0.11	0.000081	0.00361	0.3940
0.60	0.2464	1.8068	0.98455	0.10	0.000051	0.00249	0.3639
0.59	0.2288	1.7107	0.98262	0.09	0.000030	0.00165	0.3327
0.58	0.2122	1.6177	0.98052	0.08	0.000017	0.00104	0.3003
0.57	0.1965	1.5279	0.97823	0.07	0.000009	0.00062	0.2669
0.56	0.1816	1.4413	0.97573	0.06	0.000004	0.00034	0.2323
0.55	0.1676	1.3579	0.97301	0.05	0.000002	0.00016	0.1965
0.54	0.1545	1.2775	0.97007	0.04	0.000001	0.00007	0.1597
0.53	0.1421	1.2002	0.96683	0.03	0.000000	0.00002	0.1216
0.52	0.1305	1.1259	0.96344	0.02	0.000000	0.00000	0.0823
0.51	0.1196	1.0547	0.95973	0.01	0.000000	0.00000	0.0418

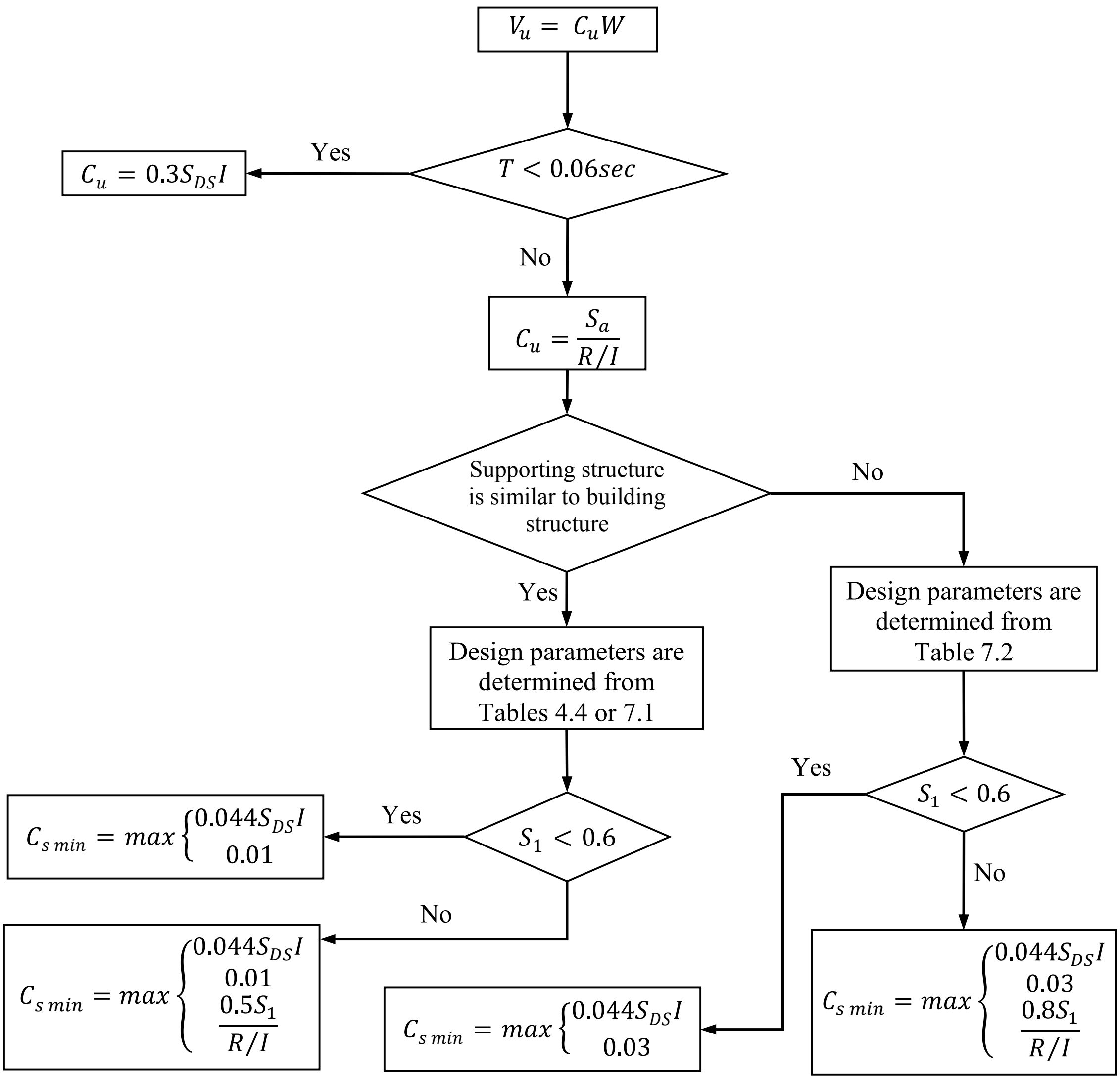
Appendix 3
Flowcharts for Seismic Design of Non-Building Structures & Non-Structural Components

Appendix 3. Flowcharts for Seismic Design of Non-Building Structures and Non-Structural Components

A3.1 Non-Building Structure and Non-Structural Component Distinction



A3.2 Base Shear Determination of the Supporting Structure



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