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Revision Memo

- 12/03** Technical update to the December 1997 version to incorporate the latest design provisions from ASCE-7, and state-of-the-practice engineering reports from various industry committees.
- Details:
- Updated references
 - Revised Basic design loads to match GP 04-01-03 & ASCE 7.
 - UBC-94 Seismic Procedures moved to appendix D.



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SCOPE

This section discusses the types and nature of the basic design loads that act on process plant structures. It also provides procedures for calculating the magnitude and distribution of design loads based on the requirements of the applicable Global Practices (GP) and industry codes and standards.

REFERENCES

GLOBAL PRACTICES

GP 04-01-03	Structural Design Loads
GP 04-01-04	Surge Vibration Design Loads
GP 04-03-02	Blast Resistant Buildings
GP 04-06-01	Reinforced Concrete Foundations
GP 04-06-02	Supporting Structures and Foundations for Heavy Machinery
GP 04-10-01	Concrete Design and Construction

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Section XXIX-A	Civil Design Data
Section XXIX-F	Machinery Foundation Design
Section XXIX-I	Blast-Resistant Structures

OTHER REFERENCES

ACI 318-02	"Building Code Requirements for Reinforced Concrete," American Concrete Institute, Detroit, MI
ACI 313-1997	"Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials"
AISC-1989	"Specification for Structural Steel Buildings - Allowable Stress Design and Plastic Design"
AISC	"Supplement No. 1 to the Specifications for Structural Steel Buildings - Allowable Stress Design and Plastic Design"
AISC-1994	"Load and Resistance Factor Design Specification for Structural Steel Buildings"
API 650-1988	"Welded Steel Tanks for Oil Storage"
ASCE 4-98	"Seismic Analysis of Safety-Related Nuclear Structures"
ASCE 7-02	"Minimum Design Loads for Buildings and Other Structures"
ASCE 37-02	"Design Loads on Structures during Construction".
UBC-1997	"Uniform Building Code"
EE.135E.91	"Process Plant Earthquake Performance and Design Criteria"
ASCE Petrochemical Energy Committee, "Wind Loads on Petrochemical Facilities," 1997.	
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ASSE/SAFE A14.3," Safety Requirements for Fixed Ladders"	
ASSE/SAFE A1264.1," Safety Requirements for Workplace Floor and Wall Openings, Stairs, and Railings Systems"	

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INTRODUCTION

This subsection is intended as a supplementary guide to **GP 04-01-03**, Structural Design Loads, and includes the calculation formulas, charts and procedures needed to meet the GP provisions for loads on plant structures. It can be used as a “roadmap” and interpretative guide for the key provisions of the referenced codes and standards in **GP 04-01-03**. It is not intended to supercede the GP and referenced codes and standards in the GP with this DP. In locations where the referenced documents are unfamiliar, this DP subsection may be used to assist in achieving the design intent. Consistent with the GP and reference codes and standards, this DP provides loading conditions and associated load combinations, to be considered for design, and the corresponding load factors, safety factors, and allowable stresses. The site specific data needed to determine environmental loads are usually provided in the Job Specifications (GI). [Section XXIX-A](#), Civil Design Data describes the methodology for determining/obtaining these data. The subsection also lists values for the seismic zone factors and design wind speeds for a number of ExxonMobil plant sites worldwide and provides procedures that can be used to determine design values for locations not listed.

[Appendix A](#) provides formulas for calculating the fundamental period of vibration for common process plant structures. This parameter is needed to calculate the earthquake loads on a structure, and the wind loads for flexible structures.

[Appendix B](#) provides the formulas for calculating the “gust effect factor” for determining the wind loads for flexible buildings and other structures according the **ASCE 7** methodology.

Examples are presented in [Appendix C](#) to illustrate load calculation procedures. In particular, several examples of wind and earthquake load calculations are included, as discrepancies in load calculations have often arisen in these areas.

Some jurisdictions require that UBC-94 be used for earthquake load calculations. [Appendix D](#) details the procedures for using UBC-94

Design loads for process equipment and support structures is the primary focus of this subsection. For detailed discussions of design loads for machinery foundations and blast-resistant structures, reference should be made to [Section XXIX-F](#), “Machinery Foundation Design,” and [Section XXIX-I](#), “Blast-Resistant Structures.”

BASIC DESIGN LOADS

General

The basic loads normally considered for the design of process plant structures are as follows:

Dead loads.

Live loads.

Fluid Forces

Self-Straining Forces.

Environmental loads including earthquake, snow, ice, rain, and wind loads.

Loads due to lateral earth pressure, groundwater pressure, or pressure of bulk materials

Construction loads including erection forces and transportation loads.

Operating loads including fluid, impact, piping restraint, surge vibration, thermal, and vibration loads.

Maintenance loads including bundle pull loads.

These basic loads are specified and combined for various load conditions to yield safe, reliable and economic designs for process plant structures. The nature of each basic design load is described in the following sections.

Dead Loads

Dead loads comprise the weight of all materials of construction incorporated into the structure. These include the weights of the structure, walls, floors, roofs, built-in partitions, finishes, cladding and fixed empty equipment and appurtenances permanently fastened to or supported on the structure. Dead load items frequently encountered in process plant designs are as follows:

- Structural components including platforms, walkways and stairs.
- Vessels, exchangers and other equipment permanently supported by the structure, not including any liquid or process chemical loads.
- Piping, valves, instruments, etc., attached to the supported equipment or otherwise supported by the structure.
- Insulation and fireproofing on the structure, equipment and piping.

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When calculating dead loads for the purposes of design, the actual weights of materials, structures, and fixed equipment should be used. These weights are usually given in Design Specifications, vendor data sheets, or material design manuals such as the **AISC** "Manual of Steel Construction." In the absence of specified data, the "Minimum Design Dead Loads" values given by the **ASCE 7** "Minimum Design Loads for Buildings and Other Structures," or a similar standard, may be used.

Live Loads

Live loads are the loads superimposed on or specified for a structure as a result of its use, operation or occupancy, but do not include construction or environmental loads such as snow load, rain load, wind load, earthquake load, or dead loads. Live loads include the following:

- Personnel, portable machinery, tools and equipment.
- Material to be temporarily stored during maintenance, such as exchanger parts, pipe and fittings, and valves, etc.
- Material normally stored during operation such as tools, maintenance equipment, catalyst and chemicals.
- Vehicular loads, maintenance equipment loads on floors or platforms, etc.

Certain other loads are treated as live loads for design purposes. These include impact, surge and machinery dynamic forces.

Generally live loads can be obtained from the data provided in the Job Specifications, Vendor's specifications and data sheets, or from national or local codes and standards, such as **ASCE 7**.

Shown in [Table 1](#) are selected typical live loads for the flooring, roofs and appurtenances of plant structures.

Table 1 - Live Loads For Plant Structures

COMPONENT DESIGN	LOADED AREA	MINIMUM LIVE LOAD
a. Floor Plate, Grating and Slabs	1. Walkways and Access Platforms	100 psf (4.8 kN/m ²)
	2. Platforms for Operating Storage or Maintenance Storage Loads	125 psf (6.0 kN/m ²)
b. Floor Framing and Bracing	1. Walkways and Access Platforms	60 psf (2.9 kN/m ²) of a Moving Concentrated Load of 1000 lb (4.4 kN) (2)
	2. Platforms for Operating Storage or Maintenance Storage Loads	Per a.2 (1)
c. Columns and Brackets	1. Walkways and Access Platforms	Per b.1 (1)
	2. Platforms for Operating Storage or Maintenance Storage Loads	Per a.2 (1)
d. Ladders	Ladder Framing and Rungs	See ASSE A14.3
e. Stairs	Stair Framing Treads	See ASSE A1264.1
f. Guard Railing and Posts for Platforms and Stairways		See ASSE A1264.1
g. Roof (Sloped or Flat)		20 psf (1.0 kN/m ²)

Notes:

- (1) May be reduced based on influence area per **ASCE 7**

Fluid Loads

Fluid loads are the gravity loads of liquid or solid materials in equipment and piping during operation or hydrotest. The weights of fluid materials in equipment and piping can be calculated from the corresponding fluid density and volume. These are usually provided in the Job Specifications, and vendor's specifications and data sheets. Fluid load defines structural actions in structural supports, framework, or foundations of a storage tank, vessel, or similar container due to stored liquid products. The product in a storage tank shares characteristics of both dead and live load. It is similar to a dead load in that its weight has a maximum calculated value, and the magnitude of the actual load may have a relatively small dispersion. However, it is not permanent; emptying and filling causes fluctuating forces in the structure, the maximum load may be exceeded by overfilling; and densities of stored products in a specific tank may vary. When their volumes cannot be well defined or they are highly variable, they should be treated as live loads.

It should be emphasized that uncertainties in lateral forces from bulk materials, are higher than those in fluids, particularly when dynamic effects are introduced as the bulk material is set in motion by filling or emptying operations. Accordingly, lateral loads from bulk materials should be included in loads due to lateral earth pressure, ground water pressure, or pressure of bulk materials

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Self-Straining Forces

Self-straining forces can be caused by differential settlement foundations, creep in concrete members, shrinkage in members after placement, expansion of shrinkage compensating concrete, and changes in temperature of members during the service life of the structure. In some cases, these forces may be a significant design consideration. In concrete or masonry structures, the reduction in stiffness that occurs upon cracking may relieve these self-straining forces, and the assessment of loads should consider this reduced stiffness. Forces resulting from temperature changes in piping and equipment should be treated as live loads due to the uncertainties in determining the effects of equipment and piping temperature changes on the structure.

PROCESS LOADS

The operation of equipment or machinery in a process unit generally results in process loads, which affect the design of the equipment, its support structure and foundation. These loads include the effects of vibration, impact, surge and thermal expansion/contraction. Process loads are often dynamic in nature. However, for design purposes their effects are usually determined using static equivalent loads. Types of process loads are discussed below.

Surge Loads

Surge loads, as addressed herein, relate to the operation of fluid-solid units. They do not include the surge of fluid flow in piping and other equipment. Surge is the lateral cyclic loading, which results from the upward flow of fluids (gases, liquids) through a solid-particle bed in fluidized-solids units. These fluids flow with sufficient velocity to overcome the weight of the bed, such that the particles are suspended by the flowing fluid. Large masses of solids in these fluidized beds can separate and recombine with resulting inertia (surge) forces. Such surge forces act horizontally against the vessel wall and are transmitted from the vessel to the support structure. Surge loads are considered for both normal operating conditions and abnormal conditions associated with process startup and upset. The effect of surge is treated in design in terms of equivalent static loads.

GP 04-01-04, "Surge Vibration Design Loads," provides the procedures for calculating the magnitude and distribution of surge loading for common types of fluid-solid process units. The magnitude of the surge force is proportional to the weights of fluidized solids contained in the process vessels and piping. These weights, determined from the fluid bed volume and densities in each vessel and piping, should be obtained from the process designer for the following conditions:

- **Normal surge:** the maximum "operating" weight, with corresponding fluid-bed height and density, in the process vessels and piping during normal operation of the unit.
- **Abnormal surge:** the maximum upset fluid-bed weight corresponding to the most severe combination of fluid-bed height and density in the process vessels and piping during startup or upset of the unit.

Both conditions should be investigated during design.

Machinery Dynamic Forces

These forces are caused by the unbalance of moving parts of rotating and reciprocating equipment. The unbalanced moving mass result in dynamic (inertial) loading and vibration of the equipment and support structure.

GP 04-06-02, "Supporting Structures and Foundations for Heavy Machinery," specifies the design dynamic forces for heavy rotating machinery, normally encountered in refineries and chemical plants, such as motors and centrifugal compressors. The design dynamic loading for reciprocating machinery are usually provided by the vendor or manufacturer. [Section XXIX-F](#) "Machinery Foundation Design" describes the loads for rotating and reciprocating machinery, and gives information on the types of foundations normally used to support this machinery. It also provides procedures to calculate machinery dynamic loads, and the methodology for the analysis and design of foundations supporting heavy machinery. In addition, reference can be made to the text "Design of Structures and Foundations for Vibrating Machines" by Arya, O'Neill and Pincus.

In addition to conventional refinery and chemical plant processing machinery, there are certain heavy equipment items in synfuel and other chemical processes that can generate dynamic loads during operation. These include mixers, rotating drums, skip hoists, primary and secondary crushers, and retorts. Little experiential data, however, is available for determining dynamic loads associated with the operation of these heavy equipment items. These loads should be estimated by or with the assistance of the equipment manufacturer. Dynamic loads for such equipment are usually based on vendor data or on operating experience for similar equipment.

Thermal Loads

Thermal loads result from the partial or complete restraint of equipment or piping which are subjected to a change in temperature from the ambient. Structural members subjected to a change in temperature are treated as self-straining forces. Thermal loads include:

- Forces due to anchorage of piping and equipment
- Friction forces caused by sliding or rolling of equipment on supporting members

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For a piping system, thermal loads on supports should be obtained from the results of the stress (piping flexibility) analysis of the piping system. A computerized thermal stress analysis is normally performed by the pipe stress engineers. Restraints are included in the analysis at the location of the pipe anchors. The resultant reactions on the anchors are obtained from the analysis and applied as loads on the support structure.

For an exchanger, drum, furnace, etc., the operating temperature for the particular piece of equipment, given in Design Specification, should be used to calculate thermal growth and resultant load on the supports. This resultant load should then be compared to the frictional force required to cause sliding at the support, using the following coefficients of static friction, as specified in **GP 04-01-03**:

PTFE on PTFE*	0.10
Steel on Steel	0.40
Steel on Concrete	0.45

*PTFE are prefabricated teflon expansion pads.

For flexible supports, consideration should be given to the support deflections relative to the thermal growth of the vessel. In extreme cases this growth may be accommodated with forces in the supports not large enough to overcome friction.

Impact Loads

Traveling cranes, elevators, hoists and similar equipment can impart impact loads on their tracks and supports during operation. These loads are usually accounted for in design by applying impact factors to the normal static loads being handled. **ASCE 7** specifies impact loads for various handling facilities, in terms of impact factors to be applied to the normal static loads being handled. A listing of such loads is shown in [Table 2](#). In addition, all elevator loads should be increased by 100% for impact.

Table 2 - Impact Loads For Traveling Cranes, Elevators, And Hoists

HANDLING FACILITY	LOAD APPLICATION	IMPACT LOAD
Traveling Crane Runway	1. Vertical on the supports	25% of the maximum wheel loads
	2. Longitudinal on the supports	10% of the maximum wheel loads applied at the top of the runway
	3. Lateral on the supports	20% of the sum of the lifted load and the weight (mass) of the crane trolley applied at the top of the runway and considered as acting in either direction normal to runway
Trolley Beams	1. Vertical on the trolley beams	25% of the maximum wheel loads
	2. Longitudinal on the trolley beams	10% of the wheel loads
Davits	1. Vertical on the davit	25% of the lifted load
	2. Lateral on the davit	20% of the weight (mass) of the moving equipment

Dynamic effects from vehicular traffic are also treated as impact loads in the design of bridges, in accordance with **AASHTO** Specifications, as discussed below under "Vehicular Loads."

Impact loads for other moving equipment should be determined in conjunction with the equipment manufacturer.

Solid Flow Loads

Silos, bin, and bunkers storing granular materials are subject to pressures against the walls and bottoms due to the stored or flowing material. Significant pressure increases above the static values may be caused by filling, withdrawal of material, arching, collapse of arched material, aeration, and eccentric discharge.

The effects of the motion of the stored material can be included in design by applying overpressure factors or impact factors to the static pressure values. There are various methods to calculate the pressures. American practice is, generally, to use Janssen's formula whereas in parts of Europe, Reimbert's method is preferred. Rankine's method is sometimes used for silos having small height to diameter ratios. Methods other than Janssen's may be used to compute wall pressures. There are a large variety of hopper pressure formulas available in the literature including Jenike, McLean and Walker. All are based on different assumptions and may yield significantly different pressure distributions.

Refer to **ACI 313** for the calculation of loads from the motion of stored materials. **ACI 313** describes procedures to calculate static material pressures, and provides the overpressure and impact factors to be applied to the static pressure values, to evaluate the design loads due to solid flow.

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ENVIRONMENTAL LOADS

The most common environmental loads generally applicable to process plant facilities result from snow, ice, rain, wind and earthquake. Environmental loads are based on the meteorological and seismological conditions specific to the plant site under consideration. The design loads resulting from these conditions are extreme and rare events. They are expected to occur infrequently during the normal design life of the facilities. Consequently, when considering these loads with other design loads, reduced safety factors or load factors are used in the design process, to reflect the low probability that the loads will all achieve their design magnitudes at the same time. Furthermore, some environmental loads (wind, earthquake) are assumed not to occur simultaneously. Specific considerations for each type of environmental load are discussed below.

Snow Loads

Snow loading results from the accumulation of snow on building roofs, open structures, and equipment. Design snow loading is determined from the ground snow load specified for the plant site. The ground snow load is based on the weather records of snow on the ground for that geographic location.

Snow loads to be used for sites in the U.S.A. are given in **ASCE 7**. For other locations, ground snow load should be based on local conditions, and may be specified in local codes and standards. For such locations, ground snow loads should be based on a 50 year mean recurrence interval (MRI). For instance, the loads given in the Canadian Building Code are based on a 30 year MRI and should be adjusted to a 50 year MRI. At some locations, heavy rain may fall on roofs already sustaining snow loads. At such locations, a rain-on-snow surcharge load is applied.

Ice Loads

Ice loading results from the accretion of ice on exposed structural members components, appurtenances, and equipment. Ice loads should be considered in the design of ice-sensitive structures. Ice-sensitive structures are structures for which the effect of an atmospheric icing load governs the design of part or all of the structure. These structures include, but is not limited to, lattice structures, guyed masts, overhead lines, light suspension and cable-stayed bridges, open catwalks and platforms, flagpoles, and signs. Most ice on structures is formed from freezing rain. Freezing rain is rain or drizzle that falls into a layer of subfreezing air at the earth's surface and freezes on contact with the ground or an object to form glaze ice. Glaze ice is clear high-density ice. Ice is sometimes formed from snow or in-cloud icing.

Ice thickness to be used for sites in the U.S.A. are given in **ASCE 7**. For other locations, ice thickness should be based on local conditions, and may be specified in local codes and standards. For such locations, ice thickness should be based on a 50-year mean recurrence interval (MRI).

Wind speeds are often low to moderate in freezing rainstorms and revised lower wind speeds are provided in **ASCE 7** for these conditions. This concurrent wind speed used with the nominal ice thickness is based on both the winds that occur during the freezing rainstorm and those that occur between the time the freezing rain stops and the time the temperature rises to above freezing. When the temperature rises above freezing, **ASCE 7** assumes that the ice melts enough to fall from the structure. In colder regions, the ice will generally stay on structures for a longer period of time following the end of a storm that results in higher concurrent wind speeds. The results of an extreme value analysis show that the concurrent wind speed does not change significantly with mean recurrence interval.

Rain Loads

Where adequate slope to drain does not exist, or where drains may be blocked, rainwater can collect and pond in low areas. As additional water flows to such an area, the structure tends to deflect more, allowing a deeper pond to form there. If the structure does not possess enough stiffness to resist this progression, localized overloading may result.

When evaluating the potential for rain ponding loads, consideration should be given to the long-term deflection of the structure under dead load. The ponding potential increases as the flexibility of the framing increases.

Generally, roofs with a slope of 1/4 in/ft (20 mm/m) or more are not susceptible to ponding instability from rain, unless drain blockages allow deep ponds to form. Avoiding deep ponding if one drain becomes blocked is particularly important for flexible roof systems.

Each portion of a roof should be designed to sustain the load of all rainwater that could accumulate on it if the primary drainage system for that portion is blocked. Ponding instability should be considered in this situation.

The **AISC** "Manual of Steel Construction," Part 5, Sect. K2, provides a procedure for evaluating ponding instability.

Wind Loads

Wind load is the pressure exerted on an object in the path of the wind. The design wind loads for structures and equipment are based on the design wind speed specified for the plant site, determined from wind data for the site. These loads also depend on the surface roughness and topography of the site, and the geometric shape and height of the structure. In addition, dynamic amplification of wind loading occurs for flexible structures.

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GP 04-01-03 “Structural Design Loads,” gives the requirements governing the determination and application of wind loads for the design of structures. The requirements in this GP are based on the wind load calculation procedures of **ASCE 7**.

Before wind loads can be determined, the design wind speed for the site must be established. [Section XXIX-A](#), “Civil Design Data,” gives procedures for determining the design wind speed for a site. **ASCE 7** defines the basic design wind speed as the 3-second gust wind speed, at 33 ft (10 m) above ground level, with an annual probability of occurrence of 0.02, associated with flat, open country terrain (Exposure C category). For a particular project, the design wind speed is usually stated in the **General Instructions and Information (GII) Specification** for the plant. In earlier versions of **ASCE 7** [Section XXIX-A](#), the design wind was based on a “fastest mile wind speed” thus, one should be clear on the wind speed basis when taking information from the GII’s of past projects. The 3-second gust design wind speeds for selected **ExxonMobil** locations is included in [Section XXIX-A](#).

Outside the U.S.A., local codes and standards for the calculation of wind design loads may be used, subject to the requirements of **GP 04-01-03**. Such codes may be based on a design wind speed averaged over a certain time interval or “duration,” different from the basic 3-second gust wind speed. [Section XXIX-A](#) of this DP describes how to convert between wind speeds averaged over different durations.

Earthquake Loads

Earthquake loads are the inertial loadings imposed on a structure resulting from ground motions during an earthquake. The magnitude of the design earthquake load is a function of the seismicity of the location, the flexibility, ductility and damping of the structure, and the soil-structure interaction. The seismicity of the location is expressed in terms of an acceleration, a seismic zone factor or acceleration coefficient. These are related to the effective peak ground acceleration (EPA) and reflect the seismicity of the site. Seismic accelerations, zone factors or acceleration coefficients are usually given in the **General Instructions and Information (GII) Specification** for the plant. [Section XXIX-A](#) provides a list of current seismic probability coefficients for selected **ExxonMobil** locations, for use with **GP 04-01-03** and this DP.

GP 04-01-03 provides the requirements for determining and applying earthquake loads for the design of structures based on the earthquake load provisions of **ASCE 7**. The following is a brief description of these building codes/standards.

ASCE 7 - This standard is the reference document for building codes. The 2002 earthquake provisions are based on the 2000 NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings and Other Structures. The lateral earthquake forces of **ASCE 7** are based on strength design (limit state design or load and resistance factor design).

National Earthquake Hazards Reduction Program (NEHRP) - This program is sponsored by the Federal Emergency Management Agency (FEMA), the U.S. Geological Survey (USGS), and National Institute of Standards and Technology (NIST). It defines comprehensive zone specific lateral forces, methods of analysis, and detailing requirements seismic design.

International Building Code (IBC) An initiative was developed to define a common US based International Building Code (IBC) which would harmonize the seismic and other provisions for buildings and other structures. The load provisions in the IBC are essentially the same as ASCE 7. The GP is based on **ASCE 7** or other national codes yielding comparable design for plant facilities.

Uniform Building Code (UBC) - This is a Model Building Code which was used primarily in the western United States and overseas in some countries including Saudi Arabia and Dominican Republic, etc. It was supported by the International Conference of Building Officials (ICBO). The UBC seismic provisions are based primarily on the recommendations for earthquake design from the Structural Engineers Association of California (SEAOC) Blue Book. This code has been superseded by the International Building Code (IBC). Some jurisdictions still require the use of either UBC 94 or UBC 97 and the procedures are left in this DP for reference. The UBC was historically the basis for Exxon’s seismic design requirements in the IPs. UBC 97 was used as a basis for the California Building Code 2001 with additions and revisions.

The minimum design lateral forces prescribed in the UBC 94 edition are at a service level, hence design is normally based on the working stress approach or used with applicable load factors. Consequently, because of the difference in philosophy for lateral force levels, when comparing the base shear of the UBC with **ASCE 7** or UBC 97, it (UBC 94) must be multiplied by a factor of 1.4 to 1.5.

SEAOC Blue Book - This document, focusing primarily on California and other western states, addresses the various zones, structural systems, lateral load resisting systems and detailing requirements. It is revised frequently to reflect new information and learnings after major seismic events.

A brief comparison of the primary static force requirements of the UBC and **ASCE 7** are shown below:

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Table 3 - Earthquake Loads; Primary Static Force Requirements

SEISMIC PARAMETER	ASCE 7	UBC-97	UBC-94
Base Shear	$V = C_s W$	$V = \frac{C_v I}{R \cdot T} W$	$V = \frac{ZIC}{R_w} W$
Coefficient C or C_s	$C_s = \frac{S_{DS}}{R/I}$	Based on soil profile and Z	$C = \frac{1.25 S}{T^{2/3}}$
Upper Limits.	$C_s \leq \frac{S_{D1}}{T \cdot (R/I)}$	$V \leq \frac{2.5 \cdot C_a \cdot I}{R \cdot T} W$	$C \leq 2.75$
Lower Limits for Buildings	$C_s \geq 0.044 S_{DS} I$	$V \geq 0.11 \cdot C_a \cdot I \cdot W$	$\frac{C}{R_w} \geq 0.075$
Lower Limits for Non-Buildings	$C_s \geq 0.14 S_{DS} I$	$V \geq 0.56 \cdot C_a \cdot I \cdot W$	$\frac{C}{R_w} \geq 0.25$
Seismic Zone Factor	S_s and S_1 acceleration	Z	Z
Importance Factor	I	I	I
Response Modification Coefficient	R, Strength level response	R , strength level response	R_w , working level response
Site Class / Soil Profile / Site Coefficient	Soil Profile Type A B C D, E or F	Soil Profile Type A B C D, E or F	Site Coefficient $S_1 = 1.0$ $S_2 = 1.2$ $S_3 = 1.5$ $S_4 = 2.0$

The seismic provisions of **ASCE 7** provide the minimum criteria consistent with life safety considerations. Some structural and non-structural damage can be expected because the design allows for inelastic energy dissipation by utilizing the deformability of the structural system. For ground motions in excess of the design levels, the intent is that there be a low likelihood of collapse.

ACCIDENTAL LOADS

In addition to the normally specified design loads for facilities in refineries and chemical plants some facilities may be subjected to loading from the effects of accidental events such as explosions and fires. The AIChE Center for Chemical Process Safety (CCPS) provides guidelines for addressing these phenomena. [Section XV-H](#), "Safety in Plants - Blast Protection and Fireproofing" covers the company practice concerning designing for these events. The following paragraphs discuss the structural design for explosions and fires.

Blast Loads

Blast loads may result from accidental vapor-cloud explosions in a process plant. [Section XV-H](#) provides techniques for determining the blast effects from such incidents. **GP 04-03-02**, "Blast Resistant Structures" provides design blast loads and design criteria for buildings and structures required to be blast resistant. Design blast loading for such facilities is dependent on the spacing from a potential explosion source, and the function of the building. [Section XXIX-I](#), "Blast Resistant Structures," describes the special design requirements for a blast resistant process plant building and provides procedures to calculate blast loads. ER&E Technical Manual **TMEE065** provides details of the procedures for analyzing and designing blast-resistant structures.

Fire Loads

Severe thermal effects can result from certain accidental conditions such as fires, or spill of cold or hot products. However, these are not normally explicitly considered in structural design, except in special cases.

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OTHER LOADS

Hydrostatic Pressure and Buoyancy

Inground structures such as basins, pipes, and vessels that are located below the level of the natural ground water are subject to hydrostatic pressure including buoyancy.

Hydrostatic pressure at a point below ground water level is the force per unit area due to the weight of water above that point. Hydrostatic pressure acts in a direction normal to the surface of the submerged object. **GP 04-01-03** requires that the water level be assumed to be at grade.

Buoyancy is an upward force equal to the weight of water displaced by submergence of an object. Unless properly designed for, buoyancy forces can cause a submerged structure to be lifted from their intended design positions.

Lateral Earth Pressure

Soil exerts a lateral pressure on any structure which keeps the soil from assuming its natural angle of repose. Inground structures such as retaining walls, thrust blocks, deadmen, and sulfur pits are subject to lateral earth pressure loads. This lateral earth pressure is a function of the properties of the confined soil, embedded depth of the structure, slope of and loading on soil adjacent to the structure, and surface roughness of the structure in contact with the soil.

There are three states of lateral earth pressure for a soil-structure system:

- Active: structure moves away from soil
- Passive: structure pushes against soil
- At rest: no movement of structure

In general, the active case is assumed for most retaining walls. However, in some situations, if insufficient movement of the wall takes place the active earth pressure condition cannot be developed. In such a case, the soil would be in the at rest condition, resulting in higher lateral pressures. For such situations the designer should obtain the appropriate input from a geotechnical specialist.

Test Loads

Storage tanks, drums, process vessels, and piping are often proof-tested by filling with water. Test load refers to the weight of water necessary for such testing. When there is more than one vessel to be tested in a structure, the vessels may be tested concurrently or separately. Structural design should be based on the worst possible loading condition.

The weight of water necessary to pressure test vessels, equipment or piping is usually provided in the Job Design Specifications, and vendor's specifications and data sheets. The test load can be computed from the water volume contained during test, determined from the equipment size.

Construction Loads

These loads imposed on a partially completed or temporary structure during and as a result of the construction process. Construction loads include, but are not limited to, materials, personnel and equipment, such as loading from guy rigging, shoring, etc. In addition, inertial loading effects may result from ground transmitted vibration due to blasting or pile driving.

These loads, caused by the erection and construction of the structure or equipment, are dependent on the construction methods and equipment used by the contractor. The contractor should therefore, specify erection and construction loads. Further information can also be obtained from ASCE 37-02 "Design Loads on Structures during Construction".

Maintenance Loads

Maintenance loads are temporary loads caused by the dismantling, repair, or painting of equipment or structures. Maintenance loads include davit loads, and heat exchanger bundle pulling forces.

As specified in **GP 04-01-03**, the bundle pull force acting on a heat exchanger support structure is taken as equal to the bundle weight. This force should be applied longitudinally, at the centroid of the tube bundle. The shear force due to bundle pulling should be assumed to be transmitted solely through the fixed shell support, for design purposes.

Per **GP 04-01-03**, davit design loads (excluding manhole davits) consist of the weight of the heaviest piece of equipment that they may be required to lift, plus the weight of rigging equipment, plus the corresponding impact loads. A minimum davit design load of 1000 lb (450 kg) should be applied.

Other maintenance loads associated with the dismantling, repair, or painting of equipment or structures should be determined in conjunction with the contractor performing the work.

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Transportation Loads

Modern construction techniques often involve the transportation of large modules of a facility by ship, rail and/or road to the construction site. The loads caused by such transport can be significant and sometimes governing in the design of these modules and therefore should be included during detailed engineering.

For structures and equipment that are transported from a fabrication site to the installation site, significant inertial forces may result from the motion of the structure or equipment. The magnitude of the loads will depend on the mode of transportation and the motions induced, and should be determined in conjunction with the transport company.

Vehicular Loads

Appropriate design of roads, culverts, and pipeline and electrical crossings requires information on the types of vehicles anticipated. This information includes vehicle wheel and axle arrangement and loads, track arrangements and loadings, and traffic density by type and speed.

When detailed traffic data is unavailable, the single-axle loadings prescribed by **AASHTO** for an H-20 truck form a reasonable minimum design basis for most plant roads. In addition, main plant roads should be designed for a minimum tandem-axle load of 50 short tons (45 tonnes). A sketch showing axle and wheel arrangements for these loadings is included in [Figure 1](#).

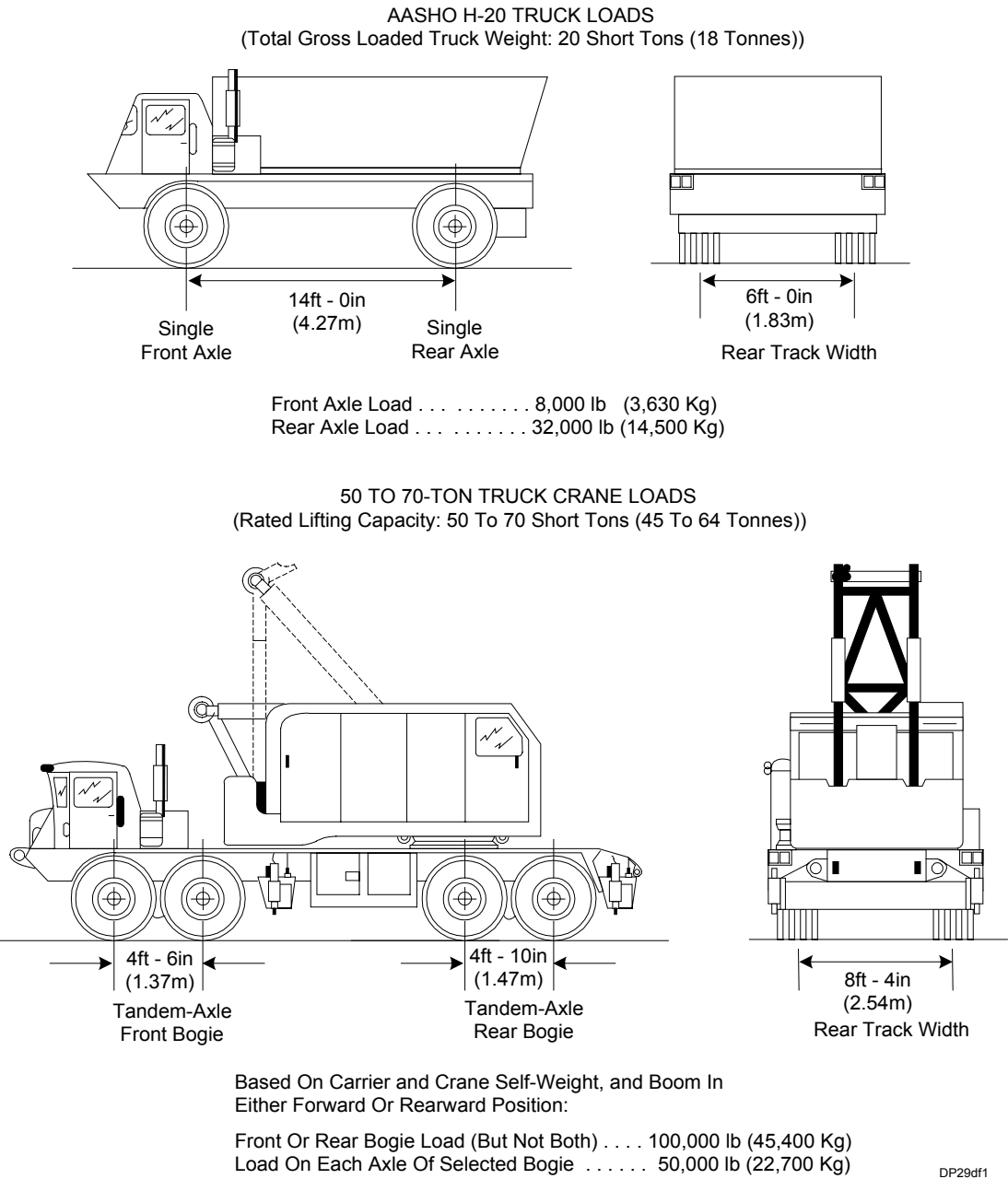
Bridges, trenches, and other structures accessible to vehicles should be designed to withstand the impact effects of truck loading, as defined by **AASHTO Standard Specifications for Highway Bridges** as follows:

$$I = \frac{50}{L + 125}$$

where:

- I = Impact fraction (maximum 30 percent)
- L = Length in ft of the portion of the span that is loaded to produce the maximum stress in the member

Figure 1 - Minimum Loads For Roadway And Culvert Design



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WIND LOAD CALCULATION PROCEDURES

SUMMARY OF KEY PROVISIONS

Wind loads for the design of structures, equipment and plant buildings are to be in accordance with **ASCE 7** and the ASCE guide "Wind Loads on Petrochemical Facilities" as specified per **GP 04-01-03**. Below is a summary of primary provisions for calculating these loads.

- The design wind pressure (q_z) and the design wind force (F) for buildings and other structures per the GP and this DP are based on the procedures specified in **ASCE 7** or a similar wind load standard. Force coefficients (C_f) and projected areas normal to the wind (A_f) for petrochemical structures, such as pipe racks, vessels with appurtenances, and equipment support structures are specified and discussed herein.
- The basic (design) wind speed, (V) for the site is defined as the 3-second gust speed corresponding to a 50-year mean recurrence interval, at a height of 33 ft (10 m) and for an "Exposure C" terrain conditions. The *basic wind speed* corresponds to a 50-year mean recurrence interval (annual probability of 0.02 of being equaled or exceeded in one year), and is dependent on location. It is usually provided in the Job Specifications (GII). Values for some ExxonMobil sites are listed in [Section XXIX-A](#).
- Other Exposure categories (B or D) may be used, subject to the Owners approval or, where the terrain conditions of the site justify.
- The gust effect factor (G) should be taken equal to 0.85 unless the structure is flexible (i.e., has a natural frequency less than 1 Hz).

ASCE 7 METHOD

The procedures provided in **ASCE 7** standard serve as the basis for determining the wind loads on petrochemical plant facilities. A summary of the wind calculation procedure plus supplementary recommendations for calculation of wind on petrochemical plant structures are provided in the following sections.

Velocity Pressure

The velocity pressure, q_z , at elevation z for buildings and structures is calculated as follows:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \quad (0.613 K_z K_{zt} K_d V^2 I)$$

where:	q_z	=	Velocity pressure at height z above ground, psf (kPa)
	K_z	=	Velocity pressure coefficient evaluated at height z
	K_{zt}	=	Topographic factor
	K_d	=	Wind Directionality factor
	V	=	Basic wind speed, mph (m/sec)
	I	=	Importance factor

The *velocity pressure coefficient*, K_z , reflects the dependence of the velocity pressure on the height above ground and on the roughness of surrounding terrain (exposure category). For purposes of design exposure categories are limited to the following:

- Exposure C - Open terrain with scattered obstructions having heights generally less than 33 ft (10 m). This category applies to typical process plant facilities.
- Exposure D - Flat, unobstructed areas exposed to wind flowing over open water for a distance of at least 1 mile (1.6 km). Exposure D extends inland from the shoreline a distance of 1,500 ft (460 m) or 10 times the height of the building or structure, whichever is greater. Shorelines in Exposure D include inland waterways, the great lakes and coastal areas of California, Oregon, Washington State and Alaska. It excludes shorelines in hurricane prone regions.

However, in some situations use of Exposure B may be justified, such as for evaluating existing structures surrounded by the appropriate terrain condition within a plant site. Exposure B is defined as "urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger." See **ASCE 7** for aerial photographs showing typical exposure types.

Velocity Pressure Exposure Coefficient, K_z , can be determined from [Table 4](#).

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Table 4 - Velocity Exposure Coefficient

HEIGHT ABOVE GROUND LEVEL, z		EXPOSURES		
ft	m	B	C	D
0-15	0-4.6	0.57	0.85	1.03
20	6.1	0.62	0.90	1.08
25	7.6	0.66	0.94	1.12
30	9.1	0.70	0.98	1.16
40	12.2	0.76	1.04	1.22
50	15.2	0.81	1.09	1.27
60	18	0.85	1.13	1.31
70	21.3	0.89	1.17	1.34
80	24.4	0.93	1.21	1.38
90	27.4	0.96	1.24	1.40
100	30.5	0.99	1.26	1.43
120	36.6	1.04	1.31	1.48
140	42.7	1.09	1.36	1.52
160	48.8	1.13	1.39	1.55
180	54.9	1.17	1.43	1.58
200	61.0	1.20	1.46	1.61
250	76.2	1.28	1.53	1.68
300	91.4	1.35	1.59	1.73
350	106.7	1.41	1.64	1.78
400	121.9	1.47	1.69	1.82
450	137.2	1.52	1.73	1.86
500	152.4	1.56	1.77	1.89

Notes:

Linear interpolation may be used for intermediate values of height Z.

K_z can be calculated from:

$$K_z = 2.01 \left(\frac{z}{z_g} \right)^{2/\alpha} \quad \text{for } 15 \text{ ft (4.6 m)} \leq z \leq z_g$$

$$K_z = 2.01 \left(\frac{15}{z_g} \right)^{2/\alpha} \quad \text{for } z < 15 \text{ ft (4.6 m)}$$

where: z = Height above local ground level in ft (m)

z_g = Gradient height in ft (m)

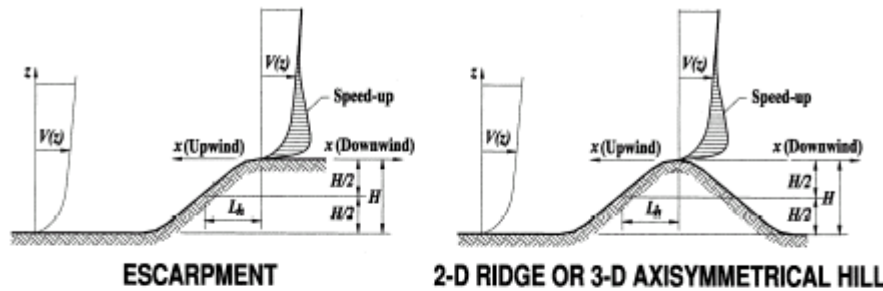
α = Power law coefficient

EXPOSURE	α	z_g ft (m)
B	7.0	1,200 (366)
C	9.5	900 (274)
D	11.5	700 (213)

The *importance factor*, I , should be specified by the Owner. For most ordinary cases, $I = 1.0$. For a more detailed discussion on the selection of importance factor, see [Section XXIX-A](#).

The *topographic factor*, K_{zt} , accounts for the higher wind speeds that may be experienced by structures sited on the upper half of an isolated hill or escarpment than those situated on level ground. K_{zt} is equal to 1.0 if no such terrain features are present, otherwise it should be determined from the following equation and tables:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$



ESCARPMENT

2-D RIDGE OR 3-D AXISYMMETRICAL HILL

Topographic Multipliers for Exposure C										
H/L _h	K ₁ Multiplier			x/L _h	K ₂ Multiplier		z/L _h	K ₃ Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

- (1) For values of H/L_h , x/L_h and z/L_h other than those shown, linear interpolation is permitted.
- (2) For $H/L_h > 0.5$, assume $H/L_h = 0.5$, and substitute $2H$ for L_h in x/L_h and z/L_h .
- (3) Multipliers assume that wind approaches the hill or escarpment along the direction of maximum slope.
- (4) Effect of wind speed-up shall not be required to be accounted for when $H/L_h < 0.2$ or when $H < 15$ ft (4.5 m) for Exposure C & D, or < 60 ft (18 m) for exposure B.
- (5) Notation:
 - H: Height of hill or escarpment relative to the upwind terrain, in ft (m).
 - L_h : Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in ft (m).
 - K_1 : Factor to account for shape of topographic feature and maximum speed-up effect.
 - K_2 : Factor to account for reduction in speed-up with distance upwind or downwind of crest.
 - K_3 : Factor to account for reduction in speed-up with height above local terrain.
 - x: Distance (upwind or downwind) from the crest to the building site, in ft (m).
 - z: Height above local ground level, in ft (m).

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The directionality factor K_d , accounts for two effects: (1) The reduced probability of maximum winds coming from any given direction and (2) the reduced probability of the maximum pressure coefficient occurring for any given wind direction. The factor should only be used in conjunction with the ASCE load combination factors given in the load combination section. It should be determined from the following table:

Table 5 - Directionality Factor, K_d

STRUCTURE TYPE	DIRECTIONALITY FACTOR, K_d
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks and Similar Structures	
Square	0.95
Hexagonal	0.95
Round	0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

This table is taken from ASCE 7-02. It is noted that the K_d value for round chimneys, tanks, and similar structures is given as 0.95 in recognition of the fact that the wind load resistance may not be exactly the same in all directions as implied by a value of 1.0. A value of 0.85 might be more appropriate if a triangular trussed frame is shrouded in a round cover. 1.0 might be more appropriate for a round chimney having a lateral load resistance equal in all directions.

Wind Forces

The wind force on a structure is calculated from the following:

$$F = q_z G C_f A_f$$

where: F = Design wind force, lbs (N)

q_z = Velocity pressure evaluated at height, z , of the centroid of area A_f using the defined exposure. Psf (N/m²)

G = Gust effect factor

C_f = Force coefficient

A_f = Projected area normal to wind, except where C_f is specified for the actual surface area, ft² (m²)

The *gust effect factor*, G , accounts for the loading effects in the along-wind direction due to the effect of wind turbulence on the structure. It also accounts for along-wind loading due to dynamic amplification for flexible buildings and structures. It does not include cross-wind loading effects, vortex shedding, instability due to galloping and flutter, or dynamic torsion effects. It is determined as follows:

- For structures located in Exposure C and D terrain with a natural frequency equal to or greater than 1 Hz, G may be taken equal to 0.85.
- For structures with a natural frequency less than 1 Hz, G should be calculated according to the procedure provided in [Appendix B](#).

The *force coefficient*, C_f , accounts for along-wind effects due to the shape or drag of the structure. The selection of force coefficient for petrochemical structures/equipment is discussed in the next section.

The *projected area*, A_f , represents the area normal to the wind force. The calculation of projected areas for petrochemical structures is discussed in the next section.

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WIND ON PETROCHEMICAL STRUCTURES

Guidelines for calculating wind on process equipment and petrochemical support structures are discussed in this section. It should be noted that, shielding effects due to the presence of other buildings and structures are not normally considered in the determination of overall wind loads, excepted as provided below. These guidelines are based, in general, on the **ASCE** Petrochemical energy committee publication, "Wind Loads on Petrochemical Facilities".

Pipe Racks

Shielding effects for a group of overhead parallel pipelines and cable trays should be calculated based on wind striking at an angle from the horizontal (slope of 1 to 10). The tributary area should be determined as follows.

$$A_f = (D + 0.1W) L$$

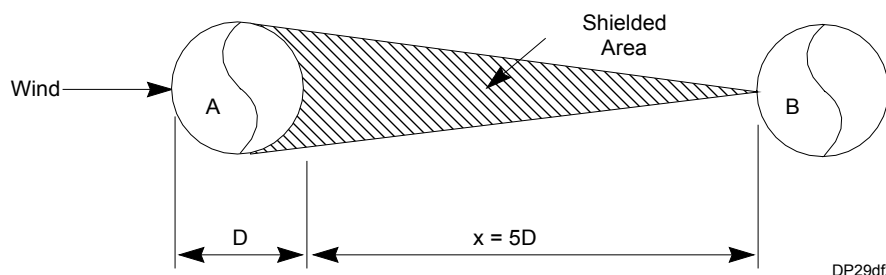
- where:**
- A_f = Tributary area of piping or cable trays, excluding any structural framing ft² (m²)
 - D = Diameter of largest pipe or height of cable tray. If unknown assume 12 in. (0.30 m)
insulated pipes, outside diameter = 1.3 ft (0.40 m)
 - W = Width of rack ft (m)
 - L = Length of rack ft (m)

Notes:

- (1) Force coefficients for piping should be in accordance with [Table 6](#) and should be 0.7 as a minimum.
- (2) Force coefficients for cable trays should be 2.0.
- (3) Wind loads on structural framing should be calculated with a force coefficient of 1.8 applied to all structural members without shielding.

Shielding effects in rows of pipes are included in the equation above. The shielding effect of one line on another is shown in [Figure 2](#).

Figure 2 - Wind Shielding On Parallel Pipes



Note: When "X" is more than 5D, no appreciable shielding of pipe "B" will occur.

Horizontal Vessels and Spheres

Wind loads on horizontal vessels and spheres may be estimated as follows:

1. Increase vessel diameter by 1.5 ft (0.46 m) to account for ladders, nozzles and pipes 8 in. (0.2 m) and smaller, plus
2. estimate separately the wind forces on piping larger than 8 in. (0.2 m) using a force coefficient (C_f) 0.7, plus
3. estimate the wind forces on platforms and supports using unshielded projected vertical area of platforms and handrails using force coefficient of 2.0.

The Force Coefficient for wind perpendicular to the long axis of the vessel should be determined from [table 8](#). Use a Force coefficient of 0.5 for a rounded head and 1.2 for a flat head for wind in the longitudinal direction. Force coefficient for a sphere (vessel only) shall be determined by using the values from [Table 6](#) based on $h/D = 1$. Wind force on appurtenances should be added separately. (See [Table 9](#))

Vertical Process Vessels

The wind loads on vertical cylindrical vessel can be estimated as follows depending on information on appurtenances:

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Simplified Method

If detailed information (number of platforms, platform size, etc.) is unknown at the time of design of the foundation, wind loads on vertical process vessels with appurtenances such as ladders, platforms, and piping attached to the vessel may be estimated as follows.

1. Increase vessel diameter by 5 ft (1.52 m), or 3 ft (0.91 m) plus the diameter of the largest pipe, whichever is greater, and
2. increase vessel height by the average diameter of the top third of vessel, and
3. determine the force coefficient based on the original vessel height and diameter (average of top 1/3) per [Table 6](#).

Detailed Method

If the details of platforms, piping, ladders, etc. are known, wind loads on vertical vessels may be estimated as follows.

1. Increase vessel diameter by 1.5 ft (0.46 m) to account for ladders, nozzles and piping 8 in. (0.2 m) or smaller, plus the diameter of the largest line coming from the top portion of the vessel.
2. The force coefficient applied to the projected width of the vessel should be taken from [Table 6](#) based on the original height and diameter (average of top 1/3) and on the appropriate roughness of the vessel surface.
3. Add separately the force from pipes larger than 8 in. (0.2 m) that are outside the projected area of the vessel or more than 5 pipe diameters from the vessel surface. Use a force coefficient (C_f) 0.7 for these pipes.
4. For platforms use the vertical projected area, including handrails, toe plate, etc. and a force coefficient (C_f) of 2.0.

Table 6 -Force Coefficients For Tanks, Rooftop Equipment And Similar Structures
(Including Piping and Equipment)

CROSS-SECTION	TYPE OF SURFACE	C _f FOR h/D VALUES OF:		
		1	7	25
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagonal)	All	1.0	1.1	1.5
Hexagonal or octagonal	All	1.0	1.2	1.4
Round: $D\sqrt{q_z} > 2.5$ ($D\sqrt{q_z} > 5.3$)	Moderately smooth	0.5	0.6	0.7
	Rough ($D'/D \approx 0.02$)	0.7	0.8	0.9
	Very rough ($D'/D \approx 0.08$)	0.8	1.0	1.2
Round: $D\sqrt{q_z} \leq 2.5$ ($D\sqrt{q_z} \leq 5.3$)	All	0.7	0.8	1.2

Notes:

- (1) The design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.
- (2) Linear interpolation is permitted for h/D values other than shown.
- (3) Notation:
D: Diameter of circular cross-sections and least horizontal dimension of square, hexagonal or octagonal cross-sections at elevation under consideration, in ft (m);
D': depth of protruding elements such as ribs and spoilers, in ft (m); and
h: height of structure, in ft (m); and
 q_z : velocity pressure evaluated at height z above ground, psf (N/m²).

Equipment Structures

The total wind force on a typical process plant open frame equipment structure can be estimated as that on an enclosed structure that would completely envelope the structure and attachments. The overall shape factor should be based on the structure's aspect ratio (height/width) and the values in [Table 6](#). Wind loading on individual equipment supports, should be based on the unshielded wind load.

Alternatively, the total wind force on the structure may be estimated as the sum of the forces on each component in the structure. Components should include: equipment and supports (without considering shielding), piping, structural framing (as estimated from [Tables 7](#) and [8](#)), ladders, stairs, and other appurtenances attached to the structure. For ordinary structures, the total wind force need not exceed that of the enclosed structure in the previous paragraph.

Trusses and Open Frame Structures

Shape factors for trusses and open frame structures should be as follows:

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Table 7 - Force Coefficients For Trusses And Open Frame Structures

TOWER CROSS SECTION	C_f
Square	$4.0 \epsilon^2 - 5.9 \epsilon + 4.0$
Triangle	$3.4 \epsilon^2 - 4.7 \epsilon + 3.4$

For all wind directions considered, the area A_f consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration, not to the gross area.

The specified force coefficients are for towers with structural angles or similar flat-sided members.

For towers containing rounded members, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members:

$$0.51 \epsilon^2 + 0.57, \text{ but not } > 1.0$$

Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal:

$$1 + 0.75 \epsilon, \text{ but not } > 1.2$$

Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.

Loads due to ice accretion shall be accounted for.

Notation:

ϵ : ratio of solid area to gross area of one tower face for the segment under consideration.

$$\text{Solidity Ratio } \epsilon = \frac{\text{Total solid area on a face}}{\text{Gross area of face (h x d)}}$$

The shape factors for trussed towers and open frame towers are based on structures with not more than two trusses/frames normal to the direction of the wind, and take in account shielding effects and pressures acting on the leeward (back) face. The pressures in these cases should be applied to the projected solid area of the front face only.

For a triangular tower, the design wind force should be assumed to act normal to the tower face. When the wind is oblique to the faces of a square section tower, the wind load should be based on the solid area of one face, and assumed to act perpendicular to a diagonal. If the faces have different solid areas, the larger value should be used.

It should be noted that, for trussed towers and open frame structures, the shape factors given are only for square and triangular towers. Discretion should be used when trying to extrapolate these values to other tower shapes.

The value of $C_f = 1.6$ for a single truss or lattice is based on a solidity ratio (ϵ) of approximately 0.3. Typical solidity ratios for a process plant framework are in the range 0.2 to 0.3. See **ASCE 7** for other solidity ratios.

For guyed towers, the cantilever portion of the tower should be designed for 125% of the design force. In addition, a reduction of 25% of the design force in any span between guys should be made for the determination of controlling moments and shears.

Multiple Bay Open Frame Structures

Force coefficients for wind loads on multiple-bay (more than 2 frames deep), open-frame structures (excluding equipment) may be determined as follows, for frame spacing ratio S_f/B ranging from 0.1 to 0.5.

Table 8 - Force Coefficients On Gross Area Of Multiple Bay Open Frame Structures

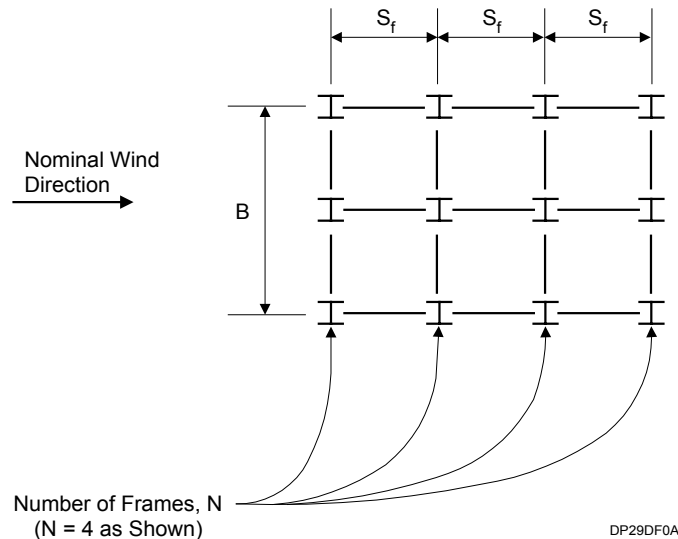
NUMBER OF FRAMES (N)	SOLIDITY RATIO ϵ		
	0.1	0.2	0.3
2	0.35	0.55	0.75
3	0.45	0.75	0.95
4	0.55	0.90	1.15
6	0.75	1.20	1.55



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This table is based on figure 4 in the ASCE Petrochemical energy committee publication, "Wind Loads on Petrochemical Facilities". See this reference for more exact coefficients.

Figure 3 - Plan View Of Framing



where: ε = Solidity, ratio of solid area to gross area of frame
 S_f = Frame spacing, measured from centerline to centerline
 B = Frame width, measured from outside to outside of frame
 N = Number of frames perpendicular to wind

Notes:

- (1) Linear interpolation may be used for solidity values not shown.
- (2) For structures where the solid area of the windward frame is equal to or exceeds the other frames, the solidity is based on the windward frame. Otherwise, the average solidity of all frames should be used.

The solid area of the frame is defined as the solid area of each element in the plane of the frame projected normal to the nominal wind direction. Elements considered part of the solid area of a frame include beams, columns, bracing, cladding, stairs, ladders, handrails, etc. Items such as vessels, tanks, piping and cable trays are not included in calculation of solid area of frame; wind loads on these items are calculated separately.

For structures having frames of equal solidity or if the solid area of the windward frame exceeds that of the other frames, the effective solid area should be based on the windward frame. If the solid area of the windward frame is less than that of the other frames, the effective solid area should be taken as the average of all frames.

In some instances, the equipment supported in a structure can be in the wind shadow of other equipment or parts of the structure. In such cases, it may be appropriate to reduce the equipment load by the shielding factor η .

$$\eta = (1 - \varepsilon)^{(k+0.3)} > 0.4$$

where: ε = The solidity ratio
 k = Volumetric solidity ratio for the level under consideration, defined as the sum of the volumes of all equipment vessels, exchangers, etc. on a level of the structure to the gross volume of the structure at that level
 η = 1.0 if there is no equipment shielding

As the position of equipment may change over time, the formula above should be used with caution and is not normally considered in the design of new facilities.

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Appurtenances

Wind loads for the design of individual appurtenances may be calculated using a force coefficient $C_f = 2.0$ and projected areas as listed in [Table 9](#).

Table 9 - Wind Area Of Appurtenances On Cylindrical, Spherical And Open Frame Structures

APPURTENANCE	PROJECTED AREA sq. ft/ft (sq. m/m)
Handrail	0.8 (0.24)
Ladder without cage	0.5 (0.15)
Ladder with cage	0.7 (0.21)
Stair with handrail: Side elevation End elevation with open risers	handrail area plus channel depth 50% gross area

Plant Buildings

Wind loads on the main wind-force resisting systems of plant buildings of all heights should be determined as follows:

$$p = q G C_p - q_i (GC_{pi})$$

- where:**
- p = design wind pressure for main wind force resisting system, psf, (N/m²)
 - q = q_z for windward wall at height Z above ground.
= q_h for leeward wall, side walls and roof at mean roof height h .
 - q_i = q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings;

 q_z for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$);
 - G = Gust Effect Factor
 - C_p = Pressure coefficients per 0
 - (GC_{pi}) = 0.0 for open buildings
= + 0.55, - 0.55 for partially enclosed buildings
= + 0.18, - 0.18 for enclosed buildings

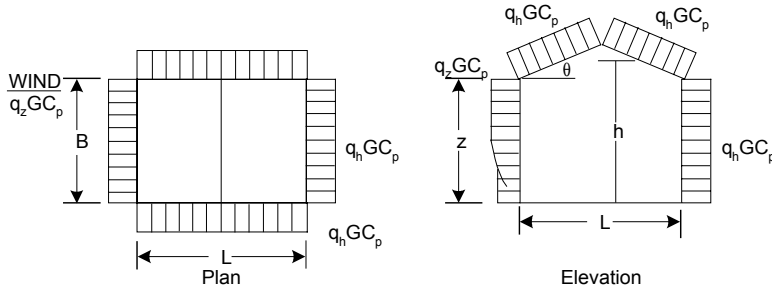
Notes:

- (1) Plus and minus signs signify pressures acting toward and away from the internal surfaces.
- (2) Values of GC_{pi} shall be used with q_z or q_h as specified in [Table 10](#).
- (3) Two cases shall be considered to determine the critical load requirements for the appropriate condition: a positive value of G_{cpi} applied to all internal surfaces and a negative value of G_{cpi} applied to all internal surfaces.
- (4) Glazing that is not impact resistant or protected with an impact-resistant covering, shall be treated as an opening.



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Table 10 - Pressure Coefficients For Plant Buildings



DP29dT8

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	D.B	q_z
Leeward Wall	0-1 2 ≥ 4	-0.5 -0.3 -0.2	q_h
Side Walls	All Values	-0.7	q_h

ROOF PRESSURE COEFFICIENTS, C_p FOR USE WITH Q_H

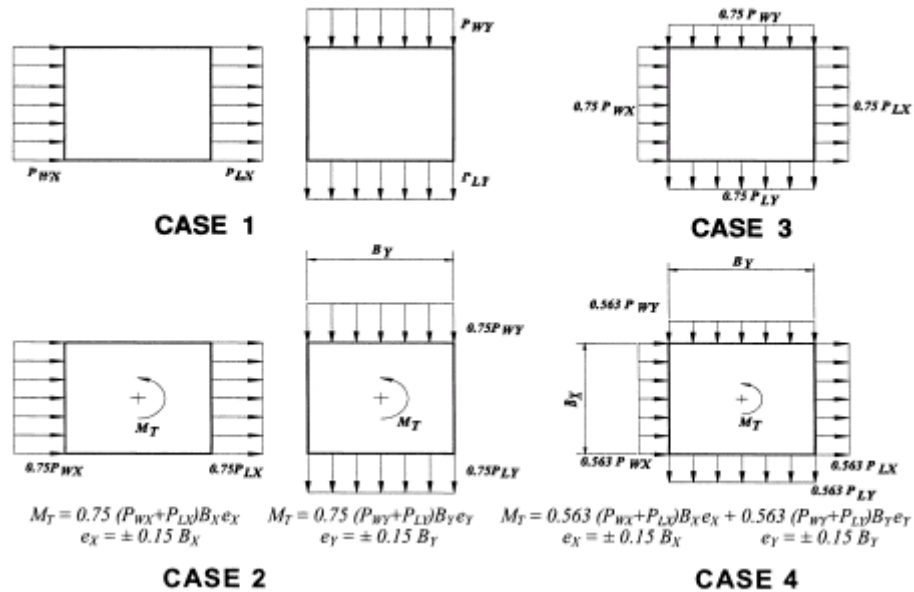
WIND DIRECTION	WINDWARD									LEEWARD			
	ANGLE, θ (DEGREES)									ANGLE, θ (DEGREES)			
	H/L	10	15	20	25	30	35	45	≥60 ⁽⁴⁾	10	15	≥20	
Normal to ridge for θ ≥ 10°	< 0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.5	0.01 θ	-0.3	-0.5	0.06	
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6	
	> 1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6	
Normal to ridge for θ < 10° and Parallel to ridge for all θ	≤ 0.5	Horizontal distance from windward edge 0 to h/2 -0.9, -0.18 h/2 to h -0.9, -0.18 h to 2h -0.5, -0.18 > 2h -0.3, -0.18					* Value is provided for interpolation purposes. ** Value can be reduced linearly with area over which it is applicable as follows: Area (sq ft)						

Notes:

- (1) Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
- (2) Linear interpolation is permitted for values of L/B , h/L and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
- (3) Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
- (4) For roof slopes greater than 80° , use $C_p = 0.8$.
- (5) For monoslope roofs, entire roof surface is either a windward or leeward surface.
- (6) For flexible buildings use appropriate G_f as determined by rational analysis.
- (7) Notation:

B : Horizontal dimension of building, in ft (m), measured normal to wind direction.
 L : Horizontal dimension of building, in ft (m), measured parallel to wind direction.
 h : Mean roof height in ft (m), except that eave height shall be used for $\theta < 10$ degrees.
 z : Height above ground, in ft (m).
 G : Gust effect factor.
 q_z, q_h : Velocity pressure, psf (N/m²), evaluated at respective height.
 θ : Angle of plane of roof from horizontal, in degrees.

The plant building should be designed for the load cases below. The eccentricity, e , assuming that the structure is considered rigid for wind loads, shall be measured from the geometric center of the structure face and shall be considered for each principal axis. The sign of the eccentricity shall be plus or minus, whichever causes the most effect.



Case 1 - Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2 - Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3 - Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4 - Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notation:

P_{WX} , P_{WY} : Windward face design pressure acting in the x, y principal axis, respectively.

P_{LX} , P_{LY} : Leeward face design pressure acting in the x, y principal axis, respectively.

e (e_X , e_Y): Eccentricity for the x, y principal axis of the structure, respectively.

M_T : Torsional moment per unit height acting about a vertical axis of the building.

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EARTHQUAKE LOAD CALCULATION PROCEDURES

SUMMARY OF KEY PROVISIONS

Earthquake loads for the design of structures, equipment and plant buildings are covered in **GP 04-01-03**. Below is a summary of Key provisions for earthquake load calculations.

- Minimum design earthquake loads shall be calculated per the **ASCE 7** standard. Currently the latest version is **ASCE 7-02**. Other project approved standards may be used.
 - Earthquake loads from **ASCE 7** are based on ultimate strength design, hence base shears are 1.4-1.5 times higher than those from service level standards such as **UBC-94**.
- The static and dynamic analysis procedures as appropriate, should be per **ASCE 7**, as supplemented herein.
- For tanks designed to **API 650** and **API 620**, earthquake loads should be per the seismic provisions in the Appendices of those standards.
- The Seismic Design Category (SDC), based on site seismicity, soil characteristics and structure importance, and the structure characteristics determine whether a static or dynamic analysis should be used. Two options are available for the seismic analysis of structures depending on their complexity; namely:
 - Equivalent Lateral Force Procedure
 - Dynamic Lateral Force Procedures
- Dynamic analysis, where required, should be based on the following:
 - Time-dependent response or modal response method using a response spectrum.
 - Ground motion used for dynamic analysis is that having a 2% probability of being exceeded in 50 years and reflecting the influence of local geology, seismicity and soil characteristics, where appropriate.
- Response spectrum is defined using 5% damping unless otherwise specified.
 - The base shear for a given direction should not be less than 100% of the base shear calculated using the equivalent lateral force procedure when using Modal Response Spectrum Analysis. The base shear should not be less than 100% of the equivalent lateral force procedure when using response history analysis.
 - Dynamic analysis should include all significant modes that include at least 90% of the participating mass of the structure in each principal direction.
- A three dimensional structural model should be used for the dynamic analysis of structures with irregular plan configurations.

The design forces are intended only as approximations to produce the same deformations, when multiplied by a deflection amplification factor, as would occur in the same structure should an earthquake ground motion at the design level occur. There are no external forces applied to the above ground portion of a structure during an earthquake. The design limit state for earthquakes is unlike that for any other load is that it is based on system performance not member performance. It is assumed that there is considerable energy dissipation through repeated cycles of inelastic straining. The reason is the large demand exerted by the earthquake and the associated high cost of providing enough strength to maintain linear elastic response. This unusual limit state makes it difficult to separate design provisions for loads from those for resistance. This means that most seismic codes have provisions that modify customary provisions for proportioning and detailing structural systems and construction quality assurance requirements.

For the majority of structures, the static equivalent lateral force procedure is sufficient for use in determining the magnitude and distribution of the lateral forces. However, unusual structures that have significant irregularities in shape, mass or stiffness or are affected by interaction with other structures may require dynamic analysis.

If a dynamic analysis is performed, **ASCE 7** requires that the earthquake forces be cross checked with those calculated per the equivalent lateral force procedure. Scaling of the results, which is covered in more detail below, is also recommended.

The type of earthquake analysis required depends on the characteristics of the structure and on the relative seismology of the site.

SEISMIC USE GROUP

ASCE 7 assigns structures into Seismic Use Group (SUG) categories based on the occupancy category. The importance factor is based on the SUG. Groups and importance factors are as follows;

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Table 11 - Seismic Use Group (SUG) And Importance Factor (I) For ExxonMobil Facilities

CATEGORY	TYPICAL FACILITY	SUG	SEISMIC I_E
IV - Essential	Fire, rescue and emergency response shelters.	III	1.5
III - Hazardous	Determined in conjunction with Plant SOC based on local codes and conditions	II	1.25
II - Standard	All other facilities	I	1.00

The default Seismic Occupancy Category is III, Hazardous, Seismic Use Group is II, and Importance Factor = 1.25. Seismic Use Group of I, and thus Importance Factor = 1.0 may be used with ExxonMobil approval for a specific building, etc. that is determined to be Occupancy Category II. Such determination must be documented in the design basis for that specific building.

SITE SEISMICITY AND CHARACTERISTICS

The maximum considered earthquake ground motion defines the seismicity. It is represented by spectral response accelerations, 5% damped, at short periods, 0.1 seconds, and at one second periods. The maximum considered earthquake has a 2% probability of exceedance in 50 years (2500 year return period) for most areas of the US. It is the largest (deterministic) earthquake that can be generated by known sources in coastal California. It is usually provided in the Job Specifications (GII). Values for some ExxonMobil sites are listed in [Section XXIX-A](#).

These values are then modified for soil conditions by a acceleration-related factor, F_a , and velocity related soil factor F_v .

The *Site Classification* for the site is determined from [Table 12](#) :

Table 12 - ASCE 7 Site Classification

SITE CLASSIFICATION	SHEAR WAVE VELOCITY (V_s)	STANDARD PENETRATION RESISTANCE (N OR N_{ch})	UNDRAINED SHEAR STRENGTH (S_u)
A: Hard Rock	> 5,000 fps (> 1500 m/s)	Not applicable	Not applicable
B: Rock	2,500 to 5,000 fps (760 to 1500 m/s)	Not applicable	Not applicable
C: Very Dense Soil and Soft Rock	1,200 to 2,500 fps (370 to 760 m/s)	>50	> 2000 psf (> 100 kPa)
D: Stiff Soil	600 to 1200 fps (180 to 370 m/s)	15 to 50	1,000 to 2,000 psf (50 to 100 kPa)
E: Soil	< 600 fps (< 180 m/s)	< 15	< 1,000 psf (< 50 kPa)
F: Soils Requiring Site Specific Evaluation	1. Soils vulnerable to potential failure or collapse 2. Peats and or highly organic clays 3. Very high plasticity clays 4. Very thick soft/medium clays		

The soil classification procedure is as follows:

- Step 1: If the site corresponds to any of the four categories of Soil Profile F, conduct a site-specific evaluation.
- Step 2: Classify site as Soil Profile Type E if there exists a soft layer > 10 ft (3 m) thick. ($PI > 20$, $w > 20\%$ and/or $S_u < 500$ psf)
- Step 3: Categorize the site by one of the following three methods. Soil Profile Types A and B should not be assigned to a site if there is more than 10 ft (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.
- Compare the shear wave velocity (V_s) for the top 100 ft (30.5 m) of soil with the values in the table above.
 - Compare the standard penetration (N or N_{ch}) resistance for the top 100 ft (30.5 m) of soil with the values in the table above.

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- c) Compare the undrained shear strength (S_u) of cohesive soil layers ($PI > 20$) for the top 100 ft (30.5 m) with the values in the table above. For cohesionless soil layers ($PI < 20$) determine N_{ch} for the top 100 ft (30.5 m) and compare to the table above. When the N_{ch} and S_u criteria differ, assign the category corresponding to the softer soil (e.g., Soil Profile Type E is softer than D).

Use Site class D when the soil properties are not known in sufficient detail to determine the site class.

Soil profiles containing distinctly different soil layers should be subdivided into layers and V_s , N and S_u should be determined based on a weighted average per the procedure recommended in **ASCE 7**.

The seismic coefficients, F_a and F_v , are listed in [Table 13](#) and [Table 14](#) for the various Site class and S_s or S_1 values.

Table 13 - Acceleration Based Coefficient F_a

SITE CLASS	MAPPED MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_s < 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	(1)	(1)	(1)	(1)	(1)

Note:

Use straight line interpolation for intermediate values of S_s

- (1) Requires site specific geotechnical investigation and dynamic site response analysis

Table 14 - Velocity Based Coefficient F_v

SITE CLASS	MAPPED MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIODS				
	$S_1 < 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	(1)	(1)	(1)	(1)	(1)

Note:

Use straight-line interpolation for intermediate values of S_1

- (1) Requires site specific geotechnical investigation and dynamic site response analysis

The maximum considered earthquake spectral response acceleration for short periods (SMS) and at 1-sec (SM1), adjusted for site class effects, is

$$S_{MS} = F_a S_s$$

$$S_{M1} = F_v S_1$$

where: S_{MS} = the maximum considered earthquake, 5% damped, spectral response acceleration at short periods adjusted for site class effects.

S_{M1} = the maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec adjusted for site class effects.

F_a = acceleration-based site coefficient (at short period).

F_v = velocity-based site coefficient (at 1.0-sec period).

S_s = the mapped maximum considered earthquake spectral response acceleration at a period of 1-sec

S_s = the mapped maximum considered earthquake spectral response acceleration at short periods

These values are then adjusted to 2/3 of their value to produce a design spectral response acceleration parameter. The two-thirds is the reciprocal of 1.5, which is agreed to be the "seismic margin" built into structures designed by the 1994, and older editions of the NEHRP Provisions. In other words, a structure designed by the 1994 or older editions of the NEHRP Provisions is believed to have a low likelihood of collapse under an earthquake that is one and one-half times as large as the design earthquake of those documents. The redefinition of the design earthquake in the 1997 NEHRP Provisions is intended to provide a uniform level of safety across the US against collapse during the Maximum Considered Earthquake.

$$S_{DS} = 2/3 S_{MS}$$

$$S_{D1} = 2/3 S_{M1}$$

where: S_{DS} = the design, 5% damped, spectral response acceleration at short periods.

S_{D1} = the design, 5% damped, spectral response acceleration at a period of 1 sec.

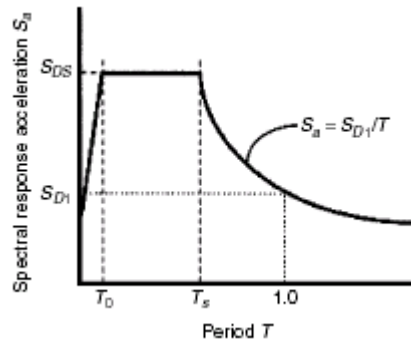
S_{MS} = the maximum considered earthquake, 5% damped, spectral response acceleration at short periods adjusted for site class effects.

S_{M1} = the maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec adjusted for site class effects.

General Procedure Response Spectrum.

Where a design response spectrum is required by these provisions and site-specific procedures are not used, the design response spectrum curve shall be developed as indicated in **ASCE 7** and as follows:

Figure 4 - Design Response Spectrum



1. For periods less than or equal to T_0 , the design spectral response acceleration, S_a , shall be taken as given by:

$$S_a = S_{DS} \cdot \left(0.4 + 0.6 \cdot \frac{T}{T_0} \right)$$

2. For periods greater than or equal to T_0 and less than or equal to T_s , the design spectral response acceleration, S_a , shall be taken as equal to S_{DS} .

3. For periods greater than T_s , the design spectral response acceleration, S_a , shall be taken as given by:

$$S_a = \frac{S_{D1}}{T}$$

where S_a = the design spectral response acceleration

S_{DS} = the design spectral response acceleration at short periods



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S_{D1} = the design spectral response acceleration at 1-sec period, in units of g-sec

T = the fundamental period of the structure (sec)

$T_0 = 0.2S_{D1}/S_{DS}$

$T_S = S_{D1}/S_{DS}$.

SEISMIC DESIGN CATEGORY

In **ASCE 7**, the prescribed analysis procedures are dependent on the **Seismic Design Category**. There are four seismic Performance Categories (A, B, C, or D) and they are established based on the use of the structure (Seismic Use Group) and on the level of expected seismic ground motion. The Seismic Performance Category determines the type of analysis and the special quality assurance required during construction for Seismic Design Categories C, D, E or F. See **ASCE 7** for provisions.

Table 15 - Seismic Design Category Based On Short Term Period Response Accelerations

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{DS} < 0.167 \text{ g}$	A	A	A
$0.167 \text{ g} \leq S_{DS} < 0.33 \text{ g}$	B	B	C
$0.33 \text{ g} \leq S_{DS} < 0.50 \text{ g}$	C	C	D
$0.50 \text{ g} \leq S_{DS}$	D ⁽¹⁾	D ⁽¹⁾	D ⁽¹⁾

Note:

- (1) Seismic Use Group I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1-second period, S_1 , equal to or greater than 0.75g shall be assigned to Seismic Design Category E and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

Table 16 - Seismic Design Category Based On 1 Second Period Response Accelerations

Value of S_{D1}	Seismic Use Group		
	I	II	III
$S_{D1} < 0.067 \text{ g}$	A	A	A
$0.067 \text{ g} \leq S_{D1} < 0.133 \text{ g}$	B	B	C
$0.133 \text{ g} \leq S_{D1} < 0.20 \text{ g}$	C	C	D
$0.20 \text{ g} \leq S_{D1}$	D ⁽¹⁾	D ⁽¹⁾	D ⁽¹⁾

Note:

- (1) Seismic Use Group I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1-second period, S_1 , equal to or greater than 0.75g shall be assigned to Seismic Design Category E and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

The most severe category shall be used. A structure assigned to Category E or F shall not be sited where there is a known potential for an active fault to cause rupture at the ground surface at the structure.

The seismic design categories defined above determine the applicability of the detailing requirements totally irrespective of the severity of wind effects. The seismic design categories are used in the **ASCE 7** code, irrespective of the severity of wind effects, to determine permissible structural systems, limitations on height and irregularity, those components of the structure that must be designed for seismic resistance, and the type of lateral force analysis that must be performed.

Typical seismic design categories for selected US sites are given in example 6 in appendix C.

STRUCTURAL DESIGN CRITERIA, ANALYSIS AND PROCEDURES

Design Basis

Structural Design for acceptable seismic resistance includes:

- Selection of vertical and lateral force resisting systems that are appropriate to the anticipated seismic intensity
- A continuous, regular and redundant load path capable of ensuring that the structure act integrally in its response.
- Adequate lateral and vertical strength and stiffness to limit damage to acceptable levels by correctly proportioning members and connections.

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The design ground motions shall be assumed to occur in any horizontal direction. Allowable stress design is permitted to be used to evaluate sliding, overturning, and soil bearing at the soil-structure interface regardless of the design approach used in the seismic design of the structure.

In **ASCE 7**, the proportioning of structures' elements (sizing of individual members, connections, and supports) is typically based on the distribution of internal forces computed based on linear elastic response spectrum analyses using response spectra that are representative of, but substantially reduced from the anticipated design ground motions. As a result, under the severe levels of ground shaking anticipated for many regions of the nation, the internal forces and deformations produced in most structures will substantially exceed the point at which elements of the structures start to yield and buckle and behave in an inelastic manner. This approach can be taken because historical precedent, and the observation of the behavior of structures that have been subjected to earthquakes in the past demonstrates that if suitable structural systems are selected, and structures are detailed with appropriate levels of ductility, regularity, and continuity, it is possible to perform an elastic design of structures for reduced forces and still achieve acceptable performance. Therefore, these procedures adopt the approach of proportioning structures such that under prescribed design lateral forces that are significantly reduced, by the response modification coefficient R , from those that would actually be produced by a design earthquake they will not deform beyond a point of significant yield. The elastic deformations calculated under these reduced design forces are then amplified, by the deflection amplification factor to estimate the expected deformations likely to be experienced in response to the design ground motion. Considering the intended structural performance and acceptable deformation levels, **ASCE 7** prescribes the story drift limits for the expected (i.e. amplified) deformations. These procedures differ from those in earlier codes and design provisions wherein the drift limits were treated as a serviceability check.

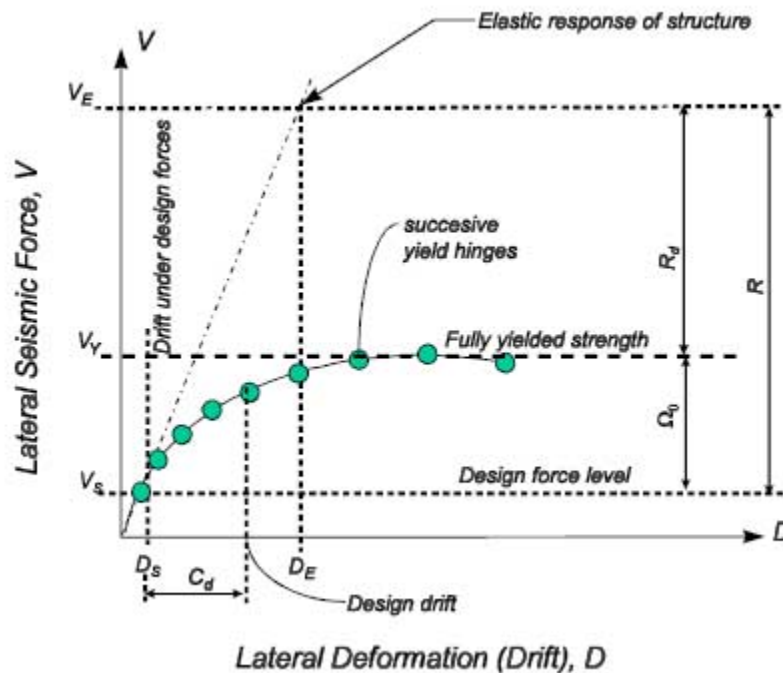
The term "significant yield" is not the point where first yield occurs in any member but, rather, is defined as that level causing complete plastification of at least the most critical region of the structure (e.g., formation of a first plastic hinge in the structure). A structural steel frame comprised of compact members is assumed to reach this point when a "plastic hinge" develops in the most highly stressed member of the structure. A concrete frame reaches this significant yield when at least one of the sections of its most highly stressed component reaches its strength. For other structural materials that do not have their sectional yielding capacities as easily defined, modifiers to working stress values are provided. These requirements contemplate that the design includes a seismic force resisting system with redundant characteristics wherein significant structural overstrength above the level of significant yield can be provided. [Figure 5](#) shows the lateral load-deflection curve for a typical structure. Significant yield is the level where plastification occurs at the most heavily loaded element in the structure, shown as the lowest yield hinge on the load-deflection diagram. With increased loading, causing the formation of additional plastic hinges, the capacity increases (following the solid curve) until a maximum is reached. The overstrength capacity obtained by this continued inelastic action provides the reserve strength necessary for the structure to resist the extreme motions of the actual seismic forces that may be generated by the design ground motion.

The *response modification coefficient*, R , is an empirical factor used to account for both damping and the ductility inherent in the structural system at displacements great enough to exceed initial yield and approach the ultimate load displacement of the structural system. These values are based on **ASCE 7 -02** table 9.14.5.1.1. These values are based on engineering judgment of the performance of the various materials and systems in past earthquakes. The values of R must be chosen and used with careful judgment. For example, lower values must be used for structures possessing a low degree of redundancy wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength.

The system overstrength factor, Ω_0 , approximates the inherent overstrength in typical structures having different seismic force-resisting systems. The overstrength results from the development of sequential plastic hinging in a properly designed, redundant structure. Several other sources will further increase structural overstrength. First, material overstrength (i.e. actual material strengths higher than the nominal material strengths specified in the design) may increase the structural overstrength significantly. For example, a recent survey shows that the mean yield strength of A36 steel is about 30 to 40 percent higher than the minimum specified strength, nominally used in design calculations. Second, member design strengths usually incorporate a strength reduction (or resistance) factor, ϕ , to ensure a low probability of failure under design loading. Third, designers themselves introduce additional overstrength by selecting sections or specifying reinforcing patterns that exceed those required by the computations. Similar situations occur when minimum requirements of design codes, for example, minimum reinforcement ratios, control the design. Finally, the design of many flexible structural systems, such as moment resisting frames, are often controlled by the drift rather than strength limitations of the Code, with sections selected to control lateral deformations rather than provide the specified strength. The results is that structures typically have a much higher lateral resistance than specified as a minimum by the Code and first actual significant yielding of structures may occur at lateral load levels that are 30 to 100 percent higher than the prescribed design seismic forces. If provided with adequate ductile detailing, redundancy and regularity, full yielding of structures may occur at load levels that are two to four times the prescribed design force levels.

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Figure 5 - Inelastic Force-Deformation Curve.



Basic seismic force resisting systems are listed in [table 16](#). Moment frames in dual systems shall be capable of resisting as least 25% of the design seismic forces. The appropriate response modification coefficient, R , system overstrength factor, Ω_0 , and the deflection amplification factor, C_d , shall be used in determining the base shear, element design forces and design story drift.

Table 17 - Seismic Coefficients

Structure Type	Response Modification Coefficient, R	System Overstrength Factor, Ω_0	Deflection Amplification Factor, C_d	Structural System and Height Limits (ft) (1)			
				Seismic Design Category			
				A & B	C	D	E & F
Nonbuilding frame systems:							
Concentric braced frames of steel	5	2	4.5	NL	NL	NL	NL
Special concentric braced frames of steel	6	2	5	NL	NL	NL	NL
Nonbuilding Moment-resisting frame systems:							
Special moment frames of steel	8	5	5.5	NL	NL	NL	NL
Ordinary moment frames of steel	3.5	3	3	NL	NL	50	50
Special moment frames of concrete	8	3	5.5	NL	NL	NL	NL
Intermediate moment frames of concrete	5	3	4.5	NL	NL	50	50
Ordinary moment frames of concrete	3	3	2.5	NL	50	NP	NP
Steel storage racks	4	2	3.5	NL	NL	NL	NL
Elevated tanks, vessels, bins, or hoppers(2):							
On braced legs	3	2	2.5	NL	NL	NL	NL
On unbraced legs	3	2	2.5	NL	NL	NL	NL
Irregular braced legs single pedestal or skirt-supported	2	2	2	NL	NL	NL	NL
Welded steel	2	2	2	NL	NL	NL	NL
Concrete	2	2	2	NL	NL	NL	NL

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Structure Type	Response Modification Coefficient, R	System Over-strength Factor, Ω_0	Deflection Amplification Factor, C_d	Structural System and Height Limits (ft) (1)			
				Seismic Design Category			
				A & B	C	D	E & F
Horizontal, saddle supported welded steel vessels	3	2	2.5	NL	NL	NL	NL
Tanks or vessels supported on structural towers similar to buildings	3	2	2	NL	NL	NL	NL
Flat bottom, ground supported tanks, or vessels:							
Anchored (welded or bolted steel)	3	2	2.5	NL	NL	NL	NL
Unanchored (welded or bolted steel)	2.5	2	2	NL	NL	NL	NL
Reinforced or prestressed concrete:							
Tanks with reinforced nonsliding base	2	2	2	NL	NL	NL	NL
Tanks with anchored flexible base	3	2	2	NL	NL	NL	NL
Tanks with unanchored and unconstrained:							
Flexible base	1.5	1.5	1.5	NL	NL	NL	NL
Other material	1.5	1.5	1.5	NL	NL	NL	NL
Cast-in-place concrete silos, stacks, and chimneys having walls continuous to the foundation	3	1.75	3	NL	NL	NL	NL
All other reinforced masonry structures not similar to buildings	3	2	2.5	NL	NL	50	50
All other nonreinforced masonry structures not similar to buildings	1.25	2	1.5	NL	50	50	50
All other steel and reinforced concrete distributed mass cantilever structures not covered herein including stacks, chimneys, silos, and skirt-supported vertical vessels that are not similar to buildings	3	2	2.5	NL	NL	NL	NL
Trussed towers (freestanding or guyed), guyed stacks and chimneys	3	2	2.5	NL	NL	NL	NL
Cooling towers:							
Concrete or steel	3.5	1.75	3	NL	NL	NL	NL
Wood frame	3.5	3	3	NL	NL	50	50
Telecommunication towers							
Truss: Steel	3	1.5	3	NL	NL	NL	NL
Pole: Steel	1.5	1.5	1.5	NL	NL	NL	NL
Wood	1.5	1.5	1.5	NL	NL	NL	NL
Concrete	1.5	1.5	1.5	NL	NL	NL	NL
Frame: Steel	3	1.5	1.5	NL	NL	NL	NL
Wood	1.5	1.5	1.5	NL	NL	NL	NL
Concrete	2	1.5	1.5	NL	NL	NL	NL
Inverted pendulum-type structures (except elevated tanks, vessels, bins, and hoppers)	2	2	2	NL	NL	NL	NL
Signs and billboards	3.5	1.75	3	NL	NL	NL	NL
All other self-supporting structures, tanks, or vessels not covered above or by ASCE 7 approved standards that are similar to buildings	1.25	2	2.5	NL	50	50	50



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Notes:

(1) Height shall be measured from the base.

(2) Support towers similar to building-type structures, including those with irregularities (see above for definition of irregular structures) shall comply with the detailing requirements below.

See Table 9.5.2.2 in **ASCE 7** for other types of basic seismic force-resisting systems.

NL = No limit. NP = Not permitted.

The combined system Response Modification Coefficient, R , shall be a maximum of 3 for supported non-building structures that have non-rigid dynamic characteristics.

Intermediate moment frames of reinforced concrete shall not be used for non-building structures similar to buildings at sites where the seismic coefficient, S_{DS} , is greater than or equal to 0.50 unless 1) The non-building structure is less than 50 ft (15.2 m) in height and 2) $R = 3.0$ is used for design.

Structures having one or more of the irregularities in [Table 18](#) shall be designated as having plan irregularities and shall comply with the requirements in the **ASCE 7** listed sections when there are assigned to the correct appropriate Seismic Design Categories.

Non-building structures with irregularities at sites where the seismic coefficient, S_{DS} , is greater than or equal to 0.5 and that cannot be modeled as a single mass shall use the Modal Analysis procedures.

Table 18 - Plan Structural Irregularities

	IRREGULARITY TYPE AND DESCRIPTION	ASCE 7 REFERENCE	SEISMIC DESIGN CATEGORY APPLICATION
1a.	Torsional Irregularity Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion, 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. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	9.5.2.6.4.2 9.5.5.5.2	D, E, and F C, D, E, and F
1b.	Extreme Torsional Irregularity Extreme Torsional Irregularity is defined to exist where the maximum story drift, computed including accidental torsion, 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. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	9.5.2.6.4.2 9.5.5.5.2 9.5.2.6.5.1	D C and D E and F
2	Re-entrant Corners Plan configurations of a structure and its lateral force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15% of the plan dimension of the structure in the given direction.	9.5.2.6.4.2	D, E, and F
3	Diaphragm Discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	9.5.2.6.4.2	D, E, and F
4	Out-of-Plane Offsets Discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	9.5.2.6.4.2 9.5.2.6.2.11	D, E, and F B, C, D, E, or F
5	Nonparallel Systems The vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	9.5.2.6.3.1	C, D, E, and F